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George Read Hall – The University of Delaware
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Technical Assignment #2
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Executive Summary:

This report is a comparison of alternate floor systems for George Read Hall. Four different systems were designed and compared to the existing Hambro floor system. The four systems were a one-way slab, a two-way flat plate slab, precast hollow-core planks, and non-composite steel joists.

The one-way slab system reduced the floor depth and is economical to construct because of the ability to reuse the formwork. The cons of this system are that it requires a redesign of other gravity load and lateral load members and increases the total weight of the building.

The two-way flat plate system is economical to construct because of the simplicity of the formwork. It also provides the shallowest floor depth. On the other hand, it greatly increases the total weight of the building, which could produce higher seismic loads and increase the footing sizes. Furthermore, it requires a minimum of 29" square columns.

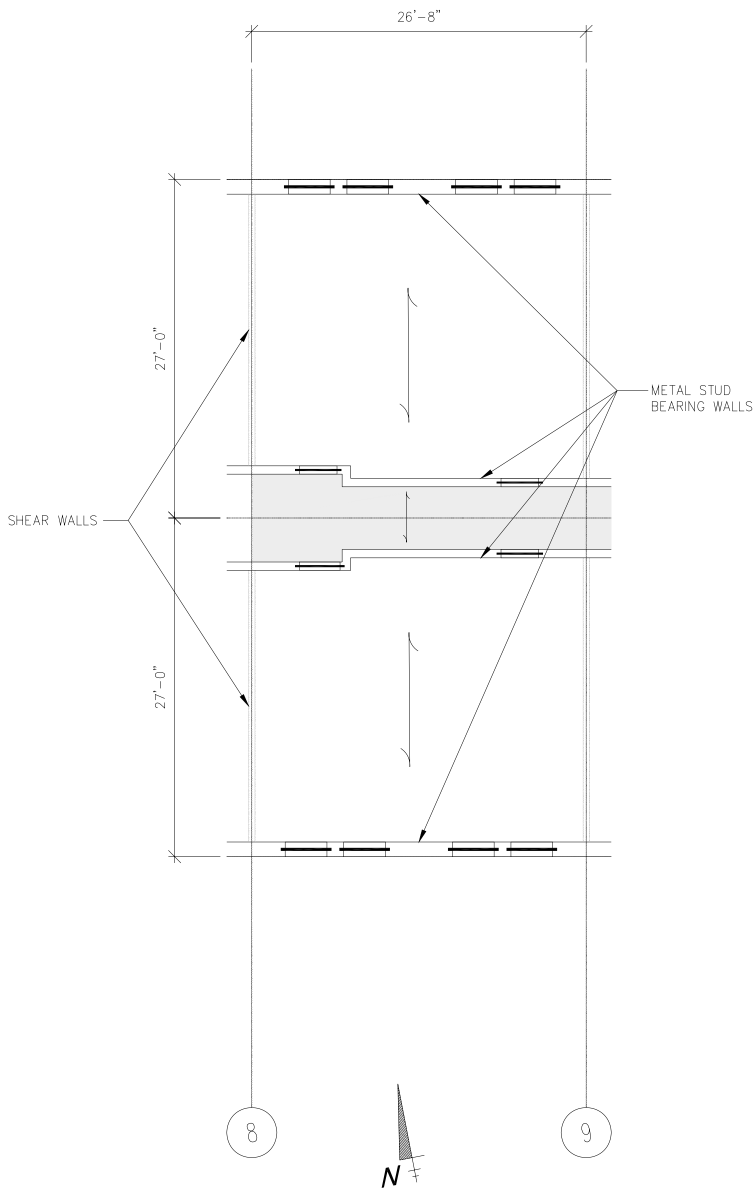
A precast hollow-core plank system is simple to construct and can be done rather quickly. It can also be economical by decreasing labor time. This system also results in a shallower floor, but also brings significant additional weight to the building. The support for this system could be a variety of things, including masonry bearing walls, steel beams, and possibly metal stud bearing walls.

Non-composite joists with a 4" slab is easy to contruct and lightweight. The downfall of this system is that it requires much deeper joists than the existing composite system in order to be economical. Additionally, the spacing of the joists is half as much, resulting in the need for twice as many members.

Therefore, after analyzing and comparing these alternate systems, it was determined that three of the four systems should be considered with further investigation. The non-composite joist system does not warrant further investigation because it has no real advantages over the existing composite system. The other three systems warrant further investigation because they do have advantages over the original Hambro composite system and could provide more economical options.

Existing System:

The existing floor system in George Read Hall is composed of a Hambro composite floor system. This system uses 14" deep steel 50 ksi steel joists working with a $2^3/_4$" concrete slab. The joists are spaced at $4'$-$1^1/_4$" on center. The typical span for the joists is 23'-6" with an interior span of 6'-0" for the corridor. The typical bay is shown below.



THE SHADED AREA REPRESENTS THE CORRIDOR AREA WHERE THE LIVE LOAD IS INCREASED TO 100 PSF. THE SPAN ARROWS SHOW THE DIRECTION OF THE STEEL JOIST FRAMING IN THE HAMBRO FLOOR SYSTEM. THE LIVE LOAD IN THE NON-SHADED AREA IS 40 PSF AS DETERMINED BY IBC.

ALTERNATE SYSTEMS:

FOUR ALTERNATE SYSTEMS WERE ANALYZED AND COMPARED TO THE EXISTING SYSTEM TO DETERMINE IF A MORE APPROPRIATE DESIGN COULD BE INCORPORATED.

ALTERNATE SYSTEM #1: ONE-WAY CONCRETE JOIST SYSTEM

SUPERIMPOSED LOAD = 1.2(25) + 1.6(40) = 94 PSF → LIVING SPACES
SUPERIMPOSED LOAD = 1.2(25) + 1.6(100) = 190 PSF → CORRIDOR

USING THE CRSI MANUAL FOR CONCRETE CONSTRUCTION, 6"WIDE X 8" DEEP RIBS SPACED AT 26" ON CENTER WITH A 3" CONCRETE TOPPING CAN SAFELY SPAN 24'-0" WITH A FACTORED SUPERIMPOSED LOAD OF 104 PSF WHEN REINFORCED WITH #4@8.5" TOP BARS AND 1-#4 AND 1-#5 BOTTOM BARS. THE TABLES ALSO TAKE DEFLECTION INTO ACCOUNT. THE DEFLECTION WAS LIMITED TO L/360, BUT THIS SPAN IS CONTROLLED BY FLEXURAL STRENGTH. IT CAN ALSO BE SEEN FROM THE TABLE THAT THE SAME JOIST CAN EASILY SPAN THE 6'-0" CORRIDOR UNDER THE CALCULATED LOADS.

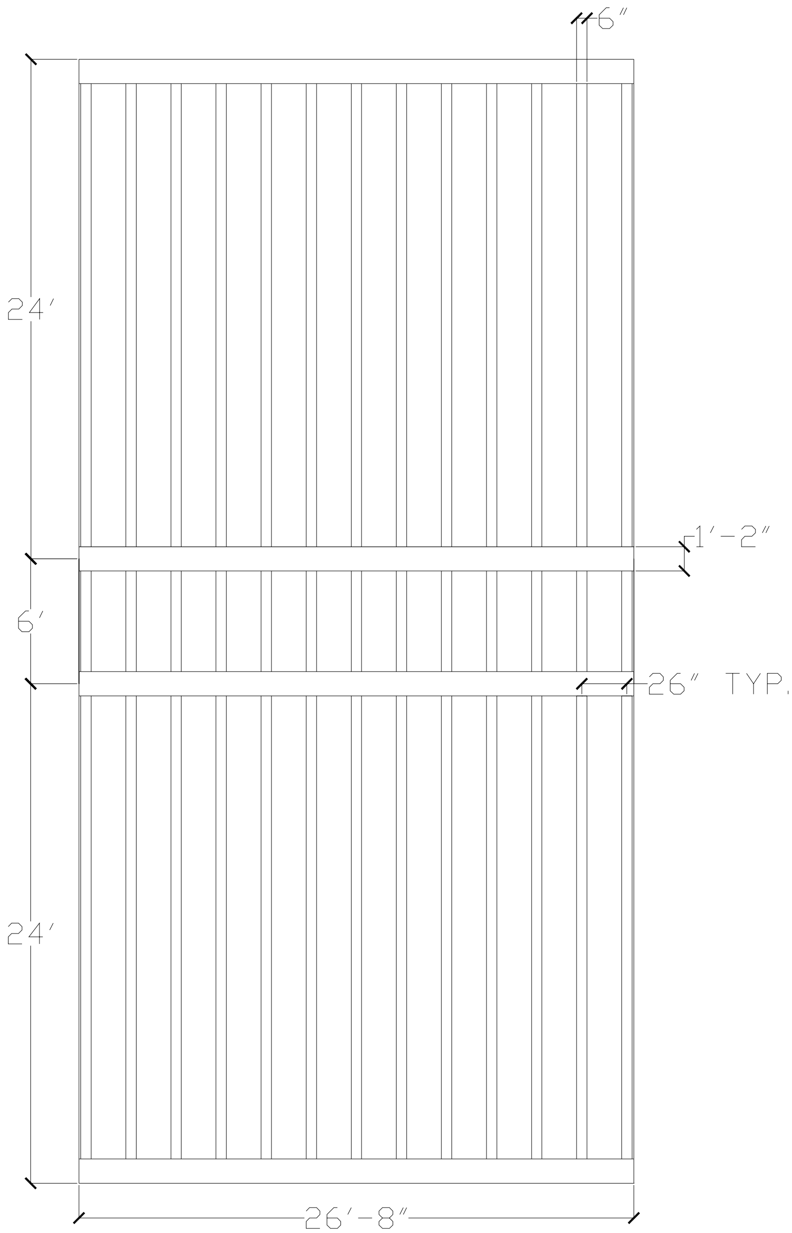
STANDARD ONE-WAY JOISTS (1) MULTIPLE SPANS	20" Forms + 6" Rib @ 26" c.-c. (2) FACTORED USABLE SUPERIMPOSED LOAD (PSF)											f'_c = 4,000 psi f_y = 60,000 psi
8" Deep Rib + 3.0" Top Slab = 11.0" Total Depth												
TOP BARS Size @	# 4 12	# 4 11	# 4 8.5	# 5 10.5	# 6 12	End Span Defl. Coeff. (3)	# 4 12	# 4 10.5	# 4 8	# 5 9.5	# 6 10.5	Int. Span Defl. Coeff. (3)
BOTTOM BARS # #	# 3 # 4	# 4 # 4	# 4 # 5	# 5 # 5	# 5 # 6		# 3 # 3	# 3 # 4	# 4 # 4	# 4 # 5	# 5 # 5	
Steel (psf)	.61	.76	.97	1.20	1.46		.64	.79	1.04	1.33	1.70	
CLEAR SPAN	END SPAN						INTERIOR SPAN					
14'-0"	260	360	450*	462*	476*	.334	275	418	514*	523*	534*	.206
	0	0	477	596	678*		0	0	563	734	770*	
15'-0"	215	302	404	414*	426*	.440	228	353	465*	472*	482*	.271
	0	0	0	508	613*		0	0	479	628	700*	
16'-0"	178	255	344	374*	384*	.570	189	299	411	430*	438*	.351
	0	0	0	436	538		0	0	0	541	641*	
17'-0"	148	215	295	340*	348*	.726	158	255	353	393*	400*	.447
	0	0	0	376	467		0	0	0	469	587	
18'-0"	122	182	253	311*	317*	.913	131	218	306	362*	367*	.562
	0	0	0	326	407		0	0	0	409	514	
19'-0"	101	155	218	283	291*	1.133	109	187	265	334*	339*	.697
	0	0	0	0	356		0	0	0	358	452	
20'-0"	82	131	188	247	268*	1.391	89	160	231	309*	313*	.856
	0	0	0	0	313		0	0	0	314	399	
21'-0"	66	111	163	216	247*	1.691	73	137	201	277	291*	1.041
	0	0	0	0	275		0	0	0	0	354	
22'-0"	52	93	140	189	229*	2.037	58	117	175	244	271*	1.254
	0	0	0	0	243		0	0	0	0	315	
23'-0"	40	77	121	165	213*	2.433	46	99	153	216	253*	1.497
	0	0	0	0	215		0	0	0	0	280	
24'-0"		64	104	144	190	2.885		84	133	191	236*	1.775
		0	0	0	0			0	0	0	250	
25'-0"		52	89	126	168	3.397		70	116	169	222*	2.090
		0	0	0	0			0	0	0	224	
26'-0"		41	75	110	149	3.974		58	100	150	200	2.445
		0	0	0	0			0	0	0	0	
27'-0"			63	95	132	4.621		48	87	132	179	2.844
			0	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 ≥ ℓ_n/18.5 for end spans, ℓ_n/21 for interior spans).
(4) Exclusive of bridging joists and tapered ends.
*Controlled by shear capacity. +Capacity at elastic deflection = ℓ_n/360.

The corresponding girder was designed for this span, carrying half of the larger span living area as well as half of the corridor span. The design length was taken as 26'-8". Calculations show that an 11" x 20" girder with 7-#9 top bars and 5-#9 bottom bars is required to carry the applied loads. The depth was limited to 11" to match the depth of the joists. This will make the construction process of this system much more efficient. Detailed calculations of the girder design are shown in Appendix A.

In addition to a more efficient construction process, this system can be more economical than other concrete systems because the formwork is reusable throughout the entire project. The total depth of the system is 11". This is approximately 6" less than the existing floor system. This could provide beneficial if the floor to ceiling height is constricted. However, the floor to ceiling height in George Read Hall is approximately 8'-7". Therefore, the need to reduce the floor system depth does not seem to be very critical. This system is also more economical because fire proofing is not needed since the slab acts as the fire protection.

This system may be a more economical concrete floor system, but that does not mean that it is the most economical overall system. The labor costs of concrete are much higher than that of a steel system. Additionally, this system would require the use of interior concrete columns and girders in lieu of the existing cold formed metal studs. These new columns could be quite sizable in comparison to the existing HSS columns, possibly creating a loss of architectural space. The extra concrete will also greatly increase the total weight of the building which will cause greater seismic forces. The additional building weight could also cause an increase in the footings sizes. The typical one-way joist layout is shown below.


6"
24'
1'-2"
6'
26" TYP.
24'
26'-8"

ALTERNATE SYSTEM #2: TWO-WAY FLAT PLATE

SUPERIMPOSED LOAD = 94 PSF

AGAIN USING THE CRSI MANUAL, WITH A DESIGN SPAN OF 27' IT CAN BE SEEN THAT A 9" THICK SLAB IS NEEDED. THE REQUIRED COLUMN STRIP REINFORCEMENT IS 14-#5 TOP BARS AT THE EXTERIOR SUPPORT, 9-#7 BOTTOMS BARS, AND 12-#8 TOP BARS AT THE INTERIOR SUPPORT. IN THE MIDDLE STRIP 11-#5 TOP BARS AND 14-#4 BOTTOM BARS ARE REQUIRED. THESE NUMBERS ASSUME A SQUARE PANEL. THE ACTUAL PANEL DIMENSIONS ARE 26'-8" X 23'-6". THE DIFFERENCE BETWEEN THE TWO SPANS IS RELATIVELY CLOSE, SO THE SAME REINFORCEMENT CAN BE USED IN BOTH DIRECTIONS WITHOUT BEING UNECONOMICAL.

FLAT PLATE SYSTEM (WITHOUT SHEARHEADS) — f'_c = 4,000 psi, Grade 60 Bars

SQUARE EDGE PANEL — 9 in. = TOTAL THICKNESS OF SLAB — 0.750 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. +	Bottom	Top Int.	Each Middle Strip: Bottom	Top Int.	End Panel Steel (psf), Location of Panel: E	EC	C
23	50	16	0.767	74	148	200	12-#4 5	9-#5	11-#6	8-#5	8-#5	2.31	2.34	2.23
23	100	20	0.748	89	179	240	14-#4 7	16-#4	13-#6	8-#5	8-#5	2.52	2.54	2.50
23	150	24	0.677	103	207	278	16-#4 5	9-#6	15-#6	8-#5	8-#5	2.78	2.81	2.84
23	200	27	0.709	117	234	315	12-#5 5	10-#6	13-#7	14-#4	8-#5	3.12	3.16	3.25
23	250	31	0.641	129	257	346	13-#5 4	11-#6	11-#8	10-#5	13-#4	3.47	3.49	3.58
23	300	34	0.626	140	279	376	14-#5 3	9-#7	12-#8	8-#6	14-#4	3.79	3.82	3.89
23	350	40	0.610	147	293	395	22-#4 4	8-#8	13-#8	12-#5	10-#5	4.04	4.13	4.26
24	50	18	0.736	84	168	226	13-#4 5	10-#5	12-#6	8-#5	8-#5	2.33	2.35	2.27
24	100	22	0.724	101	201	271	15-#4 7	12-#5	11-#7	8-#5	8-#5	2.62	2.64	2.64
24	150	26	0.713	116	232	313	12-#5 5	10-#6	13-#7	9-#5	8-#5	2.97	3.01	3.12
24	200	30	0.652	131	262	352	13-#5 4	16-#5	12-#8	16-#4	13-#4	3.37	3.41	3.49
24	250	34	0.611	144	288	388	22-#4 4	17-#5	13-#8	8-#6	10-#5	3.69	3.74	3.86
24	300	39	0.610	154	307	414	23-#4 4	8-#8	17-#7	12-#5	10-#5	4.02	4.06	4.19
24	350	45	0.609	161	322	433	16-#5 2	8-#8	14-#8	9-#6	16-#4	4.29	4.31	4.45
25	50	20	0.733	94	188	252	14-#4 7	11-#5	14-#6	13-#4	13-#4	2.45	2.47	2.44
25	100	24	0.724	113	225	303	17-#4 8	10-#6	13-#7	9-#5	13-#4	2.86	2.88	2.88
25	150	29	0.651	130	260	349	13-#5 4	11-#6	19-#6	16-#4	13-#4	3.19	3.21	3.32
25	200	33	0.633	146	292	393	22-#4 7	10-#7	13-#8	12-#5	10-#5	3.66	3.71	3.84
25	250	39	0.610	158	316	425	16-#5 2	10-#7	14-#8	19-#4	16-#4	3.91	3.96	4.08
25	300	45	0.609	167	334	450	25-#4 3	20-#5	15-#8	13-#5	11-#5	4.17	4.19	4.35
25	350	51	0.608	175	351	472	13-#6 1	9-#8	16-#8	10-#6	12-#5	4.53	4.62	4.85
26	50	22	0.705	105	209	281	16-#4 6	9-#6	15-#6	13-#4	13-#4	2.53	2.56	2.53
26	100	27	0.658	125	251	337	19-#4 6	11-#6	14-#7	10-#5	13-#4	2.98	3.00	3.02
26	150	31	0.655	145	290	390	22-#4 7	10-#7	13-#8	12-#5	10-#5	3.51	3.54	3.57
26	200	37	0.636	161	322	433	16-#5 5	14-#6	14-#8	9-#6	16-#4	3.82	3.87	3.98
26	250	44	0.609	172	345	464	17-#5 4	11-#7	15-#8	10-#6	12-#5	4.17	4.23	4.39
26	300	51	0.608	182	364	490	13-#6 1	9-#8	16-#8	22-#4	12-#5	4.46	4.49	4.67
26	350	58	0.607	189	378	509	19-#5 1	10-#8	17-#8	11-#6	9-#6	4.75	4.82	5.05
27	50	24	0.717	116	232	313	12-#5 5	10-#6	13-#7	9-#5	9-#5	2.72	2.75	2.71
27	100	29	0.693	139	279	375	14-#5 6	9-#7	12-#8	11-#5	14-#4	3.18	3.21	3.26
27	150	34	0.654	160	321	432	16-#5 5	14-#6	14-#8	19-#4	16-#4	3.60	3.66	3.79
27	200	41	0.630	176	351	473	13-#6 3	9-#8	16-#8	10-#6	12-#5	4.12	4.19	4.35
27	250	49	0.608	188	376	506	19-#5 3	12-#7	17-#8	11-#6	9-#6	4.43	4.48	4.64
27	300	56	0.607	198	396	533	14-#6 1	10-#8	18-#8	16-#5	13-#5	4.71	4.78	4.95
27	350	64	0.606	205	410	552	15-#6 0	11-#8	18-#8	9-#7	10-#6	5.02	5.08	5.27
28	50	26	0.709	129	258	347	19-#4 10	23-#4	14-#7	10-#5	14-#4	2.78	2.79	2.77
28	100	31	0.679	154	308	414	15-#5 6	10-#7	13-#8	12-#5	10-#5	3.33	3.36	3.47
28	150	38	0.662	175	351	472	13-#6 4	15-#6	15-#8	10-#6	12-#5	3.92	3.92	4.06
28	200	46	0.609	191	381	513	19-#5 3	10-#8	17-#8	11-#6	13-#5	4.35	4.38	4.56
28	250	54	0.608	203	407	547	20-#5 3	13-#7	18-#8	16-#5	10-#6	4.62	4.68	4.93
28	300	62	0.607	213	426	574	16-#6 2	11-#8	19-#8	12-#6	10-#6	5.00	5.07	5.33
28	350	70	0.606	221	442	595	16-#6 0	11-#8	20-#8	10-#7	11-#6	5.22	5.30	5.60

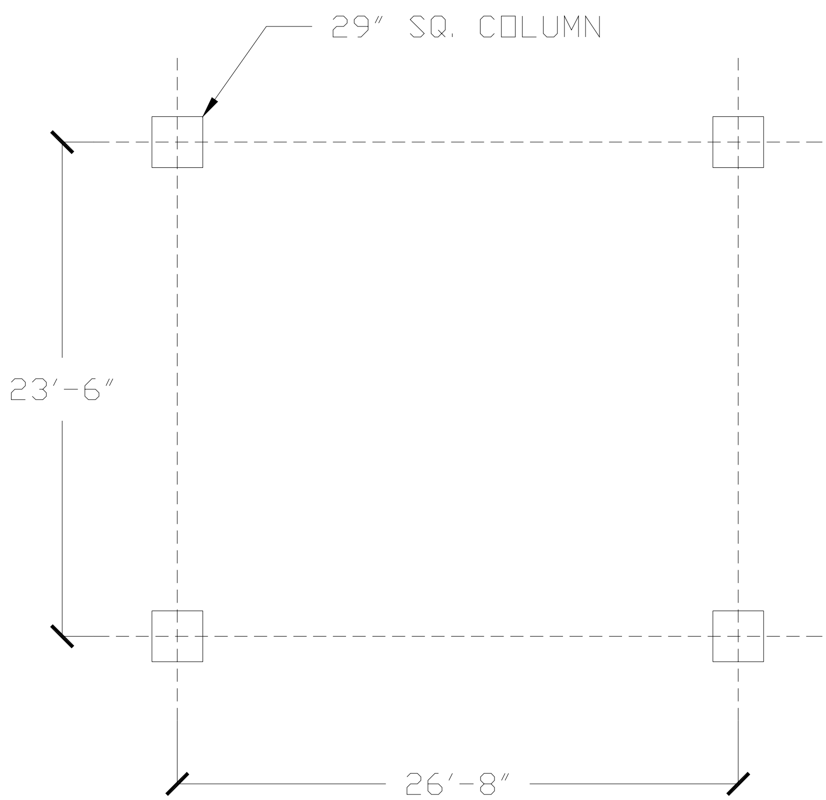
SQUARE INTERIOR PANEL — 9 in. = TOTAL THICKNESS OF SLAB — 0.750 c.f./s.f.

(2) Span c.-c. (ft)	(3) Load (psf)	(1) Min. Sq. Col. (in.)	Reinforcing Bars, Column Strip: Top	Bottom	Middle Strip: Top	Bottom	Steel (psf), Location of Panel: I	IE	IC
23	50	11	14-#5	8-#5	8-#5	8-#5	2.37	2.39	2.42
23	100	15	17-#5	8-#5	8-#5	8-#5	2.55	2.57	2.60
23	150	19	20-#5	13-#4	8-#5	8-#5	2.77	2.79	2.81
23	200	23	12-#7	10-#5	8-#5	8-#5	3.11	3.14	3.17
23	250	28	11-#8	16-#4	8-#5	8-#5	3.42	3.43	3.43
23	300	33	11-#8	8-#6	13-#4	8-#5	3.56	3.61	3.65
23	350	40	12-#8	12-#5	9-#5	8-#5	3.82	3.87	3.92
24	50	12	16-#5	8-#5	8-#5	8-#5	2.38	2.39	2.41
24	100	17	11-#7	8-#5	8-#5	8-#5	2.70	2.70	2.69
24	150	22	12-#7	14-#4	8-#5	8-#5	2.90	2.92	2.95
24	200	26	11-#8	16-#4	8-#5	8-#5	3.27	3.30	3.34
24	250	31	12-#8	12-#5	9-#5	8-#5	3.61	3.65	3.70
24	300	39	13-#8	19-#4	10-#5	8-#5	3.86	3.86	3.86
24	350	47	13-#8	9-#6	10-#5	13-#4	4.01	4.05	4.09
25	50	14	13-#6	13-#4	13-#4	13-#4	2.50	2.52	2.54
25	100	19	12-#7	9-#5	13-#4	13-#4	2.82	2.85	2.87
25	150	24	14-#7	16-#4	13-#4	13-#4	3.13	3.13	3.12
25	200	29	12-#8	12-#5	9-#5	13-#4	3.48	3.52	3.57
25	250	37	13-#8	9-#6	10-#5	13-#4	3.78	3.82	3.86
25	300	46	14-#8	20-#4	16-#4	9-#5	4.01	4.06	4.10
25	350	54	14-#8	10-#6	16-#4	9-#5	4.17	4.26	4.36
26	50	16	20-#5	13-#4	13-#4	13-#4	2.48	2.49	2.51
26	100	21	13-#7	10-#5	13-#4	13-#4	2.88	2.90	2.92
26	150	26	12-#8	12-#5	9-#5	13-#4	3.33	3.37	3.41
26	200	33	13-#8	13-#5	10-#5	13-#4	3.61	3.64	3.68
26	250	43	14-#8	10-#6	16-#4	9-#5	3.94	3.99	4.05
26	300	52	15-#8	22-#4	11-#5	10-#5	4.19	4.24	4.28
26	350	62	15-#8	11-#6	12-#5	10-#5	4.43	4.51	4.59
27	50	18	12-#7	10-#5	9-#5	9-#5	2.74	2.76	2.78
27	100	23	12-#8	17-#4	9-#5	9-#5	3.17	3.17	3.16
27	150	29	13-#8	13-#5	10-#5	13-#4	3.45	3.48	3.52
27	200	39	14-#8	10-#6	11-#5	14-#4	3.78	3.86	3.93
27	250	49	15-#8	11-#6	12-#5	10-#5	4.18	4.26	4.33
27	300	59	16-#8	16-#5	12-#5	16-#4	4.38	4.46	4.54
27	350	69	17-#8	16-#5	9-#6	16-#4	4.61	4.67	4.72
28	50	19	14-#7	16-#4	14-#4	14-#4	2.82	2.82	2.82
28	100	26	13-#8	19-#4	10-#5	14-#4	3.29	3.29	3.29
28	150	33	15-#8	14-#5	11-#5	10-#5	3.78	3.78	3.79
28	200	45	16-#8	23-#4	12-#5	10-#5	4.05	4.09	4.13
28	250	56	17-#8	16-#5	13-#5	16-#4	4.37	4.41	4.46
28	300	67	17-#8	12-#6	13-#5	11-#5	4.57	4.65	4.73
28	350	77	18-#8	17-#5	20-#4	12-#5	4.79	4.89	4.99

(1) Columns same above and below plate. (2) Center-to-center of columns; $\ell_1 = \ell_2$. (3) Superimposed factored load (factored dead load has been deducted).

One of the primary advantages of the two-way flat plate system is its economy. The formwork is very simple, resulting in less labor time to form and strip the concrete. Another advantage of the flat plate system is that the total depth of the floor is only 9". However, as discussed above, the depth of the floor is not a critical issue. Again, fire proofing is not necessary for this system because of the use of the 9" thick concrete slab.

The major downfall of this system in the context of this report is that it also would require the use of interior concrete columns and beams. This system requires minimum 29" square columns which could reduce the amount of usable space. The use of a two-way flat plate would also increase the total weight of the building due to a thicker slab and larger concrete columns than in the existing Hambro system. This will again cause an increase in the seismic loads and introduce greater loads into the footings.



Alternate System #3: Pre-stressed Hollow Core Plank

Superimposed Load = 94 psf

Using the PCI Design Handbook's provided load tables for hollow core planks, it was determined that 4'-0" wide x 8" deep lightweight planks with a 2" normal weight concrete topping are sufficient. The reinforcing for these planks is 6-$^3/_8$" straight prestressing strands located 1$^1/_2$" up from the bottom of the planks. Additional calculations show that a W16x50 is required to support the edges of the planks. These calculations can be seen in Appendix B. The typical plank cross section is shown below along with the corresponding load tables.

HOLLOW-CORE

4'-0" x 8"
Lightweight Concrete

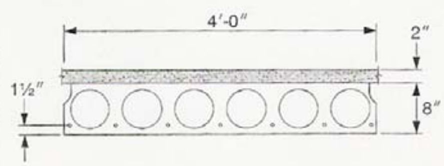
Strand Pattern Designation

76-S
- S = straight
- Diameter of strand in 16ths
- No. of strand (7)

Safe loads shown include dead load of 10 psf for untopped members and 15 psf for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load. Check availability of lightweight sections.

Capacity of sections of other configurations are similar. For precise values, see local hollow-core manufacturer.

Key
- 346 — Safe superimposed service load, psf
- 0.3 — Estimated camber at erection, in.
- 0.4 — Estimated long-time camber, in.



f'_c = 5,000 psi
f'_{ci} = 3,500 psi

Section Properties

		Untopped		Topped	
A	=	215	in²	—	
I	=	1,666	in⁴	3,529	in⁴
y_b	=	4.00	in.	5.70	in.
y_t	=	4.00	in.	4.30	in.
S_b	=	416	in³	619	in³
S_t	=	416	in³	821	in³
b_w	=	12.00	in.	12.00	in.
wt	=	184	plf	272	plf
		46	psf	68	psf
V/S	=	1.92	in.		

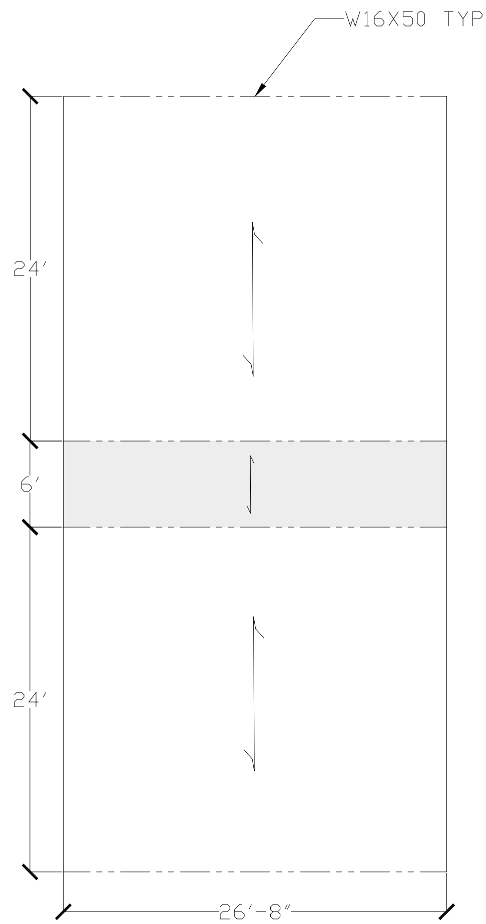
4LHC8+2

Table of safe superimposed service load (psf) and cambers (in.) — **2" Normal Weight Topping**

Strand Designation Code	Span, ft																					
	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38
66-S	320	277	242	211	186	163	144	127	113	100	88	78	69	60	53	45							
	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.4	0.4	0.3	0.3	0.2							
	0.4	0.5	0.5	0.5	0.5	0.4	0.4	0.3	0.3	0.2	0.0	−0.1	−0.3	−0.5	−0.7	−1.0							
76-S		327	286	251	222	196	174	155	138	123	109	98	87	77	69	61	52	43					
		0.5	0.5	0.6	0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.6	0.5	0.4	0.3					
		0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.5	0.4	0.3	0.2	0.1	−0.1	−0.3	−0.6	−0.9	−1.2					
58-S				327	290	258	231	206	185	167	150	135	122	110	99	90	81	72	62	53	45		
				0.8	0.8	0.9	0.9	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.0	1.0	1.0	0.9	0.8	0.7		
				0.9	0.9	1.0	1.0	1.0	1.0	0.9	0.9	0.8	0.7	0.6	0.4	0.2	0.0	−0.2	−0.5	−0.9	−1.3		
68-S					323	304	278	250	225	204	184	167	151	138	125	114	103	93	83	73	64	56	48
					1.1	1.1	1.2	1.3	1.3	1.4	1.5	1.5	1.5	1.6	1.6	1.6	1.6	1.6	1.5	1.5	1.4	1.3	1.2
					1.2	1.3	1.3	1.4	1.4	1.4	1.4	1.3	1.3	1.2	1.1	0.9	0.8	0.6	0.3	0.0	−0.3	−0.7	−1.2
78-S					332	313	297	279	263	238	216	197	179	163	149	136	125	113	102	91	81	72	64
					1.3	1.4	1.5	1.6	1.7	1.7	1.8	1.9	2.0	2.0	2.1	2.1	2.1	2.2	2.2	2.2	2.1	2.1	2.0
					1.5	1.6	1.7	1.7	1.8	1.8	1.8	1.8	1.8	1.8	1.7	1.6	1.5	1.3	1.1	0.9	0.6	0.2	−0.1

Strength based on strain compatibility; bottom tension limited to $6\sqrt{f'_c}$; see pages 2-2–2-6 for explanation.

The typical bay is shown below. The span arrows indicate the direction of the hollow-core span. The shaded area represents the area where the live load is increased to 100 psf.



Lightweight concrete was used for this design because it decreases the total weight of the building. Lightweight 8" planks are actually lighter than 6" normal weight planks. In additon to less weight, less reinforcing is needed because of the added depth. The total depth of this system is only 10", which is the second shallowest of all the systems investigated in this report. This depth assumes that a bulkhead would be built around the beams. It may be possible to bear the panels on metal stud wall, but further investigation is required to determine its feasibility. Another positive of this system is that a variety of options

are available for plank bearing. A wide flange beam was designed as the bearing member in this report, but masonry walls and metal stud walls could be other options. This will allow for the most economical system to be used to support the planks. Because the concrete is precast, the construction time is a lot less for this system. This will be more cost efficient. Fire protection is also already provided by the planks.

Even with lightweight concrete, this system is still slightly heavier than the existing system. The bulkheads built around the wide flange supports could present an architectural problem.

Alternate System #4: Non-composite Steel Joists

Assuming a 4"slab, the distributed load on the joists is:

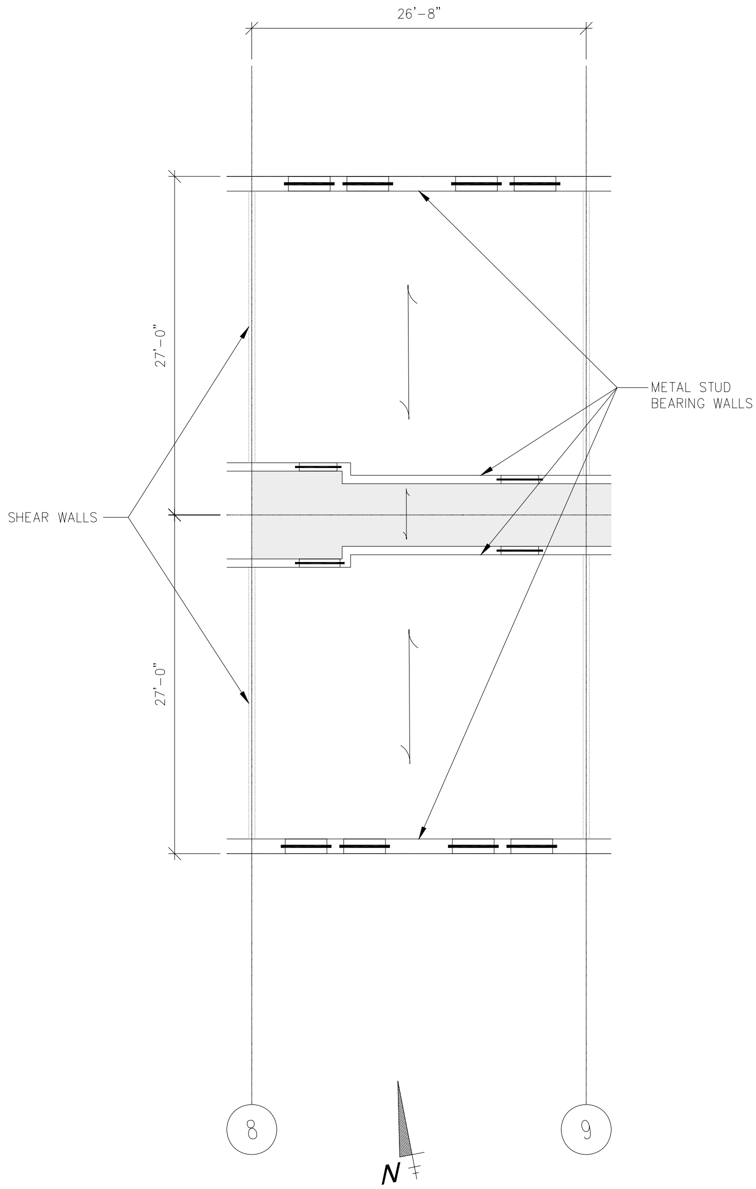
$$w = 1.2(50) + 1.2(25) + 1.6(40) = 154 \text{ psf}$$

At 2'-0" on center, 18K3 steel joists are required. This was determined using the K-Series economy table from the Columbia Joist Company catalog. The slab reinforcement is #3@12 bottom bars.

Joist Designation	14K6	18K5	22K4	16K6	20K5	24K4	18K6	16K7	22K5	20K6	18K7	22K6	20K7	24K5	22K7	24K6
Depth (In).	14	18	22	16	20	24	18	16	22	20	18	22	20	24	22	24
Approx. Wt. (lbs./ft)	7.7	7.7	8.0	8.1	8.2	8.4	8.5	8.6	8.8	8.9	9.0	9.2	9.3	9.3	9.7	9.7
Span (ft)																
14	550 550															
15	550 507															
16	550 467			550 550				550 550								
17	550 443			550 526				550 526								
18	550 408	550 550		550 490			550 550	550 490			550 550					
19	550 383	550 523		550 455			550 523	550 455			550 523					
20	525 347	550 490		550 426	550 550		550 490	550 426		550 550	550 490		550 550			
21	475 299	550 460		548 405	550 520		550 460	550 406		550 520	550 460		550 520			
22	432 259	518 414	550 548	498 351	550 490		550 438	550 385	550 548	550 490	550 438	550 548	550 490		550 548	
23	395 228	473 362	518 491	455 307	529 451		518 393	507 339	550 518	550 468	550 418	550 518	550 468		550 518	
24	362 199	434 318	475 431	418 269	485 386	520 516	473 345	465 296	536 483	528 430	526 382	550 495	550 448	550 544	550 495	550 544
25	334 175	400 281	438 381	384 238	446 350	479 456	435 305	428 263	493 427	486 380	485 337	537 464	541 421	540 511	550 474	550 520
26	308 156	369 249	404 338	355 211	412 310	442 406	402 271	395 233	455 379	449 337	448 299	496 411	500 373	499 453	550 454	543 493
27	285 139	342 222	374 301	329 188	382 277	410 361	372 241	366 208	422 337	416 301	415 267	459 367	463 333	462 404	512 406	503 439
28	265 124	318 199	348 270	306 168	355 246	381 323	346 216	340 186	392 302	386 269	385 239	427 328	430 298	429 362	475 364	467 393
29		296 179	324 242	285 151	330 223	354 290	322 194	317 167	365 272	360 242	359 215	398 295	401 268	400 325	443 327	435 354
30		276 161	302 219	266 137	308 201	331 262	301 175	298 151	341 245	336 218	335 194	371 266	374 242	373 297	413 296	406 319

This system can be more economical than a composite system because it doesn’t require the use of Hambro specialists. This would result in less labor costs. Another positive aspect of this system is that cold formed metal studs can still be used. The non-composite system is also closer in weight to the existing structure, so the affects on the seismic loads and footing capacities are not as great as in the other alternate systems. The existing shear wall system can also be used.

The depth of this system totals 22”. This is 5” deeper than the existing system. In addition to this additional depth, the joists are spaced at 2’-0” on center instead of 4’-1$^{1}/_{4}$” on center. This means that twice as many joists are required, significantly increasing the material costs as well as labor costs to install more joists. More joists also requires more fireproofing. All of the joists must have adequate fireproofing applied as required by code. The typical bay looks the same as the existing system; however, in this diagram, the span arrows represent 18K3 joists at 2’-0” on center.


26'-8"
27'-0"
27'-0"
METAL STUD
BEARING WALLS
SHEAR WALLS
8
9
N

Summary:

System	Pros	Cons	Further Investigation
Existing	-Lightweight -Economical -Allows room for mechanical equip.	-Deep floor depth -Harder to fireproof	--
One-Way Slab	-Economical constuction -No additional fireproofing required	-Requires redesign -Heavy -No room for mechanical equip.	Yes
Two-Way Slab	-Economical construction -Shallowest floor depth -No additional fireproofing required	-Requires large columns -Heaviest system -Requires Redesign -No room for mechanical equip.	Yes
Hollow-Core Planks	-Shallow floor depth -Simple/Fast construction -Cost effective due to less construction time -No additional fireproofing required	-Heavy -Requires deep support beams or redesign -No room for mechanical equip.	Yes
Non-Composite Joists	-Simple construction -Lightweight -Allows room for mechanical equip.	-Deepest floor depth -More joists required -Harder to fireproof	No

Conclusions:

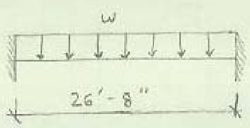
After analyzing four alternate floor systems and comparing them to the existing system, several conclusions can be made. Three of the four systems could require a different structural system than the cold formed metal studs. In addition, they would require a different lateral load resisting system. The non-composite steel joist system does not require a different structural system than the existing one; however, it results in a heavier system than the original with significantly deeper members. Therefore, I feel that no further investigation is needed on this system, and it is not a viable alternate option because it has no real advantages over the existing system. The other three systems may require redesigns of other gravity load and lateral load resisting members, but may be beneficial due to other advantages.

Appendix

Appendix A:

<u>One-Way Slab Girder Design</u>

$w = 1.2(25) + 1.2(65) + 1.6(40) = 172$ psf → living spaces
$w = 1.2(25) + 1.2(65) + 1.6(100) = 268$ psf → corridors
Self-weight $= 1.2(11)(16)(150)(\frac{1}{144}) = 220.0$ lb/ft



$w = 172(0.5)(23.5) + 268(0.5)(6) + 220.0 = 3045.0$ lb/ft

$M_{max} = \frac{wl^2}{11} = \frac{3.045(26.67)^2}{11} = 196.90$ k·ft → Negative

$M_{max} = \frac{wl^2}{16} = \frac{3.045(26.67)^2}{16} = 135.37$ k·ft → Positive

<u>Try 11" × 16" Girder:</u>

Top Reinforcement:

$M_x = 196.90 - \frac{3.045}{2}\left(26.67 - \frac{8.5}{12}\right) = 157.37$ k·ft

$M_{n,req} = \frac{157.37}{0.9} = 174.86$ k·ft

$A_{s,req} = \frac{174.86(12)}{60(0.9)(9.5)} = 4.57$ in^2 → Try 5-#9 Bars, $A_s = 5.00$ in^2

$a = \frac{5.0(60)}{0.85(4)(16)} = 5.51$ in.

$M_n = 5.00(60)(8.5 - \frac{5.51}{2}) = 1723.50$ k·in $= 143.63$ k·ft < 174.86 → NO GOOD.

<u>Try 11" × 20" Girder:</u>

Try 7-#9 Bars, $A_s = 7.00$ in^2

$a = \frac{7.0(60)}{0.85(4)(20)} = 6.18$ in.

$M_n = 7.0(60)(8.5 - \frac{6.18}{2}) = 2272.94$ k·in $= 189.41$ k·in → OK

Bottom Reinforcement:

$M_{n,req} = \frac{135.37}{0.9} = 150.41$ k·ft

$A_{s,req} = \frac{150.41(12)}{60(0.9)(8.5)} = 3.93$ in^2 → Try 5-#9 Bars, $A_s = 5.00$ in^2

$a = \frac{5(60)}{0.85(4)(20)} = 4.41$ in.

$M_n = 5.0(60)(8.5 - \frac{4.41}{2}) = 1888.24$ k·in $= 157.35$ k·ft

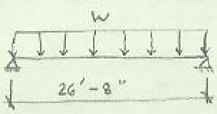
∴ 11" × 20" GIRDER w/ 7-#9 TOP BARS & 5-#9 BOTTOM BARS IS OK

Appendix B:

Beam Design

$w = 1.2(74) + 1.2(25) + 1.6(40) = 182.8$ psf → living spaces

$w = 1.2(74) + 1.2(25) + 1.6(100) = 278.8$ psf → corridor



$w = 182.8(0.5)(23.5) + 278.8(0.5)(6) = 2.984$ k/ft

$M_{max} = \frac{w\ell^2}{8} = \frac{2.984(26.67)^2}{8} = 265.3$ k.ft

– Using beam curves, most economical section is W16 × 40

$\Delta = \frac{5(2.23)(26.67)^4 1728}{384(29000)(518)} = 1.69$ in.

$\Delta_{allow} = \frac{26.67(12)}{240} = 1.33$ in. → NO GOOD

$I_{req} = \frac{5(2.23)(26.67)^4 1728}{384(29000)(1.33)} = 656.5$ in^4

∴ USE W16 × 50

Appendix C:

Non-composite Joist Design

- Assume 4" Slab

$W_{slab} = \frac{4}{12}(150) = 50$ psf

$w = 1.2(50) + 1.2(25) + 1.6(40) = 154$ psf

@ 4'-0" Spacing

$w = 154(4) = 616$ lb/ft. → exceeds all K-series joist safe loads

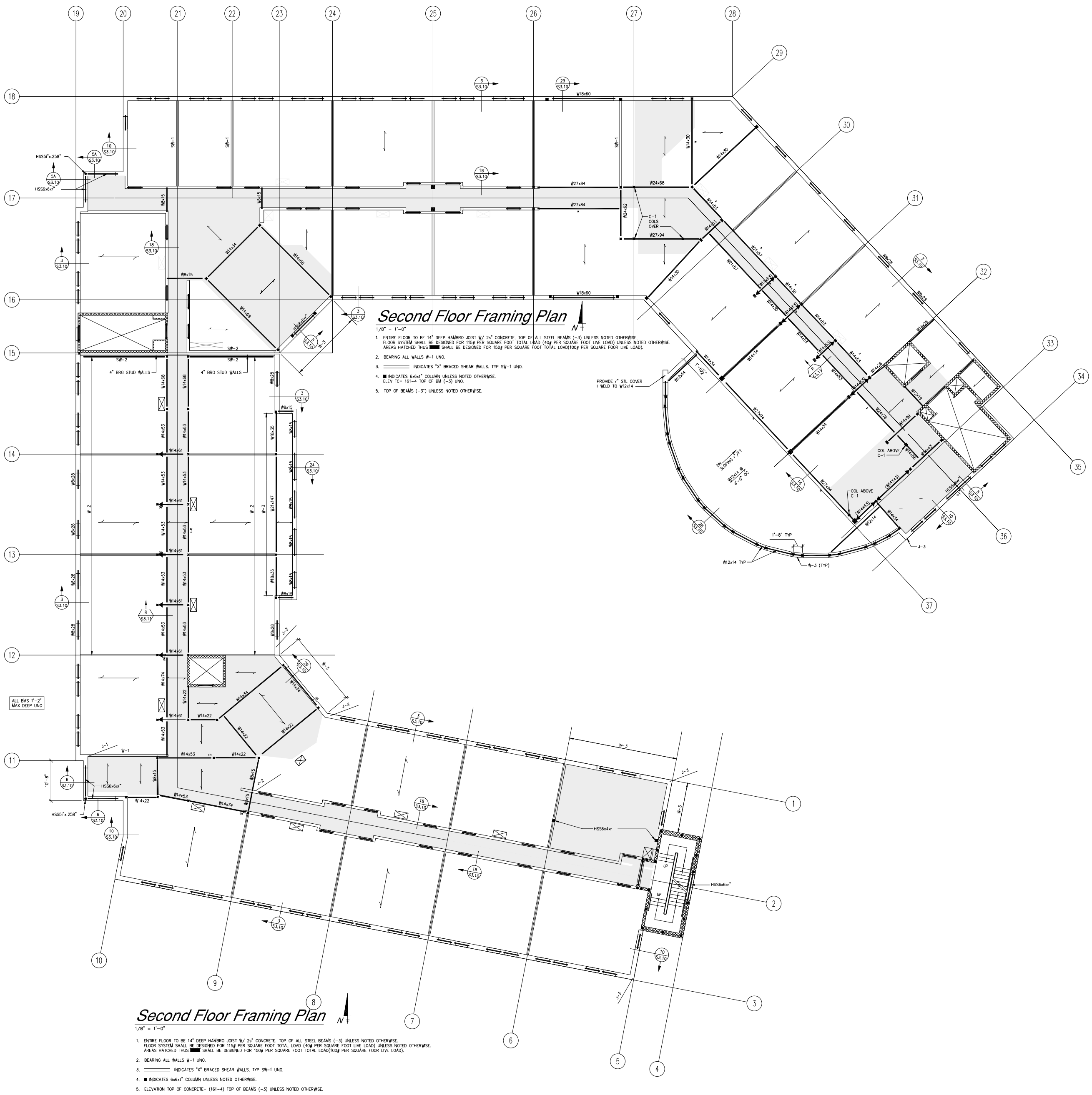
@ 2'-0" Spacing

$w = 154(2) = 308$ lb/ft.

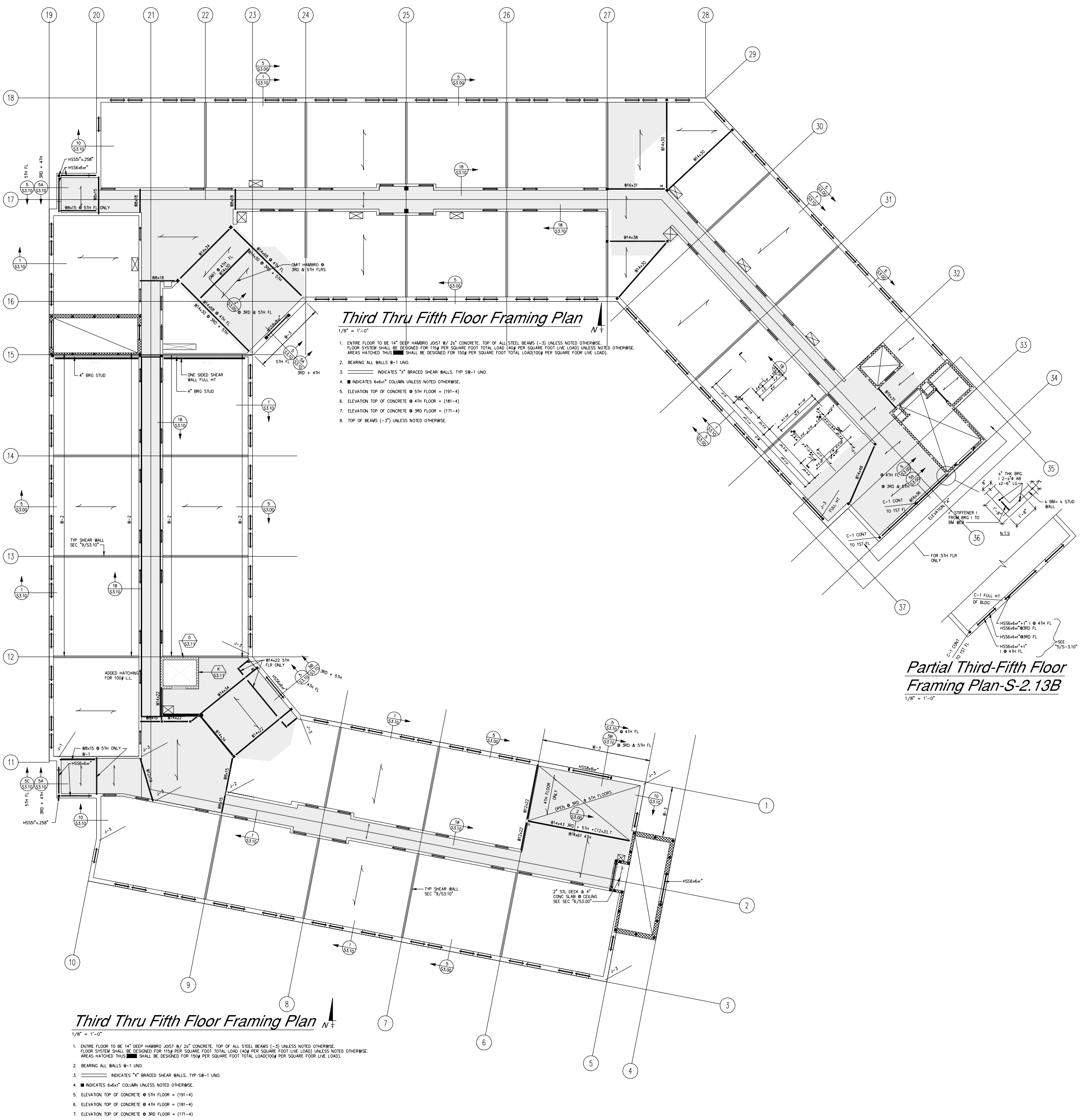
- Using K-series economy table, Use 18K3 Joists

Appendix D:

Second floor framing plan



Third through fifth floor plan



Building Section