Structural Technical Report 2
Pro-Con Structural Study of Alternative Floor Systems

PricewaterhouseCoopers
Oslo, Norway

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AE 481W Senior Thesis
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Executive Summary

This report contains a pro – con structural study of alternate floor systems for the PricewaterhouseCoopers (PwC) building. For reference, existing conditions, architectural background and a structural systems discussion is provided. The focus of the report is a written comparison on how three alternative floor systems relate to the PwC building. To indicate preliminary member and slab sizes, each system contains a schematic design for gravity loads only. The report is concluded with a chart that summarizes features of each floor system.

The main selection criteria of alternative systems were structural depth and ability to provide flexible column free space. Other items addressed were weight, structural depth, cost, fire protection, speed of construction, susceptibility to vibration and local labor expertise.

The following floor systems were evaluated:

+ Prestressed hollow core concrete *(existing)*
+ Composite steel beam and deck
+ Girder slab system
+ 2 way - Post tensioned concrete

The result of my study indicated the existing structural system is the best alternative for the location of Oslo, Norway. If the building were built in the US, Germany or England, composite deck could potentially yield a more economical alternative. Schematic design however, revealed a composite deck system would incur an increase in structural depth of 8”. If this system were selected for further investigation, impacts on architectural expression would need to be addressed. A schematic study on spanning precast concrete decking in the opposite direction revealed the existing layout provides a more efficient use of precast concrete elements. Post tensioned concrete was studied as a concrete alternative, but had drawbacks in relation to structural weight and spanning of continuous tendons in relation to the existing architectural layout.

1 – Existing Conditions

1.1 Architectural Background

In 2003 *Oslo S Utvikling* hosted an international architecture competition for the lot located south of the *Oslo S* train lines - between the outrun of *Akerselven* and *Middeladerparken*. The competition was jointly won by *MVRDV*, *Dark Arkitekter*, and *A-lab* with their proposal for the *Barcode* development. The new *PricewaterhouseCoopers* (PwC) building is the first building to be completed in the *Barcode* strip and will be "the face" of the *Barcode* towards the west.

The *Barcode* concept is based on a series of parallel building strips aligned in a formation that will ensure a lot of air between buildings and provide good views onto and out of the site, says *A-lab* architect Mathias Eckman (Figure 1, 2). The strip will contain a row of eight to ten buildings, each with their own individual form and character. They will abide by certain formulas and guidelines set forth by the zoning plan that regulates shape, size, function, material use, public spaces, roofing, and entrances. There is a volume guide with specific principle forms that the buildings may take on. Each building must adhere to one of the principle forms and must be completely different from the adjacent buildings. The intention is to provide unique multifunctional architecture with a lot of light, variation and accessibility.

Figure 1: Barcode Concept

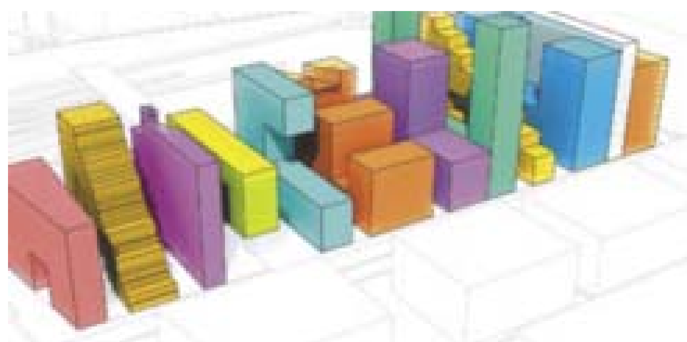


Figure 2: Image Barcode Concept

Images courtesy of Oslo S Utvikling

The exterior shape of the PwC building is simple and defined. The east side runs perpendicular to *Nydalen Alle* and the west side follows the property line, creating a rhombus like shape in plan. There are of two stories below grade and twelve above grade with a five story opening in the center of the façade indicating the main entrance.

The program inside mainly conforms to the needs of the professional services firm, PricewaterhouseCoopers. Technical rooms and parking are located on sub grade floors. The first three floors above grade contain an auditorium, a reception area, meeting rooms, and towards *Nydalen Alle*, shops and restaurants. The forth through the eleventh floors hold conference rooms and office spaces. A grand cafeteria with spectacular views and outdoor dining options is located on the top floor. The core consists of a permanent technical zone that contains communication, technical installations and wet services, in addition to zones that can be designed differently depending on the need of the different departments. The story height is 12 ft which will be similar to all buildings in the *Barcode* development.

The building envelope consists of curtainwall glazing, metal paneling and tar paper roof, intended to give off an impression of lightness, openness and technological sophistication. The attachment of the curtainwall to the building is made using steel brackets welded to the outside edges of the steel deck framing. The glass type chosen is Glaverbels Stopray Carat, and there are 8 different variations of this glass on the building. Determining which glass to use was challenging says *A-Lab* architect Mathias Eckman, as criteria for fire, sound, solar shading and safety had to be considered.

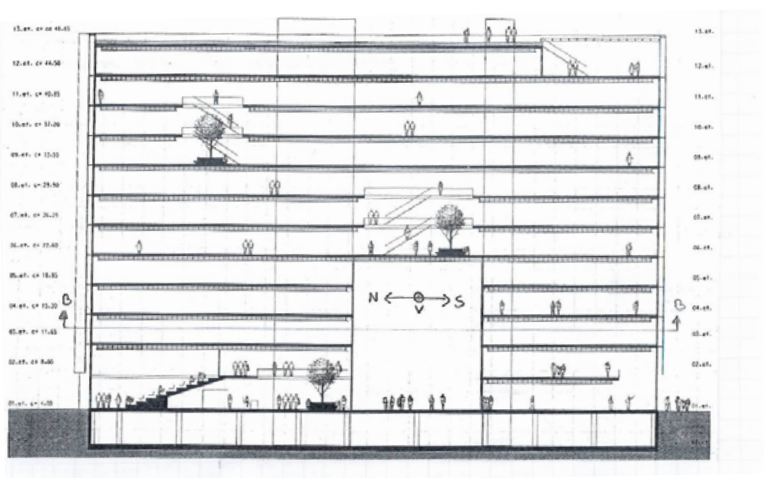


Figure 3: Building Section

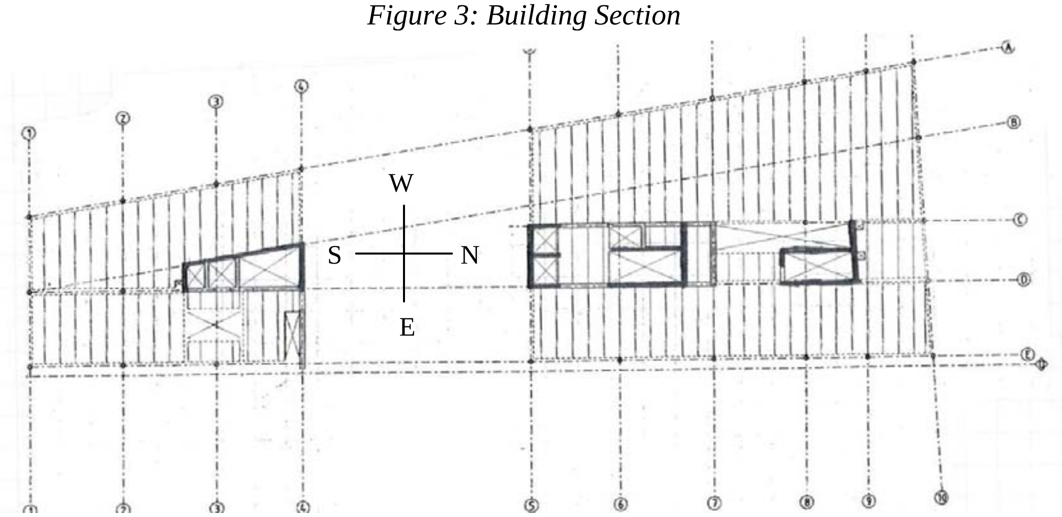


Figure 4: Typical framing plan for floors 1 – 4

1.2 Structural Systems Discussion

The superstructure of the building consists of precast concrete decking on a steel frame with cast in place concrete shearwalls at the core. The decking is prestressed hollow core concrete plank that have typical sections of 120cmx30cm, with spans ranging from 10 to 20 meters. Along the interior of the building, planks typically rest on steel angles fastened to the concrete core. Along the exterior, planks typically rest on the bottom flange of a special steel beam (HSQ). The beams are fabricated by precast engineer and conceal the flange and web within the plane of the slab, creating extremely low floor to floor height. The beams are supported by circular hollow structural steel columns filled with reinforced concrete.

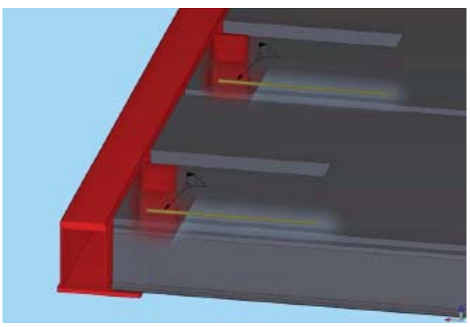


Figure 5: Principle connection of deck elements with one sided HSQ beam.

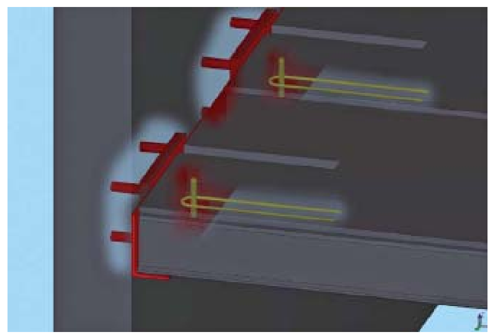


Figure 6: Principle connection of deck elements with interior concrete wall.

The grand opening at the center of the façade is allowed through three trusses comprised of hollow circular steel tubing for diagonal/vertical members and HSQ beams for horizontal members. During construction the structure was supported by three temporary columns that were removed after the integrity of the truss was intact.

Lateral resistance is provided by cast in place concrete cores, located at the center of each leg of the building. Concrete plank decking acts as a rigid diaphragm that transfers loads to the shear walls. The building is tall and narrow in the short direction and therefore requires thick shear walls. Walls are typically 400mm thick in the short direction and 300mm in the long direction. The narrow building shape also causes large overturning moments. Cores are integrated into the cast in place concrete substructure and acts as a base to distribute the overturning moments to the foundation.

There are two stories below grade comprised of cast in place concrete. The lowest level has a slab thickness of 500mm with recessed areas for elevator shafts. All other floor slabs are 300mm thick, with exception of areas below outdoor areas where slab thickness is increased to 400mm.

The foundation uses steel and concrete piles to transfer axial tension, axial compression and lateral loads to the ground. There are five different types of piles used which are driven between 100 and 130ft to bedrock. Pile capacities are dependent on pile type, connection type, and whether bending is about strong or weak axis.

1.3 Materials

Steel

Item	Euronorm	ASTM	Fu (ksi)	Fy (ksi)	Ea (ksi)	Va	Density (Ib/ft^3)
Columns	S355	A572Gr50	51	74	30 500	.3	50
Beams	S355	A572Gr50	51	74	30 500	.3	50
Reinforcing	B500C	-	-	72	30 500	-	-

Concrete

Item	Norwegian Standard	Eurocode CEN	f_{ck} (ksi)	f_{ctm} (ksi)	E_{cm} (ksi)
Cast in place	B35	C35/45	5	0.46	4 850
Prefabricated	B45	C45/55	6.5	0.55	5 222
Columns	B45	C45/55	6.5	0.55	5 222

f_{ck} - compressive cylinder strength at 28days
f_{ctm} - value of mean axial tensile strength of concrete
E_{cm} – Secant modulous of elasticity

Notes

1. Metric material strengths are converted to imperial form using 1psi = .006894 N/mm^2. Values are rounded down to nearest whole number.

2 - Alternative Floor System Discussion

2.1 Systems Selected

The existing floor system plus three alternative systems have been evaluated for the PwC building. The main selection criteria for alternatives were structural depth and ability to provide flexible column free space. The following floor systems were evaluated:

+ Prestressed hollow core concrete *(existing)*
+ Composite steel beam and decking
+ Girder slab system
+ 2 way - Post tensioned concrete

A schematic design of each system was performed to determine preliminary framing members and slabs. Since the PwC building has bays that range in size and shape, two separate bays were studied. A 24ft x19ft was chosen because it is the most reoccurring bay in the structure and will indicate typical member sizes (Bay A, figure 7). The second bay chosen is located over the auditorium and contains the largest span in the building. Although its actual shape in plan is not rectangular, it has been approximated to 24ft x 40ft (Bay B, figure 7). This will indicate floor systems applicability to the largest span in the building.

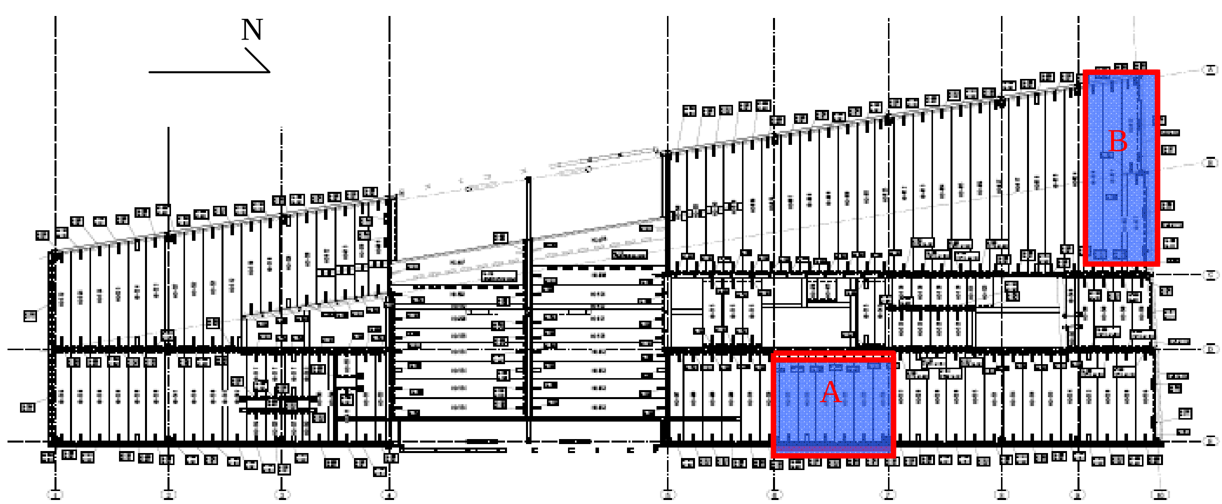


Figure 7: Bay Selection

2.2 Document and Code Review

The PwC building was designed in accordance with various sections and editions of the Norwegian Standards. As the purpose of this report is to conduct a schematic comparison, I have used reference standards most available to me. To keep design loads similar to those used by design engineer, gravity loads were determined in accordance with the Eurocodes 1991-1. Post tensioned concrete follows design methods presented in ACI 318 – 05 and IBC 2003. Load and Resistance Factor Design according to the thirtheenth edition of the AISC Manual of Steel Construction was used for steel design checks.

2.3 Gravity loads

The following gravity loads were used for conducting schematic design of alternative floor systems:

Dead

Material / Occupancy	Unit Weight (kN/m^2)	Unit Weight (psf)
Floor and Ceiling Finishes, M.E.P.	1.5	31
Façade	.7	15

Live

Area	Reference	Category	Unit Weight (kN/m^2)	Unit Weight (psf)
Office spaces	EN 1991-1 2002, Table NA.6.2	B	3	63
Corridors	EN 1991-1 2002, Table NA.6.2	C3	5	105

Notes

1. Metric unit weights have been converted to imperial form using 1psf = .04784kN/m2. Values are been rounded up to nearest whole number

2.4 Precast Hollow Core Concrete – Existing Structure

The existing structural system is adequate to handle the structural and architectural requirements. It also meets requirements for vibration and acoustic performance for office purposes. Although this system is more costly than a conventional precast concrete system, I believe the additional costs are outweighed by the architectural benefits. The interior spaces and exterior facades of the building give off an expression of lightness and transparency. I believe this is greatly due to the thin structural sandwich achieved through the floor system.

Prefabricated concrete decking is a popular floor system in northern Europe. The reason for this could be the fast speed of erection. Due to prefabrication of beams, columns, and rebar the superstructure is assembled on site incredibly fast. Considering it is commonly used, I can also conclude there is local labor expertise within this field, which potentially makes it more economical alternatives. There may however, be other factors involved which I have not addressed and needs further research.

The floor system is supported by circular HSS columns filled with reinforced concrete. According to *Design guide for concrete filled columns* by *Corus UK limited*, advantages to this column system are:

- They provide architects and engineers with a robust and inherently fire resistant column.
- During construction the steel sections dispenses with the need for formwork and erection schedule is not depended on concrete curing time.
- During finishing concrete filling is protected against mechanical damage.
- When completed, columns provide greater usable floor area, higher visibility, reduced maintenance, and are aesthetically pleasing

Overall the existing structural system does a good job achieving architectural and structural requirements for the location of Oslo, Norway.

2.5 Composite Steel Beam and Deck

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. The concrete contributes to distribution of loads and improves the acoustic and fire protection properties of the sandwich. The structure also yields flexible use floor area and placing of partition walls. For schematic design, decking was determined using *united steel deck* load span tables, while beams girders were sized using the AISC steel manual.

The schematic design began with a composite deck and concrete slab. The thinnest result was a 4" concrete slab on composite deck assembly. The decking is supported by beams spaced at 8' running in the East - West direction. This direction was chosen in order to locate deep members along the perimeter and towards the cores, thus minimizing interruption of MEP and partition layout. Trial designs for composite beams and girders were made, however trial members did not meet serviceability criteria of construction deflection under pre composite conditions. Therefore cambering of the beam or shoring during construction would be required. Since shoring has significant cost and scheduling impact, I opted for larger members. Members were not chosen on a basis of most economical shape, but on minimal structural depth. With larger members, composite action was no longer needed to satisfy flexural requirements. Therefore beams and girders do not act compositely.

A drawback of this floor system compared to the existing is the increase in structural depth. The max structural depth was estimated to be 20.5", which is an increase of approximately 8" when compared to the existing structure. Since the floor to floor height is required to be a constant of 12ft for the entire Barcode district, this would mean that the floor to ceiling height would decrease by 8".

The weight of this system is comparable to the existing structure and similar foundations could be used. The lateral system could be kept concrete or changed to an all steel option. Vertical support is typically provided by wide flange columns. As columns are exposed in the existing structure a change in columns requires consideration of architectural expression and floor plan.

If the PricewaterhouseCoopers building were built in the US, England or Germany composite deck on steel framing would be a likely choice of floor system. However in the northern part of Europe it is more common with precast concrete decking and therefore composite deck may not be as economical of an alternative. Determination of this needs further investigation. Setting structural depth and local labor expertise aside, I think composite deck is a good alternative to the existing floor system.

Composite Deck Schematic Design:

Loading

Live Load = 63psf
SIMP Dead Load = 32psf
Façade = 25psf

Material Properties

Normal weight concrete
f'c = 3000psi
fy = 50ksi

Deflection Criteria

Under Live Load = l/360
Under Total Load = l/240

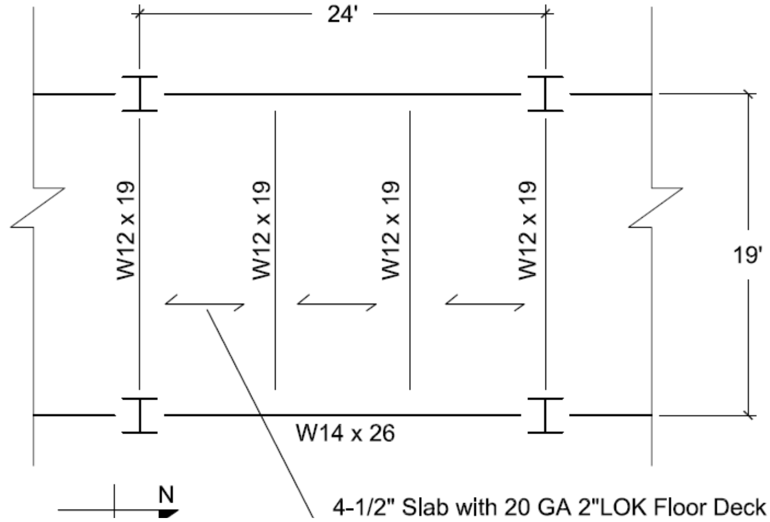


Figure 8: Framing Layout for Bay A

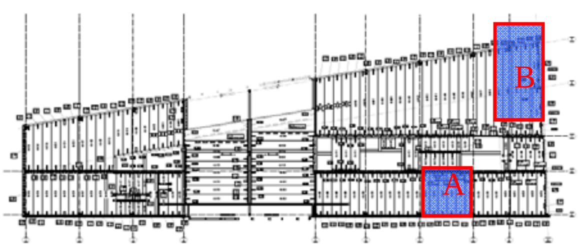


Figure 9: Overview

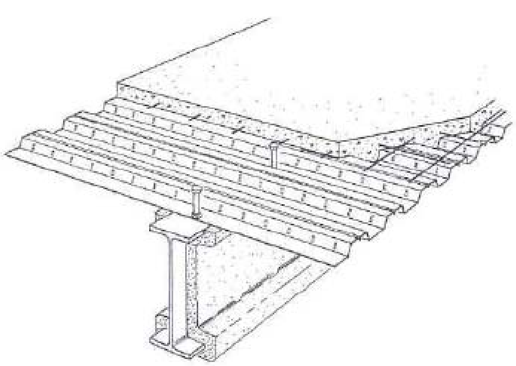


Figure 10: sketch of typical composite deck framing

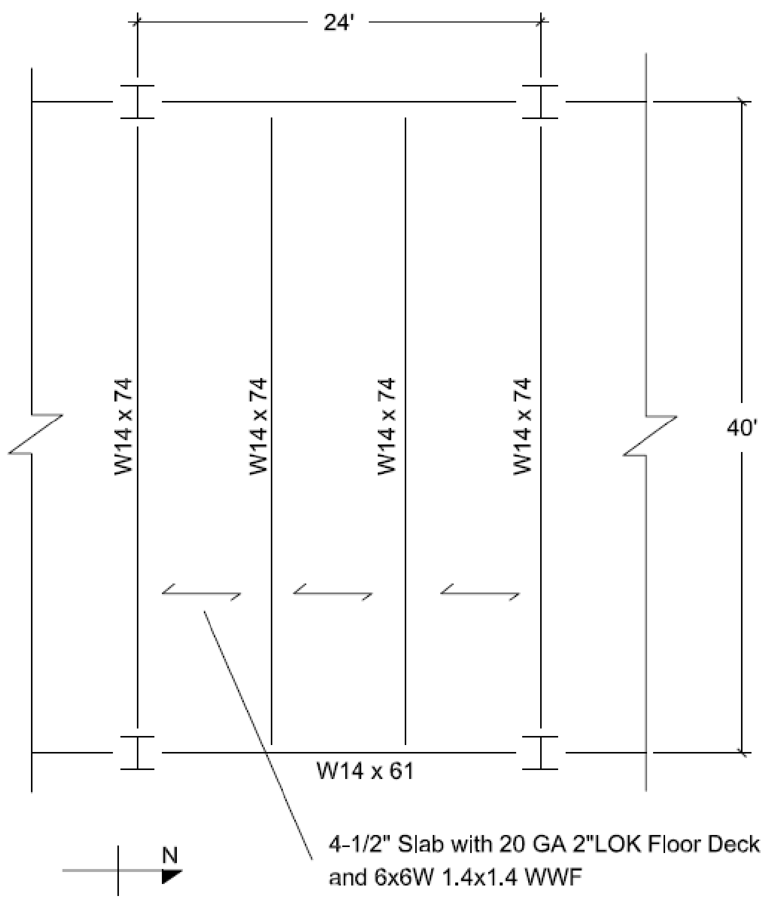


Figure 11: Framing Layout Bay B

2.6 Girder Slab System

Currently the PwC building spans precast hollow core plank in the East - West direction. A comparison was made using a girder slab system spanning precast concrete elements in the North - South direction. As this is very similar to the existing structure it has many same features as discussed in section 2.4 of this report. The system proposed consists of interior open web dissymmetric beams (D-beam) and prestressed hollow core slabs, connected by cementious grout. For schematic design, Nitterhouse load tables were used to determine precast decking and the *Girder Slab Design Guide v1.4* was used to size beams.

For bay A (figure 7) the decking selected was an 8” x 4’ prestressed concrete hollow core plank with a 2” topping. Using the plank size and subsequent weight, an interior girder was sized to be DB 9x46, yielding an overall sandwich depth of 11” inches.

For bay B (figure 7) the decking selected was the same as of Bay A, however given the required loading there are not any standard D-beams with the capacity to spanning 40ft. This does not mean that a girder slab system wouldn’t work. One option is to change the column layout and decrease the span length. This is not an alternative for the PwC building as it requires column free space over the auditorium. The most likely solution is to fabricate special beams with larger capacities (Figure 12). This comes at an additional cost and would need to be evaluated.

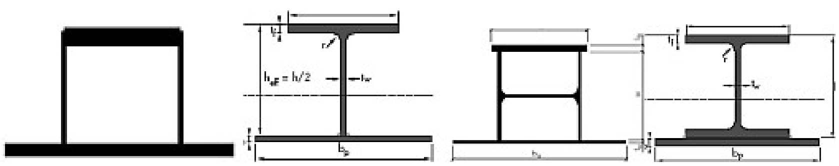


Figure 12: beams that integrate beam with deck height.

One of the design challenges with using precast concrete plank decking for the PwC building is the irregular bay shapes and sizes. The current design accommodates triangular bays by cutting the ends of planks at an angle. With the elements spanning in the North – South direction a different approach would need to be made. One solution is to use cast in place concrete where triangular sections occur. This would incur considerable cost and schedule increase and is one of the drawbacks to spanning the elements in this direction. Overall the existing structure appears to be technologically superior and have a better layout than the girder slab system proposed.

Girder Slab Schematic Design:

Loading:

Live Load = 60psf
SIMP Dead load = 32psf

Material Properties:

Plank f'c = 6000psi
Grout f'c = 4000psi

Deflection Criteria:

Live Load = l / 360

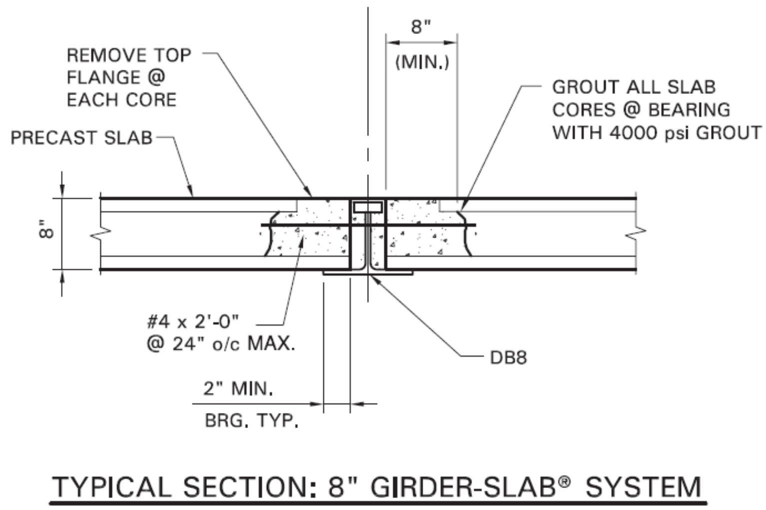


Figure 13: Typical Girder slab section
- Courtesy of Girder Slab Technologies, LLC

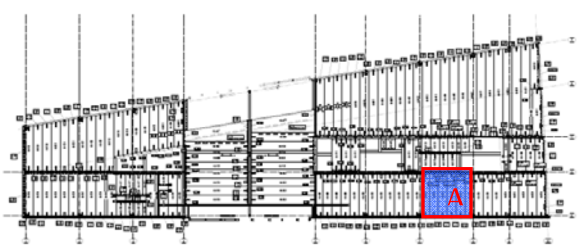


Figure 14: Framing Layout Bay B

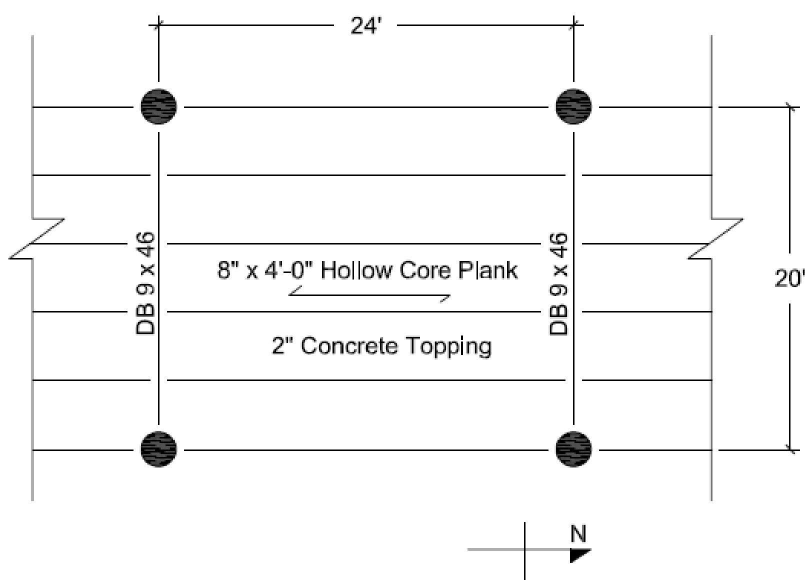


Figure 15: Framing Layout, Bay A

2.7 Post tensioned concrete – 2 way

2 way post tensioned concrete was investigated as a concrete alternative to the structural system of the PwC building. The system was chosen for it's capability of economically achieving long spans while maintaining a low structural depth. Other advantages to post tensioned systems are deflection and vibration control and structural integrity under abnormal or catastrophic loading.

Using the design aids provided by Portland Cement Association a schematic design analysis was conducted for three continuous 20ft spans. Banded tendons were run in the North - South direction and Uniform tendons in the East - West. This schematic design was mainly conducted to form an understanding of the design procedures for two way post tensioned concrete slabs. Although my knowledge on the subject is limited, I believe this system would be difficult to employ if the existing architectural layout were to be maintained. In many areas, the central cores prevent uniform tendons from running continuously across the width of the building. Another issue is the irregular bay size. When attempting to conduct simplified analysis a 15' interior span next to a 40' exterior span, I encountered difficulty in balancing the loads.

Using L/h = 45 to approximate slab thickness, the largest span requires a 12" thick slab. This is a very thin structural sandwich and would provide good of room for MEP installations and provide flexible partition layout.

An all concrete structure would yield an overall heavier building than the existing structre and capacity of foundation conditions would need to be considered. A heavier structure would however reduce overturning moments which could be a benefit considering it's narrow shape.

Overall there are a number of issues that need further investigation to determine the feasibility of this structural system. Currently the major concerns are weight in relation to foundation conditions and layout of continuous tendons in accordance with architectural layout.

Post Tensioned Concrete Schematic Design:

Loading:

Live Load = 40psf
SIMP Dead Load = 32psf

Material Properties:

<u>Concrete:</u>
f'c = 5000psi
fci = 3000psi

<u>Rebar:</u>
Fy = 60ksi

<u>Post Tensioning:</u>
½ ø strands
Fpu = 270ksi
Prestress losses = 15ksi

Deflection Criteria:

Not considered

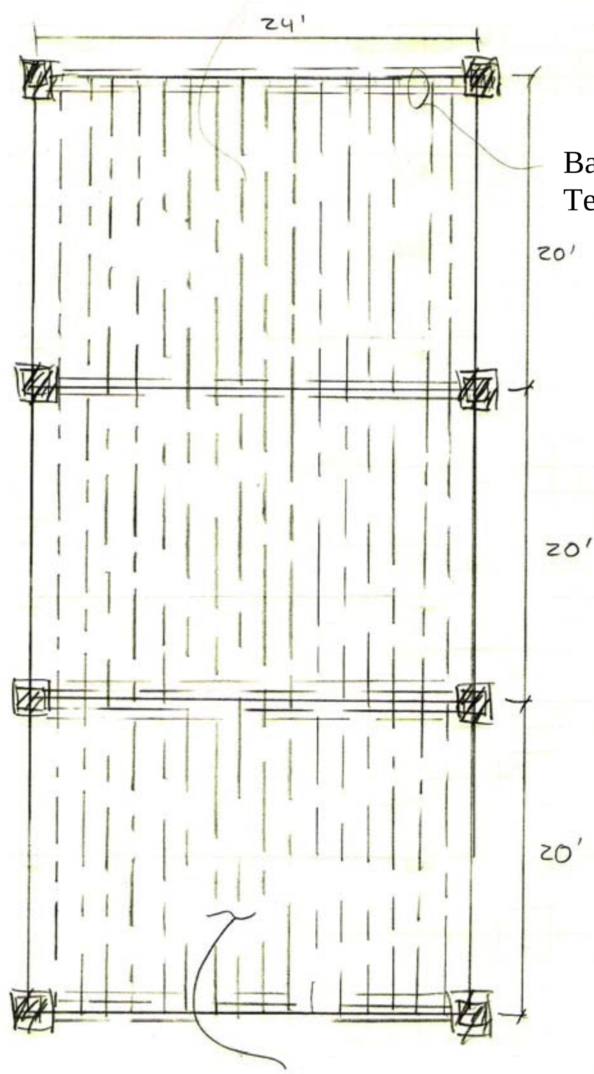


Figure 16: Plan

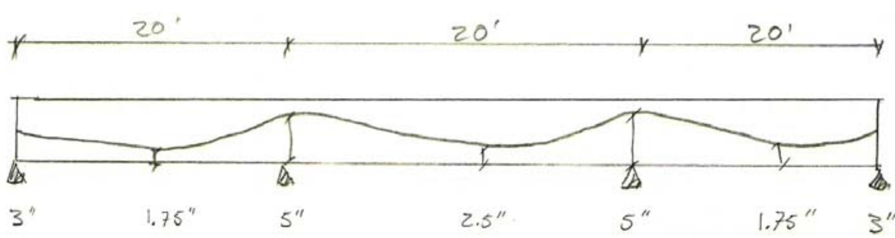


Figure 17: Section

2.8 Summary Chart

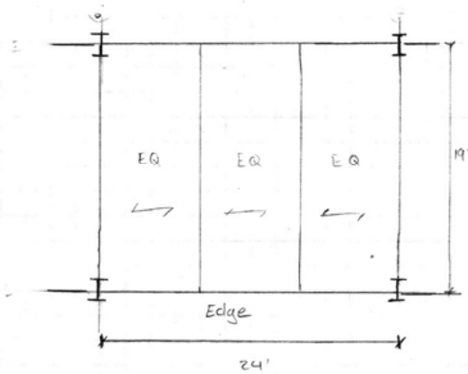
	Precast Concrete *(North – South)*	**Composite Deck**	**Girder Slab** *(East – West)*	**Post Tension Concrete**
Approximate Structural Weight	75 psf	55 psf	85 psf	75 psf / 150 psf
Approximate Max Depth	12”	20.5”	11”	6” / 12”
Vibration	-	Potentially more stiff than original system	Comparable with existing system	Heavier than existing, should dampen better
Additional Fire Proofing	SOFP Required	SOFP Required	SOFP Required	No additional fire proofing required
Constructability	Easy	Medium	Easy	Medium/Hard
Relative Cost	Medium/High	Medium/Low	Medium/High	Medium/Low
Sound Transmission	Fair	Fair / Poor	Fair	Good
Formwork required	No	No	No	Yes
Lead Time	Long	Long	Long	Short
Speed of erection	Fast	Medium	Fast	Medium
Local Labor expertise	High	Low	High	Medium
Need for ceiling finish	No	Yes	No	No
Effect on Column Grid	-	Potentially None	Potentially None	Requires Rearrangement
Overall Feasibility	-	Potential for further investigation	Existing structure is superior	Some potential

Appendix – Preliminary Design Calculations

A1 – Composite Deck

PwC | Oslo, Norway | Composite Steel B | 1

Small Bay B



Normal wt conc

$f'_c = 3000$ psi

$f_y = 50$ ksi

$w_{LL} = (1.6)(63) = 100.8$ psf

$w_{DL} = (1.2)(31) = 37.2$ psf

Slab Design

Try 4.5" slab with 20 GA. 2" Lock floor

Allowable load: -1 stud/ft = 330 psf > 135 psf ∴ ok

-No stud/ft = 200 psf > 135 psf ∴ ok

Max unshored span: 2 span = 8.97 ft > 8 ft ∴ ok

Reinforcing: Required $A_{WWF} = 0.023$ in²/ft

Use $A_{6x6\ W1.4\times1.4\ WWF} = 0.028$ in²/ft

Check Deflection: $\Delta = \frac{5 w_{LL} \ell^4}{384 E I}$

$$\Delta_L = \frac{(5)(63)(8^4)(1728)}{(384)(29.5\times10^6)(6.3)}$$

$$\Delta_L = .03 < \frac{\ell}{360} = .267 \therefore ok$$

$$\Delta_T = \frac{(5)(138+(1.2)(42))(8^4)(1728)}{(384)(29\times10^6)(6.3)}$$

$$\Delta_T = .095 < \frac{\ell}{240} = \frac{8\times12}{240} = .4" \therefore ok$$

$\boxed{\text{Use } 4\tfrac{1}{2}\text{" Slab w 20 GA. 2" Lok Floor Deck and } 6\times6 \text{ W1.4} \times 1.4 \text{ WWF}}$

Beam Design

$$M_n = \frac{w\ell^2}{8} = \frac{(138+(1.2)(42))(8)(19^2)}{8000} = 68^{1k}$$

assume $a = 1"$

$Y_2 = 4.5 - \frac{1}{2} = 4"$

$b_{eff} = \begin{cases} (8)(12) = 96" \\ \frac{(19)(12)}{4} = 57" \Leftarrow \end{cases}$

- From table 3-19 try W10×12 PNA @ 7 $I = 55.8$
  $\phi M_n = 73.2 > 68$ $\Sigma Q_n = 44.2$
- Check assumption

$$a = \frac{\Sigma Q}{(.85)(f'c)(b_{eff})} = \frac{44.2}{(.85)(3)(57)} = .3" < 1" \therefore ok$$

- Check deflection criteria

$w_D = (31+42+12)(8)$
$w_D = 680$

$$\Delta_D = \frac{\ell}{240} < \frac{5}{384}\frac{w\ell^4}{EI}$$

$$\frac{(19)(12)}{240} < \frac{(5)(680)(19^4)(1728)}{(384)(29\times10^6)(I)}$$

$$I > 72 in^4$$

PwC | Oslo, Norway | Composite Steel | 3

$$\Delta_L = \frac{(19)(12)}{360} < \frac{5}{384}\frac{(68\times 8)(19)^4(1728)}{(29\times 10^6)(I)}$$

$$I > 86 \text{ in}^4$$

Since live load deflection controls selection, the beam selection will not work compositly without scring. Opt to not use scring and go non-composite

- Try W12 × 16 $I = 103\text{in}^4$ $\phi M_p = 75.4^{1k}$

$$\Delta_{DL} = \frac{(19)(12)}{240} < \frac{(5)(31+42+16)(8)(19^4)(1728)}{384\ (29\times 10^6)(I)}$$

$$I = 78\text{in}^4 > 103\text{in}^4 \therefore \text{ok}$$

$$w_u = (1.6(63) + 1.2(31+42+16) = 207$$

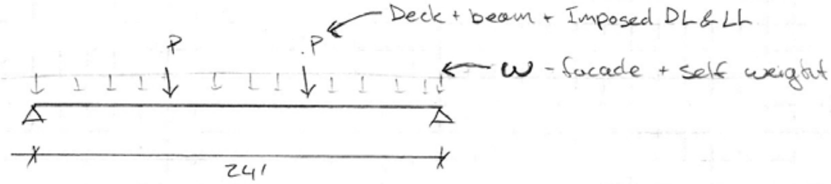
$$M_u = \frac{w\ell^2}{8} = \frac{(207)(8)(19^2)}{8} = 74.7^{1k} < 75^{1k} \therefore \text{ok}$$

$$V_u = \frac{(207)(8)(19)}{2} = 7452^k < 79.1^k \therefore \text{ok}$$

Mu is very close to ØMu ⇒ go one size up

$\boxed{\text{Use W 12} \times \text{19 Non Composite}}$

Girder Design - Exterior



- Dead Load Deflection

$P_D = (31+42)(8)(\frac{19}{2}) + (19)(\frac{19}{2})$

$P_D = 5.7^k$

$\omega_D = (15)(12) = .180 \text{ klf}$

$\Delta_D = \frac{(.0357)(5.7)(24^3)(1728)}{(29\times10^6)(I)} + \left(\frac{5}{384}\right)\frac{(180)(24^4)(1728)}{(29\times10^6)}$

$I_{req} = 167 + 46 = 213 \text{ in}^4$

- Moment on girder

$P_u = (1.2)(5.7) + \frac{1.6(63)(8)(\frac{19}{2})}{1000} = 14.5^k$

$w_u = (1.2)(180) = .216 \text{ klf}$

$M_u = (14.5)(8) + \frac{(.216)(24^2)}{8} = 131.552^{'k}$

- Try W14×26 $I = 245 \text{ in}^4$ $\phi M_p = 151^{'k}$ $\phi V_u = 106$

$M_u = (14.5)(8) + \frac{(.216+.026)(24^2)}{8} = 133^{'k}$ ∴ ok

$\Delta = 167 + \left(\frac{5}{384}\right)\left(\frac{(180+26)(24^4)(1728)}{29\times10^6}\right) = 220$ ∴ Ok

$V_u = 14.5 + \frac{(.180+.026)(24)}{2} = 17^k$ ∴ ok

$\boxed{\text{Use W14} \times 26 \text{ Non Composite}}$

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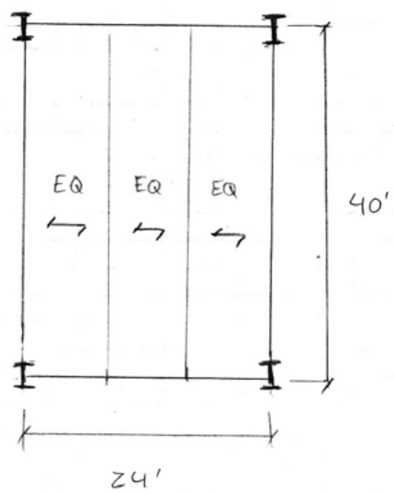
Long Bay A



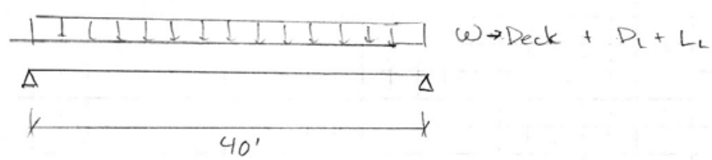
Normal weight Concrete
$f'_c = 3000psi$
$f_y = 50 ksi$
$\omega_{LL} = 63$
$\omega_{DL} = 31$

Slab Design

Loads and spans are same as with bay C therefore use same deck

Use 4 ½" slab w 20 GA 2" LOK Floor Deck and 6×6 W1.4×1.4 WWF

Beam Design



* Assume full lateral support

• Check Requirerd I for Deflection

$w_L = (63)(8) = 504$ plf

$\Delta_{LL} = \frac{l}{360} = \frac{5wL^4}{384EI}$

$\frac{(12)(40)}{360} = \frac{(5)(504)(40^4)(1728)}{(384)(29\times10^6)(I)}$

$I_{req} = 750$ in^4

$w_D = \frac{l}{240} = \frac{5wL^4}{384EI}$

$\frac{(12)(40)}{240} = \frac{(5)(31+42)(8)(40^4)(1728)}{(384)(29\times10^6)(I)}$

$I_{req} = 580$ in^4

• Required Moment

$W_u = 8[1.6(63) + 1.2(31+42)] = 1507$ plf

$M_u = \frac{wl^2}{8} = \frac{1507(40^2)}{8} = 301.4^k$

Since deflection controls, steel sections will not work under composite action without suring. Opt to use non-composite

• Try W14x74 $I = 795$ in^4 $\emptyset M_p = 473$ $\emptyset V_n = 191$

$w_u = (32+42)(8) + 74 = 666$ plf

$\Delta_{DL} = \frac{(12)(40)}{240} = \frac{(5)(666)(40^4)(1728)}{(384)(29\times10^6)\ I}$

$I = 661 < 795$ in^4 $\therefore$ OK

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- Check bending capacity

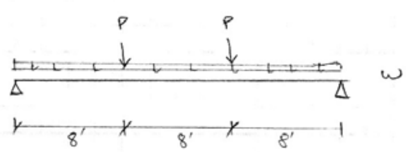
$$W_u = 1.2(32+42)(8) + (1.2)(74) + (1.6)(62)(8)$$
$$W_u = 1.6^k$$
$$M_u = \frac{(1.6)(40^2)}{8}$$
$$M_u = 320 < 473 = \phi M_p \quad \text{Ok}$$

- Check Shear

$$V_u = \frac{(1.6)(40)}{2} = 32^k < 191^k \therefore \text{ok}$$

$$\boxed{\text{Use } W14 \times 74 \text{ Non Composite}}$$

Girder Design - Exterior beam



$$P_D = (32+42)(8)(20) + (74)(20) = 13.3^k$$
$$\omega_D = (15)(12) + \text{self weight} = .18\,k/ft + \text{self weight}$$
$$P_L = (63)(8)(20) = 10^k$$

$$M_u = ((1.2)(13.3) + (10)(1.6))(8) + \frac{(.18)(24)^2}{8}$$
$$M_u = 268^{\prime k}$$

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- Determine required moment of inertia

$$\Delta_{LL} \frac{\ell}{360} = \frac{0.0357\, PL^3}{EI} +$$

$$\frac{(24)(12)}{360} = \frac{(0.0357)(10)(24^3)(1728)(1000)}{(29\times10^6)(I)}$$

$$I = 367.6$$

- Try W14 × 61 $\phi M_n = 383$ $I = 640$ $\phi V_n = 104$

- Check live load deflection

$$\Delta_{DL} = \frac{\ell}{240} = \frac{(.0357)(13.3)(24^3)(1728)(1000)}{(29\times10^6)\, I} + \left(\frac{5}{384}\right)\frac{(180+61)(24^4)(1728)}{(29\times10^6)}$$

$$\frac{(12)(24)}{240} = 391.15 + 62 I$$

$$I = 378 < 640 \therefore OK$$

$\boxed{\text{Use W14 × 61 Non Composite exterior girder}}$

United steel deck, inc. – 2” LOK - FLOOR

	COMPOSITE PROPERTIES												
	Slab Depth	ϕM_{sf} in.k	A_c in²	Vol. ft³/ft²	W psf	S_c in³	I_{av} in³	ϕM_{no} in.k	ϕV_{nt} lbs.	Max. unshored spans, ft. 1span	2span	3span	A_{wwf}
22 gage	4.50	40.27	32.6	0.292	42	1.05	5.9	29.40	5030	5.82	7.83	7.92	0.023
	5.00	46.44	37.5	0.333	48	1.23	8.0	34.53	5480	5.54	7.47	7.56	0.027
	5.25	49.53	40.0	0.354	51	1.32	9.2	37.16	5720	5.41	7.31	7.39	0.029
	5.50	52.61	42.6	0.375	54	1.42	10.5	39.81	5960	5.30	7.16	7.24	0.032
	6.00	58.78	48.0	0.417	60	1.61	13.5	45.21	6460	5.09	6.89	6.97	0.036
	6.25	61.87	50.8	0.438	63	1.71	15.3	47.95	6720	5.03	6.76	6.84	0.038
	6.50	64.95	53.6	0.458	66	1.81	17.1	50.70	6980	4.97	6.65	6.72	0.041
	7.00	71.12	59.5	0.500	73	2.01	21.2	56.26	7530	4.85	6.43	6.51	0.045
	7.25	74.21	61.9	0.521	76	2.11	23.5	59.07	7750	4.79	6.32	6.41	0.047
	7.50	77.29	64.3	0.542	79	2.21	26.0	61.98	7970	4.74	6.22	6.31	0.050
20 gage	4.50	48.60	32.6	0.292	42	1.26	6.3	35.43	5450	6.81	8.97	9.27	0.023
	5.00	56.18	37.5	0.333	48	1.48	8.6	41.65	5900	6.47	8.55	8.83	0.027
	5.25	59.96	40.0	0.354	51	1.60	9.8	44.84	6140	6.32	8.36	8.63	0.029
	5.50	63.75	42.6	0.375	54	1.71	11.3	48.07	6390	6.18	8.18	8.45	0.032
	6.00	71.32	48.0	0.417	60	1.95	14.5	54.63	6890	5.94	7.85	8.11	0.036
	6.25	75.11	50.8	0.438	63	2.07	16.3	57.96	7140	5.86	7.70	7.95	0.038
	6.50	78.90	53.6	0.458	66	2.19	18.2	61.31	7400	5.79	7.56	7.80	0.041
	7.00	86.47	59.5	0.500	73	2.43	22.6	68.09	7950	5.65	7.29	7.53	0.045
	7.25	90.26	61.9	0.521	76	2.55	25.0	71.50	8170	5.58	7.17	7.41	0.047
	7.50	94.05	64.3	0.542	79	2.67	27.6	74.93	8390	5.52	7.05	7.28	0.050

1 Stud/ft

	L, Uniform Live Loads, psf *														
	Slab Depth	ϕMn in.k	6.00	6.50	7.00	7.50	8.00	8.50	9.00	9.50	10.00	10.50	11.00	11.50	12.00
22 gage	4.50	40.27	400	365	310	265	230	200	175	155	135	120	105	95	85
	5.00	46.44	400	400	360	305	265	230	200	175	155	140	125	110	95
	5.50	52.61	400	400	400	350	300	260	230	200	175	155	140	125	110
	6.00	58.78	400	400	400	390	335	295	255	225	200	175	155	140	125
	6.50	64.95	400	400	400	400	370	325	285	250	220	195	175	155	135
	7.00	71.12	400	400	400	400	400	355	310	275	240	215	190	170	150
	7.25	74.21	400	400	400	400	400	370	325	285	250	225	200	175	155
	7.50	77.29	400	400	400	400	400	385	340	295	260	230	205	185	165
20 gage	4.50	48.60	400	400	380	325	285	245	215	190	170	150	135	120	110
	5.00	56.18	400	400	400	380	330	285	250	220	195	175	155	140	125
	5.50	63.75	400	400	400	400	375	325	285	250	225	200	175	160	140
	6.00	71.32	400	400	400	400	400	365	320	285	250	225	200	180	160
	6.50	78.90	400	400	400	400	400	400	355	315	280	245	220	195	175
	7.00	86.47	400	400	400	400	400	400	390	345	305	270	240	215	195
	7.25	90.26	400	400	400	400	400	400	400	360	320	285	255	225	205
	7.50	94.05	400	400	400	400	400	400	400	375	330	295	265	235	210

No studs

20 gage	4.50	35.43	375	315	270	230	200	170	150	130	115	100	90	80	70
	5.00	41.65	400	375	315	270	235	205	175	155	135	120	105	95	85
	5.50	48.07	400	400	365	315	270	235	205	180	160	140	125	110	95
	6.00	54.63	400	400	400	360	310	270	235	205	180	160	140	125	110
	6.50	61.31	400	400	400	400	350	300	265	230	205	180	160	140	125
	7.00	68.09	400	400	400	400	390	335	295	260	230	200	180	160	140
	7.25	71.50	400	400	400	400	400	355	310	270	240	210	190	165	150
	7.50	74.93	400	400	400	400	400	370	325	285	250	225	200	175	155

A2 – Girder Slab

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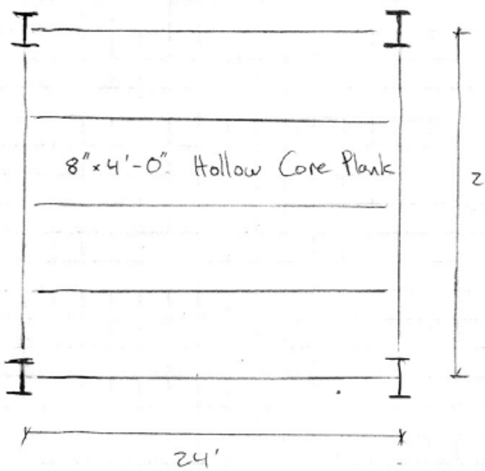

Small Bay B

Plank DL = 60psf
Live load 60psf
Topping DL = 25psf
Plank f'c = 6000psi
Grout f'c = 4000psi
Plank Span = 24'
DB Span = 20'

- Size interior D-Beam

Live Load Reduction = 26.6%

Allowable Δ_{LL} = L/360 = (20)(12)/360 = .67"

Initial Load - Precomposite

$$M_{DL} = \frac{(24)(0.06)(20)^2}{8} = 72 \text{ kft} < 84 \text{ ok}$$

$$\Delta_{DL} = \frac{(5)(24)(.06)(20)^4(1728)}{(384)(195)(29000)} = .92 \text{ in}$$

Total Load Composite

$$M_{sup} = 82$$

$$M_{TL} = 154.9$$

$$S_{req} = \frac{(154.9)(12)}{(.6)(50)} = 61.9 < 68.6 \text{ in}^3 \text{ ok}$$

$$\Delta_{sup} = \frac{(5)(24)(.065)(20^4)(1728)}{(384)(356)(29000)} = .58 < .67 \text{ in. ok}$$

Check Superimposed Compressive strength on concrete

$$N = \frac{E_s}{E_c} = \frac{29000}{(57000)\sqrt{4000}} = 8.04$$

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$S_{tc} = (8.04)(68.6) = 552 \text{ in}^4$

$f_c = (.82)(12) / 552 = 1.8$ ksi

$F_c = (.45)(4) = 1.8$ ksi ≥ 1.8 ksi ∴ ok

Check bottom flange tension stress

$f_b = \frac{(.82)(12)}{50.8} + \frac{(.82)(12)}{80.6} = 29.3$ ksi

$F_b = (.9)(50) = 45 \text{ ksi} > 23.6$ ∴ ok

Check Shear

Total Load $= (60)(.73) + 25 + 60 = 129$ psf

$w = (.129)(24) = 3.1$ K/ft

$R = (3.1)(20)/2 = 31$ K/ft

$f_v = (31)/(0.375)(5.75) = 14.4$ ksi

$F_v = (.4)(50) = 20 \text{ ksi} > 14.4$ ksi ∴ ok

use a DB 9×46 w/ 8" hollow core slabs w/ 2" concrete topping

CAMPAD

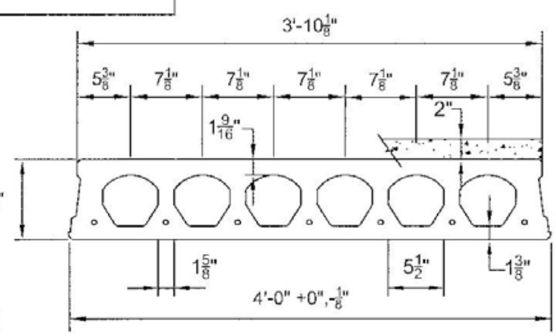
Prestressed Concrete
8"x4'-0" Hollow Core Plank

1 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES
Composite Section

A_c = 301 in.^{2}	Precast S_{bc} = 617 in.^{3}
I_c = 3134 in.^{4}	Topping S_{tc} = 902 in.^{3}
Y_{bc} = 5.09 in.	Precast S_{tc} = 1076 in.^{3}
Y_{tc} = 2.91 in.	Wt.= 245 PLF
	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
   7-1/2"Ø, 270K = 147.7 k-ft
7. Maximum bottom tensile stress is $7.5\sqrt{f'c}$ = 580 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 allowable service stresses.
15. Load values will 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 2003 & ACI 318-02 (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)	277	247	220	198	179	164	148	135	116	99	84	70	57						
7 - 1/2"ø	LOAD (PSF)	367	342	319	299	281	265	247	225	205	186	165	145	128	112	98	85	75	63	53

NITTERHOUSE
CONCRETE PRODUCTS

2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17201-0813
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 1 Hour & 0 Minute fire resistance rating.

08/10/07

8SF1.0T

A3 – Post Tension Concrete – 2 Way

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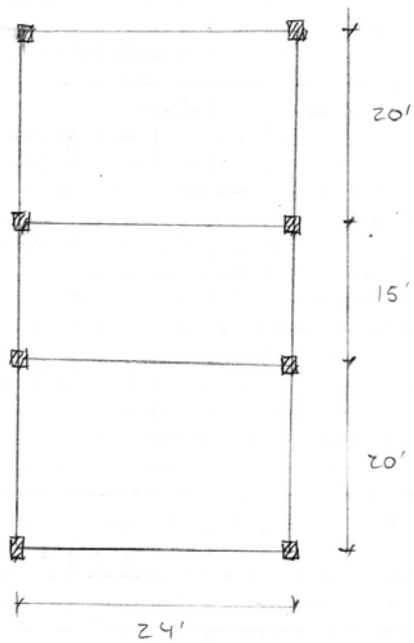
Two Way Post-Tension Design - Interior Frame



Loads

Framing D_L = Self Weight

Super imposed D_L = 31 psf

LL_o = 63 psf

2 hour fire rating

Materials

Concrete: Normal weight 150 pcf

f'_c = 5000 psi

f'_{ci} = 3000 psi

Rebar: f_y = 60 000 psi

PT: Unbounded tendons

1/2" Ø, 7 wire strands $A = 0.153 \text{ in}^2$

f_{pu} = 270 ksi

PwC | Oslo, Norway | Post - Tension 2

Materials

Concrete: Normal weight 150 pcf

$f'_c = 5000$ psi

$f_{ci} = 3000$ psi

Rebar: $f_y = 60\,000$ psi

PT: Unbounded tendons

1/2" Ø, 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

$P_{eff} = A^* f_{se} = (0.153)(174) = 26.6$ kips / tendon

Determine Preliminary Slab Thickness

Start with L/h = 45

Longest span = 20'

$h = (20)(12)/45$

$h = 5.3''$

Use h = 6" preliminary slab thk.

Loading

DL = Self weight = (6)(150) = 75 psf

SIDL = 31 psf

LL_o = 63 psf

· Live load Reduction

$A_T = (20)(24) = 480 ft^2$

$K_{LL} = 1$

$LL = (63)(.93) = 59 psf$

Design of interior frame

· Use equivilent frame method
· Total bay width = 24'
· Ignore column stiffness

Calculate Section Properties

$A = bh = (288)(6) = 1728 in^2$

$S = bh^2/6 = (288)(6^2)/6 = 1728 in^3$

Set Design Parameters

Allowable stresses: Class U

$f'_c = 3000 psi$

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

Tension $= 3\sqrt{f'_{ci}} = 3\sqrt{3000} = 164 psi$

At service Loads

$f'_c = 5000 psi$

Compression $= .45 f'_c = .45(5000) = 2250$

Tension $= 6\sqrt{f'_c} = 6\sqrt{5000} = 424 psi$

Average precompression limits

$P/A = 125$ psi min

300 psi max

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Target load balance

$.75\ w_{DL} = (.75)(75) = 56.25$ psf

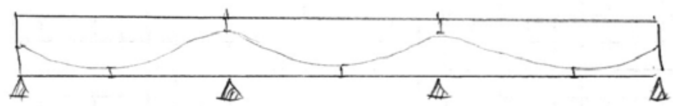
Cover Requirements

Restrained slabs = 3/4" bottom

Unrestrained slabs = 1½" bottom

3/4" top

Tendon Profile.



Tendon Ordinate	Tendon Location
Exterior support - anchor	3"
Interior support - top	5"
Interior span - bot	2.5"
End span - bot	1.75"

$a_{int} = 5 - 2.5" = 2.5"$

$a_{ext} = \frac{3+5}{2} - 1.75" = 2.25"$

Prestress Force Required

Assume ext span governs

$w_b = (.75)(75)(24) = 1.35$ k/ft

$P = w_b L^2 / 8^2 a_{end} = (1.35)(20^2)/(8)(2.25/12)$

$P = 360^k$

Check Precompression Allowance

Determine number of tendons

\# tendons = (360) / (26.6) = 13.53

∴ Use 16 tendons

P_{actual} = (16)(26.6) = 425.6 ^{k}

$w_b = \left(\frac{425.6}{360}\right) 1.35 = 1.596$

P_{actual} / A = 425.6 / 1800 = 236 > 125 ok
< 300 ok

Check interior Span Force

P = (1.35)(20²) / (8)(2.5/12)

P = 324

w_b = (425.6)(8)(2.5/12) / (20²)

w_b = 1.77

w_b / w_{DL} = 98 % < 100 % ok

Effective prestress force $\boxed{P_{eff} = 425.6^k}$

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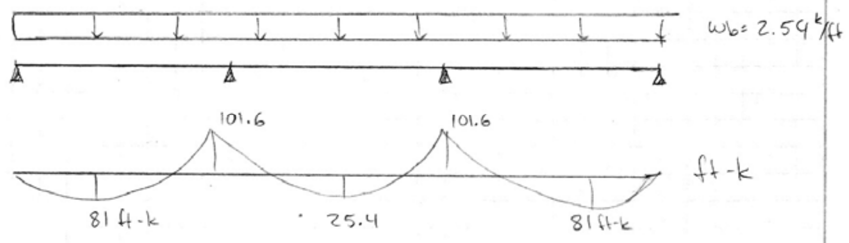
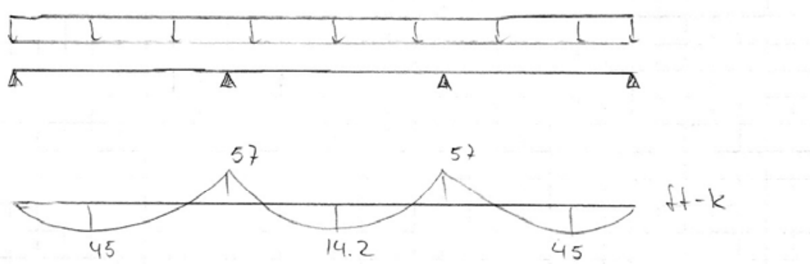
Check Slab Stresses

• Dead Load Moments $w = (75 + 31)(24)/1000$

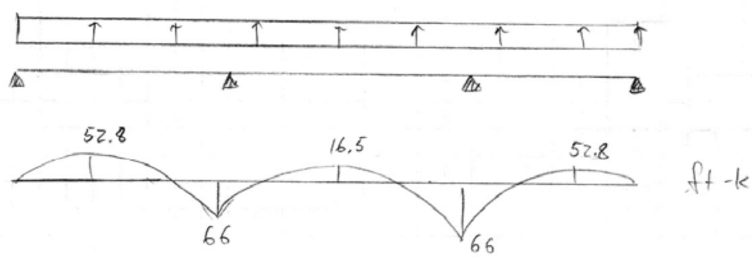


• Live Load Moments $w_{LL} = (59)(24)/1000 = 1.42\ ^k/ft$



• Total Balancing Moments, M bal

$w_b = -1.65\ k/ft$ (average of 3 bays)



Stresses emmediatly after jacking

	Midspan stresses		support stresses
	interior	end	
M dead load (k-ft)	25.4	81	101.6
M live load (k-ft)	14.2	45	57
M balance (k-ft)	16.5	52.8	66
S (in3)	1728	1728	1728
P (k)	425.6	425.6	425.6
A (in2)	1800	1800	1800
ftop (psi)	-298.3	-432.3	10.8
fbot (psi)	-174.6	-40.6	-483.7
Allowable top (psi)	1800	1800	164
Allowable bot (psi)	1800	164	1800
Within Limits	OK	OK	OK

Stresses at servive loads

	Midspan stresses		support stresses
	interior	end	
M dead load (k-ft)	25.4	81	101.6
M live load (k-ft)	14.2	45	57
M balance (k-ft)	16.5	52.8	66
S (in3)	1728	1728	1728
P (k)	425.6	425.6	425.6
A (in2)	1800	1800	1800
ftop (psi)	-396.9	-744.8	406.6
fbot (psi)	-76.0	271.9	-879.5
Allowable top (psi)	2250	2250	424
Allowable bot (psi)	2250	424	2250
Within Limits	OK	OK	OK

Ultimate Strength

Factored moments

$M_1 = P \cdot e$

$e = 0$ @ exterior support

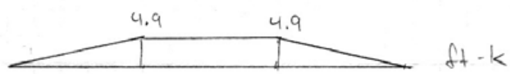
$e = 2''$ @ interior support

$M_1 = \frac{(425.6)(2)}{12} = 70.9$

Secondary post tensioning moments

$M_{sec} = M_{bal} - M_1$

$M_{sec} = 66 - 70.9 = -4.9$ ft-k



Typical load combination for ultimat strength

$M_u = 1.2 M_{DL} + 1.6 M_{LL} + 1 M_{sec}$

@ Midspan $M_u = 164.3$ ft-k

@ Support $M_u = -218$ ft-k

Determine minimum bounded reinforcement

Positive moment region:

Interior span: $f_t = -76$ No positive reinf required

end span: $f_t = 271.9 < 141$ psi

Minimum positive reinforcement required

$y = f_t / (f_t + f_c) h$

$= 271.9 / (271.9 + 744.8)(8)$

$= 2.14$ in

$N_c = (M_{DL} + M_{LL} / S)(.5)(y)(l_2)$

$= (81 + 45 / 1728)(.5)(2.14)(24)(12)$

$= 269.64^k$

$A_{s,min} = N_c / .5 f_y$

$= 269.64 / (.5)(60)$

$= 8.99$ in^2

$A_{smin} = 9$ in^2 / 24 ft

$= .3745$ in^2/ft

Use #5 @ 10 in O.C

Negative Moment Region

$A_{s\,min} = .00075 A_{cf}$

Interior supports

$A_{cf} = (8)(20)(12) = 1920$

$A_{s,min} = (.00075)(1920)$

$A_{s,min} = 1.44$ in^2 8 - #4 top (1.6 in^2)

Exterior Supports

$A_{ct} = (8)(20)(12) = 1920$

$A_{s\,min} = (.00075)(1920)$

$= 1.44 in^2$

8 # 4 Top ($1.6 in^2$)

Place bars (1.5)(6) = 9" away from supports

Max bar spacing 12"

Check minimum reinforcement for ultimate Strength

$M_n = (A_s f_y + A_{ps} f_{ps})(d - a/2)$

$A_{ps} = (0.153)(14)$

$= 2.142 in^2$

$f_{ps} = 174\,000 + 10\,000 + \frac{(5000)(24 \cdot 12)d}{(300)(2.142)}$

$f_{ps} = 184{,}000 + 2241d$

At Supports

$d = 6" - 3/4" - 1/4" = 5"$

$f_{ps} = 195{,}205$ psi

$a = [(1.6)(60) + (2.142)(195)] / [(0.85)(5)(24 \times 12)$

$= .42$

$\phi M_n = .9[(1.6)(60) + (2.142)(195)][6" - (.42)/2] / 12$

$= 223 > 218k$ Governs over ultimat serength

$A_{s\,min} = 1.6 in^2$ still ok

8 # 4 Top @ interior support
8 # 4 Top @ exterior support

Midspan (end span)

$d = 6 - 1\frac{1}{2}" - \frac{1}{4}" = 4.25"$

$f_{ps} = 184000 + (2241)(4.25) = 184952$ psi

$a = [(9)(60) + (2.142)(185)] / [(.85)(5)(24 \times 12)] = .76$

$\phi M_n = .9\,[(9)(60) + (2.142)(185)][4.25 - (\frac{.76}{2})] / 12$

$\phi M_n = (.9)(936)(3.87) / 12$

$\phi M_n = 271.8 > 164.3$ ft-k Need to increase reinf

$y = [272 / (272 + 744.8)]\, 6$

$= 1.6$ in

$M_c = (81 + 45 / 1728)(.5)(1.6)(24)(12)$

$= 202$

$A_{s\,min} = 202 / .5 f_y$

$= (202)(.5)(60)$

$= 6.7\ in^2$

$\boxed{\#5 @ 12" \text{ oc Bottom at end Spans}}$