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Structural Option
Building: Vickroy Hall
Location: Duquesne University
Pittsburgh, PA 15282
Date: October 27, 2006
Title of Report: Technical Report 2
Faculty Consultant: Dr. Boothby

Executive Summary

The purpose of this report is to analyze, design, and evaluate four alternative floor systems in the building. The report will give preliminary sizes of members, depths, and other pertinent information about each system. Figures from Handbooks are present as well as hand calculations and tables.

The four alternative floor systems that I chose were Hollow Core Planking on Steel Supports, One-Way Concrete Joist System, Two-Way Flat Plate System, and finally Hollow Core Planking on Concrete Beams and Masonry Bearing Walls.

Through calculations and tables, I have decided that all systems, including the original system require further investigation. The only system that I am remotely unsure about is the Hollow Core on Masonry Bearing Walls. It is difficult to tell if placing bearing walls in place of the columns will result in a change in the architecture or physical spaces in the building. Therefore, I will still investigate further into the system.

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1. Introduction

Completed in 1997, standing eight stories above ground, and encompassing 77,000 square feet within its walls, Vickroy Hall provides Living and Learning areas for up to 280 upper class students of Duquesne University. The living quarters are suites with two double rooms and an attached private bathroom. The learning quarters are multiple meeting rooms complete with tables and lounge chairs.

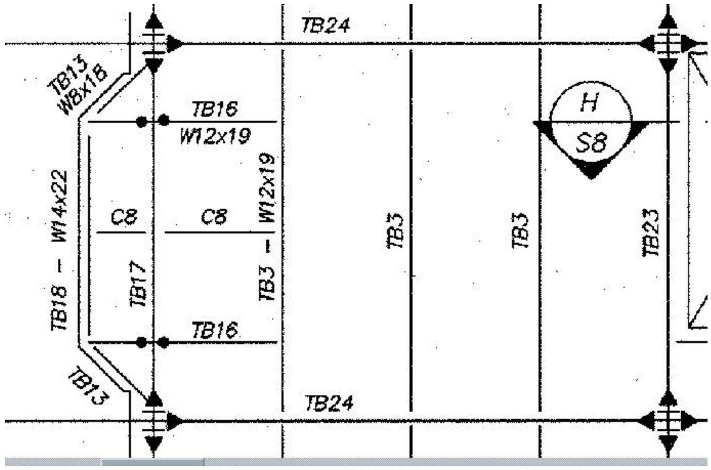
The building sports multiple protrusions to give it interesting dimension when compared to the buildings around it. There are also two story columns on the exterior of the building to give it a ‘floating’ look and add to its prestigious façade. The first two floors are atypical due to the columns and the need for a lobby, large meeting rooms, and offices. However, the floors above take on a more typical structure.

This report is designed to take a closer look at the typical floor structure of Vickroy Hall and undertake the task of designing alternative systems that could have worked in the building. It will evaluate four alternative floor systems and compare them to the original system.

2. The Current System

2.1 Current System in Drawings

The main structural system consists of structural steel members including W-shapes and C-channels. The W-shapes are the framing for typical members and the C-channels provide support for the cantilevers and other protrusions. They are usually oriented perpendicular to the other framing members. The main members extending from column to column are detailed as moment connections. These moment connections are either classified as a wind moment connections or a moment resisting connections. The typical floor plan generally calls for W12 to W16’s. (See partial framing plan below or Figure 1 in the Appendix which illustrates the typical full original framing plan.)



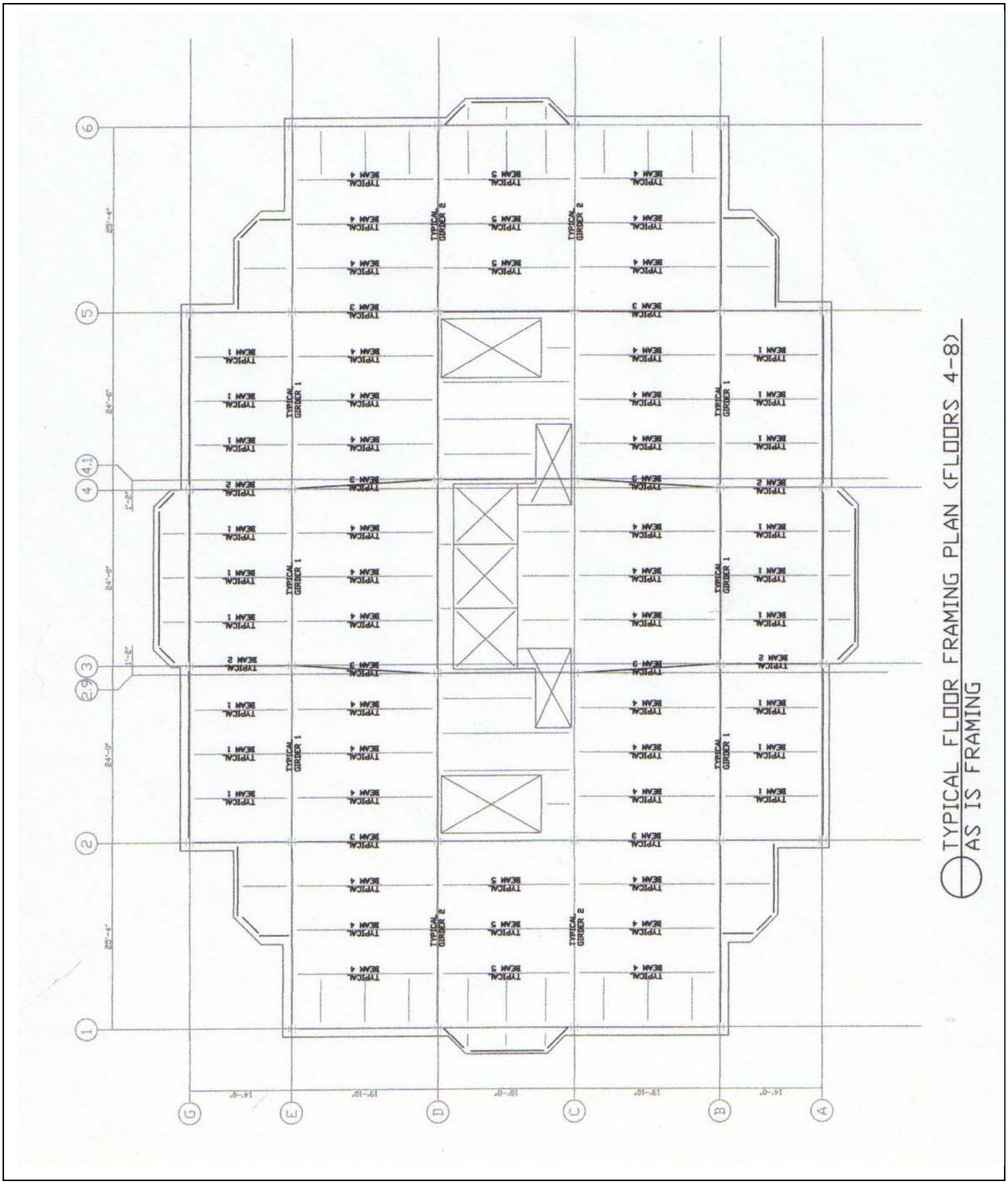
Typical Partial Framing Plan

The floor system is a non-composite metal and concrete deck. On a typical floor, the deck is 2” – 20 gage corrugation with 3-1/4” light weight concrete and 6x6 – W2.9 x W 2.9 welded wire fabric. The deck was to be welded to the supporting structural member. (See photo below)

Typical Floor System: Shows corrugated metal deck supported by steel framing

2.2 Analysis

My analysis used the typical floor system to design the typical members of the system. I did not take into account wind or seismic forces, but designed strictly for gravity loads. The moment connections, as they exist, were taken into account as fixed-fixed beams when designing. All other connections were assumed to be simply supported. My loads were revised from Technical Assignment 1 to reflect IBC 2003 instead of BOCA 1993. With this revision, some of my members were the same as the original, but most differed. (See Framing Plan) Calculations for the current system can be found in the Appendix as Figures 6-9. The typical member sizes of my analysis versus that of the original are shown in the table below.


TYPICAL FLOOR FRAMING PLAN (FLOORS 4-8)
AS IS FRAMING

Typical Beam Name (Current/Analyzed)	Current Size	Analyzed Size
Typical Beam 4/Typical Beam 1	W 10x15	W 10x15
Typical Beam 5/Typical Beam 2	W 18x35	W 10x15
Typical Beam 2/Typical Beam 3	W 21x44	W 10x15
Typical Beam 1/ Typical Beam 4	W 14x22	W 12x22
Typical Beam 3/Typical Beam 5	W 12x19	W 12x19
Typical Beam 14/Typical Girder 1	W 21x62	W 18x71
Typical Beam 24/Typical Girder 2	W 21x62	W 21x73

2.3 Evaluation of the System

This evaluation will highlight the pros and cons of the current system.

<u>Current System Pros</u>

- Has withstood the test of time
- Steel is constructed relatively fast
- Building did not show stress cracking in masonry facade
- Relatively light system
- Plenty of plenum space between floors for MEP

<u>Current System Cons</u>

- Moment Frames are expensive
- Moment Frames take longer constructability time
- No shear walls – moment connections take all of the wind and seismic loads

3. Alternative System 1: Hollow Core Planking on Steel Supports

3.1 The System

Hollow core planking is a type of precast concrete system that can be constructed a multitude of ways. The planks are cast in long lengths and cut to size to accommodate the project. The hollow cores can be filled with grout for added strength if need be. A topping slab may also be added for either structural purposes or strictly leveling. For this system, the precast will be supported by structural steel members. The system I analyzed has a two-inch topping for both structural integrity and to make sure the floor is level. The Nitterhouse Concrete Products website provided free specifications and details for their typical planks and coinciding connections.

3.2 Analysis

From the Nitterhouse Concrete Products site, I chose the J952 planking system. The full PDF of the specifications can be found in the Appendix as Figure 2. The planks are four stranded 8” x 4’ wide members. The weight of each plank is 82.5 psf or 330 plf. The strength of the member is 3000 psi when it arrives on site, and the 28-day strength is 5000 psi. The allowable loads are located on the bottom of the PDF from Nitterhouse Concrete Products.

Following the original floor plan, the columns were kept the same and the orientation of the planks followed that of the original beams for simplicity at the cantilevered and protruding sections. There were four typical planks. The planks were assumed to be simply supported with minor tack welds to the supporting members. The sections where planks were not designed for were atypical, such as around the core of the building, which houses the elevator shafts and stairwells. Such analysis was beyond the scope of this report. The typical supporting members were also designed for. However, they were not designed as fixed-fixed members as in the original system, but simply supported.

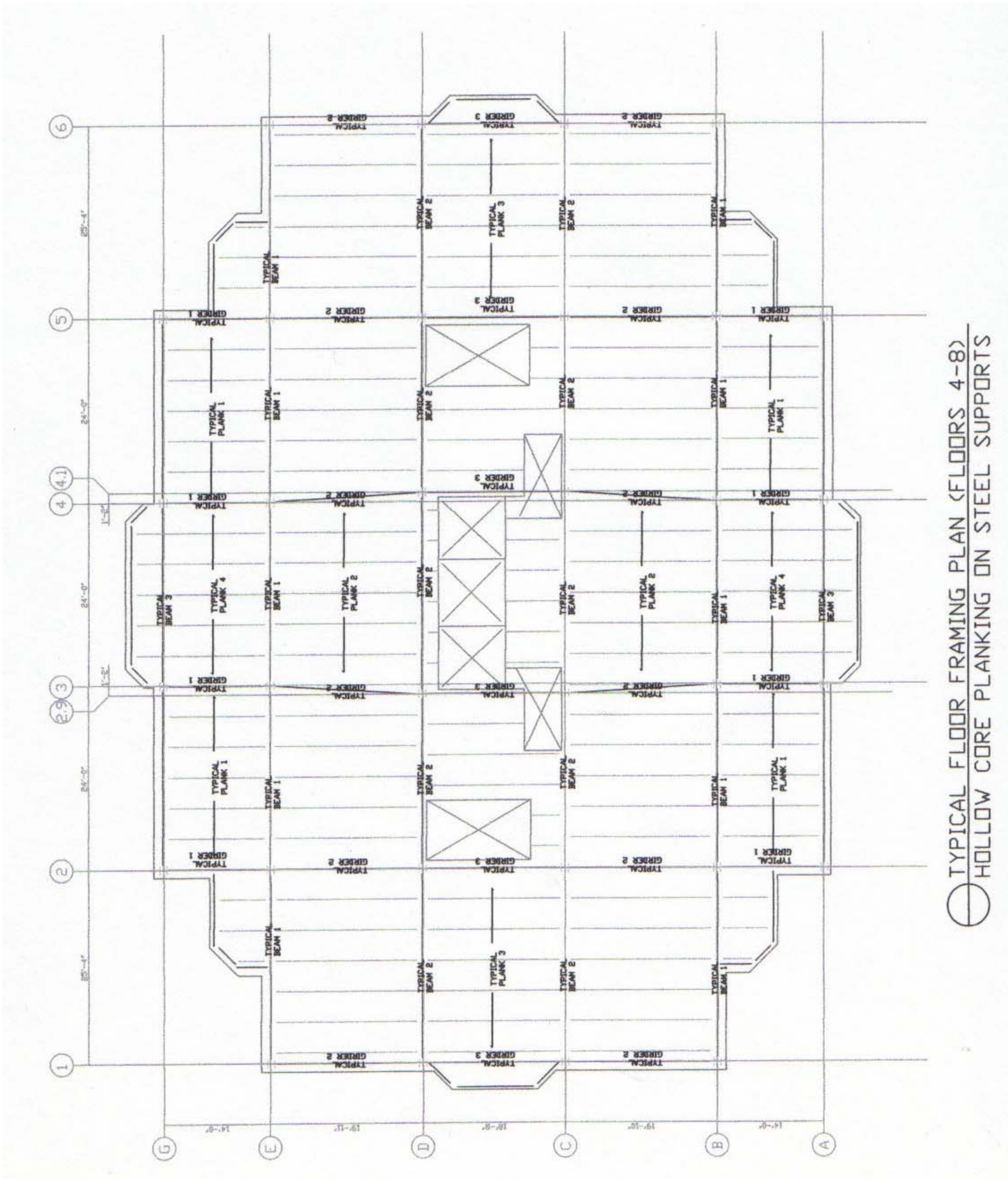
Other Assumptions:

- Deflection of the planks is calculated into the allowable loads given in the Nitterhouse Concrete Products specifications
- Since there is topping on the planks, the planks will not serve as point loads on the supporting members, but as a distributed load along the length of the member.
- Supporting members are unbraced except for necessary tack welds along the length as defined by the precastor
- Only gravity was accounted for in the floor system

The typical floor plan of this system is shown below. The beams and girders are labeled as typical member (#). The summary for each member is shown in the table below. Deflection controlled for most members and the most economical size was not chosen. The member with the closest value to the controlling property was chosen.

Typical Member	**Typical Member Size**
Typical Beam 1	W16x89
Typical Beam 2	W21x83
Typical Beam 3	W10x22
Typical Girder 1	W14x38
Typical Girder 2	W16x77
Typical Girder 3	W12x106

Further calculations may be found in the Appendix as Figures 10-15.


TYPICAL FLOOR FRAMING PLAN (FLOORS 4-8)
HOLLOW CORE PLANKING ON STEEL SUPPORTS

3.3 Evaluation of the System

This evaluation will highlight the pros and cons of the precast hollow core plank on structural steel members system.

Hollow Core System Pros	**Hollow Core System Cons**
o Durable system[1]	o Cannot change column spacing due to façade
o Inherently Fire Resistant[1]	o Much cutting will be needed to size to the original column spacing as planks come in 4' sections
o Fast Installation[1]	o While spanning in shorter direction
o Noise Attenuation[1]	o Easier to construct but more pieces
o Less expensive	o Cantilevered sections
	o Larger supporting member sizes

4. Alternate System 2: One Way Concrete Joist System with Concrete Beams

4.1 The System

One way concrete joist systems can basically take a bay that can be a two way system and force it to be a one way system. The joist system is a 'monolithic combination of regularly spaced joists (ribs) and a thin slab of concrete cast in place to form an integral unit with the supporting beams, columns, or walls.'[2] The system uses forms repeatedly to construct the floor system. Many sizes and depths are available. The system was 'developed to save dead weight and reinforcement.'[2]

4.2 Analysis

For the analysis, I chose a form that would require the least amount of atypical formwork. Vickroy Hall has many unusual spans that do not necessarily divide easily into the forms that are generally used for the concrete joist system. The form that was analyzed was thirty inches plus another six inches for the rib. The smallest depth was chosen to allow for plenum space and to keep the typical floor to ceiling height. This height consists of a ten inch deep rib with a three inch top slab for a total depth of thirteen inches. To use the Joist tables, the unit weight per length must be known. In this case, it was 138 psf, which was determined from the current system floor load.

Other Assumptions:

- At large openings:
  - Will need header joists and more reinforcing
- Deflection: all load capacities have been investigated for deflection by CRSI
- Unequal Continuous Spans:
  - Cause differences in moments
  - Limitations
    - Live Load <= 3 Dead Load

- Check 40< 3(60)
    - Larger span to adjacent span shall not be greater than 20% the length of the shorter span
        - Check span: 14' to 19'-10" : 42% greater, therefore, must span in the long direction (lettered column lines)
- Material Strengths
    - f'c = 4000 psi
        - Normal weight concrete
    - fy = 60000 psi
- Loadings
- CRSI Factors loads such that: 1.4D + 1.7L
    - This will be conservative to IBC 2003

f'_c = 4,000 psi
f_y = 60,000 psi

JOIST-BAND BEAMS, END SPANS

STEM h in.	STEM b in.	BARS(1) BOTTOM ℓ_n + 12 in.	BARS(1) BOTTOM 0.875 ℓ_n	Layers (2)	TOP	SPAN, ℓ_n = 12 ft LOAD (4) k/ft	STIR. TIES (5)	ϕT_n ft-kips	$A\ell$ sq. in.	STEEL WGT lb.	SPAN, ℓ_n = 14 ft LOAD (4) k/ft	STIR. TIES (5)	ϕT_n ft-kips	$A\ell$ sq. in.	STEEL WGT lb.
		2#6	1#6	1	4#6	4.4	103C	6	-	166	3.2	113C	6	-	187
				1			143C	25	1.0	218		173C	25	1.0	256
		2#6	2#6	1	4#7	5.7	123C	6	-	215	4.2	133C	6	-	241
				1			143C	25	1.0	255		173C	25	1.0	299
	24	4#6	3#6	1	4#9	9.5	143C	6	-	336	7.0	163C	6	-	383
				1			144C	25	1.0	474		174C	25	1.0	556
		10#4	9#4	2	4#10	10.6	183B	6	-	426	7.8	203B	6	-	482
				1			184B	25	1.0	603		214B	25	1.0	694
		2#6	2#6	1	5#6	5.9	N/A	11	-	140	4.3	N/A	10	-	160
				1			243A	43	1.5	367		283A	42	1.5	425
		3#6	3#6	1	5#8	8.6	123C	11	-	312	6.3	133C	10	-	352
				1			243A	43	1.5	460		283A	42	1.5	530
12.5	36	5#6	5#6	1	5#10	13.7	143C	11	-	482	10.1	163C	10	-	548
				1			144C	43	1.5	650		174C	42	1.5	760
		7#6	6#6	1	5#11	17.2	144C	11	-	681	12.6	174C	10	-	788
				1			244A	43	1.5	947		284A	42	1.5	1089

f'_c = 4,000 psi
f_y = 60,000 psi

JOIST-BAND BEAMS, INTERIOR SPANS

STEM h in.	STEM b in.	BARS(1) BOTTOM ℓ_n + 12 in.	BARS(1) BOTTOM 0.875 ℓ_n	Layers (2)	TOP	SPAN, ℓ_n = 20 ft LOAD (4) k/ft	STIR. TIES (5)	ϕT_n ft-kips	$A\ell$ sq. in.	STEEL WGT lb.	SPA LOAD (4) k/ft	STI TIE (5
		2#6	1#6	1	4#6	2.1	103C	6	-	222	1.7	093
				1			243C	24	1.0	351		263
		2#7	1#7	1	4#7	2.7	143C	6	-	304	2.3	143
				1			243C	24	1.0	411		263
	24	2#8	1#8	1	4#9	3.9	173C	6	-	429	3.2*	183
				1			243C	24	1.0	519		263
		2#9	1#9	1	4#10	4.8*	233B	6	-	553	3.9*	243
				1			303B	24	1.0	655		333
		2#6	2#6	1	5#7	3.1	N/A	10	-	244	2.5	N/
				1			403A	41	1.5	623		443
		2#7	2#7	1	5#8	4.1	N/A	10	-	325	3.4	N/
				1			403A	41	1.5	704		443
12.5	36	2#9	2#9	1	5#10	6.4*	223B	10	-	678	5.3*	233

f'_c = 4,000 psi
f_y = 60,000 psi

JOIST-BAND BEAMS, INTERIOR SPANS

STEM h in.	STEM b in.	BARS(1) BOTTOM ℓ_n + 12 in.	BARS(1) BOTTOM 0.875 ℓ_n	Layers (2)	TOP	TOTAL CAPACITY SPAN, ℓ_n = 12 ft LOAD (4) k/ft	STIR. TIES (5)	ϕT_n ft-kips	$A\ell$ sq. in.	STEEL WGT lb.	SPAN, ℓ_n = 14 ft LOAD (4) k/ft	STIR. TIES (5)	ϕT_n ft-kips	$A\ell$ sq. in.	STEEL WGT lb.
		2#6	1#6	1	4#6	5.7	103C	6	-	157	4.2	103C	6	-	173
				1			143C	26	1.1	209		173C	25	1.0	248
		2#6	2#6	1	4#7	7.6	113C	6	-	194	5.6	123C	6	-	221
				1			144C	26	1.1	348		173C	25	1.0	284
	24	3#6	3#6	1	4#9	11.6	133C	6	-	281	8.6	143C	6	-	321
				1			144C	26	1.1	425		174C	25	1.0	505
		4#6	3#6	1	4#10	13.8	133C	6	-	330	10.2	153C	6	-	383
				1			144C	26	1.0	473		174C	25	1.0	561
		2#6	2#6	1	5#7	8.5	103C	11	-	214	6.3	N/A	11	-	172
				1			243A	44	1.6	375		283A	43	1.5	437
		3#6	2#6	1	5#8	10.5	113C	11	-	265	7.7	123C	11	-	303
				1			243A	44	1.6	420		283A	43	1.5	489
12.5	36	4#6	4#6	1	5#10	16.3	133C	11	-	391	12.0	143C	11	-	447
				1			144C	44	1.6	566		174C	43	1.5	672
		5#6	5#6	1	5#11	19.9	134C	11	-	548	14.6	154C	11	-	636
				1			244A	44	1.6	826		284A	43	1.5	963

f'_c = 4,000 psi
f_y = 60,000 psi

JOIST-BAND BEAMS, END SPANS

STEM h in.	STEM b in.	BARS(1) BOTTOM ℓ_n + 12 in.	BARS(1) BOTTOM 0.875 ℓ_n	Layers (2)	TOP	SPAN, ℓ_n = 20 ft LOAD (4) k/ft	STIR. TIES (5)	ϕT_n ft-kips	$A\ell$ sq. in.	STEEL WGT lb.	SPAN LOAD (4) k/ft	STIR. TIES (5)
		2#6	1#6	1	4#6	1.6	093C	6	-	223	1.3	073C
				1			243C	24	1.0	357		263C
		2#8	1#8	1	4#7	2.5*	163C	6	-	364	2.0*	163C
				1			243C	24	1.0	458		263C
	24	2#10	1#10	1	4#9	3.8*	253B	6	-	592	3.1X	273B
				1			303B	24	1.0	682		333B
		2#10	1#10	1	4#10	4.0*	263B	6	-	652	3.3X	273B
				1			303B	24	1.0	737		333B
		2#7	2#7	1	5#6	2.4	N/A	10	-	262	2.0	N/A
				1			403A	40	1.5	641		443A
		2#8	2#8	1	5#8	3.6*	N/A	10	-	395	3.0*	N/A
				1			403A	40	1.5	774		443A
12.5	36	2#10	2#10	1	5#10	5.4*	253B	10	-	811	4.5*	263B
				1			403A	40	1.5	1023		443A

CRSI Handbook Charts for Concrete Joist Band Beams

SPAN 1 SPAN 2

STANDARD ONE-WAY JOISTS (1) MULTIPLE SPANS		30" Forms + 6" Rib @ 36" c.-c. (2) FACTORED USABLE SUPERIMPOSED LOAD (PSF)											f'_c = 4,000 psi f_y = 60,000 psi
		10" Deep Rib + 3.0" Top Slab = 13.0" Total Depth											
TOP BARS	Size @	# 4 12	# 4 12	# 4 9.5	# 5 12	# 5 10	End Span Defl. Coeff. (3)	# 4 12	# 4 11.5	# 4 8.5	# 4 7	# 5 8.5	Int. Span Defl. Coeff. (3)
BOTTOM BARS	# #	# 4 # 4	# 4 # 5	# 5 # 5	# 5 # 6	# 6 # 6		# 3 # 4	# 4 # 4	# 4 # 5	# 5 # 5	# 5 # 6	
Steel (psf)		.58	.69	.86	1.05	1.25		.63	.74	.95	1.18	1.46	
CLEAR SPAN		END SPAN						INTERIOR SPAN					
17'-0"		180	251	309*	317*	328*	1.006	215	301	358*	365*	374*	.619
		0	0	322	404	478*		0	0	404	508	570*	
18'-0"		151	214	278	288*	297*	1.264	182	259	328*	334*	342*	.778
		0	0	0	351	425		0	0	351	444	525*	
19'-0"		127	184	241	263*	271*	1.569	155	224	301*	307*	314*	.966
		0	0	0	306	373		0	0	306	390	481*	
20'-0"		106	157	209	241*	248*	1.926	131	194	268	284*	289*	1.185
		0	0	0	268	328		0	0	0	343	429	
21'-0"		88	135	182	221*	228*	2.342	111	168	235	263*	267*	1.441
		0	0	0	235	290		0	0	0	303	381	
22'-0"		73	115	158	204*	210*	2.820	94	145	207	244*	248*	1.736
		0	0	0	207	256		0	0	0	269	340	
23'-0"		59	98	137	182	194*	3.369	78	126	182	227*	231*	2.073
		0	0	0	0	227		0	0	0	239	303	
24'-0"		48	83	119	160	180*	3.995	65	108	160	212*	215*	2.458
		0	0	0	0	202		0	0	0	0	272	
25'-0"			70	103	141	167*	4.703	53	93	141	189	201*	2.894
			0	0	0	179		0	0	0	0	244	
26'-0"			58	89	124	156*	5.502	43	80	123	168	188*	3.386
			0	0	0	159		0	0	0	0	219	
27'-0"			48	76	108	141	6.398	68	108	150	176*		3.938
			0	0	0	0		0	0	0	197		
28'-0"				65	95	125	7.400		57	95	133	166*	4.554
				0	0	0			0	0	0	177	
29'-0"				54	82	111	8.516		47	82	118	156*	5.240
				0	0	0			0	0	0	159	
30'-0"				45	71	98	9.752			71	105	143	6.001
				0	0	0				0	0	0	

(1) For gross section properties, see Table 8-1.
(2) First load is for standard square joist ends; second load is for special tapered joist ends.
(3) Computation of deflection is not required above horizonal line (thickness ≥ $\ell_n/18.5$ for end spans, $\ell_n/21$ for interior spans).
(4) Exclusive of bridging joists and tapered ends.
*Controlled by shear capacity. +Capacity at elastic deflection = $\ell_n/360$.

PROPERTIES FOR DESIGN (CONCRETE .41 CF/SF) (4)

NEGATIVE MOMENT											
STEEL AREA (SQ. IN.)	.60	.60	.76	.93	1.12		.60	.63	.85	1.03	1.31
STEEL % (UNIFORM)	.73	.73	.92	1.14	1.37		.73	.76	1.03	1.25	1.61
(TAPERED)	.43	.43	.54	.66	.80		.43	.44	.60	.73	.94
EFF. DEPTH, IN.	11.8	11.8	11.8	11.7	11.7		11.8	11.8	11.8	11.8	11.7
– ICR/IGR	.179	.179	.214	.246	.280		.179	.185	.232	.268	.314
POSITIVE MOMENT											
STEEL AREA (SQ. IN.)	.40	.51	.62	.75	.88		.31	.40	.51	.62	.75
STEEL %	.09	.12	.15	.18	.21		.07	.09	.12	.15	.18
EFF. DEPTH, IN.	11.8	11.7	11.7	11.6	11.6		11.8	11.8	11.7	11.7	11.6
+ICR/IGR	.162	.200	.239	.280	.323		.128	.162	.200	.239	.280

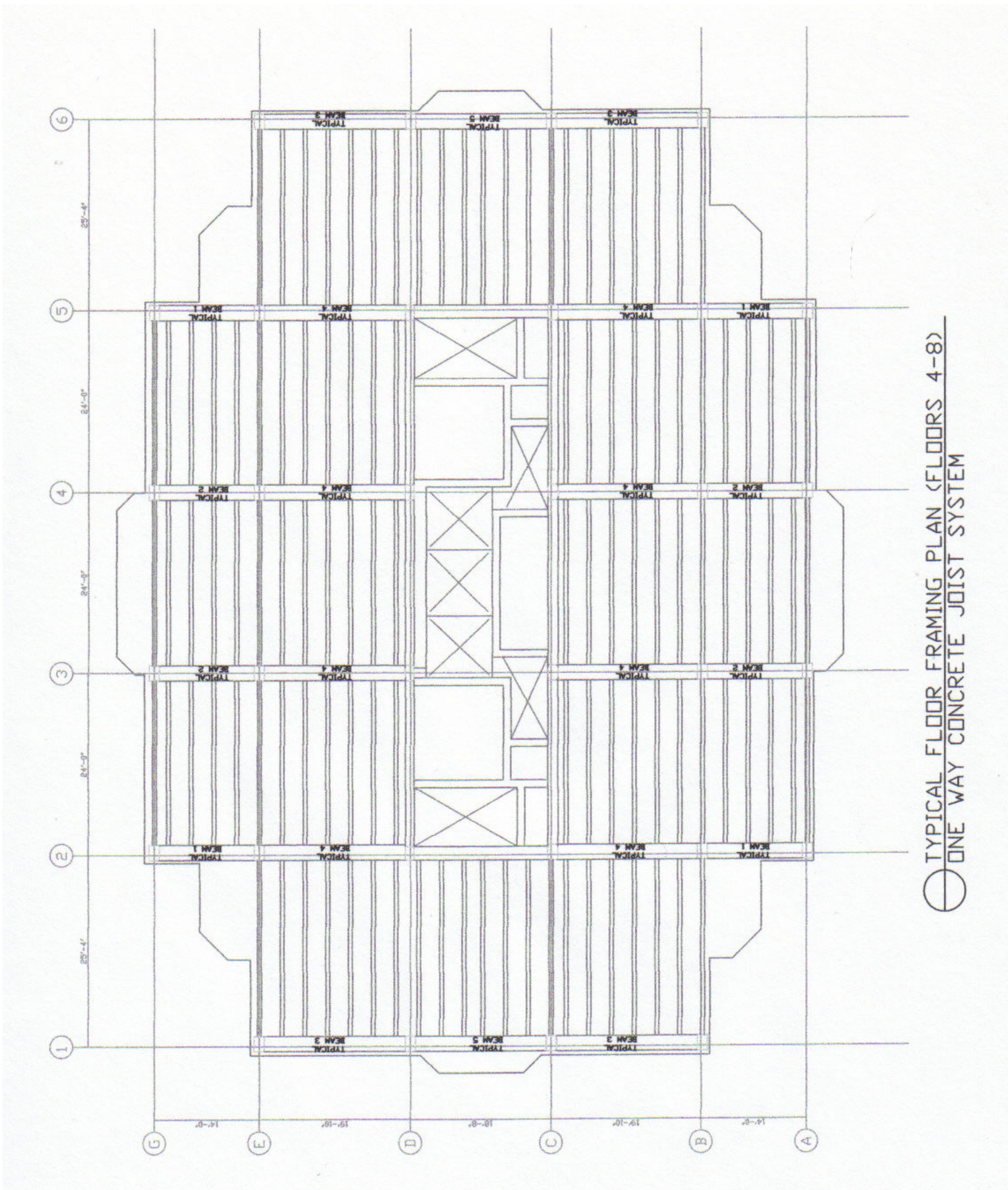
STANDARD (1) ONE-WAY JOISTS SINGLE SPAN	30" Forms = 6" Rib @ 36" c.-c. FACTORED USABLE SUPERIMPOSED LOAD (PSF)(2)									f'_c = 4,000 psi f_y = 60,000 psi
	10" Deep Rib + 3.0" Top Slab = 13.0" Total Depth									
BOTTOM BARS	# 4 # 4	# 4 # 5	# 5 # 5	# 5 # 6	# 6 # 6	# 6 # 7	# 7 # 7	# 7 # 8	# 8 # 8	# 8 # 9	Deflection Coeff. (3)
Steel (psf)	.45	.57	.70	.85	1.00	1.18	1.36	1.57	1.78	2.02	
CLEAR SPAN	FACTORED USABLE SUPERIMPOSED LOAD (PSF)										
16'-0"	133	191	249	316	385	465*	482*				1.214
	0	0	0	0	0	0	548				
17'-0"	108	159	211	270	331	402	445*				1.547
	0	0	0	0	0	0	476				
18'-0"	87	133	179	232	286	350	412*				1.944
	0	0	0	0	0	0	415				
19'-0"	69	110	152	199	248	305	364				2.414
	0	0	0	0	0	0	0				
20'-0"	54	91	129	172	215	267	320				2.964
	0	0	0	0	0	0	0				
21'-0"	41	75	109	148	187	234	282				3.602
	0	0	0	0	0	0	0				
22'-0"		60	91	127	163	206	249	299	340*		4.339
		0	0	0	0	0	0	0	344*		
23'-0"		48	76	109	142	181	221	266	305+		5.183
		0	0	0	0	0	0	0	0		
24'-0"			63	93	123	159	196	237	269+		6.145
			0	0	0	0	0	0	0		
25'-0"			51	79	107	140	174	212	237+	260+	7.235
			0	0	0	0	0	0	0	0	
26'-0"			41	66	92	123	154	189	211+	231+	8.464
			0	0	0	0	0	0	0	0	
27'-0"				55	79	108	137	169	188+	206+	9.844
				0	0	0	0	0	0	0	
28'-0"				45	68	94	121	152	169+	185+	11.385
				0	0	0	0	0	0	0	
29'-0"					57	82	107	135	152+	166+	13.101
					0	0	0	0	0	0	

(1) For gross section properties, see Table 8-1.
(2) First load is for standard square joist ends; second load is for special tapered joist ends.
(3) Computation of deflection is not required above horizonal line (thickness ≥ $\ell_n/16$).
(4) Exclusive of bridging joist and tapered ends.
*Controlled by shear capacity. +Capacity at elastic deflection = $\ell_n/360$.

PROPERTIES FOR DESIGN (CONCRETE .41 CF/SF)(4)

POSITIVE MOMENT										
STEEL AREA (SQ. IN.)	.40	.51	.62	.75	.88	1.04	1.20	1.39	1.58	1.79
STEEL PERCENT	.09	.12	.15	.18	.21	.25	.29	.34	.38	.43
DEPTH, D IN.	11.75	11.69	11.69	11.63	11.63	11.56	11.56	11.50	11.50	11.44
+ICR/IGR	.162	.200	.239	.280	.323	.369	.418	.469	.523	.573

A typical floor plan is shown below. All of the members are the same size, but hold different loads. Their size is 12.5" x 24". Further calculations may be found in the Appendix in Figures 16-19.



4.3 Evaluation of the System

This evaluation will highlight the pros and cons of the One way concrete joist system.

One Way Concrete Joist System Pros

- Easy to construct
- Shallow System
- With a drop ceiling, allows for a large plenum space beneath members
- Inherently fire resistant
- Less dead load

One Way Concrete Joist System Cons

- May take longer to construct
- MEP would have to drill holes or go beneath the members
- Many atypical corners and widths
- Atypical spaces may increase costs

5. Alternative System 3: Two Way Flat Plate System

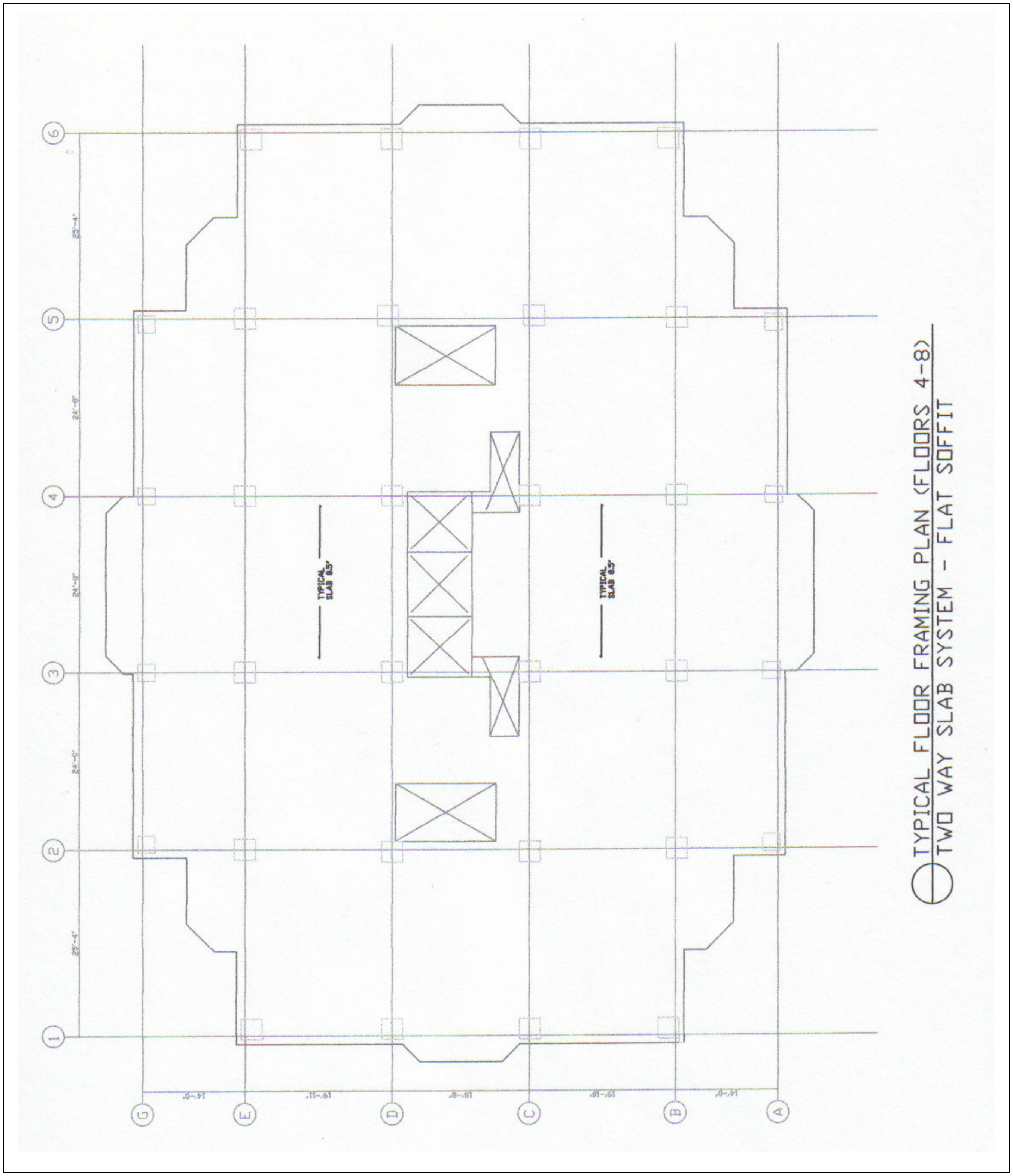
5.1 The System

The two-way flat plate system that I analyzed had no drop panels or beams. I chose strictly a flat slab supported by columns. This was to increase the amount of plenum space and decrease the amount of obstructions for the MEP systems. 'The two-way flat plate is one of the most efficient structural systems for economy.'[3] The formwork for the system is very easy, with little oddities, even in atypical spans and protrusions, as is the case with Vickroy Hall. I used the CRSI Handbook tables for ease of design.

5.2 Analysis

The system I designed ended up being an 8.5" thick slab supported by columns in the range of 23 inches square to 34 inches square. This worked well to stay within the current floor to floor heights and even allowed an increase in the plenum space. A chart with a summary of all of the values from the CRSI Handbook, along with the typical floor plan is shown below. For the values in context, please refer to the Appendix, Figures 3-4. For further calculations, please refer to the Appendix, Figures 20-21.

Panel	Column Lines	Span	Panel Type	Ratio (l_2/l_1)
Typical Panel 1	(2-3,G-E)(4-5,G-E) (2-3,a-B)(4-5, A-B)	14'x24'	C	1.71
Typical Panel 2	(3-4,E-G)(3-4,A-B)	14'x24'	IC	1.71
Typical Panel 3	(1-2,D-E)(5-6, D-E) (1-2,B-C)(5-6, B-C)	26'x19'	C	1.28
Typical Panel 4	(D-E, 2-5)(B-C, 2-5)	24'x20'	IC	1.2
Typical Panel 5	(C-D,1-2)(C-D, 5-6)	26'x19'	C	1.36


TYPICAL FLOOR FRAMING PLAN (FLOORS 4-8)
TWO WAY SLAB SYSTEM - FLAT SOFFIT
TYPICAL SLAB 8.5"
TYPICAL SLAB 8.5"
25'-4"
24'-0"
24'-0"
24'-0"
25'-4"

Panel	Col Size	Slab t	Reinforcement					
			Column Strip			Middle Strip		Steel
			Top Ext +	Bott	Top Int	Bot	Top	(psf)
Typical Panel 1	28”x28”	8.5”	12-#5	11-#6	14-#7	10-#5	12-#4	3.23
Typical Panel 2	23”x23”	8.5”	-	10-#5	13-#7	14-#4	12-#4	3.11
Typical Panel 3	34”x34”	8.5”	15-#5	10-#7	13-#8	12-#5	10-#5	3.70
Typical Panel 4	23”x23”	8.5”	-	10-#5	13-#7	13-#4	12-#4	3.11
Typical Panel 5	34”x34”	8.5”	15-#5	10-#7	13-#8	12-#5	10-#5	3.70

5.3 Evaluation of the System

This evaluation will highlight the pros and cons of the Two-Way flat plate system with no drop panels or beams.

Two-Way Flat Plate System Pros
- Ease of construction
- Allows for large plenum space
- Economical
- Inherently fire resistant
- Shallow system

Two-Way Flat Plate System Cons
- Larger Columns
- Cantilevered sections
- Must design for shear
- Punching shear is typical
- If current column lines are kept, there
- will be eccentricities in the columns

6. Alternative System 4: Hollow Core Planks on Concrete Beams and Masonry Bearing Walls

6.1 The System

Hollow core planking is a type of precast concrete system that can be constructed a multitude of ways. The planks are cast in long lengths and cut to size to accommodate the project. The hollow cores can be filled with grout for added strength if need be. A topping slab may also be added for either structural purposes or strictly leveling. For this system, the precast will be supported by concrete beams and/or masonry bearing walls. The system I analyzed has a two-inch topping for both structural integrity and to make sure the floor is level. The Nitterhouse Concrete Products website provided free specifications and details for their typical planks and coinciding connections.

6.2 Analysis

From the Nitterhouse Concrete Products site, I chose the J952 planking system. The full PDF of the specifications can be found in the Appendix as Figure 2. The planks are four stranded 8” x 4’ wide members. The weight of each plank is 82.5 psf or 330 plf. The strength of the member is 3000 psi when it arrives on site, and the 28-day strength is 5000 psi. The allowable loads are located on the bottom of the PDF from Nitterhouse Concrete Products.

Following the original floor plan, the columns were kept the same and the orientation of the planks followed that of the original beams for simplicity at the cantilevered and protruding sections. There were four typical planks. The planks were assumed to be simply supported with minor tack welds to the supporting members. The sections where planks were not designed for were atypical, such as around the core of the building, which houses the elevator shafts and stairwells. Such analysis was beyond the scope of this report. The typical supporting members were also designed for. However, they were not designed as fixed-fixed members as in the original system, but simply supported.

Other Assumptions:

- Deflection of the planks is calculated into the allowable loads given in the Nitterhouse Concrete Products specifications
- Since there is topping on the planks, the planks will not serve as point loads on the supporting members, but as a distributed load along the length of the member.
- Supporting members are unbraced except for necessary tack welds along the length as defined by the precastor
- Only gravity was accounted for in the floor system

After reviewing the typical architectural floor plan, I determined the places where masonry bearing walls could be placed without ruining the architectural beauty of the building. (see Appendix Figure 5 for Typical Architectural Floor Plan) The reason I chose these spots were because of the large amount of empty space between the columns that were not being used in the current system. Therefore, if it was not being used in the current system, it could be used in the alternative system. The plank layout is the same as the alternative system 1 due to logistics (See framing below for bearing walls and plank layout). For further calculations, please refer to Figure 22 in the Appendix.

6.3 Evaluation of the System

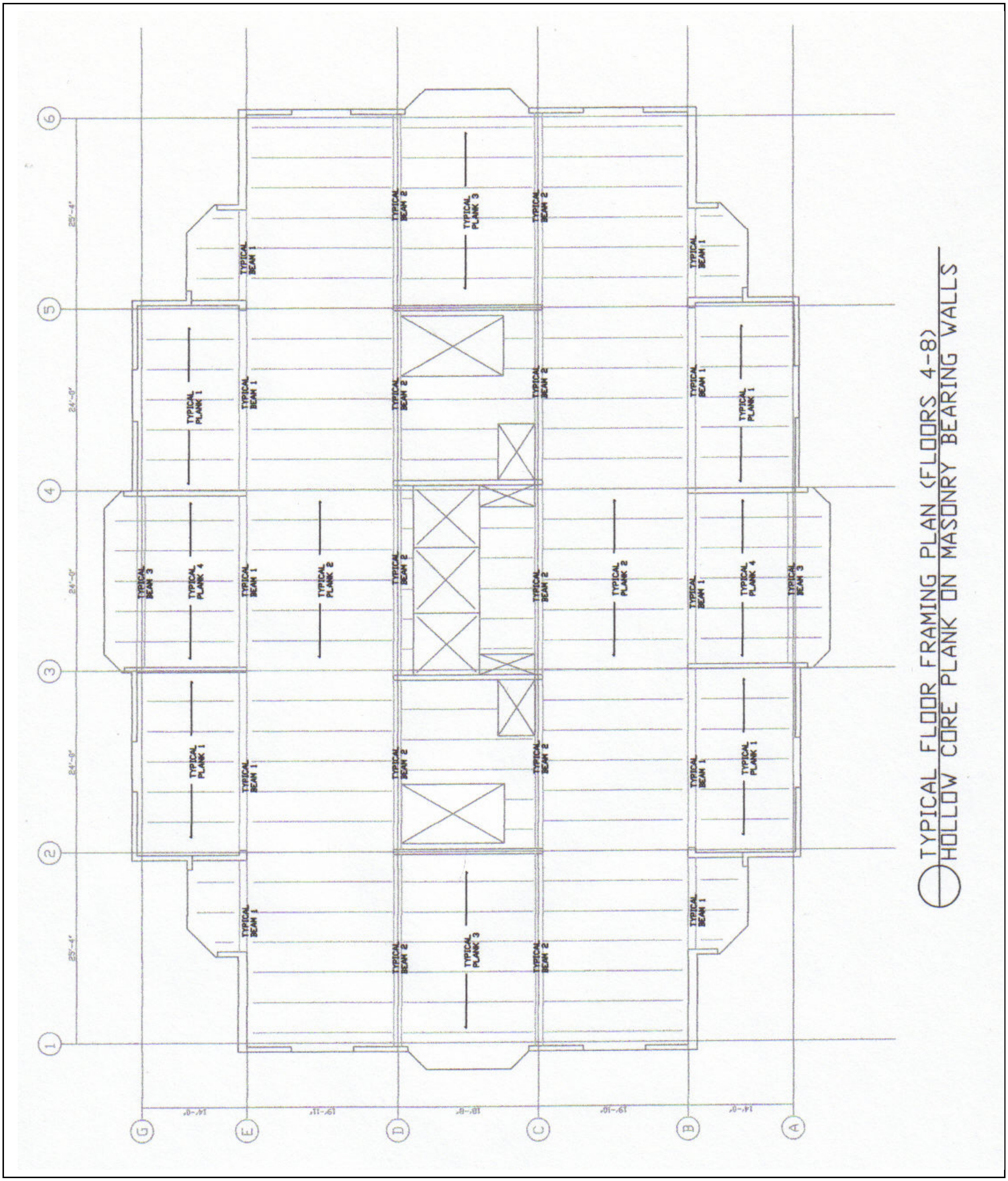
This evaluation will highlight the pros and cons of the precast hollow core plank on structural steel members system.

Hollow Core System Pros

- Durable system[1]
- Inherently Fire Resistant[1]
- Fast Installation[1]
- Noise Attenuation[1]
- Less expensive
- Less empty space between columns
- Bearing walls take the place of columns

Hollow Core System Cons

- Cannot change column spacing due to façade
- Much cutting will be needed to size to the original column spacing as planks come in 4' sections
- While spanning in shorter direction
  - Easier to construct but more pieces
- Cantilevered sections
- Planks must bear on beams which in turn bear on bearing walls


TYPICAL FLOOR FRAMING PLAN (FLOORS 4-8)
HOLLOW CORE PLANK ON MASONRY BEARING WALLS

7. Overall Evaluation

System	Current	Hollow Core on Steel	One-Way Joist	2-Way Flat Plate	Hollow Core on Masonry
Features	o Moderate member sizes o Easy Constructability o Withstood test of time	o Light system o Easy to construct o Fire Resistant	o Light system o Reusable formwork saves money o Fire Resistant	o Easiest to construct o Largest floor to floor height o Fire Resistant	o Less empty spaces o Easy to construct o Fire Resistant
Cost	o Moment frames are expensive	o Atypical spaces may prove to be pricey	o Atypical spaces may prove to be pricey	o Atypical spaces may prove to be pricey	o Atypical spaces may prove to be pricey
Least Depth	Moderate Depth	Largest of 5	Moderate Depth	Least Depth	2nd Largest
Further Evaluation	Yes	Yes	Yes	Yes	Maybe – Placement of bearing walls may become and issue architecturally

Appendix

Figure 1: Typical Framing Plan

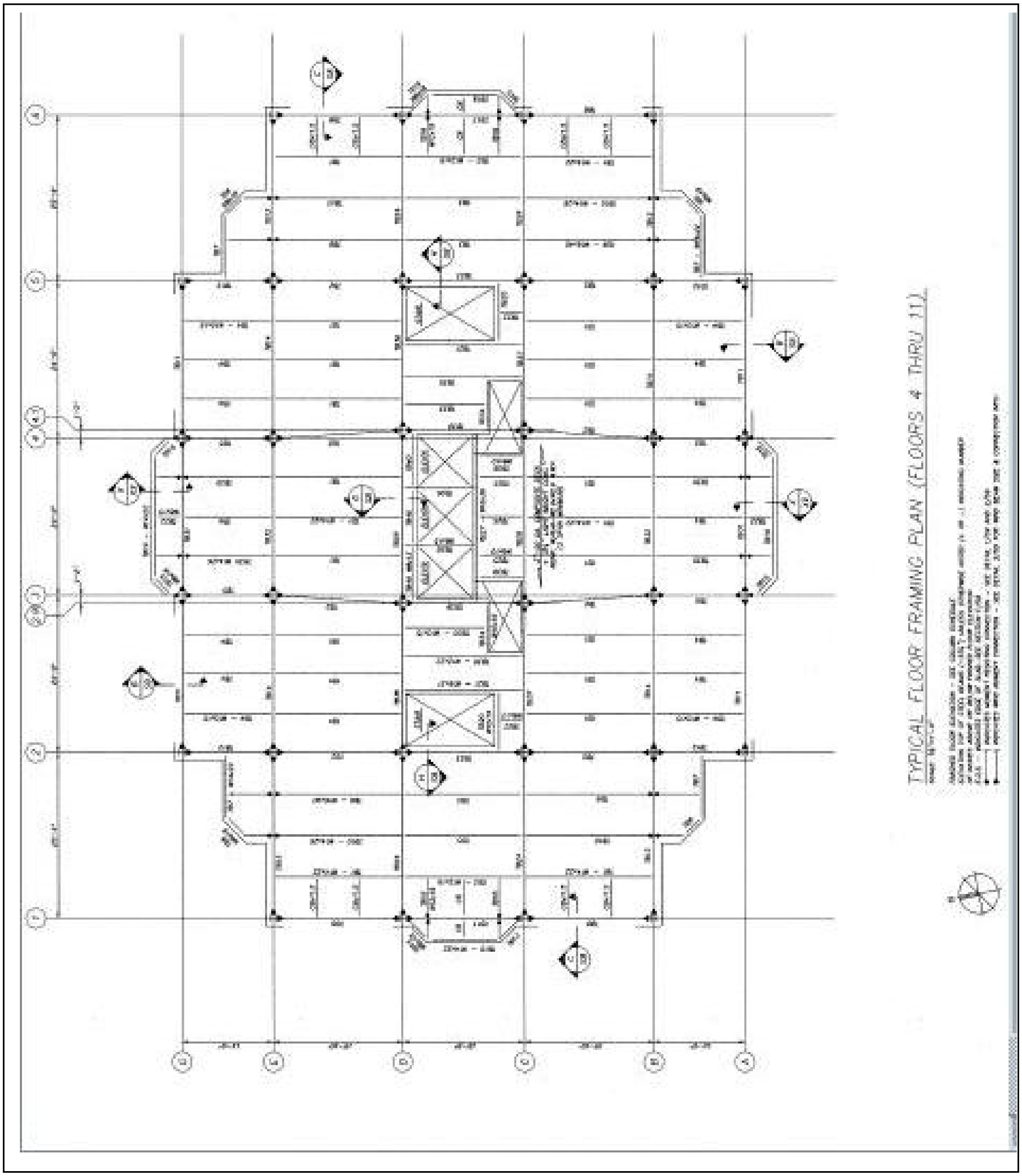


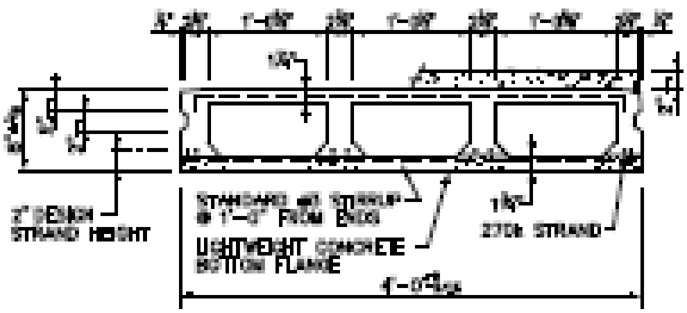
Figure 2: Nitterhouse PDF for J952 Hollow Core Planking

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

PHYSICAL PROPERTIES

Composite

$A' = 285\ in^2$	$S'_b = 468\ in^3$
$I' = 2624\ in^4$	$S'_t = 1096\ in^3$ (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 = 88.7K
   6 – 1/2"Ø, 270K = 124.0K
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 determining 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	336	291	252	218	191	167	148	128	112	98	85	74	63	51	41	31	X
Shear 4 – 1/2"Ø	527	466	421	382	346	317	294	272	252	235	218	197	176	157	140	128	122	110	98	88	78	70	X
Flexure 6 – 1/2"Ø	[illegible]	903	898	764	679	580	502	437	383	336	296	263	232	207	185	165	147	132	116	101	87	74	63
Shear 6 – 1/2"Ø	542	483	434	393	358	329	303	280	261	243	227	212	199	188	176	167	152	137	124	112	101	91	88

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

Figure 3: Flat Plate System from CRSI

FLAT PLATE SYSTEM
(WITHOUT SHEARHEADS)

$f'_c = 4,000$ psi
Grade 60 Bars

SQUARE EDGE PANEL

8.5 in. = TOTAL THICKNESS OF SLAB — 0.708 c.f./s.f.

SPAN c.-c. Cols. $\ell_1 = \ell_2$ (ft)	Factored Superimposed Load (psf)	(1) Min. Square Column (in.)	γ_f	Total Panel Moments −M Ext. (ft-kip)	+M Int. (ft-kip)	−M 1st int. (ft-kip)	Reinforcing Bars, Each Column Strip, Top Ext.	+	Each Column Strip, Bottom	Each Column Strip, Top Int.	Each Middle Strip, Bottom	Each Middle Strip, Top Int.	End Panel Steel (psf), Location of Panel E	EC	C	
22	50	16	0.748	62	124	167	11-# 4	5	8-# 5	14-# 5	11-# 4	11-# 4	2.18	2.19	2.17	
22	100	19	0.737	76	151	204	12-# 4	6	10-# 5	12-# 6	11-# 4	11-# 4	2.41	2.43	2.33	
22	150	23	0.658	88	175	236	14-# 4	4	8-# 6	11-# 7	11-# 4	11-# 4	2.69	2.71	2.68	
22	200	26	0.680	99	199	267	16-# 4	6	9-# 6	12-# 7	13-# 4	11-# 4	2.98	3.00	3.06	
22	250	30	0.612	110	220	296	12-# 5	2	10-# 6	13-# 7	14-# 4	8-# 5	3.32	3.34	3.41	
22	300	34	0.611	118	237	319	19-# 4	4	11-# 6	11-# 8	10-# 5	13-# 4	3.56	3.60	3.70	
22	350	39	0.609	125	250	336	20-# 4	3	16-# 5	12-# 8	16-# 4	9-# 5	3.75	3.78	3.90	
23	50	17	0.752	71	141	190	12-# 4	5	9-# 5	11-# 6	11-# 4	12-# 4	2.22	2.24	2.17	
23	100	21	0.739	86	171	231	14-# 4	7	11-# 5	11-# 7	11-# 4	12-# 4	2.54	2.57	2.54	
23	150	25	0.686	99	199	267	16-# 4	6	9-# 6	12-# 7	13-# 4	12-# 4	2.85	2.87	2.94	
23	200	29	0.675	112	224	302	12-# 5	5	11-# 6	11-# 8	10-# 5	12-# 4	3.31	3.34	3.40	
23	250	33	0.645	123	246	332	13-# 5	4	16-# 5	12-# 8	16-# 4	9-# 5	3.51	3.56	3.68	
23	300	39	0.609	131	262	352	14-# 5	3	9-# 7	13-# 8	11-# 5	14-# 4	3.84	3.89	4.03	
23	350	45	0.608	137	274	369	22-# 4	2	10-# 7	13-# 8	12-# 5	10-# 5	4.08	4.16	4.30	
24	50	19	0.721	80	159	214	13-# 4	5	10-# 5	17-# 5	12-# 4	13-# 4	2.33	2.33	2.24	
24	100	24	0.666	96	192	259	16-# 4	5	12-# 5	15-# 6	13-# 4	13-# 4	2.64	2.67	2.67	
24	150	28	0.657	112	223	300	12-# 5	4	11-# 6	14-# 7	10-# 5	12-# 4	3.13	3.16	3.23	TP1
24	200	32	0.634	126	251	339	20-# 4	6	16-# 5	12-# 8	16-# 4	9-# 5	3.36	3.41	3.53	
24	250	38	0.610	136	271	365	22-# 4	4	10-# 7	13-# 8	12-# 5	10-# 5	3.82	3.88	4.00	
24	300	45	0.608	143	286	385	23-# 4	4	10-# 7	14-# 8	12-# 5	10-# 5	4.02	4.09	4.20	
24	350	51	0.607	150	300	403	16-# 5	2	14-# 6	14-# 8	9-# 6	16-# 4	4.27	4.29	4.48	
25	50	21	0.720	89	178	240	15-# 4	6	17-# 4	14-# 6	12-# 4	13-# 4	2.41	2.41	2.36	
25	100	26	0.699	108	215	290	12-# 5	5	10-# 6	13-# 7	9-# 5	13-# 4	2.85	2.89	2.93	
25	150	30	0.649	125	249	336	13-# 5	4	16-# 5	12-# 8	16-# 4	9-# 5	3.23	3.28	3.35	
25	200	37	0.610	138	276	372	15-# 5	3	10-# 7	13-# 8	12-# 5	10-# 5	3.68	3.73	3.86	
25	250	43	0.609	149	297	400	16-# 5	2	14-# 6	14-# 8	19-# 4	16-# 4	4.03	4.05	4.19	
25	300	50	0.608	157	314	423	12-# 6	2	9-# 8	15-# 8	20-# 4	11-# 5	4.32	4.38	4.54	
25	350	57	0.607	163	326	438	13-# 6	1	9-# 8	16-# 8	10-# 6	12-# 5	4.61	4.67	4.84	
26	50	23	0.728	99	199	268	16-# 4	9	13-# 5	16-# 6	13-# 4	13-# 4	2.51	2.54	2.56	
26	100	28	0.663	120	240	323	19-# 4	7	23-# 4	14-# 7	10-# 5	13-# 4	2.94	2.96	3.10	
26	150	34	0.655	138	277	373	15-# 5	5	10-# 7	13-# 8	12-# 5	10-# 5	3.55	3.58	3.70	TP3 TP5
26	200	41	0.619	151	303	408	16-# 5	5	14-# 6	14-# 8	13-# 5	11-# 5	3.90	3.92	4.07	
26	250	49	0.608	162	324	436	26-# 4	5	15-# 6	16-# 8	10-# 6	12-# 5	4.27	4.31	4.40	
26	300	56	0.607	170	340	457	13-# 6	1	12-# 7	16-# 8	22-# 4	12-# 5	4.51	4.57	4.77	
26	350	64	0.606	176	353	475	19-# 5	1	10-# 8	17-# 8	11-# 6	13-# 5	4.77	4.85	4.99	

SQUARE INTERIOR PANEL

8.5 in. = TOTAL THICKNESS OF SLAB — 0.708 c.f./s.f.

(2) Span c.-c. (ft)	(3) Load (psf	(1) Min. Sq. Col. (in.)	Reinforcing Bars, Column Strip, Top	Column Strip, Bottom	Middle Strip, Top	Middle Strip, Bottom	Steel (psf), Location of Panel I	IE	IC	
22	50	10	13-# 5	11-# 4	11-# 4	11-# 4	2.24	2.25	2.26	
22	100	14	11-# 6	11-# 4	11-# 4	11-# 4	2.42	2.44	2.46	
22	150	19	13-# 6	8-# 5	11-# 4	11-# 4	2.68	2.72	2.76	
22	200	23	11-# 7	13-# 4	11-# 4	11-# 4	2.90	2.93	2.95	
22	250	27	13-# 7	10-# 5	11-# 4	11-# 4	3.28	3.29	3.30	
22	300	33	13-# 7	10-# 5	8-# 5	11-# 4	3.36	3.41	3.45	
22	350	40	11-# 8	16-# 4	13-# 4	11-# 4	3.60	3.65	3.70	
23	50	12	11-# 6	11-# 4	12-# 4	11-# 4	2.33	2.33	2.33	
23	100	16	13-# 6	12-# 4	12-# 4	11-# 4	2.55	2.59	2.63	
23	150	21	11-# 7	13-# 4	12-# 4	11-# 4	2.79	2.81	2.84	
23	200	25	13-# 7	10-# 5	12-# 4	11-# 4	3.16	3.19	3.23	
23	250	31	11-# 8	16-# 4	13-# 4	11-# 4	3.39	3.43	3.48	
23	300	39	12-# 8	11-# 5	13-# 4	12-# 4	3.69	3.74	3.78	
23	350	47	12-# 8	12-# 5	9-# 5	12-# 4	3.84	3.89	3.94	
24	50	13	12-# 6	12-# 4	13-# 4	12-# 4	2.42	2.41	2.41	
24	100	18	20-# 5	13-# 4	13-# 4	12-# 4	2.64	2.66	2.68	
24	150	23	13-# 7	10-# 5	13-# 4	12-# 4	3.08	3.09	3.11	TP2 TP4
24	200	28	14-# 7	11-# 5	13-# 4	11-# 4	3.25	3.30	3.36	
24	250	37	12-# 8	12-# 5	9-# 5	12-# 4	3.62	3.66	3.71	
24	300	46	13-# 8	12-# 5	14-# 4	13-# 4	3.87	3.92	3.96	
24	350	54	13-# 8	19-# 4	10-# 5	13-# 4	3.98	4.02	4.06	
25	50	15	13-# 6	12-# 4	13-# 4	12-# 4	2.40	2.42	2.43	
25	100	20	12-# 7	14-# 4	13-# 4	12-# 4	2.76	2.78	2.81	
25	150	26	14-# 7	11-# 5	13-# 4	12-# 4	3.15	3.20	3.24	
25	200	33	12-# 8	12-# 5	14-# 4	12-# 4	3.44	3.49	3.53	
25	250	43	13-# 8	9-# 6	10-# 5	13-# 4	3.81	3.85	3.89	
25	300	52	14-# 8	20-# 4	16-# 4	9-# 5	4.04	4.09	4.13	
25	350	62	14-# 8	10-# 6	16-# 4	9-# 5	4.22	4.31	4.41	
26	50	17	15-# 6	13-# 4	13-# 4	12-# 4	2.51	2.52	2.54	
26	100	23	14-# 7	16-# 4	13-# 4	12-# 4	2.95	2.95	2.95	
26	150	28	13-# 8	12-# 5	14-# 4	12-# 4	3.41	3.42	3.43	
26	200	39	13-# 8	13-# 5	10-# 5	13-# 4	3.64	3.68	3.72	
26	250	49	14-# 8	10-# 6	16-# 4	14-# 4	3.96	4.04	4.13	
26	300	60	15-# 8	22-# 4	11-# 5	10-# 5	4.24	4.28	4.33	
26	350	69	15-# 8	11-# 6	12-# 5	10-# 5	4.47	4.55	4.64	

Figure 4: Flat Plate System from CRSI

FLAT PLATE SYSTEM (WITHOUT SHEARHEADS)

f'_c = 4,000 psi
Grade 60 Bars

SQUARE EDGE PANEL

8 in. = TOTAL THICKNESS OF SLAB — 0.667 c.f./s.f.

SPAN c.-c. Cols. $\ell_1 = \ell_2$ (ft)	Factored Superimposed Load (psf)	(1) Min. Square Column (in.)	γ_f	Total Panel Moments −M Ext. (ft-kip)	+M Int. (ft-kip)	−M 1st. int. (ft-kip)	Reinforcing Bars: Each Column Strip Top Ext.	+	Each Column Strip Bottom	Each Column Strip Top Int.	Each Middle Strip Bottom	Each Middle Strip Top Int.	End Panel Steel (psf) Location of Panel E	EC	C
16	50	10	0.724	23	45	61	8-#4	1	7-#4	8-#4	7-#4	8-#4	1.75	1.75	1.76
16	100	10	0.814	29	57	77	8-#4	3	7-#4	10-#4	7-#4	8-#4	1.80	1.82	1.81
16	150	12	0.731	34	67	90	8-#4	1	7-#4	8-#5	7-#4	8-#4	1.88	1.90	1.86
16	200	13	0.763	39	77	104	8-#4	2	8-#4	9-#5	7-#4	8-#4	1.99	2.01	2.02
16	250	15	0.720	44	87	117	8-#4	2	6-#5	10-#5	7-#4	8-#4	2.10	2.13	2.12
16	300	17	0.748	48	97	130	9-#4	4	10-#4	8-#6	7-#4	8-#4	2.29	2.31	2.30
16	350	19	0.626	53	106	143	9-#4	1	11-#4	9-#6	8-#4	8-#4	2.44	2.46	2.41
17	50	10	0.773	27	55	74	9-#4	1	8-#4	10-#4	8-#4	9-#4	1.87	1.87	1.86
17	100	11	0.802	34	69	93	9-#4	2	8-#4	12-#4	8-#4	9-#4	1.95	1.96	1.93
17	150	13	0.791	40	81	109	9-#4	3	9-#4	14-#4	8-#4	9-#4	2.05	2.07	2.00
17	200	16	0.717	46	92	124	9-#4	3	10-#4	11-#5	8-#4	9-#4	2.18	2.21	2.19
17	250	18	0.705	52	104	140	9-#4	4	7-#5	12-#5	8-#4	9-#4	2.28	2.30	2.25
17	300	20	0.671	58	116	156	10-#4	3	8-#5	10-#6	8-#4	9-#4	2.47	2.49	2.53
17	350	22	0.659	63	126	170	11-#4	3	13-#4	11-#6	9-#4	9-#4	2.66	2.67	2.71
18	50	10	0.815	33	66	88	9-#4	2	8-#4	12-#4	8-#4	9-#4	1.83	1.83	1.80
18	100	13	0.770	41	82	110	9-#4	3	9-#4	10-#5	8-#4	9-#4	1.96	1.98	1.94
18	150	15	0.763	48	96	129	9-#4	4	10-#4	11-#5	8-#4	9-#4	2.09	2.10	2.09
18	200	18	0.695	55	109	147	10-#4	3	8-#5	13-#5	8-#4	9-#4	2.27	2.29	2.25
18	250	21	0.618	61	123	166	11-#4	2	13-#4	11-#6	9-#4	9-#4	2.47	2.50	2.51
18	300	23	0.615	68	135	182	12-#4	2	14-#4	16-#5	10-#4	9-#4	2.65	2.66	2.68
18	350	25	0.656	73	147	198	13-#4	4	10-#5	10-#7	10-#4	9-#4	2.88	2.91	3.03
19	50	12	0.779	38	77	103	10-#4	2	9-#4	14-#4	9-#4	10-#4	1.96	1.97	1.93
19	100	15	0.738	48	96	129	10-#4	3	10-#4	11-#5	9-#4	10-#4	2.08	2.10	2.09
19	150	18	0.674	56	112	150	10-#4	3	8-#5	13-#5	9-#4	10-#4	2.28	2.29	2.16
19	200	20	0.677	64	128	172	11-#4	3	9-#5	11-#6	9-#4	10-#4	2.48	2.49	2.43
19	250	23	0.675	71	143	192	13-#4	4	10-#5	12-#6	10-#4	10-#4	2.69	2.71	2.73
19	300	26	0.613	78	156	210	14-#4	3	8-#6	10-#7	11-#4	10-#4	2.98	2.98	3.02
19	350	28	0.654	84	169	227	10-#5	3	12-#5	20-#5	8-#5	10-#4	3.20	3.22	3.32
20	50	13	0.783	45	90	121	10-#4	3	10-#4	16-#4	9-#4	10-#4	1.96	1.97	1.92
20	100	17	0.687	55	110	148	10-#4	3	8-#5	13-#5	9-#4	10-#4	2.17	2.18	2.12
TP3 20	150	20	0.651	65	129	174	11-#4	3	9-#5	11-#6	9-#4	10-#4	2.42	2.41	2.33
20	200	23	0.658	74	147	198	13-#4	4	10-#5	10-#7	10-#4	10-#4	2.64	2.66	2.73
20	250	26	0.623	82	164	220	14-#4	4	8-#6	11-#7	8-#5	10-#4	2.98	3.00	3.01
20	300	29	0.611	90	179	241	16-#4	2	9-#6	12-#7	8-#5	11-#4	3.18	3.22	3.33
20	350	33	0.610	95	190	255	11-#5	2	13-#5	10-#8	13-#4	11-#4	3.33	3.40	3.59

SQUARE INTERIOR PANEL

8 in. = TOTAL THICKNESS OF SLAB — 0.667 c.f./s.f.

(2) Span c.-c. (ft)	(3) Load (psf	(1) Min. Sq. Col. (in.)	Reinforcing Bars: Column Strip Top	Column Strip Bottom	Middle Strip Top	Middle Strip Bottom	Steel (psf) Location of Panel I	IE	IC
16	50	10	8-#4	7-#4	8-#4	7-#4	1.76	1.76	1.76
16	100	10	10-#4	7-#4	8-#4	7-#4	1.86	1.86	1.86
16	150	14	11-#4	7-#4	8-#4	7-#4	1.93	1.95	1.97
16	200	16	13-#4	7-#4	8-#4	7-#4	2.04	2.06	2.07
16	250	17	14-#4	7-#4	8-#4	7-#4	2.10	2.12	2.15
16	300	18	16-#4	7-#4	8-#4	7-#4	2.21	2.24	2.27
16	350	19	12-#5	8-#4	8-#4	7-#4	2.43	2.45	2.47
17	50	10	9-#4	8-#4	9-#4	8-#4	1.85	1.87	1.88
17	100	10	12-#4	8-#4	9-#4	8-#4	2.00	2.00	2.00
17	150	14	13-#4	8-#4	9-#4	8-#4	2.06	2.08	2.09
17	200	16	10-#5	8-#4	9-#4	8-#4	2.20	2.22	2.24
17	250	17	17-#4	8-#4	9-#4	8-#4	2.27	2.30	2.32
17	300	18	9-#6	8-#4	9-#4	8-#4	2.44	2.47	2.50
17	350	20	10-#6	9-#4	9-#4	8-#4	2.64	2.67	2.70
18	50	10	11-#4	8-#4	9-#4	8-#4	1.84	1.85	1.86
18	100	10	9-#5	8-#4	9-#4	8-#4	1.98	2.00	2.01
18	150	14	16-#4	8-#4	9-#4	8-#4	2.09	2.10	2.11
18	200	16	12-#5	8-#4	9-#4	8-#4	2.22	2.24	2.26
18	250	17	10-#6	9-#4	9-#4	8-#4	2.47	2.50	2.52
18	300	20	11-#6	10-#4	9-#4	8-#4	2.65	2.65	2.65
18	350	23	9-#7	7-#5	9-#4	8-#4	2.87	2.91	2.94
19	50	10	13-#4	9-#4	10-#4	9-#4	2.00	2.01	2.02
19	100	10	16-#4	9-#4	10-#4	9-#4	2.13	2.14	2.16
19	150	14	13-#5	9-#4	10-#4	9-#4	2.34	2.33	2.33
19	200	17	10-#6	9-#4	10-#4	9-#4	2.45	2.47	2.49
19	250	20	16-#5	10-#4	10-#4	9-#4	2.62	2.64	2.67
19	300	23	10-#7	11-#4	10-#4	9-#4	2.96	2.96	2.96
19	350	27	10-#7	8-#5	10-#4	9-#4	3.06	3.07	3.08
20	50	10	10-#5	9-#4	10-#4	9-#4	2.01	2.01	2.01
20	100	12	12-#5	9-#4	10-#4	9-#4	2.14	2.16	2.17
20	150	16	11-#6	9-#4	10-#4	9-#4	2.41	2.41	2.41 TP9
20	200	19	12-#6	11-#4	10-#4	9-#4	2.63	2.67	2.71
20	250	23	10-#7	8-#5	10-#4	9-#4	2.88	2.91	2.95
20	300	26	20-#5	13-#4	10-#4	9-#4	2.96	3.03	3.09
20	350	33	15-#6	13-#4	10-#4	9-#4	3.11	3.18	3.26

(Continued on next page)

Figure 5: Typical Floor showing Architectural Elements

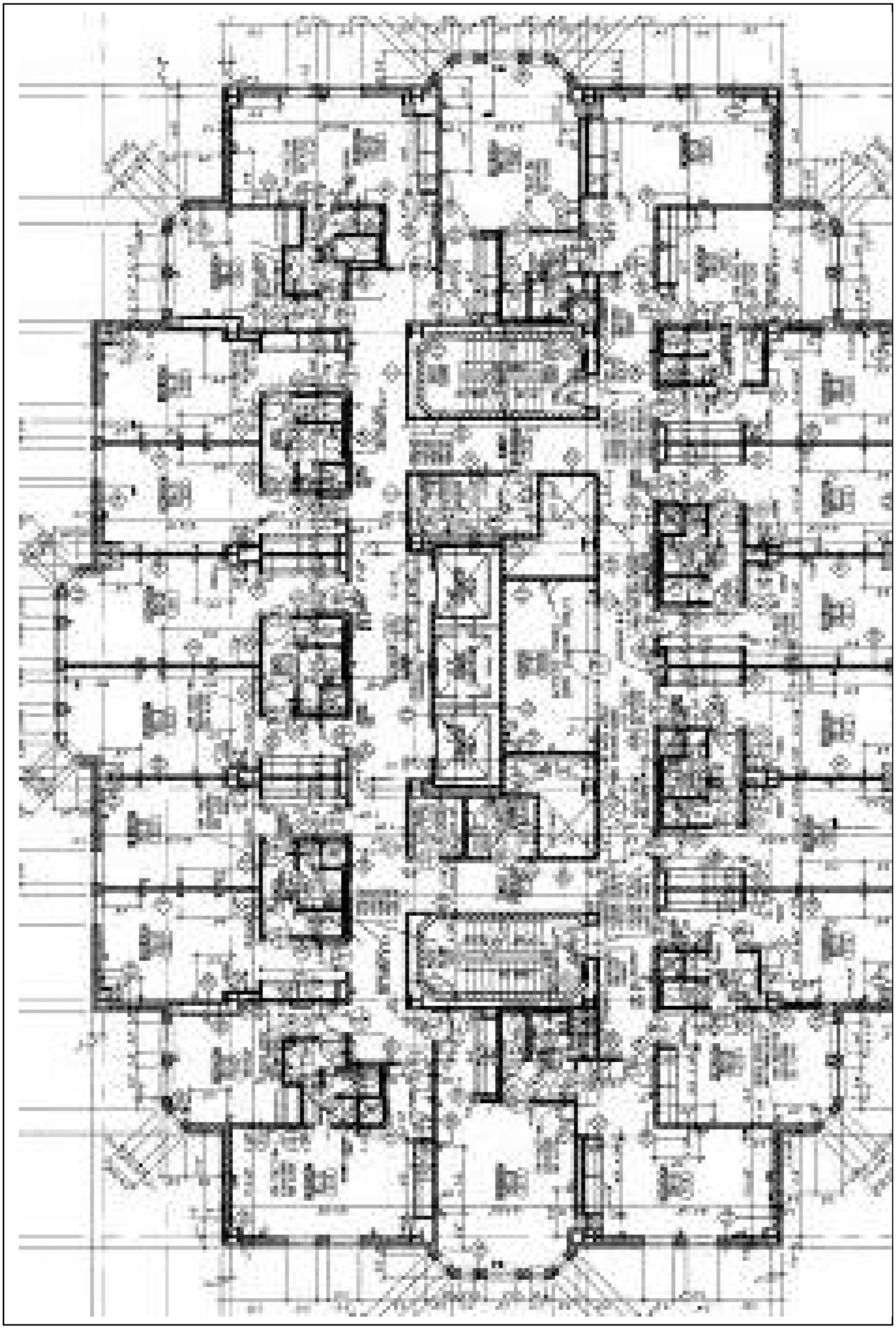


Figure 6: Current System Calculations Page 1

CURRENT SYSTEM | TECHNICAL REPORT 2 | DONNA KENT

Floor loading

Dead Load

deck	2psf
reinforcing	2 psf
lightweight concrete	38 psf = 140 pcf (3.25"/12"/')
floor covering	2psf
Ceiling	2psf
MEP	10 psf
Collateral	5 psf
	61 psf

• partition load not needed

Live load

Dwelling Units 40 psf

• Assume fixed-fixed for girders due to moment frame detail
• Assume simply supported for most beams, fixed for 2 Typical
• Assume gravity loads only
* revision of Technical Report 1
- floor loading
- end conditions for girders

$w_u = 1.2(61) + 1.6(40)$
$w_u = 138\text{ psf}$

Typical Beam 1

Span = 14'-0"
simply supported
Tributary width = 6'

$w_u = 138\text{psf}(6') = 828\text{ plf}$

$M_{max} = \frac{w\ell^2}{8} = 21'^k$
$V_{max} = \frac{w\ell}{2} = 6^k$
$\Delta_{max} = \ell/360 = \frac{5w\ell^4}{384EI}$

$$I_{req} = \frac{360(5)w\ell^3}{384E} = \frac{1800(828)(14)^3(12)^2}{384(29000000)}$$

$I_{req} = 53\text{ in}^4$

use W10x15 $I = 68.9 > 53\text{in}^4$ ∴ ok
$\phi M_p = 60 > 21'^k$ ∴ ok
$\phi V_n = 62 > 6^k$ ∴ ok

Figure 7: Current System Calculations Page 2

CURRENT SYSTEM	TECHNICAL REPORT 2	DONNA KENT	P

Typical Beam 2

Span 14'-0"

fixed-fixed

Tributary width 6'

$W_u = 828$ plf

$M_{max} = \frac{w\ell^2}{12} = 14'^k$

$V_{max} = \frac{w\ell}{2} = 6^k$

$\Delta_{max} = \ell/360 = \frac{w\ell^4}{384EI}$

$I_{req} = \frac{360 w\ell^3}{384E} = \frac{360(828)(14)^3(12)^2}{384(29000000)}$

$I_{req} = 11 \text{ in}^4$

use W10x15 $\quad I = 68.9\text{in}^4 > 11\text{in}^4 \therefore ok$

$\phi M_p = 60.0'^k > 14'^k \therefore ok$

$\phi V_n = 62^k > 6^k \therefore ok$

Typical Beam 3

Span 19'-10"

fixed-fixed

TW = 6'

$W_u = 828$ plf

$M_{max} = 27'^k$

$V_{max} = 8^k$

Δ_{max}:

$I_{req} = 31 \text{ in}^4$

use W10x15 $\quad I = 68.9\text{in}^4 > 31\text{in}^4 \therefore ok$

$\phi M_p = 60'^k > 27'^k \therefore ok$

$\phi V_n = 62^k > 8^k \therefore ok$

Typical Beam 4

Span 19'-10"

simply supported

TW = 6'

$W_u = 828$ plf

$M_{max} = 41'^k$

$V_{max} = 8^k$

$\Delta_{max} =$

$I_{req} = \frac{1800 w\ell^3}{384E} = 150 \text{ in}^4$

use W12x22 $\quad I = 156\text{in}^4 > 150\text{in}^4 \therefore ok$

$\phi M_p = 110'^k > 41'^k \therefore ok$

$\phi V_n = 86.3^k > 8^k \therefore ok$

Figure 8: Current System Calculations Page 3

CURRENT SYSTEM | TECHNICAL REPORT 2 | DONNA KENT

Typical Beam 5

Span: 18'8"

Simply Supported

$TW = 6'$

$W_u = 828$ plf

$M_{max} = 36'^k$

$V_{max} = 8^k$

Δmax:

$I_{req} = 126 in^4$

use W12x19 $\quad I = 130 in^4 > 126 in^4 \therefore ok$

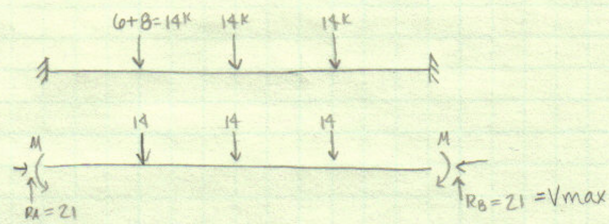
$\phi M_p = 92.6'^k > 36'^k \quad \therefore ok$

$\phi V_n = 77.4^k > 8^k \quad \therefore ok$

Typical Girder 1

Span: 24'-0"

fixed-fixed



by Superposition: (maximums at center)

$$M_{max} = \frac{Pl}{8} + \left(R_1 x - \frac{Pab^2}{l^2}\right)2 \qquad a = 18' \quad b = 6' \quad x = 12'$$

$$= \frac{14^k(24)}{8} + 2\left(21(12) - \frac{14(18')(6')^2}{24^2}\right)$$

$$M_{max} = 514'^k$$

$$\Delta max = \frac{Pl^3}{192EI} + 2\left(\frac{Pb^2x^2}{6EIl^3}(3al - 3ax - bx)\right)$$

$$\frac{(12)24}{360} = \frac{14(24)^3(12^3)}{192(29000I)} + 2\left(\frac{14(6')^2(12)^2(12)}{6(29000)\ I(24)^3}\left(3(18)(24)(12^2) - 3(18)(12)(12^2) - (6)(12)(12)^2\right)\right)$$

$$0.8 = \frac{60.06}{I} + 2\left(\frac{3.62e\text{-}4}{I}(82944)\right)$$

$$0.8I = 120.1$$

$$I_{req} = 150 in^4$$

use W18x71 $\quad I = 1170 in^4 > 150 in^4 \quad \therefore ok$

$\phi M_p = 548'^k > 514'^k \quad \therefore ok$

$\phi V_n = 247^k > 21^k \quad \therefore ok$

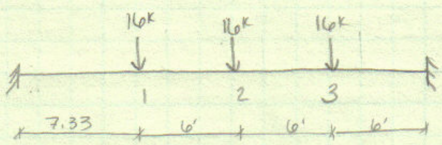
Figure 9: Current System Calculations Page 4

CURRENT SYSTEM | TECHNICAL REPORT 2 | DONNA KENT

Typical Girder 2

Span 25'-4"

fixed - fixed



$V_{max} = 24^{k}$

By superposition (maximums at middle point load ~ assumption)

$$M_{max} = \frac{2Pa_2^2 b_2^2}{l^3} + \left(R_1 x - \frac{Pa_1 b_1^2}{l^2}\right) + \frac{R_1 x - Pa_3 b_3^2}{l^2}$$

$$= \frac{2(16)(13.33)^2(12)^2}{25.33^3} + 24(13.33) - \frac{16(18)(7.33)^2}{25.33^2} + 24(12) - \frac{16(19.33)(6)^2}{25.33^2}$$

$M_{max} = 617^{'k}$

$$\Delta_{max} = L/360 = \frac{Pa_2^3 b_2^3}{3EIl^3} + \frac{Pb_1^2 x^2}{6EIl^3}(3a_1 l - 3a_1 x - b_1 x) + \frac{Pb_3^2 x^2}{6EIl^3}(3a_3 l - 3a_3 x - b_3 x)$$

$$\frac{25.33(12)}{360} = \frac{16(13.33)^3(12)^3(12)^3}{3(29000)I(25.33)^3} + \frac{16(7.33)^2(13.33)^2(12)}{6(29000)I(25.33)^3}\left(3(18)(25.33)(12)^2 - 3(18)(13.33)(12^2) - (7.33)(13.33)(12)\right)$$

$$+ \frac{16(6)^2(12)^2(12)}{6(29000)I(25.33)^3}\left(3(19.33)(25.33)(12^2) - 3(19.33)(12)(12^2) - 6(12)(12^2)\right)$$

$$0.84 = \frac{0.046}{I} + \frac{51.37}{I} + \frac{35.53}{I}$$

$0.84I = 86.95$

$I = 104\ in^4$

use W21x73 $\quad I = 1600 > 104\ in^4 \quad \therefore$ ok

$\phi M_p = 648^{'k} > 617^{'k} \quad \therefore$ ok

$\phi V_n = 260^{k} > 24^{k} \quad \therefore$ ok

SUMMARY	Size
Typical Beam 1	W10x15
2	W10x15
3	W10x15
4	W12x22
5	W12x19
Girder 1	W18x71
2	W21x73

Figure 10: Hollow Core On Steel Support Calculations Page 1

HOLLOW-CORE PLANKING ON STEEL	TECHNICAL REPORT 2	DONNA KENT

wght: 330 plf or 82.5 psf

4 typical Plank sizes
- not including around elevator shafts
- 3 simply supported : spans → 14'-0", 19'-10", 18'-8"
- 1 with edge cantilever : span: 14'-0" with 4' cantilever

4' widths, 8" depth, 2" CIP Concrete Topping

- See Nitterhouse PDF in Appendix
- assuming deflection is within limits on specifications

Plank 1:

Span 14'-0" using J952

Weight ≠ 4620 lbs

Max deflection: $\frac{5wl^4}{384EI}$

I from Nitterhouse = 2624 in^4

Strength = 5000 psi

$E_c = 33w_c^{1.5}\sqrt{f'c}$ where $w_c = 145$ psf

$E_c = 4.07e6$ psi

$E_c = 4.07e3$ ksi

$\Delta = [5(148)(14)^4(12)^3]/[384(4.07e6)(2624)]$

$= 0.012"$

$\Delta = L/360$

$= 14(12)/360$

$= 0.47" > 0.012"$ ∴ ok

Allowable superimposed load from Nitterhouse (Assuming 4 strand Pattern)
- Flexure = 462 psf
- Shear = 382 psf

Load (from Technical Report 1)

$TL = 1.2D + 1.6L = 1.2(70psf) + 1.6(40psf)$

$TL = 148$ psf

- both allowable loads are greater than imposed, therefore ok

Figure 11: Hollow Core Planks on Steel Supports Page 2

HOLLOW-CORE PLANKING | TECHNICAL REPORT 2 | DONNA KENT

Plank 2:

Span: 19'-10" using J952 w_p = 330 plf 8" x4' 4 strand

weight = 6545 lbs

E = 4.07e6 psi
I = 2624 in^4 from J917 Nitterhouse PDF

Max Deflection:

Δ_{max} = 0.048

$\Delta_{allowable}$ = L/360
= 19'-10"/360
$\Delta_{allowable}$ = 0.056

$\Delta_{max} < \Delta_{allowable}$ therefore, ok

Allowable loads at 20' with 4 strand pattern
- Flexure: 191 psf
- Shear: 219 psf

$>$ 148 psf therefore ok

Plank 3:

Span: 18'-8" using J952 8"x4' 4 strand

weight = 6160 lbs

E = 4.07e6 psi
I = 2624 in^4

Max Deflection:

Δ_{max} = 0.037"
$\Delta_{allowable}$ = (18+8/12)/360
= 0.052"

$\Delta_{max} < \Delta_{allowable}$ ∴ ok

Allowable Superimposed loads at 19' assuming 4 strand pattern
- Flexure: 218 psf
- Shear: 235 psf

$>$ 148 psf therefore ok

Figure 12: Hollow Core Planks on Steel Supports Page 3

HOLLOW-CORE PLANKING	TECHNICAL REPORT 2	DONNA KENT	P3

Plank 4

Span: 14'-0" w/ 4' cantilever
8"x4' J952 4 strand

$E = 4.07e6$ psi
$I = 2624\ in^4$

Max Deflection

Noncantilever: $\Delta = \frac{wx}{24EI\ell}(\ell^4 - 2\ell^2x^2 + \ell x^3 - 2a^2\ell^2 + 2a^2x^2)$

$$= \frac{148(7')((14\cdot 12)^4 - 2(14)^2(7)^2(12^4) + 12^4(14)(7)^3 - 2(4)^2(14)^2(12)^4 + 2(4)^2(7)^2(12)^4)}{24(4.07e6)(2624)(14)(12)}$$

$\Delta = 0.0096"$

Cantilever: $\Delta = \frac{wx_1}{24EI}(4a^2\ell - \ell^3 + 6a^2x_1 - 4ax_1^2 + x_1^3)$

$$= \frac{148(4)}{24(4.07e6)(2624)}[4(4)^2(14)(12^3) - 12^3(14)^3 - 6(4)^2(4)(12^3) - 4(4)^3(12^3) + 4^3(12^3)]$$

$\Delta = -0.0097"$

$\Delta_{allowable}$:

Allowable loads ok ⇒ refer to plank 1

Typical Beam 1:

- beam supporting abuttment of Plank 1 and 2
- since topping will be placed on planks, the planks will not be point loads, but instead, distributed evenly along beams

Plank 1: self weight = 4620 lbs ⇒ shear = 2310 lbs ⇒ w = 578 plf
imposed load = 148 psf (14')/2 ⇒ w = 1036 plf

Plank 2: self: 6545 lbs ⇒ shear = 3273 lbs ⇒ w = 819 plf
imposed: 148 (19+10/12)/2 ⇒ 1468 plf

Total load on beam: 578+1036+819+1468 plf
TL = 3901 plf

Span: 24'-0 (unbraced length)
∴ assuming simply supported

$M_{max} = w\ell^2/8$
$= 280872$ lbs
$\Rightarrow 281'^k$

$V_{max} = w\ell/2$

Figure 13: Hollow Core Planks on Steel Supports Page 4

HOLLOW-CORE PLANKING | TECHNICAL REPORT 2 | DONNA KENT

$\Delta_{allowable} = L/360$

$I_{required} = \dfrac{360(5w\ell^3)}{384E} = \dfrac{1800(3901\ (24)^3(12^2))}{384\ (29000000\text{psi})}$

$I = 1255\ in^4$

Try W16x89 $\quad I = 1310\ in^4$

$\phi M_p = 660'^k > 281'^k \therefore ok$

$\phi V_n = 238^k > 47^k \therefore ok$

Typical Beam 2:

Plank 2 to Plank 3 abuttment support

Plank 2: 2287 plf

Plank 3: self weight: 6160 lb ⇒ shear = 3080 lbs ⇒ w = 770 plf

imposed: 148 (18 + 8/12)/2 ⇒ w = 1382 plf

Total load = 4439 plf

Span (unbraced length): 25'-4" ⇒ 25.33' ≈ 26'

$M_{max} = 375'^k$

$V_{max} = 58^k$

$I_{required} = 1800w\ell^3/384E = \dfrac{1800(4439)(26)^3(12^2)}{384(29000000)}$

$I = 1816\ in^4$

Try W21x83 $\quad I = 1830\ in^4 > I_{req}$

$\phi M_p = 735'^k > 375'^k \therefore ok$

$\phi V_n = 298^k > 58 \therefore ok$

Typical Beam 3:

Plank 4 Cantilever support

*will be holding shear from Plank 1 and 4' cantilever

4' weight: 4'(82.5 psf) = 330 plf

imposed: 148 (4') = 592 plf

Plank 1: 1619 plf

Total Load = 1619 plf

Unbraced length = 14'

Figure 14: Hollow Core Planks on Steel Supports Page 5

HOLLOW-CORE PLANKING	TECHNICAL REPORT 2	DONNA KENT	P5

$$I_{required} = \frac{360(5wl^3)}{384E} = \frac{360(5)(1614)(14')^3(12)^2}{384(29000000)}$$

$I = 103 \text{ in}^4$

Try W10x22 $I = 118 \text{ in}^4 > I_{req}$ ∴ ok
$\phi M_p = 97.5'^k > 40'^k$ ∴ ok
$\phi V_n = 66.1^k > 15'^k$ ∴ ok

Deepest System:

	d_b	d_{plank}	$d_{topping}$	total
W16x89	16.8	8"	2"	26.8"
W21x83	21.4	8"	2"	31.4"

Current:

	d_b	$d_{corrugation}$	$d_{concrete}$	total
W14x22	13.7"	2"	3.25"	18.95"
W14x26	13.9"	2"	3.25"	19.15"
W16x40	16.0"	2"	3.25"	21.25"
W16x31	15.9"	2"	3.25"	21.15"

Floor to Floor : 11'-4"

- would lose at most 10" floor to ceiling
- gain 10" for plenum

Girders

Typical Girder 1:
Plank1 Area / Plank 4 Area

Tributary Area:
12' for Plank 1
12' for Plank 4 (cantilever goes straight to column)

Loading:
Planks: 102.5 psf + 148 psf = 250.5 psf
x 24' = 6012 plf

Span 14'-0"
- assume simply supported

$M_{max} = 147'^k$
$V_{max} = 42^k$

Figure 15: Hollow Core Planks on Steel Supports Page 6

Max Deflection (I required)

$$I = \frac{360(5w\ell^3)}{384E} = \frac{360(5)(6012)(14)^3(12)^2}{384(29000000)}$$

$I_{req} = 384\ in^4$

Try W14x38 $I = 385\ in^4$

$\phi M_p = 229'^k > 147'^k \therefore ok$

$\phi V_n = 118^k > 42^k \therefore ok$

Typical Girder 2

Plank 2 Area

· same loading: 6012 plf

Span: 19'-10"

$M_{max} = 296'^k$

$V_{max} = 60^k$

Δmax (I_{req})

$I - 1092\ in^4$

use W16x77 $I = 1120\ in^4$

$\phi M_p = 566'^k > 296'^k \therefore ok$

$\phi V_n = 203^k > 60 \therefore ok$

Typical Girder 3

Plank 3 Area

· same loading: 6012 plf

· Span: 18' 8"

$M_{max} = 262'^k$

$V_{max} = 56^k$

Δmax:

$I_{req} = 910\ in^4$

use W12x106 $I = 933\ in^4 > \ \ in^4 \therefore ok$

$\phi M_p = 615'^k > 262'^k$

$\phi V_n = 212^k > 56^k \therefore ok$

Figure 16: One-Way Concrete Joist System Page 1

ONE-WAY CONCRETE JOIST | TECHNICAL REPORT 2 | DONNA KENT

Openings
- will need header joists near elevators and stair wells
- joists near to openings will need more reinforcing

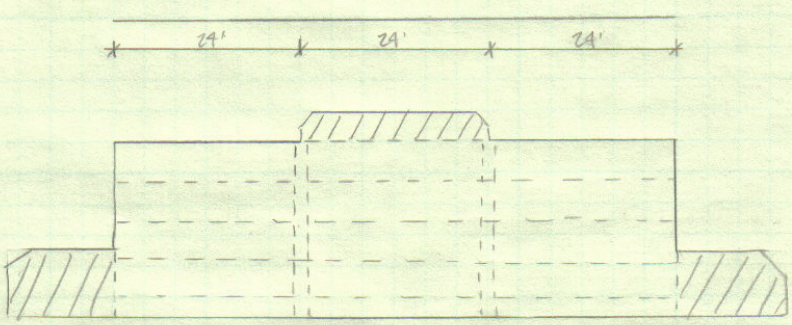
Deflection
- all load capacities have been investigated for deflection

Unequal Continuous Spans
- cause differences in moments
  - limitations
    - Live Load ≤ 3 Dead Load
      - check $40 \leq 3(61)$ ✓
    - larger span to adjacent span shall not be greater than 20%
      - check span in short direction
        14' to 19'10"
        ↳ 42% larger, therefore must span in long direction (lettered columns)

Material Strengths
- $f'c = 4000$ psi
- $fy = 60000$ psi

Loadings
- CRSI factors loads such that: $1.4D + 1.7L$
  - this will be conservative to IBC 2003
- Analyzing $w_u = 138$ psf

Span 1: Between Column lines G & E



- shaded areas disregarded in this report (for simplified check)

From Page 8-21
use 30" + 6" rib   10" deep rib + 3.0" top slab ⇒ 13" total depth
End Span
use #5 at 12" on top
1#5 + 1#6 per rib on bottom
weight of bars: 1.05 psf
capacity = 160 psf
Top bars at exterior edge $A_s = \frac{1}{3}(0.31 + 0.44) / 3' = 0.083\ in^2/ft$
use #4 at 12" with 90° end hooks

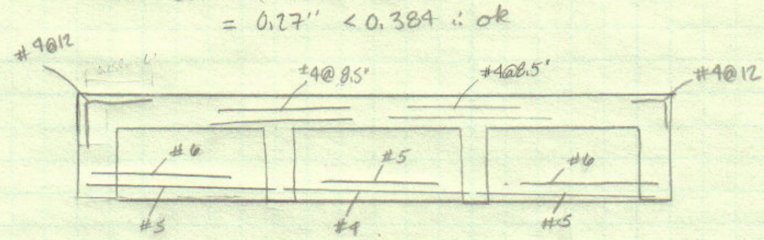
Figure 17: One-Way Concrete Joist System Page 2

ONE-WAY CONCRETE JOIST | TECHNICAL REPORT 2 | DONNA KENT | P2

Interior Spans
use #4 at 8.5" on top
1#4 & 1#5 in rib
weight = 0.95 psf
capacity = 160 psf

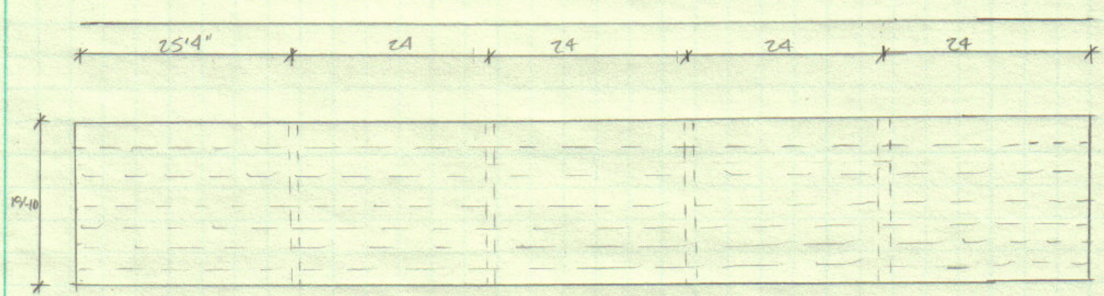
Deflection $\left(\frac{LL}{TL/1.6}\right)\left(\frac{\ell n}{480}\right) = \left(\frac{40}{138/1.6}\right)\left(\frac{24(12)}{480}\right) < \frac{\ell n}{750}$

$= 0.27'' < 0.384 \therefore ok$



SECTION THRU RIB

Span 2: Between D&E Column Lines



From Page 8-21
use 30"+6" rib 10" deep rib + 3.0" top slab ⇒ 13" total depth
ext. #5 @ 12" on top
2#6 in rib
weight of bars = 1.25 psf
capacity through interpolation: 163 psf

Top bar at exterior edge $A_s = 1/3(2)(0.44)/36' = 0.10\ in^2/ft$
use #4 at 12" with 90° end hooks

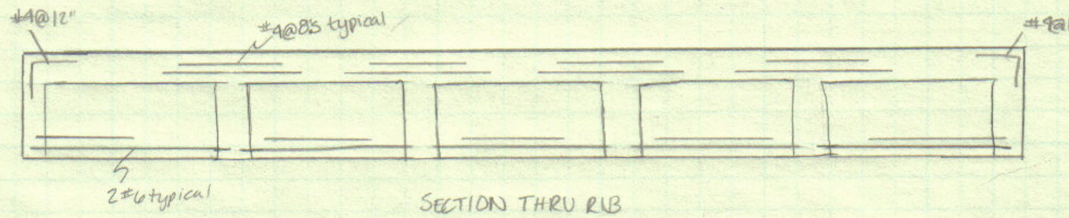
Interior Spans
(same as span 1)

Deflection
Interior (same as span 1)
Exterior $\left(\frac{40}{138/1.6}\right)\left(\frac{25.33(12)}{480}\right) < \frac{25.33(12)}{750}$

$0.29'' < 0.41'' \therefore ok$

Figure 18: One-Way Concrete Joist System Page 3

ONE-WAY CONCRETE JOIST | TECHNICAL REPORT 2 | DONNA KENT



SECTION THRU RIB

Span 3: Between Column Lines C&D

Single span due to building core : 25'4"

From page 8-42

use 30" +6" rib 10" deep rib + 3.0" top slab ⇒ 13" total depth (for uniformity)

#7 in rib

weight of bars = 1.36

capacity through interpolation: 167.4

Top bar at exterior edge:

$A_s = 1/3(0.60)/3 = 0.07$

use #3 @ 12" with 90° hooks

Deflection

$$\left(\frac{40}{138/1.6}\right)\left(\frac{25.33(12)}{480}\right) < \frac{25.33(12)}{750}$$

$0.29" < 0.41"$ ∴ ok

BEAM DESIGN: CONCRETE

From Table 8-1 concrete weight = 61 psf

• CRSI BEAMS

- $f'c$ = 4,000 psi
- f_y = 60000 psi
- deflections taken into account

• assume all beams simply supported due to casting

Typical Beam 1: Exterior Beam Between Columns G+E, A+B

Span: 14'-0

Tributary Width = 12'

w_u = 732 plf

M_{max} = 18'K

V_{max} = 5K

From page 12-80 h = 12.5" b = 24" will hold 3.2 k/ft ∴ ok

Figure 19: One-Way Concrete Joist System Page 4

ONE-WAY CONCRETE JOIST | TECHNICAL REPORT 2 | DONNA KENT

Typical Beam 2: Interior Beam Between column lines G+E, A+B

Span = 14'-0"

Tributary width = 24'

Wu = 1464 plf

Mmax = 36'K

Vmax = 10K

From Page 12-94 h = 12.5" b = 24" will hold 4.2 K/ft ∴ ok

Typical Beam 3: Exterior Beam between Column lines D+E, B+C

Span = 19'-10"

Tributary Width = 12.17'

Wu = 743 plf

Mmax = 37'K

Vmax = 8K

From page 12-81 w/ ℓ_n = 20' h = 12.5" b = 24" will hold 1.6 K/ft ∴ ok

Typical Beam 4: Interior Beam between Column lines D+E, B+C

Span = 19'-10"

Tributary Width = 24.17'

Wu = 1474 plf

Mmax = 73'K

Vmax = 15K

From Page 12-95 h = 12.5" b = 24" will hold 2.1 K/ft ∴ ok

-from 20' span

Typical Beam 5: Exterior Beam between Column lines C+D

Span = 18'-8"

Tributary width = 12.17'

Wu = 743 plf

Mmax = 33'K

Vmax = 7K

From Page 12-81 w/ ℓ_n = 20' h = 12.5' b = 24" will hold 1.6 K/ft ∴ ok

Figure 20: Two-Way Flat Plate System Page 1

CRSI

- Material Strengths
  - $f'_c = 4000$ psi
    (Normal Weight Concrete)
  - $f_y = 60{,}000$ psi
- Design Factors
  - $1.4D + 1.7L$
    - conservative for IBC 2003
- Requirements Satisfyed
  - minimum thickness
  - deflection
  - crack control
- Limitations
  - 3 spans continuous in each direction ✓
  - Ratio lengths to width not exceeding 2.0 ✓
  - Successive spans not to differ more than 1/3 the span of longer ✓
  - Individual columns not to be offset more than 10% of span ✓
  - Live load ≤ 2 Dead Load
- Assumptions for non-square
  - when ℓ_2/ℓ_1 is close to 1.0, use values for larger span
  - no shear heads required

Design

$w_u = 138$ psf as determined from current system Analysis

Typical Panel 1: Column lines (2-3, G-E), (4-5, G-E), (2-3, A-B), (4-5, A-B)
Span 14'-0" x 24'-0"
$\ell_2/\ell_1 = 1.71$
will use larger values due to large ratio
- Square Edge Panel C
- minimum square column to hold 28"x28", 8.5" slab

Typical Panel 2: Column lines (3-4, E-G), (3-4, A-B)
Span 14'-0" x 24'-0"
Interior panel, IC
$\ell_2/\ell_1 = 1.71$
- use larger values due to large ratio
- Minimum square column to hold 23" x 23", 8.5" slab

Typical Panel 3: Column lines (1-2, DE), (5-6, DE), (1-2, BC), (5-6, BC)
Span 25'-4" x 19'-10"
Edge Panel, C
$\ell_2/\ell_1 = 1.28$
→ compare reinforcing due to closeness to 1.0
use 26 x 20, 8.5" slab

Figure 21: Two-Way Flat Plate System Page 2

2-WAY FLAT PLATE | TECHNICAL REPORT 2 | DONNA KENT | P2

	Col Strip			Middle		Steel
	Top Ext	Bott.	Top int	Bott.	Top int	C
20'	11-#4 3	9#5	11-#6	9-#4	10-#4	2.33
26'	15-#5 5	10#7	13-#8	12-#5	10-#5	3.70

- cannot really cut back on steel, so use 26' reinforcing
- minimum square to hold: 34"x34"

Typical Panel 4: Between Column lines (DE, 2-5) + (BC, 2-5)

Span: 24' x 19'-10" → 20'

$l_2/l_1 = 1.2$

Interior Panel, IC

compare reinforcing

	Col Strip		Middle		Steel
	Top	Bot	Top	Bot	IC
20'	11-#6	9-#4	10-#4	9-#4	2.41
24'	13-#7	10-#5	13#4	12-#4	3.11

use 24' reinforcing

- minimum square column to hold: 23" x 23", slab = 8.5"

Typical Panel 5: Between Column lines (CD, 1-2) and (CD, 5-6)

Span: 25'-4" x 18'-8" ⇒ 26 x 19

$l_2/l_1 = 1.36$ ⇒ use larger reinforcement,

Exterior Panel, C

- minimum square column to hold: 34" x 34", slab = 8.5"

Figure 22: Hollow Core Planking On Masonry Bearing Walls and Concrete Beams

HOLLOW CORE PLANKING ON MASONRY + BEAMS	TECHNICAL REPORT 2	DONNA KENT	P1

* use plank design from other system

BEAM DESIGN
- using designs from other system for loading

Typical Beam 1
Span 24'
Total Load = 3036 plf
$M_{max} = 219^{k}$
$V_{max} = 36^{k}$

From Page 12-25 use $\ell n = 24'$, $h = 16''$, $b = 16''$ will hold 3.1 k/ft

Typical Beam 2
Span: 26'
Total Load = 3174 plf
$M_{max} = 268^{k}$
$V_{max} = 41^{k}$

From page 12-25 use $\ell n = 26'$, $h = 16''$, $b = 16''$ will hold 3.1 k/ft

Typical Beam 3
Span 14' + 4' cantilever
Total Load = 1614 plf
$M_{max} = 40'^{k}$
$V_{max} = 15^{k}$

From page 12-50, use $\ell n = 18'$, $h = 12''$, $b = 10''$; will hold 1.7 k/ft $\therefore$ ok

Concrete Beams will bear on masonry bearing walls
- did not design because it is the bearing system of the floor system