Crane Girder Design

Jules Van de Pas P.E.,S.E. CSD Structural Engineers

Crane Girder Design

Presenter: Jules Van de Pas P.E.,S.E.

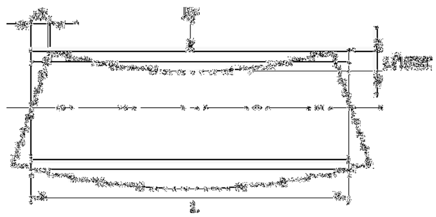
Learning Objectives:

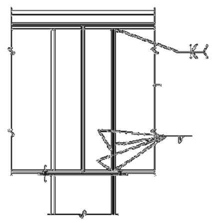
- Discuss the design of crane girder connections for good fatigue performance.
- Demonstrate the design of industrial crane girders.
- Demonstrate design of crane girders to resist seismic loads.

Crane Girder Details

Proper detailing is the key to good fatigue performance

The vast majority of crane girder performance issues occur at the crane girder to column connection.





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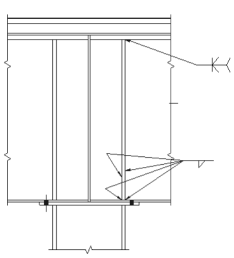
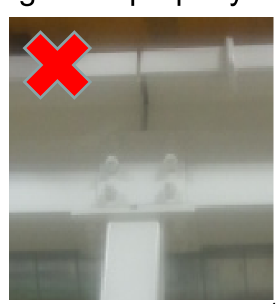
Column or Bracket Support

- Do not use framed or clip angle type connections.
- Extend bearing stiffeners the full height of the girder
- Weld to the girder top flange with full penetration welds or welds sized for the wheel loads.
- Weld to the girder web and bottom flange with properly sized continuous fillet welds.





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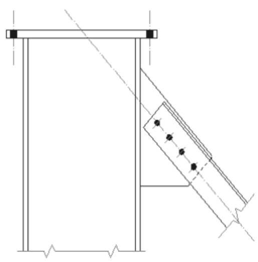
Crane Column Cap Plates

- Allow girder end rotation
  - avoid thick cap plates
  - place the bolts outside of the column flanges.
- Do not use knee braces
- Allow for adjustment in placing the crane girder.



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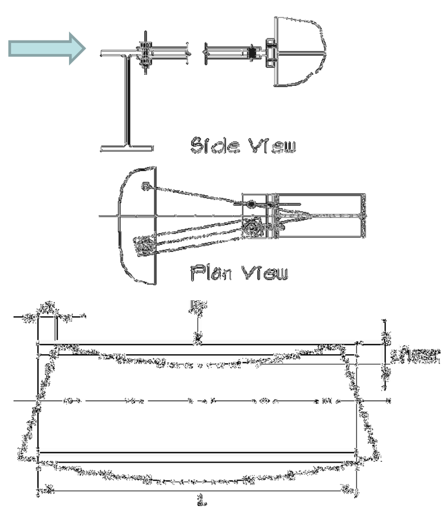
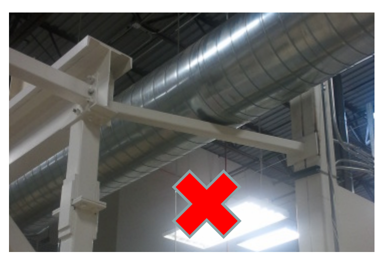
Tie Back Design





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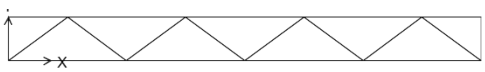
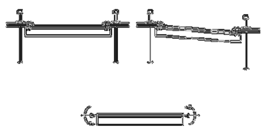
Girder Lacing

Using a truss to stabilize the top and bottom flange for lateral loads



Detail to avoid creating inadvertent continuity in adjacent spans

Consider bending in the angles due to relative vertical movement.



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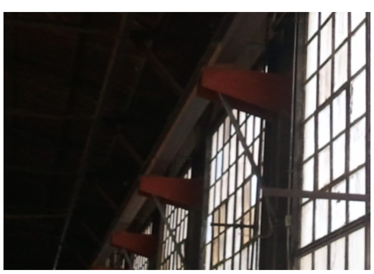
Problematic Girder Support



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Crane Girder Example

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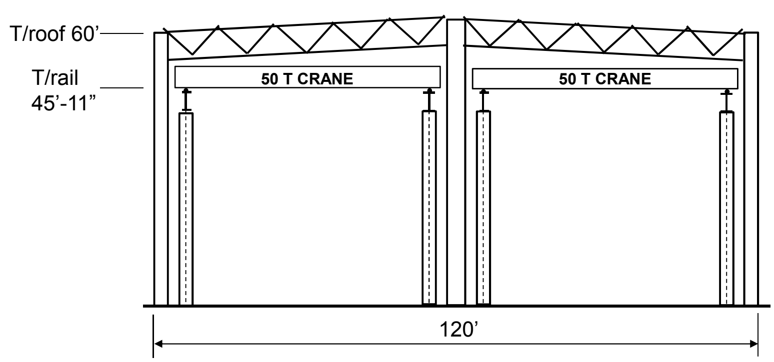
Cross Section



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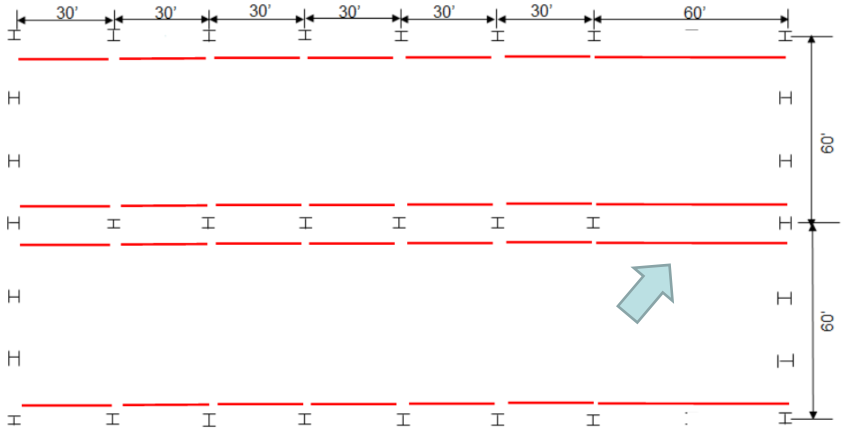
Building Footprint



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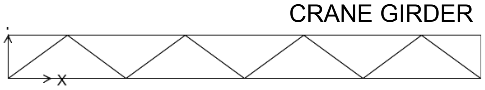
60 ft Runway Girder:

Center Condition:



Use angles to create a truss at the top and bottom of the girders.

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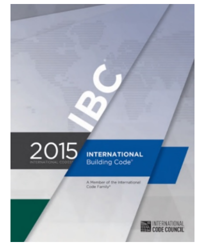
Codes, Standards & Ref’s

- Building Code: IBC 2015
- Minimum Design Loads For Buildings And Other Structures (ASCE 7-10)
- Guide for the Design and Construction of Mill Buildings (AISE Tech Report No. 13, 2003)
- Industrial Buildings Roofs to Anchor Rods 2nd ed. (AISC Steel Design Guide Number 7, 2004)



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Design of a Runway Girder

Crane capacity = 50 tons (CMAA Class D)
Bridge weight = 90.8 kips
Trolley and hoist weight = 31.2 kips
Wheel load = 78 kips (Maximum with lifted load)
Wheel spacing = 11.0 ft.
Rail weight = 175 lbs./yard

Vertical impact = 25% of wheel loads
Lateral load= 20% of lifted load + trolley and hoist
Longitudinal load = 10% of the maximum wheel loads.

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Performance Requirements

Crane Girder Deflection Limits

– Vertical L/800

– Horizontal L/400

Service Life

-500,000 cycles

CMAA Class D:

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Load Combinations

Strength Design Loads: ASCE 7-10

Chapter 2 Section 2.3 Basic Load Combinations

2) 1.2D + 1.6L

Chapter 12 Section 12.4 Seismic Load Effects, Combinations

5a) $(1.2 + 0.2S_{DS})D + \rho Q_E + L + .2S$

5a) $(1.2 + 0.2S_{DS})(D+C_d) + \rho Q_E$

Serviceability

Vertical wheel loads without impact & 100% lateral load

Fatigue Life Design Load

wheel loads without impact (one crane)

50% of maximum lateral load (one crane)

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Seismic Loads

- Spectral Acceleration, S_s: 1.054 G
- Spectral Acceleration, S_1: 0.400 G
- Occupancy Category: II
- Site Class: D
- Importance Factor, I: 1.0

$$S_{MS} = F_a S_s = (1.1)(1.054) = 1.16g \quad S_{M1} = F_v S_1 = (1.6)(0.400) = 0.640g$$

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(1.16) = 0.77g \quad S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.64) = 0.43g$$

- Seismic Design Category = D

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Design of a Runway Girder

Evaluate for Seismic Loads: Design Crane Girder to resist loads based on ASCE 7-10 Chapter 13: Seismic Design Requirements for Nonstructural Components

$$F_p = \frac{.4 a_p S_{DS} W_p}{\frac{R_p}{I_p}} \left(1 + 2\frac{z}{h}\right) \qquad a_p\text{=2.5,} \qquad R_p\text{=3.5}$$

(Table 13.5-1 "Other flexible components, High deformability element and attachments)

$I_p = 1.0$ (section 13.1.3)

z =45.9 ft. h= 60.0 ft. S_{DS}=.770

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Runway Girder -Seismic

$F_p = \frac{.4(2.5)(.77)W_p}{\left(\frac{3.5}{1.0}\right)}\left(1 + 2\frac{45.9}{60.0}\right) = .56W_p$

Total Bridge + Trolley = 90.8 k + 30.2 k =122 k.

$F_p = .56(122\ kips) = 68.3\ k.\quad F_p/wheel = 17.1\ k.\ (ult.)$

$C_d = \frac{90.8}{4} + \frac{30.2}{2} = 35.8\ \ kips/wheel\ \ (Vertical\ Load)$

For Comparison: Max wheel load = 78 k.

F_{ulat}*=1.6*.2(100+31.2)=42.0 kip* F_{ulat}*/wheel=10.5 kips*

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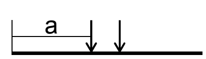
LRFD Design of 60' Crane Girder

Deflection requirements: Locate wheel loads symmetrically placed about the girder centerline. a =24.5 ft. (294 in.)

Vertical: L/800 = (60 ft.)(12)/800 = 0.9 in.

I_{xreqd}: $\Delta_{max} = \frac{P_a a}{24EI}(3L^2 - 4a^2)$



$$\Delta_{max} = \frac{(78)(294)}{24(29000)I}(3(720)^2 - 4(294)^2) = \frac{39849}{I}$$

$$I_{xreqd} = \frac{39849}{\Delta_{max}} = \frac{39849}{.9} = 44277\quad W40x593\ Ix = 50{,}400in^4$$

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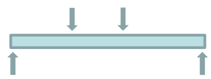
Design of 60 ft Runway Girder

Calculate Moments & Forces 1.2D+1.6L:

DL (Girder + Rail + Clamps) = 593+175/3 + 20 = 671 lbs/ft

$M_{DL} = (1/8)wL^2 = (1/8)(0.67 \text{ kips/ft})(60 \text{ ft})^2 = 302$ k-ft

M_{LL} :See table 3-23 case 44



$$M_{LL} = \frac{1.25 * 78}{2(60)}(60 - 11/2)^2 = 2413 \; kip\text{–}ft.$$

$M_{ux} = 1.2{*}M_{DL} + 1.6{*}M_{ULL} = 1.2{*}302 + 1.6{*}2413 = 4223$ k-ft.

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Design of 60 ft Runway Girder

Calculate Moments & Forces 1.2D+1.6L :

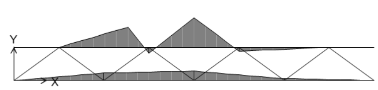
$$M_{uy} = \frac{1.6{*}6.6}{2(60)}(60 - 11/2)^2 = 261 \; k\text{–}ft. \; (total)$$

Calculate chord force in truss:

$$P_u = \left(\frac{261}{5.5}\right) = 47 \; k \; (top\ flange)$$

Bending Between the panel points:

$M_{uy} = 49 \; k\text{–}ft.$ *(per model)*



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Design of 60 ft Runway Girder

Available Moment: L_b = 15 ft., L_r = 63.9 ft., L_p = 13.4 ft.
See table 3-2

$L_p \leq L_b \leq L_r$ *Therefore use Equation F2 – 2:*

$$M_n = C_b[M_p - (M_p - .7F_yS_x)(\frac{L_b - L_p}{L_r - L_p})] \leq M_p$$

$$M_n = 1.0[11400 - (11400 - .7(50)(\frac{2340}{12}))(\frac{15 - 13.4}{63.9 - 13.4})] \leq 11400$$

$M_n = 11255$ *k-ft.* $\varphi M_n = .9 * 11255 =$ *10130 k-ft*

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Design of 60 ft Runway Girder

Check combined forces on girder 1.2D+1.6L

$\frac{kl}{r_y} = \frac{15*12}{4.82} = 37.3$ ⇨ $\emptyset P_n = 40.7 * 3.23 * 16.7 = 2195\ k$

$\varphi\ M_{ny} = \varphi F_y Z_y/2 = (.9)(50)(481)/2)/12 = 902$ *kip-ft.*

$\varphi\ M_{nx} = 10130$ *kip-ft.*

Interaction per AISC Chapter H

$$\frac{P_u}{2\varphi P_n} + \frac{M_{ux}}{\varphi M_{nx}} + \frac{M_{uy}}{\varphi M_{ny}} \leq 1.0 \quad \frac{47}{2*2195} + \frac{4223}{10130} + \frac{49}{902} = .51 \ \text{OK}$$

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Design of 60 ft Runway Girder

Evaluate for Seismic Loads (Continued):

Recall: $M_{DL} = 302\,k - ft.$

$M_{LL} = 2413\ k - ft.$ *(1.25*Wheel load of 156 kips)*

$P_u = 69\ k$ *(For Lat. wheel loads of 10.6 kips)*

$M_{uy} = 49\ \ k - ft.$ *(For Lat. wheel loads of 10.6 kips)*

For load Case 5a: $(1.2+.2S_{DS})(D + C_d) + \rho Q_E$

$M_{ux} = (1.35)[(302) + (35.8/(1.25*156))(2413)] = 1006\ k\text{-}ft.$

$P_u = \frac{(17.1)}{(10.6)} 47 = 76\ kip\ (total)$

$M_{uy} = (17.1/10.6)49\ \ kip-ft. = 79\ k\text{-}ft.$

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Design of 60 ft Runway Girder

For Load Case 5a W40 Crane Girder

Interaction per AISC Chapter H

$$\frac{P_u}{2\varphi P_n} + \frac{M_{ux}}{\varphi M_{nx}} + \frac{M_{uy}}{\varphi M_{ny}} \leq 1.0 \qquad \frac{76}{2*2195} + \frac{1009}{10130} + \frac{79}{902} = .21\ \text{OK}$$

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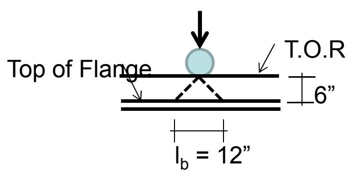
Design of 60 ft Runway Girder

Check Limit states for:

- **Shear**
- **Web Local Yielding J10.2 (wheel loads)**
- **Web Local Crippling J10.3 (wheel loads)**
- **Web Sidesway Buckling J10.4 (wheel loads)**



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Design of 60 ft Runway Girder

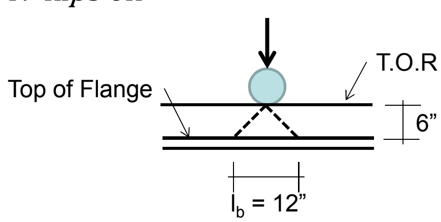
Check Web Local Yielding- AISC J10.2 W40x593

$\varphi = 1.$

$\varphi R_n = \varphi F_{yw} t_w (5k + l_b)$ J10-2

$= (1.0)(50\ ksi)(1.79 in.)[(5)(4.41\ in.) + (12\ in.)] = 3047\ kips$

$156\ kips < 3047\ kips$ ok



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Design of 60 ft Runway Girder

Check Web Local Crippling- AISC J10.3 W40X593

t_f =3.23 in., t_w = 1.79 in., d = 43.0 in., l_b = 12 in. φ =.75

$$\varphi Rn = \varphi .8 t_w^{\,2} [1 + 3(\frac{l_b}{d})(\frac{t_w}{t_f})^{1.5}]\sqrt{\frac{EF_{yw}t_f}{t_w}} \qquad \text{J10-4}$$

$$\varphi Rn = .75 * .8(1.79)^2[1 + 3\left(\frac{12}{43.0}\right)(\frac{1.79}{3.23})^{1.5}]\sqrt{\frac{29000(50)(3.23)}{1.79}}$$

φRn=4184 $\geq$ 156 k. OK

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Design of 60 ft Runway Girder

Check Sidesway Web Buckling AISC J10.4(b) compression flange not restrained against rotation

$W40x593$: $h/t_w = 19.1$, $b_f = 16.7$ in., $L_b = 180$ in.

$(h/t_w)/(L_b/b_f) = 19.1/(180/16.7) = 1.77 > 1.7$

Sidesway web buckling limit state OK

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Laced Crane Girders

- Horizontal truss attached to the top flange of the girder and a back up girder to form a truss to stabilize the top flange of the girder
  - usually economical for spans over 40 feet

Horizontal loads for the 60’ Girder Lacing

- Seismic Loads
- Crane Lateral Loads
- AIST Bottom flange bracing criteria

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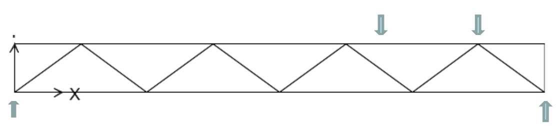
Crane Girder Lacing

Determine brace force:

Seismic: F_p=17.1 kips/wheel

lateral crane load: F_u=1.6*6.6=10.6 kips/wheel):



R_{end}=17.1(2)(52.5+41.5)/60=53.6 kips

Max. Angle Force P_u= 53.6(112/66)=90.9 kips

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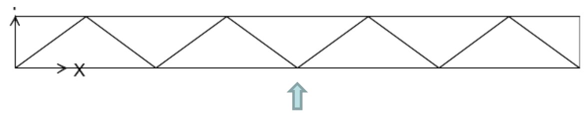
Runway Girder Design

Determine brace force: 2.5% of W40 flange force



Average f_b in bottom flange= 19.98 ksi (Ult. load level)

Brace force = $.025f_bA$ =.025(19.98)(3.23 in.)(16.7) = 26.9 kips

Applied at mid span

Max Angle Force = (26.9/2)(112/66) = 22.8 kips < 90.9 kips

Seismic Load Controls

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Design of 60 ft Runway Girder

Angle Strength: L=9.33 ft.

Try 2L4x4x5/16 AISC Manual Table 4-8:

$\varphi P_n = 100.9\ kips > 90.9 kips\ \ OK$

Table 4-8 (continued)
Available Strength in Axial Compression, kips
Double Angles—Equal Legs

2L4 F_y = 36 ksi

Shape	2L4×4× 3/4		5/8		1/2		7/16		3/8		5/16		1/4[c]	
lb/ft	37.0		31.4		25.6		22.6		19.6		16.4		13.2	
Design	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
X-X Axis 0	235	353	199	299	162	243	142	214	123	185	103	155	75.9	114
2	230	346	195	293	158	238	139	210	121	182	101	152	74.6	112
4	215	324	183	275	149	224	131	197	114	171	95.4	143	70.7	106
6	193	290	164	247	134	202	118	178	103	155	86.4	130	64.7	97.3
8	166	249	142	213	116	174	103	154	89.5	134	75.3	113	[illegible]	[illegible]
10	136	205	117	176	96.3	145	85.5	128	74.7	112	63.1	94.8	[illegible]	[illegible]
12	107	161	93.1	140	76.7	115	68.3	103	59.9	90.1	50.8	76.4	40.1	60.3
14	80.8	121	70.7	106	58.5	87.9	52.3	78.6	46.1	69.3	39.3	59.1	31.9	47.9
16	61.9	93.0	54.1	81.4	44.8	67.3	40.1	60.2	35.3	53.0	30.1	45.2	24.6	37.0
18	48.9	73.5	42.8	64.3	35.4	53.2	31.6	47.6	27.9	41.9	23.8	35.7	19.4	29.2
20			34.6	52.1	28.7	43.1	25.6	38.5	22.6	33.9	19.3	28.9	15.7	23.7

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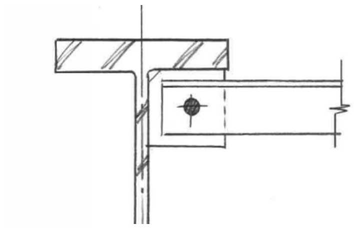
Design of 60 ft Runway Girder

Check Connection for applicable limit states:

- Bolt shear (J3.1)
- Bolt bearing (J3.10)
- Weld and base metal shear rupture (J2.4, J4.2)
- Block shear on angle and gusset (J4.3)
- Tensile rupture on net angle section (D2)
- Fatigue per Appendix 3. *



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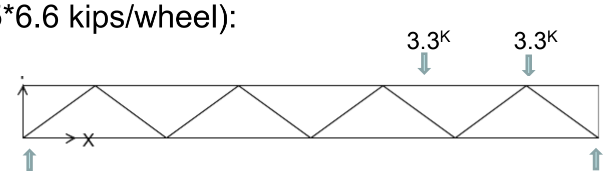
Design of 60 ft Runway Girder

Fatigue Condition: Service Load 50% of max lateral loads (.5*6.6 kips/wheel):



R_{end}=3.3(52.5+41.5)/60=5.2 kips

Angle Force P_s= 5.2(112/66)=+/-8.8 kips

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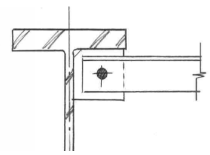
Fatigue Evaluation

Check Connection at the bolt:



- Base Metal at net section of high strength bolted joints designed …bearing resistance, but fabricated and installed to all requirements for slip critical connections.

Section 2: Stress Category B C_f=12 F_{th}=16 Ksi

$F_{SR} = 1000(\frac{C_f}{n_{sr}})^{.333} \geq F_{TH}$ n_{sr}=500,000 cycles

$F_{SR} = 1000(\frac{12}{500{,}000})^{.333} = 28.9\ ksi$

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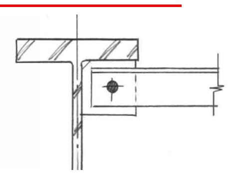
Fatigue Evaluation

Check Connection PL. at the bolt:



5/8" x 6" plate (1)1 1/4" dia. A325 Bolt

A_{net}=.625(6-1.375)=2.89

f=(2)(8.8)/2.89=6.1 ksi < 28.9 ksi OK (<16 ksi)

Also check net section on the angles

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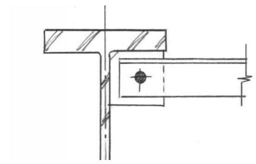
Fatigue Evaluation

Check connection at the gusset weld:

Section 5.7 Base Metal and weld metal at transverse end connections of tension-loaded plate elements....



(2) Cases require evaluation

- Crack initiating from the weld toe
- Crack initiating from the weld root

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Fatigue Evaluation

Check connection at the gusset weld (continued):

Crack initiating from weld toe Stress Category C

C_f=4.4 F_{th}=10 Ksi

$F_{SR} = 1000(\frac{4.4}{n_{SR}})^{.333} \geq F_{TH}$ $F_{SR} = 1000(\frac{4.4}{500{,}000})^{.333}$ =20.7 ksi

Crack initiating from weld root Stress Category C ''

$F_{SR} = 1000Rfil(\frac{4.4}{n_{SR}})^{.333}$ $R_{fil} = .\frac{06+.72(w/tp)}{t_p^{`167}}$ F_{th}=N/A

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Fatigue Evaluation

Check connection at the web weld (continued):

Section 5 Crack initiating from weld root Stress Category C
"

$R_{fil}=\frac{.06+.72(.3125/.625)}{.625^{.167}}=.454$

$F_{SR} = 1000(.454)(\frac{4.4}{500,000})^{.333} = 9.4\ ksi$ *(CONTROLS)*

5/16 fillet weld capacity =9.4ksi(.3125.7071)=2.08 k/in.*

Elastic analysis of the L shaped weld group f=.7 k/in.

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Fatigue Evaluation

Check fatigue at the welded plate to the girder web and tension flange.

- Impact of the stress riser on the strong axis bending strength.

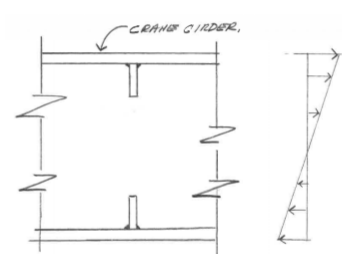

Table A-3.1, Section 5.7, “Base metal of tension loaded plate elements and on girders and rolled beam webs or flanges at toe of transverse fillet welds adjacent to welded transverse stiffeners.”

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Fatigue Evaluation

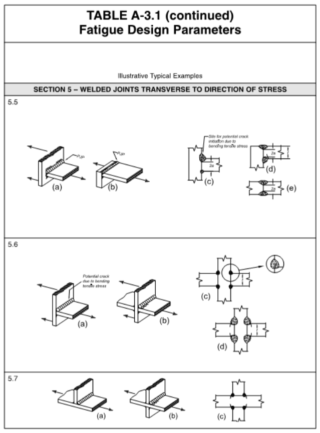

TABLE A-3.1 (cont.)
Fatigue Design Parameters

Description	Stress Category	Constant C_f	Threshold F_{TH} ksi (MPa)	Potential Crack Initiation Point
SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS (cont'd)				
5.6 Base metal and filler metal at transverse end connections of tension-loaded plate elements using a pair of fillet welds on opposite sides of the plate. F_{SR} shall be the smaller of the toe crack or root crack stress range.				Initiating from geometrical discontinuity at toe of weld extending into base metal or, initiating at *weld root* subject to tension extending up and then out through weld
Crack initiating from weld toe:	C	44×10^8	10 (69)	
Crack initiating from weld root:	C″	Eqn. A-3-5 or A-3-5M	None provided	
5.7 Base metal of tension loaded plate elements and on girders and rolled beam webs or flanges at toe of transverse fillet welds adjacent to welded transverse stiffeners.	C	44×10^8	10 (69)	From geometrical discontinuity at *toe of fillet* extending into base metal

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Fatigue Evaluation

Section 5.7 (stress cat. C) $C_f = 44x10^8$, $F_{TH} = 10\ ksi$

$F_{SR} = (\frac{C_f}{n_{SR}})^{0.333} \geq F_{TH}$ $F_{SR} = (44x10^8/500{,}000)^{0.333} = 20.6\ ksi$

$f_b = M_{ll}/S_x = (1931\ kip\text{-}ft.)(12)/\ 2340\ in.^3 = 9.9\ ksi \leq 20.6\ ksi\ ok$

For Comparison without the welded attachment

Section 1.1 (stress cat. A) $C_f = 250x10^8$, $F_{TH} = 24\ ksi$

$F_{SR} = (250x10^8/500{,}000)^{0.333} = 36.7\ ksi$

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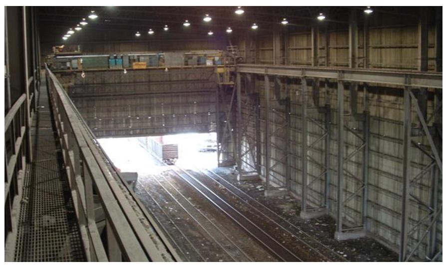
GIRDER DESIGN

CONCLUSION
1. Follow Good Detailing Practices
2. Design for serviceability then strength
3. Evaluate fatigue life per AISC Appendix 3



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