Technical Assignment 2

Medical Office Building

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October 27, 2004

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Executive Summary

This report looked at alternative floor framing systems for the Medical Office Building. The original monolithically cast slab on beam system was compared to a composite slab on steel beam system, a slab on steel joists system, a flat plate system and a precast panel system. The systems were compared on the basis of cost, impact on usable space, and compatibility with the architecture.

The composite slab on steel beam system resulted in a 4" slab resting on W14x38 beams. Three beams spanned 28'-0" between W30x124 girders. The cost of this system was $3.69/S.F. higher than that of the original cast-in-place (CIP) slab on beam and the floor sandwich depth increased 16". Because of the increased cost and decreased usable space, this system was ruled a nonviable option.

The slab on steel joist system had a 2" slab over 16K2 joists. The joists were spaced 2'-0" O.C. and spanned 28'-0". The girders were classified as 24G14N8.5K joist girders. The steel joists did save 69 cents/S.F. compared to the original system, but they had major architectural shortcomings. First, the system could at best achieve a 2 hr. fire rating. Second, the system could not handle the architectural curve along the southeastern wall. Finally, the floor sandwich depth increased 8". If longer spans could be accomplished this system may be viable, but the current design does not offer enough benefits for further consideration.

The flat plate system is an 11.5" slab resting directly on the columns. The reinforcing in the slab is #8 bars spaced evenly across the section of the slab. Thermal expansion requirements governed for the positive section of the middle strip. The column strips required 15 bars on the bottom and 6 bars on the bottom. The middle strip required 5 bars top and bottom. The flat plate costs $3.27/S.F. less than the original system and offers several benefits. The most notable benefit is the 6.5" decrease of the floor sandwich depth. In addition, this system offers equivalent fire protection because its slab is thicker than the original, and it can match the architectural curve. However, this system would require a new strategy for lateral support in the building. Even with this shortcoming, this system offers a viable alternative to the CIP slab on beam.

The precast concrete system consists of 8DT24 with 68-S strands planks spanning 28'-0". The planks rest on a 28IT36 precast inverted tee beam. No topping was provided because a raised floor system will rest on the slab. This system saves $7.95/S.F. compared to the original, making it less than half the cost. However, similar to the joist system it would have difficulties managing the curve so some CIP work would still be necessary. Another draw back is that the floor sandwich depth increases 18", doubling in size. Switching to a hollow core slab could alleviate this problem, but that switch would require that the beams be CIP because no precast beams have a ledge with the same depth as the hollow core planks. Based on the possibility of using a hollow core plank on CIP beams, this system remains a viable option.

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Introduction

The selection of floor systems in buildings has a large impact on several other design features. The existing floor system (Figure 1) in the Medical Office Building is a one-way slab on beam system monolithically cast in concrete. Filigree beams, 8'-0" wide and 18" deep, carry the weight of the slab, which varies between 9" and 10" of thickness. In addition, these beams act as part of a moment frame lateral system. This system has fairly large spans, 28'-0" in both directions and a relatively small sandwich height at 18", but other systems may provide a better value.

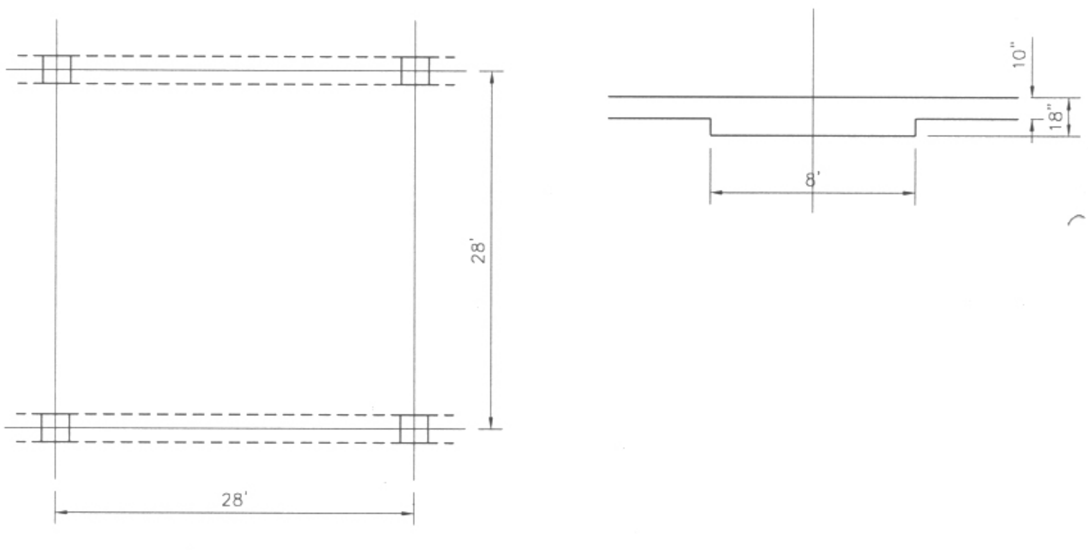


Figure 1 – Existing Floor System

A look at the framing plan (Figure 2) shows that most of the bays are uniform in size, besides the bays that intersect the curve along the southeastern face. Although cast-in-place (CIP) concrete is beneficial around the curves, using unit pieces could greatly speed up the construction process. Further time savings, and therefore money savings, might be achieved by switching to a steel system. Although time and money are important, the amount of usable space in a building is often a major consideration of owners, occupants and designers. A system that would result in a smaller floor sandwich depth could have greater benefits than construction cost savings during construction. This report explores four floor systems and the impact they have on the overall cost and quality of the building.

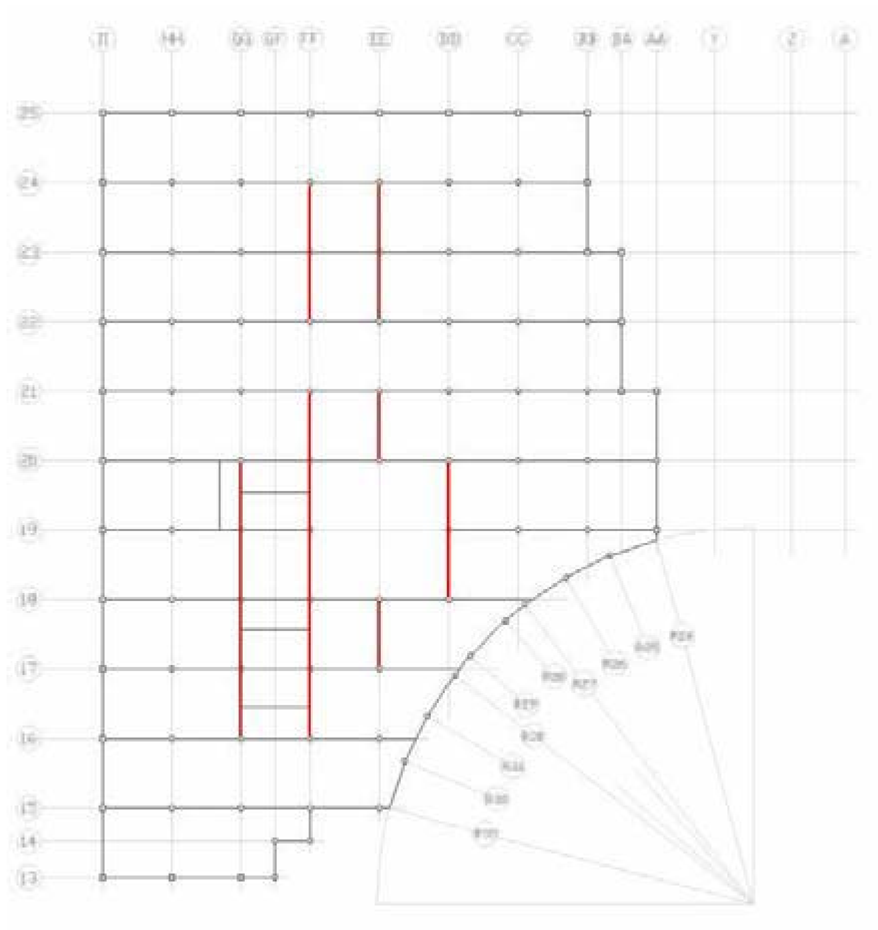


Figure 2 – Typical Framing Plan

The four systems being considered are:

- Composite Slab on Beam
- Slab on Steel Joists
- Flat Plate
- Precast Concrete Panels and Beams

The floor systems are designed based on gravity loading only. The design assumed an interior bay, but used the highest values for moment and shear, which often occur on an exterior bay. The loading was selected for a typical 2nd floor open office area. Certain amounts of superimposed dead load were added based on existing architectural conditions (Table 1).

	Regular	**Construction**
Live Load for a Typical floor	100 psf	20 psf
Superimposed Dead Load	25 psf	25 psf
Elevated Floor	5 psf	5 psf
Mechanical & Electrical	10 psf	10 psf
Flooring	5 psf	5 psf
Drop Ceiling	5 psf	5 psf

Table 1 – Loads for Floor Design

Composite Slab on Beam

The composite slab on beam system (Figure 3) consists of a 4" CIP concrete deck resting on three W14x38 beams spaced at 7'-0" on center (OC). Forty-four (44) shear studs develop a composite stress in the concrete of 540k. The beams bear on W30x124 girders that carry the load to the columns. Both the beam and girder design were controlled by deflection. Smaller sizes may be achievable by cambering the members, but no such design was undertaken for this report.

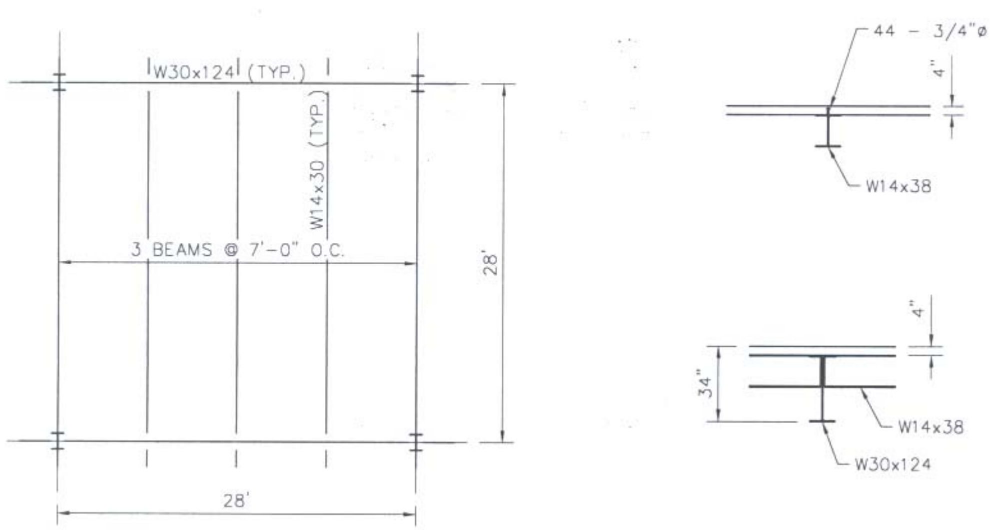


Figure 3 – Composite Slab on Beam Schematic

Benefits

The use of a structural steel framing system has several advantages over CIP concrete. First, structural steel is modular, and can be quickly placed in the uniform bays. Second, the additional support under the slab saves up to 6" of concrete over all the floors. It is also possible to purchase a curved beam for the southeastern face of the building, so the architecture is not affected.

Disadvantages

There are several drawbacks to a steel frame floor system. The most noticeable drawback is that the beams intersect an area that was once open plenum space. This has a large impact on the mechanical and electrical (MEP) professionals. Also, even though 6" were removed from the slab, the total depth of the composite floor frame increased to 18". Even worse, the girders that replaced the concrete beams are 30" deep, making the floor sandwich over 34", 16" more than the original 18". Not only does the sandwich depth increase and the plenum become cluttered, but steel requires fire proofing to meet the same protection standards as the CIP concrete system.

Slab on Steel Joists

A 2" slab reinforced with welded wire fabric (WWF) on non-composite metal deck serves as the floor in the slab on steel joist system (Figure 4). The joists carrying the slab are Vulcraft 16K2 joists spaced 2'-0" OC. The joists rest on 24G14N8.5K girders that transfer the load to steel columns. Although joists are capable of much longer spans, a brief investigation suggested that doubling the span would result in an infeasible design so the original 28'-0" spans were maintained.

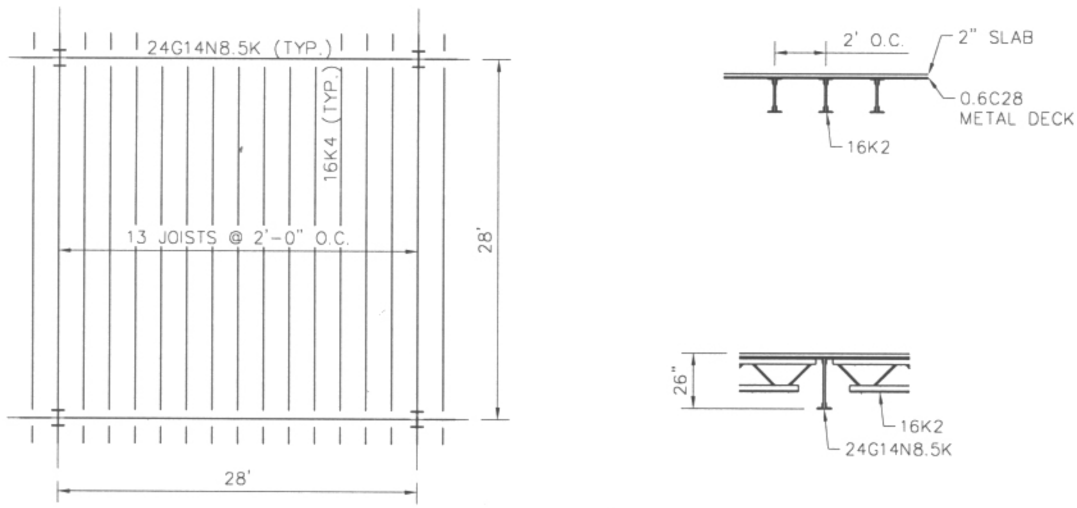


Figure 4 – Slab on Steel Joists Schematic

Benefits

Steel joist systems work best over long spans with light loads. The tight spacing of the joists resulted in a much smaller slab, saving 8" of concrete throughout most of the building. Also, similar to the beams, the joists' modular nature makes construction much easier and faster compared to CIP concrete. Although joists also interfere with the plenum space, their open nature makes them less intrusive than beams. Another benefit of using joists is that the atrium already uses joists, so the equipment needed for the process would be put to more use than in the current plan.

Disadvantages

Joists have all the same drawbacks of steel beams, including interfering with plenum space, increasing floor sandwich depth, and requiring fire protection. The total sandwich depth after installing joists would be 26", which is 8" more than the original system. Another major drawback of this system is its inability to handle the curve on the southeastern wall.

Flat Plate

The flat plate system (Figure 5) supports itself on columns spaced at 28'-0". The depth of the system was chosen as 11.5" to meet deflection requirements. This depth also allowed the system to meet punching shear requirements at the column without the need of shear reinforcing. The reinforcing of the slab is handled by #8 bars placed in both the column strips and middle strips. The negative reinforcing for the column strips required 15 #8 bars over 14'-0", while the positive reinforcing required only 6 #8's over 14'-0". The middle strip reinforcing both top and bottom was 5 #8's over 14'-0". The positive moment of the middle strip called for 4 # 8's but the minimum steel for thermal stresses required five.

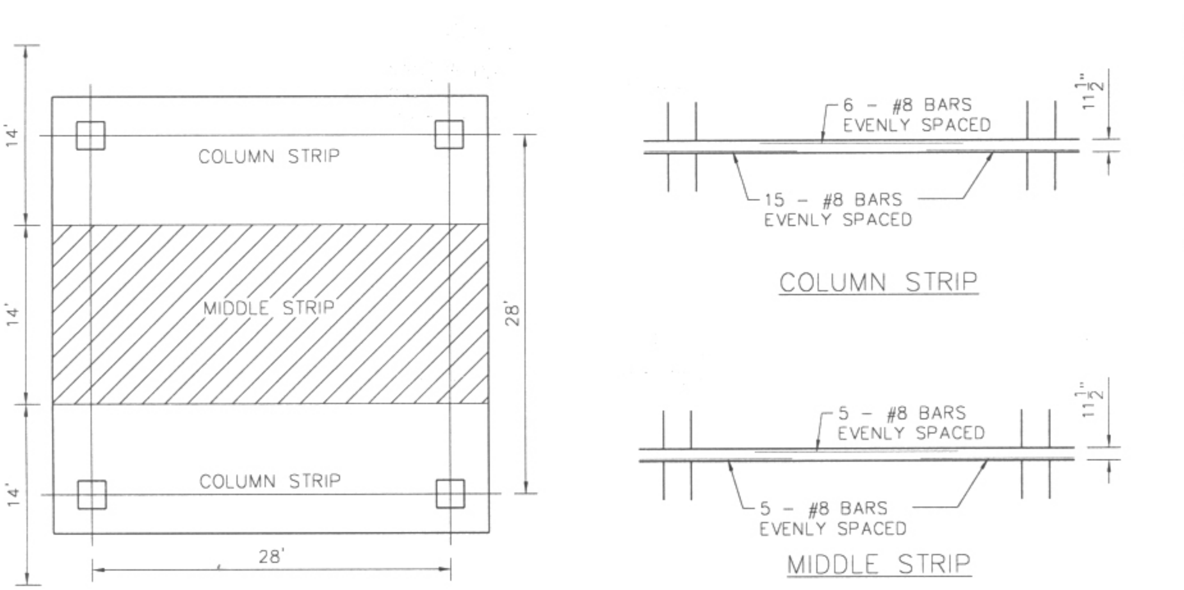


Figure 5 – Flat Plate Schematic

Benefits

The flat plate system addresses the problem of increasing usable building space very effectively. The new slab depth of 11.5" is 1.5" thicker than the previous slab, but the floor sandwich depth is reduced 6.5" from 18". This space can either be recovered for usable space or given to the MEP for a simpler design. This system is also much easier to cast than a slab with beams because there is less formwork. Also as a CIP system the flat plate will not be affected by the southeastern curve.

Disadvantages

Although flat plates are easier to form, they require a lot of shoring. Also, intuition says that without the stiffness lent by beams vibration could be an issue. The other drawback to this system, which also applies to both previous systems, is that it removes one of the lateral systems from the building. This means that a completely new lateral resisting system would be necessary.

Precast Concrete Panels and Beams

The precast concrete panel system (Figure 6) consists of double tee planks resting on an inverted tee beam. The planks are classified as 8DT24 with strand 68-S and the beams are 28IT36 with strand #16. This system was designed using the PCI Handbook. No topping was specified in consideration of reducing the floor sandwich depth. Along the same lines, hollow core slabs were considered, but no inverted tee beam had a height above it's ledge equal to the depth of the hollow core slab required.

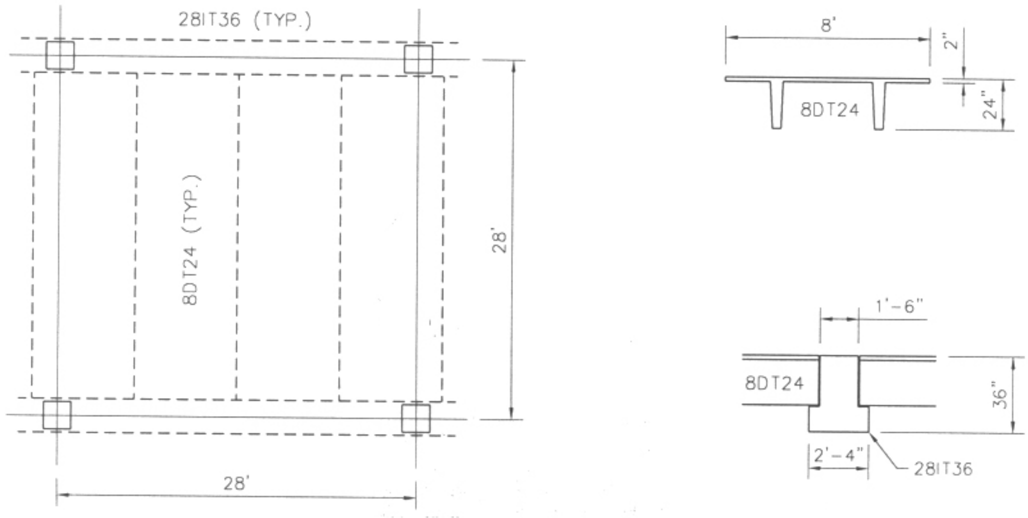


Figure 6 – Precast Concrete Panels and Beams

Benefits

Precast concrete combines the benefits of modular design with the efficiency of concrete. The system can be constructed very quickly and is generally lighter than a CIP concrete system. Also, because no topping was specified there may be no need for a concrete pump on the site.

Disadvantages

Double tee beams are very deep, which puts them in the same category as beams and trusses. The panels interfere with plenum spaces and reduce the usable space. Furthermore, because of the need for a ledge to rest the planks on, the floor sandwich depth increases to 36”, which is twice the original depth. Also, similar to the joist system, the precast concrete panels and beams do not fit to the curved shape.

Cost Comparison

The cost of each of the systems was estimated using an assembly estimate from R.S. Means 2004. Because the area of the floor is equal in all systems, a comparison of the per square foot cost accurately reflects the total cost of the system. The prices were not adjusted for time, but were adjust for location near Reading, PA:

Materials: 93.5% Installation: 104.6% Total: 98.5%

The cost of construction extends beyond monetary expense alone. In addition to the price of construction, the effects on the floor depth and the fire rating need to be considered. The following table (Table 2) summarizes the costs associated with each system.

System	Material $/S.F.	Installation $/S.F.	Total $/S.F.	Sandwich Depth	Fire Rating	Consider Further?
Existing (CIP slab on beam)	4.68	10.46	14.78	18"	3 hr	-
Composite Slab on Beam	8.42	10.09	18.47	34"	varies	N
Slab on Steel Joists	7.85	6.17	14.09	26"	≤2 hr	N
Flat Plate	4.24	7.48	11.51	11.5"	3 hr	Y
Precast Concrete Panels and Beams	4.96	1.70	6.83	36"	3 hr	Y

Table 2 – Cost Summary

If only the financial costs are considered, precast concrete panels are the ideal system, saving nearly 40% against the next least expensive system. However, the precast system is also the deepest system and interferes with the plenum space. The flat plate has the minimum floor sandwich depth, but costs a bit more, although it still saves money compared to the existing system. Neither steel system is significantly competitive with the concrete systems.

Conclusions

Several floor systems could be used for the Medical Office Building. Of the four systems tested, three cost less than the original system and one cost more. Also, one system increased the usable space, whereas the other three systems decreased the usable space. The steel system both costs more and uses more space and is therefore not worth further investigation. The flat plate system offers both savings and increased usable space.

The flat plate system decreased the floor sandwich depth by 6.5" and costs roughly $3.00 less than the original system. A more thorough investigation of this system may result in an improved design. However, the flat plate system does remove the existing lateral support system, which means that an entirely new lateral support system would need to be designed. The precast system could be built to maintain the current lateral system.

The precast system is the least expensive of the systems explored, but it also results in the biggest floor sandwich depth. A careful redesign of this system or an integration of a CIP beam instead of a precast beam could allow for a smaller floor sandwich. This is particularly true if a CIP slab were designed to accommodate the hollow slab planks that are only 8" thick. The curved surfaces could also be taken care of by using CIP along the southeastern face. The joist system also had problems with curved surfaces.

Considering the inflexibility of steel joists, they do not appear to be a good system for use around the curve. This shortcoming in addition to the marginal cost savings and increased floor sandwich depth makes the steel joist system seem very unattractive.

Appendix

Technical Report #2 | Overview | Brendon J. Burley | 1/1

Loads

Typical Floor Live Load - 100 psf

Superimposed Dead Load - 25 psf
- Elevated Floor - 5 psf
- Mechanical & Electrical - 10 psf
- Flooring - 5 psf
- Drop Ceiling - 5 psf

Systems

One-way slab on steel girders

Slab on beam steel system

Composite slab on beam steel system ⟵

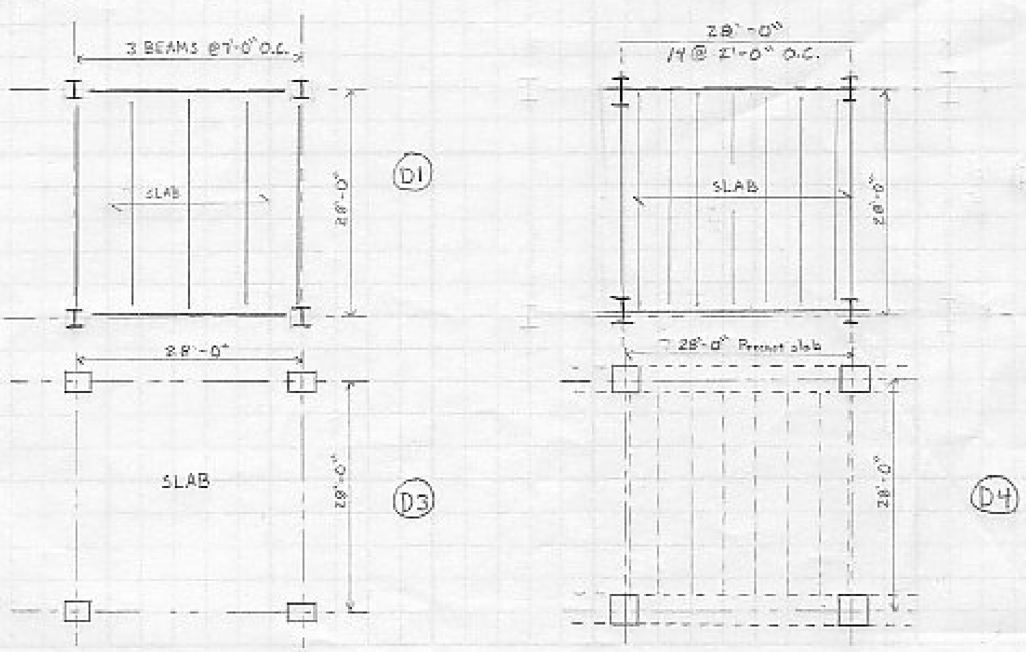
Slab on steel joists ⟵

Two-way slab on beams

Two-way slab on columns ⟵

Precast slab with beam ⟵

Plan



Technical Report #2	D1	Brendon J. Burley	1/3

Slab

$l_n = 7'\text{-}0''$ slab does not carry ceiling

$w_L = 100$ psf

$w_D = 20$ psf

minimum depth to neglect deflection:

interior : $h \geq \frac{l_n}{28}$

exterior : $h \geq \frac{l_n}{24}$ ← controls $h \geq \frac{7(12)}{24} = 3.5''$

Choose $h = 4''$, $f'_c = 4000$ psi, #3 bars

$d = h - .75 - \frac{d_b}{2} = 3.0625''$

$w_{self} = \frac{4}{12}(150) = 50$ psf $1.4D = 98$ psf

$w_D = 20 + 50 = 70$ psf $1.2D + 1.6L = 244$ psf ← controls

Analyze a 1' wide strip

$w_L = 100$ plf

$w_d = 70$ plf

$w_u = 244$ plf

Use ACI Moment Coefficients

exterior spans are critical: $M_{u-} = \frac{1}{10} w_u l_n^2 = 1.20$ 'k/ft.

$M_{u+} = \frac{1}{14} w_u l_n^2 = 0.85$ 'k/ft.

$V_u = 1.15\left(\frac{w_u l_n}{2}\right) = 0.98$ k/ft.

$M_u \leq \phi M_n = \phi .85 f'_c b a (d - \frac{a}{2})$ Assume $\phi = .9$

$1.20(12) = .9(.85)(4)(12) a (3.0625 - \frac{a}{2})$

$18.36 a^2 - 112.455 a + 14.4 = 0 \Rightarrow a = 0.13''$

$A_s = .85 \frac{f'_c}{f_y} b a = .85 (\frac{4}{60})(12)(0.13) = 0.0884$ in²/ft ← controls

$A_{s\,min} = 0.0018bh = 0.0018(12)(4) = 0.0864$ in²/ft

$s = 3h = 3(4) = \underline{12''} < 18'' \Rightarrow$ #3's @ 12'' $A_s = 0.11$ in²/ft.

#3's @ 12'' represents the smallest area of steel provided by #3's in this slab, so it will suffice for the positive moment as well.

$a = \frac{A_s f_y}{.85 f'_c b} = \frac{0.11(60)}{.85(4)(12)} = 0.16''$

$\varepsilon_t = \frac{d - a/\beta_1}{a/\beta_1}(0.003) = \frac{3.0625 - 0.19}{0.19}(0.003) = 0.045 > 0.005 \therefore \phi = 0.9$

$\phi M_n = \phi A_s f_y (d - \frac{a}{2}) = .9(0.11)(60)(3.0625 - \frac{0.16}{2}) = 17.72$ "k/ft. = 1.47 'k/ft.

$\phi V_n = .85(2\sqrt{f'_c}\, bd) = .85(2\sqrt{4000}\,(12)(3.0625)) = 3951.27$ lb/ft. = 3.95 k/ft.

1.47 'k/ft. > 1.20 'k/ft. o.k.

3.95 k/ft. > 0.98 k/ft. o.k.

Technical Report #2	D1	Brendon J. Burley	2/3

Beam

$L_n = 28'-0"$ Construction Load

$w_L = 100$ psf $w_L = 20$ psf

$w_D = 75$ psf $w_D = 50$ psf

$\Delta_{max} = \frac{l_n}{360}$

$\Delta = \frac{5 w_u L_n^4}{384 EI}$ => $I_{min} = 4.6875 \frac{w_u l_n^3}{E}$

Assume $w_{self} = 80$ plf

$1.4D = 1.4(75(7)+80) = 847$ plf

$1.2D+1.6L = 1.2(75(7)+80)+1.6(100)(7) = 1846$ plf ← controls

Construction

$1.4D = 1.4(50(7)+80) = 602$ plf

$1.2D+1.6L = 1.2(50(7)+80)+1.6(20)(7) = 740$ plf ← controls

$I_{min} = 4.6875 \frac{(1.846)(28)^3(1.44)}{29000} = 943 \text{ in}^4$

$I_{beam} = 4.6875 \frac{(0.740)(28)^3(144)}{29000} = 378 \text{ in}^4$

Assume $a = 1"$ => $Y_2 = h - \frac{a}{2} = 4 - \frac{1}{2} = 3.5"$

Table 5-15 : W14×38 $I_{beam} = 385 \text{ in}^4$, $Y_1 = 0.129"$, $\Sigma Q_n = 473$ k

$I = 956 \text{ in}^4$

$\Sigma Q_n = .85 f'_c b a$

$b = TA = 7'(12) = 84"$

$b = \frac{l_n}{4} = \frac{28(12)}{4} = 84"$ either controls

$a = \frac{\Sigma Q_n}{.85 f'_c b} = \frac{473}{.85(4)(84)} = 1.66"$ no good => $Y_2 = 3"$

Table 5-15 : W14×38 $I_{beam} = 385 \text{ in}^4$; $Y_1 = 0"$, $\Sigma Q_n = 560$ k

$I = 951 \text{ in}^4$

$a = \frac{560}{.85(4)(84)} = 1.96"$ o.k.

Table 1-1 : $d = 14.1"$ $t_w = 0.31$

Table 5-14 : $\phi M_p = 231$'k $\phi M_n = 399$'k

$M_u = \frac{w_u l_n^2}{8} = \frac{1.846(28)^2}{8} = 180.91$'k

$V_u = \frac{w_u l_n}{2} = \frac{1.846(28)}{2} = 25.84$ k

$M_{u\,beam} = \frac{w_u l_n^2}{8} = \frac{0.740(28)^2}{8} = 72.52$'k

399'k > 180.91'k o.k.

231'k > 72.52'k o.k.

Use 3/4" Φ studs

Table 5-13 : $Q = 26.1$ k

$\# = \frac{2\Sigma Q_n}{Q} = \frac{2(560)}{26.1} = 42.9$ Use 44 studs

$\phi V_n = \phi(0.6) f_y t_w d = .9(.6)(50)(0.310)(14.1) = 118.02$ k

118.02k > 25.84 k o.k.

Technical Report #2	D 1	Brendon J. Burley	3/3

Girder

$l_n = 28'$

$w_L = 100$ psf

$w_D = 75$ psf   Assume beams distribute as 114 plf on girder

Assume $w_{self} = 86$ plf

$w_L = 100(28) = 2800$ plf

$w_d = 75(28) + 114 + 86 \cong 2300$ plf

$1.4D = 3220$ plf

$1.2D + 1.6L = 7240$ plf ← controls

$$M_u = \frac{w_u l_n^2}{8} = \frac{7.240(28)^2}{8} = 709.52'k$$

$l_b = 7'-0"$

Table 5-5 : W30 x124

$w_d = 75(28) + 114 + 124 = 2338$ ∴ $w_u = 7286$ plf ∴ $M_u = 713.99'k$

Table 1-1 : $d = 30.2"$ $t_w = 0.585"$ $I = 5360\ in^4$

$$\Delta_{max} = \frac{28(12)}{360} = 0.93"$$

$$\Delta = \frac{5(7.286)(28)^4(1728)}{384(29{,}000)(5360)} = 0.65"$$

$$V_u = \frac{w_u l_n}{2} = \frac{7.286(28)}{2} = 102.00\ k$$

$$\Phi V_n = .9(.6)(50)(0.585)(30.2) = 477.01\ k$$

$0.65" < 0.93"$ o.k.

$477.01k > 102.00k$ o.k.

Technical Report #2 | D2 | Brendon J. Burley | 1/1

Slab

$L_n = 2'-0"$ ∴ Slab does not carry ceiling

$w_L = 100$ psf

$w_D = 20$ psf ∴ Superimposed Load = 120 psf

Try a 2" slab w/ 6×6-W2.1×W2.1 W.W.F. on 0.6C28 metal deck

allow = 285 psf , w_{self} = 23 psf

285 psf > 125 psf o.k.

Joist

$L_n = 28'-0"$

$w_L = 100$ psf

$w_D = 25$ psf

$w_{slab} = 23$ psf Superimposed Load = 148 psf

Try a 16K2

$Total_{allow}$ = 186 psf , $Live_{allow}$ = 106 psf , w_{self} = 5.5 plf

186 psf > 148 psf o.k.

106 psf > 100 psf o.k.

Girder

$L_n = 28'-0"$

Point Load = $\frac{1}{2}$ (148 (2) + 5.5)(28) = 4221 lb.

2 Joists / panel point ∴ P = 8.44 k

Try a 24G 14N 8.5K

w_{self} = 72 plf

Technical Report #2 | D3 | Brendon J. Burley | 1/2

Two-way Slab

$l_1 = 28'$

$l_2 = 28'$

$w_L = 100$ psf

$w_D = 25$ psf

Minimum thickness for column supported slabs w/o edge beams

exterior : $\frac{l_n}{30}$ ← controls

interior : $\frac{l_n}{33}$

$h_{min} = \frac{28(12)}{30} = 11.2"$ choose $11.5"$ ∴ deflection ok.

$w_{slab} = \frac{11.5}{12}(150) = 144$ psf ∴ $w_D = 169$ psf

Using direct design method

$1.4D = 236.6$ psf

$1.2D + 1.6L = 362.8$ psf ← controls

$M_o = \frac{w_u l_2 l_n^2}{8} = \frac{362.8(28)^3}{8} = 995523.2'\text{lb} = 995.52'\text{k}$

$M_- = .65 M_o = 647.09'\text{k}$

$M_+ = .35 M_o = 348.43'\text{k}$

Assume #8 bars

$d = h - .75 - \frac{d_b}{2} = 11.5 - .75 - \frac{1}{2} = 10.25"$

$M_u \le \phi M_n = \phi .85 f'_c ba(d - \frac{a}{2})$

Column strip : $b = 2 \times \frac{L}{4} = 2 \times \frac{28(12)}{4} = 168"$

Middle strip : $b = \frac{L}{2} = \frac{28(12)}{2} = 168"$

for an interior bay column strip takes 75% M_- and 60% M_+

Column(-): Assume $\phi = .9$

$.75(647.09)(12) = .9(.85)(4)(168)a(10.25 - \frac{a}{2})$

$257.04a^2 - 5269.32a + 5823.81 = 0 \Rightarrow a = 1.17$

$A_s = .85\frac{f'_c}{f_y}ba = .85\frac{4}{60}(168)(1.17) = 11.14\ in^2$ ∴ need 15 #8's $A_s = 11.85\ in^2$

$a = \frac{A_s f_y}{.85 f'_c b} = \frac{11.85(60)}{.85(4)(168)} = 1.24"$

$\varepsilon_t = \frac{d - a/\beta_1}{a/\beta_1}(0.003) = \frac{10.25 - 1.46}{1.46}(0.003) = 0.018 > 0.005$ ∴ $\phi = .9$

$\phi M_n = \phi A_s f_y (d - \frac{a}{2}) = .9(11.85)(60)(10.25 - \frac{1.24}{2}) = 6162.24"\text{k} = 513.52'\text{k}$

$513.52'\text{k} > .75(647.09) = 485.32'\text{k}$ o.k.

Column(+): Assume $\phi = .9$ $M_{+col} = .6(348.43) = 209.06'\text{k}$

$257.04a^2 - 5269.32a + 2508.72 = 0 \Rightarrow a = 0.49"$

$A_s = .85(\frac{4}{60})(168)(0.49) = 4.66\ in^2$ ∴ need 6 #8's $A_s = 4.74\ in^2$

$a = \frac{4.74(60)}{.85(168)(4)} = 0.50"$ $\varepsilon_t = \frac{10.25 - .59}{.59}(0.003) = 0.049 > 0.005$ ∴ $\phi = .9$

$\phi M_n = .9(4.74)(60)(10.25 - \frac{.50}{2}) = 2559.6"\text{k} = 213.3'\text{k}$

$213.3'\text{k} > 209.06'\text{k}$ o.k.

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Middle (−): Assume $\phi = .9$ $M_{-mid} = .25(647.09) = 161.77'k$

$257.04a^2 - 5269.32a + 1941.24 = 0 \Rightarrow a = 0.38''$

$A_s = .85 \frac{4}{60}(168)(0.38) = 3.62\ in^2$ ∴ 5 #8's $A_s = 3.95\ in^2$

$a = \frac{3.95(60)}{.85(168)(4)} = 0.41''$ $\varepsilon_t = \frac{10.25 - .49}{.49}(0.003) = 0.060 > 0.005$ ∴ $\phi = .9$

$\phi M_n = .9(3.95)(60)(10.25 - \frac{.41}{2}) = 2142.60''k = 178.55'k$

$178.55'k > 161.77'k$ o.k.

Middle (+): Assume $\phi = .9$ $M_{+mid} = .40(348.43) = 139.37'k$

$257.04a^2 - 5269.32a + 1672.46 = 0 \Rightarrow a = 0.32''$

$A_s = .85 \frac{4}{60}(168)(0.32) = 3.05\ in^2$ ∴ 4 #8's $A_s = 3.16\ in^2$

$a = \frac{3.16(60)}{.85(168)(4)} = 0.33''$ $\varepsilon_t = \frac{10.25 - .39}{.39}(0.003) = 0.076 > 0.005$ ∴ $\phi = .9$

$\phi M_n = .9(3.16)(60)(10.25 - \frac{.33}{2}) = 1720.90''k = 143.41'k$

$143.41'k > 139.37'k$ o.k.

$A_{s\,min} = 0.0018bh = 0.0018(168)(11.5) = 3.48''$ ∴ need 5 #8's $A_s = 3.95\ in^2$

Shear

Assume 26" ϕ columns $\frac{b_o}{4} = 26 + d$

$b_o = 4(26 + 10.25) = 145''$

$V_c = 4\sqrt{f'_c}\, b_o d = 4\sqrt{4000}(145)(10.25) = 375995\ lb. = 376.00\ k$ ← controls

$V_c = (\frac{40d}{b_o} + 2)\sqrt{f'_c}\, b_o d = (\frac{40(10.25)}{145} + 2)\sqrt{4000}(145)(10.25) = 453787\ lb. = 453.79\ k$

$\phi V_c = .75(376.00) = 282.00\ k$

$V_u = w_u(L^2 - (\frac{b_o}{4})^2) = 362.8(28^2 - (\frac{145}{48})^2) = 281124\ lb. = 281.12\ k$

$282.00\ k > 281.12\ k$ o.k.

Slab

$l_n = 28'$

$w_L = 100\ psf$

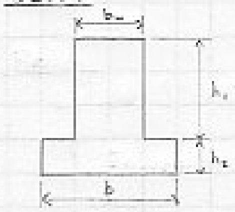
$w_D = 25\ psf$

Superimposed Load = 125 psf

	Allow	Initial Camber	Long-time Camber	W_{self}	width
Double T	173 psf	0.5"	0.7"	62 psf	8'-0"
Hollow Slab	141 psf	1.1"	1.3"	56 psf	4'-0"

Try hollow slab 4HC8 Strand 78-S

Beam



$l_n = 28'$

$w_L = 100(28) = 2800\ plf$

$w_d = 25(28) = 700\ plf$

$w_{slab} = 56(28) = 1568\ plf$

Superimposed Load = 5068 plf

$h_1 = 8"$ no such beam.

add 4" topping or use double T

Using Double T : 8DT24 Strand 68-S ∴ Superimposed Load = 4956 plf

$h_1 = 24"$ ∴ choose 28 IT 36 Strand #16

$W_{allow} = 7236\ plf$ camber = 0.4" long-time camber = 0.1"