J-1 ELEVATOR GUARD RAIL BRACING

a. *Introduction.*

(1) Purpose. The purpose of this example problem is to illustrate the design of elevator counterweight guide rail bracing using Chapter 10 and Chapter 6 of FEMA 302 (Components). A common occurrence in high-rise buildings is loss of elevator service due to the counterweight leaving the guide rails and impacting the cab. The problem is attributed to flexibility or failure of the counterweight guide rails and their supports.

(2) Scope. The problem follows the steps in Tables 4-5 and 4-6 to analyze the counterweight guide rails and their supports. The building housing the elevator is required to be functional after an earthquake.

b. *Component description.*

The elevator used in this example problem is located in a five-story steel moment frame building. The elevator counterweight is assumed to be 5,000 pounds (22.24KN). Typical elevator details are shown in Figure 10-11.

c. *Component design.*

A.1 *Determine appropriate Seismic Use Group*

Due to the requirement that the building be functional after an earthquake, the elevator is given a performance level of immediate occupancy (IO). The Seismic Use Group and other performance parameters are determined from Table 4-4, as follows;

Performance Level:	IO	(per problem statement)
Seismic Use Group:	IIIE	(Table 4-4)
Ground Motion:	3/4 MCE (B)	(Table 4-4)
Performance Objective:	3B	(Table 4-4)

A.2 *Determine site seismicity.*

The following values are assumed for this example:

$S_S = 1.20g$ (MCE Maps)

A.3 *Determine site characteristics.*

Soil type D is assumed for this problem

Soil type: D (Table 3-1)

A.4 *Determine site coefficients.*

$F_a = 1.02$ (interpolated) (Table 3-2a)

A.5 *Determine adjusted MCE spectral response accelerations.*

$S_{MS} = F_a S_S = 1.02(1.20)g = 1.22g$ (EQ. 3-1)

A.6 *Determine design spectral response accelerations.*

$S_{DS} = 3/4\ S_{MS} = 3/4(1.22) = 0.92g$ (EQ. 3-23)

A.7 *Bracing system.*

The guide rails are supported at each story by the floor framing (see Figure J1-1). Since it is not feasible to brace the rails between floor levels, the rails will be stiffened to span between supports to resist the seismic forces from the counterweight.

A.8 *Select R_p, a_p, and I_p factors.*

$a_p = 1.0$ (Table 10-2)
$R_p = 2.5$ (Table 10-2)
$I_p = 1.5$ (per Paragraph 10-1d)

A.10 *Determine member sizes for gravity load effects.*

No gravity load design is required.

Note: Table 4-6 was created as an aid for building design and is not entirely applicable in the design of nonstructural systems and components. The following steps are based on the intent of TI 809-04, and do not have a one to one correspondence to steps as listed in table 4-6.

F.1 *Determine seismic force effects.*

Seismic forces (F_p) shall be determined in accordance with chapter 10 as follows (see Figure J1-1):

$$F_p = \frac{0.4a_pS_{DS}W_p}{R_p / I_p}\left(1+2\frac{z}{h}\right) \quad \text{(EQ. 10-1)}$$

where; $z/h = 1.0$ (top story of building)
$W_p = 5{,}000\text{-lb}\ (22.24KN)$ (counterweight)

F_p is not required to be greater than:

$$F_p = 1.6S_{DS}I_pW_p \quad \text{(EQ. 10-2)}$$

nor less than:

$$F_p = 0.3S_{DS}I_pW_p \quad \text{(EQ. 10-2)}$$

$$F_p = \frac{0.4(1.0)0.92(5{,}000-\text{lb})}{2.5/1.5}(1+2(1)) = 0.66(5{,}000-\text{lb}) = 3{,}300-\text{lb or } 3.30\text{-kips}\ \ (14.68KN)$$

$$(F_p)_{max} = 1.6(0.92)1.5(5{,}000-\text{lb}) = 11{,}040-\text{lb} > 3{,}300-\text{lb} = F_p\ \ (49.11KN > 14.68KN)$$ **O.K.**

$$(F_p)_{min} = 0.3(0.92)1.5(5{,}000-\text{lb}) = 2{,}070-\text{lb} < 3{,}300-\text{lb} = F_p\ \ (9.21KN < 14.68KN)$$ **O.K.**

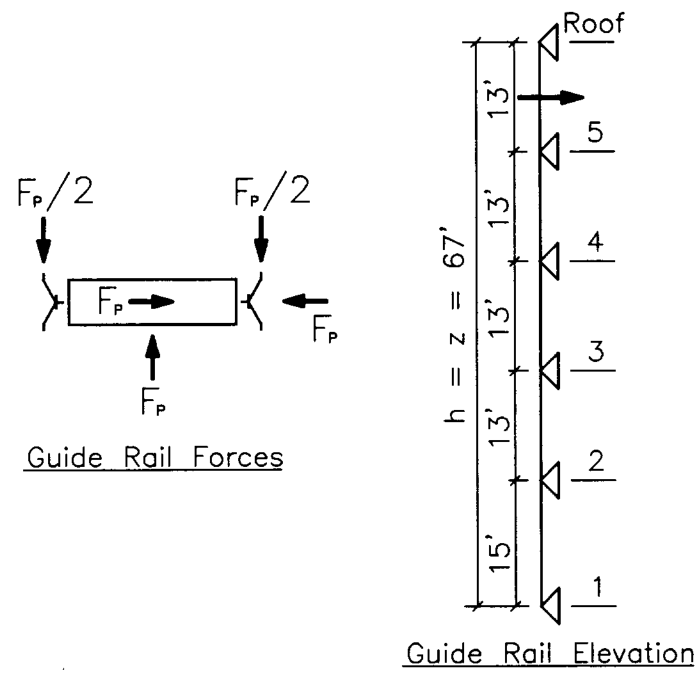
Because of the importance of the guide rails, assume that equation 4-1 of AISC Seismic Provisions (dated April 15, 1997) will apply with $\Omega_0 = 2.0$.

$$\therefore F_p = 3.30^k(2) = 6.60^k\ \ (29.36KN)$$

F.2 *Design members.*

Check guide rail;
Assume elevator uses an 18.5plf (0.27KN/m) guide rail. The properties for an 18.5plf (0.27KN/m) guide rail, obtained from the guide rail manufacturer, are as follows;

$A = 5.44\text{-in}^2\ (3.51X10^3\ mm^2)$	$I_x = 9.67\text{-in}^4\ (4.02X10^6\ mm^4)$	$I_y = 7.45\text{-in}^4\ (4.02X10^6\ mm^4)$
$\bar{y} = 1.26\text{-in}\ (32.0mm)$	$S_x = 3.23\text{-in}^3\ (52.93X10^3\ mm^3)$	$S_y = 2.71\text{-in}^3\ (44.41X10^3\ mm^3)$
$J = 0.78\text{-in}^4\ (324.7X10^3\ mm^4)$	$r_x = 1.33\text{-in}\ (33.8MM)$	$r_y = 1.17\text{-in}\ (29.7mm)$



Note: For metric equivalents; 1-ft = 0.30m

Figure J1-1. Guide rail elevation

Check flexure in rail;

Assume F_y = 36ksi (288.2MPa) (obtained from the guide rail manufacturer), and E = 29 x 10^6psi (200X10^3 MPa).

Note: The guide rail is essentially a tee section. Therefore, per AISC LRFD 2nd edition section F1.2c, its flexural capacity may be calculated as follows;

$$\phi_b M_n > M_u$$

where; $\phi_b = 0.9$

$$M_n = M_{cr} = \frac{\pi\sqrt{EI_yGJ}}{L_b}\left[B + \sqrt{1+B^2}\right] \qquad \text{(EQ. F1-15 AISC LRFD)}$$

$$M_n < 1.0M_y = 1.0S_xF_y$$

$$E = 29{,}000\text{ksi} \ \ (200\text{X}10^3 \text{ MPa})$$

$$I_y = 7.45\text{-in}^4 \ \ (3.10\text{X}10^6 \text{ mm}^4)$$

$$G = \frac{E}{2(1+\mu)} = \frac{29{,}000\text{ksi}}{2(1+0.3)} \approx 11{,}200\text{ksi} \ \ (77.2\text{X}10^3 \text{ MPa}),$$

μ is poisson's ratio ≈ 0.3

$$J = 0.78\text{-in}^4 \ \ (324.7\text{X}10^3 \text{ mm}^4)$$

$$L_b = 13\text{-ft.} = 156\text{-in.} \ \ (3.97\text{m})$$

$$B = \pm 2.3(d / L_b)\sqrt{I_y / J} \qquad \text{(EQ. F1-16 AISC LRFD)}$$

(the minus sign will be used for the stem in compression, and d, the depth of the rail section, = 4.25" (108.0mm) per manufacturers specifications)

$$\therefore B = -2.3(4.25''/156'')\sqrt{7.45-\text{in}^4 / 0.78-\text{in}^4} = -0.194$$

$$\therefore M_n = \frac{\pi\sqrt{29{,}000\text{ksi}(7.45-\text{in}^4)11{,}200\text{ksi}(0.78-\text{in}^3)}}{156-\text{in}}\left[-0.194+\sqrt{1+(-0.194)^2}\right] = 722^{\text{in-k}} \ \ (81.59\text{KN-m})$$

$$M_y = 1.0S_xF_y = 1.0(3.23-\text{in}^3)36\text{ksi} = 116.3^{\text{in-k}} < 722^{\text{in-k}} = M_{cr} \ \ (13.14\text{KN-m} < 81.59\text{KN-m})$$

$$\therefore \phi_b M_n = \phi_b S_x F_y = 0.9(116.3^{in-k}) = 105^{in-k} \quad (11.89\text{KN-m})$$

Determine factored load;
Assume simple beam moment for rail spanning between floors;

$$(M_u)_x = \frac{PL}{4} = \frac{6.60^k(13')(12''/1')}{4} = 257^{in-k} \quad (29.04\text{KN-m}),$$

and $(M_u)_y = 0.5(M_u)_x = 129^{in-k}$ (14.58KN-m)
Therefore,

$$(\phi_b M_n)_x = 105^{in-k} < 257^{in-k} = (M_u)_x \quad (11.87\text{KN-m} < 29.04\text{KN-m})$$

N.G.
Determine required plastic section modulus;

$$(Z_x)_{req'd} = \frac{M_u}{\phi F_y} = \frac{257^{in-k}}{0.9(36ksi)} = 7.93 - in^3 \quad (129.9\text{X}10^3\ mm^3)$$

$$(Z_y)_{req'd} = \frac{M_u}{\phi F_y} = \frac{129^{in-k}}{0.9(36ksi)} = 3.98 - in^3 \quad (65.2\text{X}10^3\ mm^3)$$

Check deflection;
Assume maximum deflection to be limited to 1/2-in. (12.7mm);

$$\Delta_x = \frac{PL^3}{48EI} = \frac{6.60^k(13')^3(1{,}728 - in^3/ft^3)}{48(30{,}000ksi)9.67 - in^4} = 1.80 - in > 0.5 - in \quad (45.7mm > 12.7mm) \qquad \textbf{N.G.}$$

$$\Delta_y = \frac{3.30^k(13')^3(1{,}728 - in^3/ft^3)}{48(30{,}000ksi)7.45 - in^4} = 1.17 - in > 0.5 - in \quad (29.7mm > 12.7mm) \qquad \textbf{N.G.}$$

Determine required moment of inertia's;

$$I_x \geq \frac{PL^3}{48E\Delta_x} = \frac{6.60^k(13')^3(1{,}728 - in^3/ft^3)}{48(30{,}000ksi)0.5''} = 34.8 - in^4 \quad (14.48\text{X}10^6\ mm^4)$$

$$I_y \geq \frac{PL^3}{48E\Delta_x} = \frac{3.30^k(13')^3(1{,}728 - in^3/ft^3)}{48(30{,}000ksi)0.5''} = 17.4 - in^4 \quad (7.24\text{X}10^6\ mm^4)$$

Note: To simplify calculations, the contribution of the rail will be ignored in determining flexural strength.

Try TS 5x5x1/4 welded to back of guide rails (see Figure J1-2);
Properties of TS 5x5x1/4:

$Z_x = Z_y = 8.07\text{-in}^3$ ($132.2\text{X}10^3\ mm^3$) $I_x = I_y = 16.9\text{-in}^4$ ($7.03\text{X}10^6\ mm^4$)
$S_x = S_y = 6.78\text{-in}^3$ ($111.1\text{X}10^3\ mm^3$) $A = 4.59\text{-in}^2$ ($2.96\text{X}10^3\ mm^2$)
$\bar{y} = 2.50$-in (63.5mm) $r = 1.92$-in (48.8mm)
$J = 27.4\text{-in}^4$ ($11.40\text{X}10^6\ mm^4$)

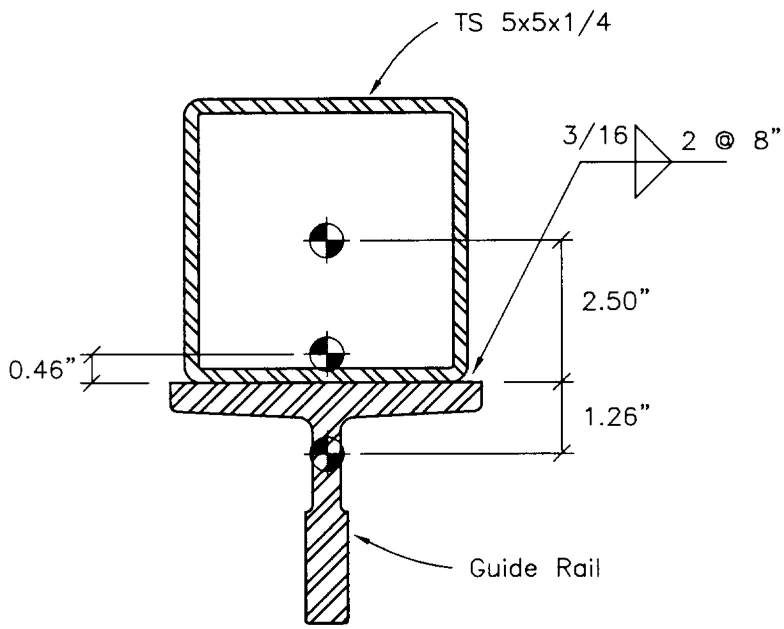
Check deflection;

Determine properties of composite section for deflection calculations;

$$\bar{y} = \frac{\sum A_i y_i}{\sum A_i} = \frac{(5.44 - in^2)(-1.26'') + (4.59 - in^2)(2.5'')}{5.44 - in^2 + 4.59 - in^2} = +0.46 - in \quad (1.7mm)$$

Calculate moment of inertia about the neutral axis

$$I = \sum (I_o + Ad^2)$$



Note: For metric equivalents; 1-in = 25.4mm

Figure J1-2. Composite section of stiffened guide rail

Calculations about the x-axis are performed in tabular form as follows;

Element	A (in^2)	d (in)	Ad^2 (in^3)	I_0 (in^4)
TS 5x5x1/4	4.59	2.04	19.10	16.90
Guide rail	5.44	1.72	16.09	9.67
			35.19	26.57

1-in = 25.4mm
$1\text{-in}^2 = 645.2\text{mm}^2$
$1\text{-in}^3 = 16.4\text{X}103\ \text{mm}^3$
$1\text{-in}^4 = 416.2\text{X}103\ \text{mm}^4$

$I_x = 26.57\text{-in}^4 + 35.19\text{-in}^4 = 61.76\text{-in}^4$ $(25.7\text{X}10^6\ \text{mm}^4)$
$I_y = 7.45\text{-in}^4 + 16.90\ \text{-in}^4 = 24.35\ \text{-in}^4$ $(10.1\text{X}10^6\ \text{mm}^4)$

Therefore;

$$\Delta_2 = \Delta_1\left(\frac{I_1}{I_2}\right)$$

$$\Delta_x = 1.80-\text{in}\left(\frac{9.67-\text{in}^4}{61.76-\text{in}^4}\right) = 0.28" < 0.50" \quad (7.1\text{mm} < 12.7\text{mm})$$ **O.K.**

$$\Delta_y = 1.17-\text{in}\left(\frac{7.45-\text{in}^4}{24.35-\text{in}^4}\right) = 0.36" < 0.50" \quad (9.1\text{mm} < 12.7\text{mm})$$ **O.K.**

Check flexural capacity of rail assuming TS 5X5X1/4 (TS 127.0mmX127.0mmX6.4mm) supports all loads;

L_b = 13-ft or 156-in (3.97m)

$$L_p = \frac{300r_y}{\sqrt{F_{yf}}} = \frac{300(1.92")}{\sqrt{36\text{ksi}}} = 96.0-\text{in} \quad (2.44\text{m})$$ (EQ. F1-4 AISC LRFD)

Since $L_b > L_p$ must calculate L_r, and M_r;

$$L_r = \frac{r_y X_1}{F_L}\sqrt{1+\sqrt{1+X_2F_L^2}} \qquad \text{(EQ. F1-6 AISC LRFD)}$$

$$M_r = F_L S_x \qquad \text{(EQ. F1-7 AISC LRFD)}$$

where; F_L = smaller of (F_{yf} - F_r) or F_{yw}
Since $F_{yf} = F_{yw}$, and F_r = 10ksi (69.0MPa)
F_L = 36ksi – 10ksi = 26ksi (179.3MPa)

$$\therefore M_r = 26\text{ksi}(6.78-\text{in}^3) = 176^{\text{in-k}} \quad \text{(19.89KN-m)}$$

$$X_1 = \frac{\pi}{S_x}\sqrt{\frac{EGJA}{2}} \qquad \text{(EQ. F1-8 AISC LRFD)}$$

where; E = 29,000ksi ($200X10^3$ MPa)
G = 11,200ksi ($77.2X10^3$ MPa)
J = 27.4-in^4 ($11.4X10^6$ mm^4)

$$\therefore X_1 = \frac{\pi}{S_x}\sqrt{\frac{EGJA}{2}} = \frac{\pi}{6.78-\text{in}^3}\sqrt{\frac{29{,}000\text{ksi}(11{,}200\text{ksi})27.4-\text{in}^4(4.59-\text{in}^2)}{2}} = 66{,}220\text{ksi}$$

($456.6X10^3$ MPa)

$$X_2 = 4\frac{C_w}{I_y}\left(\frac{S_x}{GJ}\right)^2 = 0 \quad (C_w = 0 \text{ for a doubly symmetric closed section}) \qquad \text{(EQ. F1-9 AISC LRFD)}$$

$$\therefore L_r = \frac{1.92''(66{,}221\text{ksi})}{26\text{ksi}}\sqrt{2} = 6{,}916'' \quad \text{(175.78m)}$$

$$M_n = C_b\left[M_p - (M_p - M_r)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p \qquad \text{(EQ. F1-2 AISC LRFD)}$$

where; $M_n \le 1.5M_y = 1.5S_xF_y$

$M_p = Z_xF_y = 8.07-\text{in}^3(36\text{ksi}) = 291^{\text{in-k}}$ (32.88KN-m)

$C_b = 1.0$ (conservative for all load cases)

$$M_n = 1.0\left[291^{\text{in-k}} - (291^{\text{in-k}} - 176^{\text{in-k}})\left(\frac{156''-75.5''}{6{,}916''-75.5''}\right)\right] = 290^{\text{in-k}} \quad \text{(32.77KN-m)}$$

Check that $M_n \le 1.5M_y$;

$$\frac{Z_x}{S_x} = \frac{8.07-\text{in}^3}{6.78-\text{in}^3} = 1.19 < 1.5$$

$\therefore (\phi M_n)_x = 0.9(290^{\text{in-k}}) = 261^{\text{in-k}} > 257^{\text{in-k}} = (M_u)_x$ (29.48KN-m > 29.04KN-m) **O.K.**

$\therefore (\phi M_n)_y = 261^{\text{in-k}} > 129^{\text{in-k}} = (M_u)_y$ (29.49KN-m > 14.58KN-m) **O.K.**

Check shear assuming guide rail supports all loads;
Note: The worst case shear occurs when the counterweight is positioned close to the support. The TS 5x5x1/4 section will be stopped short of the support thus placing the entire shear load on to the guide rail (see Figure J1-4). Assume data obtained from the manufacturer of the guide rail shows the guide rail stem and flange to be 0.5-in. (12.7mm) thick at their thinnest points, the overall depth of the rail section to be 4.25-in. (108.0mm), and the width of the flange to be 5.50-in. (139.7mm) By inspection, loading on the stem is the worst case to be investigated because it has the smallest shear area supporting the largest load.

Check the stem parallel to the 6.60^k (29.4KN) load;

$\phi_vV_n \le V_u$ where; $\phi_v = 0.90$

$$\frac{h}{t_w} = \frac{4.25"}{0.50"} = 8.5 < 69.7 = \frac{418}{\sqrt{36ksi}} = \frac{418}{\sqrt{F_y}}$$

$$\therefore V_n = 0.6F_{yw}A_w \qquad \text{(EQ. F2-1 AISC LRFD)}$$

$$\phi_v V_n = 0.9(0.6)36ksi(4.25"(0.5")) = 41.3^k \ (183.7KN)$$

$$\phi_v V_n = 41.3^k > 6.60^k = V_u \quad (183.7KN > 29.4KN) \qquad \textbf{O.K.}$$

Use TS 5x5x1/4 (TS 127mmX127mmX6.4mm) attached to guide rail

Design weld of TS to guide rail

Vertical shear at the TS to guide rail interface is determined from;

$$v = \frac{VQ}{I} \ \text{(kips/in)}$$

where; $V_{max} = 6.60^k$ (29.4KN)
Q = static moment of the TS about the interface
= 2.50"(4.59-in^2) = 11.48-in^3 (188.1X10^3 mm^3)

$$R_u = v = \frac{6.60^k(11.48-in^3)}{61.76-in^2} = 1.23^{k/in} \quad (0.22KN/mm)$$

Since the thicker part being joined is 0.5-in. thick, the minimum thickness of weld per Table J2.4 of AISC LRFD is 3/16-in. Try a 3/16" fillet weld using E70 electrodes 2-in. in length spaced at 8-in. on center on both sides;

$$\phi R_n = 0.75t_e(0.60F_{EXX}) \qquad \text{(per AISC LRFD J2.2)}$$

$$\text{where; } t_e = \frac{3"}{16}\left(\frac{1}{\sqrt{2}}\right) = 0.133-in \quad (3.38mm)$$

$$\phi R_n = 0.75(0.133")(0.60(70ksi)) = 4.19^{k/in} \text{ for a 3/16-in. (4.76mm) weld}$$

For two welds 2-in. long and spaced at 8-in. (203.2mm) on center;

$$\therefore \phi R_n = \frac{4.19^{k/in}(2")}{8"}x2 = 2.10^{k/in} > 1.23^{k/in} = R_u \qquad (0.37KN/mm > 0.22KN/mm) \qquad \textbf{O.K.}$$

Use two 2-in. (50.8mm) by 3/16" (4.76mm) fillet welds spaced at 8-in. (20.32mm) on center to connect TS section to guide rail

Design bolted connection of guide rail to horizontal spreader bracket (Figures J1-3, and J1-4);
Note that the 6.60^k (29.4KN) load can only push against the guide rail as shown and can not pull it. Because the bracket angle is as yet not designed, the connecting members are conservatively assumed to be 3/8-in. (9.53mm) thick for these calculations.
Try 2-5/8" (15.9mm) ϕ bolts in a slip critical connection;
Check Shear;
By inspection, the worst case occurs for loads perpendicular to the long leg of the bracket angle.

$R_u = 6.60^k$ (29.4KN)

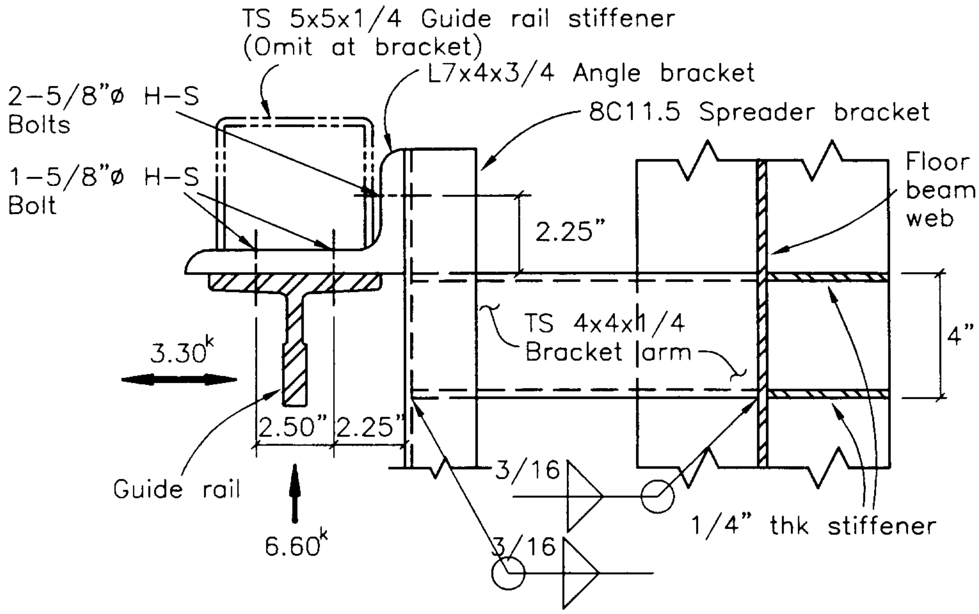
Per Table 8-16 of Volume 2 of AISC LRFD 2nd edition;

$$\phi R_n = 5.22^{k/bolt} \ (23.2KN)$$

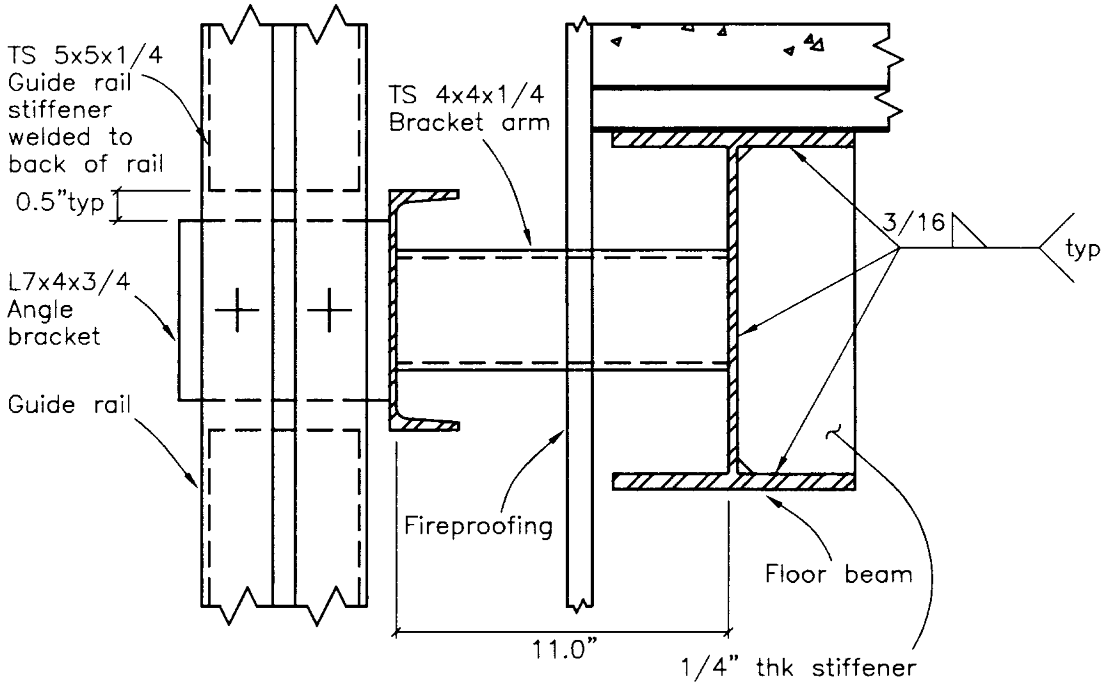
$$\therefore \phi R_n = 2(5.22^k) = 10.4^k > 6.60^k = R_u \ (46.3 > 29.4KN) \qquad \textbf{O.K.}$$

Note: Although slip critical connections theoretically are not subject to bearing, they must have sufficient strength in the event of an overload that may cause slip to occur.

Check bearing;
From Figure J1-3, the maximum spacing of bolts aligned in the direction of load is 2.5-in. (63.5mm) > 3d = 3(5/8-in.) = 1.88-in. (47.8mm), and the minimum edge distance is 1.5-in. (38.1mm) >1.5d = 1.5(5/8-in.) = 0.938-in. (23.8mm). Therefore, spacing and edge distance requirements of AISC LRFD J3.3 are satisfied.
Per Table 8-13 of Volume 2 of AISC LRFD 2nd edition;



Note: For metric equivalents; 1-in = 25.4mm, 1-kip = 4.48KN
Figure J1-3. Plan view of guide rail bracket



Note: For metric equivalents; 1-in = 25.4mm, 1-kip = 4.48KN
Figure J1-4. Section through guide rail bracket

$$\phi R_n = 65.3^{k/in}(0.375") = 24.5^{k/bolt} \quad (109.0\text{KN/bolt})$$

$$\therefore \phi R_n = 2(24.5^k) = 49^k > 6.60^k = R_u \quad (218.0\text{KN} > 29.4\text{KN})$$ **O.K.**

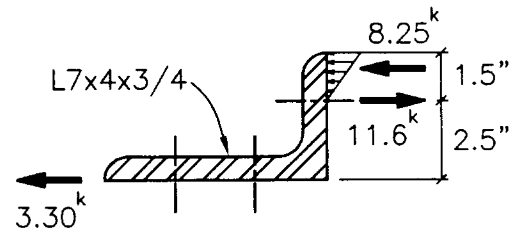
Check prying action;
The worst case tensile load in the bolts connecting the angle bracket to the C channel spreader bracket must be determined. This load can result from either one of the load cases shown in Figure J1-3.
Consider bending about the bolt centerline at angles attachment to the spreader bracket;

Load condition (1); a 3.30^k (14.7KN) load parallel to the long leg on bracket angle (Figure J1-5):

$3.30^k(2.5") = C(2/3)1.5"$

$$C = \frac{(3.30^k)2.5"}{1.5}\left(\frac{3}{2}\right) = 8.25^k \quad (36.7KN)$$

$P_u/\text{bolt} = (8.25^k + 3.30^k)/2 = 5.78^{k/bolt}$ (25.7KN/bolt)



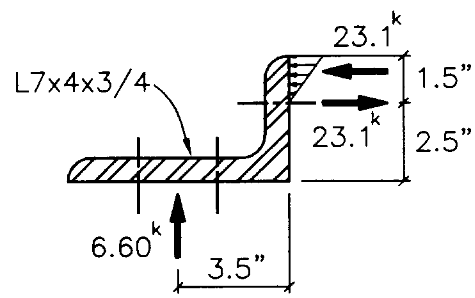
(1-in = 25.4mm, 1-kip = 4.448KN)
Figure J1-5. Detail of forces in angle bracket

Load condition (2); a 6.60^k (29.4KN) load perpendicular to the long leg on bracket angle (Figure J1-6):

$6.60^k(3.5") = C(2/3)1.5"$

$$C = \frac{(6.60^k)3.5"}{1.5}\left(\frac{3}{2}\right) = 23.1^k \quad (102.7KN)$$

$P_u/\text{bolt} = (23.1^k)/2 = 11.6^{k/bolt}$ (51.6KN/bolt)
(governs)



(1-in = 25.4mm, 1-kip = 4.448KN)
Figure J1-6. Detail of forces in angle bracket

Per Table 8-15 of volume 2 of AISC LRFD 2nd edition;

$\phi R_n = 20.7^{k/bolt} > 11.6^{k/bolt} = R_u$ (92.1KN/bolt > 51.6KN/bolt) **O.K.**

Use 2-5/8" (15.9mm) ϕ bolts at each angle bracket leg

Design bracket angle that connects guide rail to the horizontal spreader bracket (Figure J1-3, and J1-4);
Try L 7x4x3/8x0'-6" (L177.8mmX101.6mmX9.5mmX0.15m)
Design for flexure;
Critical section for bending occurs about the bolt centerline at angles attachment to the spreader bracket.
By inspection, the shape is compact and the unbraced length is negligible so that equation F1-1 of AISC LRFD may be used.

$\phi M_n = \phi Z_x F_y > M_u$ with; $\quad M_n \le 1.5M_y = 1.5S_x F_y$

As shown in the design of the bolts, the worst case moment is caused by the 6.60^k (29.4KN) load perpendicular to the Long leg of the angle bracket. Also, the reduction in cross sectional area due to the boltholes is considered in calculating the plastic section modulus.

$M_u = 6.60^k(3.5") = 23.1^{in\text{-}k}$ (2.61KN-m)

$$Z_x = \frac{bh^2}{4} = \frac{(6"-2(0.75"))(0.375")^2}{4} = 0.158-\text{in}^3 \quad (2.59X10^3\ \text{mm}^3)$$

$$S_x = \frac{bh^2}{6} = \frac{(6"-2(0.75"))(0.375")^2}{6} = 0.106-\text{in}^3 \quad (1.74X10^3\ \text{mm}^3)$$

$$\frac{Z_x}{S_x} = \frac{0.158}{0.106} = 1.49 < 1.5$$

$$\therefore \phi M_n = 0.9 Z_x F_y = 0.9(0.158 - in^3)36ksi = 5.12^{in-k} < 23.1^{in-k} = M_u \quad (2.02KN\text{-}m < 9.14KN\text{-}m) \ \textbf{N.G.}$$

Try L 7x4x3/4x0'-8"

$$Z_x = \frac{bh^2}{4} = \frac{(8"-2(0.75"))(0.75")^2}{4} = 0.914 - in^3 \quad (14.98X10^3\ mm^3)$$

$$S_x = \frac{bh^2}{6} = \frac{(8"-2(0.75"))(0.75")^2}{6} = 0.609 - in^3 \quad (9.98X10^3\ mm^3)$$

$$\frac{Z_x}{S_x} = \frac{0.914}{0.609} = 1.50$$

$$\therefore \phi M_n = 0.9 Z_x F_y = 0.9(0.914 - in^3)36ksi = 29.6^{in-k} > 23.1^{in-k} = M_u \quad (11.71KN\text{-}m > 9.14KN\text{-}m)$$

O.K.

Use L7x4x3/4x0'-8" (L177.8mmX101.6mmX19.1mmX0.20m) for angle bracket

Design horizontal spreader bracket (Figures J1-3, and J1-4);
By inspection, the worst case moment is caused by the 6.60^k (29.4KN) load perpendicular to the Long leg of the angle bracket.

$M_u = 6.60^k(3.5") = 23.1^{in-k}$ (9.14KN/m)

By inspection, the shape is compact and the unbraced length is negligible so that equation F1-1 of AISC LRFD may be used.
Try C8x11.5;

$Z_x = 9.55\text{-}in^3$ ($156.5X10^3\ mm^3$) $\quad S_x = 8.14\text{-}in^3$ ($144.4X10^3\ mm^3$) $\quad \frac{Z_x}{S_x} = \frac{9.55}{8.14} = 1.17 < 1.5$

$$\therefore \phi M_n = 0.9 Z_x F_y = 0.9(9.55 - in^3)36ksi = 309^{in-k} > 23.1^{in-k} = M_u \quad (122.20KN\text{-}m > 9.14KN\text{-}m)$$

O.K.

Use C8x11.5 (C203.2mmX0.17KN/m) for horizontal spreader

Design bracket arm to connect horizontal spreader to web of floor beam (Figure J1-3, and J1-4);
Assume that the spreader bracket distributes the load evenly between two bracket arms supporting the two guide rails on either side of the counter weight. Therefore, a simple bracket arm takes half of the loading. Also, note that the welded connection is eccentrically loaded.

Try TS 4x4x1/4 (TS 101.6mmX101.6mmX6.4mm) with a 3/16-in. (4.8mm) fillet weld all around;

Check TS 4x4x1/4 in flexure;
By inspection, the worst case moment is caused by the 6.60^k (29.4KN) load perpendicular to the Long leg of the angle bracket. The moment arm is L = 11" + 3" + 0.22" = 14.22" (361.2mm) using the C8x11.5 web thickness of 0.22-in (5.6mm).

$\therefore M_u = (1/2)6.60^k(14.22") = 46.9^{in-k}$ (5.30KN-m)

For a TS 4x4x1/4;

$Z_x = 4.97\text{-}in^3 \quad S_x = 4.11\text{-}in^3 \quad \frac{Z_x}{S_x} = \frac{4.97}{4.11} = 1.21 < 1.5$

$$\therefore \phi M_n = 0.9 Z_x F_y = 0.9(4.97 - in^3)36ksi = 161^{in-k} > 46.9^{in-k} = M_u \quad (18.16KN\text{-}m > 5.30KN\text{-}m)$$

O.K.

Use TS 4x4x1/4 (TS 101.6mmX101.6mmX6.4mm) *for bracket to floor beam web*

Check weld of TS4x4x1/4 to beam web and to spreader bracket;

$\phi R_n = 0.75 t_e (0.60 F_{EXX})$ (per AISC LRFD J2.2)

$$\text{where;} \quad t_e = \frac{3''}{16}\left(\frac{1}{\sqrt{2}}\right) = 0.133-\text{in} \quad (3.38\text{mm})$$

$$\phi R_n = 0.75(0.133'')(0.60(70\text{ksi})) = 4.19^{k/in} \text{ for a 3/16-in. (4.8mm) weld}$$

Combined shear and bending of welds:

Note: Even through the tube steel section is welded all around, only the two vertical welds opposite the stiffeners on the other side of the beam web will be considered effective for bending. Formula for the section modulus of the weld group is taken from Table 5.18.1 of 'Steel Structures' by C. Salmon and J. Johnson, 3rd edition.

$$f_s = \frac{P}{A} = \frac{6.60^k}{(4)4''} = 0.41^{k/in} \quad (71.8\text{N/mm})$$

For a weld pattern formed from two parallel lines of length 'b' and spaced a length 'd' apart;

f_b: $$I_p = bd = 4''(4'') = 16.0 - \text{in}^3/\text{in} \quad (262.2\text{X}10^3 \text{ mm}^3)$$

where; b = 4" (101.6mm) = the length of the horizontal welds
d = 4" (101.6mm) = the length of the vertical welds

$$f_b = \frac{M_u}{S} = \frac{46.9^{in-k}}{16.0 - \text{in}^3/\text{in}} = 2.93^{k/in} \quad (0.51\text{KN/mm})$$

Therefore,

$$R_u = f_r = \sqrt{f_s^2 + f_b^2} = \sqrt{(0.41^{k/in})^2 + (2.93^{k/in})^2} = 2.96^{k/in} \quad (0.52\text{KN/mm})$$

$$\phi R_n = 4.19^{k/in} > 2.96^{k/in} = R_u \quad (0.73\text{KN-m} > 0.52\text{KN-m})$$ **O.K.**

Use a 3/16-in. (4.8mm) weld to attach TS bracket to beam web and spreader bracket

Design stiffeners for beam web;

To prevent out of plane buckling of the beam web, stiffeners will be added as shown in Figures J1-3, and J1-4. By inspection, two 1/4-in. (6.4mm) thick stiffeners spaced at 4-in. (101.6mm) apart and connected with a 3/16-in. (4.8mm) fillet weld as shown will be sufficient.

Note: By inspection, the deflection of the guide rail due to the deflection of the stiffeners combined with the deflection of the bracket arm is less then 1/2-in. (12.7mm).

Use two 1/4-in. (6.4mm) thick stiffeners attached with a 3/16-in. (4.8mm) fillet weld