Project 1 - Report

SE 140A - Group 18

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February 3, 2026

3. Composite Girder 2 Design
    a. Cost Estimate
    b. Stud Placement
    c. Design Drawing

I. Introduction

This project details the gravity and seismic load-carrying system of an eight story structure. The structure is made of concrete slabs on a ___**fill in**____ metal deck and supported with steel framing. Each story is 15 feet high and a 2 hour fire rating is required.

The first portion, Project 1A, involved the design of steel gravity columns along the full length of the building, the required column splices, and the base plates.

Project 1B involves composite beam and girder design.

Table 1.1 - Building Information

Building Information	
# of Stories	8
Story Height (ft)	15
Bay Width (ft)	35
Requirements:	2-hr fire rating

Table 1.2 - Dead Load

DL Type	Floor DL (psf)	Roof DL (psf)
Concrete Slab	43.5	0
3-in Metal Deck	2.2	2.2
Steel Framing	15	12
Superimposed (MEP, misc.)	15	12
TOTAL:	75.7	26.2

Table 1.3 - Additional Loads

Load Type	Notes	Floor Load (psf)	Roof Load (psf)
Facade (DL)	Only considered for edge columns	13	13
Partitions (LL)	Unreduced live load (added separately)	15	0

II. Gravity Load Resisting System: Columns

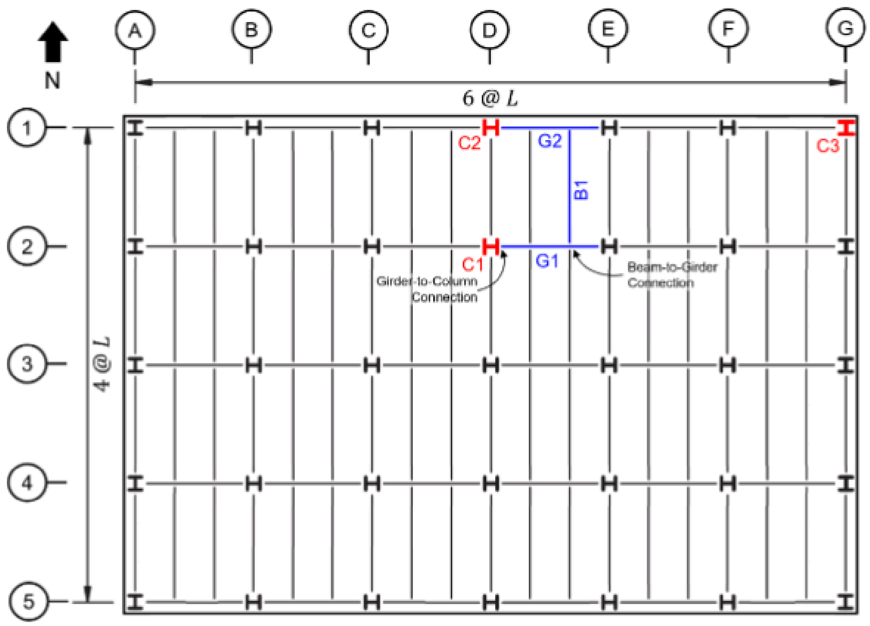
The building's steel columns are designed with the following specifications:

- Spans the full height of the building (all eight floors)
- Two splices (at the third and sixth floor)
- Base plate at the bottom-most column (column 3)
- The bay width between each column is 35 feet (L=35').

Each team member was responsible for designing a different column.

- Column C1: Chris Liu
- Column C2: Hannah Ahn
- Column C3: Isaac Kim

Figure 1 - Building Floor Plan



1. Gravity Column Design

The loading on each floor was determined by loading combinations of the dead load, live load, and roof load. The maximum loading from the various floors each beam spanned was used as the gravity loading demand. The best fit W shape was then found in the steel manual Table 4-1a.

Dead loads were calculated by calculating the weight of a single floor, Pd, and the curtain wall at that floor.

$$Pd = A_T * Area\ DL$$

$$Curtain\ Wall = Facade\ Load * Story\ Height * Bay\ Width$$

The loading of the floor and curtail wall of the set floor and all the ones above it were summed and is highlighted as "Stacked Pd".

Live loads were reduced with the reduction factor, Ψ, and added with all the loadings of the floors above it. L refers to the reduced live area load and R_LL refers to reduced roof live area load. P_L is the concentrated loading on the column found by multiplying the live load with tributary area, A_T.

$$\Psi_i = 0.25 + \frac{15}{\sqrt{K_{LL}(8-i)(A_T)}} \geq 0.4$$

$$L = \Psi_i * L_0$$

P_L = L * A_T

The partition live load, Partition LL, is the partition weight of 15 psf multiplied by the tributary area.

The total live load of a single floor, Total LL, was calculated by adding the concentrated loading, P_L, and partition loading, Partition LL. The stacked live load is the sum of the loading on a single floor and all the floors above it.

Column C1:

Table 2.1.1 - Column C1 Dead Loads

		Dead Load		
Floor	At (ft^2)	Area DL (psf)	Pd (lb)	Stacked Pd (lb)
Roof	1225	26.2	32095	32095
8	2450	75.7	92732.5	124827.5
7	3675	75.7	92732.5	217560
6	4900	75.7	92732.5	310292.5
5	6125	75.7	92732.5	403025
4	7350	75.7	92732.5	495757.5
3	8575	75.7	92732.5	588490
2	9800	75.7	92732.5	681222.5

Table 2.1.2 - Column C1 Live Loads

	Live Load								
Floor	Ψ	Lo (psf)	L (psf)	R_LL (psf)	P_L (lb)	Partition LL (lb)	Total LL (lb)	Stacked Live Load (lb)	P_RLL(lb)
Roof	0.6	20	0	12.00	0.0	0.00	0.00		14700.00
8	0.402	80	32.12		39349.2	18375.00	57724.24	57724.24	
7	0.374	80	32.00		39200.0	18375.00	57575.00	115299.24	
6	0.357	80	32.00		39200.0	18375.00	57575.00	172874.24	
5	0.346	80	32.00		39200.0	18375.00	57575.00	230449.24	
4	0.337	80	32.00		39200.0	18375.00	57575.00	288024.24	
3	0.331	80	32.00		39200.0	18375.00	57575.00	345599.24	
2	0.326	80	32.00		39200.0	18375.00	57575.00	403174.24	

Table 2.1.3 - Column C1 Load Combinations

Column C1		**LC 1**	**LC 2**	**LC 3**	**LC Final**
Story	Floor	Pu = 1.4D (kips)	Pu = 1.2D + 1.6L + 0.5Lr (kips)	Pu = 1.2D + **0.5L +** 1.6Lr (kips)	**Pu_final (kips)**
8	Roof	44.93	45.86	62.03	62.03
7	8	174.76	249.50	202.18	249.50
6	7	304.58	452.90	313.38	**452.90**
5	6	434.41	656.30	424.66	656.30
4	5	564.24	859.70	535.94	859.70
3	4	694.06	1063.10	647.22	**1063.10**
2	3	823.89	1266.50	758.50	1266.50
1	2	953.71	1469.90	869.77	**1469.90**

Table 2.1.4 - Columns selected for C1

Column	Column Selection	Capacity (kip)	Demand (kip)
1	W12x53	478	452.90
2	W12x106	1100	1063.10
3	W12x152	1590	1469.90

Column C2:

Table 2.1.5 - Column C2 Dead Load

Column C2				Dead Load			
Story	Floor	Above Story Height (ft)	At (ft^2)	Area DL (psf)	Curtain Wall (lb)	Pd (lb)	Stacked Pd (lb)
8	Roof	6	647.5	26.2	2730	16964.5	19694.5
7	8	15	1295	75.7	6825	49015.75	75535.25
6	7	15	1942.5	75.7	6825	49015.75	131376
5	6	15	2590	75.7	6825	49015.75	187216.75
4	5	15	3237.5	75.7	6825	49015.75	243057.5
3	4	15	3885	75.7	6825	49015.75	298898.25
2	3	15	4532.5	75.7	6825	49015.75	354739
1	2	15	5180	75.7	6825	49015.75	410579.75

Table 2.1.6 - Column C2 Live Loads

	Live Load								
Floor	Ψ	Lo (psf)	L (psf)	R_LL (psf)	P_L (lb)	Partition LL (lb)	Total LL (lb)	Stacked Live Load (lb)	P_RLL(lb)
Roof	0.6	20	0	12.00	0.0	0.00	0.00		14700.00
8	0.402	80	32.12		39349.2	18375.00	57724.24	57724.24	
7	0.374	80	32.00		39200.0	18375.00	57575.00	115299.24	
6	0.357	80	32.00		39200.0	18375.00	57575.00	172874.24	
5	0.346	80	32.00		39200.0	18375.00	57575.00	230449.24	
4	0.337	80	32.00		39200.0	18375.00	57575.00	288024.24	
3	0.331	80	32.00		39200.0	18375.00	57575.00	345599.24	
2	0.326	80	32.00		39200.0	18375.00	57575.00	403174.24	

Table 2.1.7 - Column C2 Load Combinations

Column C2		Load Comb. 1	Load Comb. 2	Load Comb. 3	Load Comb. Final
Story	Floor	Pu = 1.4D (kips)	Pu = 1.2D + 1.6L + 0.5Lr (kips)	Pu = 1.2D + **0.5L +** 1.6Lr (kips)	Pu_final (kips)
8	Roof	27.57	27.52	36.07	36.07
7	8	105.75	150.73	120.64	150.73

6	7	183.93	270.29	204.07	**270.29**
5	6	262.10	387.66	286.82	387.66
4	5	340.28	503.54	369.10	503.54
3	4	418.46	619.24	451.32	**619.24**
2	3	496.63	734.95	533.55	734.95
1	2	574.81	850.65	615.77	**850.65**

Table 2.1.8 - Columns selected for C2

Column	Column Selection	Capacity (kip)	Demand (kip)
1	W12x40	281	270.29
2	W12x72	735	619.24
3	W12x87	896	850.65

Column C3:

Table 2.1.9 - Column C3 Dead Loads

Column C3				**Dead Load**			
Story	Floor	Above Story Height (ft)	At (ft^2)	Area DL (psf)	Curtain Wall (lb)	Pd (lb)	Stacked Pd (lb)
8	Roof	6	342.25	26.2	5460	8966.95	14426.95
7	8	15	684.5	75.7	13650	25908.325	53985.275
6	7	15	1026.75	75.7	13650	25908.325	93543.6
5	6	15	1369	75.7	13650	25908.325	133101.925
4	5	15	1711.25	75.7	13650	25908.325	172660.25
3	4	15	2053.5	75.7	13650	25908.325	212218.575
2	3	15	2395.75	75.7	13650	25908.325	251776.9
1	2	15	2738	75.7	13650	25908.325	291335.225

Table 2.1.10 - Column C3 Live Loads

	Live Load								
Floor	Ψ	Lo (psf)	L (psf)	R_LL (psf)	P_L (lb)	Partition LL (lb)	Total LL (lb)	Stacked Live Load (lb)	P_RLL (lb)
Roof	0.8577	20	0	17.16	0.0	0.00	0.00		5871.30
8	0.655	80	52.43		17945.0	5133.75	23078.75	23078.75	
7	0.581	80	46.48		15908.1	5133.75	21041.86	44120.61	
6	0.537	80	42.93		14693.9	5133.75	19827.64	63948.25	
5	0.506	80	40.51		13865.3	5133.75	18999.01	82947.25	
4	0.484	80	38.72		13253.6	5133.75	18387.34	101334.59	
3	0.467	80	37.34		12778.2	5133.75	17911.95	119246.54	
2	0.453	80	36.22		12395.0	5133.75	17528.75	136775.29	

Table 2.1.11 - Column C3 Load Combinations

Column C3		**LC 1**	**LC 2**	**LC 3**	**LC Final**
Story	Floor	Pu = 1.4D (kips)	Pu = 1.2D + 1.6L + 0.5Lr (kips)	Pu = 1.2D + **0.5L +** 1.6Lr (kips)	Pu_final (kips)
8	Roof	20.20	20.25	26.71	26.71
7	8	75.58	104.64	85.72	104.64

6	7	130.96	185.78	143.71	**185.78**
5	6	186.34	264.98	201.09	264.98
4	5	241.72	342.84	258.06	342.84
3	4	297.11	419.73	314.72	**419.73**
2	3	352.49	495.86	371.15	495.86
1	2	407.87	571.38	427.38	**571.38**

Table 2.1.12 - Columns selected for C3

Column	Column Selection	Capacity (kip)	Demand (kip)
1	W12x40	281	185.78
2	W12x53	478	419.73
3	W12x65	663	571.38

Design Drawings

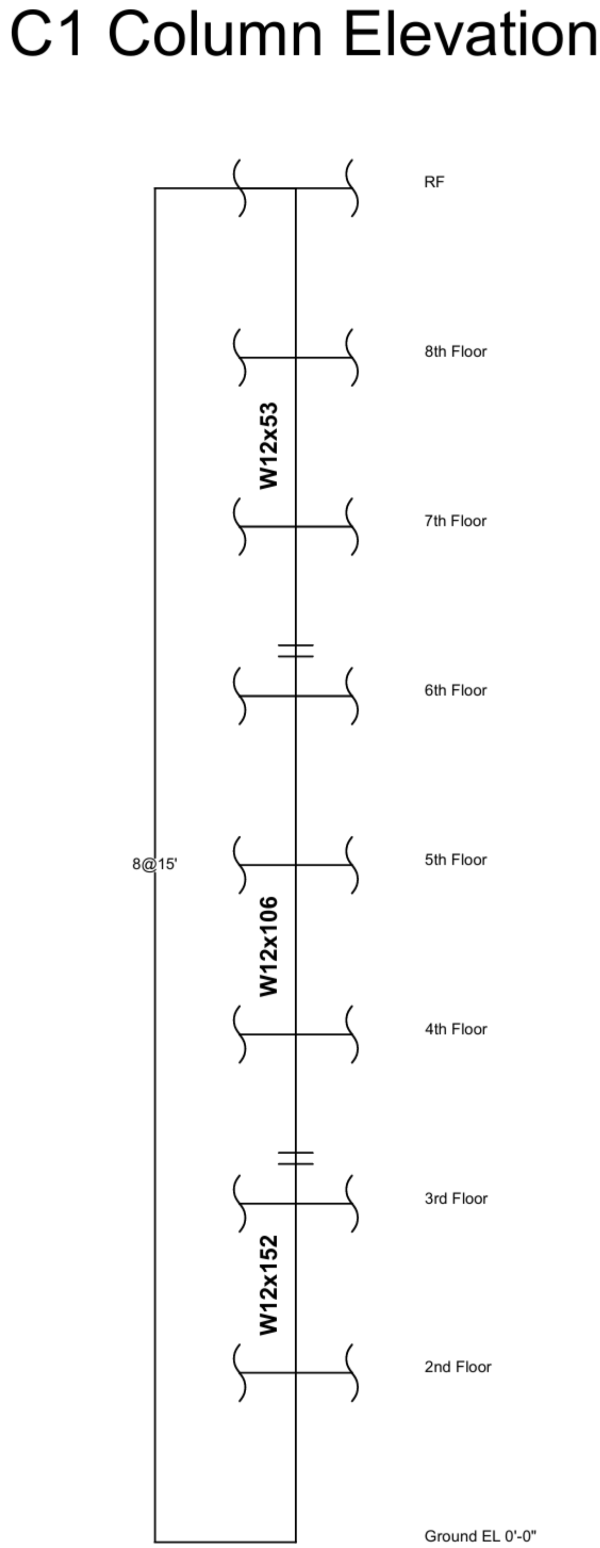


Figure 2.1.1 Column C1 elevation view with splicing locations.

C2 Column Elevation

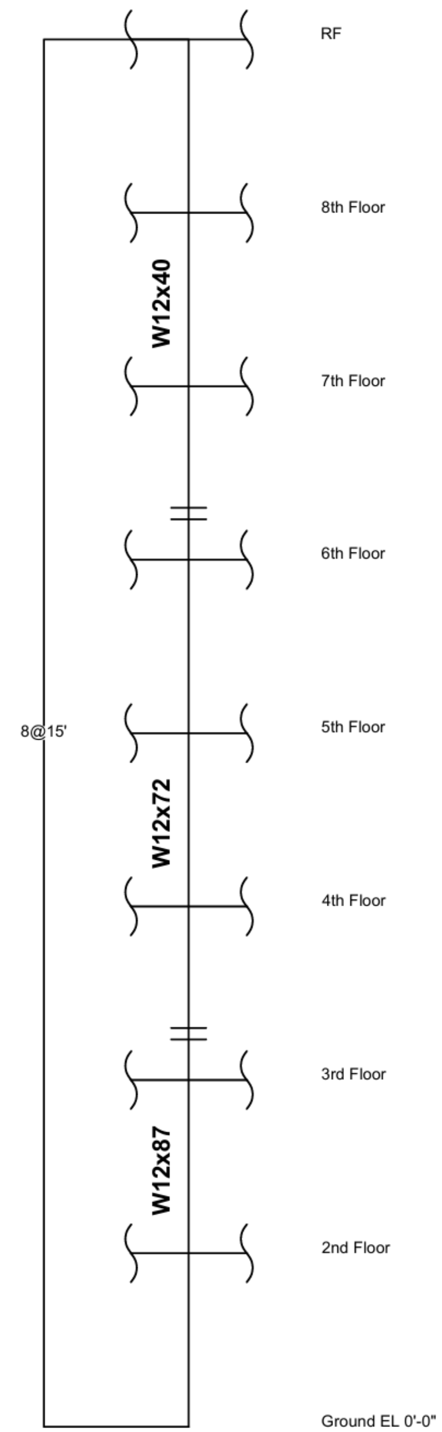


Figure 2.1.2 Column C2 elevation view with splicing locations.

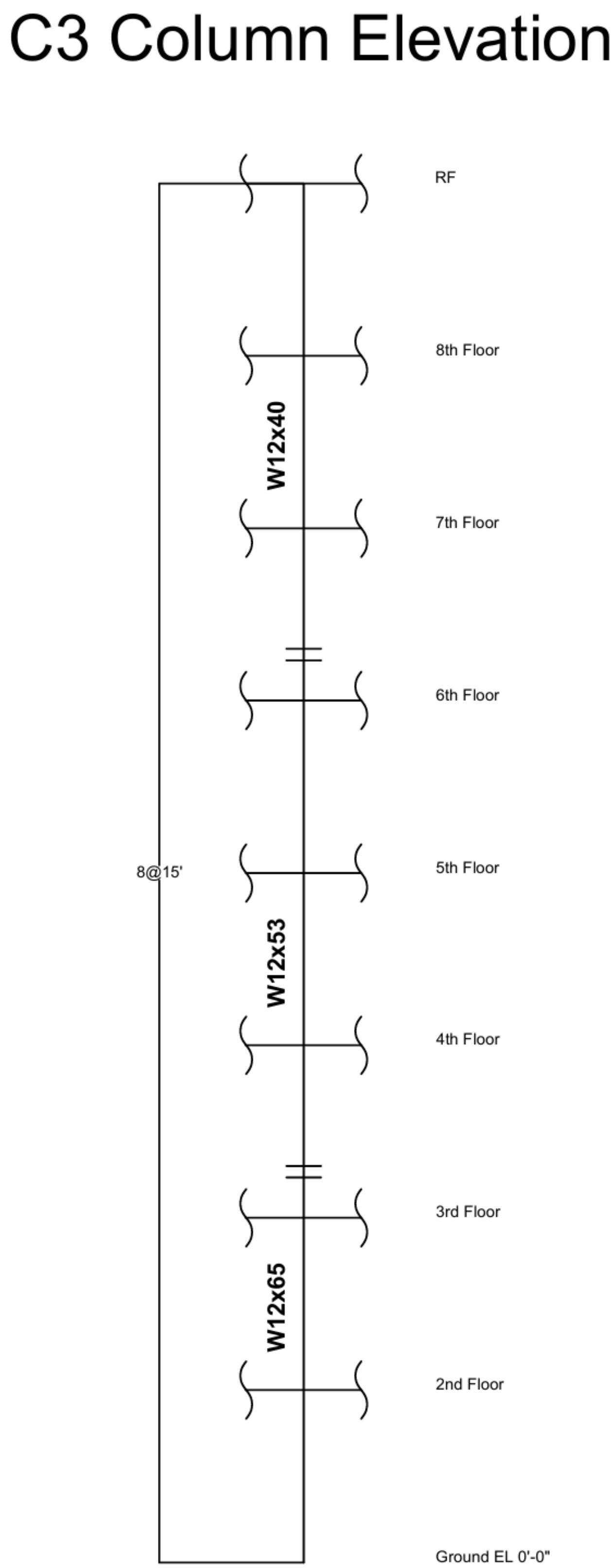


Figure 2.1.3 Column C3 elevation view with splicing locations.

2. *Column Splice Design*

Each column was spliced at the third and sixth story, but only the splice at the third story was designed. The column splice design was governed by Chapter 14 of the AISC Steel Construction Manual. A flange-plate splice was utilized for all three columns, and the splices were all fully bolted. First, a bearing strength check (AISC Eq. J7-1, p.16.1-148) was performed to determine the limit state of bearing (local compressive yielding). Next, the case was selected from AISC Table 14-3, and the sub-cases were selected as follows to determine the dimensions of the flange plates and fillers (if necessary).

Column C1:

<u>Bearing Strength Check</u>

$$\phi R_n = \phi(1.8)(F_y)(A_{pb}) \qquad \text{Eq. 2.2.1}$$

$$\phi = 0.75$$

$Fy = 50\, ksi$ → A992 grade steel

$A_{pb} = 31.2\, in^2$ → projected area in bearing (smallest area of spliced columns → W12X106)

$$\phi R_n = 0.75 * 1.8 * 50 * 31.2 = 2106\, kips$$

$$P_u = 1063.1\, kips$$

$\phi R_n \geq P_u \rightarrow 2106\, kips \geq 1063.1\, kips$ → **bearing strength satisfied**

<u>Case 1: Fully Bolted</u>

$d_l = (d_u + \frac{3}{4}\, in)$ and over   Eq. 2.2.2

$d_u = 12\frac{7}{8}$" → depth of upper column (W12X106)

$d_l = 13\frac{3}{4}$" → depth of lower column (W12X152)

$d_l - d_u = 13\frac{3}{4} - 12\frac{7}{8} = \frac{7}{8}" \geq \frac{3}{4}\, in$ → **Case 1-C**

<u>Case 1-C</u>

From AISC Table 14-3, flange plates dimensions are the same as Case 1-A → select g_u for upper column and select g_u and flange plate dimensions for the lower column. For the fillers, select the thickness as $(d_l - d_u)/2$ minus 1/8" or 3/16", whichever results in 1/8" multiples of the filler thickness. Select width to match flange plate, but not greater than upper column flange width. Select length as 1'-0" for Type 1 or 0'-9" for Type 2.

Flange plate dimensions:

- **Length = 1'-0 ½"**
- **Width = 8 in**
- **Thickness = 5/8 in**

Filler dimensions:

- **Length = 9 in**
- **Width = 8 in**
- **Thickness = 1/4 in**

Column C2:

Bearing Strength Check

$$\phi R_n = \phi(1.8)(F_y)(A_{pb}) \qquad \text{Eq. 2.2.1}$$

$$\phi = 0.75$$

$Fy = 50\ ksi$ → A992 grade steel

$A_{pb} = 21.1\ in^2$ → projected area in bearing (smallest area of spliced columns → W12X72)

$$\phi R_n = 0.75 * 1.8 * 50 * 21.1 = 1899\ kips$$

$$P_u = 619.24\ kips$$

$\phi R_n \geq P_u \rightarrow 1899\ kips \geq 619.24\ kips$ → **bearing strength satisfied**

Case 1: Fully Bolted

$$d_l = (d_u - \frac{1}{4}\ in)\ to\ (d_u + \frac{1}{8}\ in) \qquad \text{Eq. 2.2.2}$$

$d_u = 12\frac{1}{4}"$ → depth of upper column (W12X72)

$d_l = 12\frac{1}{2}"$ → depth of lower column (W12X87)

$d_l - d_u = 12\frac{1}{2} - 12\frac{1}{4} = \frac{1}{4}"$ → **Case 1-A**

Case 1-A

From AISC Table 14-3, select g_u for upper column and select g_u and flange plate dimensions for the lower column. No filler.

Flange plate dimensions:

- **Length = 1'-0 ½"**
- **Width = 8 in**
- **Thickness = 3/8 in**

Column C3:

Bearing Strength Check

$$\phi R_n = \phi(1.8)(F_y)(A_{pb}) \qquad \text{Eq. 2.2.1}$$

$$\phi = 0.75$$

$Fy = 50\ ksi$ → A992 grade steel

$A_{pb} = 15.6\ in^2$ → projected area in bearing (smallest area of spliced columns → W12X53)

$$\phi R_n = 0.75 * 1.8 * 50 * 15.6 = 1053\ kips$$

$$P_u = 420\ kips$$

$\phi R_n \geq P_u \rightarrow 1053\ kips \geq 420\ kips$ → **bearing strength satisfied**

Case 1: Fully Bolted

$$d_l = (d_u - \frac{1}{4}\ in)\ to\ (d_u + \frac{1}{8}\ in) \qquad \text{Eq. 2.2.2}$$

$d_u = 12"$ → depth of upper column (W12X53)

$d_l = 12\frac{1}{8}"$ → depth of lower column (W12X65)

$d_l - d_u = 12\frac{1}{8} - 12 = \frac{1}{8}"$ → **Case 1-B**

Case 1-B

From AISC Table 14-3, flange plates dimensions are the same as Case 1-A → select g_u for upper column and select g_u and flange plate dimensions for the lower column. For the fillers, select the thickness as $\frac{1}{8}"$ for $d_l = d_u$ and $d_l = (d_u + \frac{1}{8}in.)$ or as $\frac{1}{4}"$ for $d_l = (d_u - \frac{1}{8}in.)$ and $d_l = (d_u + \frac{1}{8}in.)$ Select width to match flange plate and length as 0’-9” for Type 1 or 0’-6” for Type 2.

Flange plate dimensions:

- **Length = 1’-0 ½”**
- **Width = 8 in**

- **Thickness = 3/8 in**

Filler dimensions:

- **Length = 6 in**
- **Width = 8 in**
- **Thickness = 1/8 in**

Design Drawings

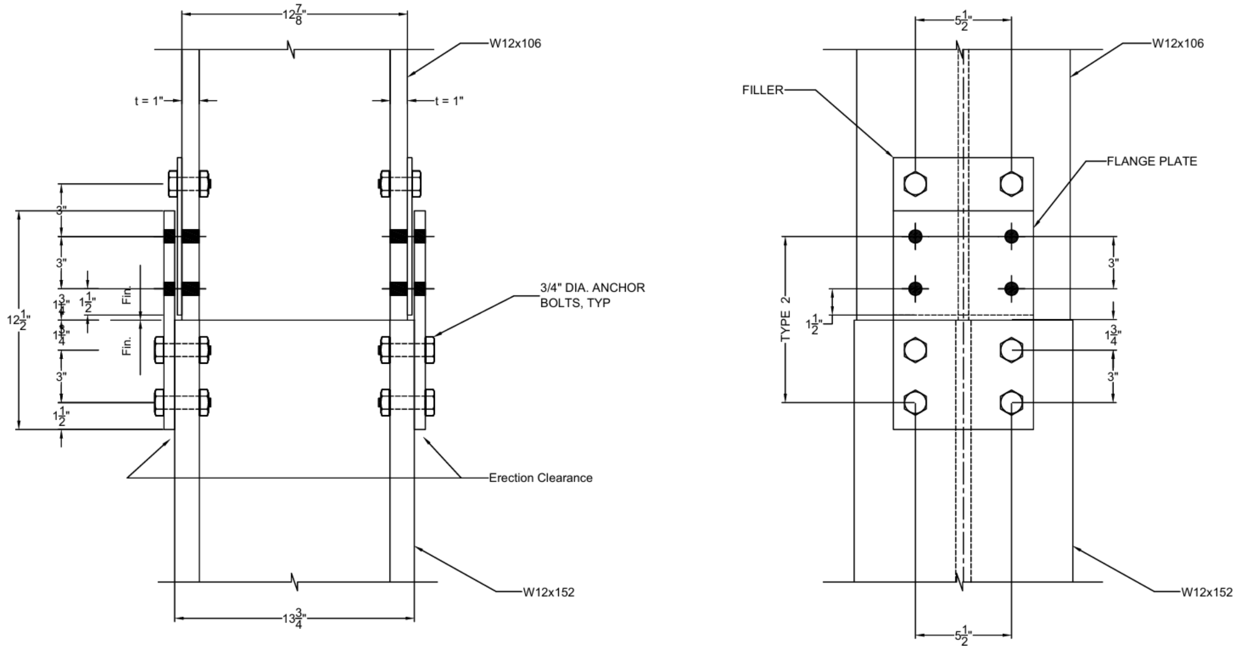


Figure 2.2.1 Column C1 splice design (Case 1-C)

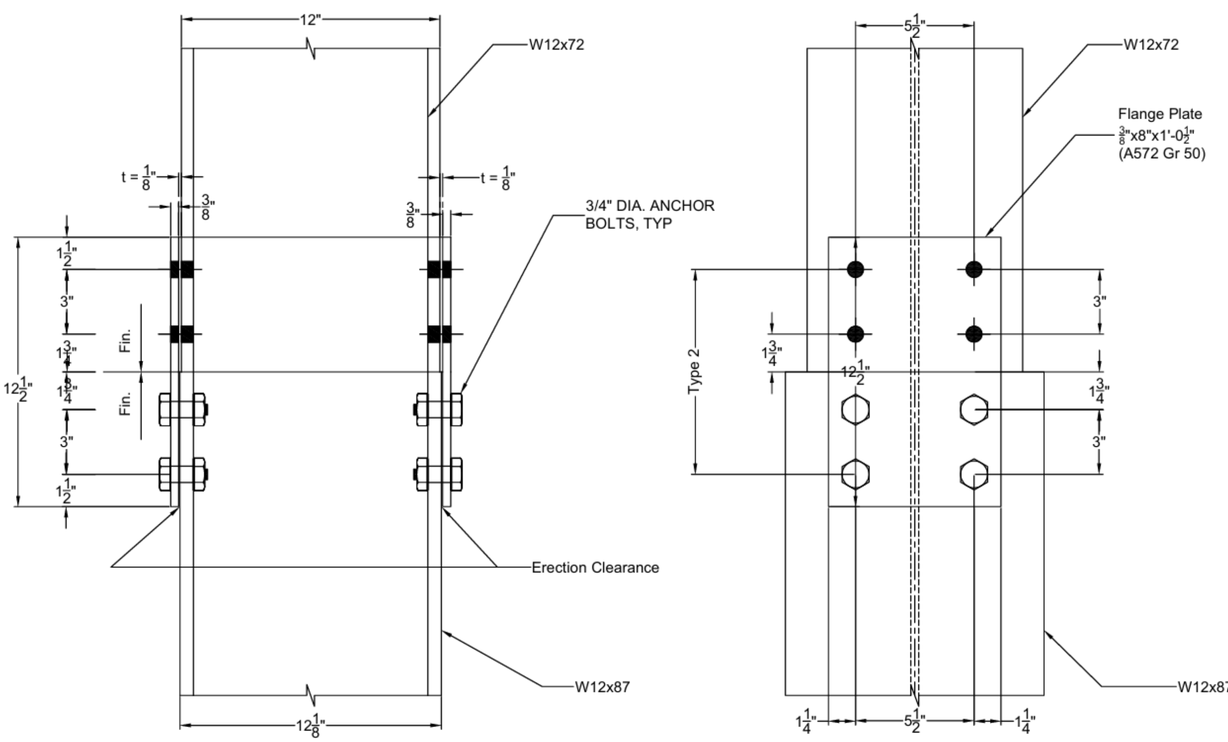


Figure 2.2.2 Column C2 splice design (Case 1-A)

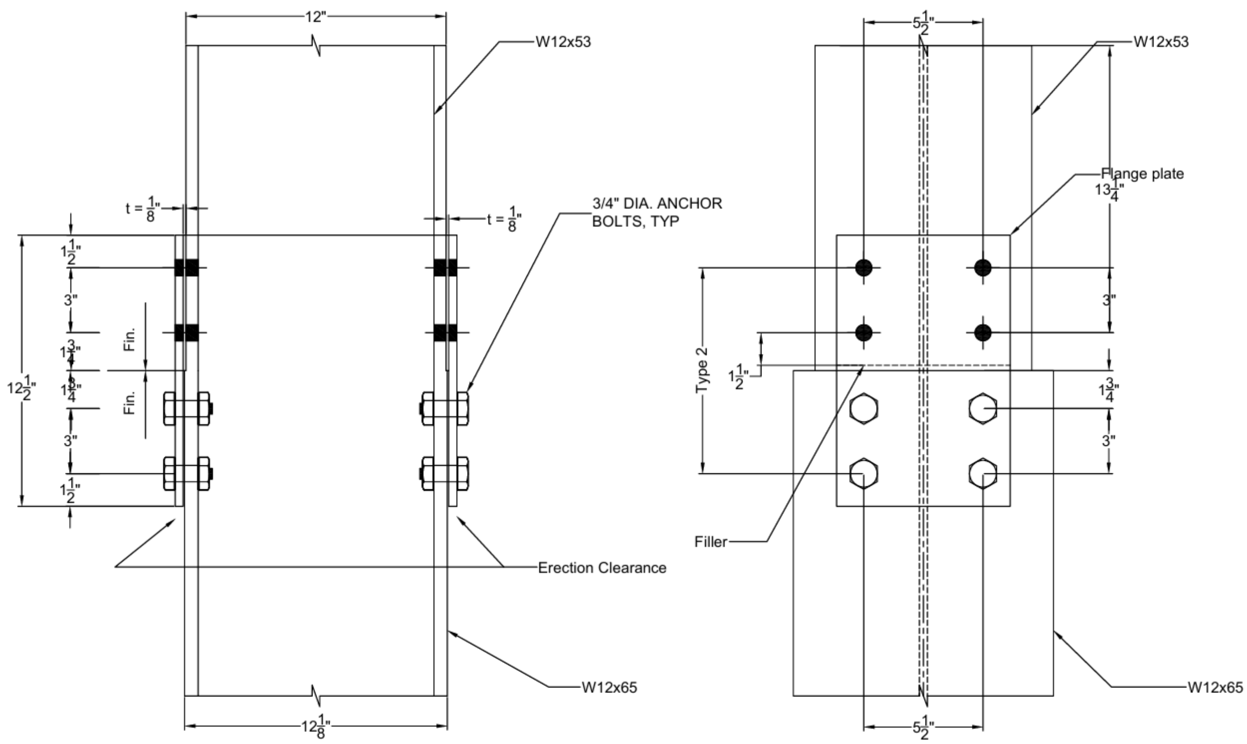


Figure 2.2.3 Column C3 splice design (Case 1-B)

3. Base Plate Design

A base plate was also to be designed for each column. The design was governed by Chapter 14 of the AISC Steel Construction Manual. First, a preliminary length and width of the base plate was determined. Next, the bearing strength of the concrete footing was checked in section J8 of the AISC Specifications, p. 16.1-149. This section also allowed the length and width of the base plate to be determined. Then, the cantilever method in AISC Chapter 14 was used to determine the thickness of the base plate.

Column C1:

Preliminary Length and Width

Length: $N > d + 3" * 2$ Eq. 2.3.1

Width: $B > b_f + 3" * 2$ Eq. 2.3.2

$$N = 13.7 + 3 * 2 = 19.7" \approx \textbf{20 in}$$

$$B = 12.5 + 3 * 2 = 18.5" \approx \textbf{20 in}$$

Concrete Bearing Strength

$\phi_c P_p = \phi_c * 0.85 * f'_c * A_1$ Eq. 2.3.2

$$\phi_c = 0.65$$

$$f'_c = 4000\, psi = 4\, ksi$$

$$A_1 = B * N = 20 * 20 = 400\, in^2$$

$$\phi_c P_p = 0.65 * 0.85 * 4 * 400 = 884\, kips$$

$$P_u = 1469.9\, kips$$

$\phi_c P_p \geq P_u \rightarrow 884\, kips \leq 1469.9\, kips \rightarrow$ **bearing strength NOT satisfied → need to resize**

Resize Using Alternative Method

$A_{1(req)} \geq \frac{P_u}{\phi_c * 0.85 * f'_c}$ Eq. 2.3.3

$$A_{1(req)} \geq \frac{1469.9}{0.65*0.85*4} = 665.11\, in^2$$

$$N = B = \sqrt{A_{1(req)}} = \sqrt{665.11} = 25.78\, in \approx \mathbf{26\ in}$$

Thickness Using Cantilever Method

$l = max\{m, n, \lambda n'\}$ Eq. 2.3.4

$m = \frac{N - 0.95d}{2}$ Eq. 2.3.5

$n = \frac{B - 0.8b_f}{2}$ Eq. 2.3.6

$n' = \frac{\sqrt{d * b_f}}{4}$ Eq. 2.3.7

$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1 - x}} \leq 1$ Eq. 2.3.8

$X = [\frac{4d * b_f}{(d + b_f)^2}] \frac{P_u}{\phi_c P_p} \leq 1$ Eq. 2.3.9

$$m = \frac{26 - 0.95*13.7}{2} = 6.49\, in$$

$$n = \frac{26-0.8*12.5}{2} = 8\ in$$

$$n' = \frac{\sqrt{13.7*12.5}}{4} = 3.27\ in$$

$$X = [\frac{4*13.7*12.5}{(13.7+12.5)^2}]\frac{1469.9}{884} = 1.65 \leq 1 \rightarrow X = 1$$

$$\lambda = \frac{2\sqrt{1}}{1+\sqrt{1-1}} = 2 \leq 1 \rightarrow \lambda = 1$$

$$\lambda n' = 1 * 3.27 = 3.27"$$

$$l = max\{6.49,\ 8,\ 3.27\} = 8\ in$$

To compute the demand, M_u: By Newton's third law, $P_u = P_p = 1469.9\ kips$

Area load: $\sigma_u = \frac{P_p}{B*N}$ Eq. 2.3.10

Uniformly distributed load: $w_u = \sigma_u * W_T$ Eq. 2.3.11

Moment demand: $M_u = \frac{w_u*l^2}{2} = [\frac{P_p}{B*N}(W_T)]\frac{l^2}{2}$ Eq. 2.3.12

$$\sigma_u = \frac{1469.9}{26*26} = 2.174\ ksi$$

$$w_u = 2.174(26) = 56.53\ kip/in$$

$$M_u = \frac{56.53*8^2}{2} = 1809\ kip \cdot in$$

Computing the thickness using supply, ϕM_p where $\phi = 0.90$

$M_p = Z_x * F_y = \frac{W_T*t_p^2}{4} * F_y$ Eq. 2.3.13

$M_d = \phi M_p \geq M_u$ Eq. 2.3.14

Substituting Eq. 2.3.13 into Eq. 2.3.14 produces the following:

$t_p = l * \sqrt{\frac{2*P_u}{\phi F_y*B*N}}$ Eq. 2.3.15

$$t_p = 8 * \sqrt{\frac{2*1469.9}{0.9*50*26*26}} = 2.48\ in$$

Rounding to the nearest 1/4" produces a thickness of **2.5 in**

Base plate dimensions: **length = 26 in; width = 26 in; thickness = 2.5 in**

Column C2:

Preliminary Length and Width

Length: $N > d + 3" * 2$ Eq. 2.3.1

Width: $B > b_f + 3" * 2$ Eq. 2.3.2

$$N = 12.5" + 3" * 2 = 18.5" \approx \mathbf{19\ in}$$

$$B = 12\frac{1}{8} + 3" * 2 = 18\frac{1}{8}" \approx \mathbf{19\ in}$$

Concrete Bearing Strength

$\phi_c P_p = \phi_c * 0.85 * f'_c * A_1$ Eq. 2.3.2

$$\phi_c = 0.65$$

$$f'_c = 4000\, psi = 4\, ksi$$

$$A_1 = B * N = 19" * 19" = 361\, in^2$$

$$\phi_c P_p = 0.65 * 0.85 * 4 * 361 = 797.81\, kips$$

$$P_u = 850.65\, kips \text{ (W12x87)}$$

$\phi_c P_p \geq P_u \rightarrow 797.8\, kips \leq 850.7\, kips \rightarrow$ **bearing strength NOT satisfied → need to resize**

Resize Using Alternative Method

$A_{1(req)} \geq \frac{P_u}{\phi_c * 0.85 * f'_c}$ Eq. 2.3.3

$$A_{1(req)} \geq \frac{850.65}{0.65*0.85*4} = 389.5\, in^2$$

$$N = B = \sqrt{A_{1(req)}} = \sqrt{389.5} = 19.7\, in \approx \mathbf{20\ in}$$

Thickness Using Cantilever Method

$l = max\{m, n, \lambda n'\}$ Eq. 2.3.4

$m = \frac{N-0.95d}{2}$ Eq. 2.3.5

$n = \frac{B-0.8b_f}{2}$ Eq. 2.3.6

$n' = \frac{\sqrt{d*b_f}}{4}$ Eq. 2.3.7

$\lambda = \frac{2\sqrt{X}}{1+\sqrt{1-x}} \leq 1$ Eq. 2.3.8

$X = [\frac{4d*b_f}{(d+b_f)^2}]\frac{P_u}{\phi_C P_p} \leq 1$ Eq. 2.3.9

$$m = \frac{20 - 0.95(12.5)}{2} = 4.06\ in$$

$$n = \frac{20 - 0.8(12\frac{1}{8})}{2} = 5.15\ in$$

$$n' = \frac{\sqrt{12.5*12\frac{1}{8}}}{4} = 3.08\ in$$

$$X = [\frac{4*12.5*12\frac{1}{8}}{(12.5+12\frac{1}{8})^2}]\frac{850.65}{797.8} = 0.962 \leq 1$$

$$\lambda = \frac{2\sqrt{0.962}}{1+\sqrt{1-0.962}} = 1.64 \leq 1 \rightarrow \lambda = 1$$

$$\lambda n' = 1 * 3.08 = 3.08''$$

$$l = max\{4.06'', 5.15'', 3.08''\} = 5.15\ in$$

To compute the demand, M_u: By Newton's third law, $P_u = P_p = 850.65\ kips$

Area load: $\sigma_u = \frac{P_p}{B*N}$ Eq. 2.3.10

Uniformly distributed load: $w_u = \sigma_u * W_T$ Eq. 2.3.11

Moment demand: $M_u = \frac{w_u*l^2}{2} = [\frac{P_p}{B*N}(W_T)]\frac{l^2}{2}$ Eq. 2.3.12

$$\sigma_u = \frac{850.65}{20*20} = 2.15\ ksi$$

$$w_u = 2.15(20) = 43.03\ kip/in$$

$$M_u = \frac{43.03*5.15^2}{2} = 1141.3\ kip \cdot in$$

Computing the thickness using supply, ϕM_p where $\phi = 0.90$

$$M_p = Z_x * F_y = \frac{W_T * t_p^{\ 2}}{4} * F_y \qquad \text{Eq. 2.3.13}$$

$$M_d = \phi M_p \geq M_u \qquad \text{Eq. 2.3.14}$$

Substituting Eq. 2.3.13 into Eq. 2.3.14 produces the following:

$$t_p = l * \sqrt{\frac{2*P_u}{\phi F_y * B * N}} \qquad \text{Eq. 2.3.15}$$

$$t_p = 5.15 * \sqrt{\frac{2*850.65}{0.9*50*20*20}} = 1.58\ in$$

Rounding to the nearest 1/4" produces a thickness of $1\frac{3}{4}$ **in**

Base plate dimensions: **length = 20 in; width = 20 in; thickness = 1 3/4 in**

Column C3:

Preliminary Length and Width

Length: $N > d + 3" * 2$ Eq. 2.3.1

Width: $B > b_f + 3" * 2$ Eq. 2.3.2

$$N = 12.125 + 3 * 2 = 18.125" \approx \mathbf{19\ in}$$

$$B = 12 + 3 * 2 = 18" \approx \mathbf{19\ in}$$

Concrete Bearing Strength

$$\phi_c P_p = \phi_c * 0.85 * f'_c * A_1 \qquad \text{Eq. 2.3.2}$$

$$\phi_c = 0.65$$

$$f'_c = 4000\ psi = 4\ ksi$$

$$A_1 = B * N = 19 * 19 = 361\ in^2$$

$$\phi_c P_p = 0.65 * 0.85 * 4 * 361 = 797.81\ kips$$

$$P_u = 420\ kips$$

$\phi_c P_p \geq P_u \rightarrow 797.81\ kips \geq 420\ kips \rightarrow$ **Bearing Strength SATISFIED**

<u>Thickness Using Cantilever Method</u>

$l = max\{m, n, \lambda n'\}$ Eq. 2.3.4

$m = \frac{N-0.95d}{2}$ Eq. 2.3.5

$n = \frac{B-0.8b_f}{2}$ Eq. 2.3.6

$n' = \frac{\sqrt{d*b_f}}{4}$ Eq. 2.3.7

$\lambda = \frac{2\sqrt{X}}{1+\sqrt{1-x}} \leq 1$ Eq. 2.3.8

$X = [\frac{4d*b_f}{(d+b_f)^2}]\frac{P_u}{\phi_C P_p} \leq 1$ Eq. 2.3.9

$$m = \frac{19-0.95*12.125}{2} = 3.74"$$

$$n = \frac{19-0.8*12}{2} = 4.7"$$

$$n' = \frac{\sqrt{12.125*12}}{4} = 3.0156"$$

$$X = [\frac{4*12.125*12}{(12.125+12)^2}]\frac{420}{798} = 0.5263 \leq 1 \rightarrow X = 0.5263$$

$$\lambda = \frac{2\sqrt{0.5263}}{1+\sqrt{1-0.5263}} = 0.8594 \leq 1 \rightarrow \lambda = 0.8594$$

$$\lambda n' = 0.8594 * 3.0156 = 2.59"$$

$$l = max\{3.74,\ 4.7,\ 2.59\} = 4.7\ in$$

To compute the demand, M_u: By Newton's third law, $P_u = P_p = 797.81/0.65 = 1227.4\ kips$

Area load: $\sigma_u = \frac{P_p}{B*N}$ Eq. 2.3.10

Uniformly distributed load: $w_u = \sigma_u * W_T$ Eq. 2.3.11

Moment demand: $M_u = \frac{w_u * l^2}{2} = [\frac{P_p}{B*N}(W_T)]\frac{l^2}{2}$ Eq. 2.3.12

$$\sigma_u = \frac{1227.4}{19*19} = 3.4\ ksi$$

$$w_u = 3.4(19) = 64.6\ kip/in$$

$$M_u = \frac{64.6*4.17^2}{2} = 561.66\ kip \cdot in$$

Computing the thickness using supply, ϕM_p where $\phi = 0.90$

$M_p = Z_x * F_y = \frac{W_T * t_p^{\ 2}}{4} * F_y$ Eq. 2.3.13

$M_d = \phi M_p \geq M_u$ Eq. 2.3.14

Substituting Eq. 2.3.13 into Eq. 2.3.14 produces the following:

$t_p = l * \sqrt{\frac{2*P_u}{\phi F_y * B * N}}$ Eq. 2.3.15

$$t_p = 4.17 * \sqrt{\frac{2*1227.4}{0.9*50*19*19}} = 1.62"$$

Rounding to the nearest 1/4" produces a thickness of **1.75”.**

Base plate dimensions: **length = 19”; width = 19”; thickness = 1.75”**

Design Drawings

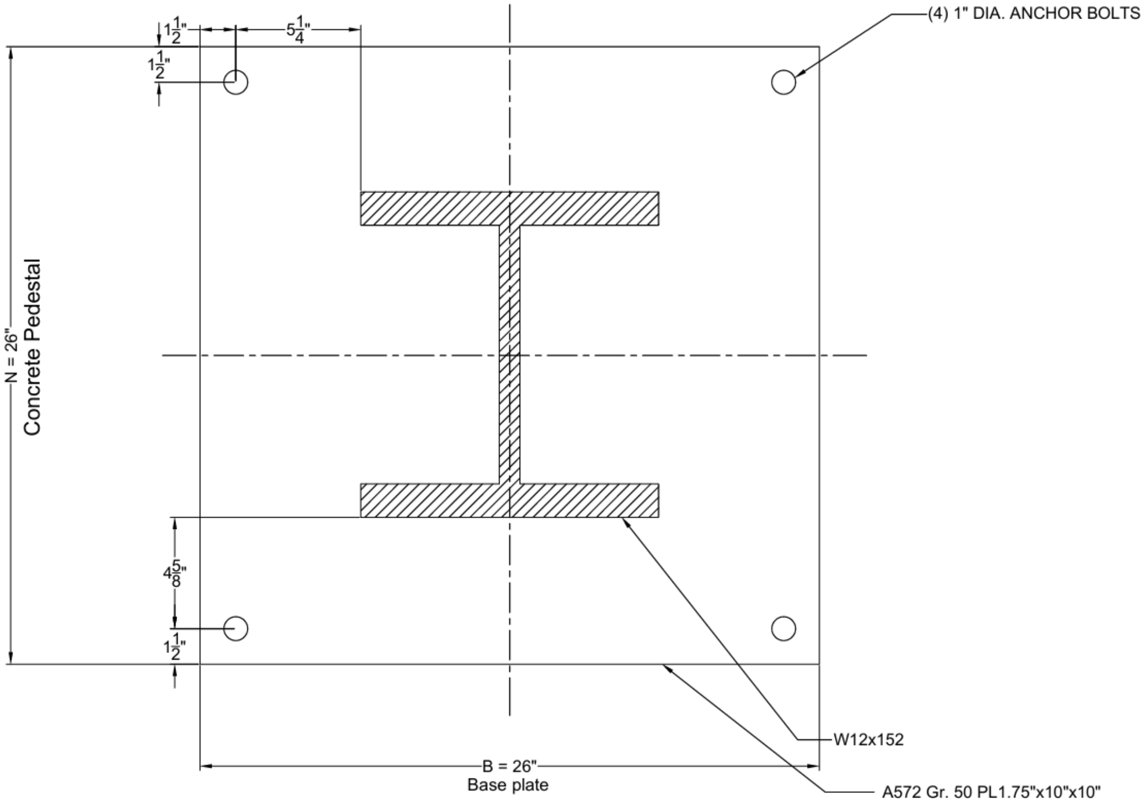


Figure 2.3.1 Column C1 Base Plate

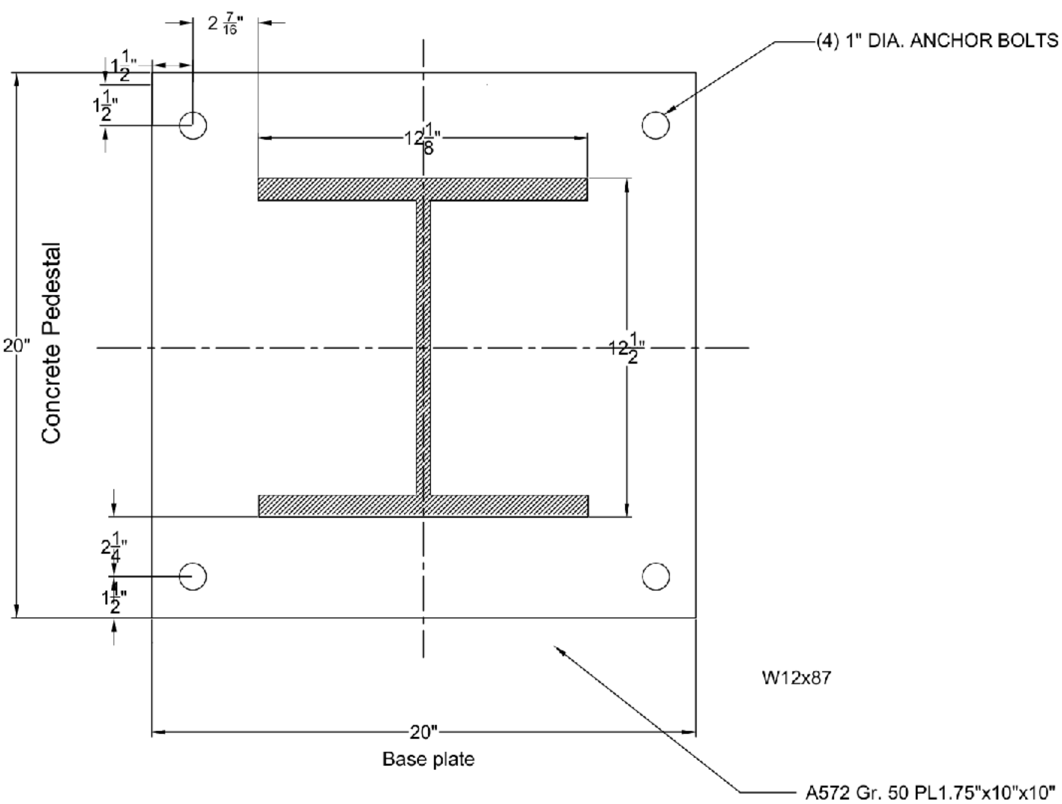


Figure 2.3.2 Column C2 Base Plate

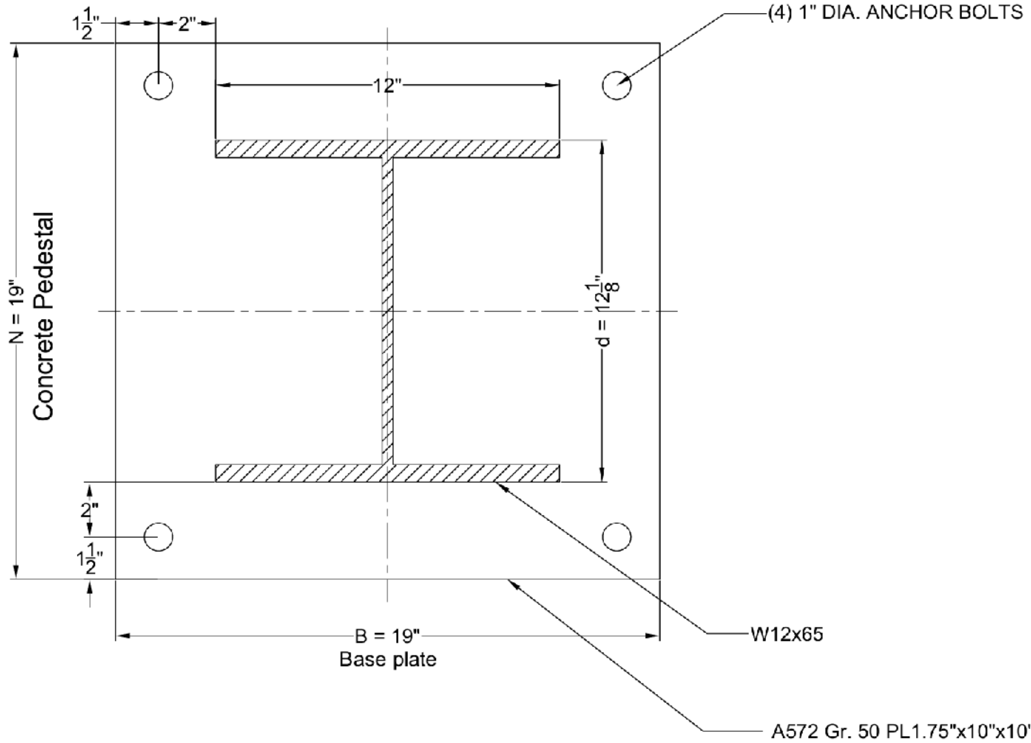


Figure 2.3.3 Column C3 Base Plate

III. Beam & Girder Design

Each beam and girder type was chosen based on what would be the most economical. W shapes that would satisfy the specific loading demands were tabulated and the most economical shape was chosen.

The cost of implementing each W shape was compared based on the sum of the cost of the steel ($5,000 per ton of steel) and the number of required studs ($5 per stud). The number of studs required were based on the spacing requirements and shear stud anchor capacity compared to the horizontal shear demand, ΣQ_n.

Table 3.0.1 - Stud Placement Requirements

	S_{stud}	S_{anchor}
Min Spacing	3"	
Max Spacing	26"	18"

Table 3.0.2 - Shear Stud Anchor Capacity

(weak studs, lightweight concrete, stud diameter = 3/4")

Deck Orientation	Stud Count / Requirement	f'c = 3 ksi
Perpendicular	1	17.1 kips
	2	14.6 kips
Parallel	Wr/hr > 1.5	17.1 kips

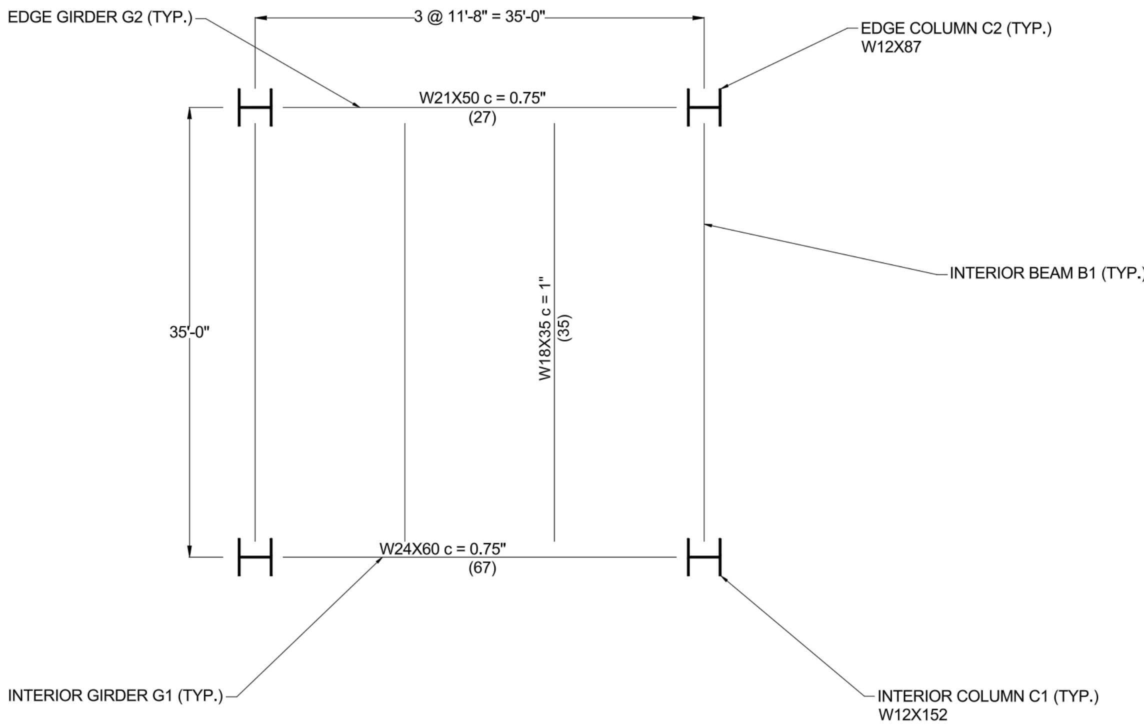


Figure 3.0.1 - Summarized Beam and Girder Plan Layout

1. *Interior Composite Floor Beam Design – B1*

The floor loads were applied to beam B1 by multiplying the relevant area loads by the tributary width of the beam, which in this case is $w_T = 35/3 = 11.67'$. For beam B1, the composite demands were assumed to govern due to the weight of the concrete. Thus, the beam was designed with the composite-stage loads and double checked later on with the construction-stage loads.

In both cases, the dead load and live load values were then separately calculated and multiplied by the applicable load case factors. After the demand load was calculated, the ultimate moment, shear, and shear were calculated. These values were then checked with the nominal design capacity values (i.e. ϕM_n, ϕV_n, $\Delta_{LL,\,limit}$) to ensure they were greater than the demand values.

Table 3.1.2 – Beam Service Loading

B1 - SERVICE		**Dead Load**		**Live Load**						
	At (ft^2)	Area DL (psf)	Line DL (plf)	KLL	Lo (psf)	Ψ	Area LL (psf)	Partition LL (psf)	Area LL (psf)	Total LL (plf)
	408.33	77.7	906.50	2	80	0.775	61.99	15	76.99	898.23

LC 1	LC 2	LC Final	M_ultimate	V_ultimate
Pu = 1.4D (klf)	Pu = 1.2D + 1.6L(klf)	Pu (klf)	Mu = wL^2 / 8 (kip-ft)	Vu = wL/2 (kip)
1.27	2.52	2.52	386.64	44.19

For the beam, the demands to be met were bending moment, shear, deflection, and checking that "a" – the assumed compression depth – was sufficient.

Table 3.1.2 – Beam Service Load Demand Checks

Trial Section	Beam Specifications					LL Deflection
	PNA	Sum_Qn	Phi_Vn	b min (in)	a	LL Deflection Limit = L/360 =
W16x40	5	237	146	105.000	0.885	0.942
	6	192	146	105.000	0.717	1.015
	7	148	146	105.000	0.553	1.120
W16x45	5	267	167	105.000	0.997	0.830
	6	217	167	105.000	0.810	0.898
	7	166	167	105.000	0.620	0.991
W18x35	5	260	159	105.000	0.971	0.875
	6	194	159	105.000	0.725	0.977
	7	129	159	105.000	0.482	1.136

Trial Section	Beam Specifications				
	PNA	a Check (a <= 2)	Mu Check	Vu Check	LL Deflection Limit < L/360
W16x40	5	Y	Y	Y	Y
	6	Y	Y	Y	Y
	7	Y	Y	Y	Y
W16x45	5	Y	Y	Y	Y
	6	Y	Y	Y	Y
	7	Y	Y	Y	Y
W18x35	5	Y	Y	Y	Y
	6	Y	Y	Y	Y

	7	Y	N	Y	Y

After a working economical beam was selected from the service phase, the beam was designed for the construction stage, including the known beam self-weight into the loads. Assuming a compression depth of $a_{trial} = 2.0"$, a sample Y2 value of 5.25 was used to calculate the nominal moments, nominal shears, and allowable live load deflections for PNA 5-7. There was also a check to ensure that for a given beam and PNA, the $a_{true} \leq (a_{trial} = 2.0")$.

Table 3.1.3 – Beam Construction Loading

B1 - Construction	**Steel Beam Properties**		**M_ultimate**	**V_ultimate**				
	Ix value (in^4)	Φb Mn (k-ft)	Mu = wL^2 / 8 (kip-ft)	Vu = wL/2 (kip)	Mu Check	Vu Check	**0.8w_DL Deflection (in)**	**Camber Specification (in)**
W16x40	518	274	176.78	20.20	Y	Y	1.031	1
W16x45	586	309	177.70	20.31	Y	Y	0.919	0.75
W18x35	510	249	175.86	20.10	Y	Y	1.038	1

Cost Estimate

Out of the W shapes that met the demand requirements, W18x35 is the most economical as the steel beam for B1.

Table 3.1.4 - Beam Cost Analysis

Trial Section	Beam Specifications	Bare Steel Section			Shear Studs				Total Cost
	PNA	Weight		Cost	SUM Qn	Required Studs (n_{req})		Cost	
		(lb/ft)	(lbs)	($)	(kips)	(half span, rounded up)	(entire beam)	($)	($)
	5	40	1400	$3,500.00	237	14	28	$140	$3,640.00
	6	40	1400	$3,500.00	192	12	24	$120	$3,620.00
W16x40	7	40	1400	$3,500.00	148	9	18	$90	$3,590.00
	5	45	1575	$3,937.50	267	16	32	$160	$4,097.50
W16x45	6	45	1575	$3,937.50	217	13	26	$130	$4,067.50

	7	45	1575	$3,937.50	166	10	20	$100	$4,037.50
	5	35	1225	$3,062.50	260	16	32	$160	$3,222.50
W18x35	**6**	**35**	**1225**	**$3,062.50**	**194**	**12**	**24**	**$120**	**$3,182.50**

Stud Placement

The beam is perpendicular to the deck so the studs could only be placed in the flutes at every 12". There are 17 flutes on one half of the beam but only 12 studs were required for half of the beam so arc spot welds were placed on the other 5 flutes.

$n_{flutes} = n_{spaces} + 1 = (\frac{35-2}{2}) + 1$ = 17.5 → 17 flutes on half of the beam (round down)

The number of studs required on one half of the beam, n_{req}, was found by dividing the shear demand, ΣQ_n, by the nominal horizontal shear strength of a single stud, Q_n 17.1 kips. For W18x35 this comes out to be 12 studs when rounded up.

$$n_{req} = \frac{\Sigma Q_n}{Q_n} = \frac{195\ kips}{17.1\ kips} = 11.4 \rightarrow 12\ studs$$

<u>W18x35</u> was chosen for the beams with 12 studs required for each half span and 5 arc spot welds filling in the other flutes. The first eight flutes closest to each end of the beam (where moment is zero) are filled with studs while the flutes in the middle alternate with studs and arc spot welds.

Design Drawing

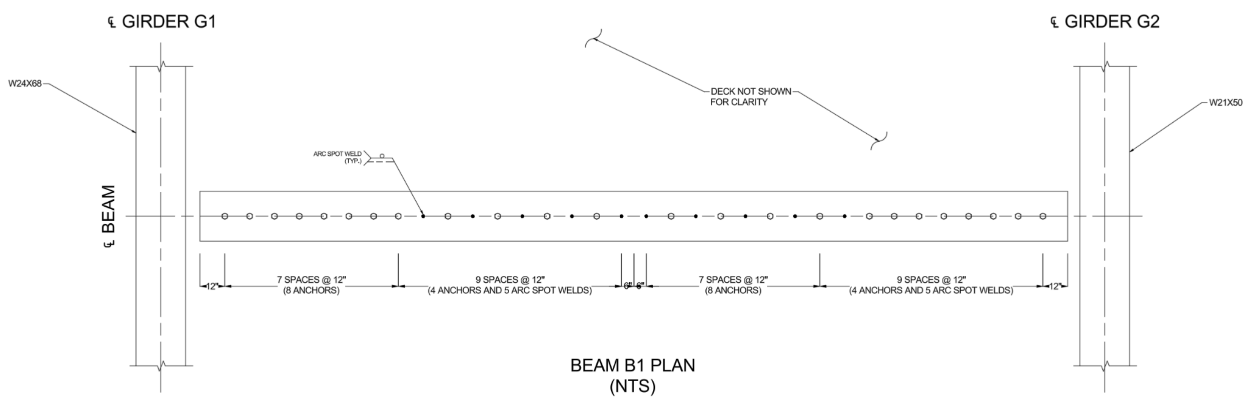


Figure 3.1.1 - Beam B1 Steel Headed Stud Anchor Layout

2. *Composite Girder 1 Design*

For Girder G1, the loads from B1 were directly through two equally spaced point loads. Each of these point loads carried double the reaction at one end of B1, due to the placement of two beams at that point. In addition, the girder's self-weight was assumed and later corrected after a working girder was designed. However, because the girder is parallel to the deck, lateral torsional buckling (LTB) was considered, which required an unbraced length of $L/3 = 35/3 = 11.67'$ and $C_b = 1$. The girder was designed assuming the construction phase governs, due to the presence of LTB, assuming there is no concrete strength contribution.

Table 3.2.1 – G1 Construction Loading

G1	Beam Loads						
	At (ft^2)	Factored B_DL (kips)	Factored B_LL (kips)	KLL	Ψ	Reduced B_LL (kips)	Total Beam Load P (kips)
Construction	1225	23.86	16.33	2	0.553	9.033	32.90

Girder Loads			
Girder SW Loads	**LC 1**	**LC 2**	**LC Final**
Self-Weight DL (klf)	Pu = 1.4D (klf)	Pu = 1.2D + 1.6L(klf)	Pu (klf)
0.07	0.098	0.08	0.098

M_beams	**M_girder SW**	**M□ot**	**V_beams**	**V_girder SW**	**V_ultimate**
Mu = Pa (k-ft)	Mu = wL^2 / 8 (k-ft)	Mu (k-ft)	V = P (kips)	V = wL/2 (kips)	Vu = wL/2 (kips)
383.79	15.01	398.79	32.90	1.715	34.61

Table 3.2.2 – G1 Lateral Torsional Buckling (LTB) Check

LTB Calculator	Cb	Lb	Lp:	Lr:	Mp (k-ft)	Mp (k-in)	Fy (ksi)	Sx (in3)
W21x111	1	11.67	10.2	31.2	1166.67	14000.00	50	249
W24x84	1	11.67	6.89	20.3	933.33	11200.00	50	196
W24x76	1	11.67	6.78	19.5	833.33	10000.00	50	176
W24x68	1	11.67	6.61	18.9	737.78	8853.33	50	154
W21x68	1	11.67	6.36	18.7	666.67	8000.00	50	140

LTB Calculator	LTB Mn (k-ft)	Mn (k-ft) = LTB <=MP	Φb Mn (k-ft)	ΦVn (k-ft)	Φb Mn > M_{ult}	ΦVn > V_{ult}
W21x111	1135.91	1135.91	1022.32	355	Y	Y
W24x84	804.51	804.51	724.06	340	Y	Y
W24x76	710.40	710.40	639.36	315	Y	Y
W24x68	619.03	619.03	557.13	295	Y	Y
W21x68	555.57	555.57	500.02	272	Y	Y

Table 3.2.3 – G1 Trial Selection

Beam Selection	**Ix (in4)**	**0.8*Pd (kips)**	**0.8*w□w (klf)**	**Δ_DL, point**	**Δ_DL, SW**	**Δ_DL, total (in)**	**Camber Specification (in)**
W21x68	1480.00	15.91	0.0544	0.975	0.043	1.018	1
W24x68	1830.00	15.91	0.0544	0.788	0.035	0.823	0.75
W24x76	2100.00	15.91	0.0608	0.687	0.034	0.721	0.5

After a working economical girder was selected from the construction phase, it was designed for the service stage, including the known girder self-weight into the loads. Assuming a compression depth of $a_{trial} = 2.0"$, a sample Y2 value of 5.25 was used to calculate the nominal moments, nominal shears, and allowable live load deflections for PNA 5-7. There was also a check to ensure that for a given beam and PNA, the $a_{true} \leq (a_{trial} = 2.0")$.

Table 3.2.4 – G1 Service Loading

G1	**Beam Loads**						
	At (ft^2)	Factored B_DL (kips)	Factored B_LL (kips)	KLL	Ψ	Reduced B_LL (kips)	Total Beam Load P (kips)
Service	1225	38.07	50.30	2	0.553	27.819	65.89

Girder Loads			
Self-Weight	**LC 1**	**LC 2**	**LC Final**
Self-Weight DL (klf)	Pu = 1.4D (klf)	Pu = 1.2D + 1.6L(klf)	Pu (klf)
0.068	0.0952	0.08	0.10

M_beams	**M_girder SW**	**M□ot**	**V_beams**	**V_girder SW**	**V_ultimate**

Mu = Pa (k-ft)	Mu = wL^2 / 8 (k-ft)	Mu (k-ft)	V = P (kips)	V = wL/2 (kips)	Vu = wL/2 (kips)
768.74	14.58	783.31	2.00	1.666	3.67

Table 3.2.5 – Girder 1 Service Load Demand Check

Trial Section	Beam Specifications					
	PNA	Sum_Qn	Phi_Vn	b min (in)	a	LL Deflection (in)
	5	434	272	105.000	1.621	0.527
	6	342	272	105.000	1.277	0.573
W21X68	7	250	272	105.000	0.934	0.636
	5	480	295	105.000	1.793	0.423
	6	366	295	105.000	1.367	0.464
W24x68	7	251	295	105.000	0.937	0.525
	5	509	315	105.000	1.901	0.378
	6	394	315	105.000	1.472	0.412
W24x76	7	280	315	105.000	1.046	0.460

Trial Section	Beam Specifications				
	PNA	a Check (a <= 2)	Mu Check	Vu Check	LL Deflection Limit < L/360
	5	Y	Y	Y	Y
	6	Y	Y	Y	Y
W21X68	7	Y	Y	Y	Y
	5	Y	Y	Y	Y
	6	Y	Y	Y	Y
W24x68	7	Y	Y	Y	Y
	5	Y	Y	Y	Y
	6	Y	Y	Y	Y
W24x76	7	Y	Y	Y	Y

Cost Estimate

After running all the relevant design checks, W24x68 was determined to be the most economical steel girder for G1.

Table 3.2.6 – Girder 1 Cost Analysis

Trial Section	Beam Specifications	Bare Steel Section			Shear Studs				Total Cost
	PNA	Weight		Cost	SUM Qn	Required Studs (n_{req})		Cost	
		(lb/ft)	(lbs)	($)	(kips)	(half span, rounded up)	(entire beam)	($)	($)
W24x68	**5**	**68**	**2380**	**$5,950.00**	**480**	**29**	**58**	**$290**	**$6,240.00**
W24x76	5	76	2660	$6,650.00	509	30	60	$300	$6,950.00
	6	76	2660	$6,650.00	394	24	48	$240	$6,890.00

Stud Placement

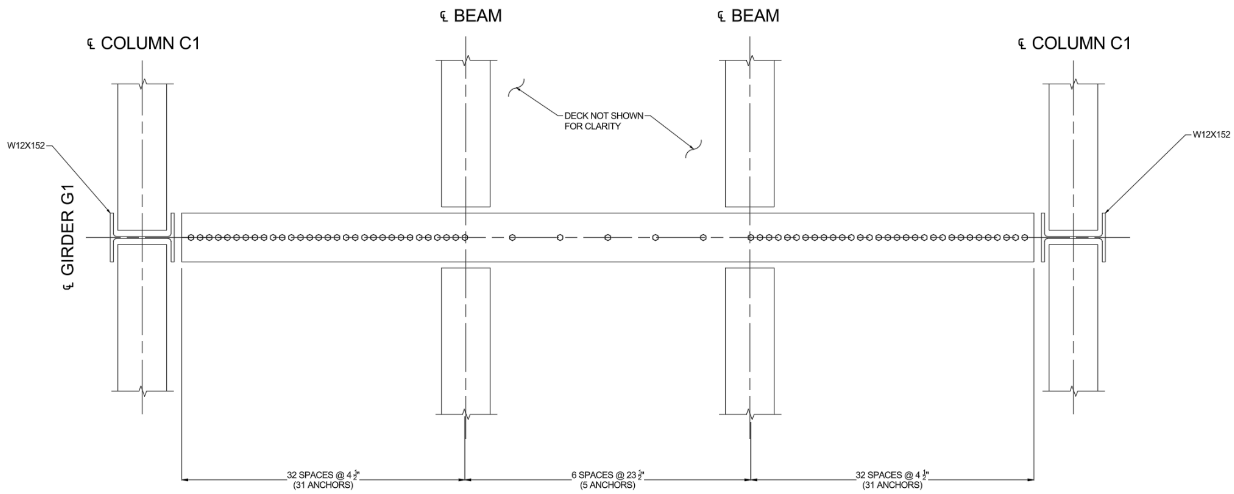
The girder is parallel to the deck so the studs can be placed at any point along the girder. Due to the two beam point loads, the end third of each end of the girder holds most of the shear while the middle third carries very little.

Outer Studs: The studs on the end thirds are designed to carry all the shear demand on half of the girder, ΣQ_n. The number of required studs, n_{req}, is spaced out in the end thirds with the spacing found by dividing the number of studs over the length.

$$stud\ spacing = \frac{L/3}{n_{req}} = \frac{35/3}{24} = 4.83'' \rightarrow 4\,\frac{1}{2}''\ \text{(rounded down)}$$

Inner Studs: There is very little shear in between the two beams so the stud spacing just has to satisfy the maximum spacing requirement, 8 times the diameter of the stud, which is 26”. The spacing for the middle studs is 5 studs 23 $\frac{1}{2}$” apart.

Design Drawing



GIRDER G1 PLAN
(NTS)

Figure 3.2.1 - Girder G1 Steel Headed Stud Anchor Layout

3. Composite Girder 2 Design

For Girder G2, the loads from B1 were directly through two equally spaced point loads. Each of these point loads were equal to the reaction at the end of B1, since G2 only has two beams placed onto it. The girder's self-weight was assumed to be 100 plf and later corrected after a working girder was designed. However, because the girder is parallel to the deck, lateral torsional buckling (LTB) was considered, which required an unbraced length of $L/3 = 35/3 = 11.67'$ and $C_b = 1$. The girder was designed assuming the construction phase governs, due to the presence of LTB, assuming there is no concrete strength contribution.

Table 3.3.1 – G2 Construction Loading

G2	Beam Loads						
	At (ft^2)	Factored B_DL (kips)	Factored B_LL (kips)	KLL	Ψ	Reduced B_LL (kips)	Total Beam Load P (kips)
Construction	647.5	12.61	8.63	2	0.667	5.757	18.37

Girder Loads			
Girder SW Loads	**LC 1**	**LC 2**	**LC Final**
Self-Weight DL (klf)	Pu = 1.4D (klf)	Pu = 1.2D + 1.6L(klf)	Pu (klf)
0.1	0.14	0.12	0.14

M_beams	**M_girder SW**	**M□ot**	**V_beams**	**V_girder SW**	**V_ultimate**
Mu = Pa (k-ft)	Mu = wL^2 / 8 (k-ft)	Mu (k-ft)	V = P (kips)	V = wL/2 (kips)	Vu = wL/2 (kips)
214.32	5.99	220.31	18.37	1.295	19.67

Table 3.3.2 – G2 Lateral Torsional Buckling (LTB) Check

LTB Calculator	Cb	Lb	Lp:	Lr:	Mp (k-ft)	Mp (k-in)	Fy (ksi)	Sx (in3)
W21x48	1	11.67	6.09	16.5	442.22	5306.67	50	93
W21x50	1	11.67	4.59	13.6	458.89	5506.67	50	94.5
W18X55	1	11.67	5.9	17.6	466.67	5600.00	50	98.3

LTB Calculator	LTB Mn (k-ft)	Mn (k-ft) = LTB <=MP	Φb Mn (k-ft)	ΦVn (k-ft)	Φb Mn > M_{ult}	ΦVn > V_{ult}
W21x48	350.63	350.63	315.57	216	Y	Y
W21x50	314.95	314.95	283.45	237	Y	Y
W18X55	377.97	377.97	340.17	212	Y	Y

Table 3.3.2 – G2 Trial Selection

Beam Selection	Ix (in4)	0.8*Pd (kips)	0.8*w□w (klf)	Δ_DL, point	Δ_DL, SW	Δ_DL, total (in)	Camber Specification (in)
W21x48	959.00	8.41	0.0384	0.795	0.004	0.799	0.75
W21x50	984.00	8.41	0.04	0.775	0.004	0.779	**0.75**
W18X55	890.00	8.41	0.044	0.857	0.004	0.861	0.75

After a safe girder was selected from the construction phase, it was designed for the service stage, using the known girder self-weight. In addition, the curtain wall loads were included into the calculations, significantly increasing the distributed loads on the girder. Assuming a compression depth of $a_{trial} = 4.0"$, a sample Y2 value of 4.25 was used to calculate the nominal moments, nominal shears, and allowable live load deflections for PNA 5-7. There was also a check to ensure that for a given beam and PNA, the $a_{true} \leq (a_{trial} = 4.0")$.

Table 3.3.3 – G1 Service Loading

G2		Beam Loads					
	At (ft^2)	Factored B_DL (kips)	Factored B_LL (kips)	KLL	Ψ	Reduced B_LL (kips)	Total Beam Load P (kips)
Service	647.5	20.12	26.59	2	0.667	17.729	37.85

Girder Loads					
Girder Self-Weight	**Curtain Wall Load**	**Total Line Load**	**LC 1**	**LC 2**	**LC Final**
Self-Weight DL (klf)	CW = Story Height * PSF	DL (klf)	Pu = 1.4D (klf)	Pu = 1.2D + 1.6L(klf)	Pu (klf)
0.1	0.195	0.295	0.413	0.12	0.41

M_beams	M_girder	M□ot	V_beams	V_girder SW	V_ultimate
Mu = Pa (k-ft)	Mu = wL^2 / 8 (k-ft)	Mu (k-ft)	V = P (kips)	V = wL/2 (kips)	Vu = wL/2 (kips)
441.63	17.67	459.29	37.85	3.82025	41.67

Table 3.3.4 – Girder 2 Service Load Demand Check

Trial Section	Beam Specifications					
	PNA	Sum_Qn	Phi_Vn (kip)	b min (in)	a (in)	**LL Deflection (in)**
	5	355	216	24.000	5.801	0.076
	6	266	216	24.000	4.346	0.083
W21x48	**7**	176	216	24.000	2.876	0.095
	5	386	237	24.000	6.307	0.072
	6	285	237	24.000	4.657	0.079
W21x50	**7**	184	237	24.000	3.007	0.092
	5	336	212	24.000	5.490	0.086
	6	269	212	24.000	4.395	0.092
W18X55	7	203	212	24.000	3.317	0.101

Trial Section	Beam Specifications				
	PNA	a Check (a <= 4")	Mu Check	Vu Check	LL Deflection Limit < L/360
	5	N	Y	Y	Y
	6	N	Y	Y	Y
W21x48	**7**	Y	Y	Y	Y
	5	N	Y	Y	Y
	6	N	Y	Y	Y
W21x50	**7**	Y	Y	Y	Y
	5	N	Y	Y	Y
	6	N	Y	Y	Y
W18X55	7	Y	Y	Y	Y

Cost Estimate

After running all the relevant design checks, W21x50 was determined to be the best steel girder for G2.

Table 3.3.5 – Girder 2 Cost Analysis

Trial Section	Beam Specifications	Bare Steel Section			Shear Studs				Total Cost
	PNA	Weight		Cost	SUM Qn	Required Studs		Cost	
		(lb/ft)	(lbs)	($)	(kips)	(half span, rounded up)	(entire beam)	($)	($)
W21x50	7	50	1750	$4,375.00	184	11	22	$110	$4,485.00
W18x55	7	55	1925	$4,812.50	203	12	24	$120	$4,932.50

Stud Placement

The girder is parallel to the deck so the studs can be placed at any point along the girder. Due to the two beam point loads, the end third of each end of the girder holds most of the shear while the middle third carries very little. The PNA value is 7 so the process of finding stud spacing remains the same as girder 1.

Outer Studs: The studs on the end thirds are designed to carry all the shear demand on half of the girder, ΣQ_n. The number of required studs, n_{req}, is spaced out in the end thirds with the spacing found by dividing the number of studs over the length.

$$stud\ spacing = \frac{L/3}{n_{req}} = \frac{35/3}{11} = 12.7" \rightarrow 12\,\frac{1}{2}" \text{ (rounded down)}$$

Inner Studs: There is very little shear in between the two beams so the stud spacing just has to satisfy the maximum spacing requirement, 8 times the diameter of the stud, which is 26”. When divided evenly, there is space for five studs but the distance is difficult to measure during construction as it comes out to be $24\frac{1}{6}$" so the spacing for the middle three studs is $24\frac{1}{2}$" and the other two are 24” apart.

Design Drawing

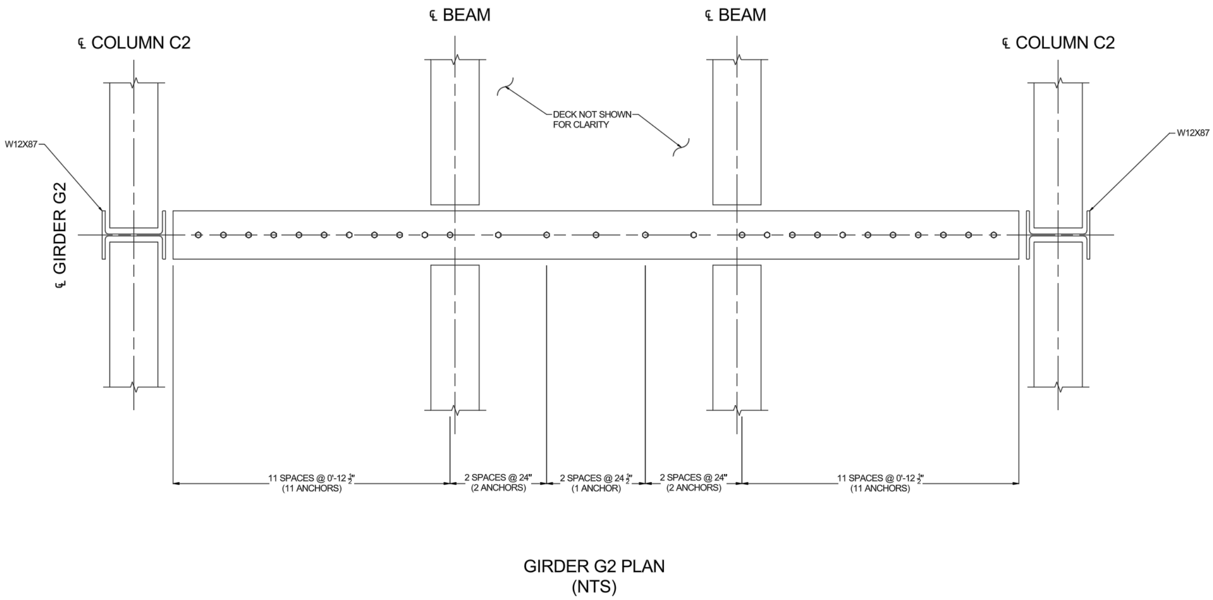


Figure 3.3.1 - Girder G2 Steel Headed Stud Anchor Layout

IV. Connection Design

To ensure the steel framing system fits together and has stable and strong joints, connections needed to be designed for the beams, girders, and columns. The two main connections that were designed were the girder-to-column and beam-to-girder connections.

1. Single Plate Beam-to-Column Flange Connection

The design of the girder-to-column connection was governed by Chapter 10 in the AISC Steel Construction Manual. First, the shear strength of the single plate was checked to ensure the plate is strong enough to withstand the shear force. Next, the minimum and maximum weld size was determined. Lastly, the shear strength of the bolts were checked. The steps of the design process are shown below.

Single Plate Shear Strength Check

Table 4.1.1 – Design Values of the Single Plate

Bolt Rows (n_b)	Bolt Diameter (in)	Thickness (in)	L (in)	L_{ev} (in)	L_{eh} (in)
4	3/4	5/16	11.5	1.25	1.5

Given the values in Table 4.1.1, an available strength of **73.1 kips** was chosen from Table 10-10a in the AISC Manual. A shear demand of **67.56 kips** from girder G1 from Project 1B was chosen. The DCR was calculated as follows:

$$DCR = \frac{R_u}{\phi R_n} = \frac{67.56}{73.1} = 0.92 \leq 1 \rightarrow \text{OK!} \qquad \text{Eq. 4.1.1}$$

Weld Size Check

To determine the minimum weld size, Table J2.4 in the AISCS was used. Given the fact that the material thickness of the thinner part joined is 5/16 in, the minimum size comes out to be **3/16 in.** To determine the maximum weld size, Section J.2b.b.2 was used. Given the fact that the material thickness is greater than 1/4 in, the maximum weld size is not to be greater than the thickness of the material minus 1/16 in, which is **1/4 in.** From Table 10-10b, the selected weld size is **1/4 in.**

Bolt Effective Strength Check

The effective strength of each individual bolt was to be checked as well. To do so, the following needed to be determined:

Eff. Strength (per bolt) = min{bolt shear, plate bearing/tearout, beam web bearing/tearout}

To find the bolt shear strength, Table 10-10b in AISC Chapter 10 was used. Given Group 120, thread (X), the LRFD strength came out to be 22.5 kips. To determine bearing and tearout strength for the single plate, the first three rows of bolts were considered a non-edge bolt and the last row was considered an edge bolt. Using $l_{ev} = 1.25\ in$ and spacing $s = 3\ in$, the non-edge and edge bolt strengths were given as 87.8 kips and 49.4 kips respectively. To determine bearing and tearout for the beam web, all bolts were considered non-edge. These strength values are given *per inch of thickness*. Determining the actual strength of the bolts is as follows:

Non-edge plate bolt: $\phi R_n = (strength)(t_p) = 87.8 * 0.3125 = 27.44\ kips$ Eq. 4.1.2

Edge plate bolt: $\phi R_n = (strength)(t_p) = 49.4 * 0.3125 = 15.44\ kips$ Eq. 4.1.3

Non-edge web bolt: $\phi R_n = (strength)(t_w) = 87.8 * 0.415 = 36.44 kips$ Eq. 4.1.4

Having determined the strength values for the three cases, the individual strength for the bolts is shown in Table 4.1.2:

Table 4.1.2 – Effective Strength of the Individual Bolt (kips)

Bolt 1	Bolt 2	Bolt 3	Bolt 4
22.5	22.5	22.5	15.44

The strength of the bolts was then summed up to 82.94 kips and multiplied by an eccentricity reduction factor found in Table 10-10b to get the bolt group effective strength. Because 4 rows were used, the factor came out to be 0.885. After multiplying by the factor, the effective strength was divided by the shear demand R_u to find the demand capacity ratio. This process is calculated below:

$$(factor)(\Sigma\phi R_n) = 0.885 * 82.94 = 73.4\, kips \qquad \text{Eq. 4.1.5}$$

$$DCR = \frac{67.56}{73.4} = 0.92 \leq 1 \rightarrow \text{OK!} \qquad \text{Eq. 4.1.1}$$

Design Drawing

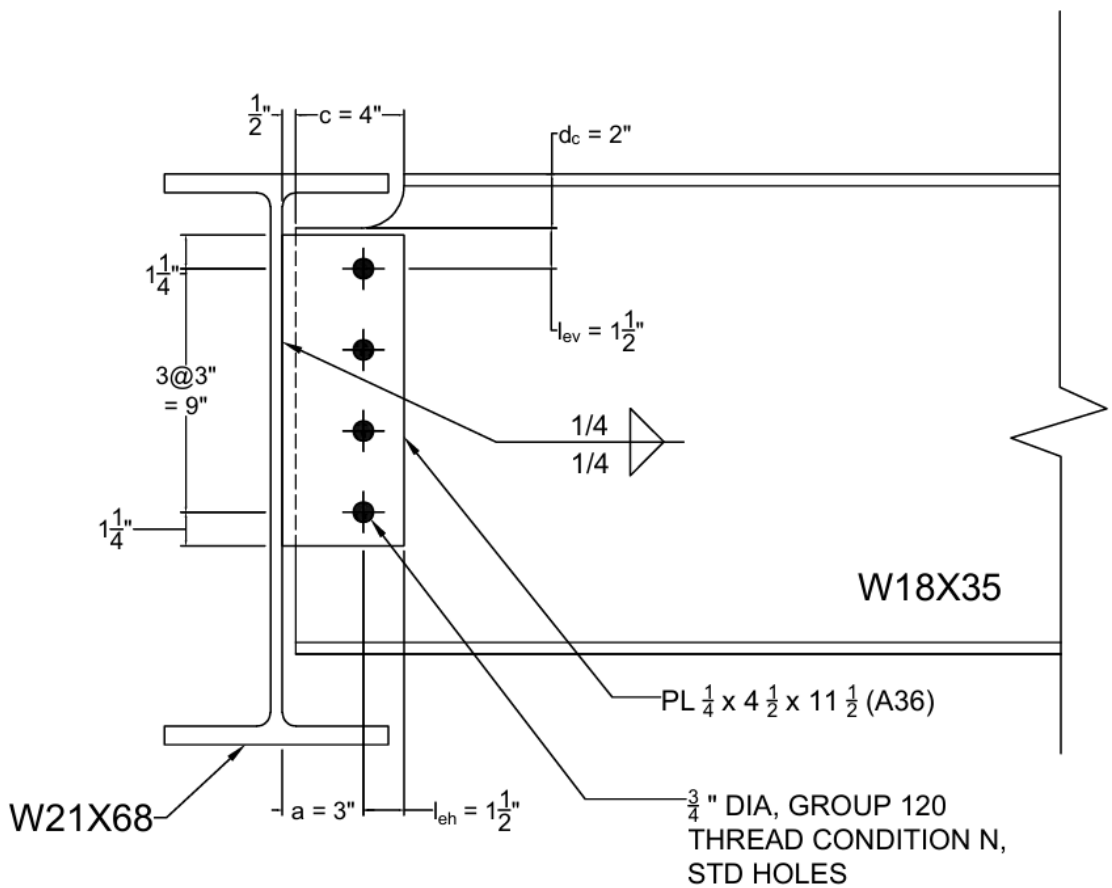


Figure 4.1 Beam to Girder Design (B1 to G1 on third floor)

2. *Single Plate Beam-to-Girder Web Connection*

The design of the beam-to-girder connection was performed using chapters 9 and 10 of the AISC Steel Construction Manual. First, the shear strength of the single plate was checked to ensure the plate is strong enough to withstand the shear force. Next, the minimum and maximum weld size was determined. The shear strength of the bolts were checked. In addition to these, the shear and flexural strengths of the beam were checked, given the "coped" condition. Furthermore, the strength of the girder web was checked to ensure no rupture at the weld. The steps of the design process are shown below.

Table 4.2.1 – Design Values of the Single Plate

Design Values					
Bolt Rows (nb)	Bolt Diameter (in)	t□ (in)	L (in)	Lev (in)	Leh (in)
4	3/4	0.25	11.5	1.25	1.5

Single Plate Shear Strength Check

Table 4.2.2 – Design Values of the Single Plate

Shear Demand Ru (kip)	Shear Capacity ΦRn (kips)	**DCR**
44.19	58.5	**0.76**

Given plate thickness $t_p = 1/4"$, bolt diameter $d_b = 3/4"$, and number of bolt rows n set to 4, the shear capacity of the plate was determined to be $\phi R_n = 58.5\ kips$, which is greater than the shear demand $R_u = 44.19\ kips$.

$$(\phi R_n = 58.5\ kips) > (R_u = 44.19\ kips) \qquad \text{Eq. 4.2.1}$$

$$DCR = \frac{R_u}{\phi R_n} = \frac{44.19}{58.5} = 0.76 \leq \quad \rightarrow \text{OK!} \qquad \text{Eq. 4.2.2}$$

Weld Size Check

To determine the minimum weld size, Table J2.4 in the AISCS was used. The thickness of the plate is $t_p = 0.25"$, while the thickness of the web of the beam is $t_w = 0.30"$. Thus, the material thickness of the thinner part joined is 0.25". Thus, the minimum size comes out to be **1/16 in.** To determine the maximum weld size, Section J.2b.b.2 was used. The maximum weld size is not to be greater than the thickness of the material minus 1/16 in. Given the plate thickness of $t_p = 1/4"$, the maximum weld size is **3/16.**

Bolt Effective Strength Check

The effective strength of each individual bolt was to be checked as well. To do so, the following needed to be determined:

Eff. Strength (per bolt) = min{bolt shear, plate bearing/tearout, beam web bearing/tearout}

To find the bolt shear strength, Table 10-10b in AISC Chapter 10 was used. Given Group 120, thread (N), the LRFD strength came out to be 17.9 kips. To determine bearing and tearout strength for the single plate, the first three rows of bolts were considered non-edge bolt and the last row was considered an edge bolt. Using $l_{ev} = 1.25\ in$ and spacing $s = 3\ in$, the non-edge and edge bolt strengths were given as 87.8 kips and 49.4 kips, respectively. To determine bearing and tearout for the beam web, the top bolt was considered an edge bolt, given the removal of the strong top flange, while the bottom three bolts were considered non-edge bolts. These strength values are given *per inch of thickness*. Determining the actual strength of the bolts is as follows:

Table 4.2.3 – Effective Strength of the Anchor Bolts

Bolt No	Dia. (in)	Bolt Shear	Plate (tp = 0.25 in.)		Beam Web (tw = 0.30 in.)		**Individual Bolt Effective Strength (kips)**
		Group 120 (N)	Edge/Non-Edge	Φrn (kips) = (Bolt strength)(t□)	Edge/Non-Edge	Φrn (kips) = (Bolt Strength)(t_w)	ΦRn_final = min(Φrn_bolt, Φrn_plate, Φrn_web)
1	3/4	17.9	Non-Edge	21.95	Edge	14.82	14.82
2		17.9	Non-Edge	21.95	Non-Edge	26.34	17.9
3		17.9	Non-Edge	21.95	Non-Edge	26.34	17.9
4		17.9	Edge	12.35	Non-Edge	26.34	12.35

						Sum	62.97
						Ecc. Reduced (0.885)	**55.73**
						DCR Ratio	**0.793**

Non-edge plate bolt: $\phi R_n = (strength)(t_p) = 87.8 * 0.25 = 21.95\ kips$ Eq. 4.2.3

Edge plate bolt: $\phi R_n = (strength)(t_p) = 49.4 * 0.25 = 12.35\ kips$ Eq. 4.2.4

Non-edge web bolt: $\phi R_n = (strength)(t_w) = 87.8 * 0.30 = 26.34\ kips$ Eq. 4.2.5

Edge web bolt: $\phi R_n = (strength)(t_w) = 49.4 * 0.30 = 14.82\ kips$ Eq. 4.2.6

The minimum strength ϕR_n values were found for the individual bolts then summed together, resulting in a total nominal bolt strength value of $\phi R_n = 62.97$. However, to account for the bolt group phenomenon, an eccentricity reduction factor of 0.885 (found in Table 10-10b) was multiplied to the summed nominal bolt strength, resulting in a final nominal bolt strength value of $\phi R_n = 55.73$.

Given plate thickness $t_p = 1/4"$, bolt diameter $d_b = 3/4"$, and number of bolt rows n set to 4, the shear capacity of the plate was determined to be $\phi R_n = 55.73\ kips$, which is greater than the shear demand $R_u = 44.19\ kips$.

$$(\phi R_n = 55.73\ kips) > (R_u = 44.19\ kips) \quad \text{Eq. 4.2.7}$$

$$DCR = \frac{R_u}{\phi R_n} = \frac{44.19}{55.73} = 0.793 \leq 1 \rightarrow \text{OK!} \quad \text{Eq. 4.2.2}$$

Coped Beam Web Shear Strength Check

Table 4.2.4 – Design Values of the Coped Beam Web

Bolt	Bolt	Beam	F_y (ksi)	F_u (ksi)	L (in)	$L_{ev,t}$ (in)	L_{eh} (in)

Rows (n_b)	Diameter (in)	Web Thicknes s (in)					
4	3/4	0.30	50	65	11.5	1.25	2.25 in

To check the shear strength of the coped beam web, Table 10-10c of the AISC Steel Design Manual was utilized. Given the parameters listed above, the shear values for the top edge hole, center hole, and block shear rupture were obtained from Table 10-10c.

The individual shear values were found to be $\phi R_{n,\,top} = 23.8\ kips$, $\phi R_{n,\,center} = 62.2\ kips$, and $\phi R_{n,\,rupture} = 76.2\ kips$ respectively. The values were combined, as shown below, then multiplied by the beam web thickness t_w to attain the final ϕR_n value.

$$\phi R_n = (\phi R_{n,\,top} + (n_b - 1)\phi R_{n,\,bottom} + \phi R_{n,\,rupture}) * t_w \qquad \text{Eq. 4.2.8}$$

$$= (23.8 + 3 * 62.2 + 76.2)(0.3)$$

$$= 85.98\ kips$$

Table 4.2.5 – DCR Check for Coped Beam Web Shear Strength

Top Edge hole Shear Rupture: L_{ev}, t = 1.25" (kips)	Center Hole Shear Rupture (3in) x3 (kips)	Block shear rupture of L_{e}□ (kips)	ΦRn	DCR CHECK
23.8	186.6	76.2	85.98	0.514

Thus, the final nominal web shear strength is greater than the demand shear, which means the design is acceptable.

$$(\phi R_n = 85.98\ kips) > (R_u = 44.19\ kips)$$

$$DCR = \frac{R_u}{\phi R_n} = \frac{44.19}{85.98} = 0.514 \leq 1 \rightarrow \text{OK!} \qquad \text{Eq. 4.2.2}$$

Coped Beam Flexural Strength Check

Since the beam is coped, the beam section must be checked for lateral torsional buckling (LTB) or local buckling, which significantly weakens the flexural strength. The LTB calculations were performed assuming an Inverted-T beam section, given the coped section of the beam.

The moment demand at the fillet of the coped section was found using

$$M_u = R_u * e = (44.19\ kips)(4.5") = 198.9\ kip\ in \quad \text{Eq. 4.2.9}$$

To determine the supply nominal moment, it was necessary to use the web slenderness ratio

$$\lambda = h_c/t_w = (17.7 - 2)/0.3 = 52.33 \quad \text{Eq. 4.2.10}$$

Before calculating the limit states and the factors used within, the factors of the factors must be calculated, namely the f and k values to determine the k_1 value. The f and k values can be found using specific equations, which depend on the c/h_c and c/d ratios respectively. Given that

$$c/h_c = 4/(17.7 - 2) = 0.25477 < 1, \quad \text{Eq. 4.2.11}$$

$$k = 2.2(h_c/c)^{1.65} = 2.2(1/0.25477)^{1.65} = 21 \quad \text{Eq. 4.2.12}$$

Given that $c/d = 4/17.7 = 0.226 < 1,$ Eq. 4.2.13

$$f = 2(c/d) = 2(0.226) = 0.452 \quad \text{Eq. 4.2.14}$$

Next, the k_1 value can be found using

$$k_1 = (fk = 0.452 * 21) = \mathbf{9.49} \geq 1.61 \quad \text{Eq. 4.2.15}$$

Now, the relevant limit states for the beam can be found.

$$\lambda_p = 0.475\sqrt{\frac{k_1 E}{F_y}} = 0.475\sqrt{\frac{(9.49)(29000)}{50}} = 25.24 \quad \text{Eq. 4.2.16}$$

$$2\lambda_p = (2)0.475\sqrt{\frac{k_1 E}{F_y}} = 0.475\sqrt{\frac{(9.49)(29000)}{50}} = 70.48 \quad \text{Eq. 4.2.17}$$

For the nominal moment calculation, the $Z_{net} = 32.1$ and $S_{net} = 18.2$ were obtained from Table 9-2b (p.9-39) and Table 9-2a (p.9-30) respectively.

The slenderness ratio for this beam $\lambda = 52.33$ falls right in between the two limit state values $\lambda_p = 35.24$ and $2\lambda_p = 70.48$. This means the relevant equation for section is

$$M_n = M_p - (M_p - M_y)(\frac{\lambda}{\lambda_p} - 1) = \quad \text{Eq. 4.2.18}$$

$$= F_y Z_{net} - (F_y Z_{net} - F_y S_{net})(\frac{\lambda}{\lambda_p} - 1)$$

$$= (50)(32.1) - (50 * 32.1 - 50 * 18.2)(\frac{52.33}{35.24} - 1)$$

= 1267.9 k-ft

$\phi M_n = 0.9 * M_n = 0.9(1267.9)$ **= 1141 k-ft** Eq. 4.2.19

Thus, the final nominal moment is greater than the demand moment, which means the design is acceptable for the flexural strength check.

$(\phi M_n = 1141\ k\ in) > (M_u = 198.9\ k\ in)$ Eq. 4.2.20

$DCR = \frac{M_u}{\phi M_n} = \frac{198.9}{1141} = 0.17 \leq 1 \rightarrow$ OK! Eq. 4.2.2

Girder Web Rupture at Weld Check

Finaly, although a weld strength check was already performed for the single plate, it is necessary to check the thickness of the girder web. The rupture strength at welds for connecting welds can be found using the equation below, assuming a weld size of 3/16".

$t_{w,girder} \geq t_{min} = \frac{6.19D}{F_u} = \frac{6.19(3/16*16)}{65} = 0.28$ Eq. 4.2.21

Girder G1 (W21x68) meets this thickness requirement, with a web thickness of $t_{w,girder} = 0.43$.

$DCR = \frac{0.28}{0.43} = 0.65 \leq 1 \rightarrow$ OK! Eq. 4.2.22

Design Drawing

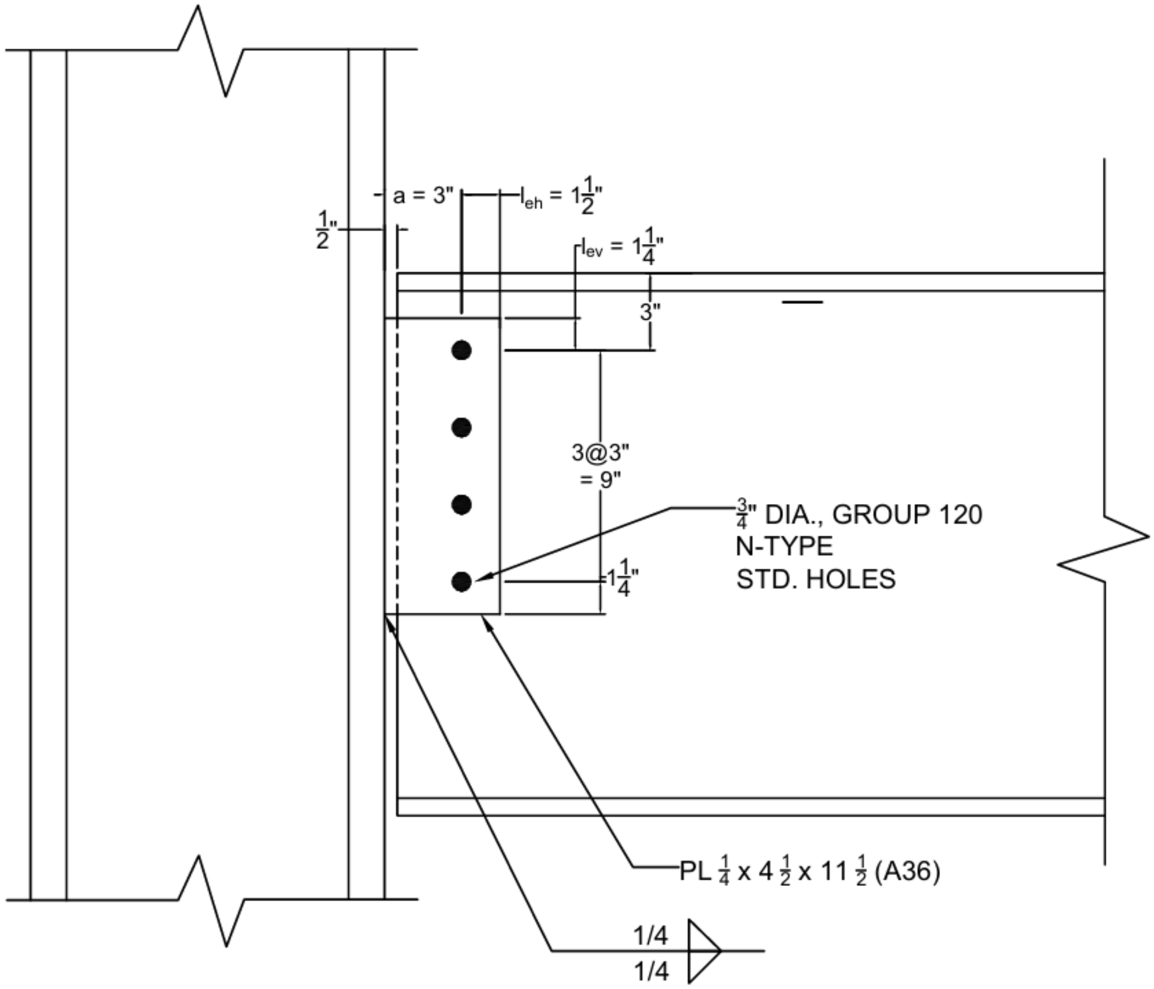


Figure 4.2 Beam to Column Design (G1 to C1 on third floor)