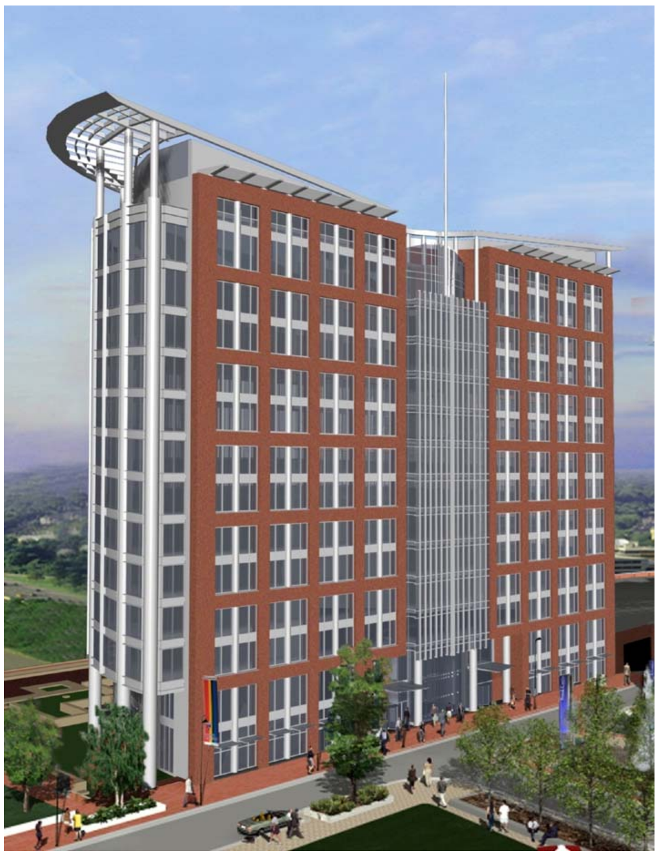
TOWERS CRESCENT
BUILDING F



CHANTILLY, VA

Benjamin M. Douglass: Structural Option
AE 481W, Tech Report 2: Floor System Comparison
October 27, 2006

Executive Summary

This report is a study of the advantages and disadvantages of various potential structural floor systems for use in Towers Crescent Building F in Chantilly, VA. The existing floor system is a flat slab with drop panels and drop bands; alternatively, I have proposed a 2-way slab, a 1-way slab with a concrete frame, precast double tees with concrete girders, precast plank on steel girders, and composite steel construction. Designs for all these systems are included in the report. Following each design is a discussion of its feasibility, cost-effectiveness, and other issues such as its fire resistance and affect on the lateral design of the building. I have concluded that the existing system has many advantages, but believe I could increase efficiency with a composite steel system.

Description of Structural System

Foundation and Slab on Grade

The building utilizes a foundation system of 80T 16"N auger cast piles. Pile caps are laid out on a roughly regular 30' x 30' structural grid as well as one semicircular line which follows the rounded face of the building. Pile groups range from 3 in the parking areas to 42 for the interior tower columns. A common pile cap for the areas supporting only parking is 6'6" x 6'6", 44" deep, contains 10 #6 reinforcing bars in each direction, and caps 4 piles. A common pile cap in the area beneath the office tower is 15' x 20', 55" deep, contains 20 #11 reinforcing bars in each direction, and caps 20 piles. The slab-on-grade is 6" thick stone concrete at f'c = 4 ksi reinforced with 6 x 6 #8/#8 W.W.F. It is placed over a vapor barrier on top of 6" of washed gravel fill.

Columns

Columns are concrete, with material strengths as follows:

Base to 2nd floor - 8 ksi
2nd floor to 8th floor - 7 ksi
8th floor to 13th floor - 6 ksi
13th floor to main roof - 5 ksi
Main roof to penthouse roof - 4 ksi.

The parking areas are held up by a mostly regular grid of concrete columns (typically 24" x 24" with 6 #9 reinforcing bars) stretching usually from the pile caps to the P-4 or P-6 level. The tower is held up by a rectangular grid of columns as well as a radial line which follows the curvature of the building. A typical internal tower column on the rectangular grid runs as follows:

Base - P2: 24"x48", 16 #18
P2 - P3: 24"x48", 16 #14
P3 - 2/P6 level: 24"x48", 20 #11
2/P6 level - 4th floor: 24"x30", 16 #11
4th floor - 5th floor: 24"x24", 16 #11
5th floor - 6th floor: 24"x24", 12 #11
6th floor - 7th floor: 24"x24", 10 #11

7th floor - 9th floor: 24"x24", 8 #11
9th floor - 13th floor: 24"x24", 6 #11
13th floor - main roof: 24"x24", 4 #11

A typical column along the semicircular line runs as follows:

Base - P3: 42" φ, 8 #11
P3 - 2/P6 level: 42" φ, 7 #11
2/P6 level - 4th floor: 36" φ, 7 #11
4th floor - main roof: 36" φ, 6 #11
Main roof - penthouse roof: 36" φ, 6 #11, W14x82

Floors

The floors are 9" minimum flat structural concrete slab (f'c = 4 ksi) reinforced by a bottom matt of #5 rebar at 12" O.C. in each direction. Where the slab is 10" thick, it is reinforced by #5 rebar at 9" O.C. in each direction, and where it is 12" thick, it is reinforced by #7 rebar at 12" O.C. in each direction. Additional reinforcement is provided as needed, almost always top reinforcement (#5 or #6) to take the tensile stresses which result from the negative moments, especially around the columns. Around every column there is a drop panel 5-1/2" below the lowest adjacent slab soffit at 1/6 the column span in each direction, a drop band 5-1/2" below the lowest adjacent slab soffit at 1/4 the column span in each direction, or a similar system.

Lateral Resistance

There is a structural core area in the center of the tower with 4 large concrete shear walls about 30' each in length, 16" thick from level P1 to P5, and 12" thick from P6 to the roof. These run through the narrow direction of the building and provide resistance against lateral loads. 6 much shorter shear walls run perpendicular to them, which leads me to believe that the natural moment frames created by the monolithically cast concrete with top reinforcement almost sufficed to provide lateral resistance in that direction. The shear walls are attached to concrete columns at either end to provide resistance against overturning moment as well as added shear capacity. Vertical reinforcement is #5 at 6" O.C. or #6 @ 6" O.C., while horizontal reinforcement is #4 at 12" or #5 at 9".

Building Codes and Design Standards

Towers Crescent Building F was designed by the 2000 USBC Virginia statewide building code, which is a variation of the IBC 2000 model code. This references the ASCE 7-98 design standard. Nevertheless, I have chosen to design by ASCE 7-02, due to my greater familiarity with it. ASCE 7 sections 6.0 and 9.0, on wind and seismic loads, respectively, are especially relevant to this assignment. Concrete structural elements would have been designed originally by the standards of ACI 318-99, but again, I will be designing based on the updated code ACI 318-02. Steel would have been designed either with the ASD manual of steel construction, 9th edition, or the LRFD manual of steel construction, 1st edition. I will use the LRFD 3rd edition. I have also taken advantage of loading tables produced by Nitterhouse Concrete Products, in sizing precast tees and planks.

Redesign

I have redesigned a section of floor for each of the following structural systems:

Flat Slab
2-Way Slab
1-Way Slab with Concrete Frame
Precast Double Tees with Concrete Girders
Precast Plank on Steel Girders
Composite Steel Construction

I have redesigned a section of four bays between the structural core and the curved face of the tower (see drawing on the next page), except while considering the two slab systems, where I redesigned a typical bay in the parking deck, in order to be able to use the direct design method.

Loads relevant to this study are as follows:

Mechanical and Lighting - 5 psf
Partitions and other finishes - 20 psf
Live - 100 psf (office building); 40 psf (parking deck)

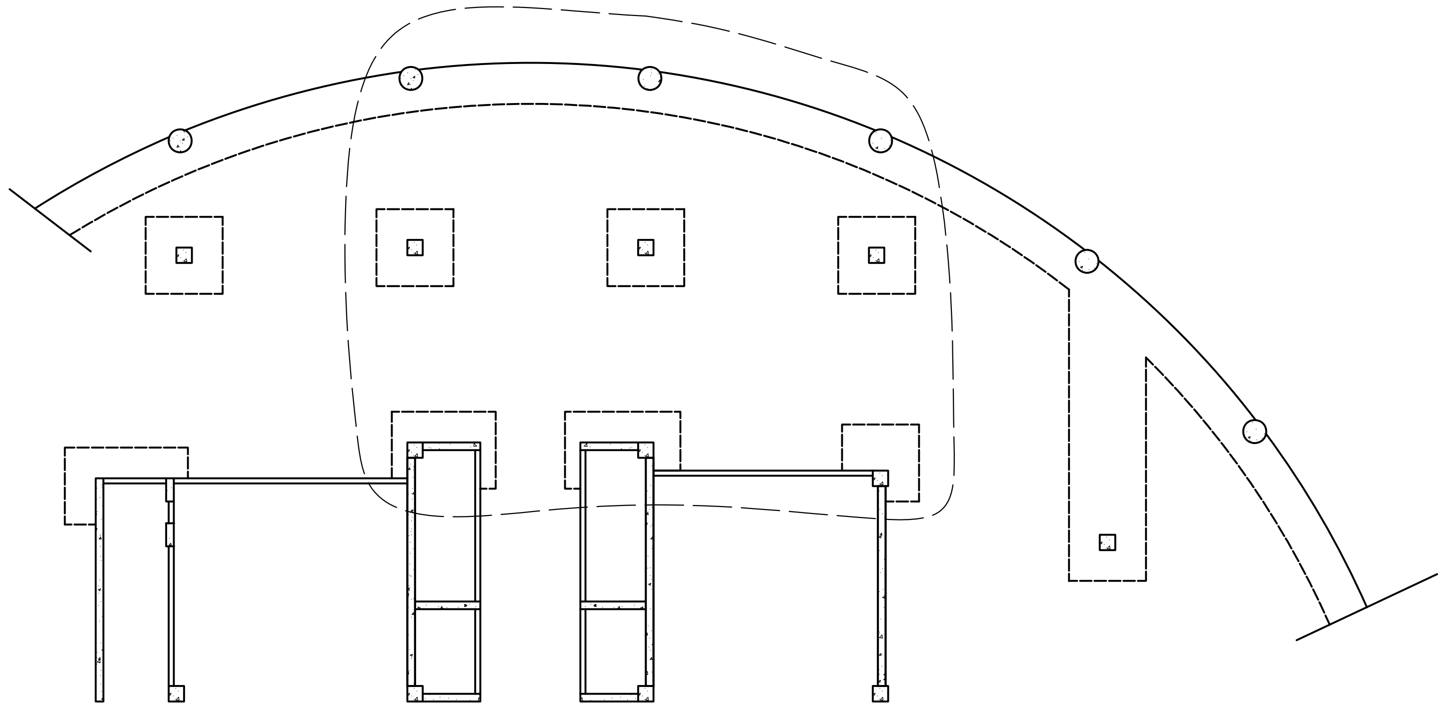
In this study all concrete is 4000 psi, all rebar reinforcement is 60 ksi, and all steel framing members are 50 ksi.

Flat Slab with Drop Panels

It is not difficult to see why the designers chose a flat slab floor system for this project. Since the tower has a curved face, naturally there are significant irregularities in the shapes of the bays. This suggests a system, such as a slab system, which is easily adaptable to irregular shapes, as opposed to a precast system where irregular shapes require expensive, specially made framing elements. Likewise, the system requires no special fire protection or consideration of vibration criteria.

Perhaps the greatest advantage of the flat slab is that it is thin. As it does not require beams, on a typical floor of the tower, the floor system is 17-1/2" at its deepest (this occurs at the drop bands which run the perimeter of the building and provide the stiffness of edge beams). In most places it is specified as being only 9" deep (though according to my calculations it ought to be 10"). But on the other hand, since this system provides no space for MEP equipment, a ceiling will have to be hung from the bottom of the slab, providing enough space for the equipment to be placed above it.

One of the great disadvantages of this system is that it is one of the heaviest. This leads to heavy loads on the columns and foundations, which require correspondingly large member sizes, as well as large seismic loads, which require a stiff lateral system. On the other hand, as is the case with any monolithically cast concrete structure, the frames have the capacity to resist bending moment and hence contribute to the lateral system. This is



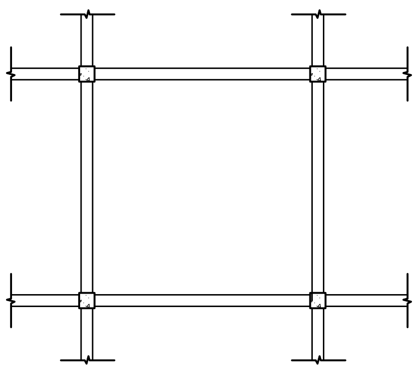
CURRENT SYSTEM: FLAT SLAB WITH DROP PANELS

BENJAMIN M. DOUGLASS	AE 481W TECH REPORT 2	FLAT SLAB	OCTOBER 27 2006

9" SLAB

BOTTOM REINFORCEMENT:	#5 @12" O.C.E.W. COLUMN STRIPS
	#5 @16" O.C.E.W. MIDDLE STRIPS
TOP REINFORCEMENT:	#5 @6" O.C.E.W. COLUMN STRIPS
	#5 @16" O.C.E.W. MIDDLE STRIPS

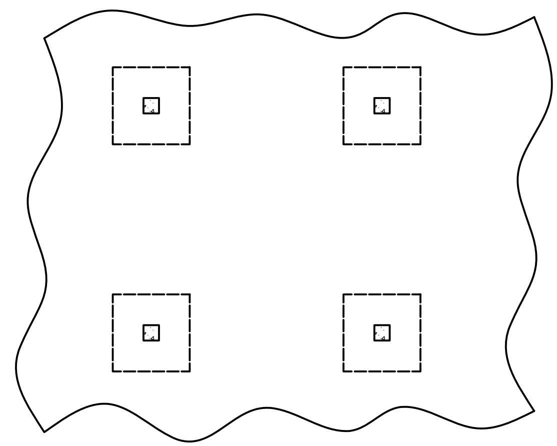
Beams: 18"x24"



2-WAY SLAB

10" SLAB

BOTTOM REINFORCEMENT:	#5 @14" O.C.E.W. COLUMN STRIPS
	#5 @16" O.C.E.W. MIDDLE STRIPS
TOP REINFORCEMENT:	#5 @6" O.C.E.W. COLUMN STRIPS
	#5 @16" O.C.E.W. MIDDLE STRIPS



FLAT SLAB

BENJAMIN M. DOUGLASS	AE 481W TECH REPORT 2	2-WAY SLAB FLAT SLAB	OCTOBER 27 2006

most likely why there are hardly any shear walls in parallel to the long direction of the building.

This system is also slow to construct. Construction crews must build elaborate and sometimes irregularly shaped formwork structures; the concrete must be given time to cure before it can resist loads, which sometimes necessitates temporary braces; and the concrete must be finished. In any case, the design is on the previous page, and calculations are in the appendix.

2-Way Slab

Most all of the above stipulations apply to 2-way slabs as well. I will note only a few significant differences. First, the slab can be somewhat thinner and lighter due to the stiffness provided it by the beams which run between the columns. In my calculations I was able to reduce the slab thickness from 10” to 9”. On the other hand, the beams need to be somewhat deep in order to do this, and this mitigates the great advantage of the slab, viz., that it is thin.

Since the formwork involved is simpler, and the amount of concrete used is slightly less, this system will probably be cheaper than the flat slab with drop panels. One final advantage is that the frames will have an increased moment capacity over the flat slab system, which might allow the designer to reduce the size of the shear walls. The design is on the previous page, and calculations are in the appendix.

1-Way Slab and Concrete Frame

The advantages and disadvantages of the concrete frame are roughly the same as those of the 2-way slab, only taken slightly farther. The slab can be still thinner and lighter since it only spans a short distance, but the beams must be deeper, though the system is still not so deep as a steel frame with a slab on top. The frames will have an even greater moment resisting capacity. One final note, the overall amount of reinforcement needed is decreased, since much less reinforcement is required in the slab. This combined with the reduction in slab thickness will result in savings in material costs. The design is below, and calculations are farther below.

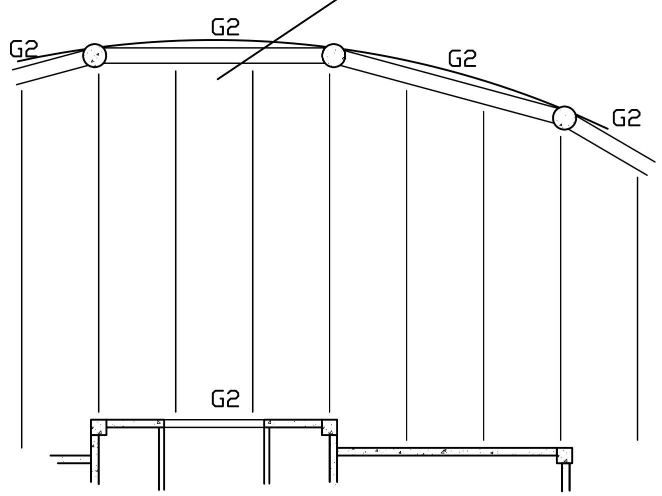
Precast Double Tee with Concrete Girders

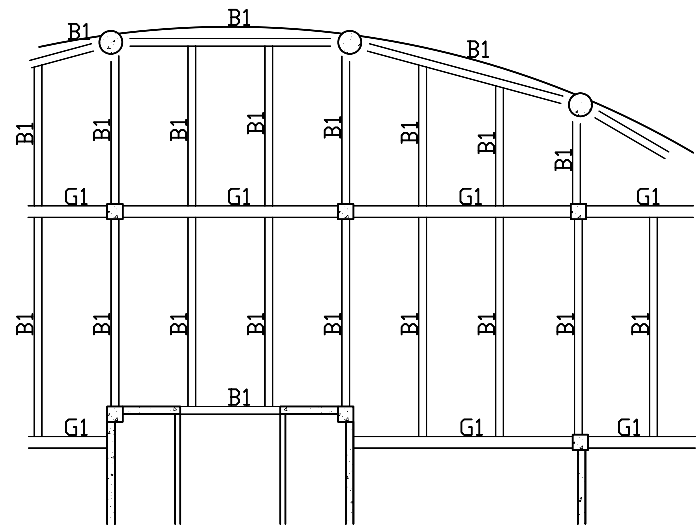
Precast double tee sections have a few significant advantages. Due to their large moment resisting capacity, they are capable of spanning large distances. In the case of this building, switching to such a system would enable the designer to, for the most part, eliminate intermediate columns and span directly from the shear wall to the columns at the edge of the building. The drawings on the two pages after the next show the existing column layout, and a proposed new column layout for use with systems which can span longer distances.

Precast tees will also give the designer the necessary 2 hour fire rating, and provide plenty of room for mechanical systems. They can also be put in place very quickly. Unfortunately, due to the nature of prestressed concrete elements, the member will tend

BEAM	B (IN.)	H(IN.)	TOP	BOTTOM	SHEAR
B1	12	24	(4) #8	(4) #8	1 LEG #4 @8" O.C.
G1	18	28	(7) #8	(7) #8	1 LEG #4 @6" O.C.

USE SECTION 32 - 12.6 PT,
WITH 10' WIDTH



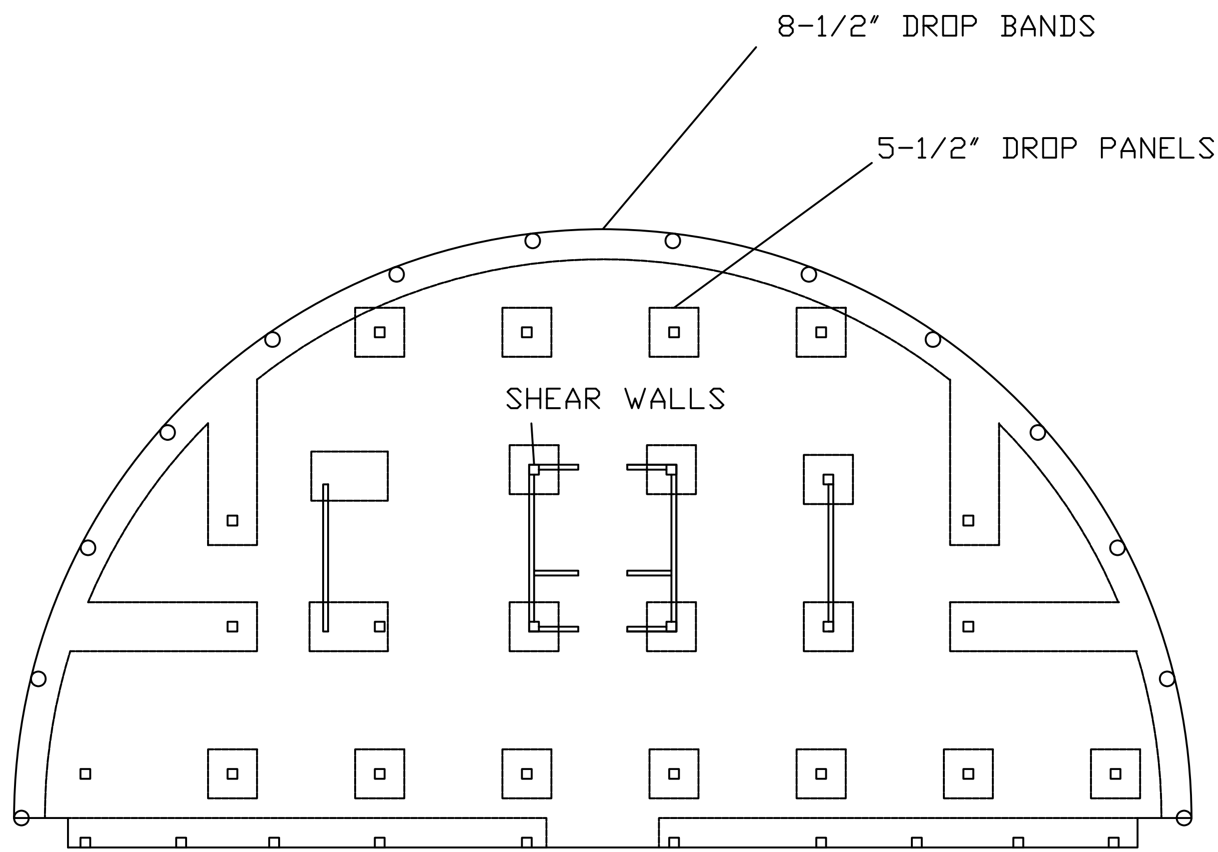


PRECAST DOUBLE Ts
WITH CONCRETE GIRDERS

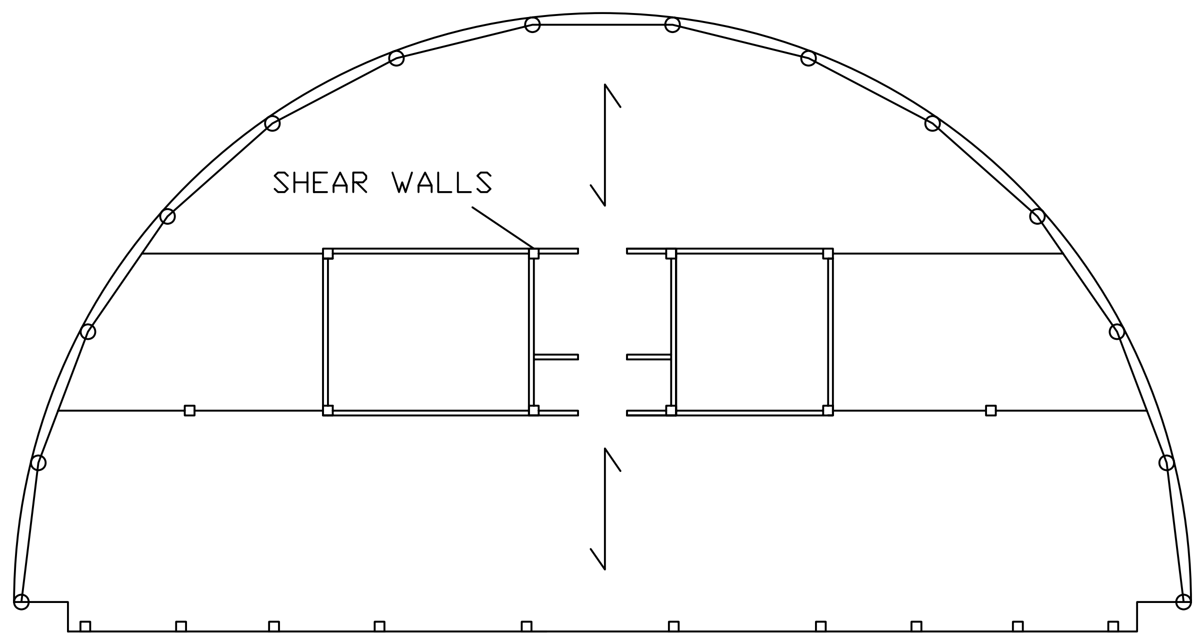
1-WAY SLAB WITH CONCRETE FRAME

BEAM	B (IN.)	H(IN.)	TOP	BOTTOM	SHEAR
G2	18	32	(7) #8	(7) #8	1 LEG #4 @6" O.C.

BENJAMIN M. DOUGLASS	AE 481W TECH REPORT 2	PRECAST TEES 1-WAY SLAB	OCTOBER 27 2006



BENJAMIN M. DOUGLASS	AE 481W TECH REPORT 2	CURRENT SYS & COLUMN LOCATIONS	OCTOBER 27 2006



BENJAMIN M. DOUGLASS	AE 481W TECH REPORT 2	COL LOCATIONS W/ LONG SPAN SYS	OCTOBER 27 2006

to deflect upwards, which situation must be ameliorated with a thin, 3” concrete topping. This will detract from the advantage of the quick construction of the floor.

Furthermore, there are very serious disadvantages which preclude the use of precast tees on this particular project. First, it would require either rearranging the basic grid structure of the building, or specially ordering 10’ wide members. The standard width for precast tees is 12’, but the grid structure of Towers Crescent Building F is 30’x30’. Ordering special tees to accommodate this situation would significantly drive up costs. For that matter, the mere fact that the floor plan of the tower is irregularly shaped in places would necessitate expensive, specially ordered and oddly shaped members. Precast tees were not made for semicircles with rectangles inside them.

Next, a system of precast tees would have no inherent moment resisting capacity. As such, which the existing shear walls running perpendicular to the long direction of the building would suffice, shear walls would have to be added in the other direction. Finally, at 35”, this system is one of the deepest, and would require either some reduction in the ceiling height of the building, or an increase in floor to floor height. See the third page back for the design, and the appendix for the calculations

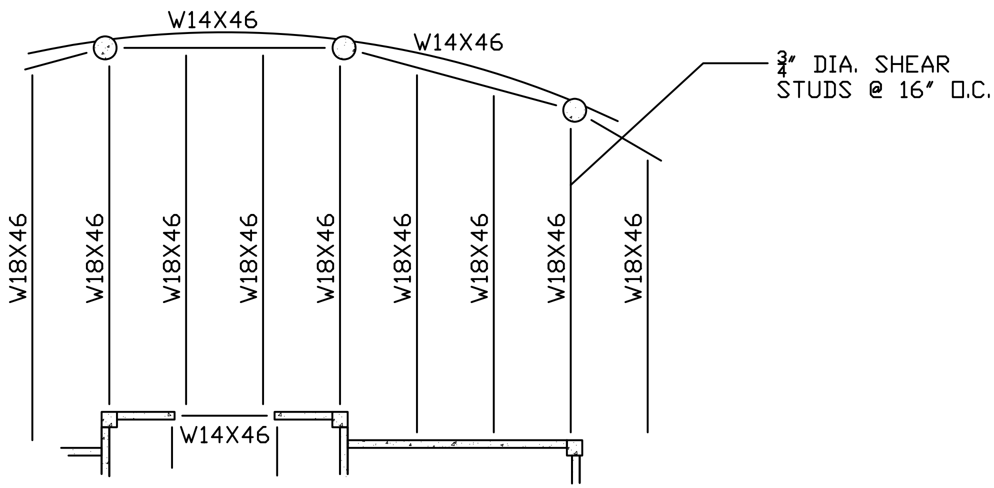
<u>Precast Plank on Steel Girders</u>

As was the case with precast tees, employing a system of precast plank will allow the designer to eliminate many of the columns in the interior of the office tower. Likewise, this system has the same advantage in its short erection time (even shorter given that the girders are steel instead of concrete), and the same catch, viz., it requires a concrete topping to mitigate problems with deflection. Unlike prestressed tees, on the other hand, the steel girders which support this system would require a coating of fireproofing to achieve the requisite 2 hour fire rating. Finally, precast plank shares the same disadvantages as precast tees of being ill-adapted to irregularly shaped floors, and not providing an inherent lateral system (and therefore necessitating a new lateral system in one direction), but at 27” deep, is at least the system does not consume so much space. It is pictured on the next page, and the design process is provided in the appendix.

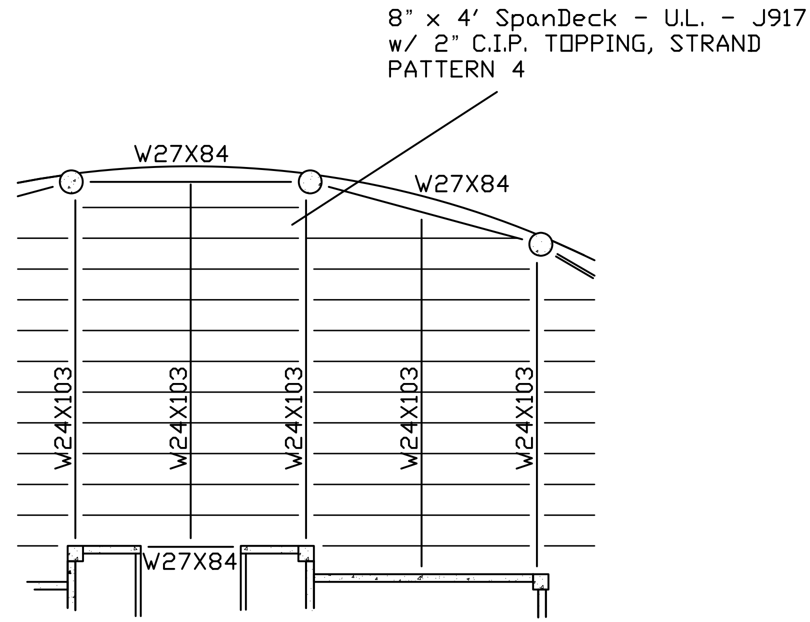
<u>Composite Steel Construction</u>

As with the above, this system obviates a number of interior columns in the tower. Furthermore, at 24”, it is not much deeper than the thickest point in the currently specified flat slab. Yet, it still leaves 18” open for mechanical space. This is by far the lightest system; the concrete slab is only 6” think and, due to the 3” metal deck, effectively weighs as much as a 4.5” slab. The steel members are also relatively light. As such, materials costs will be low, and it might be possible to decrease the size and strength of the foundation system. The system is quick to erect, besides the time required to pour the slab. Furthermore, it is easily adaptable to the irregular shape of the tower’s floor plan.

The major disadvantage of this system is that it will require an additional lateral system along the long face of the building, to compensate for the loss of moment frames. This could be done by adding shear walls perpendicular to the current shear walls, or by



COMPOSITE STEEL BEAMS



STEEL BEAMS WITH PRECAST PLANK

BENJAMIN M. DOUGLASS	AE 481W TECH REPORT 2	COMPOSITE STEEL PRECAST PLANK	OCTOBER 27 2006

eliminating the shear walls entirely in favor of braced frames or some such system. Another disadvantage is that, as is always the case with steel, it requires a coating of fireproofing to meet the requisite 2 hour fire rating. See the previous page for the design; the calculations, as always, are in the appendix.

Conclusions

The composite steel floor system appears to me to be the best choice for this particular building, for the reasons enumerated above. Though it has its own disadvantages, I believe that the advantages outweigh them enough to justify pursuing this design system further.

APPENDIX

Flat Slab with Drop Panels

Minimum thickness: ln = 30 – 2 = 28' ; ln/33 = (28)(12)/36 = 9.33" ; use 10"
Load: 1.2(150(10/12)) + 1.2(5) + 1.6(40) = 220 psf

d = 8.75; 15(12) – 12 – 8.75 = 13.271'
Vu = 0.220(13.271) = 2.92k
ΦVc = 0.75(2)(4000^.5)(12)(8.75) = 9.96 k (o.k.)
Vu = 0.220((30)(29.5) – ((24 + 8.75)/12)^2) = 193k
ΦVc = (0.75)(4)(4000)^.5(4)(32.75)(8.75) = 217k > 193k (o.k.)

Mo = (0.220)(29.5)(28^2)/8 = 636 ft-k

Column Strips
Positive: 0.21Mo = 134 ft-k
b = 29.5(12)/2 = 177" ; d = 10 – ¾ - 5/16 = 8.94"
Min As = 0.0018bt = 0.0018(177)(10) = 3.2 in.^{2}
Try #5 at 14" O.C.
As = 13(0.31) = 4.03 in.^{2}
a = 4.03(60)/(0.85(4)(177)) = 0.4018
ΦMn = (0.9)(4.03)(60)(8.94 – 0.4018/2) = 158 ft-k (o.k.)

Negative: 0.49Mo = 312 ft-k
Try #5 at 6" O.C.
As = 30(0.31) = 9.3 in.^{2}
a = 9.3(60)/(0.85(4)(177)) = 0.9272
ΦMn = (0.9)(9.3)(60)(8.94 - 0.927/2) = 355 ft-k (o.k.)

Middle Strips
Positive: 0.14Mo = 89 ft-k
Try #5 at 16" O.C.
As = 11(0.31) = 3.41 in.^{2}
a = 3.41(60)/(0.85(4)(177)) = 0.34
ΦMn = (0.9)(3.41)(60)(8.94 – 0.34/2) = 135 ft-k (o.k.)

Negative: 0.16Mo = 102 ft-k
Use same reinforcement as positive.

2-Way Slab
Assume 9” slab, 18”x24” beams

NS Interior Beams
l_2 = 30(12) = 360” ; a/h = 24/9 = 2.667 ; b/h = 18/9 = 2 ; f = 1.54
α_f = (18/360)(24/9)^3(1.54) = 1.46

EW Interior Beams
l_2 = 29.5(12) = 354” ; a/h = 24/9 = 2.667 ; b/h = 18/9 = 2 ; f = 1.54
α_f = (18/354)(24/9)^3(1.54) = 1.485

α_m = (1.46 + 1.485)/2 = 1.47
Slab supported by medium stiff beams.

ln = 360 – 24 = 336 ; t_{min} = 336(0.8 + 60/200)/(36 + 5(28/27.5)(1.47 – 0.2) = 8.7”
Use 9” slab

1.2(150(19/12)) + 1.2(5) + 1.6(40) = 205 psf
Mo = (0.205)(29.5)(28^2)/8 = 593 ft-k

Positive: 0.35Mo = 208 ft-k
60 + 30(1.485)(30)/29.5 * (1.5 – 30/29.5) = 74.5%
Column Strip: 0.745(208) = 155 ft-k ; Use #5 at 12” O.C.
Middle Strip: 0.255(208) = 53 ft-k ; Use #5 at 16” O.C.

Negative: 0.65Mo = 385 ft-k
75 + 30(1.485)(30)/29.5 * (1 – 30/29.5) = 75.5%
Column Strip: 0.755(385) = 291 ft-k ; Use #5 at 6” O.C.
Middle Strip: 0.245(385) = 94 ft-k ; Use #5 at 16” O.C.

1-Way Slab and Concrete Frame

Use 6" slab.

Typical beam load area: 10' x 28.5' = 285 ft.^{2}
L = 100(0.25 + 15/(2*285)^.5) = 87.8 psf
Girder load area: 28' x 20' = 560 ft.^{2}
L = 100(0.25 + 15/(2*560)^.5) = 70 psf

Total factored load for beam calcs: 1.2(20 + 150(6/12)) + 1.6(87.8) = 255 psf
Total factored load for girder calcs: 1.2(20 + 150(6/12)) + 1.6(70) = 226 psf
Distributed load on concrete beams: (255(10) + (1.2)(150)(1)(1.5))/1000 = 2.82 klf
Concentrated load on girders: 226(23.5)(10) + (1.2)(150)(1)(1.5)(23.5) = 59.5^k

Beams

Beams are integral with supports. ACI Moment coefficients are not applicable because the spans are not equal in length. Design 28'6" interior span for wl^2/9 negative moment and wl^2/14 positive moment (conservative). Use the same beam for the shorter spans.

Mu = 2.82(28.5^2)/9 = 255 ft-k.
Mu = 2.82(28.5^2)/14 = 164 ft-k.
Vu = (2.82)(28.5)/2 = 40.2^k

Use 12"x24" beams with (4) #8 reinforcing bars top and bottom, and (1) leg #4 shear reinforcement at 8" O.C. See spreadsheet.

Girders

Mo = 59.5(9) + (1.2)(150)(1.5)(1.5)(28.25^2)/8 = 576 ft-k.
Vu = 59.5 + (1.2)(150)(1.5)(1.5)(14) = 65.2^k
Use 18"x28" beams with (7) #8 reinforcing bars top and bottom, and (1) leg #4 shear reinforcement at 6" O.C. See spreadsheet.

<u>Precast Double Tee with Concrete Girders</u>

<u>Precast Double Tee</u>

32” x 10’ Double Tee with 3” cast in place topping. Weight: 95 psf.

Smallest load area for a Double Tee: 41.5’ x 10’ = 415 ft.^{2}

L = 100(0.25 + 15/(2*415)^.5) = 77.1 psf

Total factored, superimposed load: 1.6(77.1 + 20) = 156 psf

Use Section 32 – 12.6 PT (163 psf safe superimposed loading at 48’ span – Nitterhouse load tables).

<u>Concrete Girders</u>

Typical load area: 23(30) = 690 ft.^{2}

L = 100(0.25 + 15/(2*690)^.5) = 65.4 psf

Dist. load: (1.2)(20 + 95)(23.5) + (1.2)(150)(1)(2.667) + (1.6)(65.4)(23.5) = 6.42 klf

Mo = 6.42(28^2)/8 = 629 ft-k

Vu = 6.42(28)/2 = 90^k

Use 18”x32” beams with (7) #8 reinforcing bars top and bottom, and (1) leg #4 shear reinforcement at 6” O.C. See spreadsheet

Precast Plank on Steel Girders

8" x 4' SpanDeck – U.L. – J917 with 2" cast in place concrete topping, strand pattern 4.
347 psf allowable superimposed load at 15' span (Nitterhouse load table).
Weight: 82.5 psf.

Smallest load area for a steel beam: 41.5' x 15' = 622.5 ft.^{2}
L = 100(0.25 + 15/(2*622.5)^.5) = 67.5 psf
Smallest load area for a steel girder: 23.5' x 15' = 352.5 ft.^{2}
L = 100(0.25 + 15/(2*352.5)^.5) = 81.5 psf

Total factored load: 1.2(20 + 82.5) + 1.6(67.5) = 231 psf
Distributed load on steel beams: (231(15) + 1.2(103))/1000 = 3.59 klf

Simply Supported Beams

41.5' span: Mu = 3.59(41.5^2)/8 = 773 ft-k.
Mu/(0.9*50) = 206 in.^{3} = Z_{req}; Use W24x103.

47' span: Mu = 3.59(47^2)/8 = 991 ft-k.
Mu/(0.9*50) = 264 in.^{3} = Z_{req}; Use W24x103

48' span: Mu = 3.59(48^2)/8 = 1034 ft-k.
Mu/(0.9*50) = 276 in.^{3} = Z_{req}; Use W24x103.

28' Simply Supported Girders

Loads: 1.2(20 + 82.5) + 1.6(81.5) = 253.4
253.4(15)(23.5) + (1.2)(103)(23.5) = 92.2^k @ 14'
Vu = 92.2/2 + (1.2)(84)(14) = 47.5^k; Mu = 47.5(14) + (1.2)(84)(28^2)/8 = 675 ft-k.
Mu/(0.9*50) = 180 in.^{3} = Z_{req}; Use W27x84.
Use same for 13' girder.

Composite Steel

Smallest load area for a steel beam: 41.5' x 10' = 415 $ft.^2$
L = 100(0.25 + 15/(2*415)^.5) = 77.1 psf
Use same for girders.

Try 6" slab on with 3" metal deck. Total factored load: 1.2(20 + 150(4.5/12)) + 1.6(77.1) = 215 psf

Distributed load on steel beams: (215(10) + (1.2)(46))/1000 = 2.21 klf

Simply Supported Beams

41.5' span: Mu = 2.21(41.5^2)/8 = 475 ft-k.
Assume a = 1"
Y2 = 5", try W18x46, $\Sigma Qn = 494^k$
a = 494/(.85(4)(120) = 1.21" > 1"
Try Y2 = 4.5", W18x46, $\Sigma Qn = 494^k$
a = 494/(.85(4)(120) = 1.21" < 1.5" (o.k.)
$\Sigma Qn = 17.2n = 494^k$; n = 29 studs.

Use W18x46 with ¾" Φ shear studs at 16" O.C.

48' span: Mu = 2.21(48 ^2)/8 = 635 ft-k.
Assume a = 1"
Y2 = 5", try W18x46, $\Sigma Qn = 494^k$
a = 494/(.85(4)(120) = 1.21" > 1"
Try Y2 = 4.5", W18x46, $\Sigma Qn = 585^k$
A = 585/(.85(4)(120) = 1.434" < 1.5" (o.k.)
$\Sigma Qn = 17.2n = 585^k$; n = 34 studs.

Use W18x46 with ¾" Φ shear studs at 16" O.C.

28' Simply Supported Girders

Loads: 221(10)(23.5) + (1.2)(46)(23.5) = 53.2^k @ 9' and 19'
Vu = 53.2^k; Mu = 53.2(10) = 532 ft-k.
Try W18x46 with ¾" Φ shear studs at 16" O.C.
21 shear studs, $\Sigma Qn = 361.2^k$
a = 361.2/(.85(4)(120) = .89" < 1"
ΦMn > 532 ft-k. (o.k.)
Use same for 13" girder.

CONCRETE BEAM DESIGN

Thornton Tomasetti Engineers
Overture Phase II

Note for FLEXURE
1. Limit reinf to 75% $A_{s\ max}$
2. ρ_b (or $A_{s\ max}$) is computed without compression reinforcement
3. Using initial ϕ value of 0.9 to compute As required

			MAIN BARS			STIRRUPS			PROPERTIES												Use for Design			FLEXURAL DESIGN													SHEAR DESIGN										
	Edge Beam	TOP / BOT	# Layers	# Bars	Bar #	Total Legs	Bar #	Spacing	f_y	f'_c	β_1	L	L_n	b	h	h_{min}	clear cover	sequence	d	d_t	M_u	V_u	T_u	$A_{s\ reqd}$	$A_{s\ prov}$	% $A_{s\ max}$	$A_{s\ min}$	$A_{s\ max}$	a	ϕ	ϕM_n	M_u	$M_u/\phi M_n$	s_{clear}	s_{min}	s_{max}	$A_{v\ prov}$	$A_{v\ min}$	s_{max}	V_c	$V_{s\ reqd}$	$V_{s\ prov}$	ϕ	ϕV_n	$\phi V_{n\ max}$	V_u	$V_u/\phi V_n$
									(psi)	(psi)		(ft)	(ft)	(in)	(in)	(in)	(in)		(in)	(in)	(kip.ft)	(kip)	(kip.ft)	(in^2)	(in^2)		(in^2)	(in^2)	(in)		(kip.ft)	(kip.ft)		(in)	(in)	(in)	(in^2)	(in^2)	(in)	(kip)	(kip)	(kip)		(kip)	(kip)	(kip)	
ACI 318-99																9.5.2											10.5.1	10.3.5		9.3.2						10.6.4		11.5.5.3	11.5.4	11.3.1.1		11.5.6.2	9.3.2		11.5.6.9		
NEDA'S																			1" clr vert spacing																												
1	3	4	5	6	7	8	9	10	16	17	18	19	20	21	22	23	24	25	26	27	49	50	51	53	54	55	56	57	59	60	61	62	63	64	65	66	71	72	73	74	75	76	77	78	79	80	81
Fbeam		TOP	1	4	8	1	4	8	60000	4000	0.85	30.0	28.5	12.0	24.0	22.5	1.5	1	21.50	21.50	255	91		2.93	3.16	57%	0.86	5.52	4.65	0.90	273	255	0.94	1.3	1.00	11.0	0.20	0.08	10.8	33	20	32	0.75	49	98	40	0.81
Fbeam		BOT	1	3	8	1	4	8	60000	4000	0.85	30.0	28.5	12.0	24.0		1.5	1	21.50	21.50	164	91		1.81	2.37	43%	0.86	5.52	3.49	0.90	211	164	0.78	2.5	1.00	11.0	0.20	0.08	10.8	33	20	32	0.75	49	98	40	0.81
Fgirder		TOP	1	7	8	1	4	6	60000	4000	0.85	30.0	28.5	18.0	28.0	22.5	1.5	1	25.50	25.50	550	65		5.34	5.53	56%	1.53	9.81	5.42	0.90	567	550	***0.97***	1.2	1.00	11.0	0.20	0.09	12.8	58	29	51	0.75	82	174	65	0.80
Fgirder		BOT	1	7	8	1	4	6	60000	4000	0.85	30.0	28.5	18.0	28.0		1.5	1	25.50	25.50	550	65		5.34	5.53	56%	1.53	9.81	5.42	0.90	567	550	***0.97***	1.2	1.00	11.0	0.20	0.09	12.8	58	29	51	0.75	82	174	65	0.80
Tgirder		TOP	1	7	8	1	4	6	60000	4000	0.85	30.0	28.5	18.0	32.0	22.5	1.5	1	29.50	29.50	600	90		4.92	5.53	49%	1.77	11.35	5.42	0.90	667	600	0.90	1.2	1.00	11.0	0.20	0.09	14.8	67	53	59	0.75	95	202	90	***0.95***
Tgirder		BOT	1	7	8	1	4	6	60000	4000	0.85	30.0	28.5	18.0	32.0		1.5	1	29.50	29.50	600	90		4.92	5.53	49%	1.77	11.35	5.42	0.90	667	600	0.90	1.2	1.00	11.0	0.20	0.09	14.8	67	53	59	0.75	95	202	90	***0.95***