TECHNICAL REPORT 2

1000 CONTINENTAL SQUARE

KING OF PRUSSIA, PENNSYLVANIA

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Structural Option

January 13, 2008

Advisor: Dr. Hanagan

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EXECUTIVE SUMMARY

This is the second of three preliminary stages of analysis intended to impart a better understanding in each student of their individual building, and acts as an attempt to better focus research for the final thesis in the spring. This second report consists of a more in-depth analysis of the building's existing floor system as well as a "pro vs. con" investigation of four alternative flooring systems. The floor systems I chose to analyze in addition to the existing composite slab were:

- Precast Hollowcore Planks on Steel Beams
- Precast Double Tees on Steel Beams
- Cast-in-Place One-Way Slab with Wide Shallow Beams
- Two-Way Post-Tensioned Slab with Drop Panels.

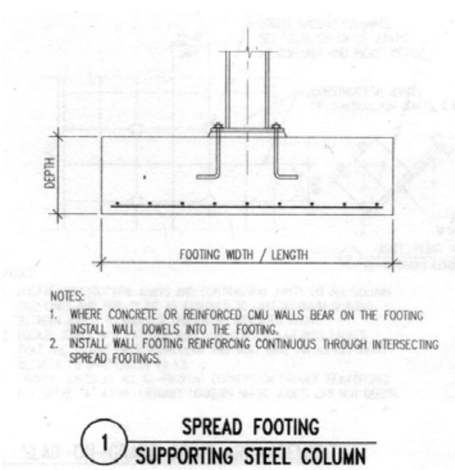
My choices in floor systems were rather limited due to the large size of the spans and heavy live load. Systems such as a standard two way flat plates and steel joists are simply unable to deal with such loading conditions. Some other options might have been available had I changed the column grid and created shorter bays, but this would have interfered with leasable space and made the value of the property drop, two scenarios which were definitely not acceptable for the original design team. By adhering to the design constraints which were placed on the original building, I arrived at several preliminary conclusions. The existing system is probably best suited to optimizing the current design, but redesigning the building in concrete with two-way PT slabs has potential. The one-way slab's thickness is appealing, but it is just too heavy and expensive to compete with the two-way. This is one design that might have fared better had I divided the bays into shorter spans. Perhaps, a combination of the two concrete designs could be used to reduce the need for deep drop panels in the PT slab. Both of the precast alternatives ended up being too thick to be practical; they left no room for mechanical systems, and less efficient beams had to be used to salvage the system depth, otherwise it would have been larger than four feet in both cases. The two-way PT slab is the only option that will be fully considered in future reports. The results from this stage of analysis are only intended to be used to rule out unacceptable alternatives. As a result, the design of the PT slab will have to be refined and drop panels, or some alternative method to reduce shear will have to be investigated in later calculations.

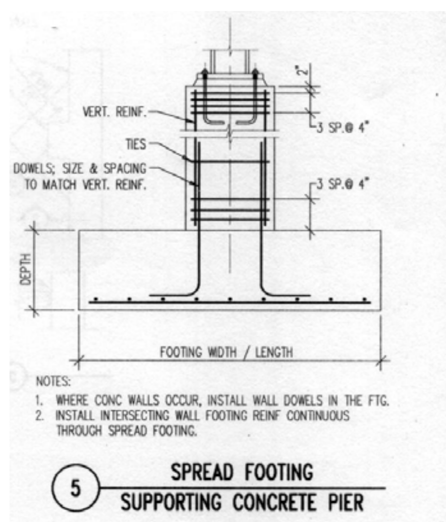
I. INTRODUCTION

1000 Continental Square is a new high end office building under construction in King of Prussia, PA. The site has a prominent location at the intersection of routes 202, 76, and 422, and is in close proximity to the PA Turnpike and King of Prussia Mall. A ground floor, partially below grade, serves mainly as space for mechanical systems and storage. Five floors of approximately 36,000 square feet of leasable space are located above that. The office space features large open floor plans with uninterrupted forty foot bays along each side of the building. The building makes use of a steel structural frame with composite metal decking and lightweight concrete slabs. Lateral loads are resisted by two moment frames along the long axis of the building and two eccentrically braced frames along the short axis.

FOUNDATIONS

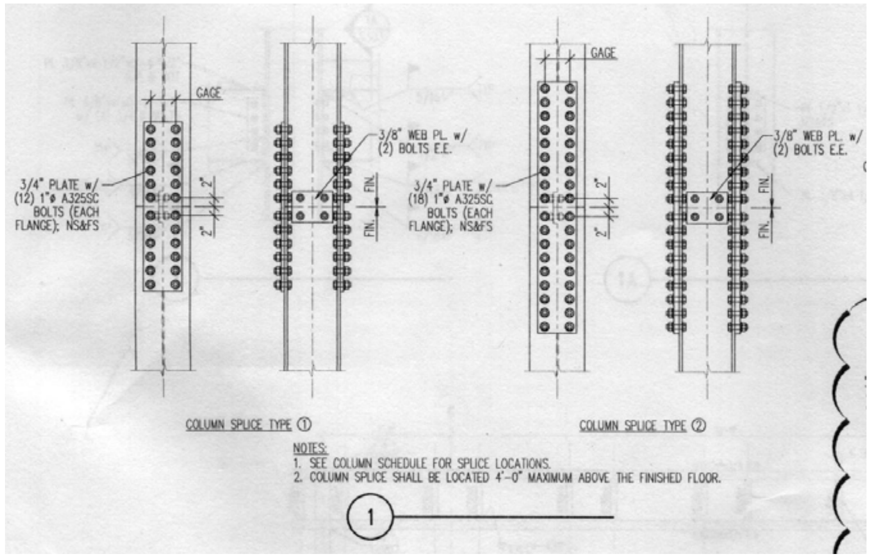
The foundations for 1000 Continental Square are a series of spread footings with continuous wall footings under the retaining walls located on the ground floor. The soils under the footings were found to withstand 4000 psf in most locations according to the geotechnical report furnished by Pennoni Associates Inc. on 24 of February 2004. Suitable bearing pressures were attained by deep dynamic compaction or partial soil exchange. Footing dimensions range from 4' x 4' x 1.5' to 20' x 20' x 4'; however, typical footings are approximately 14' x 14' x 3'. Special 55' x 18' x 3.5' spread footings are used under the braced frames. The tops of most footings are located 1.5' below grade, and minimum bearing depth is 3'. Columns either bear directly on footings or in some atypical situations concrete piers are placed on top of the footings and columns bear on those. Footings have bottom reinforcement ranging from (7) #4's to (16) #11's with typical reinforcement being approximately (12) #9's. The continuous wall footings are integrated into the spread footings they intersect, and their reinforcement is continuous throughout. Concrete in all footings has a minimum compressive strength, f'c = 3000 psi with a unit weight of 145 pcf. There is a 4" thick slab on grade which acts as the floor system for the ground floor and utilizes 4000 psi compressive strength concrete.





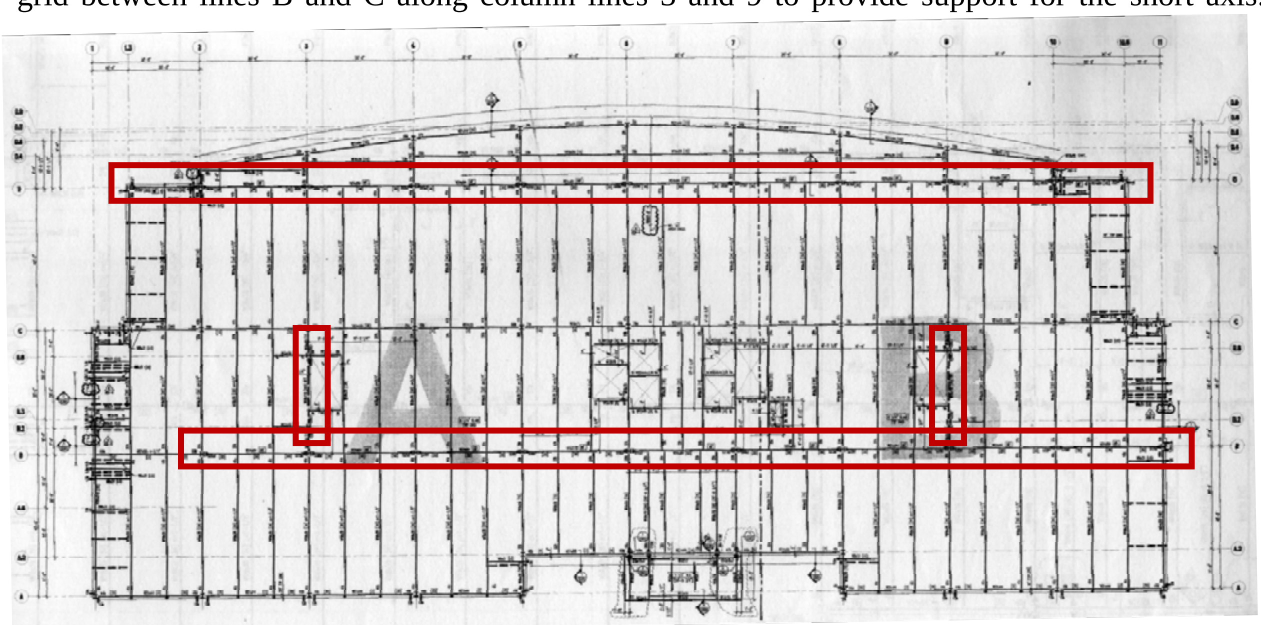
COLUMNS

The column grid for the building is laid out rectilinearly using three spans: 40’, 35’, 40’, in the N-S direction and (10) 30’ spans in the E-W, thereby creating large, uninterrupted, regular bays to simplify leasing. Column sizes vary between W 12 X 230’s on the first floor of the moment frames to W 12 X 40’s for gravity columns on the top floors. Splice levels are located a maximum of 4ft above the second and fourth floors. Typical columns are W 12 x 152’s on the bottom floors, W 12 x 96’s on the middle floors, and W 12 x 40’ on the top levels. Typical columns are fixed to foundations with four ¾” diameter anchor rods with 1’ embed depths and 4” hooks.



LATERAL LOAD RESISTING SYSTEMS

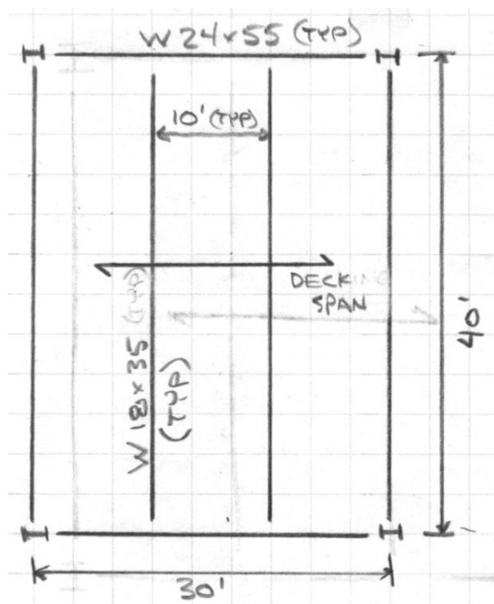
1000 Continental Square is reinforced against lateral loads by different systems along its long axis (E-W) and short axis (N-S). In the E-W direction two moment frames fit into the existing grid along column lines B and D, and act over the full height of the building and effectively its full length. In the N-S direction two full height eccentrically braced frames fit off grid between lines B and C along column lines 3 and 9 to provide support for the short axis.



II. EXISTING FLOOR FRAMING

All the floor framing above grade in the 1000 Continental Square project is 6¼" composite slabs. They consist of 3¼" lightweight concrete over 3" deep 20 gage galvanized composite floor deck. The slab is reinforced by one layer of 6 x 6 – W1.4 x W1.4 WWR, and has a weight of 115 pcf and a compressive strength of 3500 psi. This is supported by W 18 x 35's spanning 40' bays, which tie into an assortment of girders spanning 30'; W 24 x 55's being the most typical. Composite action is achieved through 6" long ¾" diameter headed studs, approximately 34, evenly spaced per beam. The W 18's feature a typical camber of 1.5". Variations in design occur at architectural features, the elevator shafts, and intersections with the moment frames, elsewhere the system is nearly identical on all floors. A typical bay is shown below.

SUMMARY



ADVANTAGES

Weight – This system has the lightest overall weight of the five which I explored. Less weight creates less seismic forces, as well as the obvious smaller gravity loads, both of which allow for smaller members and cheaper construction costs.

Thickness – With an overall depth of thirty inches, this is tied for the thinnest of the steel systems, allowing more room for MEP systems.

Constructability – Steel is light-weight in comparison to other material: easy to connect members, quick erection times, and no formwork.

DISADVANTAGES

Lead Time – Lead times for steel are longer than for materials such as concrete.

Fireproofing – Requires spray-on fireproofing everywhere, which can be expensive and time consuming.

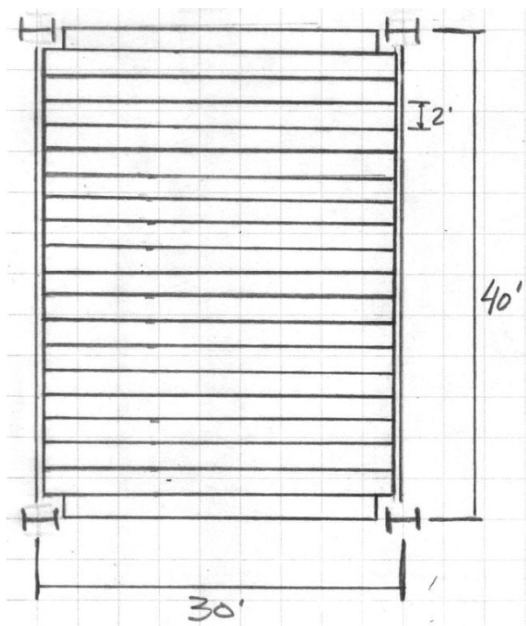
Shear Studs – Welding studs onto flanges adds time and labor to the installation of the decking.

Vibration – Although not a big issue on this job, due to its light weight, steel can have problems with dampening vibrations

III. ALTERNATIVE ONE - HOLLOWCORE ON STEEL

This design for hollowcore planks supported by steel beams fits very well into the existing column grid. The 10 " deep planks span the 30' foot direction because I could not find a manufacturer who had a plank that could span the 40' direction and support over a 100 pound superimposed load. The ends of the planks then rest on a W24 x 250 girder spanning the 40' bay. In order to minimize the total slab depth, the girder has angles welded to its webs which the planks slide onto, under the top flange. When the topping is placed it encases the top flange of the W shape creating a smooth finished floor similar to a girder slab.

SUMMARY



ADVANTAGES

Weight – This system has the second lightest weight overall. Less weight creates less seismic forces, as well as the obvious smaller gravity loads, both of which allow for smaller members and cheaper construction costs.

Constructability – The use of steel and precast allows for very simple construction because there is no need for formwork or placement of reinforcing since it has already been done.

DISADVANTAGES

Cost – The ease of construction is paid for in the extra price of materials, as well as labor to weld angles to all the girder webs, making this the most expensive option.

Fireproofing – Still requires spray-on fireproofing on exposed steel members; however, the planks do not need any additional protection thus reducing the cost and time when compared to metal deck.

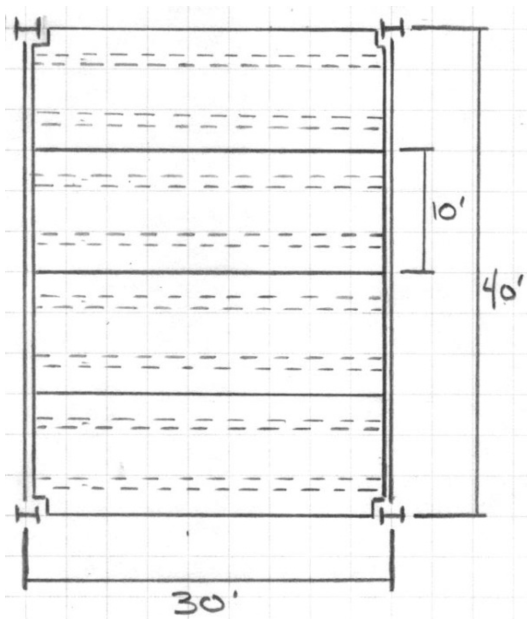
Depth – Overall depth is the second greatest only to double tees even with the creative way of mounting the planks on the beam.

Lead Time – Lead times for steel are longer than for materials like concrete.

IV. ALTERNATIVE TWO - DOUBLE TEES ON STEEL

Similar to the design of the hollowcore, this design spans the 30' direction with 20" deep 10' wide precast double tees. These rest on top of W18 x 234 girders which span N-S. Because it only takes 4 double tees to cover the 40' bay, this design is even easier to construct than the planks. An alternative design could be to span the 40' direction with the double tees, however it make almost no difference in beam size or weight. Use of precast beams and columns to replace the existing steel would be the best way to minimize the overall depth of this design. A two inch slab is placed on top of the precast to finish the surface and increases its depth so it will not need additional fireproofing.

SUMMARY



ADVANTAGES

Cost – 50¢ less cost per square foot than the composite slab shows how much the use of precast can bring down construction costs.

Constructability – The use of steel and precast allows for very simple construction because there is no need for formwork or placement of reinforcing since it has already been done.

DISADVANTAGES

Fireproofing – Still requires spray-on fireproofing on exposed steel members; however, the tees do not need any additional protection thus reducing the cost and time when compared to metal deck.

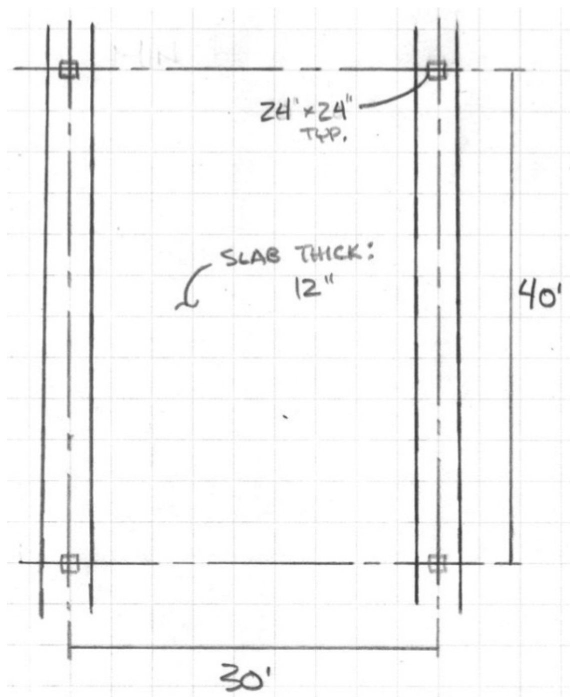
Depth – Overall depth is the biggest drawback for this system. At almost 3.5', this reduces the ceiling height (leaving no room for MEP) to 9.5'

Lead Time – Lead times for steel are longer than for materials like concrete.

V. ALTERNATIVE THREE - ONE WAY SLAB DESIGN

This design is a very elegant way of hiding what could potentially be very large beams but making them wide and shallow instead. The system used a one-way, traditionally reinforced 12" slab to span the shorter 30' direction. Then a giant 20" x 50" beam spans the longer 40' bay. This results in the thinnest overall depth of only 20". However the system pays the price for its beams in weight and cost. To utilize this system, columns would need to be redesigned in concrete. I assumed column dimensions of 24" x 24" but this would have to be evaluated and reinforcement specified in a later report in order to make this system feasible.

SUMMARY



ADVANTAGES

Thickness – The thinnest system at only 20" thus allowing 10 more inches of space for MEP systems.

Fireproofing – The overall girth of this system allows it to not need any extra fireproofing to achieve a two hour rating.

Vibration – The weight of this system makes it a natural choice for vibration resistance.

DISADVANTAGES

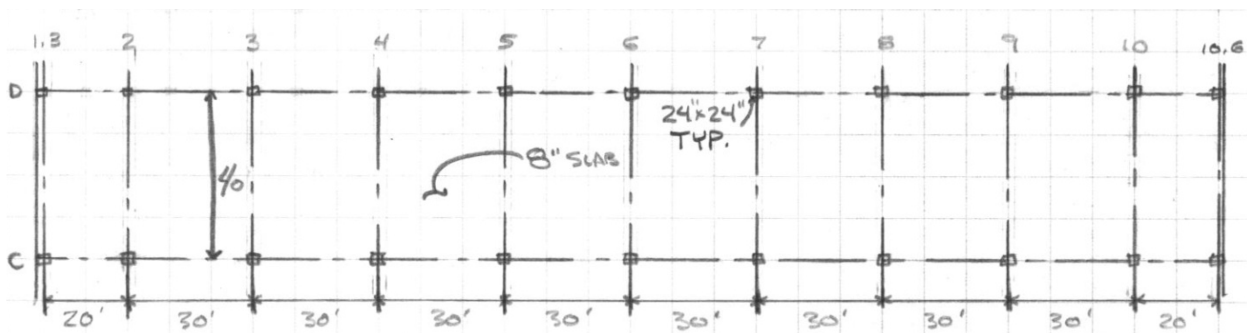
Weight – The heaviest system as a result of the amount of material in the final product. This results in increased size and reinforcement in columns and foundations.

Cost - The second most expensive system, once again, simply because of the amount of material required to build it, as well as the relative complexity of reinforcing and formwork.

Constructability – Due to the laying out of reinforcement and formwork, this gets a relatively high level of complexity

VI. ALTERNATIVE FOUR - TWO WAY PT SLAB

The most promising of the designs, a two-way PT slab is the most likely to replace the existing system. Using ultra high strength tendons, tension in concrete can be all but eliminated. This design is unfinished since shear checks showed drop panels need to be added. However, even with the complexity of construction this is still the cheapest system. This system will also require a redesign of the column system into concrete, and the assumed column dimensions of 24” x 24” will be checked, reinforcement specified, and drop panels with possible column capitals will be laid out in a future report.



SUMMARY

ADVANTAGES

Cost – PT slabs counter the added cost of complex construction by using much less material than a traditionally reinforced slab.

Thickness – With an overall depth of 22 inches, this is the second thinnest system allowing more room for MEP systems.

Fireproofing – Although thin, the slab thickness is still deep enough to not need additional fireproofing to achieve a two hour rating.

DISADVANTAGES

Constructability – By far the most complex system to design and build, precision placement of fibers as well as the density of reinforcement in column strips helps to make this the most difficult to construct. Complex formwork for drop panels adds a little more difficulty to the construction.

Weight – The second heaviest system as a result of the weight of the concrete in the final product. This results in increased dimensions and reinforcement in columns and foundations.

VII. COMPARISONS & CONCLUSIONS

	Existing	Alternate 1	Alternate 2	Alternate 3	Alternate 4
	Composite Decking	Hollowcore on Steel	Double Tees on Steel	One-Way Slab	Two-Way PT Slab
Depth	30"	39"	41"	20"	22"
System Weight	180 lbs/sq.ft.	208 lbs/sq.ft.	215 lbs/sq.ft.	264 lbs/sq.ft.	250 lbs/sq.ft.
Cost	$14.70	$15.30	$14.20	$14.90	$13.80
Fireproofing	Spray-On Req.	Spray-On Req.	Spray-On Req.	NA	NA
Vibration	-	Better	Better	Best	Better
Pros	Easy Constructability, Lightest Weight	Easy Constructability, Second Lightest Alternative	Cheap Cost, Easy Constructability	Best for Vibration, No Additional Fireproofing, Very Thin	Cheapest Alternative, Very Thin, No Additional Fireproofing
Cons	Requires Spray-On Fire Proofing, Long Lead Times, Possible Vibration Problems	Very Expensive, Requires Spray-On Fire Proofing, Very Deep Beams, Additional Labor to Weld Angles	Deepest Floor System, Requires Spray-On Fire Proofing, Aesthetically Unappealing	Very Heavy, Relatively Expensive, Moderately Difficult to Construct, Requires Formwork	Difficult to Construct, Very Heavy, Requires Extensive Formwork
Feasibility	Yes	No	No	Yes	Yes

CONCLUSIONS

From the analysis of the five floor systems in this paper, several conclusions can be drawn. It appears that the original designers made the correct choice, and a composite slab has the most to offer as far as value, adaptability, and constructability. Other forms of deck on a steel frame simply cannot compare with a composite lightweight concrete slab. Both are too heavy, and result in obscenely large steel members to support them and superfluous total assembly depths. However, if you are willing to overlook the ease and speed of construction, not easy to do with rented spaces, it is possible to use a concrete structural system with either a one-way or two-way slab. This would result in a thinner, comparably priced, or cheaper system, with better resistance to vibration and no need for supplemental fire proofing. Of the two concrete systems, the two-way PT slab has more potential as it is a whole dollar per square foot cheaper. However, the PT slab still has problems with shear failure which need to be solved, and the one-way slab does provide the smallest overall depth. Both appear to provide suitable alternatives which should be researched further.

VIII. APPENDICES

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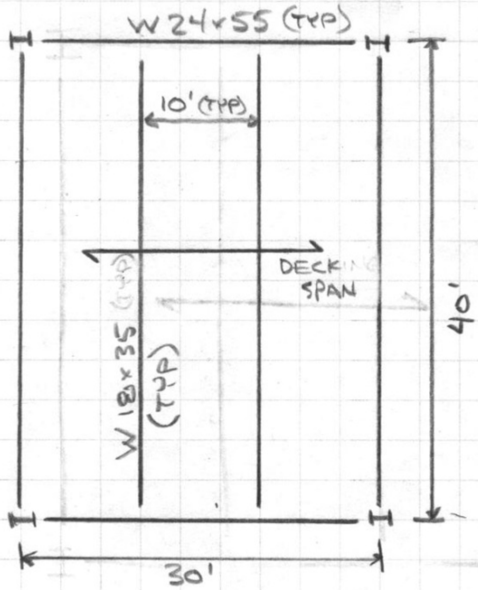
A.1 EXISTING SYSTEM – COMPOSITE METAL DECKING

EXISTING SYSTEM : COMPOSITE METAL DECKING



LL: 100 PSF
DL: 30 PSF
SI

$W_u = 1.2(30) + 1.6(100)$
$= 196$ PSF

$f'_c = 3500$ PSI
$f_y = 50{,}000$ PSI STR. STEEL
$f_y = 60{,}000$ PSI REINF.

3.0" METAL DECK, 20 GAGE
3.5" NORMAL WEIGHT TOPPING
3/4" φ, 5" HEADED SHEAR STUDS

BEAMS: W 18x35 ⇒ A_s =
GIRDERS: W 24x55 ⇒ A_s =

DECKING
$W_u = 196$ PSF UNITED STEEL DECK CATALOGUE
OK → UNSHORED SPAN FOR 3 SPAN = 10.27' > 10'
OK → MAX LOADING FOR 10' SPAN = 285 PSF > 196 PSF

BEAMS
$W_u = 196 \text{ PSF} (10')/1000 = 1.96 \text{ klf}$ → UNFACTORED = 1.30 klf

$M_u = (1.96 \text{ klf})(40')^2/8 = 392'^k$

$V_u = (1.96 \text{ klf})(40')/2 = 39.2'^k$

$\Sigma Q_n = F_y A_s = 50(10.3 \text{ in}^2) = 515$ kip

$b_{eff} = \frac{1}{2}(40') = 20'$
$= 10'$ → CONTROLS

$$a_{REQ} = \frac{\Sigma Q}{.85(f'_c)b} = \frac{515}{.85(3.5)(10\times12)} = 1.44''$$

$$Y_2 = 6.5 - 1.44/2 = 5.78''$$

TABLE 3-19

$\Sigma Q_n = 515k \geq 515$ OK

$\phi M_N = 532k \geq M_U = 392k$ OK

$\phi V_U = 143 \geq V_U = 39.2k$ OK

TABLE 3-

$I_{LB} = 1570 in^4$

$$\Delta_{MAX} = \frac{5(1.30)(40')^4(1728 in^3/ft)}{(384)(29000)(1570)} = 1.64''$$

$$\Delta_{D+L} \leq \ell/240 = (40\times12)/240 = 2.00$$

$$\Delta_{MAX} = 1.46'' < 2.00'' = \Delta_{D+L} \quad OK$$

GIRDERS

$$P_U = 1.96\left(\frac{40+35}{2}\right) = 73.5k \Rightarrow \text{UNFACTORED} = 48.75k$$

$$M_U = .33(73.5)(30') = 727.65 \text{ ft-k}$$

$$V_U = 2(73.5)/2 = 73.5k$$

$$\Sigma Q_N = F_y A_s = 50(16.3) = 815$$

$$b_{eff} = 1/2(30') = 15' \rightarrow \text{CONTROLS}$$
$$= 40'$$

$$a_{REQ} = \frac{815}{.85(f'_c)b} = \frac{815}{.85(3.5)(15'\times12)} = 1.52$$

$$Y_2 = 6.5 - \frac{1.52}{2} = 5.74''$$

TABLE 3-19

$\Sigma Q_N = 815\,k \geq 815\,k$

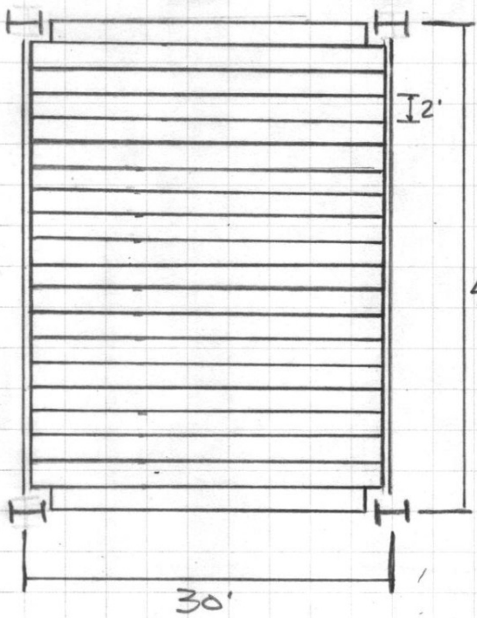
$\phi M_N = 999\,ft \cdot k \geq M_u = 727.65\,ft\text{-}k$

$\phi V_N = 252\,k \geq V_u = 73.5\,k$

OK ✓

A.2 ALTERNATIVE SYSTEM 1 –HOLLOWCORE PLANKS ON STEEL BEAMS

HOLLOWCORE PLANKS ON STEEL



- 10" THICK STANDARD HOLLOWCORE PLANKS
- 30' SPANS
- 2" STRUCTURAL TOPPING

LOADING

LIVE LOAD: 100 PSF
SELF WEIGHT: 101 PSF
DEAD LOAD: 30 PSF

$W_U = 1.6 \times 100 \text{ PSF} + 1.2 \times 131 \text{ PSF}$
$= 317.2 \text{ PSF}$

SIZE BEAMS

$317.2 \times 30' = 9516 \text{ PLF} = W_U$

$$M_U = \frac{w_u \ell^2}{8} = \frac{9.5 \text{ KLF} \times (40')^2}{8} = 1903.2 \text{ KIP-FT}$$

ACCEPTABLE BEAM SIZES:

W 44 × 230
W 36 × 231
W 33 × 241
W 30 × 235
W 24 × 250 ←

A.3 ALTERNATE SYSTEM 1 – DESIGN TABLES

SPANCRETE

4.4 Standard load tables
10" thick
.75" strand cover

4.0
Standard Spancrete

Issued 10/94

No Structural Topping
Dead Load Weight of Slab = 76 psf

FIRE RATINGS (Hours)

Code	Restrained	Unrestrained
Rational Design	—	—
SBC/UBC	3	—
UL	2	3/4
DILHR 51.045 Table 2	See 2.2	See 2.2

SECTION PROPERTIES

A = 212 in.² Y_t = 4.84 in. b = 17.5 in.
I = 3484 in.⁴ Y_b = 5.16 in. Wt = 76 psf

ϕM_n ft-k/ft	14.28	19.40	25.76	35.57	45.59	46.56	53.02
Series	.75D-10506	.75D-10606	.75D-10706	.75D-10806	.75D-10808	.75D-10712	.75D-10810
Span in Feet	ALLOWABLE SUPERIMPOSED LOAD IN POUNDS PER SQUARE FOOT						
17	170	253	357	484			
18	145	219	312	425			
19	124	190	272	375			
20	105	166	240	332			
21	90	144	212	295	408		
22	78	128	188	264	361		
23	64	110	167	236	325		
24	54	96	148	212	294	334	
25	45	84	131	190	266	303	337
26		72	117	171	241	275	306
27		63	104	154	219	251	280
28		54	92	139	199	229	256
29		46	82	125	181	209	234
30			72	113	165	191	215
31			64	102	151	175	197
32			56	92	138	161	181
33			49	83	126	147	167
34				74	115	135	153
35				66	105	124	141
36				59	96	114	130
37				53	87	104	120
38					80	96	110
39					72	88	102
40					66	80	93
41					60	73	86
42					54	67	79
43					48	61	72
44						56	66
45						50	61
46							55
47							50

2 Inch Bonded Structural Topping
Dead Load Weight of Slab With Topping = 101 psf

FIRE RATINGS (Hours)

Code	Restrained	Unrestrained
Rational Design	—	—
SBC/UBC	3	—
UL	2	1/4
DILHR 51.045 Table 2	See 2.2	See 2.2

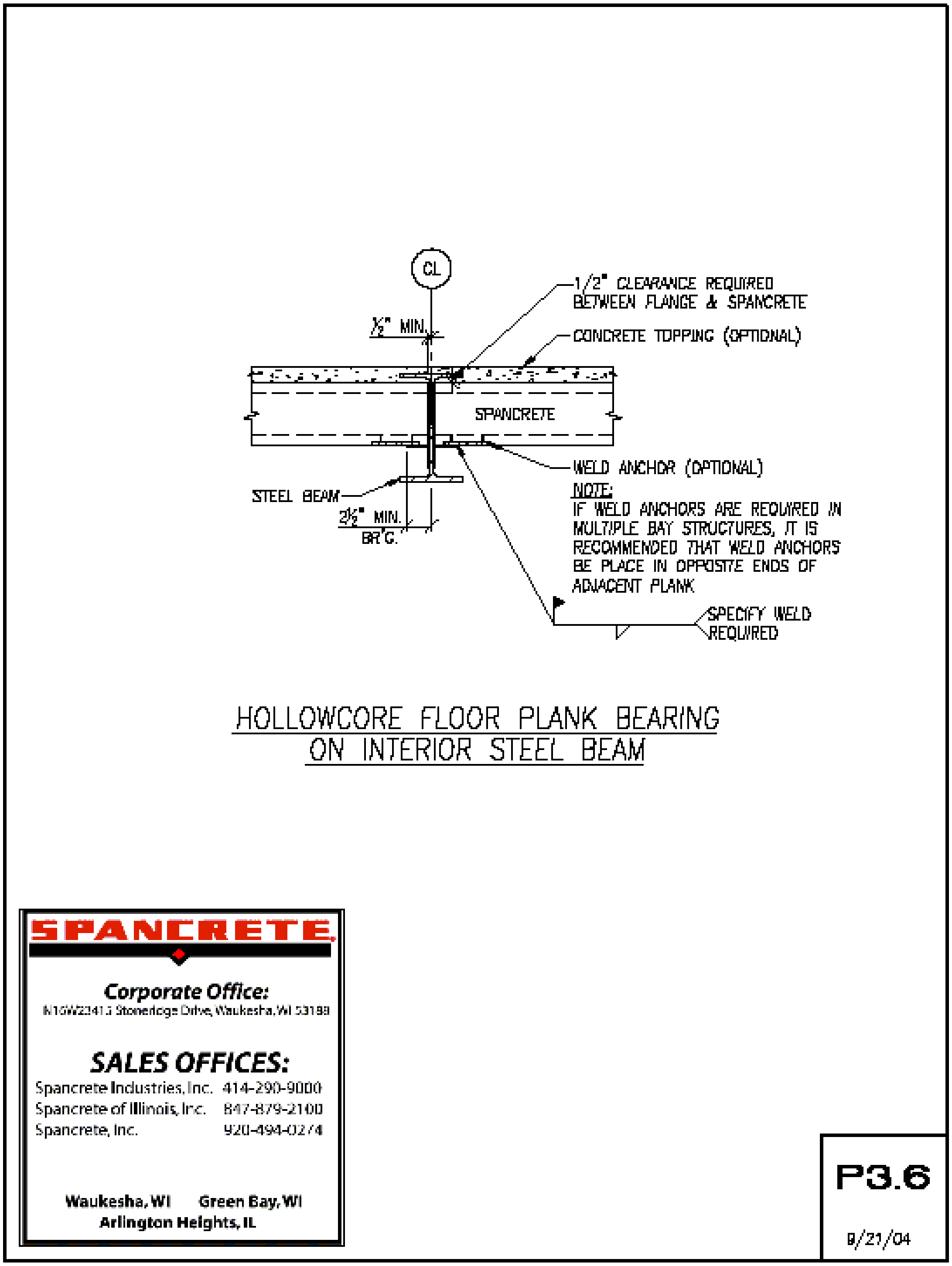
SECTION PROPERTIES

A = 367 in.² Y_t = 5.59 in. b = 17.6 in.
I = 5787 in.⁴ Y_b = 6.41 in. Wt = 101 psf

ϕM_n ft-k/ft	17.42	23.66	31.45	41.01	53.25	59.30	65.09
Series	.75D-10506T	.75D-10606T	.75D-10706T	.75D-10806T	.75D-10808T	.75D-10712T	.75D-10810T
Span in Feet	ALLOWABLE SUPERIMPOSED LOAD IN POUNDS PER SQUARE FOOT						
18	170	261	374	503			
19	144	225	327	451			
20	122	195	287	399			
21	103	169	252	354	420		
22	86	147	223	316	397		
23		127	197	282	376		
24		110	174	252	352	358	
25		95	154	226	318	340	
26			136	202	286	324	
27			120	182	261	300	308
28			106	163	237	273	294
29			93	148	215	249	281
30				131	195	227	257
31				118	178	207	233
32				105	162	189	212
33				94	147	173	192
34					[illegible]	[illegible]	[illegible]
35					122	141	158
36					108	127	144
37					96	114	130
38					85	103	117
39						92	106
40						81	95
41						72	85
42							76
43							67

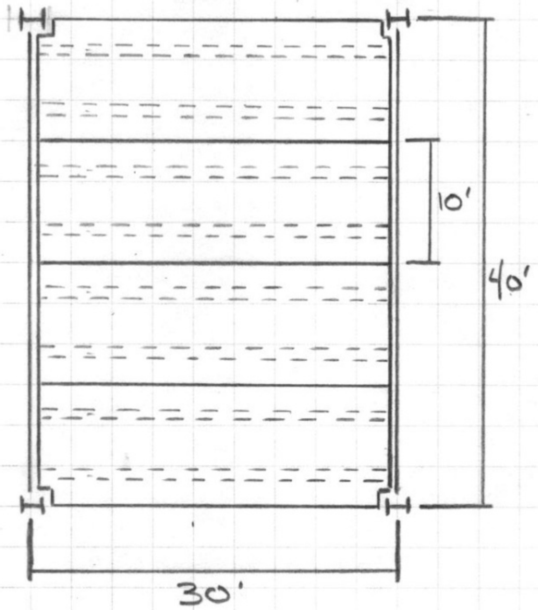
1" - 1 1/2" Camber (light shading) 1 1/2" or More Camber (dark shading)

A.4 ALTERNATIVE SYSTEM 1 – HOLLOWCORE – STEEL INTERFACE DETAIL



A.5 ALTERNATIVE SYSTEM 2 –PRECAST DOUBLE TEES ON STEEL BEAMS

DOUBLE TEES ON STEEL



- 10' WIDE, 20" DEEP DOUBLE TEES
- 30' SPANS

LOADING

LIVE LOAD: 100 PSF
SELF WEIGHT: 50 PSF
DEAD LOAD: 30 PSF

$W_U = 1.6 \times 100 + 1.2 \times 80$ PSF
$= 256$ PSF

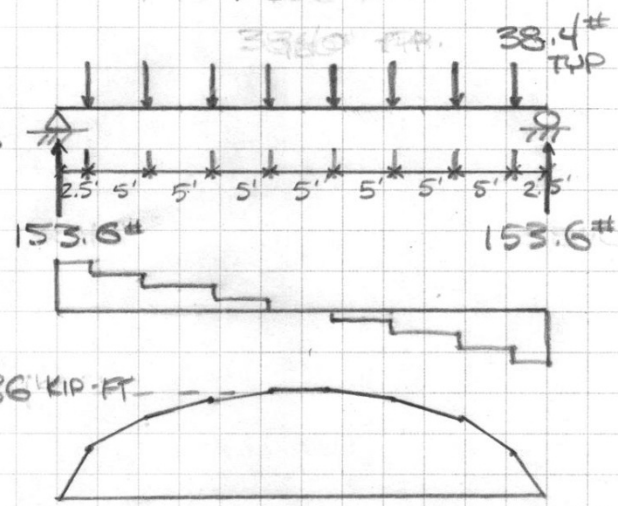
SIZE BEAMS

$256 \times 30' = 7680$ PLF $= W_U \times 40'/8 = 38400$ LBS.

$M_U = 1536$ KIP-FT

ACCEPTABLE BEAM SIZES

W 40×199
W 33×201
W 30×211
W 18×234 ←



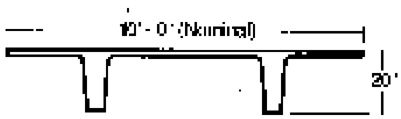
A.6 ALTERNATIVE SYSTEM 3 – DESIGN TABLE

1.0
Double
Tees

1.4 Double tee load tables
10'-0" x 20" Double tee

SPANCRETE

Issued 10/04



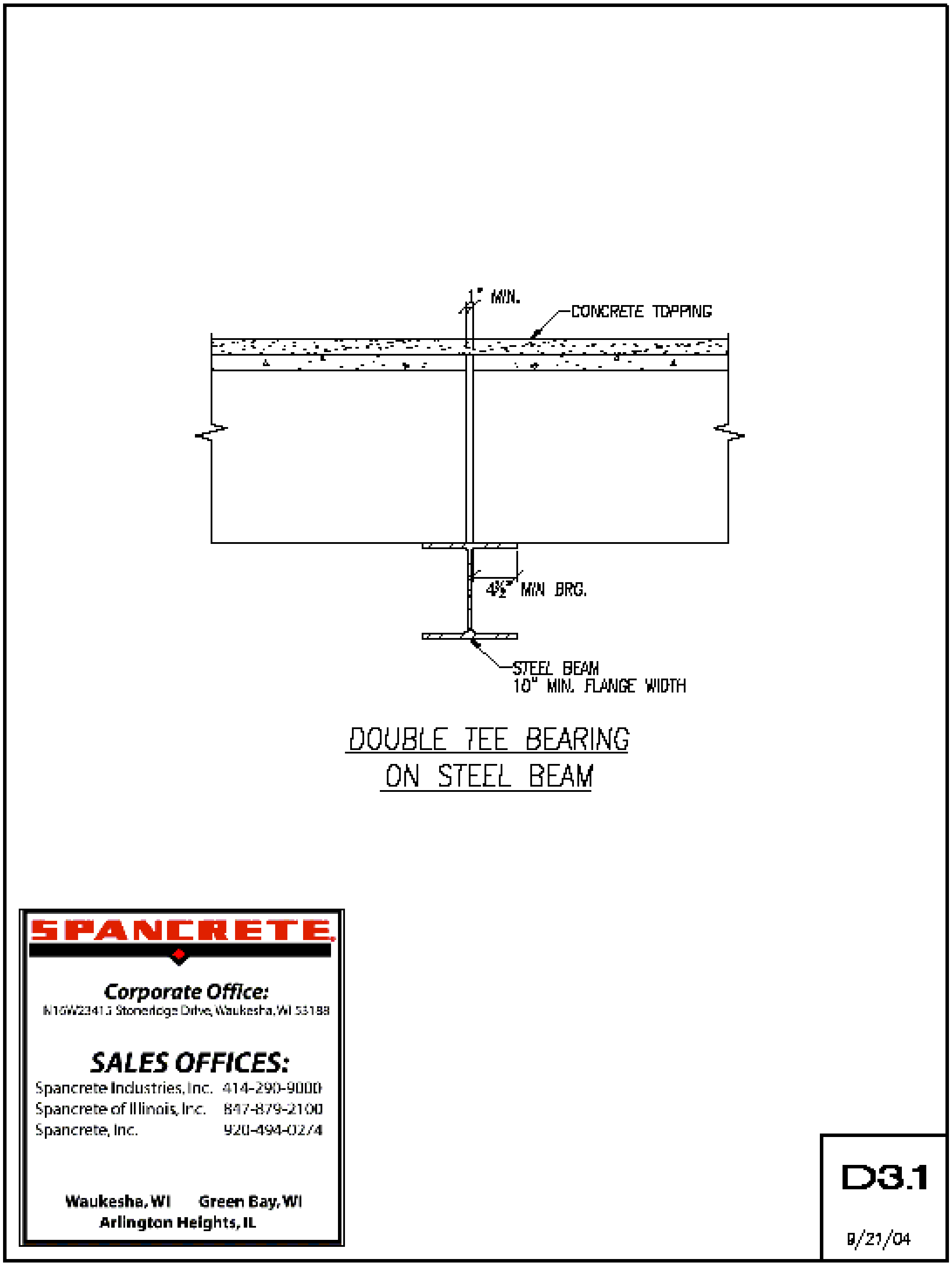
No Structural Topping
Dead Load Weight of Tee = 50 psf

SECTION PROPERTIES
A = 483 in.² I = 17,897 in.⁴ Y_t = 5.86 in. Y_b = 14.14 in.

φ M_n ft - k	207	290	359	413	454	480	592					
SERIES	10DT20-N48S	10DT20-N68S	10DT20-N88S	10DT20-N108S	10DT20-N128S	10DT20-N148S	10DT20-N128H					
Span In Feet	ALLOWABLE SUPERIMPOSED LOAD IN POUNDS PER SQUARE FOOT											
20	253											
22	200											
24	160	244										
26	129	201	260									
28	104	166	217	258	289							
30	84	138	183	218	245	262						
32	67	115	154	185	209	224						
34	54	96	131	158	179	193						
36	42	80	111	136	154	168	214					
38		66	94	116	133	144	187					
40		55	80	100	115	125	164					
42		45	68	85	98	106	144					
44			57	74	86	94	127					
46			48	63	74	82	112					
48			40	54	64	71	99					
50				45	55	61	87					
52					47	53	75					
54					40	45	67					
56							58					
58							51					
60							44					

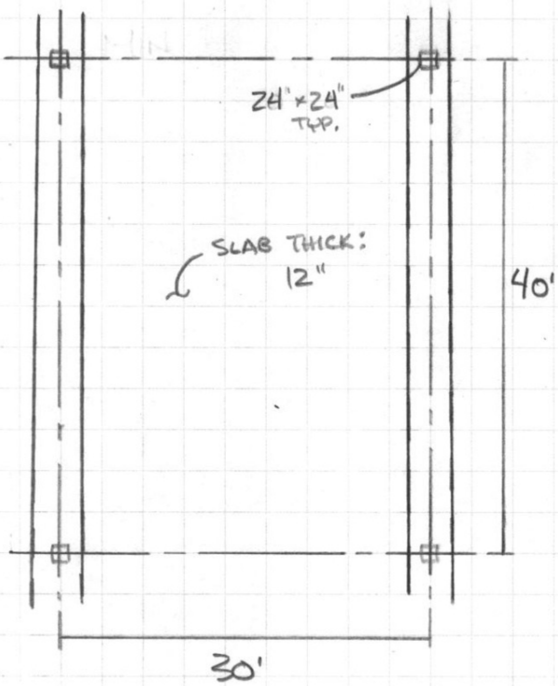
< 1 ½" Camber   1 ½" or More Camber

A.7 ALTERNATIVE SYSTEM 2 – DOUBLE TEE – STEEL INTERFACE DETAIL



A.8 ALTERNATIVE SYSTEM 3 – ONE-WAY SLAB WITH SHALLOW BEAMS

ONE WAY SLAB DESIGN



MATERIALS

- $f'_c = 4000$ PSI
- $f_y = 60000$ PSI

LOADING

LIVE LOAD: 100 PSF
DEAD LOAD: 30 PSF

$W_u = 1.6 \times 100 + 1.2 \times 30$ PSF
$= 196$ PSF

- DETERMINE MIN. SLAB THICK

ASSUMING 24" × 24" CONCRETE COLUMNS

$\ell_n = 30' - 24''/12''/' = 28'$

ACI TABLE 9.5 (a) $\Rightarrow h \geq \ell_n/28 = \frac{28' \times 12''/'}{28} = 12$ IN

TRY A 12" SLAB

- SLAB SELF WEIGHT

145 PCF × 1' = 145 PSF

$W_u = 196 + 1.2 \times 145 = 370$ PSF

- MAX MOMENTS FROM ACI COEFFICIENTS

$$M^- = \frac{w_u \ell_n^2}{10} = \frac{370 \times (28^{1/3})^2}{10} = 29.7 \text{ KIP-FT}$$

$$M^+ = \frac{w_u \ell_n^2}{16} = \frac{370 \times (28^{1/3})^2}{16} = 18.6 \text{ KIP-FT}$$

- DESIGN REINFORCEMENT

$$\rho_{MAX} = 0.85\,\beta_1 \frac{f'_c}{f_y} \frac{\varepsilon_u}{\varepsilon_u + .004} = .85 \times .85 \times \frac{4}{60}\left(\frac{.003}{.003+.004}\right)$$

$$= .0206$$

FIND EFF. DEPTH

$$d = \begin{cases} 12'' - 1.5'' = 10.5'' \longleftarrow \text{CONTROLS} \\ {}_{MAX}\sqrt{M_u/(\phi \rho f_y b (1-0.59(\rho f_y/f'_c)))} = \\ \sqrt{29.7 \times 12 / .9 \times .02 \times 60 \times 12 \left(1 - .59\left(\frac{.02 \times 60}{4}\right)\right)} = 5.7'' \end{cases}$$

FIND As AT SUPPORTS

ASSUME $a = 1$

$$A_s = M_u / \phi f_y (d - a/2) = 29.7 \times 12 / .9 \times 60 \times (10.5 - 1/2) = .66 \text{ IN}^2$$

CHECK $a = 1$

$$a = A_s f_y / (.85 \cdot f'_c \cdot b) = (.66 \times 60) / (.85 \times 4 \times 12) = .971''$$

RECALCULATE As FOR $a = .971''$

$$A_s = 29.7 \times 12 / .9 \times 60 \times (10.5 - .971/2) = .659 \text{ IN}^2$$

$A_s = .659 \text{ IN}^2/\text{ft} \implies$ #6 @ 6" ON TOP

FIND As AT MIDSPAN

$A_s = 18.6 \times 12 / .9 \times 60 \times (10.5 - .971/2) = .413 \text{ in}^2 \leftarrow$

$A_{s_{MIN}} = 0.0018\, bh = 0.0018 \times 12 \times 12 = .259 \text{ in}$

$A_s = .413 \text{ in}^2/\text{ft} \Rightarrow$ #6 @12" ON BOTTOM

- SHEAR VALUES FROM ACI COEFFICIENTS

$V_u = \frac{w_u l_n}{2} = \frac{370 \times 28\frac{1}{3}}{2} = 5.24$ kips

$\phi V_N = .75\left(2\sqrt{f'_c}\, bd\right) = .75 \times 2 \times \sqrt{4000} \times 12 \times 10.5$

$= 10.65$ kips > 5.24 kips OK ✓

- BEAM DESIGN

LIVE LOAD: 100 PSF REDUCABLE !!

$L = L_o\left(.25 + 15/\sqrt{K_{LL} A_T}\right) = 78.0$ PSF

DEAD LOAD: 25 PSF $+ 145 \times 12''/12''/1 = 170$ PSF

$W_u = 1.6 \times 78 + 1.2 \times 170 = 328.8$ PSF

$M_u^- = w_u l_n^2/12 = (328.8 \times 30') \times (40 - 2')^2/12 = 1187$ kip-ft

$M_u^+ = w_u l_n^2/24 = M_u^-/2 = 593.48$ kip-ft

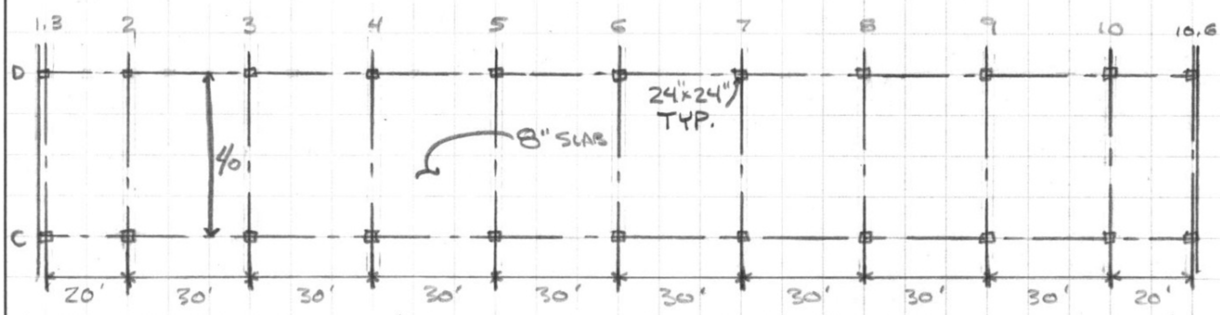
$\rho = .85(.85)(4/60)(.003/.007) = 0.0206$

$\phi M_N \geq M_u \Rightarrow .9(.0206)(60)\, bd^2 \left(1 - 0.59(.0206 \times 60)/4\right) \geq 1187$

$bd^2 \geq 15700 \text{ in}^3 \Rightarrow b = 3d \Rightarrow d = 18'' \Rightarrow b = 50''$

A.9 ALTERNATIVE SYSTEM 4 – TWO-WAY POST-TENSIONED SLAB

TWO-WAY POST-TENSIONED SLAB



COLUMNS: 24"×24"
FLOOR TO FLOOR: 13'

f'_c = 5000 PSI (SLAB)
ω = 145 PCF
f_y = 60,000 PSI
f_{pu} = 270,000 PSI

SLAB THICKNESS

$\frac{\text{SPAN}}{\text{DEPTH}} = \frac{L}{45} = \frac{30' \times 12}{45} = 8$

USE 8" SLAB

LIVE LOAD: 100 PSF
DEAD LOAD: 30 PSF

LOAD BALANCING

SELF WEIGHT = 8"/12 × (145 PCF) = 97 PSF

LIVE LOAD REDUCTION

END SPAN (20'×40') → $L = 100 \text{ PSF} \left(.25 + 15/\sqrt{(1)(800)}\right) =$
$= 78$ PSF

TYP SPAN (30'×40') → $L = 100 \text{ PSF} \left(.25 + 15/\sqrt{(1)(1200)}\right) =$
$= 68.3$ PSF

TOTAL FACTORED LOAD

$W = 1.2\,(127) + 1.6\,(68.3) = 261.7$ PSF

$W_{BAL} = 0.8 \times 97$ PSF $= 77.6$ PSF

$a = 8$ IN $- 1.25 - 1.25 = 5.5$ IN (MAX TENDON SAG)

$$F_e = \frac{W_{BAL} L^2}{8a} = (77.6)(40')^2 / 8 \times (5.5/12) = 33.9\ k/ft$$

→ ASSUME: 14 KSI LONG TERM LOSSES AND 1/2" ϕ, 7-WIRE STRAND.

$$0.153\left((0.7)(270) - 14\right) = 26.8\ k\ /\ \text{TENDON}$$

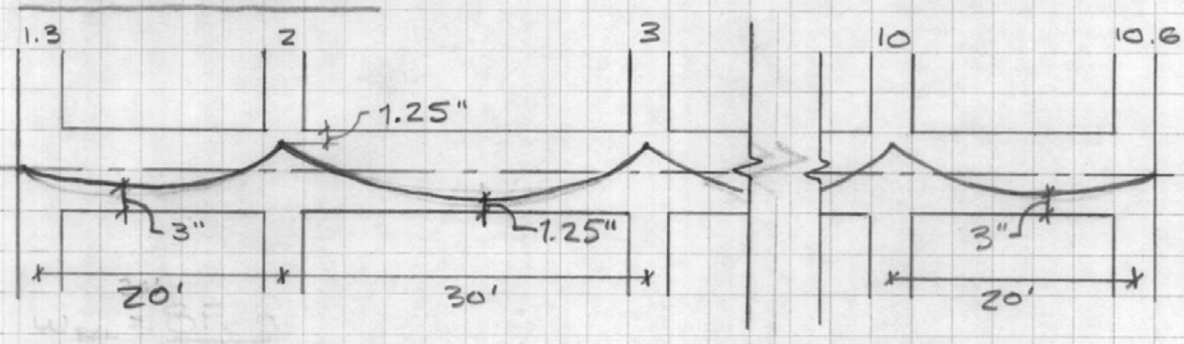
$$\#\ \text{OF TENDONS} = 40'(33.9\ k/ft) / 26.8\ k/\text{TEN} = 37.9\ \text{TEN.}$$

→ USE 38 TENDONS

$$F_e = 38(26.8)/40' = 25.5\ k/ft$$

$$F/A = 25.5 / 8 \times 12 = 0.265\ \text{KSI}$$

TENDON PROFILE



ADJUST PROFILE

$$W_{BAL} = \frac{8 F_e a}{L^2} = (8)(25.5)(5.5/12)/(30')^2 = 0.104\ \text{ksf}$$

$$a_1 = \frac{W_{BAL} L^2}{8 F_e} = 0.104(20')^2 / 8(25.5) = 0.204\ \text{FT} \times 12 = 2.44\ \text{in}$$

$$\text{MIDSPAN CGS} = (4 + 6.75)/2 - 2.44" = 2.93 \Rightarrow 3"$$

ACTUAL SAG = $(4 + 6.75)/2 - 3'' = 2.375''$

ACTUAL BALANCED LOAD = $\dfrac{8(25.5)(2.375/12)}{(20')^2} = .101\text{ ksf}$

NET LOAD CAUSING BENDING

END SPAN : $W_{NET} = .097 + .078 + .030 - .101 = 0.104$ KSF

TYP SPAN : $W_{NET} = .097 + .068 + .030 - .104 = 0.091$ KSF

EQUIVALENT FRAME METHOD

COLUMN STIFFNESS

$I = 1/12\,(24)(24)^3 = 27648\text{ in}^4$

$E = E_{col}/E_{slab} = 1.0$

$K_c = \dfrac{4EI}{L-2h} = 4(1.0)(27648)/13'(12) - 2(8\text{ in}) = 789.9\text{ in}^3$

TORSIONAL STIFFNESS

$C = \left[1 - .63\frac{x}{y}\right]\frac{x^3 y}{3} = \left[1 - .63\left(\frac{8}{24}\right)\right]\left(\frac{8^3\,24}{3}\right) = 3236\text{ in}^4$

$K_t = \dfrac{9CE}{L_2(1 - c_2/L_2)^3} = (9(3236)(1.0))/((24\times12)(1 - 1.33/40')^3) = 112\text{ in}^3$

$K_{ec} = \left[\dfrac{1}{\Sigma K_c} + \dfrac{1}{\Sigma K_t}\right]^{-1} = \left[\dfrac{1}{2(789.9)} + \dfrac{1}{2(112)}\right]^{-1} = 196.2\text{ in}^3$

SLAB STIFFNESS

$K_{s_E} = [4(1.0)(40')(8'')^3]/[12(20') - (24/2)] = 359.3\text{ in}^3$

$K_{s_T} = [4(1.0)(40')(8')^3]/[12(30') - (24/2)] = 235.4\text{ in}^3$

FIXED END MOMENTS

END SPAN = 0.104 $(20')^2/12 = 3.47$ ft·kips

TYP SPAN = 0.091 $(30')^2/12 = 6.83$ ft-kips

DISTRIBUTION FACTORS

END SPAN, EXT = $359.3/(359.3+196.2) = .65$

END SPAN, INT = $359.3/(359.3+196.2+235.4) = .45$

TYP SPAN, END = $235.4/(235.4+196.2+359.3) = .30$

TYP SPAN, TYP = $235.4/(2 \times 235.4+196.2) = .35$

	1.3		2		3		4
DF	.65	.45	.30	.35	.35	.35	.35
FEM	-3.47	3.47	-6.83	6.83	-6.83	6.83	-6.83
DIST	2.26	1.51	1.01	0	0	0	0
CO	0.76	1.13	0	0.51	0	0	0
DIST	-0.49	-0.51	-0.34	-0.18	-0.18	0	0
CO	-0.26	-0.25	-0.09	-0.17	0	-0.09	0
DIST	0.17	0.15	0.10	0.06	0.06	0.03	0.03
CO	0.08	0.09	0.03	0.05	0.02	0.03	0.02
DIST	-0.05	-0.05	-0.04	-0.02	-0.02	-0.02	-0.02
	-1.00	5.54	-6.16	7.08	-6.95	6.78	-6.80

NET TENSILE STRESSES

$-M_{MAX}$ @ FACE OF TYP. COLUMN

$$-M_{MAX} = -6.95 + \tfrac{1}{3}\left(\frac{0.091\,(30)}{2}\right)\left(\frac{24}{12}\right) = -6.04 \text{ ft-kips}$$

$$S = bh^2/6 = 12(8)^2/6 = 128 \text{ in}^2$$

$$f_{t,b} = -f_{pc} \pm \frac{M_{NET}}{S_{tb}} = -0.265 \pm \frac{-6.04\,(12)}{128} =$$

$$= -.831\,,\ .301 \text{ ksi}$$

ALLOWABLE TENSION

$6\sqrt{f'_c} = 6\sqrt{5000} = 0.424$ ksi $> .301$ ksi OK ✓

ALLOWABLE COMPRESSION

$0.6\,f'_c$ (AT TRANSFER, $f'_{c,i} = .75\,f'_c$) $= 0.6(.75)(5\text{ ksi}) = 2.25 > .831$ ksi

$0.45\,f'_c$ (AT SERVICE LOAD) $= 0.45(5\text{ ksi}) = 2.25 > .831$ ksi

OK ✓

$+M_{MAX}$ @ MIDSPAN OF TYP SPAN

$$+M_{MAX} = \left(\frac{0.091\,(30)^2}{8}\right) - 6.83 = 3.41 \text{ ft-kips}$$

$$f_{t,b} = -f_{pc} \pm \frac{M_{NET}}{S} = -0.265 \pm \frac{(3.41)(12)}{128}$$

$$= -.585\,,\ .055$$

ALLOWABLE TENSION

0.424 ksi > 0.055 ksi OK ✓

ALLOWABLE COMPRESSION

2.25 ksi > .585 ksi OK ✓

FLEXURAL CAPACITY

$FEM_{END} = (0.101)(20')^2/12 = 3.37$ ft-kips

$FEM_{TYP} = (0.104)(30')^2/12 = 7.80$ ft-kips

	1.3		2		3		4
DF	.65	.45	.30	.35	.35	.35	.35
FEM	-3.37	3.37	-7.80	7.80	-7.80	7.80	-7.80
DIST	2.19	1.99	1.33	0	0	0	0
CO	1.00	1.10	0	0.67	0	0	0
DIST	-0.65	-0.50	-0.33	-0.23	-0.23	0	0
CO	-0.25	-0.33	-0.12	-0.17	0	-0.12	0
DIST	0.16	0.20	0.14	0.06	0.06	0.04	0.04
CO	0.10	0.08	0.03	0.07	0.02	0.03	0.02
DIST	0.07	-0.05	-0.03	-0.03	-0.03	-0.02	-0.02
	-0.75	5.86	-6.78	8.17	-7.98	7.73	-7.76

SECONDARY MOMENTS

$M_{END,EXT} = 0.75 - 25.5(0/12) = 0.75$ ft-kips

$M_{END,INT} = 5.86 - 25.5(4-1.25)/12 = 0.02$ ft-kips

$M_{TYP,END} = 6.78 - 25.5(4-1.25)/12 = 0.94$ ft-kips

$M_{TYP,TYP} = 8.17 - 25.5(4-1.25)/12 = 2.33$ ft-kips

FACTORED LOAD MOMENTS

$FEM_{END} = 0.277(20')^2/12 = 9.24$ ft-kips

$FEM_{TYP} = 0.262(30')^2/12 = 19.63$ ft-kips

	1.3		2		3		4
DF	.65	.45	.30	.35	.35	.35	.35
FEM	-9.24	9.24	-19.63	19.63	-19.63	19.63	-19.63
DIST	6.00	4.68	3.12	0	0	0	0
CO	2.34	3.00	0	1.56	0	0	0
DIST	+1.52	-1.35	-0.90	-0.55	-0.55	0	0
CO	-0.68	-0.76	-0.28	-0.45	0	-0.28	0
DIST	0.44	0.47	0.31	0.16	0.16	0.10	0.10
CO	0.24	0.22	0.08	0.16	0.05	0.08	0.05
DIST	-0.16	-0.14	-0.09	-0.07	-0.07	-0.05	-0.05
FAC. MOM.	-2.58	-15.36	-17.39	-20.44	-20.04	-19.32	-19.53
2ND MOM.	-0.75	+ 0.02	+0.94	+ 2.33	+ 2.14	+ 1.88	+ 1.92
MOM @ ℄	-1.83	-15.34	-16.45	-18.11	-17.90	-17.44	-17.61

DESIGN MOMENT @ MIDSPAN

END SPAN $V_{END} = \frac{0.277 \times 20'}{2} - \frac{15.34 - 1.83}{20'} = 2.0$ kips/ft

$V_{INT} = 3.4$ kips/ft

GOVERNS

POINT OF ZERO SHEAR AND MAX MOMENT

$x = 2.0/0.277 = 7.22$ FT FROM ℄ OF EXT. COL.

POSITIVE MOMENT

$M_{MAX} = 0.5(2.0)(7.22) - 1.83 = 5.39$ ft·kips/ft

TYP SPAN

$V = \frac{0.262\ (30)}{2} - \frac{18.11 - 16.45}{(30')} = 3.8$ kips/ft

$x = 3.8/0.262 = 14.50$

$M_{MAX} = 0.5(3.8)(14.50) - 16.45 = 11.10$ ft-kips/ft

FLEXURAL STRENGTH

$A_s = 0.00075\,A_{cf} = 0.00075\,(40')(12)(8") = 2.88\ \text{in}^2$

→ TRY (5) #7

BAR LENGTH $= 2(30 - 24/12)/6 + 40/12 = 12'.8"$

$A_s = 5 \times \frac{.60}{40'} = 0.075\ \text{in}^2/\text{FT}$

CALCULATE DESIGN STRESS IN TENDONS

$f_{ps} = f_{pe} + 10000 + \frac{f'_c}{300\rho_p}$

$\rho_p = \frac{A_{ps}}{bd} = (0.153)(38\ \text{TEN.})/40 \times 12 \times (8 - 1.25) = 0.00179$

$f_{pe} = (0.7(270) - 14) = 175$ ksi

$f_{ps} = 175 + 10 + \frac{5}{3000(0.00179)} = 194.29$

$f_{ps} < 0.85\,f_{pu} = .85\,(270) = 230$ ok ✓

$f_{ps} < f_{pe} + 30 = 175 + 30 = 205$ ok ✓

$F_{su} = 194\,(0.153)\,40'/30' = 39.63$ kips/ft

$F_y = 60 \times (0.075) = 4.5$ kips/ft

$F_{TOT} = F_{su} + F_y = 39.63 + 4.5 = 44.13$ kips/ft

DEPTH OF COMPRESSION BLOCK

$a = F/0.85\, b f'_c = 44.13/0.85(12)(5) = .87 \text{ IN}$

$\varepsilon_t = (6.75 - .13)(0.003)/(0.52/0.85) = .0325$

→ ASSUME REBAR AND TENDONS IN SAME LAYER

$d - a/2 = (6.75 - 0.87/2)/12 = 0.53 \text{ ft}$

MOMENT CAPACITY @ COL ℄

$\phi M_n = 0.9(0.53)(44.13) = 21.05 \text{ ft-kips/ft} > 18.11 \text{ ft-kips/ft}$

PERMISSIBLE CHANGE IN NEGATIVE MOMENT:

$1000\, \varepsilon_t = 1000(0.0325) = 32.5\% > 20\% \text{ MAX}$

AVAILABLE INCREASE: $0.2(18.11) = 3.62 \text{ ft-kips/ft}$

ACTUAL INCREASE: $21.05 - 18.11 = 2.94 \leq 3.62 \text{ ft-kip/ft}$ OK

MOMENT CAPACITY @ MIDSPAN OF TYP. SPAN

$M_{MAX} - 2.94 = 12.13 - 2.94 = 9.19 \text{ ft-kips/ft}$

$A_{ps} f_{ps} = (0.153 \text{ in}^2/\text{TEN})(38 \text{ TEN}) \times (194.29 \text{ ksi})/40' = 16.6 \text{ k/ft}$

$a = 16.6/0.85(12)(5) = .335 \text{ IN}$

$\phi M_N = 0.9(16.6)(1/12)(6.75 - 0.335/2) = 11.10 > 6.64$

OK ✓

MOMENT CAPACITY @ MIDSPAN OF END SPAN

$\phi M_N = 0.9(16.6)(1/12)(5.00 - 0.335/2) = 6.02 > 5.39$

OK ✓

EXTERIOR COLUMNS

$A_{s\,MIN} = 0.00075\,(40')(12)(8") = 2.88\ \text{IN}^2$

$\rightarrow$ TRY $(5)\#7$

$A_s = 5 \times .60/40' = .208\ \text{IN}^2/\text{ft}$

$\rho_p = A_{ps}/bd = 38\,(0.153)/40(12)(6.75) = .0018$

$f_{ps} = 175 + 10 + \frac{5}{300\,(0.0018)} = 194\ \text{ksi}$

SAME AS INT COLUMN OK

$A_{ps}f_{ps} = 38\,(0.153)(194)/40 = 16.6\ ^k/\text{ft}$

$a = (16.6 + 60\,(.208))/(0.85)(12)(5) = 0.57\ \text{IN}$

TENDONS: $d - a/2 = 6.75 - 0.57/2 = 6.47\ \text{IN}/12 = 0.54\ \text{ft}$

REBAR: SAME PLANE AS TENDONS AT EDGE OF SLAB.

$\phi M_N = 0.9\,(16.6 + .208 \times 60)(0.54) = 14.8 > 0.75\ \text{ft-kips}/\text{ft}$

OK

SHEAR CAPACITY @ EXTERIOR COLUMN

$V_{U\,EXT} = 2.0\ ^k/\text{ft}$ ARCH PRECAST FACADE = 50 PSF

$V_U = [(1.2)(0.05)(13') + 2.0] \times 40' = 111.2\ \text{kips}$

$M_{TRANS} = 40' \times 0.75\ \text{ft-kips}/\text{ft} = 30\ \text{ft-kips}$

COMBINED SHEAR STRESS @ INSIDE FACE

$d = 0.8\,(8\text{IN}) = 6.4\ \text{IN}$

$c_1 = 24\ \text{IN}$ $b_1 = 24\text{IN} + 3.2 = 27.2\ \text{IN}$

$c_2 = 24\ \text{IN}$ $b_2 = 24\text{IN} + 6.4 = 30.4\ \text{IN}$