Technical Report 2

Student Health Center

Penn State University

Prepared By: Jacob Brambley (Structural Option)
Prepared For: Dr. Richard Behr

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Executive Summary

The Student Health Center (SHC) is a five story building on the Penn State campus that serves as a health care services and hospital facility. After completion in the fall of 2008, this building now houses University Health Services and Counseling and Psychological Services, two departments of Penn State's Division of Student Affairs.

The facility is 77 feet in height from the first level and is approximately 64,000 SF in area. It has a brick façade rising from the ground with large curtain wall on the south side the building. The structure is held up primarily by a steel frame. The overall structure sits on a mini-pile foundation through use of pile caps, piers, and grade beams. Composite steel with concrete slab on deck is use for the floor system throughout the SHC.

This technical report will explore three alternate floor systems to the concrete slab on deck already in place at the SHC. A two-way post tensioned slab, a two-way flat slab, and a hollow core concrete plank on steel beam system are the alternate floors being examined. The characteristics of the four systems will be compared and plausible alternatives to the existing system will be determined. These characteristics include cost, serviceability, constructability, self weight, as well as others. A typical interior bay was chosen as the basis for comparison. Conclusions were then drawn from this analysis as to which floor system would be a viable replacement to the composite steel with concrete deck.

As end result of this analysis, the two-way post tensioned slab and the two-way flat plate slab seem to be possible alternatives to the current system. The costs are similar to the system in place and the total floor thickness is significantly smaller. The self-weight of these proposed floors are higher creating a great load on the foundation. Also, the current partially-restrained steel frame cannot be used for these systems. Therefore, the impact of both on the foundation and an alternative for the lateral system would need to be addressed in a further report to truly confirm system efficiency.

Introduction

Located on Penn State campus, the Student Health Center serves as the center of health services for the college. The five story, 64,000 SF building was built in such a way as to bring in natural sunlight and create a healthy atmosphere for office workers and patients. The façade of the SHC is composed of the curtain wall as well as face brick accented with stone bands. The brick façade at the base of the building helps it fit in with the master plan of the rest of the university.

A composite steel floor system is utilized in the SHC and the purpose of this report is to explore three other floor systems and see how they would fit in the current building layout. By the end of this report, there will be a better idea of the viability of the current floor and the best options available if the floor construction was changed.

Structural Systems

Foundation:

The foundation of the SHC is composed of grade beams and piers that are supported by mini-piles with pile caps. The mini-piles are arranged in configurations of 1-5 piles per pile cap. They are to be at a depth of 45 feet and have an 80 ton allowable capacity. The partially-restrained moment frame employed in this building is either connected directly to a pile cap or to a concrete pier. The depth of these mini-piles will counteract the moment of the partially-restrained moment frame caused by lateral loads.

Floor System / Beams:

The floor system used in the SHC is composed of 3 1/4” lightweight concrete fill on 2”-20 gage galvanized composite floor deck LOK floor for a total slab thickness of 5 1/4". Also included are 3/4ϕ x 4” long shear studs equally spaced along the entire lengths of all interior beams and girders that are not part of the partially-restrained moment frame. The shear studs are not on the moment frame because the beams on the frame cannot be too rigid so that they can deform. This composite floor deck is supported by steel W-shape beams spanning between steel columns.

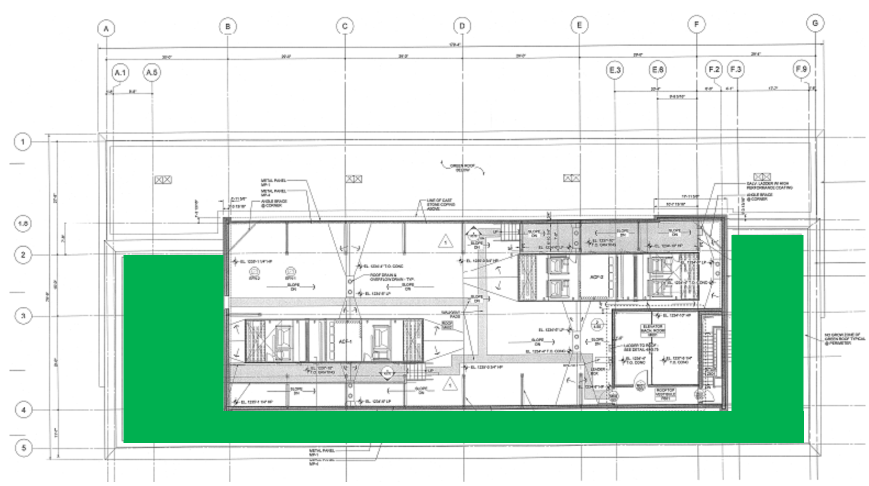
Columns:

The P.R. moment frame consists of W14 steel columns running from the foundation up to the roof level. Columns that are not part of the P.R. moment frame range in size and shape. Round HSS shapes are used both with and without concrete fill, as well as square HSS shapes and W shapes to resist gravity loads.

Roof / Penthouse Level:

The roof system is composed of 5 1/4" normal weight concrete fill on 3"-20 gage galvanized composite floor deck LOK floor for a total slab thickness of 8 1/4". The main roof is at the 6th level with a screen wall around the rooftop mechanical equipment. There is also a green roof around the perimeter of the main roof level *(Fig. 1)*. On the north end of the building, at the 5th level, there is another green roof *(Fig. 2)* that is nearly 20 feet wide and runs the length of the building.

Fig. 1 – Green Roof on Main Roof



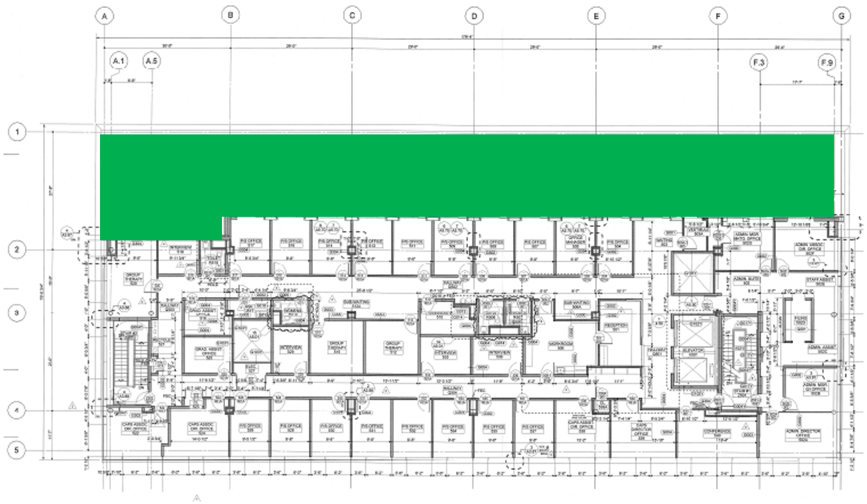


Fig 2 – Green Roof on 5th Floor

Lateral System:

A partially-restrained moment frame is used to resist lateral loads on the SHC. These frames are to have Flexible Moment Connections (FMC) designed by the steel fabricator per Part 11 of the AISC- Load & Resistance Factor Design Manual. A typical beam to column flange connection for these frames is detailed below *(Fig. 3)*. There are eight partially-restrained frames employed in this building, with seven running in the north/south direction, and one in the east/west direction *(Fig. 4)*. These frames run vertically up to the 5th Level or Main Roof Level of the building depending on the location. Frames are shown below in elevation (Fig. 5-7).

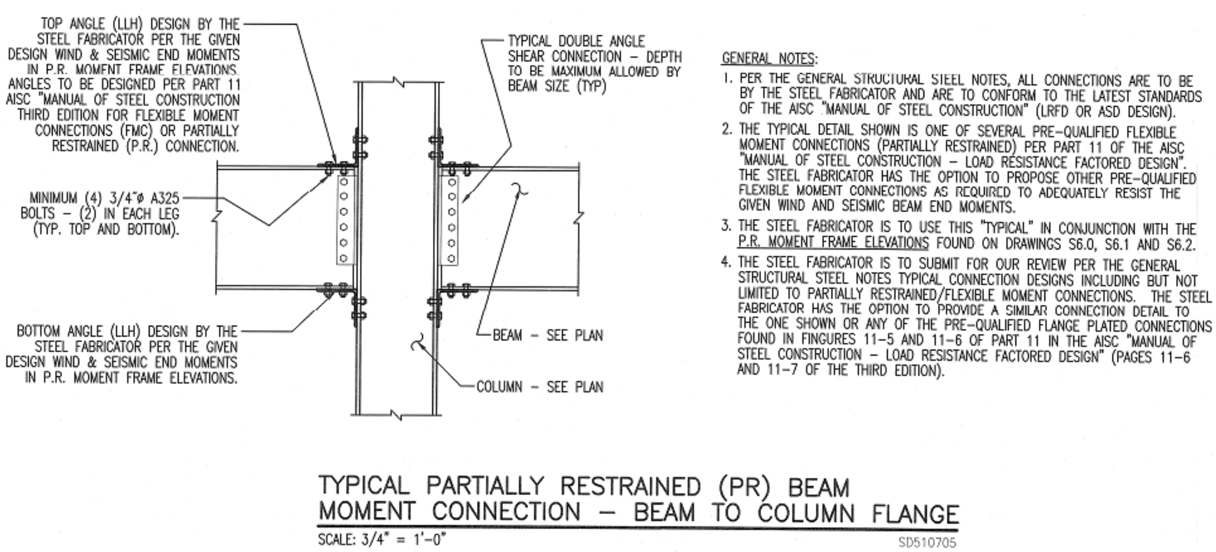


Fig. 3

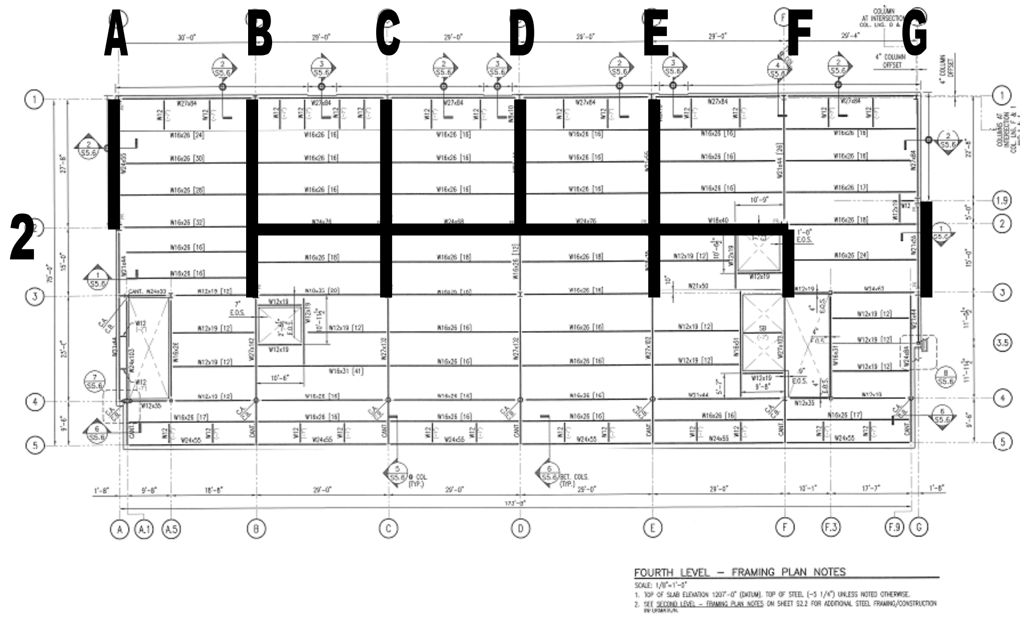


Fig. 4 – Partially-restrained Frame Locations

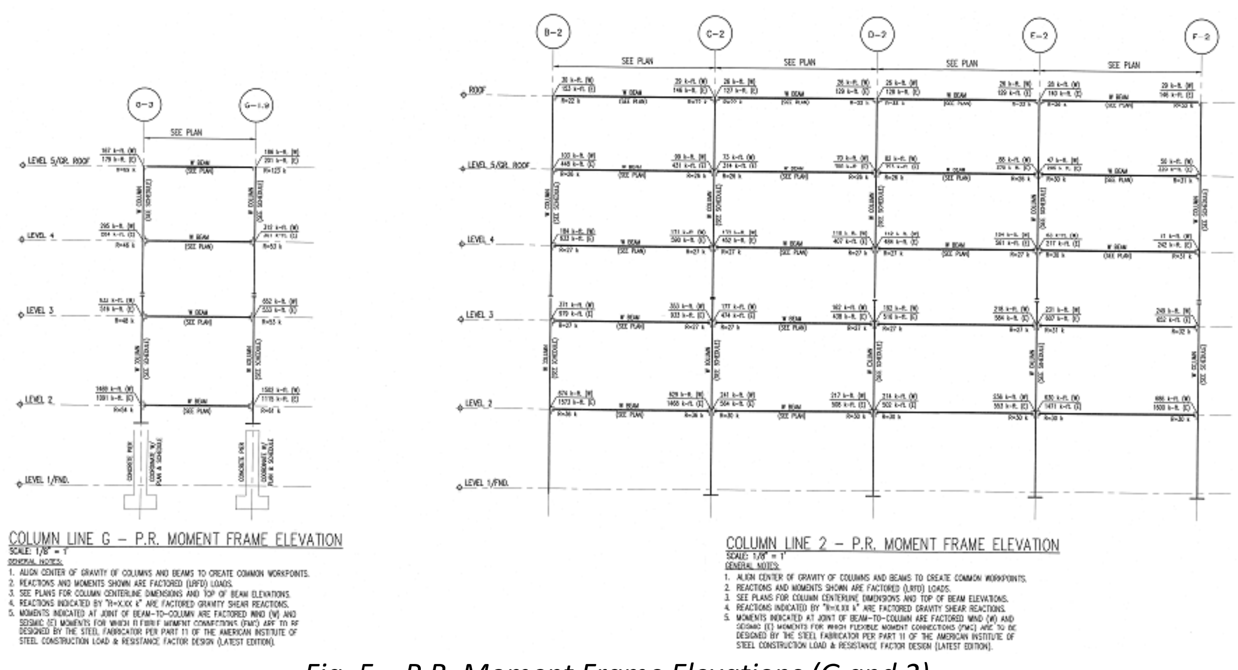


Fig. 5 – P.R. Moment Frame Elevations (G and 2)

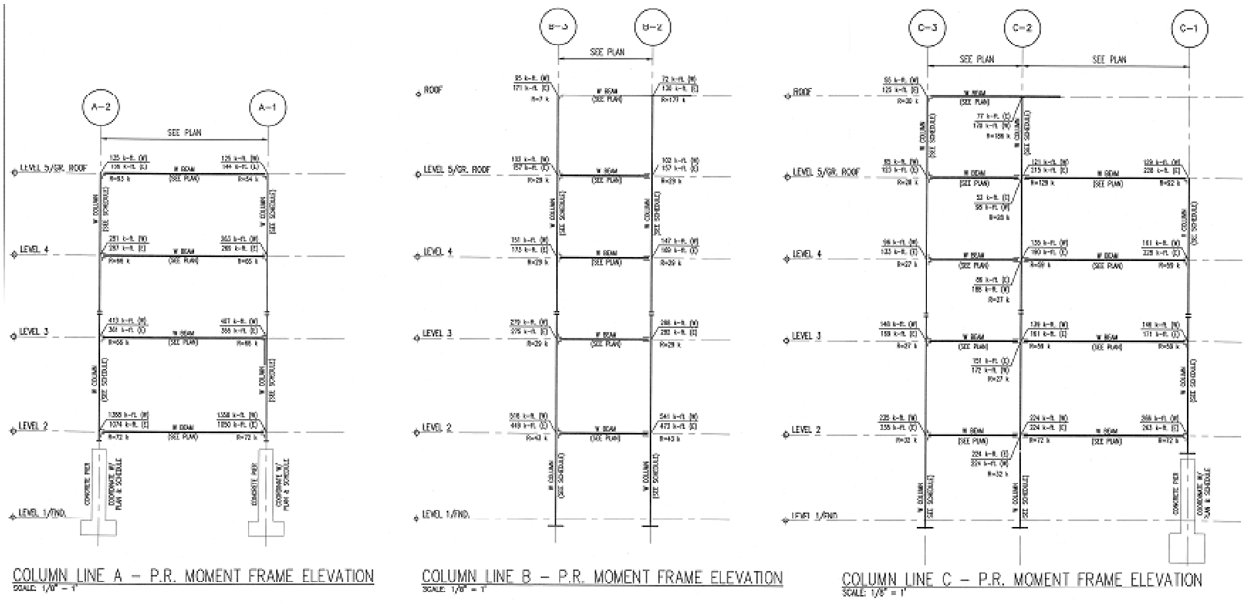


Fig. 6 – P.R. Moment Frame Elevations (A, B, and C)

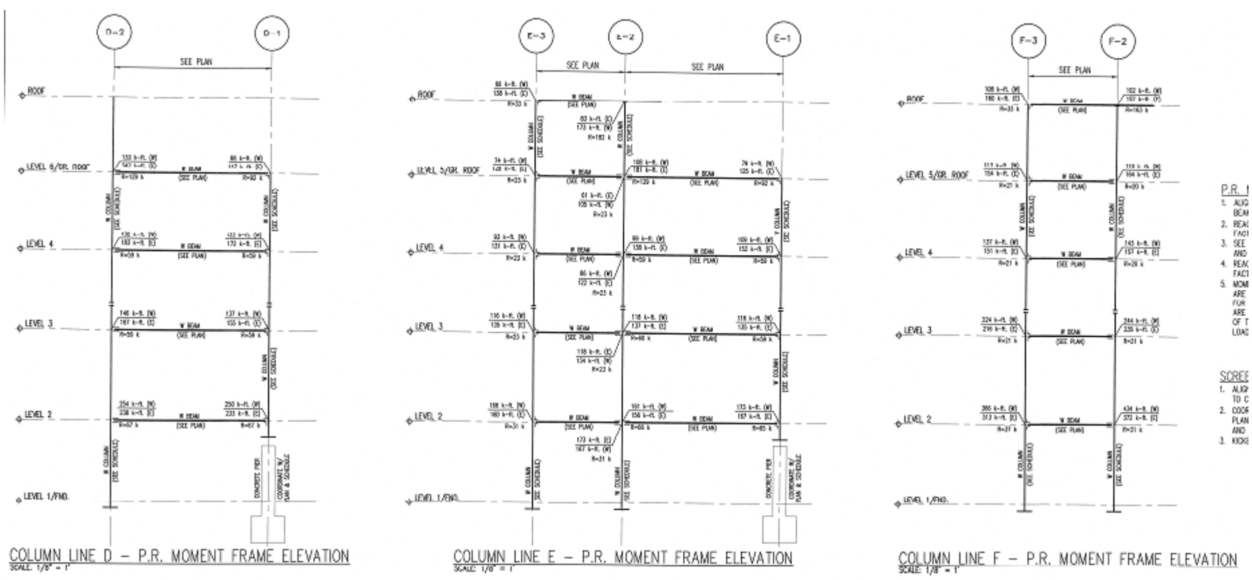


Fig. 7 – P.R. Moment Frame Elevations (D, E, and F)

Code and Design Requirements

Design Codes and References:

Codes used by Project Team:

International Building Code (IBC)/2003 with Borough Amendments
International Mechanical Code (IMC)/2003 with Borough Amendments
International Plumbing Code (IPC)/2003 with Borough Amendments
International Energy Conservation Code (IECC)/2003 with Borough Amendments
International Code Council Electrical Code (ICCEC)/2003
International Fire Code (IFC)/2003
ACI 318-05
AISC "Steel Construction Manual" (13th Edition)
ACI 530.1/ASCE 6/TMS 602 (2005)

Codes used for Thesis:

International Building Code (IBC)/2006
ACI 318-08
AISC "Steel Construction Manual" (13^{th} Edition)
ASCE 7-05

Reference Material:

RS Means Square Foot Cost Data, 2007
RS Means Assemblies Cost Data, 2008
ACI 318-08 Building Code and Commentary
Nitterhouse Concrete Products - Precast Design Aids

Deflection Criteria:

Maximum Floor Deflections:
- L/360 Live load
- L/240 Total load
- L/240 Roof

Material Properties

Material	A.S.T.M.	Minimum Strength
Concrete		
Foundation Walls, Pile Caps, Slab on Grade, Retaining Walls, Footings	-	3000 PSI
Exterior Slabs, Curbs	-	4000 PSI
Reinforcement	A615 (Grade 60)	60 KSI
WWF	A185, A497	70 KSI
Structural Tubing, Round	A500 (Grade B)	42 KSI
Structural Tubing, Shaped	A500 (Grade B)	46 KSI
Steel Pipe	A53 (Type E, Grade B)	35 KSI
Rolled Shapes	A992	50 KSI
Other Rolled Plates	A36	36 KSI
Connection Bolts	A325	92 KSI
Anchor Bolts	A307	-
Threaded Rods	A36	36 KSI
Non-shrink Grout	C1107	8000 PSI
CMU	C90 (lightweight)	2800 PSI

Loads

Gravity Loads:

Dead Load:

Dead Loads were obtained using typical design values, material specifications, or educated assumptions. My values were very similar to values stated by the Engineer of Record.

Component	Obtained Values
2" Steel Deck (on floors 1-5)	2 PSF
3-1/4" Concrete on Deck (on floors 1-5)	43 PSF
3" Steel Deck (on main roof level)	2 PSF
5-1/4" Concrete on Deck (on main roof level)	82 PSF
Green Roof	25 PSF
Ceiling with Mechanical/Electrical	15 PSF
Floor Finish	3 PSF

Live Load:

Live Loads were taken from ASCE 7-05 along with an assumption for the mechanical rooms. My obtained values were once again very similar to the values on the drawings.

Building Location	Drawing Values	Obtained Values
Corridors (first floor)	100 PSF	100 PSF
Corridors (above first floor)	80 PSF	80 PSF
Procedure/Exam Rooms	50 PSF + 20 PSF partition	40 PSF + 15 PSF partition
Lobbies	100 PSF	100 PSF
Stairs	125 PSF	100 PSF
Mechanical Rooms	75 PSF	150 PSF
Offices	50 PSF + 20 PSF partition	50 PSF + 15 PSF partition
Light Storage	125 PSF	125 PSF
Heavy Storage	250 PSF	250 PSF

Snow Load:

Snow loads were determined using IBC 2006 and Centre Region Code.

$p_f = 0.7 \times C_e \times C_t \times I \times p_g = 30.8$ psf
$p_g = 40$ psf
$C_e = 1.0$
$C = 1.0$
$I = 1.1$

Alternate Floor Systems

As mentioned in the introduction, three systems were examined to determine the best option to replace the system already in place. These alternative systems are being looked at to see if there legitimate asset to changing the current one such as a smaller floor thickness or decrease in construction cost. The three proposed options are a two-way flat slab, a two-way post tensioned slab, and a hollow core concrete plank on steel W-shapes. Analysis on the current composite steel system was completed in Technical Report 1 and will be attached to this report for reference. A typical interior bay, 23’x29’, (shown in Appendix A) was chosen to do calculations on the new systems. The existing column layout was utilized and for the concrete systems, a 20”x20” column was assumed in place of the steel columns.

Composite Steel System (existing)

A composite steel floor system is currently in place at the SHC. There is a 3 1/4” lightweight concrete fill on 2”-20 gage galvanized composite floor deck LOK floor for a total slab thickness of 5 1/4". W-shapes ranging W-12s to W-24s are used in conjunction with this slab. Also included are 3/4ϕ x 4” long shear studs equally spaced along the entire lengths of all interior beams and girders that are not part of the partially-restrained moment frame. This floor system was examined in Technical Report 1, and the calculations are shown in Appendix B of this report.

Advantages:

There are several advantages to using this system. The self-weight of the entire system is low which has a low impact on the foundation. Also, the steel goes up fairly quickly once it arrives on site. One reason for this is that there is no forming to put in place like the other concrete systems. The light, airy nature of the architecture is also accented by the slender shapes of a steel frame. If a concrete frame were used, the curtain wall areas wouldn’t seem as open and inviting.

Disadvantages:

This system does have its faults though. The overall floor thickness (slab + steel frame) is very thick compared to all-concrete systems. This affects the number of floors you can get per building height. Fireproofing is required for all the steel in the system. Also, the lead time to get the steel designed and to the site is longer than other systems.

Two-Way Flat Slab

The two-way flat slab system was the first alternative studied for this report. This system is popular due to its minimal slab thickness and low cost in certain locations. Another lateral force resisting system would have to be implemented to replace the steel moment frame, if this system was used though.

After analysis, it was determined that a 10” flat slab with 3” drop panel would suffice to carry the loads. The reinforcing used is stated in Appendix C. F’c and Fy were assumed to be 5000 psi and 60,000 psi respectively for analysis and a column size of 20”x20” was also assumed.

Advantages:

Advantages of using a flat slab include a shorter floor thickness than steel. Also, the cost is the smallest out of all of the systems studied. One reduction in cost is the lack of fireproofing needed for the concrete floor. The short lead time to get the material to the site is also highly desirable.

Disadvantages:

One major disadvantage to this proposed system is the self-weight of it. This increased weight will add load to the foundation and this may lead to higher foundation costs. The coordination of trades during constructing may also suffer due to this system because of large slab pours that need time to cure. As mentioned earlier, the larger concrete columns also may not be desirable for the areas behind curtain walls because of their bulkiness in comparison to the steel columns.

Two-Way Post Tensioned Slab

A two-way post tensioned slab was researched and analyzed as another minimal floor thickness alternative. Like the flat plate, the post tensioned system would need to implement a new lateral-force resisting system such as shear walls.

Through doing the calculations, it was determined that an 11” flat slab will work in conjunction with the post tensioning and other reinforcement. Further information about the tendons and rebar quantity and placement can be found in Appendix D.

Advantages:

One reason post tensioned slabs are used frequently are their ability to cover large spans while maintaining a relatively small thickness. The spans between columns are not extensive enough in this building to warrant using a post tensioned slab to its full potential. The current column grid could be change though and maybe it would open up the rooms a bit. The thickness of 11” was the thinnest out of all of the systems looked at. The lead time on materials is short. It is also 2-hour rated and needs no additional fireproofing.

Disadvantages:

Disadvantages may include constructability. There has to be a contractor nearby that has experience with post tensioning to build it correctly and efficiently. Much like the flat plate, it has a lot of weight to it and creates a bigger push down onto the foundation.

Prestressed Hollow Core Plank on Steel

Lastly, a prestressed hollow core plank system set on steel W-shapes was examined. Since this floor system is composed of steel and precast concrete, no labor intensive formwork or concrete pours will be happening. The whole system can be lifted up into place by a crane.

It was found, using Nitterhouse hollow core design aids, that an 8”x4’-0” hollow core plank with 4-1/2” Diam. Strand pattern will work the best to resist the building loads. This plank will sit atop W24x62 beams. Calculations and design aids are in Appendix E.

Advantages:

One major advantage, mentioned earlier, is the lack of formwork and wet concrete, which can be pretty labor intensive. In addition, the lateral-force resisting system in the original design can be kept in place. The floor is also relatively light, reducing detrimental effects on the foundation.

Disadvantages:

This system has many faults. The excessive depth of the plank and steel is a major cost and/or comfort issue, either increasing the building’s height or lowering ceiling heights. The lead time to get the steel designed, fabricated, and on the site is lengthy. Fireproofing is needed for all the steel W-shapes and the cost of the overall system is significantly higher than the other proposals.

Comparison

	Floor Systems			
	Composite Steel	Two-way Flat Slab	Two-way Post Tensioned Slab	Hollow Core Plank on Steel
System Weight (psf)	51	131	138	68
Total Depth (in)	29	13	11	32
Lead Time	Medium	Short	Short	Medium
Constructability	Medium	Medium	Hard	Medium
Formwork	No	Yes	Yes	No
Impact on Foundation	-	Yes	Yes	Little
Impact on Lateral System	-	Yes	Yes	No
Fireproofing Needed	Yes	No	No	Yes
Cost per SF	$19.95	$17.45	$21.86	$32.10
Viable Alternative	-	Yes	Yes	No
Additional Study	-	Yes	Yes	No

Conclusion

Technical Report 2 has examined several alternative floors systems to see which ones, if any, are plausible for use in the Student Health Center. Two concrete-dominate systems were looked at, as well as, another steel-dominate system. In the end, the two-way flat slab and the two-way post tensioned slab are the best options to replace the current system, as determined by this report.

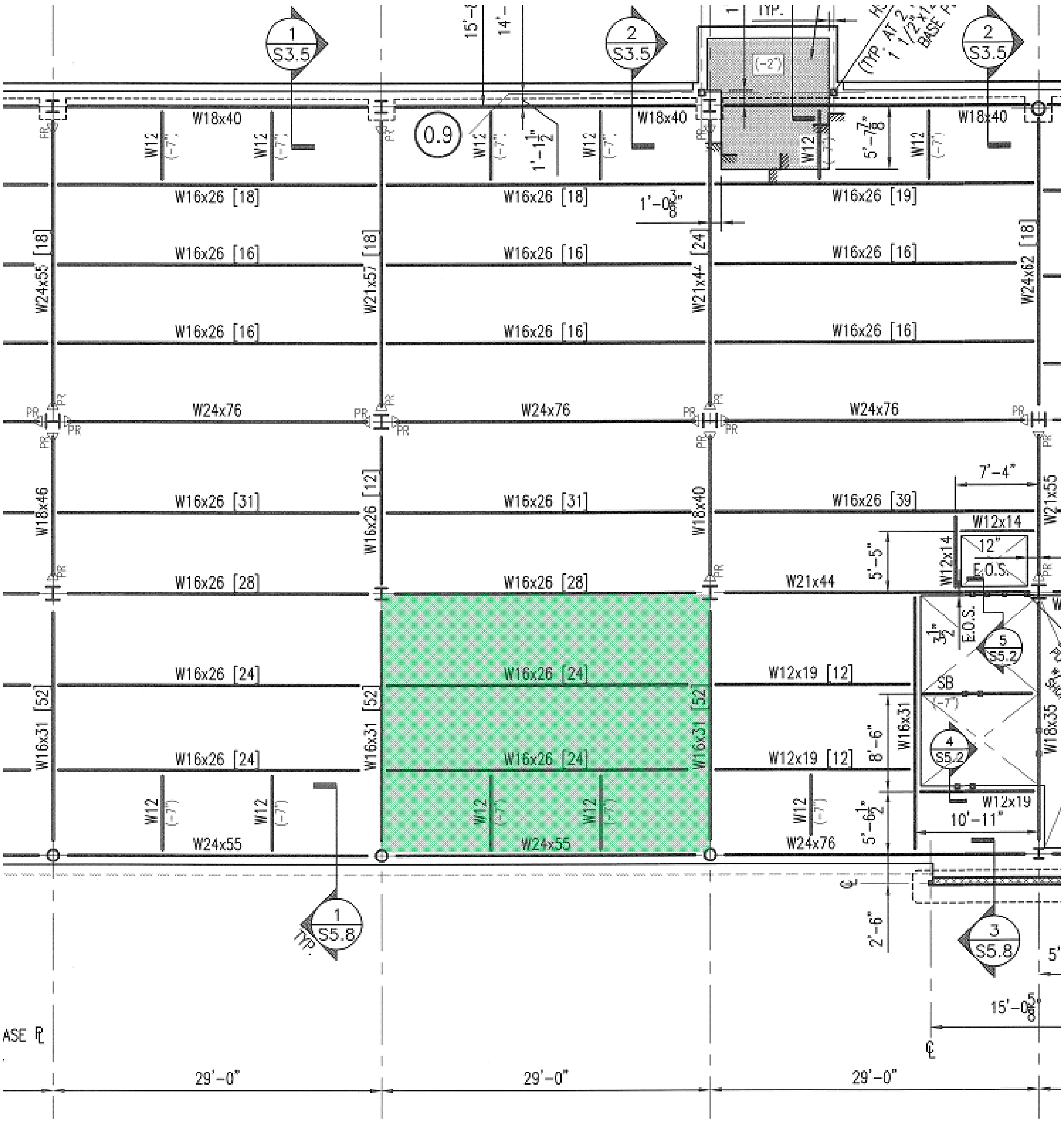
Both the flat slab and post tensioned slab reduced the thickness of the floor significantly, with the post tensioned slab being slightly thinner. They both have a short lead time and no need for fireproofing. The flat slab is slightly better in the cost and constructability categories.

The prestressed hollow core plank on steel system did not seem to be an efficient way to go in this project with cons outnumbering the pros on the majority.

Therefore, additional study will be performed on two-way flat slabs and two-way post tensioned slabs.

Appendix

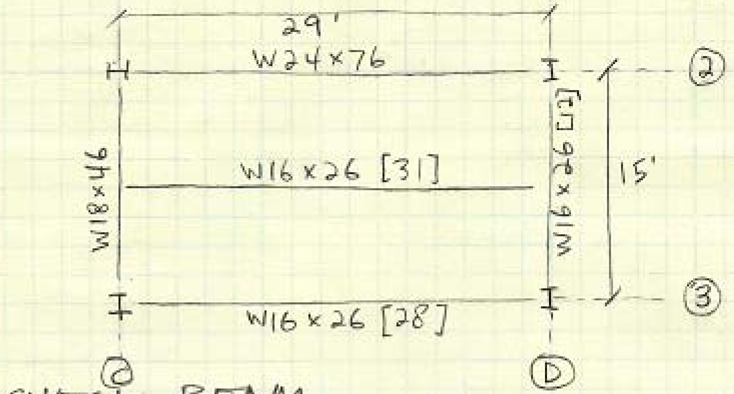
Appendix A: Existing Building Layout



Typical Bay for Calculations (29'x23') (in green)

Appendix B: Existing Composite Steel System

INTERIOR BAY ON SECOND LEVEL



CHECK BEAM

DL = 63 PSF   LL = 100 PSF (CORRIDOR)

$w_u = 1.2(\overset{0.026\,(SW)+}{0.063 \times 7.5'}) + 1.6(0.1 \times 7.5)$

$w_u = 1.798$ k/ft

$M_u = \frac{w_u l^2}{8} = \frac{(1.798)(29)^2}{8}$

$M_u = 189.0$ ft·k

$b_{eff} = \left| \begin{array}{l} 7.5' \times 12 = 90'' \\ \frac{29(12)}{4} = 87'' \leftarrow \text{CONTROLS} \end{array} \right.$

$\Delta_{CONSTRUCTION} = \frac{5 w_c l^4}{384 EI}$

$\Delta_{ALLOWABLE} = l/360 = 29(12)/360 = 0.967''$

$I_{req'd} = \frac{5 w l^4}{384 \Delta_{const} E} = \frac{5 \times (0.063 \times 7.5)(29)^4 (1728)}{384 (0.967)(29{,}000)} = 268 \text{ in}^4$

$I_{W16\times26} = 301 \text{ in}^4 > 268 \text{ in}^4 \therefore OK$

Check bending during construction

$w_u = 1.2(0.063 \times 7.5) + 1.6(0.02 \times 7.5) = 0.807$

$M_u = \frac{w_u l^2}{8} = \frac{0.807(29)^2}{8} = 84.8$ ft·k

$\phi M_{n\,W16\times26} = 166$ ft·k > 84.8 ft·k $\therefore$ OK

ASSUMING $\Sigma Q_n = 145^k$

$a = \dfrac{\Sigma Q_n}{0.85(3)(87)} = 0.654''$

$Y2 = 5.25'' - \dfrac{0.654}{2} = 4.92'' \Rightarrow 4.5''$

FROM TABLE 3-19: @ PNA = 6

$\phi M_n = 269 \text{ ft·k} > 189 = M_u \quad \therefore \text{OK}$

FROM TABLE 3-21:

DECK PERP.
3/4" ϕ STUDS ; 1 STUD/RIB
$f'_c = 3000$ ksi

} $Q_n = 17.2^k$

$\dfrac{\Sigma Q_n}{Q_n}(\times 2) = \dfrac{145}{17.2}(\times 2) = 16.9 \rightarrow 17$ STUDS REQ'D

$17 <$ ACTUAL $= 31 \quad \therefore$ OK

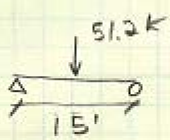
CHECK DEFLECTION:

FROM TABLE 3-20: $I_{LB} = 622 \text{ in}^4$

$\Delta = \dfrac{5 w l^4}{384 E I_{LB}} = \dfrac{5(0.100 \times 7.5)(29)^4(1728)}{384(29{,}000)(622)} = 0.66''$

$0.66 < \Delta_{Allow} = l/360 = 29(12)/360 = 0.967'' \quad \therefore$ OK

CHECK GIRDER



$P_u = 1.2(0.063 \times 7.5 \times 29) + 1.6(0.1 \times 7.5 \times 29) = 51.2^k$

$M_{max} = Pl/4 = \dfrac{51.2(15)}{4} = 192.2 \text{ ft·k}$

ASSUMING $\Sigma Q_n = 145$

$$b_{eff} = \begin{cases} 29(12) = 348" \\ 15(12)/4 = 45" \leftarrow \text{CONTROLS} \end{cases}$$

$$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{145}{0.85(3)(45)} = 1.26$$

$$Y_2 = 5.25" - 1.26/2 = 4.62" \Rightarrow 4.5"$$

FROM TABLE 3-19: @ PNA = 6

$\phi M_n = 269$ ft·k > 192.2 ft·k ∴ OK

FROM TABLE 3-21

$$\left.\begin{array}{l} \text{DECK PARALLEL} \\ w/h = 3.0 \\ 3/4"\phi \text{ STUDS} \\ \text{LT. WT. CONC.} \end{array}\right\} 17.1 = Q_n$$

$$\frac{\Sigma Q_n}{Q_n}(\times 2) = \frac{145}{17.1}(\times 2) = 16.96 \rightarrow 17 \text{ STUDS REQUIRED}$$

17 > 12 PROVIDED ∴ $\boxed{\text{NO GOOD}}$

TRY $\Sigma Q_n = 96$ @ PNA 7

$$a = \frac{96}{0.85(3)(45)} = 0.837"$$

$$Y_2 = 5.25 - 0.837/2 = 4.83" \rightarrow 4.5"$$

FROM TABLE 3-19 @ PNA = 7

$\phi M_n = 241$ ft·k > 192.2 ft·k ∴ STILL OK

$$\frac{\Sigma Q_n}{Q_n}(\times 2) = \frac{96}{17.1}(\times 2) = 11.2 \rightarrow 12 \text{ STUDS REQ'D}$$

∴ OK

<u>CHECK DEFLECTION</u>: $I_{LB} = 535 \text{ in}^4$

$$\Delta = \frac{Pl^3}{48EI_{LB}} = \frac{(0.1 \times 7.5 \times 29)(15)^3(12)^3}{48(29{,}000)(535)} = 0.17"$$

$$\Delta_{ALLOW} = 15(12)/360 = 0.5" > 0.17" \quad \therefore \text{OK}$$

CHECK COLUMN (D3 ON FIRST LEVEL)

W14x48 $h = 14'$ $A_g = 14.1\ in^2$

$I_x = 484\ in^4$ $I_y = 51.4\ in^4$

$r_x = 5.85\ in.$ $r_y = 1.91\ in.$

$\frac{KL}{r_x} = \frac{14(12)}{5.85} = 28.7$ $\frac{KL}{r_y} = \frac{14(12)}{1.91} = 87.96$

CONTROLS

$\frac{KL}{r} \le 4.71\sqrt{E/F_y} = 4.71\sqrt{\frac{29{,}000}{50}} = 113.4 > 87.96$ ∴ INELASTIC BEHAVIOR

$F_E = \frac{\pi^2 E}{(\frac{KL}{r})^2} = \frac{\pi^2(29{,}000)}{(87.96)^2} = 37.0\ ksi$

$F_{CR} = \left[0.658^{F_y/F_E}\right] F_y = \left[0.658^{50/37}\right] 50 = 28.4\ ksi$

$\phi P_n = \phi F_{CR} A_g = 0.9(28.4)(14.1) = \underline{360.4^k}$

FROM TABLE 4-1 -(IN STL MANUAL)

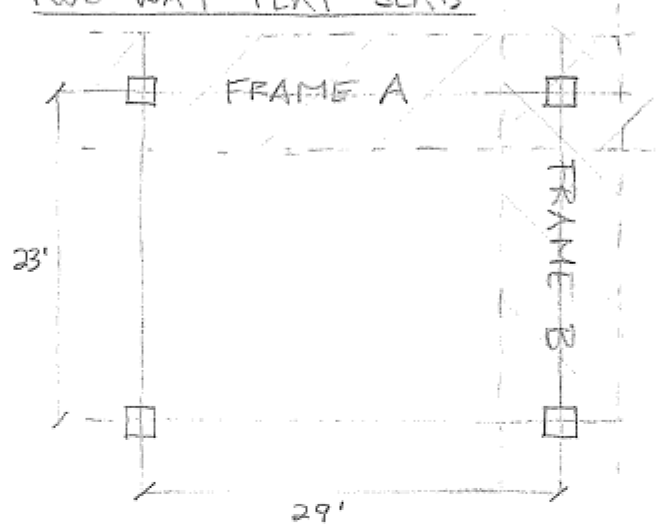
$\phi P_n = \underline{361^k}$

$361^k = \phi P_n < 442.6^k = P_u$ ∴ WILL NOT WORK

USE W14x61, $\phi P_n = 572^k < 442.6^k$ ∴ OK

Appendix C: Two-Way Flat Slab

TWO WAY FLAT SLAB



TYPICAL INTERIOR BAY

ASSUME:

$f'_c = 5000$ PSI

$f_y = 60{,}000$ PSI

20" × 26" COLUMNS

DL = 18 PSF + SW

LL = 80 PSF (CORRIDOR ON MAJORITY OF PANEL)

DETERMINE SLAB THICKNESS

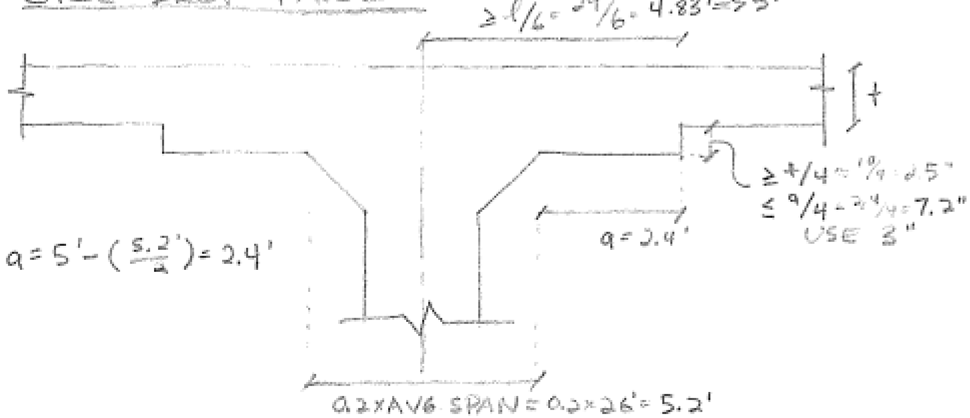
W/ DROP PANEL & W/O EDGE BEAM

FROM TABLE 9.5(c): $t_{min} = \frac{\ell_n}{36} = \frac{(29(12) - 20)}{36}$

$t_{min} = 9.11" \Rightarrow 10"$

$w_u = 1.2\left(\frac{10}{12}(150) + 18\right) + 1.6(80) = 300$ PSF

SIZE DROP PANEL

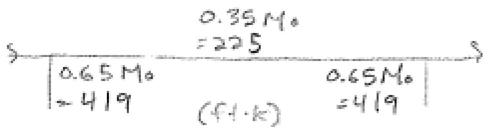


MOMENTS

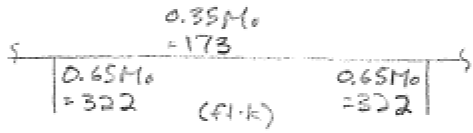
$M_o = \frac{1}{8} w_u l_2 l_n^2$

FRAME A: $\frac{1}{8}(300)(23)(29-1.67)^2/1000 = 644$ ft·k

FRAME B: $\frac{1}{8}(300)(29)(23-1.67)^2/1000 = 495$ ft·k



FRAME A



FRAME B

SUMMARY OF MOMENTS

FRAME A	M⁻ (ft·k)	M⁺
TOTAL MOM.	419	225
% to CS	75%	60%
MOM. IN C.S.	314	135
MOM. IN M.S.	105	90

FRAME B	M⁻ (ft·k)	M⁺
TOTAL MOM	322	173
% to CS	75%	60%
MOM. IN C.S.	242	104
MOM. IN M.S.	80	69

$d_A = 10 - 1.5(0.625) - 0.75 = 8.3''$

$d_B = 10 - 0.5(0.625) - 0.75 = 8.93''$

ASSUME #5 bars

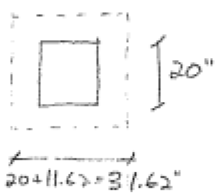
DESIGN SLAB REINFORCEMENT IN COL. STRIP

Item	Description	FRAME A M⁻	FRAME A M⁺	FRAME B M⁻	FRAME B M⁺
1	M_u (ft·k)	314	135	242	104
2	b (in)	138	138	174	174
3	d (in)	8.3	8.3	8.93	8.93
4	M_n (ft·k)	314/0.9 = 349	150	269	116
5	R (lb/in²)	441	189	233	100
6	ρ req'd	0.0074	0.0032	0.0039	0.0017
7	As req'd (in²)	8.48	3.67	6.06	2.64
8	As min (in²)	0.0018bt = 2.48	2.48	3.13	3.13
9	N	27.4 28	11.8 12	19.5 20	10.1 11
10	N min	6.9 7	6.9 7	8.7 9	8.7 9

DESIGN SLAB REINFORCEMENT IN MID. STRIP

Item	Description	FRAME A M^-	FRAME A M^+	FRAME B M^-	FRAME B M^+
1	M_u (ft·k)	105	90	80	69
2	b (in)	138	138	174	174
3	d (in)	8.3	8.3	8.93	8.93
4	M_n (ft·k)	117	100	89	77
5	R (lb/in²)	148	126	77	67
6	ρ req'd	0.0025	0.0021	0.0013	0.0011
7	A_s req'd (in²)	2.86	2.41	2.02	1.71
8	A_s min (in²)	2.48	2.48	3.13	3.13
9	N	9.2 10	8 8	10.1 11	10.1 11
10	N_{min}	7	7	9	9

PUNCHING SHEAR CHECK @ Drop Panel (3" DROP)



$$d_{avg} = \frac{11.3 + 11.93}{2} = 11.62"$$

$$V_u = \omega_u \times A = 0.3\left[(23)(29) - \left(\frac{31.62}{12}\right)^2\right] = 198^k$$

$$V_c = 4\sqrt{f'_c} \cdot b_o \cdot d$$

CONTROLLING V_c ⟶ $V_c = 4\sqrt{5000} \cdot (4 \times 31.62)(11.62)/1000 = 416^k$

$$\phi V_c = 0.75(416) = 312^k > V_u = 198^k \therefore OK$$

@ SLAB

$$d_{avg} = \frac{8.3 + 8.95}{2} = 8.62"$$

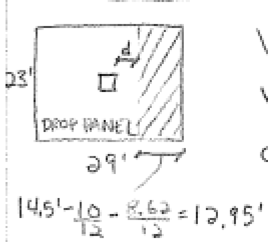
$$V_u = 0.3\left[(23)(29) - (10.72)^2\right] = 166^k$$

$V_c = \left(\frac{\alpha_s \cdot d}{b_o} + 2\right)\sqrt{f'_c}\ b_o \cdot d$ ⟵ controlling V_c

$$V_c = \left(\frac{40 \times 8.62}{4 \times 129} + 2\right)\sqrt{5000} \cdot (4 \times 129) \cdot 8.62 = 837^k$$

$$\phi V_c = 0.75(837) = 628^k > 166^k = V_u \therefore OK$$

WIDE BEAM CHECK



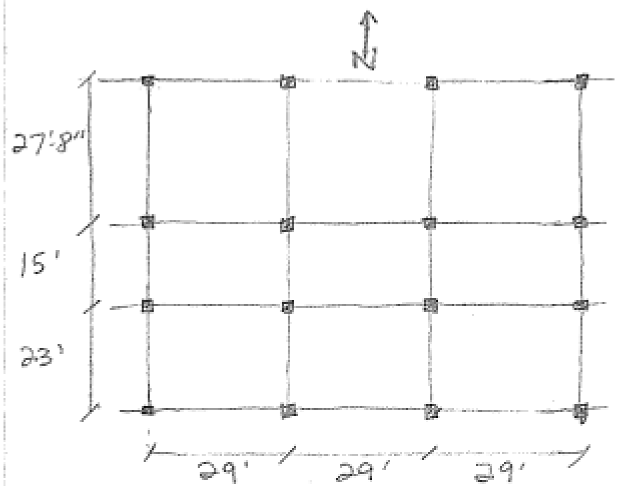
$$V_u^{wide\ bm} = 0.3 \times 12.95 \times 23 = 89^k$$

$$V_n = 2\sqrt{f'_c} \cdot b_w \cdot d = 2\sqrt{5000}(23 \times 12) \cdot 8.62/1000 = 336^k$$

$$\phi V_n = 0.75(336) = 252^k > 89^k = V_u \therefore OK$$

Appendix D: Two-Way Post Tensioned Slab

TWO WAY POST TENSIONED SLAB



ASSUME:

$f'_c = 5000$ PSI

$f_y = 60{,}000$ PSI

20" × 20" COLUMNS

DL = 18 PSF + SW

LL = 80 PSF

$f'_{ci} = 3000$ PSI

UNBONDED TENDONS – ½" φ, 7-WIRE STRANDS, $A = 0.153\ in^2$

$f_{pu} = 270$ KSI

ESTIMATED PRESTRESS LOSSES = 15 KSI

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

$P_{eff} = A \cdot f_{se} = (0.153)(174) = 26.6$ K/tendon

DETERMINE PRELIM. SLAB THICKNESS

$L/h = 45$ L = longest span = 29'

$h = 29(12)/45 = 7.7" \rightarrow 8"$ SLAB

LOADING

SW OF SLAB = $(8/12)(\overset{PCF}{150}) = 100$ PSF

SUPERIMPOSED DL = 18 PSF

LL = 80 PSF (W/O REDUCTION)

$A_T = \left(\frac{15+23}{2}\right) \times (29) = 551\ ft^2$ (SMALLEST BAY)

$A_I = 4(551) = 2204\ ft^2$

$LL_{red} = LL\sqrt{0.25 + (15/\sqrt{2204})} = 0.755(80)$

$LL_{red} = 60.4$ PSF

DESIGN OF EAST-WEST INTERIOR FRAME

EQUIVALENT FRAME METHOD
BAY WIDTH BETWEEN ℄'s $= (27.67/2) + (15/2) = 21.33'$
IGNORE COL STIFFNESS FOR SIMPLICITY
$LL/DL < 3/4$ ∴ NO PATTERN LOADING REQ'D

CALCULATE SECTION PROPERTIES:

TWO WAY SLAB MUST BE DESIGNED AS CLASS U

$A = b \cdot h = (21.33 \times 12)(8) = 2048\ in^2$

$S = b \cdot h^2/6 = (21.33 \times 12)(8)^2/6 = 2731\ in^3$

SET DESIGN PARAMETERS:

CLASS U: ALLOWABLE STRESSES

AT TIME OF JACKING:

$f'_{ci} = 3{,}000$ PSI

COMPRESSION $= 0.60 f'_{ci} = 0.6(3000) = 1800$ psi

TENSION $= 3\sqrt{f'_{ci}} = 3\sqrt{3000} = 164$ PSI

AT SERVICE LOADS:

$f'_c = 5000$ PSI

COMPRESSION $= 0.45 f'_c = 0.45(5000) = 2{,}250$ PSI

TENSION $= 6\sqrt{f'_c} = 6\sqrt{5000} = 424$ PSI

AVERAGE PRECOMPRESSION LIMITS:

$P/A = 125$ PSI MIN
$= 300$ PSI MAX

TARGET LOAD BALANCES:

60% - 80% OF SW FOR SLABS (GOOD APPROXIMATION)

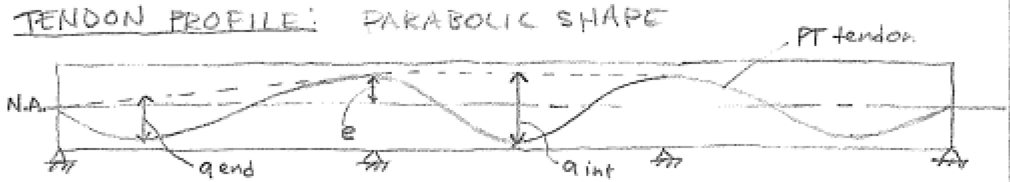
USE $0.60(SW) = 0.60(100) = 60$ PSF

COVER REQUIREMENTS:

FOR 2-HR RATING FROM THE IBC

RESTRAINED SLABS = 3/4" @ bottom

TENDON PROFILE: PARABOLIC SHAPE



TENDON ORDINATE	TENDON (CG) LOCATION	
EXTERIOR SUPPORT - ANCHOR	4.0"	from bottom of slab
INTERIOR SUPPORT - TOP	7.0"	
INTERIOR SPAN - BOTTOM	1.0"	
END SPAN - BOTTOM	1.75"	

$a_{int} = 7.0" - 1.0" = 6.0"$

$a_{end} = \left(\frac{4.0" + 7.0"}{2}\right) - 1.75" = 3.75"$

e varies along span

PRESTRESS FORCE REQ'D TO BALANCE 60% OF SW

$W_b = 0.60(W_{DL}) = 0.60(100)(21.33) = 1280 \text{ lb/ft} = 1.28 \text{ k/ft}$

FORCE IN TENDONS TO COUNTERACT THE LOAD IN END BAY

$P = W_b L^2 / 8 a_{end}$

$= (1.28)(29)^2 / 8(3.75/12) = 431 \text{ K}$

CHECK PRECOMPRESSION ALLOWANCE

OF TENDONS TO ACHIEVE 431 k

OF TENDONS = (431 k)/26.6(k/tendon) = 16.19

USE 16 TENDONS

ACTUAL FORCE FOR BONDED TENDONS:

$P_{ACTUAL} = (16 \text{ tendons})(26.6^k) = 425.6k$

$W_b = (425.6/430.6)(1.28) = 1.27 \; k/ft$

$P_{ACTUAL}/A = (425.6 \times 1000)/2048 = 208 \text{ PSI} > MIN = 125 \text{ PSI}$
$< MAX = 300 \text{ PSI}$ ∴ OK

$P_{INTERIOR} < P_{EXTERIOR}$ ∴ LESS FORCE REQ'D

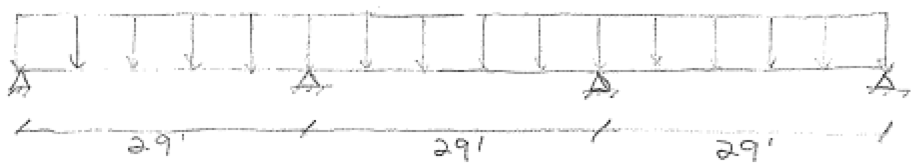
$W_b = 426(8)(6.0/12)/29^2 = 2.02 \; k/ft$ $\quad \frac{W_b}{W_{DL}} = 95\% < 100\%$ ∴ OK

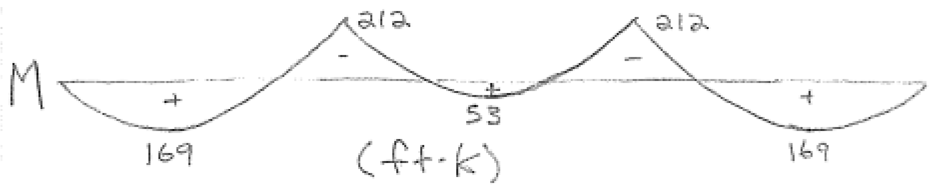
$\boxed{\text{EFF. PRESTRESS FORCE, } P_{EFF} = 426 \text{ k FOR E-W FRAME}}$

CHECK SLAB STRESSES

DEAD LOAD MOMENTS:

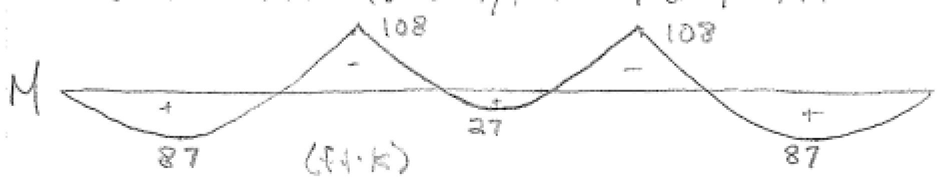
$\omega_{DL} = (118 \text{ PSF})(21.33')/1000 = 2.517 \; k/ft$





LIVE LOAD MOMENTS:

$\omega_{LL} = 60.4 \text{ PSF}(21.33')/1000 = 1.289 \; k/ft$

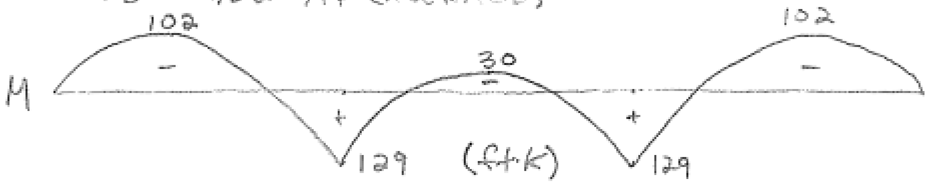


BALANCED LOAD AVERAGE:

$\frac{(2 \times 1.27) + 2.02}{3} = 1.52 \; k/ft$

TOTAL BALANCING MOMENTS, M_{bal}:

$w_b = -1.52$ k/ft (AVERAGE)



STAGE 1: STRESSES IMMEDIATELY AFTER JACKING (DL+PT)

MIDSPAN STRESSES
INT. SPAN

$f_{top} = [(-53+30)(12)(1000)]/(2731) - 208$
$= -101 - 208 = -309 \text{ PSI} < 0.6f'_{ci} = 1800 \text{ PSI} \therefore OK$

$f_{bot} = [(53-30)(12)(1000)]/(2731) - 208$
$= 101 - 208 = -107 \text{ PSI} < 3\sqrt{f'_{ci}} = 164 \text{ PSI} \therefore OK$

END SPAN

$f_{top} = [(-169+102)(12)(1000)]/(2731) - 208$
$= -294 - 208 = -502 \text{ PSI} < 0.60f'_{ci} = 1800 \text{ PSI} \therefore OK$

$f_{bot} = [(169-102)(12)(1000)]/(2731) - 208$
$= 294 - 208 = 86 \text{ PSI} < 3\sqrt{f'_{ci}} = 164 \text{ PSI} \therefore OK$

SUPPORT STRESSES

$f_{top} = [(212-129)(12)(1000)]/(2731) - 208$
$= 365 - 208 = 157 \text{ PSI} < 3\sqrt{f'_{ci}} = 164 \text{ PSI} \therefore OK$

$f_{bot} = [(-212+129)(12)(1000)]/(2731) - 208$
$= -365 - 208 = -573 \text{ PSI} < 0.60f'_{ci} = 1800 \text{ PSI} \therefore OK$

STAGE 2: STRESSES AT SERVICE LOAD (DL+LL+PT)

MIDSPAN STRESSES
INT SPAN

$f_{top} = [(-53-27+30)(12)(1000)]/(2731) - 208$
$= -220 - 208 = -428 \text{ PSI} < 0.45f'_c = 2250 \text{ PSI} \therefore OK$

$f_{bot} = [(53+27-30)(12)(1000)]/(2731) - 208$
$= 220 - 208 = 12 \text{ PSI} < 6\sqrt{f'_c} = 424 \text{ PSI} \therefore OK$

END SPAN

$f_{top} = [(-169 - 87 + 102)(12)(1000)]/(2731) - 208$
$= -677 - 208 = -885 \text{ PSI} < 0.45 f'_c = 2250 \text{ PSI} \therefore OK$

$f_{bot} = [(169 + 87 - 102)(12)(1000)]/(2731) - 208$
$= 677 - 208 = 469 \text{ PSI} > 6\sqrt{f'_c} = 424 \text{ PSI} \therefore \text{NO GOOD}$

SUPPORT STRESSES

$f_{top} = [(212 - 129 + 108)(12)(1000)]/(2731) - 208$
$= 839 - 208 = 631 \text{ PSI} > 6\sqrt{f'_c} = 424 \quad \therefore \text{NO GOOD}$

$f_{bot} = [(-212 + 129 - 108)(12)(1000)]/(2731) - 208$
$= -839 - 208 = 1047 \text{ PSI} < 0.45 f'_c = 2250 \text{ PSI} \quad \therefore OK$

* INCREASE SLAB THICKNESS

$f_{top\ req'd} + P/A = 424 + 208 = 632 \text{ PSI}$

$$\frac{[(212 - 129 + 108)(12)(1000)]}{S} = 632$$

$S = 3627 \text{ in}^3$

$3627 = (21.33 \times 12)h^2/6 \Rightarrow h = 9.2 \rightarrow \text{TRY } 11''$

$A = (21.33 \times 12)(11) = 2816 \text{ in}^2$

$S = (21.33 \times 12)(11)^2/6 = 5163 \text{ in}^3$

$W_{DL} = (11/12)(150) = 137.5 \text{ PSF}$

$w_b = 0.60(137.5)(21.33) = 1760 \text{ lb/ft} = 1.76 \text{ k/ft}$

$P = (1.76)(29)^2/8(6.0/12) = 370 \text{ k}$

$\# \text{ OF TENDONS} = 370/26.6 = 13.9 \rightarrow \text{TRY 14 Tendons}$

$P_{ACTUAL} = 14(26.6) = 372.4 \text{ k}$

$w_b = \left(\frac{372.4}{370}\right)(1.76) = 1.77 \text{ k/ft}$

$P/A = (372.4 \times 1000)/2816 = 132 \text{ PSI} \begin{matrix} > MIN \\ > MAX \end{matrix} \therefore OK$

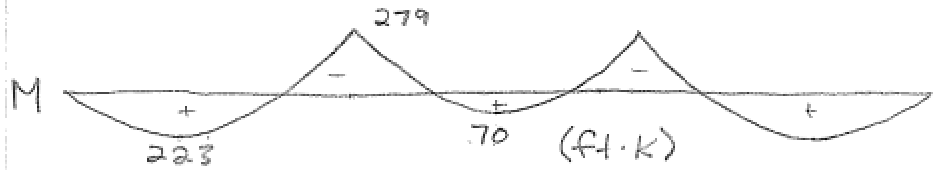
$w_{b\ interior} = 372(8)(9/12)/29^2 = 2.66 \text{ k/ft}$

$\frac{w_b}{w_{DL}} = \frac{2.66}{2.93} = 90.8\% < 100\% \therefore OK \quad \boxed{P_{EFF} = 372 \text{ k}}$

RECHECK SLAB STRESSES

DEAD LOAD MOMENTS:

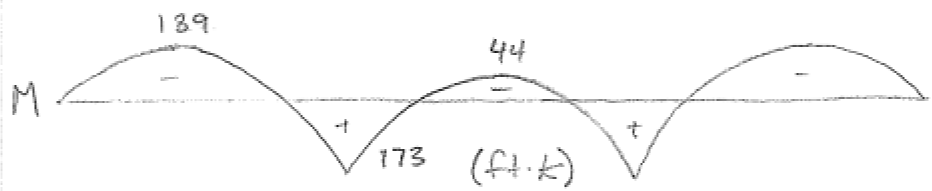
$W_{DL} = (155.5 PSF)(21.33')/1000 = 3.317\ k/ft$



LIVE LOAD MOMENTS:

(SAME AS BEFORE)

M_{bal} : $\omega_b = -2.067\ k/ft$ (AVERAGE)



STAGE 1: (DL+PT)

MIDSPAN STRESSES

INT SPAN

$f_{top} = [(-70+44)(12)(1000)]/5163 - 132 = -192\ PSI < 1800\ PSI \therefore OK$

$f_{bot} = [(70-44)(12)(1000)]/5163 - 132 = -72\ PSI < 1800\ PSI \therefore OK$

END SPAN

$f_{top} = [(-223+139)(12)(1000)]/5163 - 132 = -327\ PSI < 1800\ PSI \therefore OK$

$f_{bot} = [(223-139)(12)(1000)]/5163 - 132 = 63\ PSI < 164\ PSI \therefore OK$

SUPPORT STRESSES

$f_{top} = [(279-173)(12)(1000)]/5163 - 132 = 114\ PSI < 164\ PSI \therefore OK$

$f_{bot} = [(-279+173)(12)(1000)]/5163 - 132 = -378\ PSI < 1800\ PSI \therefore OK$

STAGE 2: (DL+LL+PT)

MIDSPAN STRESSES
INT SPAN

$f_{top} = [(-70-27+44)(12)(1000)]/5163 - 132 = -255$ PSI < 2250 PSI ∴ OK

$f_{bot} = [(70+27-44)(12)(1000)]/5163 - 132 = -9$ PSI < 2250 PSI ∴ OK

END SPAN

$f_{top} = [(-223-87+139)(12)(1000)]/5163 - 132 = -529$ PSI < 2250 PSI ∴ OK

$f_{bot} = [(223+87-139)(12)(1000)]/5163 - 132 = 265$ PSI < 424 PSI ∴ OK

SUPPORT STRESSES

$f_{top} = [(279+108-173)(12)(1000)]/5163 - 132 = 365$ PSI < 424 PSI ∴ OK

$f_{bot} = [(-279-108+173)(12)(1000)]/5163 - 132 = -629$ PSI < 2250 PSI ∴ OK

ULTIMATE STRENGTH

DETERMINE FACTORED MOMENTS:

$e = 0"$ @ EXT. SUPPORT ; $e = 4.5"$ @ INT. SUPPORTS

$M_1 = P \cdot e = (372.4)(4.5/12) = 140$ ft·k

$M_{sec} = M_{bal} - M_1 = 173 - 140 = 33$ ft·k @ INT. SUPPORTS

$M_u = 1.2 M_{DL} + 1.6 M_{LL} + 1.0 M_{sec}$

AT MIDSPAN: $M_u = 1.2(223) + 1.6(87) + 1.0(16.5) = 423$ ft·k

AT SUPPORT: $M_u = 1.2(-279) + 1.6(-108) + 1.0(33) = -475$ ft·k

DETERMINE MIN. BONDED REINF.

POSITIVE MOMENT REGION:
INTERIOR SPAN: NO POS. REINF. REQ'D

EXTERIOR SPAN: $f_t = 265$ PSI $> 2\sqrt{f'_c} = 2\sqrt{5000} = 141$ PSI
∴ MIN POS. REINF. REQ'D

$y = f_t/(f_t + f_c)h = [265/(265+529)](11) = 3.67"$

$N_c = [M_{DL+LL} / S](0.5)(y)(\ell_2)$

$= [(223+87)(12)/5163](0.5)(3.67)(21.33)(12) = 338K$

$A_{s,min} = N_c / 0.5f_y = 338/(0.5 \times 60) = 11.3 \; in^2$

$A_{s,min} = 11.3/21.33 = 0.530 \; in^2/ft$

$\boxed{\text{USE \#6 @ 9" O.C. BOTTOM} = 0.59 \; in^2/ft \text{ MIN. LENGTH SHALL BE 1/3 CLEAR SPAN AND CENTERED}}$

NEGATIVE MOMENT REGION
INTERIOR SUPPORTS:

$A_{cf} = 11"(29 \times 12) = 3828 \; in^2$

$A_{s,min} = 0.00075 A_{cf} = 0.00075(3828) = 2.871 \; in^2$

$= (15)$ #4's TOP $(3.0 \; in^2)$

EXTERIOR SUPPORTS:

$A_{cf} = 3828 \; in^2$

$A_{s,min} = 2.871 \; in^2 = (15)$ #4's TOP $(3.0 \; in^2)$

CHECK MIN. REINFORCEMENT

$d = 11" - 3/4" - 1/4" = 10"$

$A_{ps} = 0.153 \; in^2 (16) = 2.45 \; in^2$

$f_{ps} = 174{,}000 + 10{,}000 + \frac{5000(21.33)(12)(10)}{300(2.45)} = 201{,}415 \text{ PSI}$

$a = [A_s f_y + A_{ps} f_{ps}] / (0.85 f'_c b)$

$= [(3.0)(60) + (2.45)(201)] / (0.85)(5)(29)(12) = 0.455"$

$\phi M_n = 0.9 [A_s f_y + A_{ps} f_{ps}](d - a/2)$

$= 0.9[3.0(60) + 2.45(201)](10 - 0.455/2) = 549 \text{ ft-k}$
$> 475 \text{ ft-k}$
$\therefore$ MIN REINF OK

$\boxed{\text{USE (15) \#4's TOP AT INT. \& EXT. SUPPORTS}}$

AT MIDSPAN (END SPAN)

$d = 11 - 1.5 - 0.25 = 9.25"$

$f_{ps} = 200,109$ PSI

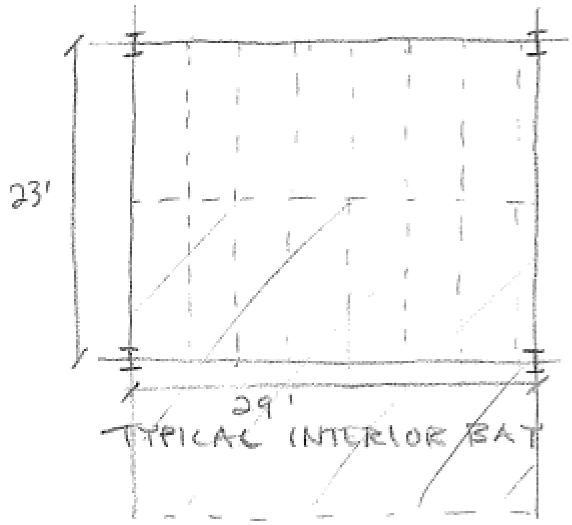
$a = [(11.3)(60) + 2.45(200)] / 0.85(5)(29)(12) = 0.79"$

$\phi M_n = 0.9[(11.3)(60) + (2.45)(200)][9.25 - \frac{0.79}{2}]/12 = 776$ ft·k
> 423 ft·k
∴ MIN REINF OK

$\boxed{\text{USE \#6 @ 9" O.C. BOTTOM}}$

Appendix E: Prestressed Hollow Core Plank on Steel

PRESTRESSED HOLLOW CORE PLANK ON STEEL



TYPICAL INTERIOR BAY

$DL = 18\,PSF + SW$

$LL = 80\,PSF$

$L_{red} = L_o\left[0.25 + \frac{15}{\sqrt{A_s}}\right]$

$= 80\left[0.25 + \frac{15}{\sqrt{2(667)}}\right]$

$= 0.66 \times 80 = 52.9\,PSF$

SERVICE LOADS (SUPERIMPOSED)

$DL + LL = 18\,PSF + 80\,PSF = 98\,PSF$

TRY 4-1/2"ϕ STRANDS IN A 8"×4' HOLLOW CORE PLANK

118 PSF CAPACITY > 98 PSF LOAD

$DL = 18 + SW = 18 + 61.25 = 79.25\,PSF$

FACTORED LOADS

$1.2(79.25) + 1.6(52.9) = 180\,PSF$

$w_u = 0.180 \times 23 = 4.14\,k/ft$

$M_u = wl^2/8 = 4.14(29)^2/8 = 435\,ft \cdot k$

TRY W21×55 ($\phi M_p = 473\,ft\cdot k$)

$\Delta_{total} = \frac{5(159.25 \times 23/1000)(29)^4(1728)}{384(29000)(1140)} = 1.76" > \frac{29(12)}{240} = 1.45"$

$\therefore$ NO GOOD

$I_{req'd} = \frac{5\left(\frac{159.25 \times 23}{1000}\right)(29)^4(1728)}{384(29000)(1.45)} = 1386\,in^4$

TRY W24×62 ($I = 1550\,in^4$, $\phi M_p = 600\,ft\cdot k$)

$$\Delta_{LL} = \frac{5\left(\frac{80(23)}{1000}\right)(29)^4(1728)}{384(29000)(1550)} = 0.65'' < \frac{l}{360} = \frac{29(12)}{360} = 0.97''$$

$\therefore$ WILL WORK

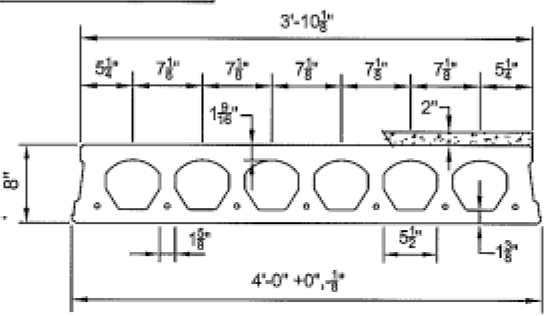
USE 8"×4'-0" HOLLOW CORE PLANKS W/ 4-1/2"ϕ STRAND PATTERN ON W24×62 BEAMS

Prestressed Concrete
8"x4'-0" Hollow Core Plank

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES Composite Section	
A_c = 301 in.^{2}	Precast b_w = 13.13 in.
I_c = 3134 in.^{4}	Precast S_{bcp} = 616 in.^{3}
Y_{bcp} = 5.09 in.	Topping S_{tct} = 902 in.^{3}
Y_{tcp} = 2.91 in.	Precast S_{tcp} = 1076 in.^{3}
Y_{tct} = 4.91 in.	Precast Wt. = 245 PLF
	Precast Wt. = 61.25 PSF

DESIGN DATA



1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
   4-1/2"Ø, 270K = 92.3 k-ft at 60% jacking force
   6-1/2"Ø, 270K = 130.6 k-ft at 60% jacking force
   7-1/2"Ø, 270K = 147.8 k-ft at 60% jacking force
7. Maximum bottom tensile stress is $10\sqrt{f'c}$ = 775 PSI
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. Topping Strength @ 28 days = 3000 PSI. Topping Weight = 25 PSF.
12. These tables are based upon the topping having a uniform 2" thickness over the entire span. A lesser thickness might occur if camber is not taken into account during design, thus reducing the load capacity.
13. Load values to the left of the solid line are controlled by ultimate shear strength.
14. Load values to the right are controlled by ultimate flexural strength or fire endurance limits.
15. Load values may be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
16. Camber is inherent in all prestressed hollow core slabs and is a function of the amount of eccentric prestressing force needed to carry the superimposed design loads along with a number of other variables. Because prediction of camber is based on empirical formulas it is at best an estimate, with the actual camber usually higher than calculated values.

SAFE SUPERIMPOSED SERVICE LOADS — IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)

Strand Pattern		SPAN (FEET)																		
		17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35
4 - 1/2"ø	LOAD (PSF)	280	248	214	185	159	138	118	102	87	74	62	52	42						
6 - 1/2"ø	LOAD (PSF)	366	341	318	299	271	239	211	187	165	146	129	114	101	88	77	67	58	50	42
7 - 1/2"ø	LOAD (PSF)	367	342	320	300	282	265	243	221	202	181	161	144	128	114	101	90	79	70	61

NITTERHOUSE
CONCRETE PRODUCTS
2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17202-9203
717-267-4505 Fax 717-267-4518

This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths. The allowable loads shown in this table reflect a 2 Hour & 0 Minute fire resistance rating.

11/03/08

8SF2.0T