North Mountain IMS Medical Office Building

Phoenix, Arizona

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Technical Assignment 2

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AE 481W-Senior Thesis

The Pennsylvania State University

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Executive Summary

Technical Assignment 2 is an exploration of different floor systems comparing structural, construction, and architectural issues. North Mountain IMS Office Building currently uses a complete pre-cast concrete structural system. This system utilizes 10’-0” feet wide by 2’-0” deep pre-stressed double tees as floor framing. This system was most likely chosen due to the great span to depth ratio of pre-stressed members. With a typical bay reaching 48’-0”, the use of double tees eliminates interior columns, which allows for maximum flexibility for the three floors of rentable office space. However, designing an alternative floor system with a typical span of 48’-0” is a challenge. To allow for thorough investigations of multiple floor systems, structural bays that span 30’-0” will also be considered for certain floors.

The floor systems explored:

Existing System: Pre-cast Concrete Double Tees
System 1: Composite Steel Framing
System 2: Hollow-Core Plank
System 3: Steel Joists
System 4: Two-way Concrete Waffle Slab

Comparisons are based on many issues such as cost, weight, efficiency, depth, constructability, and fire rating. The Summary Comparison Chart can be found on page 13. It is important to keep in mind that this report is based on simple assumptions and is intended to be a guide for future thesis redesign reports. Information for comparing the various systems was gathered from many different resources; they are noted throughout the report as needed.

Based on various criteria, the following floor systems are viable options for alternative floor systems, composite steel framing and two-way concrete waffle slab. However, for the two-way slab and the composite framing to be viable options, different bay sizes must be explored. As stated before, 30’-0” x 30’-0” bays were analyzed for the waffle slab and composite steel system.

Codes and Standards

Note: The Senior Thesis project requires the use of the most current codes and standards, those referenced for calculations in this report are listed at the end of this section.

Building Codes:

1. International Building Code (IBC), 2003 edition
2. International Energy Code (IECC), 2003 edition with 2004 supplements
3. National Electric Code (NEC), 2005 edition with Phoenix amendments
4. International Mechanical Code (IMC), 2003 edition with Phoenix amendments
5. Arizona State Plumbing Code, with 2003 supplements
6. Uniform Fire Code (UFC), 1997 edition with Phoenix amendments

Structural Codes:

1. American Concrete Institute (ACI-318), 2002 edition
2. Precast Concrete Institute (PCI), 6th edition

Building Design Loads:

1. American Society of Civil Engineers (ASCE-7), 2002 edition

Thesis Project Codes and Loads:

1. IBC 2005
2. ASCE-7, 2005
3. ACI-318 2005
4. AISC 2005
5. PCI, 6th edition

Load Summary

Live Loads:

- Roof Live Load……………………………………………..20 psf
- Floor Live Load…………………………………………….80 psf
- Stair Live Load……………………………………………100 psf
- Partition Live Load…………………………………………20 psf

Dead Loads:

- Superimposed Roof Dead Load……………………………….15 psf
- Superimposed Floor Dead Load………………………………15 psf

Wind Load:

- Total Wind Force (North-South Direction)…………………218 kips
- Total Wind Force (East-West Direction)……………...…….285 kips

Seismic Load:

- Design Base Shear…………………..……………………..1627 kips

The floor live loads for North Mountain are typical office loads. The second, third, and forth floors all feature an open floor plan with no set dimensions for walls or corridors. Because of the open floor plan, the floor live load is 80 psf. By code, corridor loading above the first floor is 80 psf. This value was used as the live load over the entire floor. In design, it is much easier to assume a uniform load over the entire floor compared to breaking the loads down between office and corridors. Also, a partition live load of 20 psf is used over the entire floor.

The floor dead load only accounts for 15 psf of superimposed load which includes mechanical, electrical, and pluming equipment. The nature of precast concrete structures makes it very simple to calculate the actual weight of the structure; a dead load in pounds per square foot is not needed because each piece of precast is detailed and the exact weight calculated.

Existing System: Pre-cast Concrete Double Tees

North Mountain IMS Office Building floor framing consists of 2'-0" deep, 10'-0" wide double tees with a minimum of 3-1/4" concrete topping. This minimum topping thickness is needed to produce the required 2 hour fire rating between the first and second floor. The tees are normal-weight concrete and have a 28-day compressive strength of 6,000 psi. The minimum prestress release strength is 4,200 psi. The prestressing strand is 7 wire, ½" diameter 270 ksi low relaxation strand. Each strand is pulled to 72.5% capacity, which results in a 30 kip force. The strand is held down at one point in the middle of the tee. Depressed strand provides greater flexural strength while reducing the stresses in the concrete during prestress release. Typical spans are 44', 48', and 54'. Figure 1 shows the second floor plan and double tee layout.

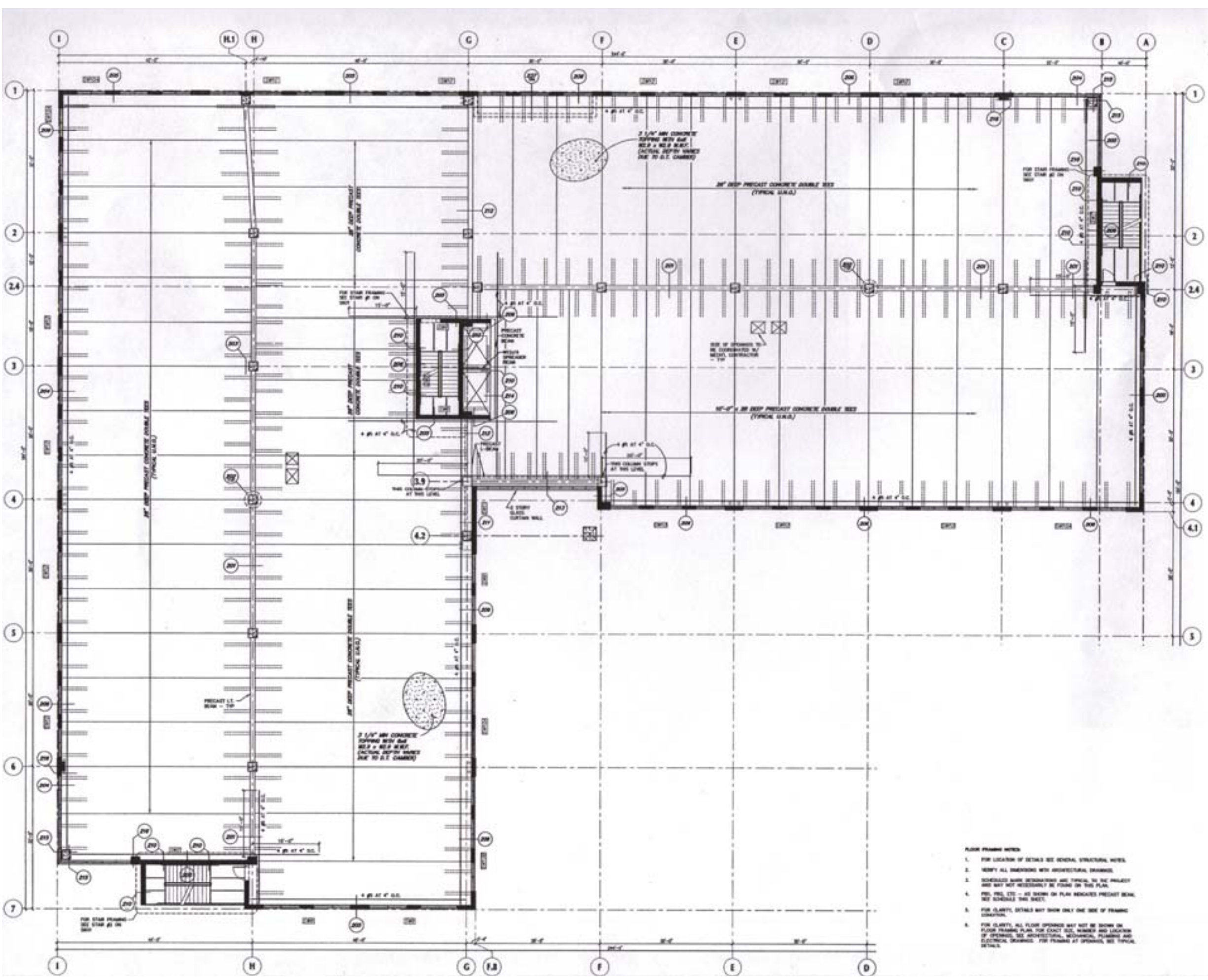


Figure 1

The 2’-0” deep double tees bear on 2’-0” deep by 2’-8” wide inverted tee girders. 28-day strength is 7,500 psi and minimum release strength is 3,750 psi. Typical inverted tee girders use 22 ½” diameter stands for tensile reinforcement. Span length for a 30’-0” bay is 28’-0” due to the columns on each end. Dapped ends on the double tees allow the top of the tee to line flush with the top of the girder. The topping is then poured over the tee and the girder at the same time, interlocking them.

Precast concrete as a material and building technique can provide structures that are very economical and efficient. North Mountain IMS Office Building is well balanced between function, economics, and efficiency. However, obtaining a balanced building project is no simple task. Precast design encompasses many different disciplines which add to the complexity of a project. There are architects, engineers, fabricators, and erectors each with their own unique objectives. Economical and efficient precast structures effectively combine all parties’ objectives. Communication and good project management is extremely important; all parties involved must work closely together.

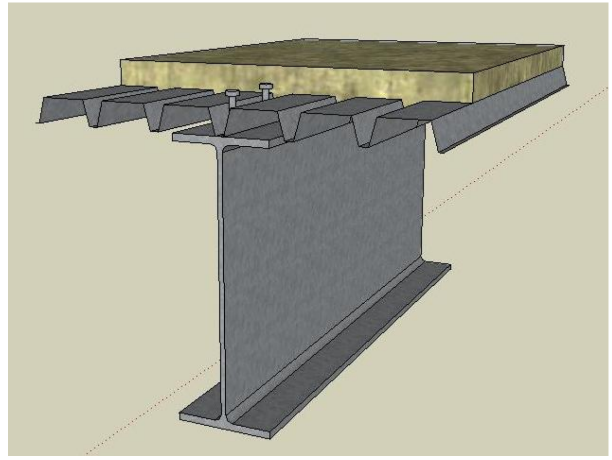
The precast fabricator is a large component because they can only make pieces in certain sizes and shapes. The architect and design engineer must first consider the capabilities of the fabricator when considering a precast structure. This is only one example why early communication is an essential part of the design process.

Repetition and similarity work best for precast; money is saved because there is less time associated with detailing and fabrication. North Mountain utilizes this principle very well. Each floor is almost identical with similar double tee and girder spans.

North Mountain exclusively uses normal-weight concrete. Due to large structure weight, the seismic load for this building is more than five times than the wind load. It would be a reasonable exercise to analyze the structure using a light-weight concrete, since reducing the weight is the only way to reduce the seismic load.

Pre-cast Concrete Double Tees	
Positives	Negatives
Span to depth ratio Fast construction time Fabricated in a controlled shop setting No fire proofing	Weight Transportation of pieces Labor intensive

System 1: Composite Steel Framing



Composite steel construction allows for shallow steel members compared to non-composite construction because the strength of the concrete floor is used in combination with the steel section. With girder spans up to 48’-0”, non-composite construction was not considered for this report. Even with a composite framing system, steel members must be deep to meet deflection requirements with large spans. A RAM Structural System Model is used to design steel sections for the typical existing bay, 48’0” x 30’-0” and a 30’-0” x 30’-0” bay; Figures 2 and 3 show the preliminary sizes for steel sections of each bay, respectively.

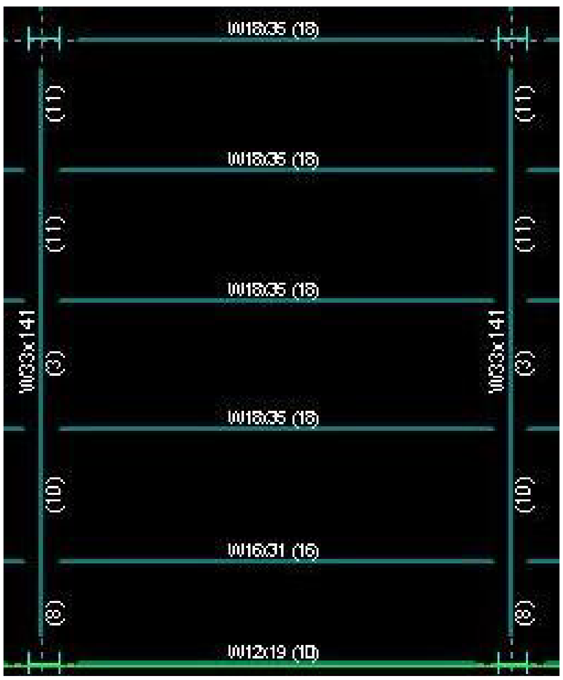


Figure 2

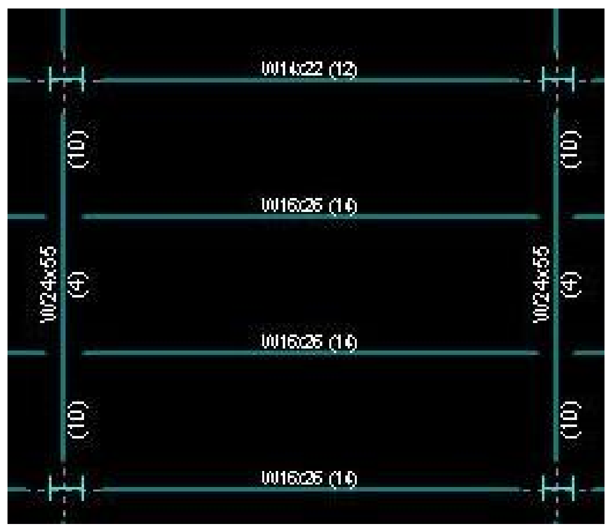


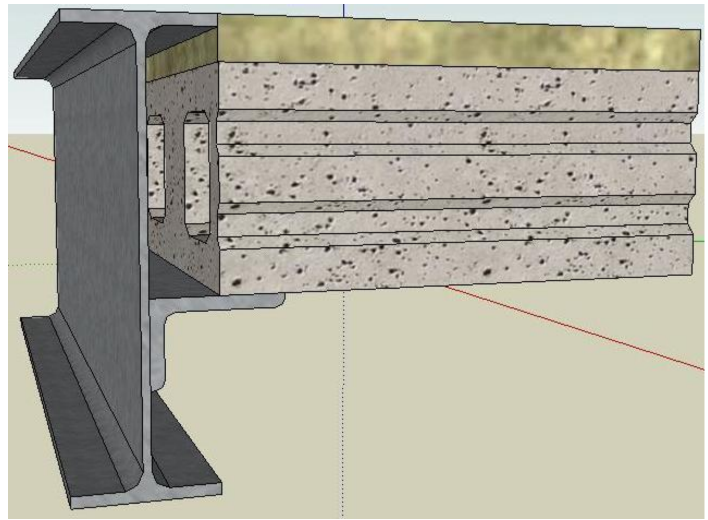
Figure 3

Intermediate beams are spaced at 10’-0” on center with a girder spanning the 48’-0” and 30’-0” dimensions. A difference of 9” (nominal) total floor depth is observed between the two bay sizes. However, there are obviously more columns resulting in more foundations and longer steel erection time, because there are more pieces to connect.

The deck and concrete slab was designed using Vulcraft load tables. The design of the deck can be found on page 15 in the Appendix. A main factor effecting slab design is fire rating. Another factor is unshored span length. It is important that the deck can support construction loads without having to be shored from below; it is an unnecessary addition to labor casts and construction time. In reference to the above criteria and strength requirements, a 2” composite deck with 3-1/4” concrete topping has been selected. This deck is designated as 2VLI19 with t=3-1/4". The combination of deck and slab chosen requires no fire proofing, which will save construction time.

Composite Steel Framing	
Positives	Negatives
Low cost Easy to construct Maintains structural grid Acoustic performance	Heavy steel sections Deep structural sandwich Needs fire proofing

System 2: Hollow-Core Plank

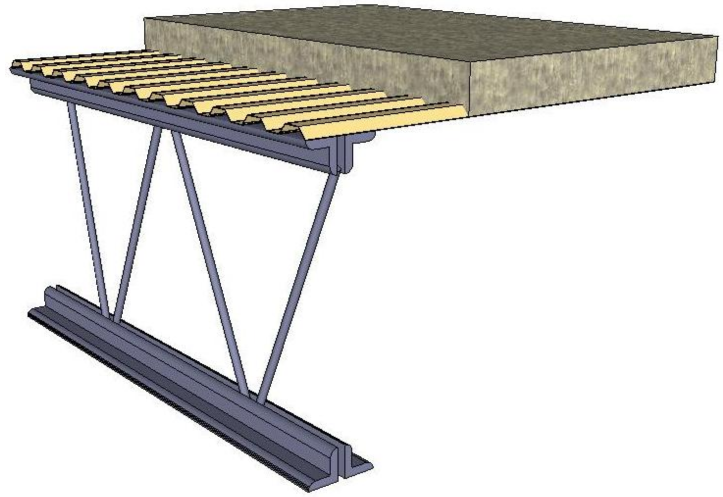


Pre-stressed hollow-core plank provides great span to depth ratio due to the pre-stressing force. Voids in the plank keep them lighter than a regular concrete slab. These voids could also be insulated to prevent thermal and sound transmission. Load tables from the PCI Design Handbook were used to size the plank. However, each hollow-core fabricator has unique section properties and specific details for producing their own product. This could result in slightly different designs from precaster to precaster.

Using the existing bay size, the plank would span 30'-0" and would bear on a large girder. The girder spans 48'-0". Based on load tables, 10" deep planks with 2" topping are required to carry the necessary floor loads. The girder supporting the plank was sized with hand deflection calculations, resulting in a W36x182. The girder is much larger than the composite framing beams due to increased dead load of the hollow-core and the lack of composite action. This calculation is shown on page 16 in the Appendix. A steel angle would have to be welded to the web of the steel girder to support the plank. Normally, the plank rests directly on the top flange of the girder, but due to the large depth of this girder, a different arrangement is necessary.

Hollow-core Plank	
Positives	Negatives
Plank span to depth ratio	Cost
Fabricated in controlled shop setting	High dead load
Quick erection time	Large steel girders
Potential one piece floor and ceiling	
Compatability with other materials	

System 3: Steel Joists



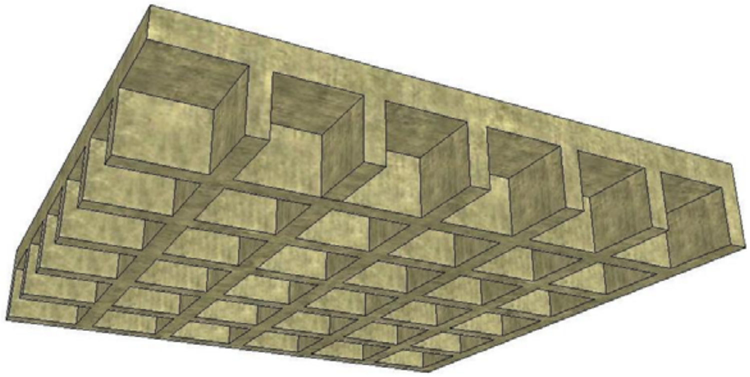
Steel joists provide a great floor system because they are light weight and relatively shallow. This system is so light weight because each member acts as a truss. However, due to this light system, vibration control can become an issue. A vibration analysis is beyond the scope of this report, but would need to be addressed if this system is chosen for future thesis assignments.

The criteria and load table used to design the steel joists can be found on page 17 in the Appendix. Spanning 30’-0” and with 5’-0” joist spacing, a 24LH7 was required to support dead and live loads. Less concrete is needed for the slab, because the joists are spaced closer together than the beams for the composite system. This creates a floor system that is nearly 30 pounds per square foot lighter than the closest competitor.

Once again, a large steel girder is needed to span 48’-0”. With a W36 x 135 and 3.5” of concrete, the total depth of the floor becomes 39.5”. This is a full foot deeper than the existing pre-cast concrete system. Steel joists also tend to be relatively expensive. With joists spaced at 5’-0”, there are a lot of members to be installed and connections to be welded; it becomes a labor intensive job. Unlike precast which also has lots of connections, one pre-cast member accounts for much more of the structural system, and erection time is significantly shorter.

Steel Joists	
Positives	**Negatives**
Light weight	Large steel girder to span 48'
Joist have good span to depth ratio	Vibration resistance
Fabricated in a controlled shop setting	Cost
Qualifies for LEED building design	Overall depth
MEP can run through open webs	Labor intensive
	Requires fire proofing

System 4: Two-way Concrete Waffle Slab



A two-way concrete system is another system that provides a great span to depth ratio. However, 30’-0” x 30’-0” bays were used to size the waffle slab system. This results in more columns in the floor plan but can provide a shorter floor to floor height.

Using CRSI Design Handbook reinforcing a relatively shallow system was selected. The slab is 3” think and the ribs are only 8” deep. The slab reinforcing is divided into column strip and middle strip. The design chart from CRSI is shown on page 18 in the Appendix. Middle strip reinforcing is 2 #6 bars in the bottom and 11 #5 in the slab. Column strip reinforcing is 1-#8 and 1-#7 in the bottom and 23 #6 in the slab.

Two-way Concrete Waffle Slab	
Positives	Negatives
Great span to depth ratio No fire proofing Shallow structural sandwich	Labor intensive Extensive formwork and shoring Changes to sturtural bays Quality control

Summary Comparison Chart

System	Double Tees	Composite Steel		Concrete Waffle Slab	Hollow-core Plank	Steel Joists
Bay Size	48'-0" x 30'-0"	30'-0" x 30'-0"	48'-0" x 30'-0"	30'-0" x 30'-0"	48'-0" x 30'-0"	48'-0" x 30'-0"
*Cost ($/ft.^{2})	$18.92	$18.95	n/a	$21.00	$21.73	$20.55
Erection Time	very fast, long lead time for tee fabrication	fast, long lead time for large steel sections		slow, uses lots of temporary form work and shoring	fast, long lead time for large steel sections	slow, many members to place and attach
Weight (lbs./ft.^{2})	Tees: 47 psf Topping: 40 psf Total: 87 psf	Concrete: 65 psf Deck: 3 psf Framing: 5 psf Total: 73 psf		Slab: 37 psf Beams: 25 psf Total: 59 psf	Topping: 25 psf Plank: 68 psf Framing: 5 psf Total: 98 psf	Concrete: 23 psf Deck: 3 psf Joists: 5 psf Total: 31 psf
Overall Depth of Floor System (in.)	Tees: 24" Topping: 3.25" Total: 27.25"	Concrete: 5.25" Girder: 24" Total: 29.25"	Concrete: 5.25" Girder: 33" Total: 38.25"	Slab: 3" Beams: 8" Total: 11"	Topping: 2" Plank: 10" Girder: 36" Total: 38"	Concrete: 3.5" Joists: 24" Girder: 36" Total: 39.5"
Effects to Other Structural Elements	n/a	connections to exterior walls, possible moment frame		moment frame analysis, increased foundations, exterior wall system	connections to exterior walls	connections to exterior walls
Vibration Resistance	very good	average		very good	above average	below average
Fire Proofing	no additional fire proofing required with 3-1/4" topping	not required with certain types of deck		not required	not required	spray-on fire proofing
Arcitectural Issues	n/a existing system	more columns in office space	deeper floor system	more columns in office space, less deep	much deeper floor	much deeper floor
Viable Solution	Existing System	Yes	No	Yes	No	No

* Cost data obtained from RS Means Assemblies pricing guide.

Conclusions

Based on the above summary comparison chart and design data, it has been determined that there are no other reasonable options at 48’-0” x 30’-0” bays for the systems analyzed within this report. Because of the large spans, the other systems are much deeper than the existing precast double tee system and deemed them unacceptable. In a medical office building, it is important to have a shallow structural sandwich so mechanical, electrical and plumbing systems can fit, and tenants can effectively use the office space.

However, the two systems analyzed with 30’-0” x 30’-0” bays are both acceptable options for redesign. These two systems are the two-way concrete waffle slab and composite steel framing. Despite adding more columns and foundations to the structure, these floor systems would provide a cost effective and efficient floor. North Mountain IMS Office Building is a great example of how prestressed, precast concrete can be very cost effective with long spans and competitive to other systems.

Appendix

Deck and Slab Design

CRITERIA:
- USE COMPOSITE DECK
- NEED 2-HOUR FIRE RATING
- BEAM SPACING 10'-0" O.C.
- OPEN WEB JOIST SPACING 5'-0" O.C.
- UNSHORED CONSTRUCTION
- SERVICE LOAD = 115psf
- LIVE LOAD = 100psf
- LIGHTWEIGHT CONCRETE

FOR COMPOSITE STEEL BEAMS:

USE 2VLI19 w/ t=3¼"

- NO ADDITIONAL FIRE PROOFING REQUIRED

(N=14) LIGHTWEIGHT CONCRETE (110 PCF)

Total Slab Depth	Deck Type	SDI Max. Unshored Clear Span			Superimposed Live Load, PSF Clear Span (ft.-in.)								
		1 Span	2 Span	3 Span	6'-0	6'-6	7'-0	7'-6	8'-0	8'-6	9'-0	9'-6	10'-0
	2VLI22	7'-2	9'-6	9'-8	238	209	186	149	133	120	108	98	90
4"	2VLI21	7'-10	10'-2	10'-6	254	223	198	178	142	128	115	105	96
	2VLI20	8'-5	10'-9	11'-1	268	235	209	187	169	135	122	110	101
(t=2")	2VLI19	9'-6	11'-11	12'-4	297	260	230	206	185	168	153	141	111
	2VLI18	10'-6	12'-10	13'-3	324	285	253	227	205	187	171	158	146
30 PSF	2VLI17	11'-5	13'-8	14'-0	352	308	273	245	221	201	184	169	156
	2VLI16	12'-1	14'-4	14'-4	377	330	292	261	235	214	195	179	165
	2VLI22	6'-9	9'-1	9'-3	276	243	195	173	155	139	126	114	104
4 1/2"	2VLI21	7'-5	9'-9	10'-1	295	259	231	185	165	149	134	122	111
	2VLI20	8'-0	10'-4	10'-8	312	273	243	217	196	157	141	128	117
(t=2 1/2")	2VLI19	9'-0	11'-5	11'-9	346	302	268	239	215	195	178	142	129
	2VLI18	10'-0	12'-3	12'-8	376	331	294	264	238	217	199	183	170
35 PSF	2VLI17	10'-10	13'-1	13'-6	400	358	318	284	256	233	213	196	181
	2VLI16	11'-5	13'-8	13'-10	400	384	340	303	273	248	227	208	192
	2VLI22	6'-6	8'-8	8'-10	315	277	222	197	176	159	144	130	119
5"	2VLI21	7'-1	9'-4	9'-8	337	296	263	211	189	169	153	139	127
	2VLI20	7'-7	9'-11	10'-3	355	312	276	248	199	179	161	146	133
(t=3")	2VLI19	8'-7	10'-11	11'-4	394	345	305	272	245	223	178	162	147
	2VLI18	9'-6	11'-10	12'-2	400	377	335	300	272	247	227	209	168
40 PSF	2VLI17	10'-3	12'-7	13'-0	400	400	362	324	292	266	243	223	207
	2VLI16	10'-11	13'-2	13'-5	400	400	387	346	311	283	258	237	219
	2VLI22	6'-4	8'-6	8'-8	334	268	236	209	187	168	152	138	126
5 1/4"	2VLI21	7'-0	9'-2	9'-6	357	314	279	224	200	180	163	148	135
	2VLI20	7'-6	9'-8	10'-0	377	331	293	263	211	190	171	155	142
(t=3 1/4")	2VLI19	8'-5	10'-9	11'-1	400	366	324	289	260	210	189	172	156
	2VLI18	9'-3	11'-7	12'-0	400	400	355	319	288	263	241	195	179
42 PSF	2VLI17	10'-1	12'-4	12'-9	400	400	384	344	310	282	258	237	219
	2VLI16	10'-8	12'-11	13'-3	400	400	400	367	330	300	274	252	232

FOR OPEN WEB STEEL JOISTS:

USE 1.5VL22 w/ t=2

- SPRAY FIRE PROOFING IS REQUIRED.

(N=14) LIGHTWEIGHT CONCRETE (110 PCF)

Total Slab Depth	Deck Type	SDI Max. Unshored Clear Span			Superimposed Live Load, PSF Clear Span (ft.-in.)									
		1 Span	2 Span	3 Span	5'-0	5'-6	6'-0	6'-6	7'-0	7'-6	8'-0	8'-6	9'-0	9'-6
	1.5VL22	5'-7	7'-5	7'-6	278	247	206	185	167	152	139	124	105	89
3 1/2"	1.5VL21	6'-3	8'-3	8'-5	293	260	233	195	177	161	147	130	110	93
	1.5VL20	6'-8	8'-11	9'-0	305	271	243	220	185	168	154	135	114	97
(t=2")	1.5VL19	7'-6	10'-0	10'-1	329	292	262	237	216	198	167	145	122	104
	1.5VL18	8'-2	10'-8	11'-0	350	311	279	252	230	211	184	153	129	110
26 PSF	1.5VL17	8'-11	11'-4	11'-8	352	312	280	253	231	212	195	163	137	116
	1.5VL16	9'-6	11'-10	12'-3	352	312	280	253	231	212	195	171	144	122

Hollow-core Plank Design

SERVICE LOAD = 115 psf
LIVE LOAD = 100 psf

USE 4HC10+2 w/ 58-S STRAND CODE

4HC10 + 2

[illegible]e of safe superimposed service load (psf) and camber (in.) **2 in. Normal Weight Topping**

[illegible]rand [illegible]nation Code	Span, ft																										
	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46
[illegible]-S	308	287	256	228	204	183	165	148	133	119	107	96	86	74	63	52	43	34	26								
	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2	–0.3	–0.4								
	0.3	0.3	0.3	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2	–0.3	–0.4	–0.6	–0.8	–1.0	–1.2	–1.4	–1.7								
[illegible]-S	317	298	282	267	252	237	219	198	180	163	148	134	120	105	92	80	69	59	50	41	33	26					
	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	0.4	0.3	0.2	0.2	0.1	0.0	–0.1	–0.3	–0.4					
	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.3	0.3	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.5	–0.7	–0.9	–1.2	–1.5	–1.8	–2.1					
[illegible]-S	326	307	291	273	258	246	234	222	212	202	188	171	153	137	122	108	96	84	74	64	55	46	38	31			
	0.5	0.5	0.6	0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.5	0.5	0.4	0.3	0.2	0.1	–0.1	–0.2			
	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.5	0.5	0.4	0.4	0.3	0.2	0.0	–0.1	–0.3	–0.5	–0.7	–0.9	–1.2	–1.5	–1.8	–2.2			
[illegible]-S	335	313	297	279	267	252	240	228	218	208	196	189	181	165	150	135	122	109	97	86	76	67	58	50	42	35	28
	0.6	0.7	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.9	0.9	1.0	1.0	1.0	0.9	0.9	0.9	0.8	0.8	0.7	0.6	0.5	0.4	0.3	0.1	0.0	–0.2
	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.4	0.3	0.2	0.0	–0.2	–0.4	–0.6	–0.9	–1.2	–1.6	–1.9	–2.3	–2.8
[illegible]-S	344	322	306	288	273	258	246	234	221	211	202	195	184	178	172	158	144	130	118	107	96	87	77	68	60	52	44
	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.1	1.1	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.1	1.1	1.0	0.9	0.8	0.7	0.5	0.3
	0.8	0.8	0.9	0.9	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	0.9	0.9	0.8	0.7	0.6	0.4	0.3	0.1	–0.1	–0.3	–0.6	–0.9	–1.3	–1.6	–2.0

STEEL GIRDER DESIGN:

w_{DL} = 93 psf (HOLLOW-CORE + 2" TOPPING)
+ 15 psf (SUPERIMPOSED)

w_{LL} = 80 psf (OFFICE)
+ 20 psf (PARTITION)

GIRDER SPAN = 48'-0"
GIRDER SPACING = 30'-0"

$w_u = 1.2(93+15) + 1.6(80+20)$
$w_u = 269$ psf
$w_u = 269\text{ psf}(30') = 8.07$ klf
$w_{\Delta L} = 100\text{ psf}(30') = 3.0$ klf
$w_{\Delta T} = 208\text{ psf}(30') = 6.2$ klf

DEFLECTION REQUIREMENTS:

$$\Delta_{LL} = \frac{l}{360} = \frac{48(12)}{360} = 1.6'' \qquad \Delta_{TL} = \frac{l}{240} = \frac{48(12)}{240} = 2.4''$$

(LIVE LOAD) $I_{REQ'D} = \frac{5 w_{\Delta} l^4}{384 E \Delta} = \frac{5(3.0)(48)^4(1728)}{384(29{,}000)(1.6)} = 7{,}722 \text{ in}^4$

(TOTAL LOAD) $I_{REQ'D} = \frac{5 w_{\Delta T} l^4}{384 E \Delta} = \frac{5(6.2)(48)^4(1728)}{384(29{,}000)(2.4)} = 10{,}640 \text{ in.}^4$

MAX MOMENT

$$M_u = \frac{w_u l^2}{8} = \frac{(8.07)(48)^2}{8} = 2324 \text{ ft·k}$$

$$Z_{x\,REQ'D} = \frac{M_u}{\phi F_y} = \frac{2324 \times 12}{0.9(50)} = 620 \text{ in}^3$$

USE W36x182

$I_x = 11{,}300 \text{ in}^4$
$Z_x = 718 \text{ in}^3$

<u>Steel Joist Design</u>

LOADS:

LL = 80 psf (OFFICE)
+20 psf (PARTITION)

DL = 26 psf (SLAB & DECK)
15 psf (SUPERIMPOSED)

SPACING: 5'-0"

SPAN = 30'-0"

$w_{LL} = 100 \cdot 5' = 500$ plf

$w_{TL} = 141 \cdot 5' = 705$ plf

USE 24LH7

Joist Type	Allowable Loads (PLF) Total	Allowable Loads (PLF) Live	Joist Weight (lbs./ft.)
29' LENGTH			
6K3	193	106	5.9
8K3	218	136	6.2
6K4	232	124	6.7
0K3	243	170	6.6
8K4	263	159	7.0
0K4	293	199	7.4
8K5	296	179	7.7
2K4	324	242	7.8
0K5	330	223	8.1
2K5	365	272	8.7
6K5	434	384	8.0
4K6	435	354	8.4
8K6	511	486	9.1
8K7	550	522	9.5
18LH5	581	345	14
20LH5	595	395	13
18LH6	648	377	15
24LH6	708	567	14
24LH7	778	623	15
20LH7	786	518	16
24LH8	830	662	16
24LH9	977	775	18
24LH10	1033	822	19
24LH11	1088	861	20
30' LENGTH			
18K3	203	123	6.1
16K4	216	112	6.6
20K3	227	153	6.5
18K4	245	144	6.9
20K4	274	179	7.3
18K5	276	161	7.7
20K5	308	201	8.0
20K6	336	218	8.7
22K6	371	266	8.2
26K5	405	346	8.0
24K6	406	319	8.4
26K6	441	377	8.8
28K6	477	439	9.0
26K7	492	417	9.2
28K7	531	486	9.5
26K8	544	457	10
26K9	550	459	10
20LH5	571	366	13
18LH6	605	340	15
24LH6	684	529	14
24LH7	752	582	15

<u>GIRDER DESIGN</u>

$w_{DL} =$ 15 psf (SUPER.)
5 psf (JOISTS)
26 psf (SLAB & DECK)

$w_{LL} = 100$ psf

SPAN = 48'-0"

SPACING: 30'-0"

$w_u = 1.2(15+5+26) + 1.6(100)$

$w_u = 215$ psf

$w_u = 215(30') = 6.45$ klf

$w_{\Delta L} = 100(30') = 3.0$ klf

$w_{\Delta T} = 46(30') = 1.38$ klf

DEFLECTION:

$$\Delta_{LL} \leq \ell/360 = \frac{48(12)}{360} = 1.6'' \quad \& \quad \Delta_{TL} \leq \ell/240 = \frac{48(12)}{240} = 2.4''$$

(LIVE LOAD) $I_{REQ'D} = \dfrac{5 w_{\Delta L} \ell^4}{384 E \Delta} = \dfrac{5(3.0)(48)^4(1728)}{384(29{,}000)(1.6)} = 7{,}722 \text{ in}^4$

(TOTAL LOAD) $I_{REQ'D} = \dfrac{5 w_{\Delta T} \ell^4}{384 E \Delta} = \dfrac{5(1.38)(48)^4(1728)}{384(29{,}000)(2.4)} = 2{,}368 \text{ in}^4$

MAX MOMENT

$$M_u = \frac{w_u \ell^2}{8} = \frac{6.45(48)^2}{8} = 1860'^k$$

$$Z_x \text{ REQ'D} = \frac{M_u}{\phi F_y} = \frac{1860(12)}{0.9(50)} = 496 \text{ in}^3$$

USE W36x135

$I_x = 7{,}800 \text{ in}^4$

$Z_x = 509 \text{ in}^3$

Two-way Concrete Waffle Slab Design

Span c.-c. Columns $\ell_1 = \ell_2$ (ft)	Factored Super-imposed Load (psf)	Square Edge Column: (1) Steel (psf)	$c_1 = c_2$ (in.)	γ_f	(2) Stirrups	Reinforcing Bars—Each Direction: Column Strip: Top Edge No.-size +	Bottom: No. Ribs	Bars per Rib	Top Interior No.-size	Middle Strip: Bottom: No. Ribs	Long Bars	Short Bars	Top Interior No.-size	Moments: −M Edge (ft-k)	+M Bot. (ft-k)	−M Int. (ft-k)	Square Interior Column: (1) Steel (psf)	$c_1 = c_2$ (in.)	(2) Stirrups	Reinforcing Bars—Each Direction: Column Strip: Bottom: No. Ribs	Bars per Rib	Top Interior No.-size	Middle Strip: Bottom: No. Ribs	Long Bars	Short Bars	Top Interior No.-size
Total Depth = 11 in.	Rib Depth = 8 in.				Total Slab Depth = 3 in.												Total Depth = 11 in.			Rib Depth = 8 in.			Total Slab Depth = 3 in.			
15'- 0" D= 6.500 RIB ON COLUMN LINE 0.545 CF/SF	50	1.88	12	0.654		11-#5+ 0	3	2-#4	11-#5	2	#4	#4	4-#5	15	29	40	1.85	12		3	2-#4	11-#5	2	#4	#4	4-#5
	100	1.88	12	0.675		11-#5+ 0	3	2-#4	11-#5	2	#4	#4	4-#5	20	42	53	1.85	12		3	2-#4	11-#5	2	#4	#4	4-#5
	150	1.88	12	0.696		11-#5+ 0	3	2-#4	11-#5	2	#4	#4	4-#5	24	57	65	1.85	12		3	2-#4	11-#5	2	#4	#4	4-#5
	200	1.88	12	0.717		11-#5+ 0	3	2-#4	11-#5	2	#4	#4	4-#5	29	72	78	1.85	12		3	2-#4	11-#5	2	#4	#4	4-#5
	300	1.99	12	0.760		11-#5+ 0	3	1-#4 and 1-#5	11-#5	2	#4	#5	4-#5	39	102	104	1.85	12		3	2-#4	11-#5	2	#4	#4	4-#5
	400	2.34	12	0.803		11-#5+ 0	3	1-#5 and 1-#6	11-#5	2	#5	#6	4-#5	48	130	130	2.03	12	3 S 4 1	3	1-#4 and 1-#5	11-#5	2	#4	#5	4-#5
18'- 0" D= 6.500 RIB ON COLUMN LINE 0.523 CF/SF	50	1.86	12	0.728		13-#5+ 0	3	2-#4	13-#5	3	#4	#4	5-#5	26	51	69	1.84	12		3	2-#4	13-#5	3	#4	#4	5-#5
	100	1.86	12	0.755		13-#5+ 0	3	2-#4	13-#5	3	#4	#4	5-#5	34	73	92	1.84	12		3	2-#4	13-#5	3	#4	#4	5-#5
	150	1.92	12	0.781		13-#5+ 0	3	1-#4 and 1-#5	13-#5	3	#4	#4	5-#5	43	100	115	1.84	12		3	2-#4	13-#5	3	#4	#4	5-#5
	200	2.03	12	0.807		13-#5+ 0	3	2-#5	13-#5	3	#4	#4	5-#5	51	127	138	1.95	12		3	1-#4 and 1-#5	13-#5	3	#4	#4	5-#5
	300	2.34	12	0.859		13-#5+ 0	3	2-#6	13-#5	3	#5	#5	5-#5	68	179	183	2.05	12	3 S 4 1	3	2-#5	13-#5	3	#4	#4	5-#5
	400	2.75	12 *	0.912	3 S 4 1	13-#5+ 0	3	2-#7	13-#5	3	#5	#6	5-#5	85	229	229	2.40	12 *	3 S 4 1	3	2-#6	13-#5	3	#4	#5	5-#5
21'- 0" D= 9.500 RIB NOT ON COLUMN LINE 0.555 CF/SF	50	1.84	12	0.792		15-#5+ 0	4	2-#4	15-#5	3	#4	#4	6-#5	43	85	115	1.82	12		4	2-#4	15-#5	3	#4	#4	6-#5
	100	1.90	12	0.822		15-#5+ 0	4	1-#4 and 1-#5	15-#5	3	#4	#4	6-#5	56	121	151	1.82	12		4	2-#4	15-#5	3	#4	#4	6-#5
	150	2.06	12	0.853		15-#5+ 0	4	2-#5	15-#5	3	#5	#5	6-#5	70	165	188	1.82	12		4	2-#4	15-#5	3	#4	#4	6-#5
	200	2.36	12 *	0.890		15-#5+ 1	4	2-#6	15-#5	3	#5	#6	6-#5	84	208	225	2.01	12		4	1-#4 and 1-#5	15-#5	3	#4	#5	6-#5
	300	2.94	12 *	0.630		15-#5+ 1	4	2-#7	17-#5	3	#6	#7	6-#5	111	292	298	2.49	12 *	3 S 4 1	4	1-#5 and 1-#6	16-#5	3	#5	#6	6-#5
	400	3.73	16 *	0.625		15-#5+ 0	4	2-#8	15-#6	3	#7	#8	7-#5	135	362	364	3.19	13 *	3 S 4 1	4	1-#6 and 1-#7	14-#6	3	#6	#6	7-#5
24'- 0" D= 9.500 RIB NOT ON COLUMN LINE 0.536 CF/SF	50	1.92	12	0.848		18-#5+ 0	4	1-#4 and 1-#5	18-#5	4	#4	#4	7-#5	63	127	171	1.86	12		4	2-#4	18-#5	4	#4	#4	7-#5
	100	2.14	12 *	0.882		18-#5+ 0	4	1-#5 and 1-#6	18-#5	4	#4	#5	7-#5	84	180	226	1.95	12		4	1-#4 and 1-#5	18-#5	4	#4	#4	7-#5
	150	2.40	12 *	0.915		18-#5+ 0	4	1-#6 and 1-#7	18-#5	4	#5	#5	7-#5	105	247	282	2.05	12 *	3 S 4 1	4	2-#5	18-#5	4	#4	#4	7-#5
	200	2.80	13 *	0.629		18-#5+ 0	4	2-#7	20-#5	4	#6	#6	7-#5	125	310	336	2.36	12 *	3 S 4 1	4	2-#6	18-#5	4	#4	#5	7-#5
	300	3.72	18 *	0.623		18-#5+ 1	4	1-#8 and 1-#9	26-#5	4	#6	#7	9-#5	162	412	435	3.24	14 *	3 S 4 1	4	2-#7	17-#6	4	#5	#6	8-#5
27'- 0" D= 9.500 RIB NOT ON COLUMN LINE 0.523 CF/SF	50	2.06	13	0.870		20-#5+ 0	4	1-#5 and 1-#6	20-#5	5	#4	#4	8-#5	90	180	242	1.94	13		4	1-#4 and 1-#5	20-#5	5	#4	#4	8-#5
	100	2.29	13 *	0.914		20-#5+ 1	4	1-#6 and 1-#7	20-#5	5	#4	#5	8-#5	119	246	321	2.02	13	3 S 4 1	4	2-#5	20-#5	5	#4	#4	8-#5
	150	2.83	15 *	0.930		20-#5+ 4	4	1-#7 and 1-#8	24-#5	5	#5	#6	8-#5	148	339	398	2.42	13 *	3 S 4 1	4	2-#6	22-#5	5	#4	#5	8-#5
	200	3.51	18 *	0.623		20-#5+ 1	4	1-#8 and 1-#9	20-#6	5	#6	#6	9-#5	175	426	472	3.10	13 *	3 S 4 1	4	2-#7	19-#6	5	#5	#5	9-#5
30'- 0" D=12.500 RIB ON COLUMN LINE 0.545 CF/SF	50	2.12	15 *	0.879		22-#5+ 1	5	1-#5 and 1-#6	22-#5	5	#4	#5	9-#5	125	249	336	1.94	15		5	1-#4 and 1-#5	22-#5	5	#4	#4	9-#5
	100	2.62	16 *	0.923		22-#5+ 5	5	1-#6 and 1-#7	26-#5	5	#5	#6	9-#5	164	329	443	2.31	15 *	3 S 4 1	5	1-#5 and 1-#6	24-#5	5	#4	#5	9-#5
	150	3.30	22 *	0.921		22-#5+ 8	5	1-#7 and 1-#8	23-#6	5	#6	#6	11-#5	201	437	541	3.02	15 *	3 S 4 1	5	1-#6 and 1-#7	22-#6	5	#5	#5	10-#5

NOTES *Factored shear stress for the column size so designated is greater than $4\sqrt{f'_c}$ but less than $6\sqrt{f'_c}$. Shear reinforcement, structural steel shearheads, or shear caps must be provided with columns indicated.

(1) Average reinforcing steel weight (psf) includes twice the flexural reinforcement tabulated (tabulated reinforcement placed in each direction). Shrinkage and temperature reinforcement, integrity reinforcement, and shear reinforcement are not included.

(2) The notation "3 S 4 1" for the stirrups: the first character "3" is the bar size, i.e., #3; the "S" stands for "stirrup"; and the "4" is the spacing of the stirrups in inches. The last character indicates the length over which stirrups are required in each joist rib; the "1" denotes the length from the face of the solid head to the first cross rib, i.e., a length of one module; a "2" indicates stirrups are required to the second cross rib from the face of the solid head, i.e., a length of two modules.

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