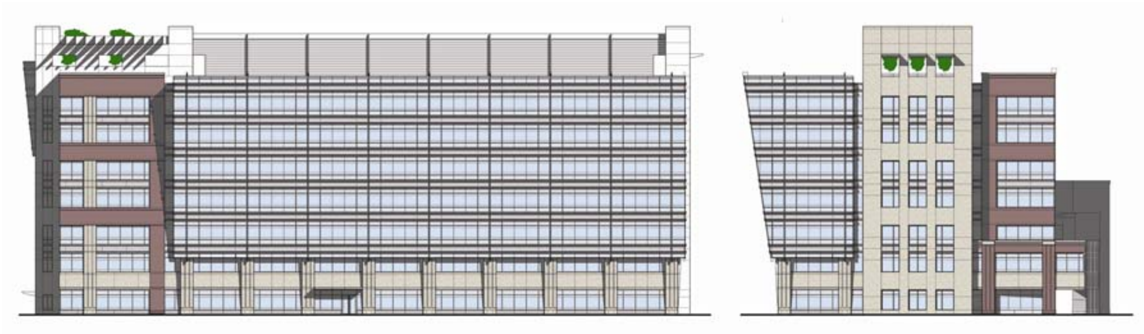


Executive Summary

This report is a description, analysis, and comparison of the existing and four alternative floor systems. The proposed floor system for Parkridge Center – Phase VI is a composite steel system. Using manufacturer design tables, the CRSI handbook, the AISC Manual of Steel Construction 13th Edition, RAM Structural system, and other design aids I have analyzed and found preliminary sizes for the following floor systems:

- Post-Tension 2-Way Flat Plate Slab
- Pre-Cast Hollow Core Plank
- Open Web Steel Joists with form deck
- Non-Composite Steel with form deck

Each system was compared against overall depth, weight, constructability, and impact on the existing foundation. From the initial analysis I found that the existing system is the most economical for the typical bay spans. Other viable options that would require more study are a Post-Tension and open web steel joist system. The post-tension systems may provide additional benefits in resisting the floor tension caused by the sloping columns on the south face. The open web steel joist system has the potential to significantly reduce the seismic base shear and impact on the shallow foundation system.

Table of Contents

Introduction

The proposed Parkridge Center – Phase VI building is a 226,000 Sq. Ft., seven story commercial office building located in Reston, VA. The framing system is a composite steel system with a total slab depth of 5 ¼”. The foundation is a shallow spread footing system with an allowable bearing pressure of 3000 PSF. The typical exterior bay is 37’-2” x 25’-0” and the typical interior bay is 35’-0” x 25’-0”. The overall depth of the floor system is limited to 4’-6” based on architectural sections showing location of ceiling tiles relative to the top of slab of the floor above. The required fire rating of the structural system is 2 hrs.

Gravity Loads

Live Loads – IBC Table 1607.1	
Roof Garden	100 PSF
Offices	70 PSF
Corridors	80 PSF
Stair and Exits	100 PSF
Lobbies and First Floor Corridors	100 PSF

The value of live load for offices includes a 20 PSF addition for partitions. To be consistent with the original design a value of 100 PSF will be used as the live load on a typical floor.

Assumed - Typical Floor Dead Loads		
Composite Floor System	41 PSF	Estimated Using United Steel Deck Catalog
Misc. (MEP, finishes, etc.)	10 PSF	Estimated Using AISC Manual of Steel Constr.
Ponding of Concrete	10 PSF	

Existing System

The existing floor system for Parkridge Center – Phase VI is a composite steel system. The system consists of beams spanning in the long direction and girders spanning in the short direction. The composite deck used is a 2" – 20 gage composite deck with 3 ¼" light weight concrete having a total slab depth of 5 ¼". The beams are cambered at 1 ¼" to counteract deflection.

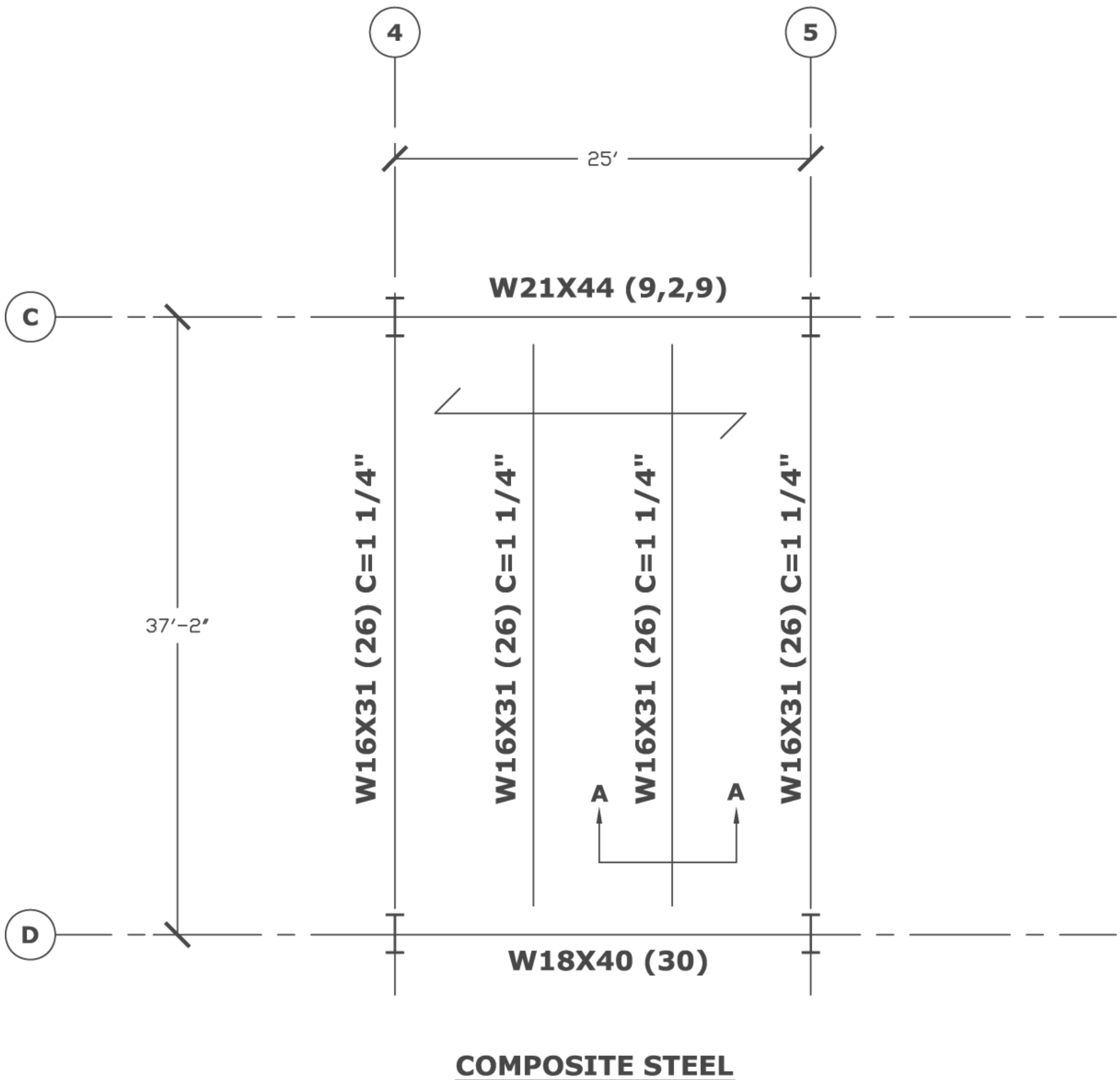


Fig. 2.1 – Existing Framing - Plan

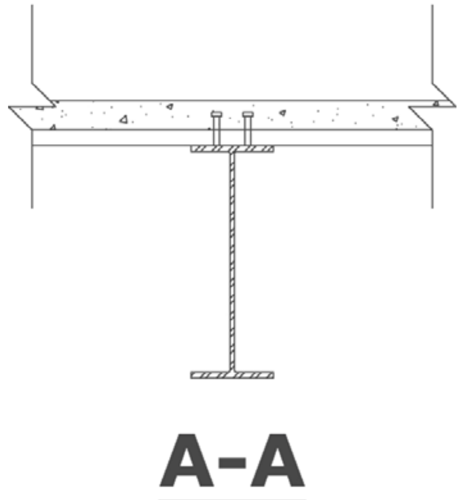


Fig. 2.2 – Existing Framing - Section

Figure 2.1 illustrates the layout of a typical exterior bay of the Parkridge office building. I have chosen to calculate my additional floor system designs using this typical bay. The W18x40 exterior girder has 30 shear studs due to the additional loading from the pre-cast curtain wall at that level.

The use of a composite system allows for the longer spans used keeping column interference with tenant space at a minimum. The system also provides ample space for MEP systems to be distributed in the allotted ceiling space. There is a potential for slight increase in price using a composite system depending on the amount of shear studs needed.

Alternative Framing Systems

The proposed alternative floor systems that will be investigated in this report are:

- Post Tension 2-Way Flat Plate Slab
- Pre-Cast Hollow Core Plank on Steel Beams
- Open Web Steel Joist with form deck
- Non-Composite Steel with form deck

These alternative systems will be checked using the typical bay illustrated in Figure 2.1.

Alternative System 1: Post Tension 2 –Way Flat Plate Slab

The first system that was chosen was a Post Tension 2-Way Flat Plate Slab. For this system I first found a preliminary column size using the axial load from technical assignment 1. For the determination of punching shear in the slab this will be conservative as the column size should increase with the change to an entirely concrete system. Using the determined column size and table 9.5(a) in ACI-318 a minimum slab thickness was determined. The determined slab thickness was 11". To use this system the typical bay had to be reduced to 27'-0" x 23'-0". The direct design method requirements are met by the typical bay and the rest of the building. The direct design method was used to determine design moments.

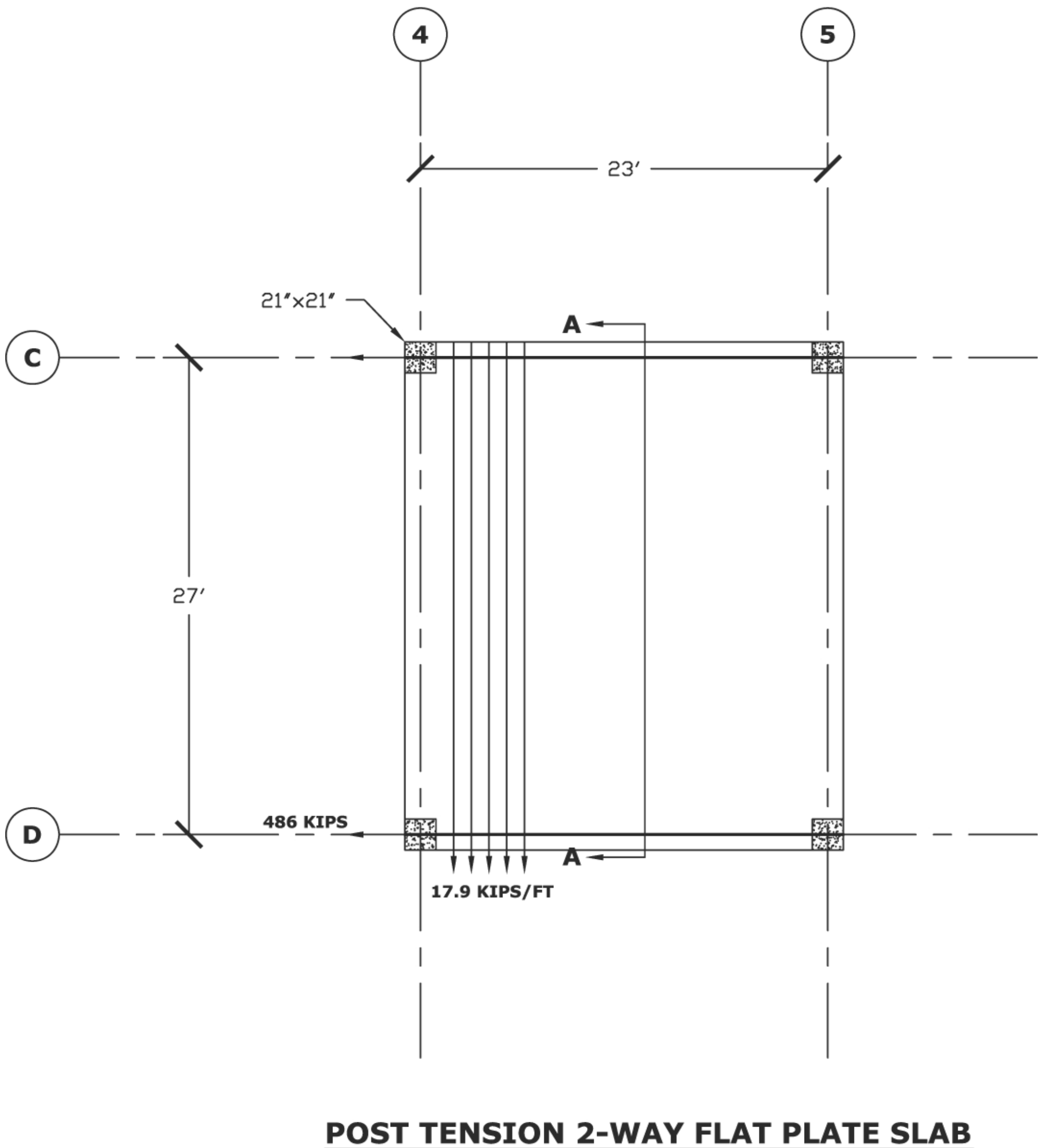


Fig. 2.3 – Alternative System 1 –Post Tension Plan

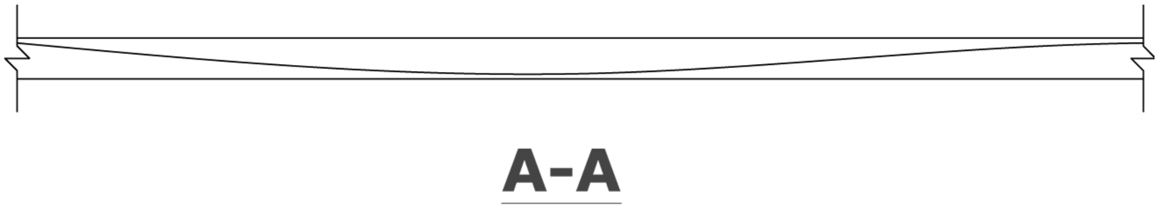


Fig. 2.4 – Alternative System 1 – Post Tension Section

I chose to band the tendons in the short direction as it has a higher tendency to accumulate load due to increased stiffness when compared to the long direction. The required jacking force for the banded tendons is 486 Kips. A required jacking force of 17.9 Kips/ft is required for the uniformly distributed tendons in the long direction.

Although the use of a post tension system requires smaller bay dimensions it significantly decrease the overall system depth. The costs associated with a post tension slab would be higher due to the increased difficulty in construction. The post tension system also meets the required fire rating of the structure without any additional fire proofing. The increased loading of the system would have a negative impact on the shallow spread footings used in the foundation. The weight would also produce larger seismic base shears negatively impacting the lateral system.

Alternative System 2: Pre-Cast Hollow Core Plank

The second system that was chosen was a pre-cast hollow core plank on steel beam system. The hollow core plank was selected based on fire rating and the Nitterhouse Concrete Products design tables. To provide a level floor service for the Parkridge office building the plank was sized with a 2" C.I.P. topping. This system also required the typical bay size to be adjusted to 36'-0" x 20'-0". This bay size was selected to minimize the number of custom planks needed. An 8" x 4' hollow core plank was selected. The controlling factor in the design of the steel support girders was deflection. A member with a moment of inertia equal to 4097.68 in^4 was required. Based on the Ix table 3-3 in the AISC Manual of Steel Construction 13th Edition the most economic member was a w30x108. The total floor system depth including allowance for MEP was 4'-2" which is with the allowable 4'-6".

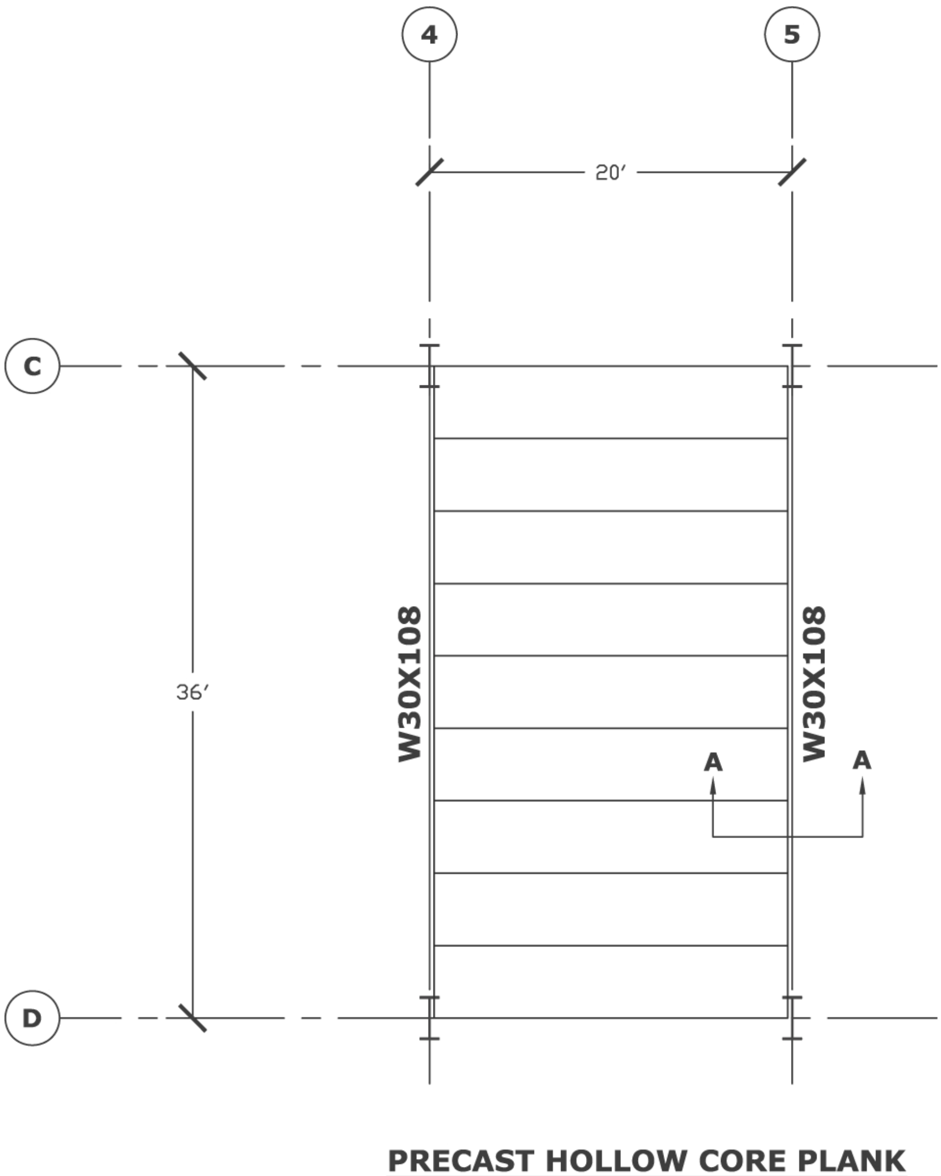


Fig. 2.5 – Alternative System 2 – Pre-cast Hollow Core Plank – Plan

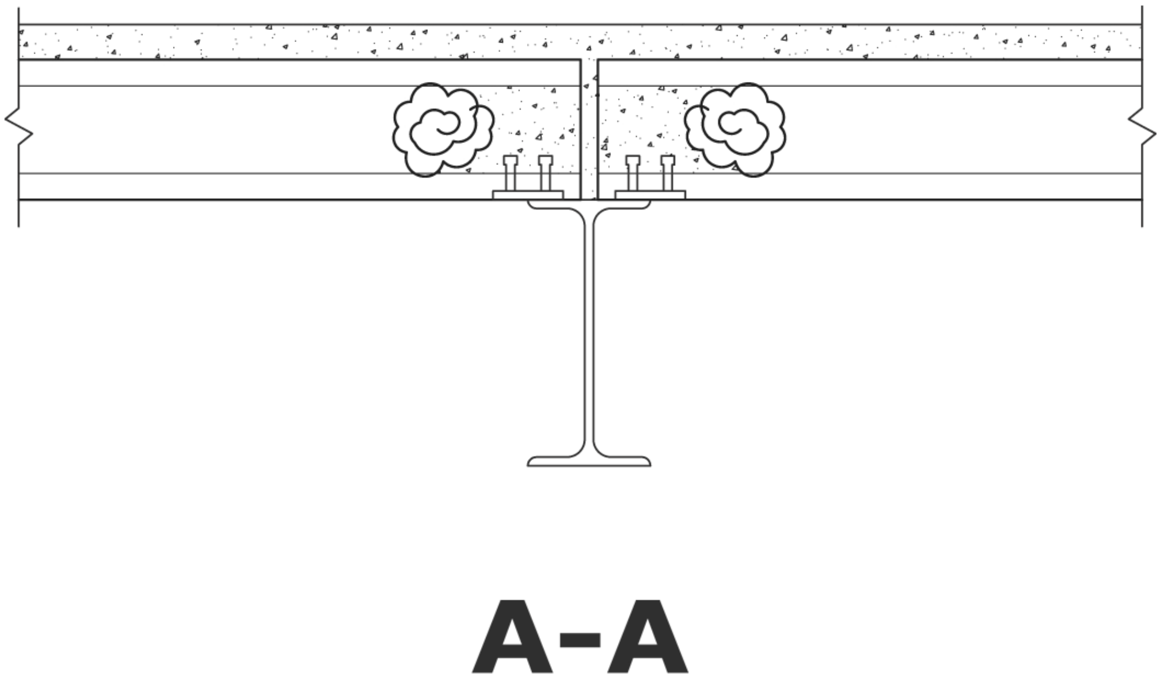


Fig. 2.6 – Alternative System 2 – Pre-cast Hollow Core Plank – Section (Detail Taken from Nitterhouse Concrete Product website)

The hollow core plank system is among the simplest and most rapid to construct. The system cost is also a minimum, but the negatives of this system for Parkridge may eliminate it from being looked into further. The hollow core plank system was the only system that challenged the depth limitation. The additional weight of the system has a negative impact on the shallow foundation system and causes an increase in the seismic base shear.

Alternative System 3: Open Web Steel Joists with Form Deck

An open web steel joist system was selected for the 3rd alternative system and was analyzed using RAM structural system. The joists were limited to an L/240 and L/360 total and live load deflection respectively. I also chose to span the joist in the long direction and have the joists spaced at 5' O.C. I chose a 5' spacing as it fits the typical bay dimension. A 20 gage UF2X deck was selected using the United Steel Desk Catalog. To achieve the required fire rating a 2 ½" concrete slab was used.

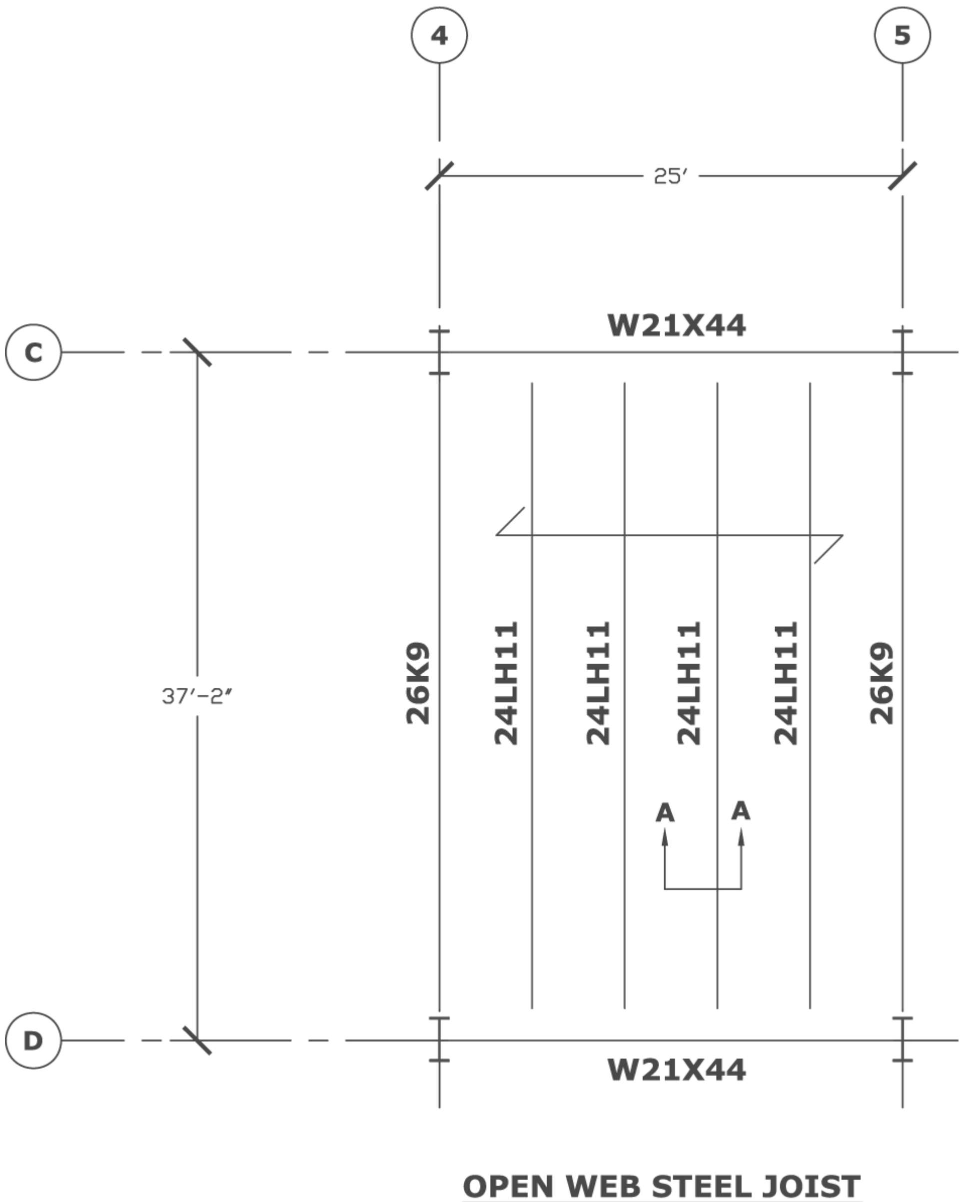
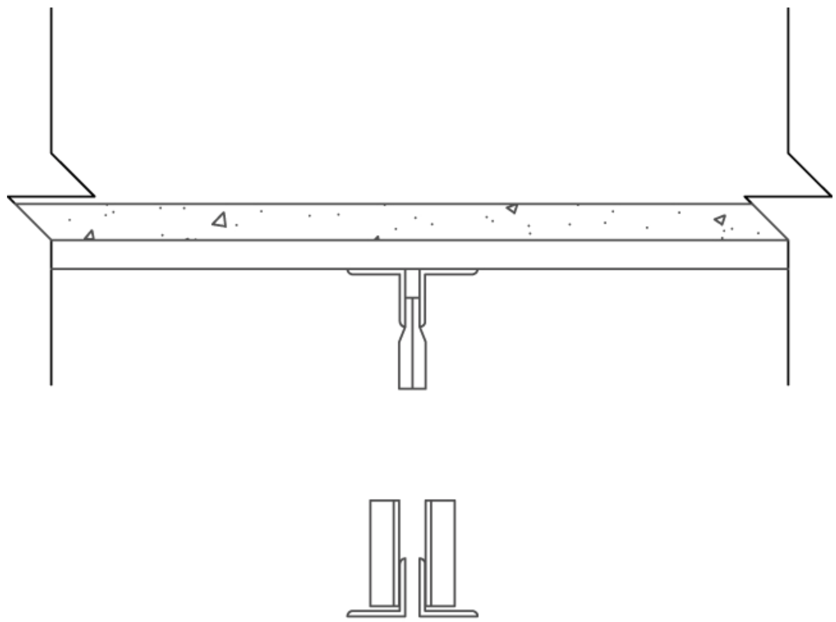


Fig. 2.7 – Alternative System 3 – Open Web Steel Joist – Plan



A-A

Fig. 2.8 – Alternative System 3 – Open Web Steel Joist – Section

The open web steel joist system is the lightest overall system out of the 5 studied. Using the open web steel joist system would decrease the seismic base shear positively impacting the lateral system. Also the decrease in weight would put less stress on the shallow foundation system. A drawback to this system however is the increased number of members per bay. A concern I have with this system is there is potential for high cost due to the need for custom members in non typical bays.

Alternative System 4: Non-Composite Steel with form deck

A non-composite steel system was selected as the final alternative system for this report. This system was analyzed using RAM structural system. Both the beams and girders were limited to an L/240 and L/360 total and live load deflection respectively. A 20 gage deck was also selected using the United Steel Deck catalog. To achieve the required fire rating a 2 ½" concrete slab was used. I chose to space the intermediate beams at the same spacing used in the existing system.

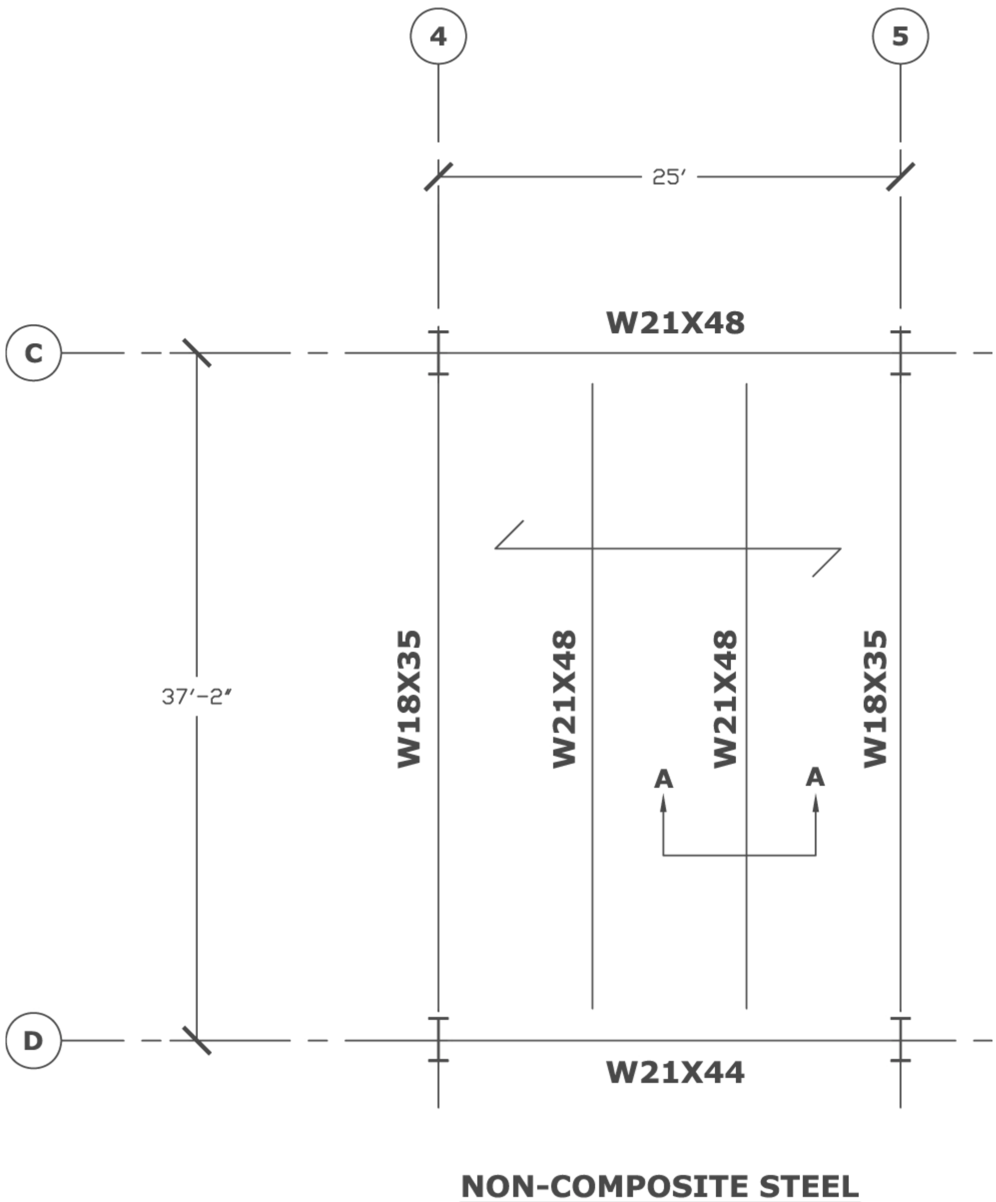


Fig. 2.9 – Alternative System 4 – Non-Composite Steel – Plan

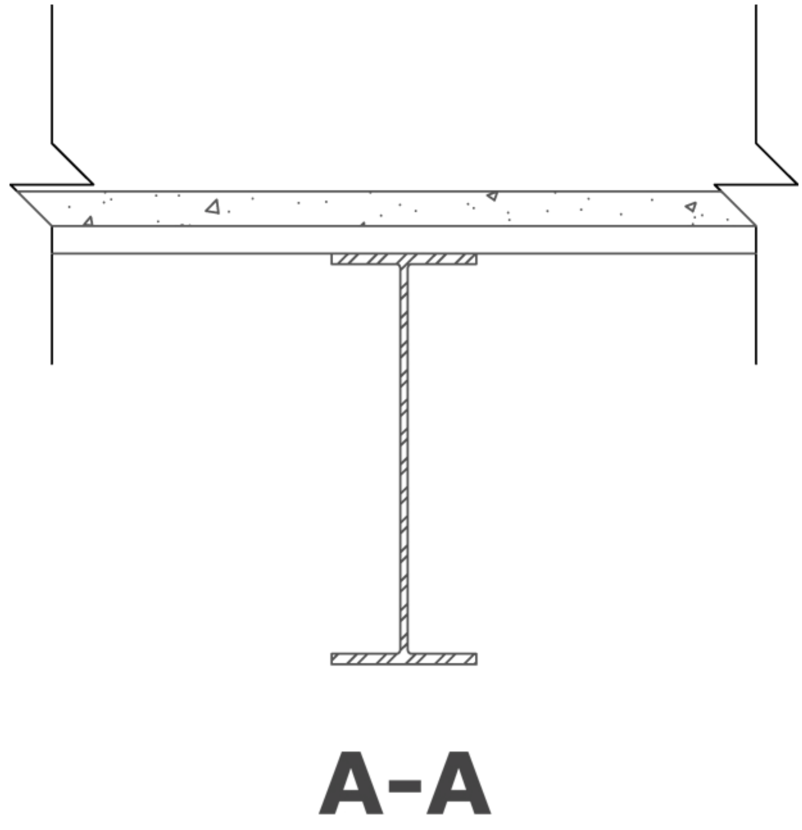


Fig. 2.10 – Alternative System 4 – Non-Composite Steel – Section

The non-composite system has the advantage of a thinner slab while keeping the original bay dimensions. Also the beams and girders are not cambered eliminating any problems that would arise with over cambering of the members. The overall depth of the system is comparable to the open web steel joist system. The increased member sizes would produce an equal cost as that of the original system. The impact from a non-composite steel system on the foundation is minimal compared to the original composite system.

Conclusions

Of the four systems analyzed in this report I feel that only the open web steel joists and 2-way Post tension slab warrant further study. A more in-depth analysis of the post tension system may yield results that minimize the resizing of typical bays. There is also a possible advantage of using the post tension with the sloping columns on the south face. The open web steel joists would allow me to keep the current bay dimensions while cutting down on the overall seismic base shear.

The following system comparison chart illustrates the differences in each system.

Floor System	Overall Depth	Span	Seismic	Foundation	Cost	Construction
Pre-Cast Hollow Core Planks	**largest**	**1 way decrease**	**increase**	**increase**	**lower**	**fast**
2-way Post Tension Slab	**smaller**	**2 way decrease**	**increase**	**increase**	**higher**	**staged**
Non-Composite Steel	**minimal change**	**no change**	**minimal increase**	**minimal change**	**minimal decrease**	**fast**
Open Web Steel Joists	**minimal change**	**no change**	**decrease**	**decrease**	**minimal decrease**	**fast**
Composite Steel	**-**	**-**	**-**	**-**	**-**	**-**

Chart 2.1 – System Comparison Chart

Appendix

Design Spreadsheet 2.1 – Direct Deign Method for 2-Way Post Tension Slab

Min. Column Size Estimation			
f'_c	4000	PSI	
P_u	1611.13	Kips	Axial Load on columns from Technical Assignment 1
$A_{req.}$	**402.78**	in^2	
B	**21**	in	
H	**21**	in	Assumed Square Columns

Minimum Slab Thickness Check			
Fy	60000	PSI	
Long Span	27.00	Ft.	
Short Span	23.00	Ft.	
$\ell_n/30$	**11**	in	
Slab Depth	11	in	
Slab DL	**137.5**	PSF	
Misc. DL	20	PSF	
LL	100	PSF	
Total Factored Load	**349**	PSF	
w_{net}	**225.25**	PSF	
Cover	0.75	in	
d	**9.75**	in	
Vu	**5.50**	Kips	
Vc	**14.80**	Kips	
Φ	0.75		
ΦVc	**11.10**	Kips	**Ok**
$V_{u,\text{two way action}}$	**139.19**	Kips	
Vc	**207.19**	Kips	
ΦVc	**155.39**	Kips	**Ok**

Check Requirements for Direct Design Method			
3 Continuous Spans EW	Y		**OK**
Span Ratio	1.17	< 2	**OK**
Span Length difference	OK		**OK**
Offset of Columns	No Offset		**OK**
Gravity Loads Only	Y		**OK**

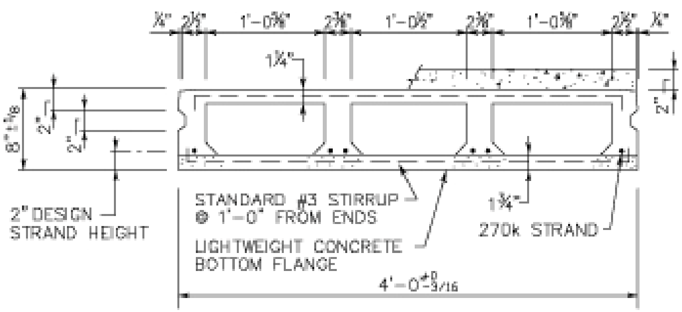
Two-Way Flat Plate System (Per 12" Width)										
Long Span				**Short Span**						
Loads				**Loads**						
Post Tension	**123.75**	PSF		Post Tension	**123.75**	PSF				
w_{net}	**225.25**	PSF		w_{net}	**225.25**	PSF				
Spans				**Spans**						
L1	27.00	Ft		L1	23.00	Ft				
L2	23.00	Ft		L2	27.00	Ft				
Factored Static Moment				**Factored Static Moment**						
Mo	**20.53**	Ft-Kips		Mo	**14.89**	Ft-Kips				
Longitudinal Distribution				**Longitudinal Distribution**						
M+	**7.18**	Ft-Kips		M+	**5.21**	Ft-Kips				
M-	**13.34**	Ft-Kips		M-	**9.68**	Ft-Kips				
Transverse Distribution				**Transverse Distribution**						
Column Strip				Column Strip						
M+	**5.39**	Ft-Kips		M+	**3.91**	Ft-Kips				
M-	**10.01**	Ft-Kips		M-	**7.26**	Ft-Kips				
Middle Strip				Middle Strip						
M+	**1.80**	Ft-Kips		M+	**1.30**	Ft-Kips				
M-	**3.34**	Ft-Kips		M-	**2.42**	Ft-Kips				
Long Span				**Short Span**						
w_{pre}	**123.75**	PSF		w_{pre}	**123.75**	PSF				
M_{pre}	**8.18**	Ft-Kips		M_{pre}	**11.28**	Ft-Kips				
a	**5.5**	in		a	**5.5**	in				
F	**17.85**	Kips		F	**24.60**	Kips				
F/A	**135.26**	PSI		F/A	**186.39**	PSI				
Average Stresses - Column Strip										
Negative Long Span				**Negative Short Span**						
S	**242**	in^3		S	**242**	in^3				
f	**360.93**	PSI	**OK**	f	**173.66**	PSI	**OK**	**379.47**	PSI	6√F'c
	-631.44	PSI	**OK**		**-546.45**	PSI	**OK**	**1800**	PSI	0.45*F'c
Positive Long Span				**Positive Short Span**						
S	**242**	in^3		S	**242**	in^3				
f	**131.92**	PSI	**OK**	f	**7.48**	PSI	**OK**	**189.74**	PSI	3√F'c
	-402.43	PSI	**OK**		**-380.27**	PSI	**OK**	**1800**	PSI	0.45*F'c

Design Table 2.1 – Nitterhouse Concrete Products Hollow Core Plank

Prestressed Concrete
8"x4' SpanDeck–U.L.–J952
(2" C.I.P. TOPPING)

PHYSICAL PROPERTIES
Composite

A' = 295 in.^{2}	S'$_b$ = 468 in.^{3}
I' = 2624 in.^{4}	S'$_t$ = 1096 in.^{3} (At Top of SpanDeck)
Y'$_b$ = 5.61 in.	S'$_{tt}$ = 597 in.^{3} (At Top of Topping)
Y'$_t$ = 2.39 in. (To Top of SpanDeck)	Wt. = 330 PLF
Y'$_{tt}$ = 4.39 in. (To Top of Topping)	Wt. = 82.5 PSF



8" SPANDECK CROSS SECTION
UL FIRE RATED J952

DESIGN DATA
1. Precast Strength @ 28 days = 5000 PSI.
2. Precast Strength @ release = 3000 PSI.
3. Precast Density = 150 PCF (Top and Webs)
   = 115 PCF (Soffit)
4. Strand = 1/2"ø, 270 K Lo-Relaxation.
5. Composite Strength = 3000 PSI.
6. Composite Density = 150 PCF.
7. Strand Height = 2.00 in.
8. Ultimate moment capacities (when fully developed)...
   4 – 1/2"ø, 270K = 88.3'K
   6 – 1/2"ø, 270K = 124.0'K
9. Maximum bottom tensile stress is $6\sqrt{f'c}$ =424 PSI.
10. All superimposed load is treated as live load in the strength analysis of flexure and shear.
11. Flexural strength capacity is based on stress/strain strand relationships.
12. Shear values are the maximum allowable before shear reinforcement is required.
13. Deflection limits were not considered when determing allowable loads in this table.
14. Load values to the left of the solid line are controlled by ultimate strength. Load values to the right are controlled by service stress.
15. All loads shown refer to allowable loads applied after the topping has hardened.

8" SPANDECK W/2" TOPPING	ALLOWABLE SUPERIMPOSED LOAD (PSF)																						
STRAND PATTERN	SPAN (FEET)																						
	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32
Flexure 4 – 1/2"ø	750	675	611	546	462	394	338	291	252	218	191	167	146	128	112	98	85	74	63	51	41	31	
Shear 4 – 1/2"ø	527	469	421	382	348	317	294	272	252	235	219	197	176	157	140	129	122	110	98	88	78	70	
Flexure 6 – 1/2"ø	1098	900	898	794	676	580	502	437	382	336	296	262	233	207	185	165	147	132	116	101	87	74	63
Shear 6 – 1/2"ø	542	483	434	393	359	329	303	280	261	243	227	212	199	188	178	167	152	137	124	112	101	91	86

NITTERHOUSE
CONCRETE PRODUCTS

This table is for simple spans and uniform loads. design data for any of these span–load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths.

2655 MOLLY PITCHER HWY. SOUTH, BOX N
CHAMBERSBURG, PA 17201-0813
717-267-4505 • FAX: 717-267-4518

REVISED 12/93

Handcalc 2.1 – System 2 Hollow Core Plank – Girder

G-46 REV 9/96

ENGINEERING [redacted] COMPUTATION SHEET

SHEET NO.

TITLE OF PROJ. OR STUDY | PROJ. OR STUDY NO.

SUBJECT | WORKS

COMPUTER | DATE | 20

HOLLOW CORE PRE-CAST PLANK

NEED 2 HR FIRE RATING

8" x 4' SPANDECK

PRECAST DL = 82.5 PSF

SUPERIMPOSED LOADS: 100 PSF LL
20 PSF SDL
120 PSF SL (UNFACTORED)

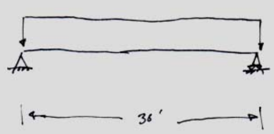
$1.2(20) + 1.6(100) = 184$ PSF

STRAND PATTERN 6 - 1/2" ϕ

SPAN 20'

MOVE LONG SPAN TO 36'

BEAM: 184 PSF + 1.2(82.5 PSF) = 283 PSF (20) = 5.66 KLF



$$M_{max} = \frac{5.66\,(36)^2}{8} = 916.92\ {'k}$$

$$V = \frac{wl}{2} = \frac{5.66\,(36)}{2} = 101.88\ k$$

$$\Delta \leq \frac{l}{240}$$

$$\frac{5\,(5.66)(36)^4\,(1728)}{384\,(29000)\,I} \leq \frac{36\,(12)}{240}$$

$$\frac{7375.83}{I} \leq 1.8$$

$I \geq 4097.68\ in^4$ TRY W30 x 108 $I_x = 4470$

$\phi M_n = 1297\ {'k} > 916.92\ {'k}$ OK

BM DEPTH: $29.8" + \overset{MEP}{10"} + \overset{\Delta}{1.8"} + \overset{PRE}{8"} = 4'\text{-}1.6" < 4'\text{-}6"$ ALLOWABLE OK

USE STANDARDS FOR MINIMUM ESSENTIAL INSTALLATIONS

Ram Printout 2.1 – System 3 Open Web Steel Joist – Joist

Standard Joist Selection

RAM Steel v10.0
DataBase: Thesis - Parkridge VI 10/27/06 12:12:31
Building Code: IBC

Floor Type: 2nd - Joists **Beam Number = 276**

SPAN INFORMATION (ft): I-End (66.00,0.00) J-End (66.00,37.17)

Maximum Depth Limitation specified = 26.00 in
Joist Size (Optimum) = 24LH11
Total Beam Length (ft) = 37.17

LINE LOADS (k/ft):

Load	Dist	DL	LL	Red%	Type
1	0.000	0.100	0.500	0.0%	Red
	37.166	0.100	0.500		
2	0.000	0.000	0.000	---	NonR
	37.166	0.000	0.000		

Maximum Total Unif. Load at any location (lbs/ft) : 600.0

Allowable Stress Ratio: 1.00

	Design Loads	Allowable Loads (lbs/ft)
Dead:	100.0	
Live:	500.0	511.6
Total:	600.0	767.4

MOMENTS:

Span	Cond	Moment kip-ft	@ ft
Center	Max +	103.6	18.6

REACTIONS (kips):

	Left	**Right**
DL reaction	1.86	1.86
Max +LL reaction	9.29	9.29
Max +total reaction	11.15	11.15

DEFLECTIONS:

Dead load (in)	=	0.242	L/D	=	1842
Live load (in)	=	1.211	L/D	=	368
Total load (in)	=	1.453	L/D	=	307

Ram Printout 2.2 – System 3 Open Web Steel Joist – Girder

Gravity Beam Design

RAM Steel v10.0
DataBase: Thesis - Parkridge VI — 10/27/06 12:12:31
Building Code: IBC — Steel Code: AISC LRFD

Floor Type: 2nd - Joists **Beam Number = 306**

SPAN INFORMATION (ft): I-End (56.00,37.17) J-End (81.00,37.17)

Beam Size (Optimum) = W21X44 Fy = 50.0 ksi
Total Beam Length (ft) = 25.00
Mp (kip-ft) = 397.50

POINT LOADS (kips):

Dist	DL	RedLL	Red%	NonRLL	StorLL	Red%	RoofLL	Red%
5.000	1.75	8.75	35.5	0.00	0.00	0.0	0.00	Snow
5.000	1.86	9.29	35.5	0.00	0.00	0.0	0.00	Snow
10.000	1.75	8.75	35.5	0.00	0.00	0.0	0.00	Snow
10.000	1.86	9.29	35.5	0.00	0.00	0.0	0.00	Snow
15.000	1.75	8.75	35.5	0.00	0.00	0.0	0.00	Snow
15.000	1.86	9.29	35.5	0.00	0.00	0.0	0.00	Snow
20.000	1.75	8.75	35.5	0.00	0.00	0.0	0.00	Snow
20.000	1.86	9.29	35.5	0.00	0.00	0.0	0.00	Snow

LINE LOADS (k/ft):

Load	Dist	DL	LL	Red%	Type
1	0.000	0.044	0.000	---	NonR
	25.000	0.044	0.000		

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 46.55 kips 0.90Vn = 195.62 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	Max +	1.2DL+1.6LL	348.3	12.5	5.0	1.00	0.90	349.69
Controlling		1.2DL+1.6LL	348.3	12.5	5.0	1.00	0.90	349.69

REACTIONS (kips):

	Left	**Right**
DL reaction	7.77	7.77
Max +LL reaction	23.27	23.27
Max +total reaction (factored)	46.55	46.55

DEFLECTIONS:

Dead load (in)	at	12.50 ft	=	-0.267	L/D =	1124
Live load (in)	at	12.50 ft	=	-0.809	L/D =	371
Net Total load (in)	at	12.50 ft	=	-1.076	L/D =	279

Ram Printout 2.4 – System 4 Non-Composite – Beam

Gravity Beam Design

RAM Steel v10.0
DataBase: Thesis - Parkridge VI — 10/27/06 12:12:31
Building Code: IBC — Steel Code: AISC LRFD

Floor Type: 2nd **Beam Number = 107**

SPAN INFORMATION (ft): I-End (64.33,0.00) J-End (64.33,37.17)

Beam Size (Optimum) = W21X48 Fy = 50.0 ksi
Total Beam Length (ft) = 37.17
Mp (kip-ft) = 445.83

LINE LOADS (k/ft):

Load	Dist	DL	LL	Red%	Type
1	0.000	0.167	0.834	14.7%	Red
	37.166	0.167	0.834		
2	0.000	0.048	0.000	---	NonR
	37.166	0.048	0.000		

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 25.92 kips 0.90Vn = 194.67 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu	@	Lb	Cb	Phi	Phi*Mn
			kip-ft	ft	ft			kip-ft
Center	Max +	1.2DL+1.6LL	240.8	18.6	0.0	1.00	0.90	398.35
Controlling		1.2DL+1.6LL	240.8	18.6	0.0	1.00	0.90	398.35

REACTIONS (kips):

	Left	**Right**
DL reaction	3.99	3.99
Max +LL reaction	13.21	13.21
Max +total reaction (factored)	25.92	25.92

DEFLECTIONS:

Dead load (in)	at	18.58 ft	=	-0.331	L/D	=	1346
Live load (in)	at	18.58 ft	=	-1.097	L/D	=	407
Net Total load (in)	at	18.58 ft	=	-1.429	L/D	=	312

Ram Printout 2.5 – System 4 Non-Composite – Girder

Gravity Beam Design

RAM Steel v10.0
DataBase: Thesis - Parkridge VI 10/27/06 12:12:31
Building Code: IBC Steel Code: AISC LRFD

Floor Type: 2nd **Beam Number = 28**

SPAN INFORMATION (ft): I-End (56.00,37.17) J-End (81.00,37.17)

Beam Size (Optimum) = W21X48 Fy = 50.0 ksi
Total Beam Length (ft) = 25.00
Mp (kip-ft) = 445.83

POINT LOADS (kips):

Dist	DL	RedLL	Red%	NonRLL	StorLL	Red%	RoofLL	Red%
8.330	3.69	14.59	31.8	0.00	0.00	0.0	0.00	Snow
8.330	3.99	15.49	31.8	0.00	0.00	0.0	0.00	Snow
16.670	3.69	14.59	31.8	0.00	0.00	0.0	0.00	Snow
16.670	3.99	15.49	31.8	0.00	0.00	0.0	0.00	Snow

LINE LOADS (k/ft):

Load	Dist	DL	LL	Red%	Type
1	0.000	0.048	0.000	---	NonR
	25.000	0.048	0.000		

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 42.78 kips 0.90Vn = 194.67 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu	@	Lb	Cb	Phi	Phi*Mn
			kip-ft	ft	ft			kip-ft
Center	Max +	1.2DL+1.6LL	354.8	12.5	8.3	1.00	0.90	369.23
Controlling		1.2DL+1.6LL	354.8	12.5	8.3	1.00	0.90	369.23

REACTIONS (kips):

	Left	**Right**
DL reaction	8.28	8.28
Max +LL reaction	20.53	20.53
Max +total reaction (factored)	42.78	42.78

DEFLECTIONS:

Dead load (in)	at	12.50 ft	=	-0.280	L/D	=	1072
Live load (in)	at	12.50 ft	=	-0.707	L/D	=	424
Net Total load (in)	at	12.50 ft	=	-0.987	L/D	=	304