THE FORENSIC MEDICAL CENTER

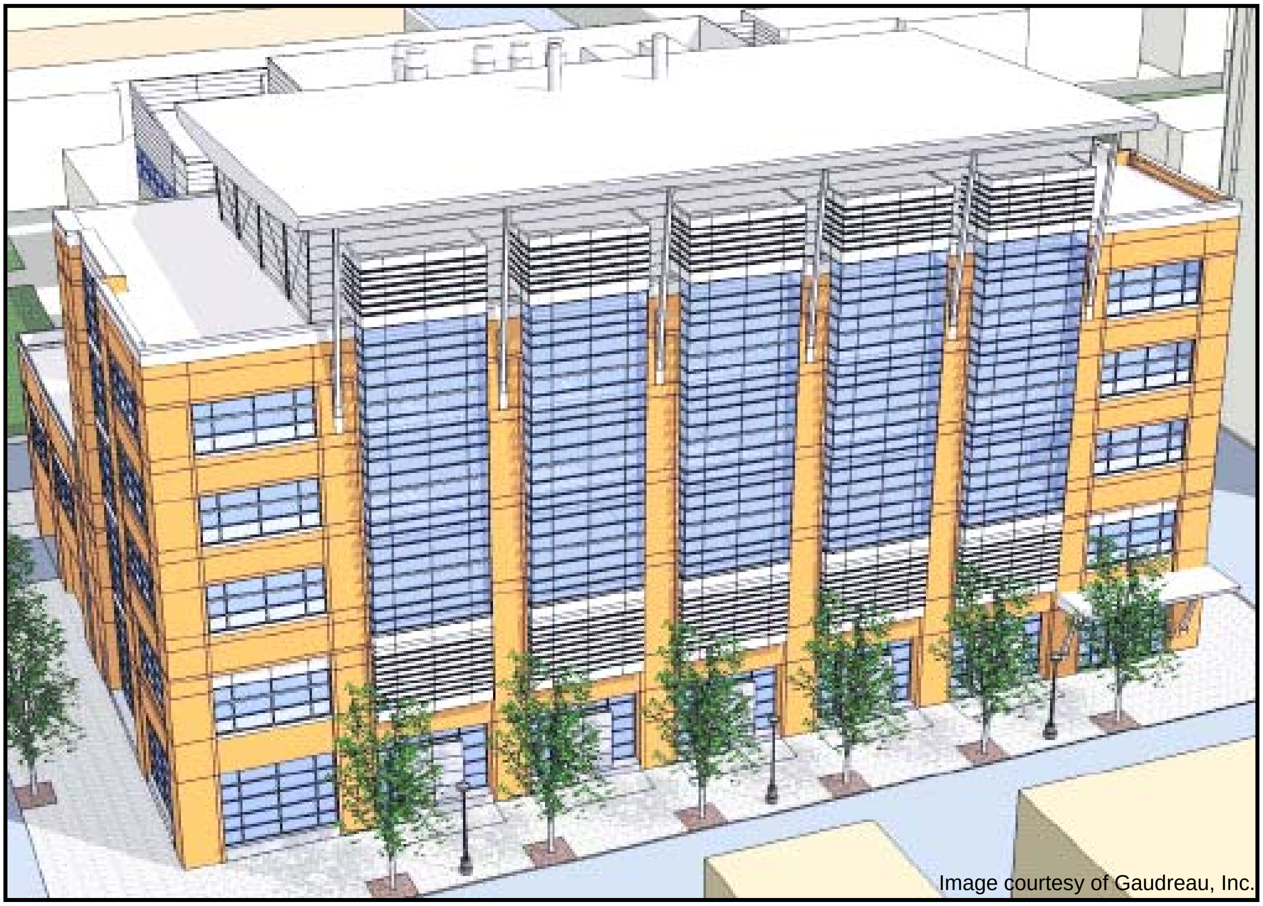

Image courtesy of Gaudreau, Inc.

TECHNICAL REPORT #2
OCTOBER 26, 2007

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STRUCTURAL OPTION

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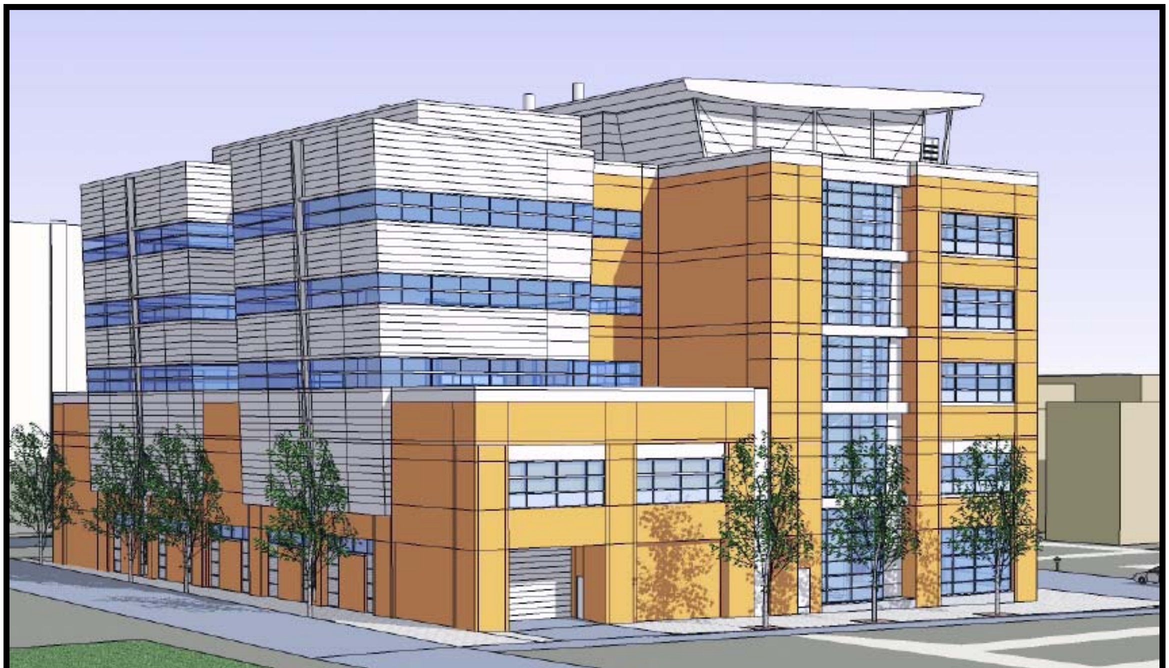
EXECUTIVE SUMMARY



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This report is an investigation of four alternative structural floor framing systems for the Forensic Medical Center:

-Composite Steel
-Two-Way Post-Tensioned Concrete
-Hollow-Core Precast Concrete Plank
-Concrete Waffle Slab

These floor systems were designed for a typical three-bay by three-bay interior section of the building. The systems were then analyzed for constructability, depth, weight, and cost. Another important factor in a high-tech laboratory building such as the Forensic Medical Center is vibration. The floor systems were not analyzed for vibration in this report, but consideration was given to typical vibration characteristics of the systems chosen.

The report concludes that the existing two-way, flat-plate concrete slab seems to be the best overall floor system for the application when taking in to account all of the factors mentioned above. However, the composite steel, post-tensioned, and waffle slab systems have some advantages over the existing system that warrant a further investigation into their capabilities to resist vibration.

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EXISTING FRAMING SYSTEM DETAILS

Columns

All of the columns in the building are normal weight concrete with a strength of 5000 psi. Typically, the columns are 24" by 24", except for ten 34" diameter circular columns in the parking garage area. Exterior columns are reinforced with eight #8 bars and #4 ties at 12" on center. Interior columns are reinforced with eight #10 bars and #4 ties at 12" on center.

Forty columns span from the foundation to the Penthouse level floor, a total of 82 feet. Four columns span 38 feet from the foundation to the low roof at the third level slab. Also at the third level, two columns are shifted 7 feet towards the center of the building (Columns D-2 and D-7). The new columns continue to the Penthouse level floor slab. At the third level, they rest on 36" by 36" transfer beams which span between the two adjacent columns (D-2 to D-3; D-6 to D-7). These transfer beams are reinforced with ten #11 bars on the top and ten #9 bars on the bottom, tied by double #4 closed stirrups at 4' on center.

Slabs

The ground floor parking garage level of the Forensic Medical Center is a 6" thick, normal weight concrete slab-on-grade, reinforced with #5 bars at 12" on-center each way. Concrete strength is 3500 psi. At the edges of this slab are concrete grade beams that are 30"-36" deep, with concrete strength of 3000 psi. The grade beams are reinforced with four #8 bars, five #9 bars, or five #10 bars, and #4 stirrups at 12" on center, depending on location.

The floor systems of levels two through five are typically 11" thick, two-way, flat-plate, normal weight concrete slabs with 26" wide by 36" deep concrete perimeter beams, reinforced with five #10 bars typically, with #4 stirrups at 8" on center. Slab reinforcement is typically #5 bars at 15" on center, each way, top and bottom at mid-span, with heavier reinforcement at the columns. Typical slab spans range from 22'-6" to 30'-0".

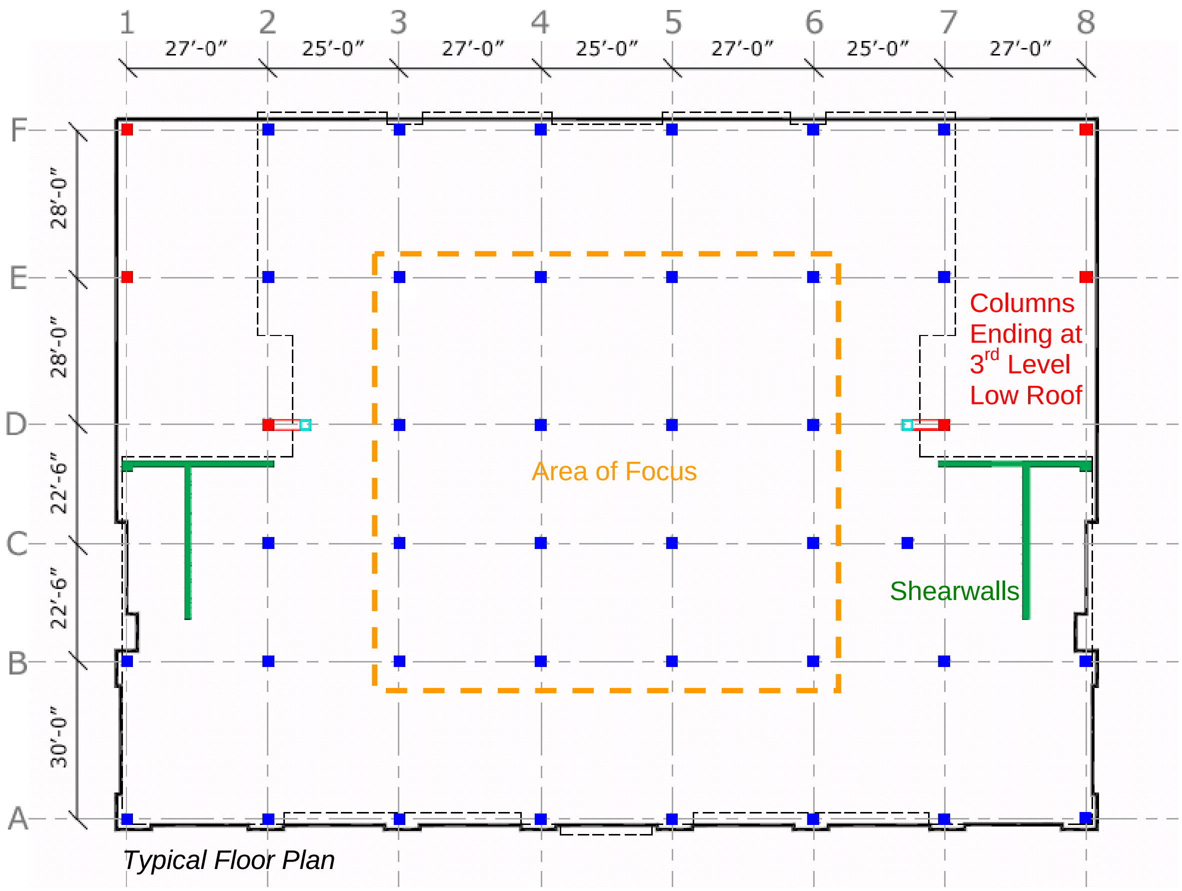
Level two contains large recessed slab areas for body storage coolers and freezers. The finished floor elevation of these slabs is 10" lower than the typical finished floor elevation. These slabs are 11" thick, one-way slabs, and are supported by monolithically-poured concrete beams with sizes ranging from 18" to 40" wide by 11" to 26" deep.

Aside from the typical 11" two-way slab, level three also has two 9" thick, two-way slab sections that serve as low roofs. A high-density file storage area requires two 24"x18" concrete beams under the mid-span of the slabs, between grid lines 3 and 4.

The Penthouse level floor slab consists of two areas. The roof areas are an 8" thick, two-way, flat-plate, normal weight concrete slab with #5 bars, typically spaced at 16", each way, top and bottom for reinforcement. The slab under the mechanical equipment is increased to 15" thick, with #5 bars at 11" each way, top and bottom, for typical reinforcement.

The loading schedule shows a typical live load of 80 psf, which includes a 20 psf partition allowance, and a superimposed dead load of 8 psf for mechanical, plumbing, and lighting, which was rounded up to 10 psf for these investigations.

This report will focus on the typical interior three-bay by three-bay area between gridlines 3 and 6, and gridlines B and E.



Typical Floor Plan

Because this building houses high-tech laboratories, vibration criteria are very important when considering alternative structural framing systems. Other considerations include cost, weight, and overall depth of the floor system.

ALTERNATIVE #1 – COMPOSITE STEEL

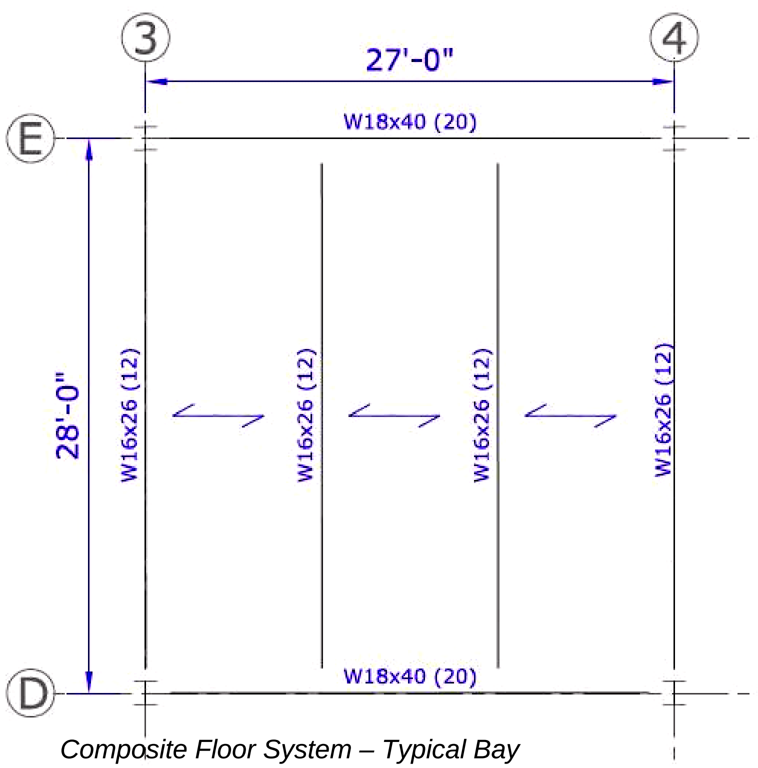
Description

Composite steel is a very popular structural framing system because it combines the tensile strength of steel with the compressive strength of concrete. The result is a relatively stiff system that is shallower than a non-composite steel system and lighter than concrete alone.

For this report, a typical 27' x 28' bay was designed for composite steel. The W18x40 girders run the 27' direction and have (20) ¾"-diameter shear studs each. The intermediate W16x26 beams span the 28' direction, and have (12) ¾"-diameter shear studs on each beam. The 1.5" 18 gauge LOK-Floor composite steel deck spans the 9' between each beam. A 2.5"-thick concrete topping slab creates a total floor depth of 20" at the beams, and 22" at the girders.

The beam sizes were found to be controlled by deflection, while the girder sizes were controlled by flexural strength. Deflections were limited to L/240 for total load, L/360 for live load, and L/240 for pre-composite action dead load.

This system would require a change to steel columns. Since the original concrete columns are 24" x 24", any typical W10, W12, or W14 steel column would easily fit within the original column footprint without disrupting the layout of the building spaces.



Composite Floor System – Typical Bay

Advantages:

Composite construction combines concrete and steel to create a system that is in between the two. It is lighter than a concrete system, which improves seismic performance and could allow for a reduction in foundation size. It isn't as deep as a non-composite steel system, allowing for less building height and more mechanical space.

Composite systems eliminate the need for formwork and shoring that are required for concrete construction, helping to reduce costs as well as speed up erection times.

Disadvantages:

Because of the use of steel beams and girders, composite construction requires a longer lead time so these components can be milled and shipped to the building site.

The composite steel system is deeper than the existing concrete slab system, which will increase the overall height of the building and make it more difficult to install the mechanical, electrical, and plumbing systems.

The vibration criteria of the equipment in the building may require even deeper beams or a thicker slab in the laboratory areas than what was used for this design.

Fire protection, in the form of a spray-on cementitious material or layers of gypsum board, must be provided on the underside of the steel deck, as well as the beams, girders, and columns.

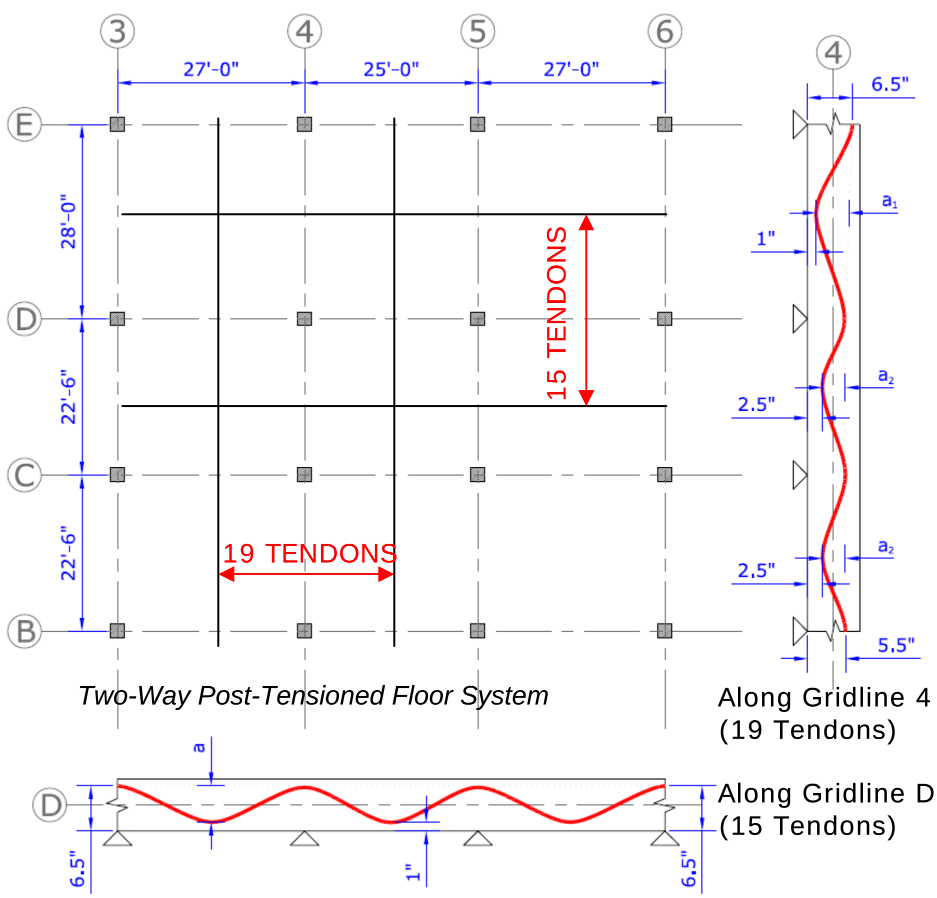
-See Appendix A for Composite Steel Calculations-

ALTERNATIVE #2 – TWO-WAY POST-TENSIONED SLAB

Description

Post-tensioning is a construction method that helps to make up for concrete's weakness in tension. Steel tendons are "draped" throughout the slab; they are placed in the top of the slab at supports, and in the bottom at midspan. After the concrete gets a chance to cure sufficiently, the tendons are stretched, imparting an upward force that works to counteract a portion of the dead load of the slab (usually 60%-80%). This allows for a thinner slab than traditional reinforced concrete because deflections are much smaller, due to this balancing of the majority of the dead load.

Using a general rule of thumb, a slab thickness of 7.5" was used for this design. Unbonded, ½"-diameter, seven-wire post-tensioning tendons were selected to balance 80% of the self-weight of the slab. Two typical three-bay sections were designed; one in each direction. Along gridline D, the tendon drape is 5.5" in each bay in each of the 15 tendons, for a total prestress force of 399 k, a balanced load of 2.14 k/ft in the 27' spans (or 90% of the self-weight), and a balancing load of 2.34 k/ft in the 25' span (98% of the self-weight). The design along gridline 4 requires 19 tendons supplying a total prestressing force of 505 k, with a 5" drape in the 28' span, and a 3" drape in the 22'-6" span. This creates a balancing load of 2.15 k/ft (88% of w_{self}) in the longer span, and 2.00 k/ft (82% of w_{self}) in the shorter spans. In addition to the tendons, conventional bonded reinforcement was required at several supports and in several spans. A summary is available in Appendix B.



Two-Way Post-Tensioned Floor System

Along Gridline 4
(19 Tendons)

Along Gridline D
(15 Tendons)

Advantages

Because of how the prestressing steel is utilized, a post-tensioned slab can span long distances with a much thinner slab than would be needed for a conventional reinforced concrete slab. This allows for a lower overall building height and increased mechanical space.

A thinner slab means less concrete is needed. As a result, concrete costs will be lower, also, the dead load will be smaller. This leads to better seismic performance, and possibly a reduction in foundation sizes.

Disadvantages:

Because of the trial-and-error method of post-tension design and the possibility of many acceptable designs for the same situation, designing a two-way post-tensioned slab is very difficult and time consuming. Nearly every individual bay would need to be examined and designed separately.

The actual jacking of the post-tensioning tendons cannot be done until the concrete has had time to cure, usually to at least 60% of its 28-day strength. As a result, construction times are slower than for many other floor systems.

Since this system is a flat-plate concrete slab, formwork and shoring are required as in the existing conventional two-way reinforced concrete slab.

The location of the high-tension tendons in the slab can vary because of bay geometries and slab openings, making it very difficult to cut openings into a post-tensioned slab. For this reason, most slab openings must be planned into the design and layout of the slab to avoid severing or bursting these tendons.

This thin, light-weight slab may not be sufficient to meet the vibration criteria of the sensitive laboratory equipment in the building.

This system does not require punching shear reinforcement when only gravity loads are analyzed. Lateral loads would probably add to the shear stresses enough, however, to require some sort of shear reinforcement at the column-slab connections.

-See Appendix B for Two-Way Post-Tensioned Slab Calculations-

ALTERNATIVE #3 – HOLLOW-CORE PRECAST PLANK

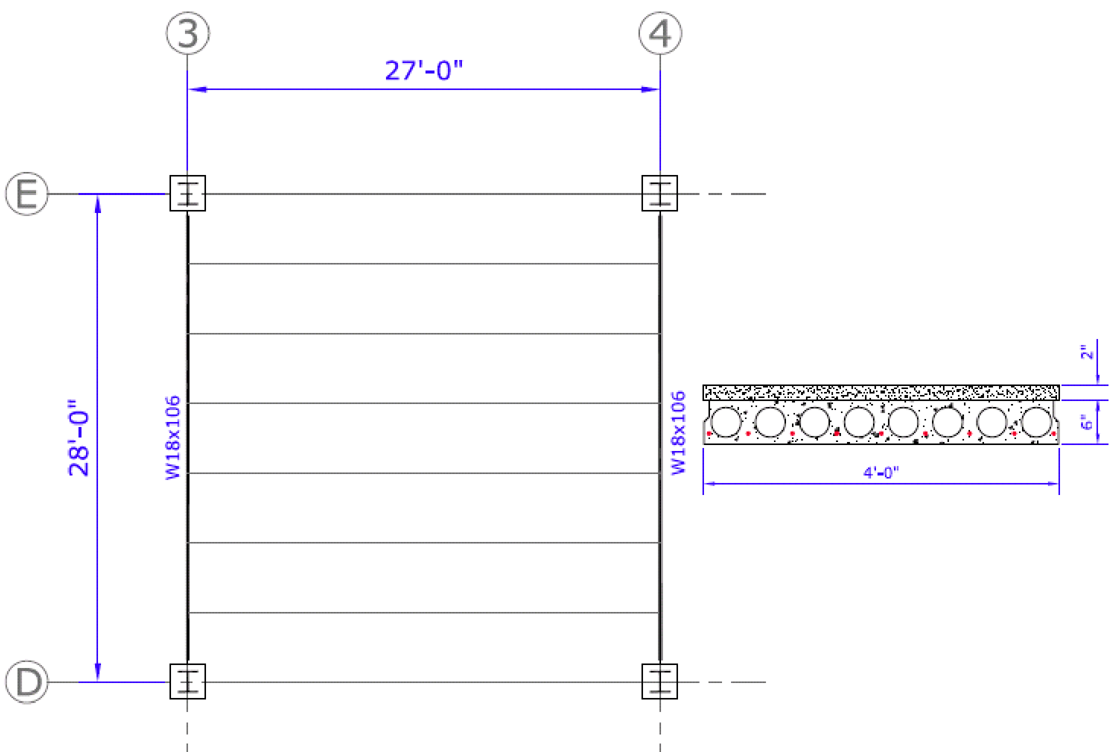
Description

Precast concrete systems are most popular in low- and mid-rise apartment buildings and other residential applications, as well as parking decks, and hospitals. These systems work best in buildings with regular square or rectangular bays. The hollow-core planks used in this system come in 4'-wide sections, so it was necessary to alter the column grid to fit this module.

The Precast Concrete Institute manual was used to select the thinnest available hollow-core plank that could support the loads in the Forensic Medical Center. A 4"-thick plank with a 2" concrete topping was selected, with strand designation code 87-S, for a 27' span. This designates (8) 7/16" straight prestressing strands.

To support the planks, a steel wide-flange girder was selected. Because these girders span in the 28' direction and are not braced against lateral torsional buckling, they are fairly large W18x106 members.

This system would require a change to steel columns. Since the original concrete columns are 24" x 24", any typical W10, W12, or W14 steel column would easily fit within the original column footprint without disrupting the layout of the building spaces.



Hollow-Core Plank Floor System – Typical Bay

Advantages:

Because the precast planks are already designed for flexure, shear, and deflection, the design of a precast system is much quicker and simpler than other systems.

The planks are poured and cured in a controlled environment off-site, so there is less uncertainty in the concrete's strength properties than for a field-cured slab.

Using a precast system eliminates the need for formwork and shoring on-site, and allows for quicker erection times, because there is no need to wait for the concrete to cure.

The shallow depth of the planks, in addition to their hollow cores, allow for very flexible mechanical, electrical, and plumbing space.

The planks are much lighter than a solid concrete slab, which decreases the structure dead load, leading to better seismic performance and a possible reduction in footing size.

Disadvantages:

Precast systems work best for buildings with regular bays, based on a 4' module. Irregular bays create a need for custom shapes and sizes of planks, which can drive up the cost of the project and make erection more complicated.

Because the planks must be poured and cured off-site, then shipped to the site, a longer lead time is required than for cast-in-place concrete.

Even though the planks themselves allow for flexible mechanical spaces, the deep steel girders that support them can get in the way of these ducts and pipes. These girders must also be protected from fire by using spray-on fireproofing or gypsum board.

Because the planks are so thin and hollow, this system will most likely not meet the vibration criteria required for the sensitive laboratory equipment that will be used in this building.

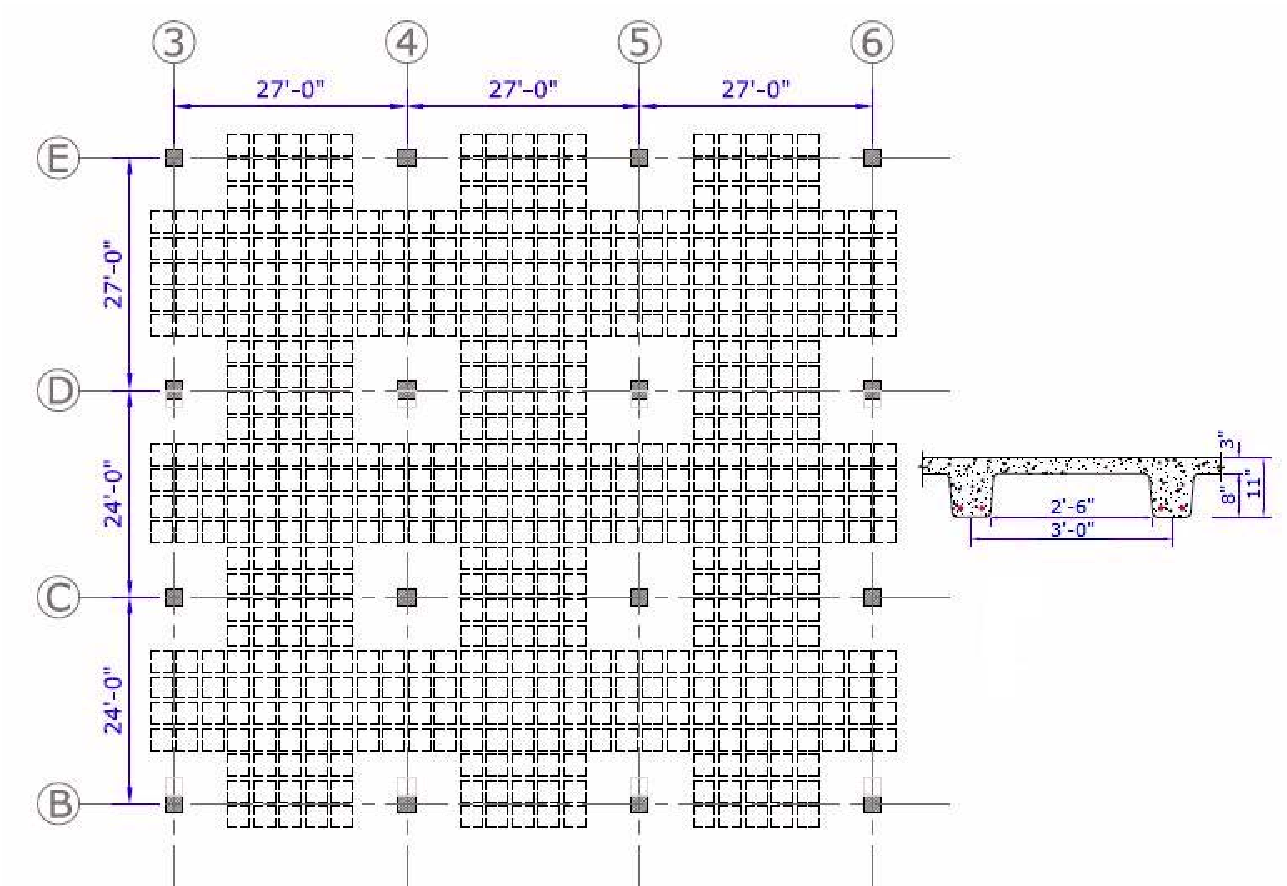
-See Appendix C for Hollow-Core Plank Calculations-

ALTERNATIVE #4 – CONCRETE WAFFLE SLAB

Description

Concrete waffle slabs are similar to the existing two-way concrete slab system, except for 30” x 30” voids in the underside of the floors. These voids create 6” ribs at three foot intervals, into which the steel reinforcement is placed. This eliminates the concrete in areas where it would not be supporting any load because it is in tension, helping to reduce the amount of concrete needed and therefore reduce the dead load of the structure.

To use this design most efficiently, the bay dimensions need to be in three-foot modules. The column grid was adjusted to allow for this. The maximum span with this geometry is 27’. A slab with a total depth of 11” was selected from the Concrete Reinforcing Steel Institute Design Handbook. This depth was selected because it is equal to the existing two-way flat-plate slab depth. This design requires two #6 bars per rib in the bottom of the column strips and a #4 long bar and a #5 short bar in each rib in the bottom of the middle strips. In the top of the slab, #5 bars are required, spaced at 7” on center in the column strip and 20” on center in the middle strip.



Waffle Slab Floor System

Advantages:

Waffle slabs are more efficient than similar flat-plate slabs because they consolidate the positive moment reinforcement into ribs, and eliminate the concrete that is useless in tension in between. This reduces the amount of concrete required, which can help reduce costs. It also reduces the dead weight of the floor, allowing for better seismic performance.

The overall thickness of this system is the same as that of the existing two-way flat-plate slab, allowing for the same amount of mechanical space.

Disadvantages:

The formwork for a waffle slab is complicated, consisting of 36" x 36" fiberglass pans that must be set up on shoring.

While unique, custom-sized forms can be created for unusual or irregular bay sizes, the waffle slab system works best for spans in modules of either 2' or 3', as the pans are available in 24" x 24" or 36" x 36" pan sizes.

Because this system is lighter than the existing two-way flat-plate system, it may not be sufficient to meet the vibration criteria of the sensitive laboratory equipment in the building.

-See Appendix D for Concrete Waffle Slab Calculations-

CONCLUSIONS

The strict vibration criteria of a high-tech laboratory building seriously limit the number of floor framing systems that can be used. Generally, stiffer floor systems perform better. Stiffness can be increased by increasing the mass of the system or the depth of the system. The existing two-way concrete slab seems to be the best combination of mass and depth for this application. It is not extremely deep, yet it still has a large amount of mass to dampen vibrations.

That being stated, several of the systems analyzed in this report could also be investigated further to see whether they meet the vibration criteria as designed, or whether they can be re-designed to meet the criteria and still be constructed faster and more cheaply than the existing system. These include the composite steel, two-way post-tensioned, and concrete waffle slab systems. The precast system was eliminated because it would require too many changes to the existing column grid, was the most expensive system analyzed, and would need to be much thicker to control vibrations enough to be considered.

The composite steel system is the deepest of these feasible systems. Using this system would add 9” to 11” to the floor thickness, adding up to nearly five feet of overall height to the building. This could hurt the seismic and wind performance of the building. Also, the cost of this system is higher than the other viable systems. The main advantage of composite steel is the ease and speed of construction.

Using the post-tensioned system would save 3.5” of floor height per story. This would save roughly a foot and a half in total height of the building, which is probably not enough to make a large difference in wind loads or seismic performance. The main advantage of this system is that it uses roughly 30% less concrete than the existing system. It’s cost is slightly lower than the existing system because of this reduction in concrete. Less concrete also reduces dead load, which improves seismic performance, and can lead to reduced foundation sizes. The disadvantage of post-tensioning is the complexity involved in designing and implementing the system. Also, as mentioned above, this system may not have enough depth or mass to meet the required vibration criteria. Further investigation would be required to check this.

To use the waffle slab design, regularity in the column grid needs to be established. This is the main drawback of this system. The elimination of the unused concrete helps significantly to reduce dead loads, improving seismic performance and allowing for smaller foundation sizes. In this case, however, the system would need to be analyzed for vibrations to make sure it meets the required criteria as designed.

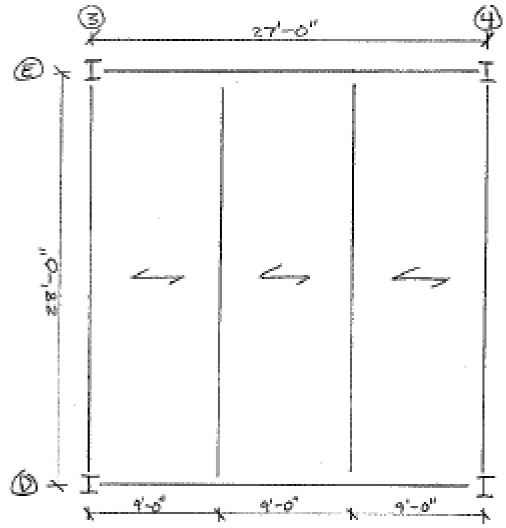
COMPARISON CHART

SYSTEM	DESCRIPTION	COST / FT^2	TOTAL DEPTH	ADVANTAGES	DISADVANTAGES	VIABLE FOR FURTHER RESEARCH
Existing System	11" 2-Way Slab	$10.54	11"	-Shallow floor system depth -Space for Mech./Elec. -Redistribution of Loads -Limits vibration	-Formwork required -Long construction time -Reinforcement detailing	
Alternative #1	Composite	$14.65	20" / 22" @ girders	-Lighter than concrete alone -No Formwork required -Limits vibration	-Long lead times for steel -Large floor system depth -Fire protection required on steel	**YES**
Alternative #2	7.5" 2-Way PT	$9.82	7.5"	-Thin floor system depth -Lighter than existing system	-Difficult to design -Longer construction time -Difficult to cut openings -Formwork required	**YES**
Alternative #3	Hollow-Core Plank	$15.58	8" / 26" @ girders	-Easier to design -Fast erection time -Flexible Mech/Elec. space	-Deep girders required -Fire protection required on steel -Bay size adjustments required -Vibration issues	NO
Alternative #4	Waffle Slab	$9.79	11"	-Lighter than solid slab -Less concrete w/ same thickness -Flexible Mech/Elec. space	-Complicated formwork -Bay size adjustments required -Noise/Vibration issues	**YES**

APPENDIX

COMPOSITE STEEL CALCULATIONS

COMPOSITE STEEL — TYPICAL BAY



CONCRETE: NORMAL WT: 150 pcf
$f'_c = 5000$ psi

STEEL:
$f_y = 50$ ksi

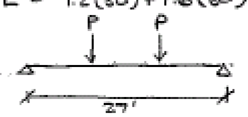
LIVE LOAD: 60 psf LABS
20 psf PARTITIONS
80 psf

SUPERIMPOSED DEAD LOAD: 10 psf

— TRY 1.5" LOK-FLOOR (18 ga.) w/ 2.5" TOP (4" TOTAL THICKNESS)
$w_D = 39$ psf SLAB + DECK + 10 psf SUPERIMPOSED ≈ 50 psf

$w_u = 1.2D + 1.6L = 1.2(50) + 1.6(80) = 188$ psf

— GIRDER DESIGN:



$b_{eff} = \text{MIN} \begin{cases} \text{SPACING} = 336" \\ \frac{1}{4}\ \text{SPAN} = 81" \leftarrow \end{cases}$

DEAD LOAD:
50 psf × (9' × 14') = 6.3 k
(PER BEAM, ×2 BEAMS)

LIVE LOAD:
80 psf × (9' × 14') = 10.1 k
(PER BEAM, ×2 BEAMS)

$P_u = 1.2(6.3 \times 2) + 1.6(10.1 \times 2)$

$P_u = 47.4$ k

$$M_u = P_u \times a = 47.4 \times 9'$$

$$M_u = 427 \text{ ft} \cdot \text{k}$$

— GIRDER DEFLECTION $\Delta = \frac{P\ell^3}{28EI}$

TOTAL: $\frac{32.8 \times (27 \times 12)^3}{28(29000)\, I_{LB_{MIN}}} = \frac{(27 \times 12)}{240} \Rightarrow I_{LB_{MIN}} = 1018 \text{ in}^4$

LIVE: $\frac{20.2 \times (27 \times 12)^3}{28(29000)\, I_{LB_{MIN}}} = \frac{(27 \times 12)}{360} \Rightarrow I_{LB_{MIN}} = 940 \text{ in}^4$

CONST. DEAD PRECOMPOSITE: $\frac{12.6 \times (27 \times 12)^3}{28(29000)\, I_{PRE_{MIN}}} = \frac{(27 \times 12)}{240} \Rightarrow I_{PRE_{MIN}} = 391 \text{ in}^4$

— W18×40 @ PNA ⑧: $I_{LB} = 1090 \text{ in}^4$, $I_{PRE} = 612 \text{ in}^4$

— W18×40 @ PNA ⑥: $\phi M_n = 438 \text{ ft} \cdot \text{k} > 427 \text{ ft} \cdot \text{k}$ OK, $\Sigma Q_n = 210$ k

$a = \frac{210}{0.85 \times 5 \times 81} = 0.61" < 1"$ OK

COMPOSITE STEEL (CONT.)

BEAM	PNA	# STUDS	STEEL WT.	EQ. STUD WT.	TOT. EQ. WT.
W16x40	4	30	1080	300	1380
W16x45	6	20	1215	200	1415
W18x40	6	20	1080	200	1280

— ASSUMING COST OF 1 STUD AS COST OF 10 lb OF STEEL

— USE W18x40 GIRDERS

TRIB. WIDTH = 9'-0" ⇒ $w_u = 188 \times 9 = 1.7$ k/ft

1.7 k/ft

28'

$$M_u = \frac{w l^2}{8} = \frac{1.7(28)^2}{8} = 167 \text{ k}\cdot\text{ft}$$

$b_{eff} = \text{MIN} \begin{cases} \text{TRIB WIDTH} = 108'' \\ 1/4 \text{ SPAN} = 84'' \leftarrow \end{cases}$ ASSUME $a = 1.0''$

$$Y_2 = 4 - \frac{a}{2} = 3.5''$$

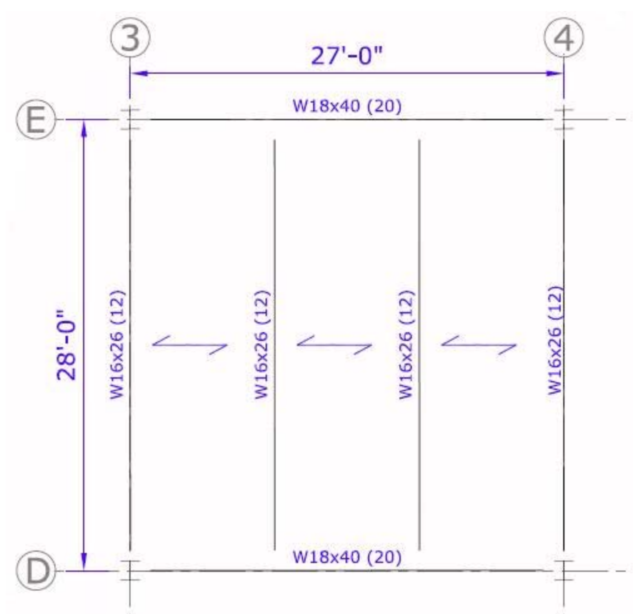
— BEAM DESIGN FOR: $M_u = 167$ ft·k
$I_{LB} = 409$ in^4, $I_{PRE} = 154$ in^4
MAX DEPTH: W16 (ALLOW FOR SINGLE COPE)

— W16x26 @ PNA 7: $I_{LB} = 499$ in^4, $I_{PRE} = 301$ in^2
$\phi M_n = 234$ ft·k, $\Sigma Q_n = 96.0$ k ($a < 1''$ ok)
w/ $Q_n = 17.2$ k ⇒ $\frac{96.0}{17.2}$ ⇒ 6 STUDS / SIDE

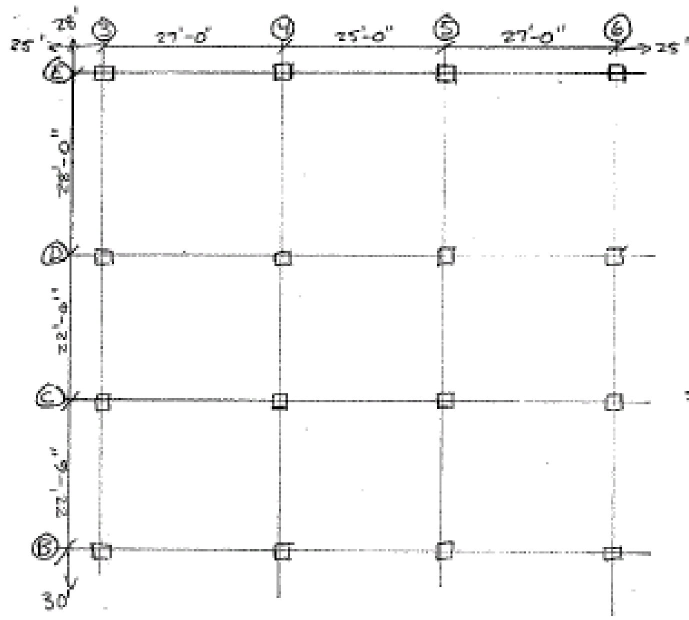
OR NON-COMPOSITE W16x36: $\phi M_n = 240$ ft·k
$I = 448$ in^4
NO STUDS REQ'D

BEAM	PNA	# STUDS	STEEL WT.	EQ. STUD WT.	TOT. EQ. WT.
W16x26	7	12	728	120	848
W16x36	—	—	1008	0	1008

— USE W16x26 PNA 7 w (12) 3/4" ϕ SHEAR STUDS



TWO-WAY POST-TENSIONED SLAB CALCULATIONS



$f'_c = 5000$ psi
$f'_{ci} = 3000$ psi
NORMAL WT. CONC. 150 pcf

LL: 60 psF LABS
$\underline{20 \text{ psF}}$ PARTITIONS
80 psF

SUPERIMPOSED DL: 10 psF

REBAR:
$f_y = 60000$ psi

→ POST-TENSIONING: UNBONDED TENDONS: 1/2" Ø, 7-WIRE STRANDS
$A = 0.153 \text{ in}^2$

$f_{pu} = 270$ ksi

EST. PRESTRESS LOSSES: 15 ksi (ACI 318-05: 18.6)

$f_{se} = 0.7(270) - 15 = 174$ ksi

$P_{eff} = A \times f_{se} = 0.153 \times 174 = 26.6$ k/TENDON

−PRELIMINARY SLAB THICKNESS — ASSUME $LL/DL < 1.0$

USE $L/h = 45$ ⇒ LONGEST SPAN = 28'-0"

$28 \times 12 / h = 45 \Rightarrow h = 7.47 \Rightarrow 7.5"$

−LOADING: DL: $150 \text{ pcf} \times \frac{7.5}{12} = 94$ psF SLAB
$+ 10$ psF SUPERIMPOSED
104 psF

LL: 80 psF $LL/DL < 1.0$ OK, $LL/DL > 0.75$
→ PATTERN LOAD TEST REQ'D

$1.2D + 1.6L = 253$ psF

−FRAME ALONG GRID LINE 4 $A = bh = (312)(7.5) = 2340 \text{ in}^2$

$S = \frac{bh^2}{6} = \frac{(312)(7.5)^2}{6} = 2925 \text{ in}^3$

POST-TENSIONED TWO-WAY SLAB (CONT.)

– DESIGN PARAMETERS: ALLOWABLE STRESSES

• AT TIME OF JACKING:

$f'_{ci} = 3000$ psi

COMP. $= 0.6 f'_{ci} = 1800$ psi

TENS. $= 3\sqrt{f'_{ci}} = 164$ psi

• AT SERVICE LOADS: $f'_c = 5000$ psi

COMP. $= 0.45 f'_c = 2250$ psi

TENS. $= 7.5\sqrt{f'_c} = 530$ psi

– AVG. PRECOMPRESSION LIMITS $P/A = 125$ psi MIN (ACI 18.12.4)
$= 300$ psi MAX

– TARGET LOAD BALANCES: 60–80% OF SELF-WEIGHT <TRY 80%>

$0.80\, w_{SELF} = 75$ psf

– COVER REQUIREMENTS: 3/4" TOP & BOTTOM (RESTRAINED SLABS)

– 28' – SPAN: PRESTRESS FORCE TO BALANCE 80% OF w_{SELF}

$w_{bal} = 0.80\, w_{SELF} = 0.80(94)(26') = 1.96$ k/ft

$P = \frac{1.96(28')^2}{8(5/12)} = 461$ k

$461/26.6 = 17.3 \Rightarrow$ TRY 19 TENDONS $P_{ACT} = 505$ k

ACTUAL $w_{bal} = \frac{505}{461} 1.96 = 2.15$ k/ft (98% OF w_{SELF} OK)

– 22'-6" SPANS: $P = \frac{1.98(22'-6")^2}{8(3/12)} = 496\text{ k} < 505\text{ k}$

$w_{bal} = \frac{505(8)(3/12)}{(22'-6")^2} = 2.00$ k/ft $\Rightarrow$ 82% OF w_{SELF} OK

– USE SAP TO CALCULATE WORST CASE MOMENTS IN FRAME

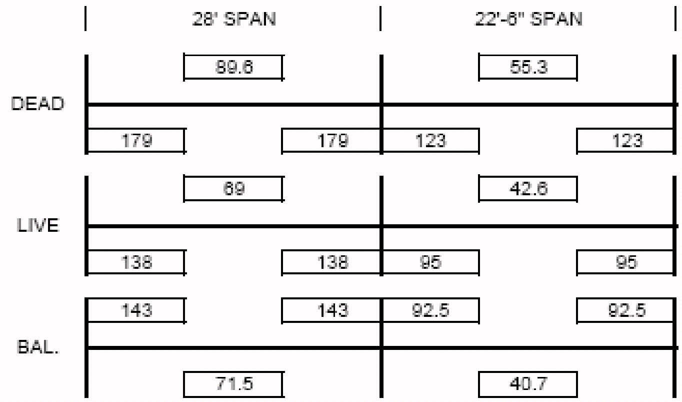
CASES: DEAD LOAD

LIVE LOAD (PATTERNED LOADING REQ'D)

BALANCED LOAD

POST – TENSIONED TWO – WAY SLAB (CONT.)

MOMENTS



SECTION PROPERTIES: $A = 2340\ in^2$ $\quad S = 2925\ in^3$ $\quad P = 505\ k$

STRESSES AT JACKING:

Max. Comp. Stress=	1800 psi
Max. Tens. Stress=	164 psi

@ MIDSPAN: $f_{top} = (-M_{DL}+M_{bal})/S - P/A$

$f_{bot} = (+M_{DL}-M_{bal})/S - P/A$

28' SPAN:	f_{top} =	-290 psi	OK
	f_{bot} =	-142 psi	OK
22'-6" SPAN:	f_{top} =	-276 psi	OK
	f_{bot} =	-156 psi	OK

@ SUPPORTS: $f_{top} = (+M_{DL}-M_{bal})/S - P/A$

$f_{bot} = (-M_{DL}+M_{bal})/S - P/A$

28' SPAN:	f_{top} =	-68 psi	OK
	f_{bot} =	-364 psi	OK
22'-6" SPAN:	f_{top} =	-91 psi	OK
	f_{bot} =	-125 psi	OK

STRESSES AT SERVICE LOADS:

Max. Comp. Stress=	2250 psi
Max. Tens. Stress=	530 psi

@ MIDSPAN: $f_{top} = (-M_{DL}-M_{LL}+M_{bal})/S - P/A$

$f_{bot} = (+M_{DL}+M_{LL}-M_{bal})/S - P/A$

28' SPAN:	f_{top} =	-573 psi	OK
	f_{bot} =	142 psi	OK
22'-6" SPAN:	f_{top} =	-450 psi	OK
	f_{bot} =	19 psi	OK

@ SUPPORTS: $f_{top} = (+M_{DL}+M_{LL}-M_{bal})/S - P/A$

$f_{bot} = (-M_{DL}-M_{LL}+M_{bal})/S - P/A$

28' SPAN:	f_{top} =	498 psi	OK
	f_{bot} =	-930 psi	OK
22'-6" SPAN:	f_{top} =	299 psi	OK
	f_{bot} =	-731 psi	OK

POST-TENSIONED TWO-WAY SLAB (CONT.)

- ULTIMATE STRENGTH

- PRIMARY PT MOMENTS $M_1 = Pe$, $e = \begin{cases} 2.75'' \text{ @ 28' SPAN SUPPORTS} \\ 2.75'' \text{ @ 28' SPAN MIDSPAN} \\ 2.00'' \text{ @ 22'-6'' SPAN SUPPORTS} \\ 1.25'' \text{ @ 22'-6'' SPAN MIDSPAN} \end{cases}$

	Primary PT Moments	=Pe		Secondary PT Moments $= M_{bal} - M_1$	
		e	M_1	M_{bal}	M_2
28'	Support	2.75	116	143	27
	Midspan	2.75	116	71.5	-44
22.5'	Support	2	84	92.5	8
	Midspan	1.25	53	40.7	-12

	Factored Moments	$Mu = 1.2D + 1.6L + 1.0M_2$
		Mu
28'	Support	408
	Midspan	262
22.5'	Support	291
	Midspan	146

- MIN. BONDED REINFORCEMENT

- POSITIVE MOMENT REGION - 28'-SPAN: $f_t = 142$ psi $> 2\sqrt{f'_c} = 141$ psi
→ BONDED REINFORCEMENT REQ'D

$$y = \left[f_t / (f_t + f_c) \right] h = \left[142 / (142 + 573) \right] 7.5 = 1.49''$$

$$N_c = \frac{M_{DL+LL}}{S} \times \frac{1}{2} y l_2 = \frac{(79.6 + 69)(12)}{2925} \times \frac{1}{2}(1.49)(26 \times 12) = 151 \text{ k}$$

$$A_{s_{MIN}} = N_c / \tfrac{1}{2} f_y = \frac{151}{\tfrac{1}{2}(60)} = 5.03 \text{ in}^2 \Rightarrow 0.19 \text{ in}^2/\text{ft}$$

USE #4 BARS @ 12'' o.c. ($A_s = 0.20$ in²/ft)
@ MIDDLE 1/3 OF SPAN

- POSITIVE MOMENT REGION - 22'-6'' SPAN: $f_t = 19$ psi $< 2\sqrt{f'_c} = 141$ psi
→ NO BONDED REINF. REQ'D.

- NEGATIVE MOMENT REGION - USE WORST CASE: 28'-SPAN

$$A_{cf} = \text{MAX} \begin{cases} 7.5'' \times \dfrac{(28' \times 12) + (22'\text{-}6'' \times 12)}{2} = 2273 \text{ in}^2 \\ 7.5'' \times (26' \times 12''/') = 2340 \text{ in}^2 \leftarrow \end{cases}$$

$A_{s_{MIN}} = 0.00075 A_{cf} = 1.76$ in²

USE (10) #4 BARS ($A_s = 2.00$ in²)
- MUST SPAN 1/6 OF SPAN ON EACH SIDE OF SUPPORT
- PLACE WITHIN 1.5h = 11.25'' OF SUPPORT ON EACH SIDE
- MAX BAR SPACING IS 12''

POST – TENSIONED TWO – WAY SLAB (CONT.)

– CHECK MIN. REINF. FOR ULTIMATE STRENGTH: SUPPORTS

$M_n = (A_s f_y + A_{ps} f_{ps})(d - a/2)$, $d = 6.5$ $a = \frac{A_s f_y + A_{ps} f_{ps}}{0.85 f'_c\ b}$

$A_{ps} = 2.91\ in^2$

$f_{ps} = 196\ ksi$ $a = 0.52''$

$\phi M_n = \frac{0.9}{12}\left[(2.00 \times 60) + (2.91 \times 196)\right]\left(6.5 - \frac{0.52}{2}\right) = 323\ ft \cdot k < 408\ ft \cdot k$

→ ULTIMATE STRENGTH REINFORCEMENT GOVERNS

$\frac{408 \times 12}{0.9} = \left[(A_s \times 60) + (2.91 \times 196)\right]\left(6.5 - \frac{(A_s \times 60) + (2.91 \times 196)}{2 \times 0.85 \times 5 \times (26 \times 12)}\right)$

$A_{s\,REQ'D} = 5.19\ in^2$ ⇒ (12) #6 BARS ($A_s = 5.28\ in^2$)

– CHECK MIN. REINF. FOR ULTIMATE STRENGTH: MIDSPAN

– 28' – SPAN: $d = 6.5$

$a = \frac{(5.03 \times 60) + (2.91 \times 196)}{0.85 \times 5 \times (26 \times 12)} = 0.66''$

$\phi M_n = \frac{0.9}{12}\left[(5.03 \times 60) + (2.91 \times 196)\right]\left(6.5 - \frac{0.66}{2}\right) = 404\ ft \cdot k > 262\ ft \cdot k$ OK

→ MINIMUM REINFORCEMENT OK

– 22'-6" SPAN: $d = 5.0$

$a = \frac{(2.91 \times 196)}{0.85 \times 5 \times (26 \times 12)} = 0.43''$

$\phi M_n = \frac{0.9}{12}\left[(2.91 \times 196)\right]\left(5.0 - \frac{0.43}{2}\right) = 205\ ft \cdot k > 146\ ft \cdot k$ OK

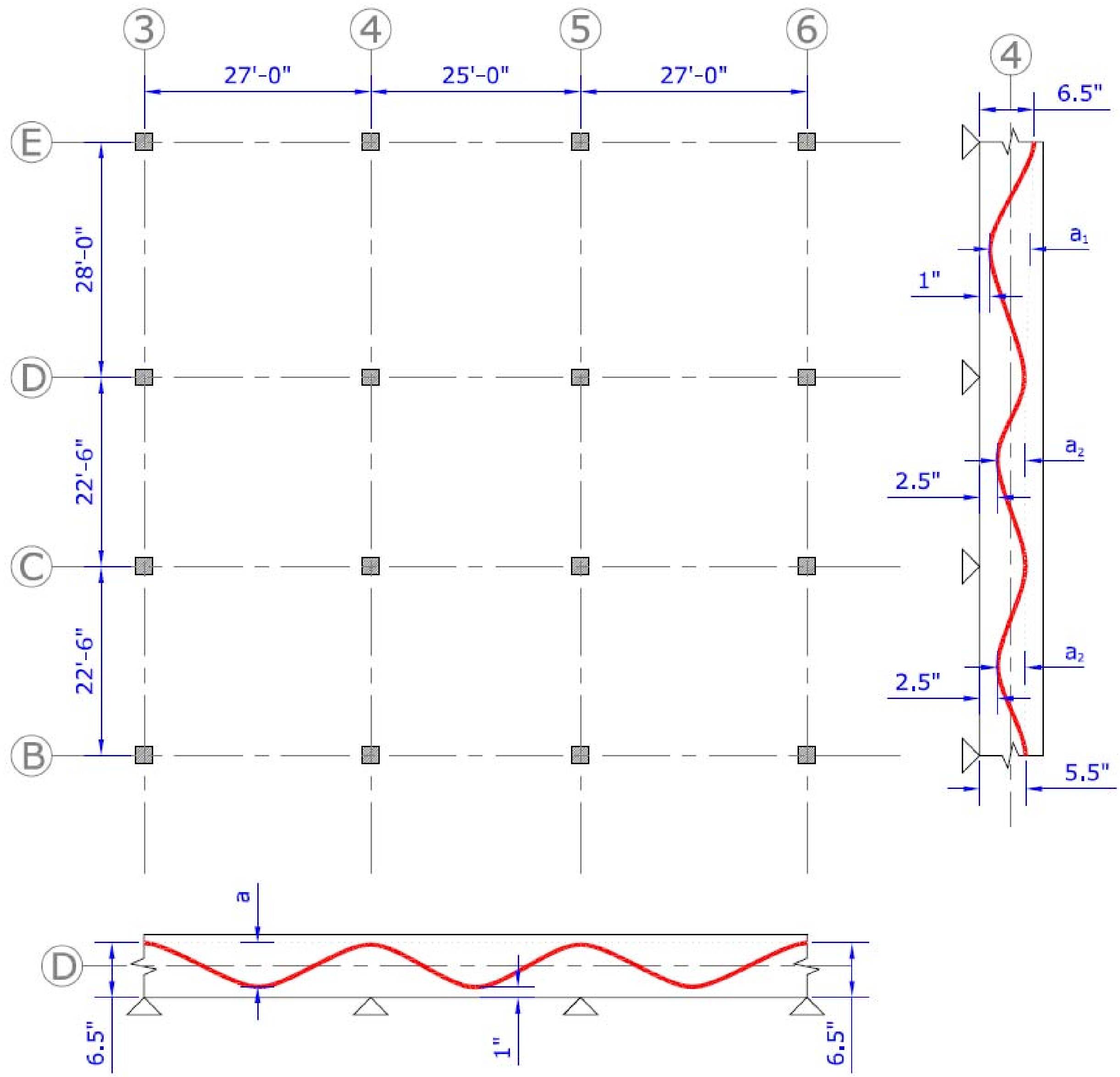
→ NO BONDED REINFORCEMENT REQ'D.

Post-Tension Strand Summary:

Frame D – (15) ½"-diameter, 7-wire strands
Frame 4 – (19) ½"-diameter, 7-wire strands

Bonded Reinforcement Summary:

		Negative Moment	Positive Moment
Frame D	27' Span	(14) #5	#4 @ 10" o.c.
	25' Span	(14) #5	Not Required
Frame 4	28' Span	(12) #6	#4 @ 12" o.c.
	22'-6" Span	(12) #6	Not Required


3
4
5
6
27'-0"
25'-0"
27'-0"
E
D
C
B
28'-0"
22'-6"
22'-6"
4
6.5"
a_1
1"
a_2
2.5"
a_2
2.5"
5.5"
D
a
6.5"
1"
6.5"

HOLLOW-CORE PRECAST PLANK CALCULATIONS

FOR SUPERIMPOSED DL OF 10 psF
AND LIVE LOAD OF 60 + 20 = 80 psF

TOTAL SERVICE LOAD = 90 psF

FROM PCI DESIGN HANDBOOK — PG 2-31

USE 4'-0" × 6" NORMAL WEIGHT CONCRETE PLANK w/ 2" TOPPING

— DESIGNATION 87-S — 94 psF LOAD

[87-S — 8 STRANDS 7/16" Ø STRAIGHT]

0.6" CAMBER AT ERECTION
—0.3" CAMBER LONG-TERM

— STEEL GIRDERS SPANNING N-S DIRECTION

WORST CASE: GIRDER D3 – E3

LOADING: DEAD: 74 psF + 10 psF = 84 psF
LIVE: 60 psF + 20 psF = 80 psF

$1.2D + 1.6L = 1.2(84) + 1.6(80) = 229$ psF

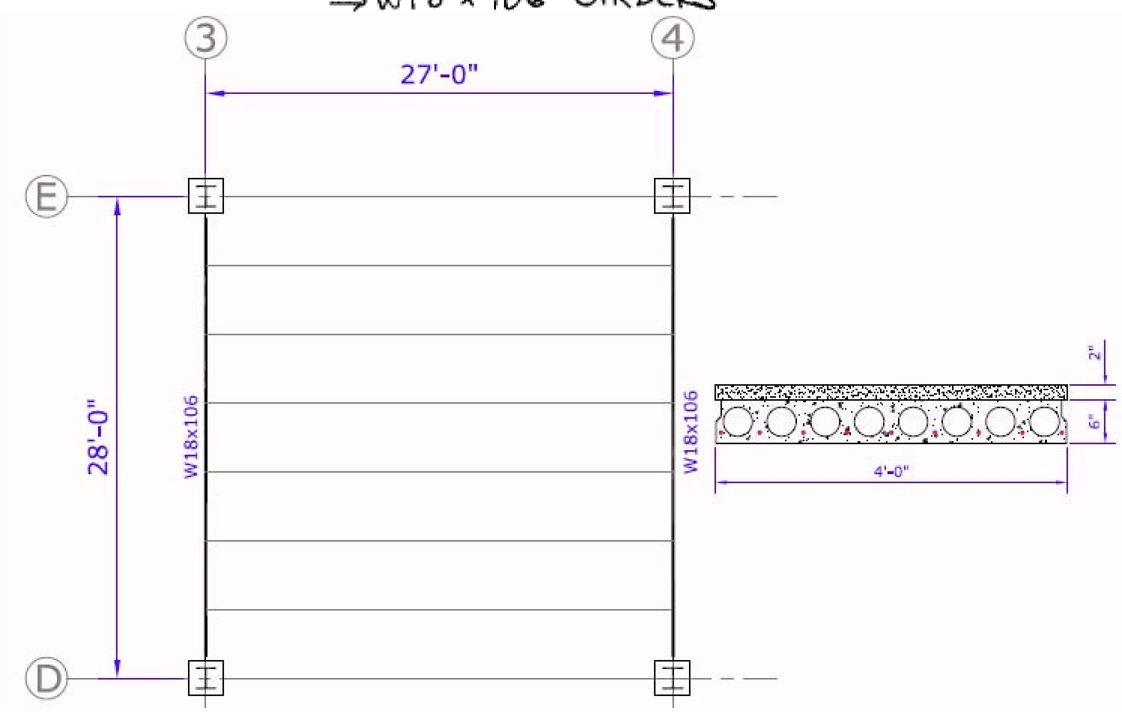
TRIB WIDTH: $26' \times 229$ psF $= 5.95$ k/ft

$M_u = \frac{w_u \ell^2}{8} = \underline{583 \text{ ft·k}}$ @ $L = 28'$

$\Delta_{LL\,MAX} = \frac{\ell}{360} = 0.93'' = \frac{5(2.08/12)(28\times12)^4}{384(29000)\,I_{min}} \Rightarrow I_{min} = 1067 \text{ in}^4$

$\Delta_{TL\,MAX} = \frac{\ell}{240} = 1.40'' = \frac{5(4.26/12)(28\times12)^4}{384(29000)\,I_{min}} \Rightarrow I_{min} = 1451 \text{ in}^4$

→ W18 × 106 GIRDERS



4HC6 + 2

Table of safe superimposed service load (psf) and cambers (in.)

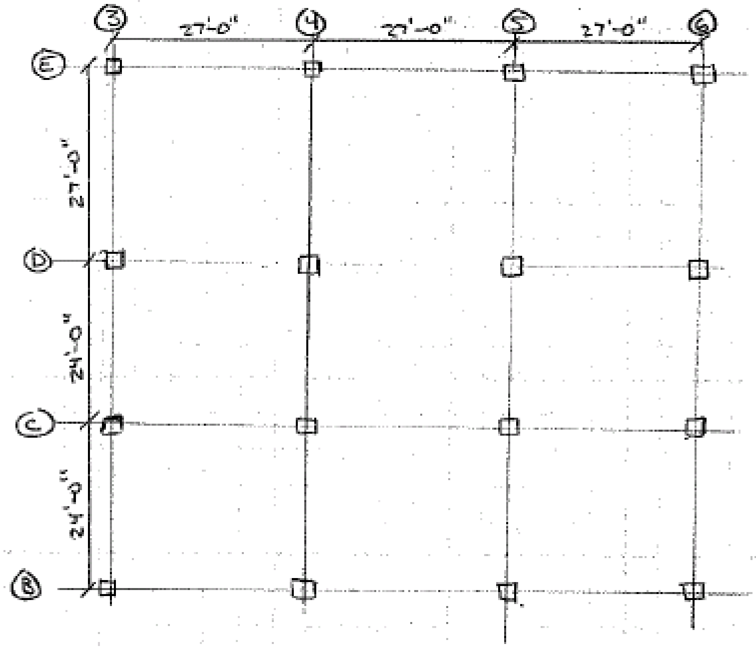
2 in. Normal Weight Topping

Strand Designation Code	Span, ft																		
	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
66-S	470	396	335	285	244	210	182	158	136	113	93	75	59	46	34				
	0.2	0.2	0.2	0.2	0.2	02	0.2	0.2	0.2	0.2	0.1	0.1	0.0	−0.1	−0.2				
	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	−0.1	−0.2	−0.3	−0.5	−0.7	−0.9	−1.2				
76-S		461	391	334	287	248	216	188	163	137	115	95	78	63	50	38	27		
		0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0.1	−0.0	−0.1	−0.3		
		0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	−0.2	−0.3	−0.5	−0.7	−0.9	−1.2	−1.5		
96-S			473	424	367	319	279	245	216	186	160	137	116	98	82	68	55	43	33
			0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.1	0.0	−0.1
			0.4	0.4	0.4	0.4	0.4	0.4	0.3	0.3	0.2	0.1	−0.1	−0.3	−0.5	−0.7	−1.0	−1.4	−1.7
87-S			485	446	415	377	331	292	258	224	195	169	147	127	109	94	80	67	55
			0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.7	0.8	0.8	0.7	0.7	[illegible]	0.6	0.5	0.4	0.3
			0.5	0.5	0.5	0.6	0.6	0.6	0.5	0.5	0.4	0.4	0.2	[illegible]	−0.1	−0.3	−0.5	−0.8	−1.2
97-S			494	455	421	394	357	327	288	251	219	192	168	146	127	110	95	82	70
			0.5	0.6	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.9	1.0	0.9	0.9	0.9	0.8	0.7	0.6
			0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.5	0.4	0.2	0.0	−0.2	−0.5	−0.8

From Pg 2-31 of PCI Design Handbook

CONCRETE WAFFLE SLAB CALCULATIONS

– COLUMN GRID MUST BE ADJUSTED TO FIT



– MULTIPLES OF 3'

– USE 30" x 30" WAFFLE SLAB SYSTEM

FACTORED LOADS, (SUPERIMPOSED DL + LL)

1.2 (10 psf) + 1.6 (80 psf) = 140 psf

MAX SPAN: 27'-0"

– USE SMALLEST AVAILABLE DEPTH: TOTAL: 11"
RIB DEPTH: 8"
SLAB DEPTH: 3"

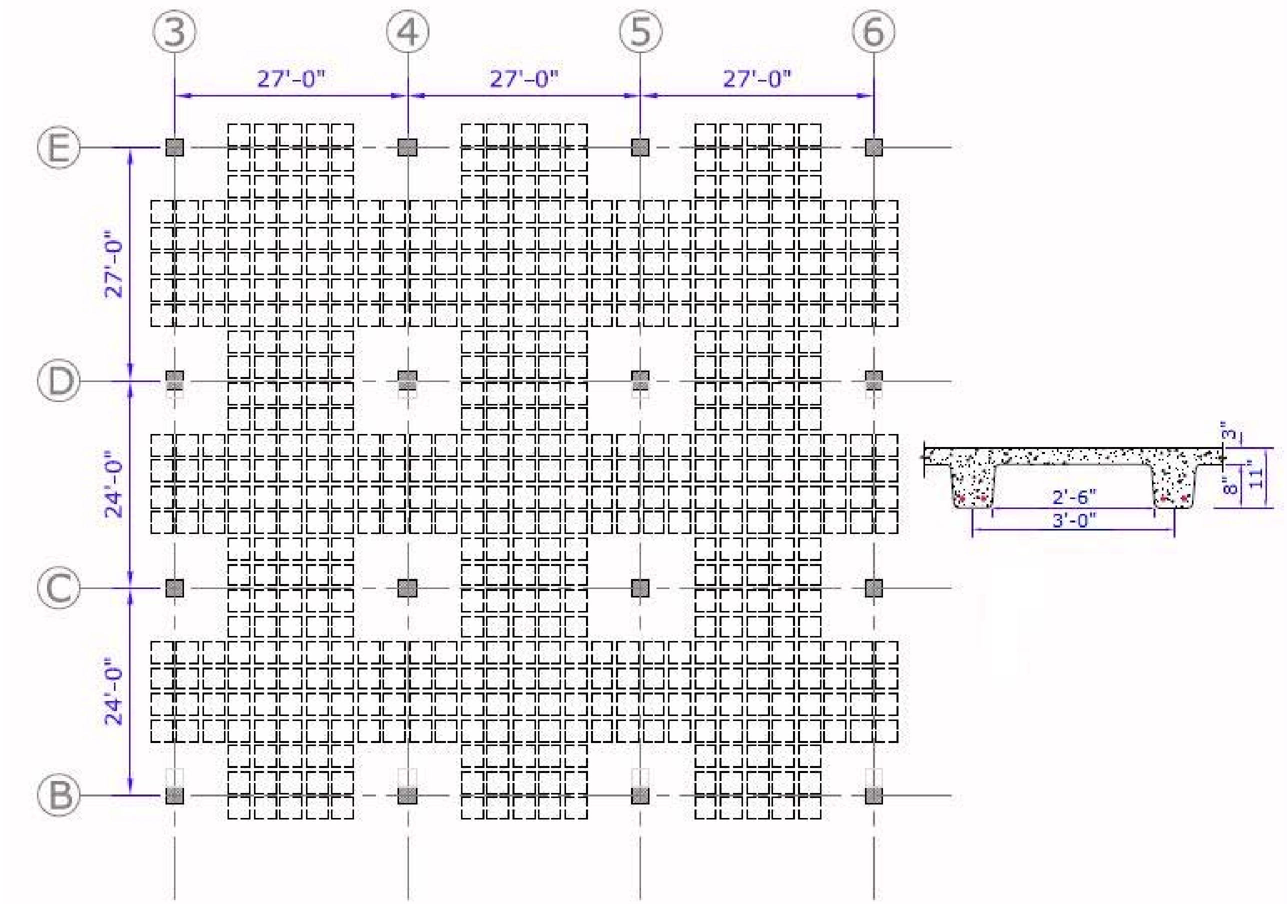
– INTERIOR PANELS
COLUMN STRIP: (2) #6 BARS PER RIB BOTTOM
#5 BARS @ 7" o.c. TOP

MIDDLE STRIP: #4 LONG BAR & #5 SHORT BAR IN EACH RIB BOTTOM
#5 BARS @ 20" o.c. BOTTOM

#3 STIRRUPS REQ'D AT 4" o.c.
FROM SOLID HEAD TO FIRST CROSS RIB

Span c.-c. Columns $\ell_1 = \ell_2$ (ft)	Factored Super-imposed Load (psf)	(1) Steel (psf)	Square Interior Column $c_1 = c_2$ (in.)	Square Interior Column (2) Stirrups	Reinforcing Bars—Each Direction: Column Strip, Bottom, No. Ribs	Column Strip, Bottom, Bars per Rib	Column Strip, Top Interior No.-size	Middle Strip, Bottom, No. Ribs	Middle Strip, Bottom, Long Bars	Middle Strip, Bottom, Short Bars	Middle Strip, Top Interior No.-size
27'-0" D= 0.500 RIB NOT ON COLUMN LINE 0.523 CF/SF	50	1.94	13		4	1-#4 and 1-#5	20-#5	5	#4	#4	8-#5
	100	3.02	13	3 S 4 1	4	2-#5	20-#6	5	#4	#4	8-#5
	150	2.42	13 *	3 S 4 1	4	2-#6	22-#5	5	#4	#5	8-#5
	200	3.18	13 *	3 S 4 1	4	2-#7	19-#6	5	#5	#5	9-#5

From Pg 11-19 of CRSI Design Handbook


3
4
5
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27'-0"
27'-0"
27'-0"
E
D
C
B
27'-0"
24'-0"
24'-0"
3"
8"
11"
2'-6"
3'-0"