Woodland Park Baptist Church
RE: S2.4 Connection at AA-A11
W16x36 (-54kip)
Check strength of tube wall for axial force (shear strength and yielding)
Reference HSS Conections manual pages 6-45 and 6-46

B = width of hollow section face with gusset attached
t = HSS wall thickness, in.
t1 = thickness of gusset plate, in.
N = length of gusset plate, in.
Fy = specified minimum yield stress of the HSS, ksi
Qf = 1.0 for tension in HSS
Qf = 1-0.3(f/Fy)-0.3(f/Fy)^2 for compression in HSS
f = magnitude of compression stress

Apply a factor of 0.67 to computed yielding value for ASD.
Apply a factor of 0.40 to computed shear value for ASD.

$\mathrm{kip} := 1000 \cdot \mathrm{lb} \qquad \mathrm{Ac} := 16 \cdot \mathrm{in}^2$

$f := \dfrac{(23 \cdot \mathrm{kip} + 15 \cdot \mathrm{kip} + 22 \cdot \mathrm{kip})}{\mathrm{Ac}} \quad B := 12 \cdot \mathrm{in} \quad t := 0.348 \cdot \mathrm{in} \qquad t1 := 0.5 \cdot \mathrm{in}$

$N := 12 \cdot \mathrm{in} \qquad Fy := 46 \cdot \dfrac{\mathrm{kip}}{\mathrm{in}^2}$

$Qf := 1 - 0.3 \left(\dfrac{f}{Fy}\right) - 0.3 \left(\dfrac{f}{Fy}\right)^2$

$Qf = 0.9735$

Compute allowable load due to yielding of the HSS wall

$$Py := \frac{\left(0.67 \cdot Fy \cdot t^2\right) \cdot \left[\left(2 \cdot \dfrac{N}{B}\right) + 4 \cdot \sqrt{\left(1 - \dfrac{t1}{B}\right)}\right] \cdot Qf}{\left(1 - \dfrac{t1}{B}\right)}$$

$Py = 22.4308 \cdot \mathrm{kip}$ **Yielding of the HSS wall**

******THRU-PLATE REQUIRED******
Allowable load = 44.8 kips using thru-plate
Assume horizontal diaphragm and column cap plate will transfer the additional 10 kips
The W14x30 frames into the corner using double 1/2" bent plates 10 1/2" long with (8) 3/4ϕ bolts in double shear.

$Ps := 0.4 \cdot Fy \cdot t \cdot N$

$Ps = 76.8384 \cdot \mathrm{kip}$ **Shear strength of the HSS wall is OK**

Check plate buckling for typical perimeter shear connection with axial force.
Connection type is thru-plate. Maximum unbraced length of thru-plate is 12" at HSS12x12 column.
Since a sidesway mode of buckling can occur, use K=1.2
Check PL5/8x12 and PL1/2x12 A36 material

$kip := 1000 \cdot lb$ $\quad E := 29000 \dfrac{kip}{in^2}$ $\quad Fy := 36 \cdot \dfrac{kip}{in^2}$

$K := 1.2$ $\quad l := 12 \cdot in$ $\quad r := \dfrac{0.625 \cdot in}{\sqrt{12}}$ $\quad Cc := \sqrt{\dfrac{\left(2 \cdot \pi^2 \cdot E\right)}{Fy}}$ $\quad Cc = 126.0993$

Slenderness ratio = Sr $\quad Sr := K \cdot \dfrac{l}{r}$ $\quad Sr = 79.8129$

When Kl/r is less than Cc, use equation E2-1 to calculate allowable stress

$$Fa := \left(1 - \frac{Sr^2}{2 \cdot Cc^2}\right) \cdot \frac{Fy}{\left(\frac{5}{3}\right) + \left(3 \cdot \frac{Sr}{8 \cdot Cc}\right) - \left(\frac{Sr^3}{8 \cdot Cc^3}\right)}$$

$$Fa = 15.3761 \circ \frac{kip}{in^2}$$

Allowable load using PL5/8x12 thru-plate (P): $\quad P := Fa \cdot 0.625 \cdot in \cdot 12 \cdot in$

$P = 115.3208 \circ kip$

$$r := \frac{0.5 \cdot in}{\sqrt{12}}$$

$Sr := K \cdot \dfrac{l}{r}$ $\quad Sr = 99.7661$

$$Fa := \left(1 - \frac{Sr^2}{2 \cdot Cc^2}\right) \cdot \frac{Fy}{\left(\frac{5}{3}\right) + \left(3 \cdot \frac{Sr}{8 \cdot Cc}\right) - \left(\frac{Sr^3}{8 \cdot Cc^3}\right)}$$

$$Fa = 13.0074 \circ \frac{kip}{in^2}$$

Allowable load using PL1/2x12 thru-plate (P): $\quad P := Fa \cdot 0.5 \cdot in \cdot 12 \cdot in$

$P = 78.0442 \circ kip$