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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.



## 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

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## AISC Night School 13

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### Design of Industrial Buildings Lesson 3



Presenter:  
Jim Fisher



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## Buildings w/o Overhead Cranes

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- Lesson 3

  - Horizontal Bracing

  - Braced Frames

  - Rigid Frames

  - Selection of the Lateral Load System

  - Examples

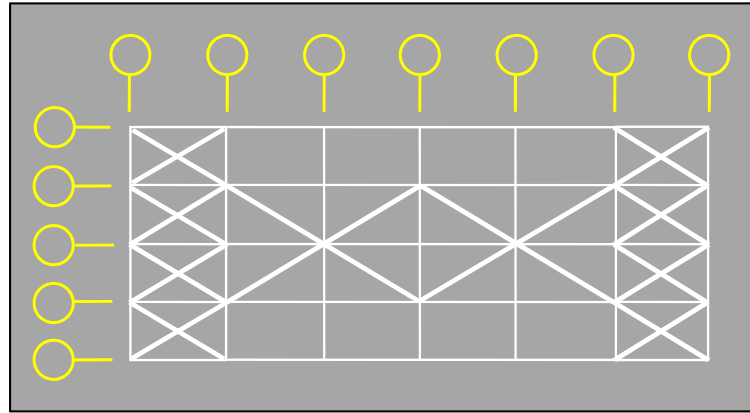


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## Horizontal Bracing

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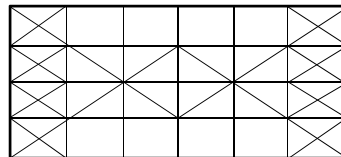
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## Horizontal Bracing

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- 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

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- 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

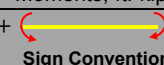
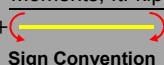
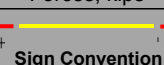
Mark	Designation	Live Load Moments, ft.-kip <sup>+</sup>		Wind or Seismic Moments, ft.-kip		Applied Chord Forces, kips	
		+  +		+  +		+  +	
		Sign Convention		Sign Convention		Sign Convention	
		Left	Right	Left	Right	Top Chord	Bottom Chord
G1	48 inch depth	50	150	± 100	∓ 100	± 50	± 50



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## Rigid Frames

GIRDER SCHEDULE USING LRFD							
Mark	Designation	Live Load Moments, ft.-kip <sup>+</sup>		Wind or Seismic Moments, ft.-kip		Applied Chord Forces, kips	
		+  Sign Convention		+  Sign Convention		+  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

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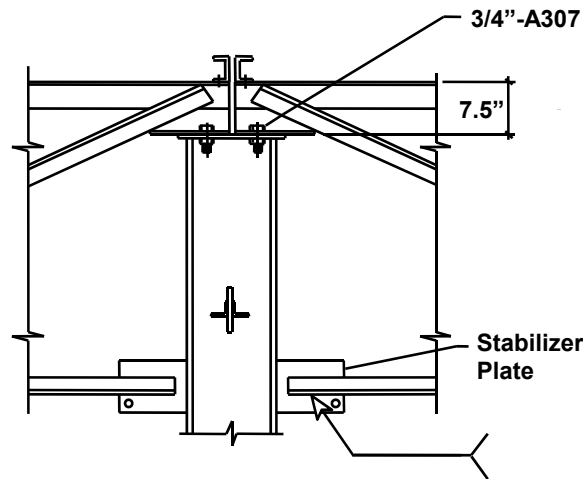
### Joist Girders



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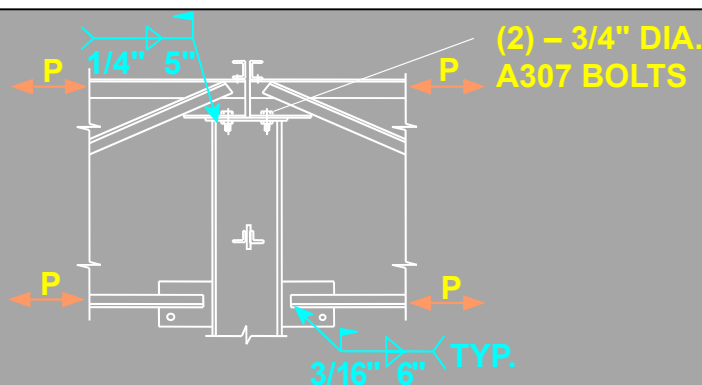
## The Basic Connection



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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

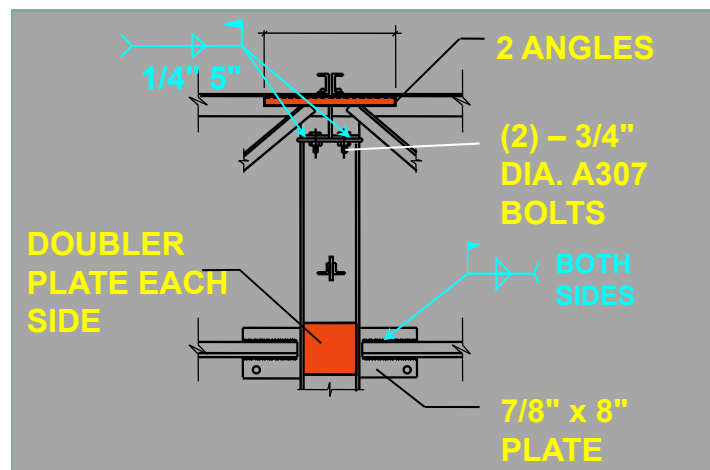
Joist Girder (7.5" seat) Top Chord Leg Size	ASD $P_a$	LRFD $\phi P_n$
2.5"	4 <sup>K</sup>	6 <sup>K</sup>
3.0"	8 <sup>K</sup>	12 <sup>K</sup>
3.5" and larger	10 <sup>K</sup>	15 <sup>K</sup>



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## The Welded Basic Connection with Ties



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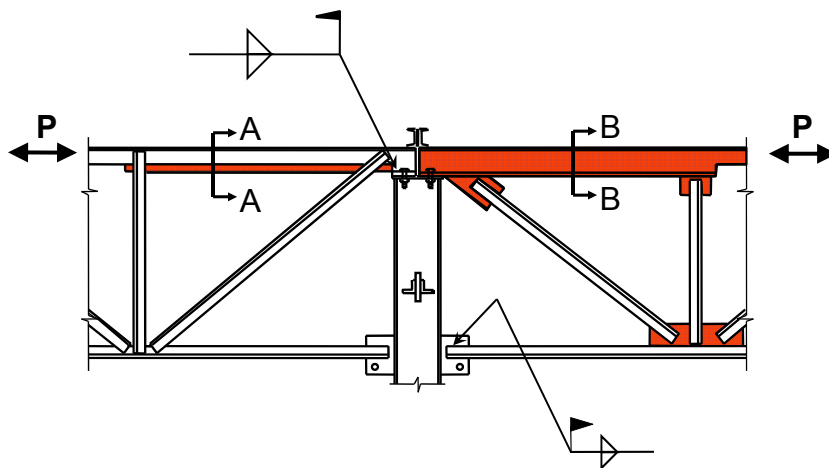
## Top Chord Transfer Angles



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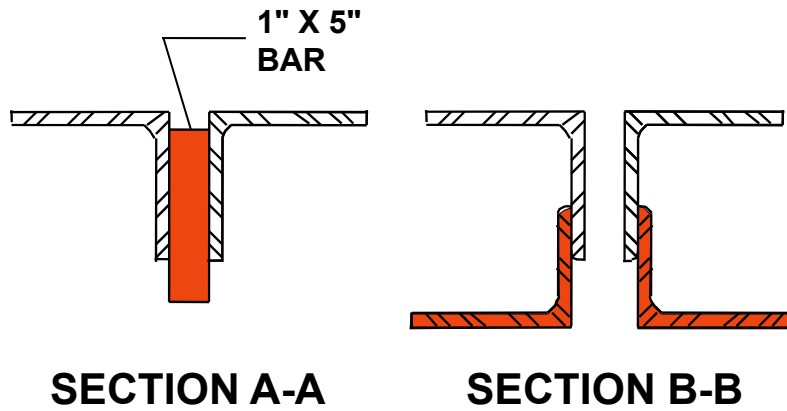
## Reinforced Chords



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## Reinforced Chords



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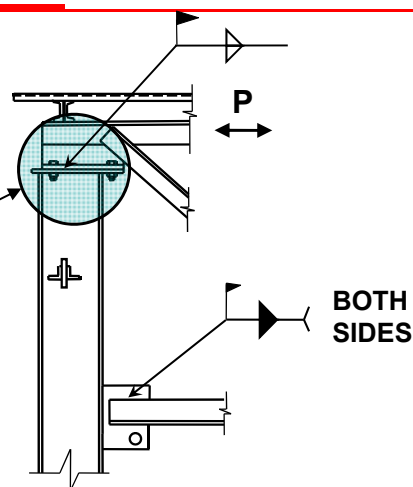
25

## Rigid Seat Connection

$$P_{\max} = 50^{\text{K}} \text{ ASD}$$

$$P_{\max} = 75^{\text{K}} \text{ LRFD}$$

Special design  
required



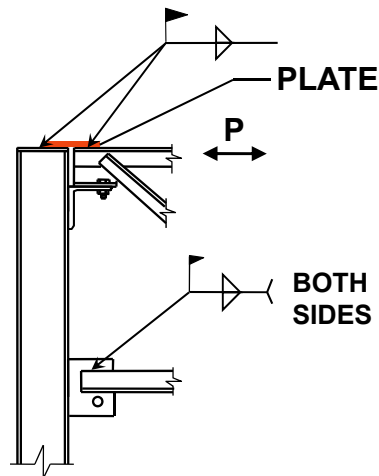
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## Moment Plate Connection

$$P_{\max} = 200\text{K ASD}$$

$$P_{\max} = 300\text{K LRFD}$$



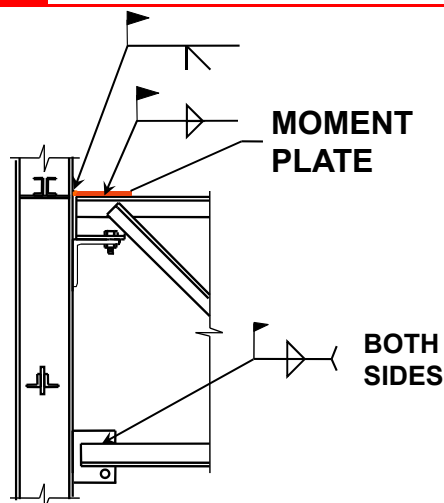
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## Floor Moment Plate Connection

$$P_{\max} = 200\text{K ASD}$$

$$P_{\max} = 300\text{K LRFD}$$



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## Bottom Chord Extensions

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Consider:

Magnitude of Bottom Chord Force

Geometrical Requirements

Chord Gap

Welding

Column Type

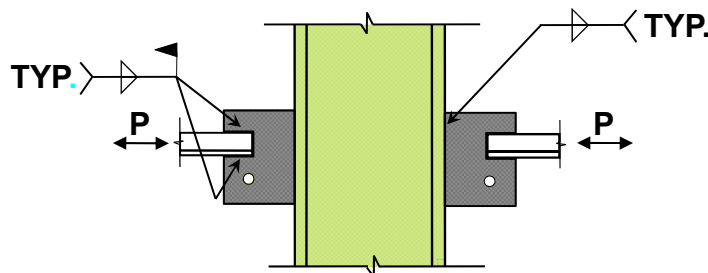


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## Bottom Chord Attachment to Flanges

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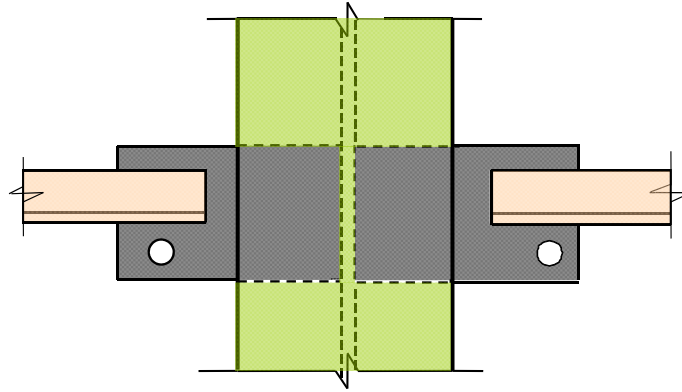


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## Framing to Column Web

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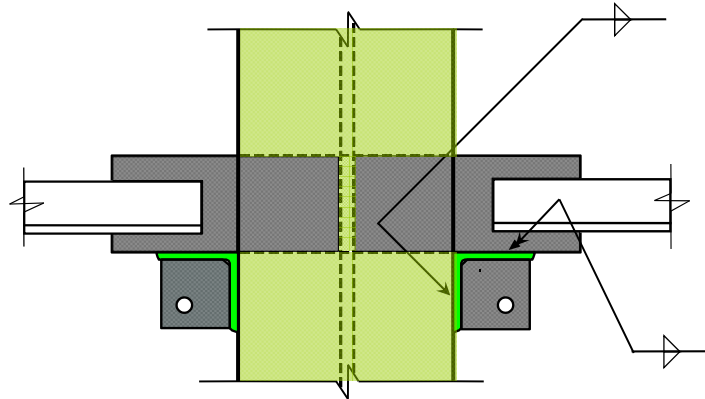


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## Angle Reinforcement to Stabilizer Plates

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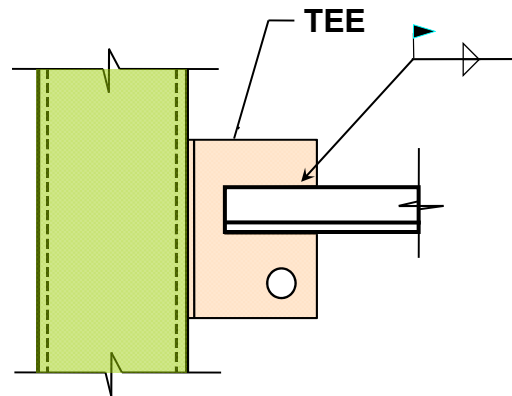


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## HSS Column

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## Rigid Frame Design

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

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

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

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

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- EXAMPLE 1A  
Building Braced at Walls using **ASD**
- EXAMPLE 1B  
Building Braced at Walls using **LRFD**
- EXAMPLE 2  
Rigid Frame Building using **ASD**

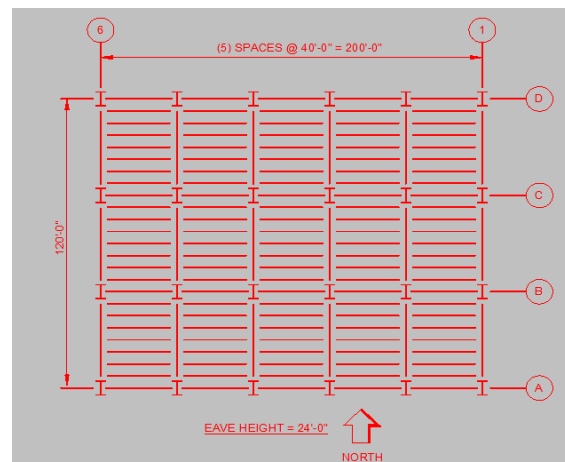


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## Example 1A: Building Braced at Walls (**ASD**)

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## Example 1A: Building Braced at Walls

Live Load	20 psf	Joists @ 5' - 0", o/c
Collateral	5 psf	Joist Girders at 40 ft. spacing
Dead Load	17 psf	22 gage Wide Rib Deck
Joist	3 psf	<b>Load Combinations:</b>
Joist Girders	1 psf	
Wind Load	= 32 psf	D + 0.6W

$$\text{Factored Dead Load} = 5 \text{ psf} + 17 \text{ psf} + 3 \text{ psf} + 1 \text{ psf} = 26 \text{ psf}$$

$$\text{Factored L}_r \text{ Load} = 1.0(20 \text{ psf}) = 20 \text{ psf}$$

$$\text{Factored Wind Load} = 0.6(32 \text{ psf}) \approx 20.0 \text{ psf}$$



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## Example 1A: Building Braced at Walls

$$\text{Joist Load} = (25 \text{ psf} + 20 \text{ psf})(5 \text{ ft.}) = 225 \text{ plf}$$

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

Select 22K7 with Total load = 231 plf

$$\text{Joist Girder Load - Interior} = (0.230 \text{ klf})(40 \text{ ft.}) = 9.2 \text{ kips}$$

$$\text{- Exterior} = (0.230 \text{ klf})(40 \text{ ft.})/2 = 4.6 \text{ kips}$$

Select Joist Girders:

JG1 42G8N4.6K (Exterior)

JG2 42G8N9.2K (Interior)

Live Load for Deflection (unfactored)

$$\text{LL} = (20 \text{ psf})(5 \text{ ft.})(40 \text{ ft.}) = 4 \text{ kips Interior; } 2 \text{ kips Exterior}$$

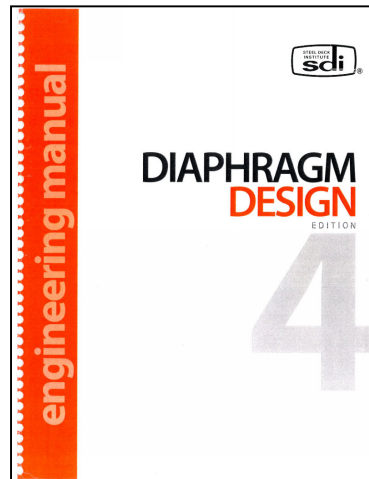


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## SDI Diaphragm Design Manual (4<sup>th</sup> Edition)

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## Diaphragm Design

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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)

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### Select Fastening Pattern from SDI Diaphragm Design Manual 4<sup>th</sup> 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.

$$V_{\text{allow}} = 555/2.35 = 236 \text{ plf where } \Omega \text{ (wind)} = 2.35$$

36/5 weld pattern without side lap screws.

$$V_{\text{allow}} = 625/2.35 = 266 \text{ plf where } \Omega \text{ (wind)} = 2.35$$



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# Diaphragm Design Manual

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

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, $S_{nf}$ , plf <sup>2</sup>										$K_{nf}$ 1/ft
		3	3.5	4	4.5	5	5.5	6	6.5	7		
309	0	1775	1560	1390	1230	1100						0.324
	1	1875	1650	1475	1325	1180	1065	970				0.272
	2	1970	1740	1560	1400	1260	1140	1040	950	860		0.234
	3	2060	1830	1640	1480	1340	1220	1120	1030	940	850	0.204
	4	2150	1920	1730	1560	1420	1300	1200	1110	1020	930	0.183
307	0	1775	1560	1390	1230	1100						0.324
	1	1875	1650	1475	1325	1180	1065	970				0.272
	2	1970	1740	1560	1400	1260	1140	1040	950	860		0.234
	3	2060	1830	1640	1480	1340	1220	1120	1030	940	850	0.204
	4	2150	1920	1730	1560	1420	1300	1200	1110	1020	930	0.183
305	0	1775	1560	1390	1230	1100						0.324
	1	1875	1650	1475	1325	1180	1065	970				0.272
	2	1970	1740	1560	1400	1260	1140	1040	950	860		0.234
	3	2060	1830	1640	1480	1340	1220	1120	1030	940	850	0.204
	4	2150	1920	1730	1560	1420	1300	1200	1110	1020	930	0.183
304	0	1775	1560	1390	1230	1100						0.324
	1	1875	1650	1475	1325	1180	1065	970				0.272
	2	1970	1740	1560	1400	1260	1140	1040	950	860		0.234
	3	2060	1830	1640	1480	1340	1220	1120	1030	940	850	0.204
	4	2150	1920	1730	1560	1420	1300	1200	1110	1020	930	0.183
303	0	1775	1560	1390	1230	1100						0.324
	1	1875	1650	1475	1325	1180	1065	970				0.272
	2	1970	1740	1560	1400	1260	1140	1040	950	860		0.234
	3	2060	1830	1640	1480	1340	1220	1120	1030	940	850	0.204
	4	2150	1920	1730	1560	1420	1300	1200	1110	1020	930	0.183
302	0	1775	1560	1390	1230	1100						0.324
	1	1875	1650	1475	1325	1180	1065	970				0.272
	2	1970	1740	1560	1400	1260	1140	1040	950	860		0.234
	3	2060	1830	1640	1480	1340	1220	1120	1030	940	850	0.204
	4	2150	1920	1730	1560	1420	1300	1200	1110	1020	930	0.183
301	0	1775	1560	1390	1230	1100						0.324
	1	1875	1650	1475	1325	1180	1065	970				0.272
	2	1970	1740	1560	1400	1260	1140	1040	950	860		0.234
	3	2060	1830	1640	1480	1340	1220	1120	1030	940	850	0.204
	4	2150	1920	1730	1560	1420	1300	1200	1110	1020	930	0.183

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

$K_{nf}$	$\phi_{nf}$
Seismic	0.55
Wind	0.70
Other	0.85



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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  
 $F_w = 60$  ksi

Loading	$\phi_{df}$	$\Omega_{df}$
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 <sup>2</sup>										$K_{nf}$ 1/ft
		3	3.5	4	4.5	5	5.5	6	6.5	7		
	0	1775	1560	1390	1230	1100						0.324
	1	1875	1650	1475	1325	1180	1065	970				0.272
36/4	0	770	675	600	530	475	425	380	340	300	260	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	465	0.318
	4	1095	995	905	825	760	705	655	610	560	520	0.267
	5	1155	1055	965	890	820	760	705	660	620	580	0.231
	6	1210	1110	1025	945	875	815	760	710	665	625	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".

$$v_{\text{allow}} = 590/2.35 = 251 \text{ plf where } \Omega(\text{wind}) = 2.35$$

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

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

$$v_{\text{allow}} = 615/2.35 = 261 \text{ plf where } \Omega(\text{wind}) = 2.35$$

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



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## Diaphragm Design (cont'd)

---

### Select Fastening Pattern from SDI Diaphragm Design Manual 4<sup>th</sup> 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.

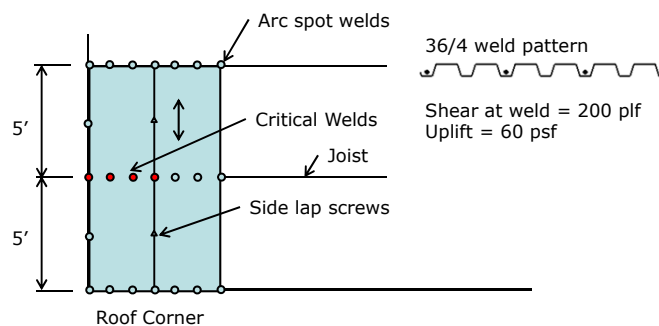
Determine the combined shear and uplift requirement  
Using the AISI Specification and Tables from the SJI  
Diaphragm Manual. ASD design.



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## Combined Weld Shear and Uplift



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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  $2 + 0.7 = 2.7$ ).

$$\begin{aligned} \text{Tension per fastener, } T &= (60 \text{ psf})(5 \text{ ft})(3\text{ft})/(2.7) \\ &= 333 \text{ lbs/weld} \end{aligned}$$



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## Combined Weld Shear and Uplift

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

Support Fastener	P <sub>nt</sub> , 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	--	--	--	↑ 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)			
	Visible Diameter (Inches)			
	0.5	0.625	0.75	1
22	867	1098	1328	1789
20	1039	1278	1598	2157
18	1341	1645	2081	2822
16	1645	2112	2580	3514

Assumes  $F_{\text{exx}} = 60$  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

$$\Omega_t = 2.5, \Omega_r = 2.55$$

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

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

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

$$P_{ntf} \neq P_{nt}$$

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

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

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



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## 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:

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

$$P_{\text{chord}} = M / \text{diaphragm depth} = 1200 \text{ ft.-kips} / 120 \text{ ft} = 10 \text{ kips}$$



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## Check Diaphragm Chord Forces (cont'd)

---

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

$$M_D = wL^2/8 = (17 \text{ psf} + 3 \text{ psf} + 5 \text{ psf})(5 \text{ ft} / 2)(40 \text{ ft})^2 / 8$$
$$= 12,500 \text{ ft.- lbs}$$

$$P_D = M/\text{depth} = (12,500)(12) / (22 \text{ in.} - 1.0 \text{ in.})$$
$$= 7,140 \text{ lbs}$$

$$P_r = 7.14 \text{ kips} + 10 \text{ kips} = 17.1 \text{ kips}$$



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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

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

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

$$26.4 \text{ kips} > 17.1 \text{ kips} \quad \text{ok}$$



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## Check Diaphragm Chord Forces (cont'd)

---

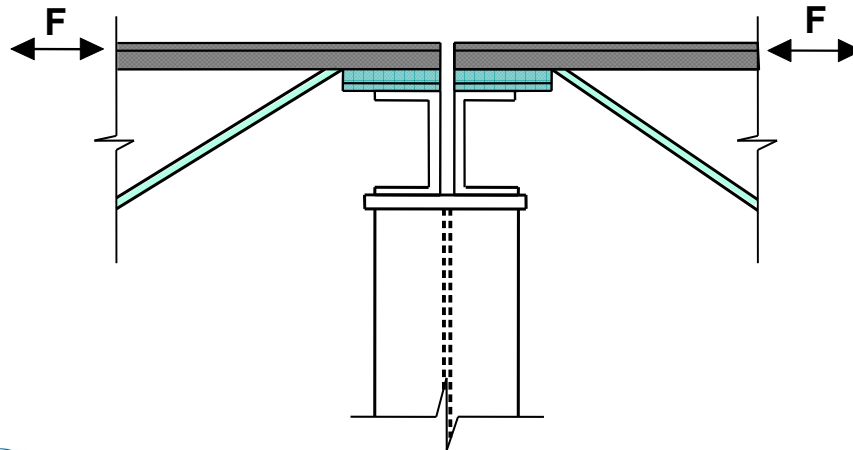
- The 22K7 joist is adequate as the diaphragm chord at A and D, provided eccentric chord bending is eliminated.
- Don't try to pass the chord force through the joist seat.



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## Diaphragm Chord Force



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## Check Diaphragm Chord Forces (cont'd)

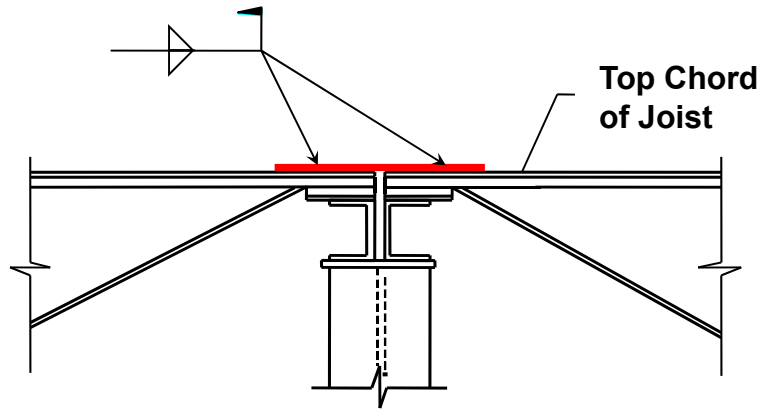
To eliminate bending stresses in the joist top chord use a tie plate between adjacent joists to transfer the chord force.



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## 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.

$$\text{Chord Force } P_{\text{chord}} = 17.1 \text{ kips}$$

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

Plate Properties:

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

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

$$L / r_x = 56$$

$$F_{cr}/\Omega = 18.3 \text{ ksi (AISC Manual Table 4-22)}$$

$$P_a = (18.3)(1.0) = 18.3 \text{ kips}$$

$$18.3 > 17.1 \text{ ok}$$



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## 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

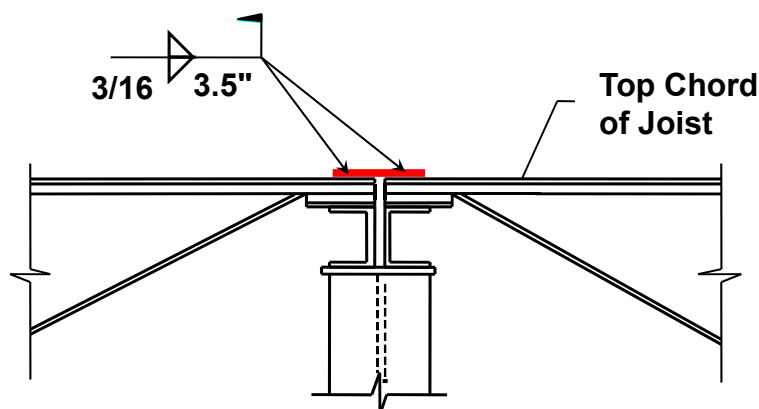
**Therefore, use 7 inches of 3/16 inch fillet on each joist end.**



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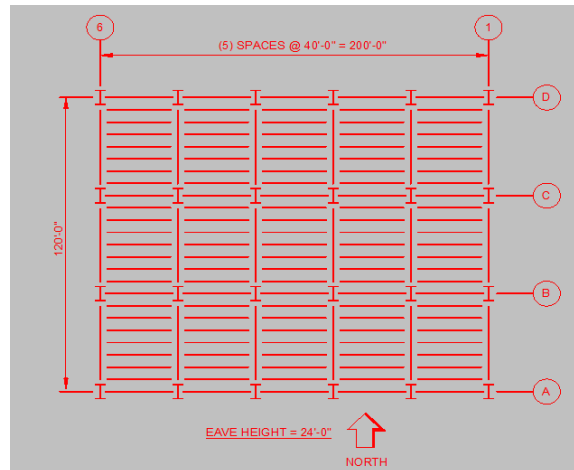
## Joist Tie Plate



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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	Joists @ 5' - 0", o/c
Collateral	5 psf	Joist Girders at 40' spacing
Dead Load	17 psf	22 gage Wide Rib Deck
Joist	3 psf	
Joist Girders	1 psf	
Wind Load	= 32 psf	

### Load Combinations:

$$1.2D + 1.6L_r$$

$$1.2D + 1.0W + 0.5L_r$$

$$\text{Factored Dead Load} = 1.2 (5 \text{ psf} + 17 \text{ psf} + 3 \text{ psf} + 1 \text{ psf})$$

$$= 31.2 \text{ psf}$$

$$\text{Factored Roof Live Load} = 1.6(20) = 32.0 \text{ psf}$$

$$\text{Factored Wind Load} = 1.0 (32 \text{ psf}) = 32.0 \text{ psf}$$



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## Example 1B: Building Braced at Walls

$$\text{Joist Load} = (31.2 \text{ psf} + 32 \text{ psf})(5 \text{ ft.}) = 316 \text{ plf}$$

From **LRFD** Load Table for 40 ft. span,  
Select 22K7 with Total load = 346 plf

$$\begin{aligned} \text{Joist Girder Load - Interior} &= (0.316 \text{ klf})(40 \text{ ft.}) = 12.6 \text{ kips} \\ &\text{- Exterior} = (0.316 \text{ klf})(40 \text{ ft.})/2 = 6.3 \text{ kips} \end{aligned}$$

Select Joist Girders:

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

Live Load for Deflection (unfactored)

$$\text{LL} = (20 \text{ psf})(5 \text{ ft.})(40 \text{ ft.}) = 4 \text{ kips Interior}; 2 \text{ kips Exterior}$$



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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:

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



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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:

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

LINES A and D:

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



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## Diaphragm Design (cont'd)

---

### Select Fastening Pattern from SDI Diaphragm Design Manual 4<sup>rd</sup> 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.

$$V_{\text{available}} = (555 \text{ plf})(0.7) = 389 \text{ plf} (\phi = 0.7)$$

36/5 weld pattern without side lap screws.

$$V_{\text{available}} = (626 \text{ plf})(0.7) = 438 \text{ plf}$$



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## Diaphragm Design (cont'd)

---

### Select Fastening Pattern from SDI Diaphragm Design Manual 4<sup>th</sup> 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.



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## 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:

$$M_{A\&D} = wL^2/8 = (32 \text{ psf})(24 \text{ ft})^2/8/1000$$
$$= 1,920 \text{ ft.-kips}$$

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



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## Check Diaphragm Chord Forces (cont'd)

---

From  $1.2D + 1.0W + 0.5L_r$

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

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

$$P_{\text{chord } D} = M_D / \text{depth} = (15,000 \text{ ft.- lbs})(12)/(22 \text{ in.- } 1.0 \text{ in.}) \\ = 8,570 \text{ lbs} = 8.6 \text{ kips}$$

$$P_{\text{chord } L} = M_L / \text{depth} = (5,000 \text{ ft.-lbs})(12)/(22 \text{ in.- } 1.0 \text{ in.}) \\ 2,860 \text{ lbs} = 2.9 \text{ kips}$$

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



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## Check Diaphragm Chord Forces (cont'd)

---

$$P_{22K7} = [wL^2/8] / \text{depth} = [0.346 \text{ klf } (40)^2 / 8](12)/(21) \\ = 39.5 \text{ kips}$$

Evaluate joist top chord forces:

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

$$21.5 \text{ kips} < 39.5 \text{ kips} \text{ ok}$$



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## 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:

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

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

$$L / r_x = 56$$

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

$$P_a = (27.5)(1.0) = 27.5 \text{ kips}$$

$$27.5 > 21.5 \text{ ok}$$



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## 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

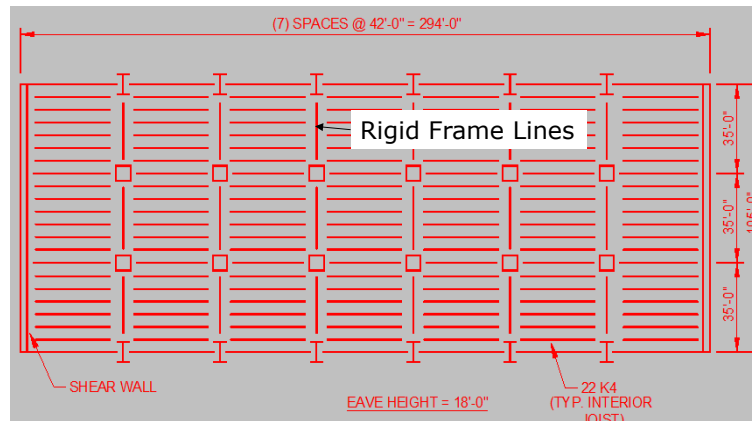
**Therefore, use 6 inches of 3/16 inch fillet  
on each joist end.**



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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:

$$D + 0.6W; D = (10 \text{ psf})(42 \text{ ft}) = 420 \text{ plf}$$

$$\text{Eave strut load} = (0.6)(32.7)(42 \text{ ft})(18 \text{ ft}/2) = 7,420 \text{ lbs}$$

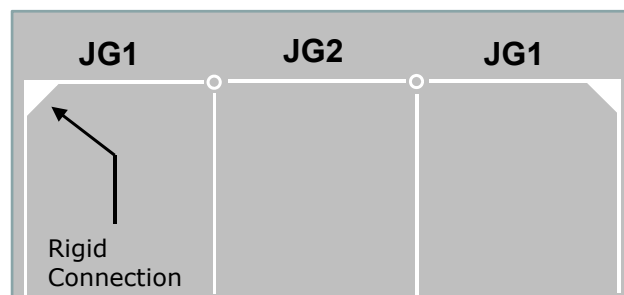
Since this is an ASD design, loads must be multiplied by 1.6 and after a second order analysis divided by 1.6.



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## 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.

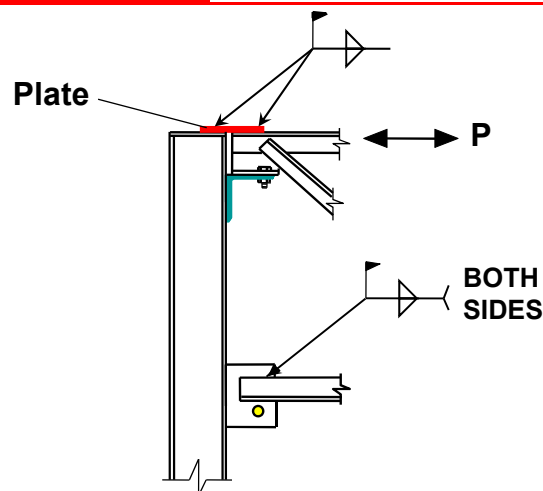
The Basic Connection is used for the Joist Girders at the interior columns.



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## Moment Plate Connection



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## Design Joist Girder Rigid Frame (cont'd)

---

Exterior Columns:	Interior Columns:
Try W14x38 ( $F_y = 50$ ksi)	Try HSS 8x8x1/4 ( $F_y = 46$ ksi)
$A = 11.2$ in. <sup>2</sup>	$A = 7.10$ in. <sup>2</sup>
$I_x = 385$ in. <sup>4</sup>	$I_x = 70.7$ in. <sup>4</sup>
$r_x = 5.87$ in.	$r_x = 3.15$ 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

$$M_r = -1005 \text{ in. kips} = 83.8 \text{ ft-kips}$$

$$P_r = 9.8 \text{ kips}$$

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.

$$= H/150$$



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## Example 2: Rigid Frame Building

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

Try the W 14x38.

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

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

$$\begin{aligned} 1/2pP_r + 9/8(b_x M_{rx}) &= (1/2)(0.11) + (9/8)(0.0109)(83.8) \\ &= 1.08 \text{ ng} \end{aligned}$$

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

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

$$+ (9/8)(0.00661)(83.8) = 0.64 \text{ ok}$$

**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:  $M_{end} = 1005 \text{ in.-kips}$

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

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

$$\begin{aligned} A_{req'd} &= P_{chord}/F_t \quad (\text{where } F_t = F_y/\Omega) \\ &= 29.13/22 = 1.32 \text{ in.}^2 \end{aligned}$$

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



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## Design Joist Girder End Connection (cont'd)

---

Check Plate for compression:

Controlling load case:  $D + 0.6W$

$$P_{\text{chord}} = -29.13 \text{ kips}$$

$$L_x = 3 \text{ in. (unbraced length)}$$

$$r_x = 0.144 \text{ in.}$$

$$L/r_x = 20.8 \leq 25$$

$$F_{cr}/\Omega_c = 21.6 \text{ ksi}$$

$$P_a = (F_{cr}/\Omega)(A) = (21.6 \text{ ksi})(2.0 \text{ in.}^2) \\ = 43.2 \text{ kips} > 29.13 \text{ kips} \text{ ok}$$



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## Design Joist Girder End Connection (cont'd)

---

Determine top plate weld:

$$P_r = 29.13 \text{ kips}$$

$$3/16 \text{ in. weld: } v \text{ allow} = 2.78 \text{ kips/in.}$$

$$\text{Required weld length: } L = 29.13 \text{ kips} / 2.78 \text{ kips/in.} = 10.5 \text{ in.}$$

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

**Use 10 in. of 3/16" weld each side 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

$$A = 4.5 \text{ in.}, P_r = 36.5 \text{ kips}$$

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

$$P_a < P_r \text{ ok}$$

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

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

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

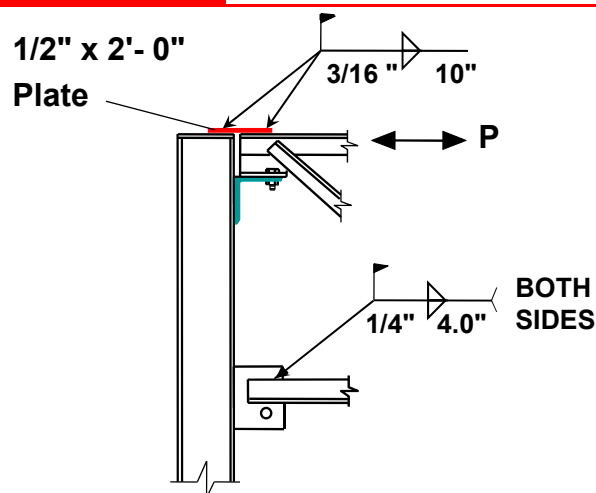
Use 4 inches of 1/4 in. fillet weld each side of stabilizer



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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:  $\Omega = 1.5$

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

$$= (50 \text{ ksi})(0.31 \text{ in.})[(5)(0.915 \text{ in.}) + 6 \text{ in.}]/1.5 = 109.3$$

kips

$$= 109.3 \text{ kips} > 29.13 \text{ kips} \text{ ok}$$



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## Design Joist Girder End Connection (cont'd)

Web Local Crippling:

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

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

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



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## SJI Spreadsheet Solutions

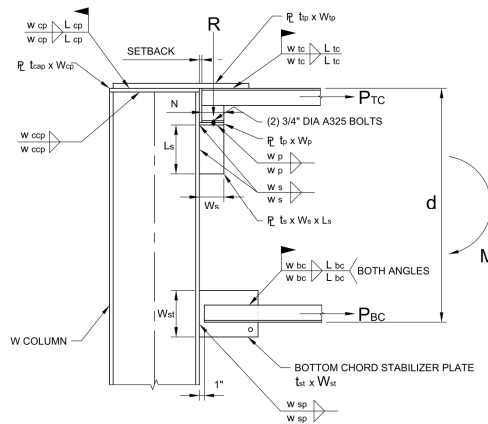
- Joist Girder Moment Connections to the Strong Axis of Wide Flange Columns
- Joist Girder Moment Connections to the Strong Axis of Wide Flange Columns- Intermediate Levels
- Joist Girder Moment Connections to the Weak Axis of Wide Flange Columns
- Joist Girder Moment Connections to HSS Columns- Top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

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All Spreadsheets are available for free  
download from:  
[Steeljoist.org](http://Steeljoist.org).



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## End of Lesson 3

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---

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## 8-Session Registrants

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### QUIZZES

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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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## Night School Resources for 8-session package Registrants

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## Night School Resources for 8-session package Registrants

Navigation links: EDUCATION, PUBLICATIONS, NASCC: THE STEEL CONFERENCE, SAFETY, STEEL SOLUTIONS CENTER, AWARDS AND COMPETITIONS

Event Table:

Event	Date
NS 13 8-Session Package	1/30/2017 7:00:00 PM

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## Night School Resources for 8-session package Registrants

Navigation links: EDUCATION, PUBLICATIONS, NASCC: THE STEEL CONFERENCE, SAFETY, STEEL SOLUTIONS CENTER, AWARDS AND COMPETITIONS, RESEARCH LIBRARY

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**

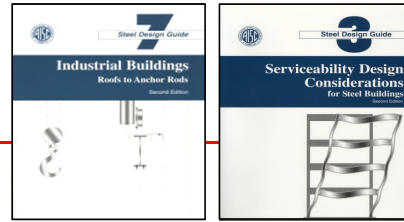
- Webinar connection information:
  - Found in your registration confirmation/receipt.
  - Reminder email sent out Monday mornings.
- Link to handouts also found here.



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## Design Guides

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Design Guides are a FREE download for AISC Members.

For Night School registrants who are non-members, we are offering a discounted price on **Design Guide 3** and **Design Guide 7** only.

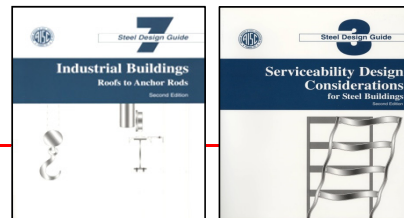
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## Design Guides

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# Thank You

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

