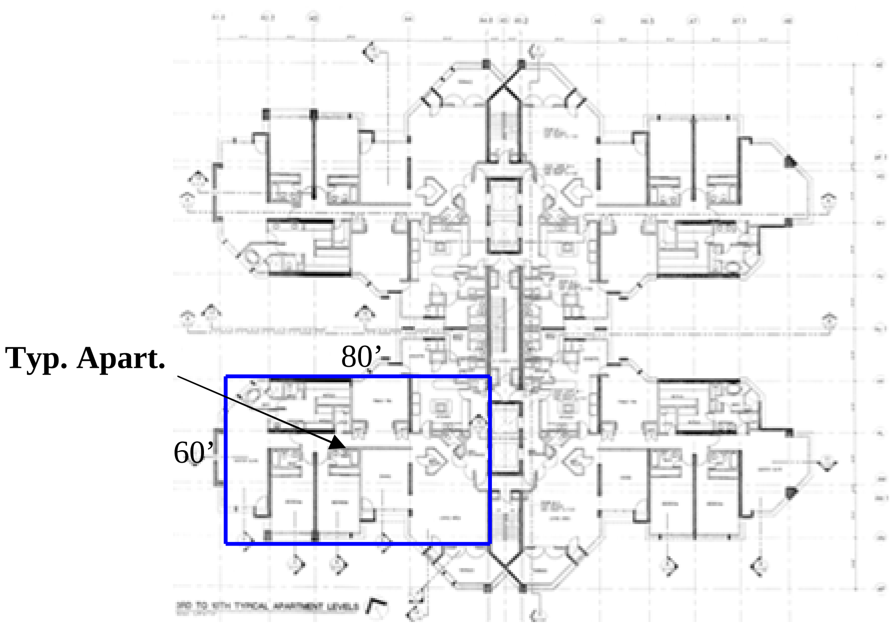
October 31, 2005

Lourdes Diaz
Structural Option
Advisor: Thomas Boothby
Building: Paseo Caribe Condominium and Parking Garage

Technical Report #2
Structural Study of Alternate Floor Systems

Building Introduction and Loading Conditions

The structure of the Condominium and Parking Garage is reinforced cast in place concrete. The building raises 14 stories above a previously constructed 10 story parking garage. There are four apartments per floor, two at each side of a 10’ wide core that contain the four elevator units and 3 sets of stairs. Each apartment is approximately a square with dimensions of 80’ east to west and 60’ north to south. Since the building is symmetrical about both axis, the analysis of the structural floor system will be based on a typical apartment span frame.



Loads and Requirements as applicable to the design of the structural floor are:

A) Live Loads
   a. Roof 40psf
   b. Floor **40psf**

c. Stairs **100psf**
d. Corridors **100psf**
e. Terrace **60psf**
f. Parking 50psf
g. Storage 125psf
h. Pool Deck 100psf

B) Dead Loads

a. Slab – 8” thick	100psf
b. Non – Bearing Concrete Block Walls	20 psf
c. Superimposed MEP	25 psf
d. Shear walls - 9’ 2” High (per longitudinal area of wall)	1375 psf

C) Strength Requirements

a. Concrete (28 day strength)

- Structural Slabs:	4,500psi
- Beams:	4,500psi
- Columns:	5,000psi
- Walls:	4,000psi
- Stairs:	4,000psi

b. Steel (Yield Strength, F_y)

- Reinforcement bars:	60,000psi
- Welded Wire Fabric:	50,000psi
- W Shapes – A992	50,000psi
- Plates, Channels, Angles, M, S Shapes	36,000psi
- Welding – E70xx	70,000psi
- Bolts – ASTM A 325	90,000psi

D) Steel Cover Requirements

c. Slab on Grade/Mat Foundation	1”
d. Slab/Joist	
- Up to #11	**¾”**
- #14 or larger	1-1/2”
e. Beams/Columns	1-1/2”

E) Post-Tensioning

f. Concrete

- Compressive strength at transfer	2,500psi

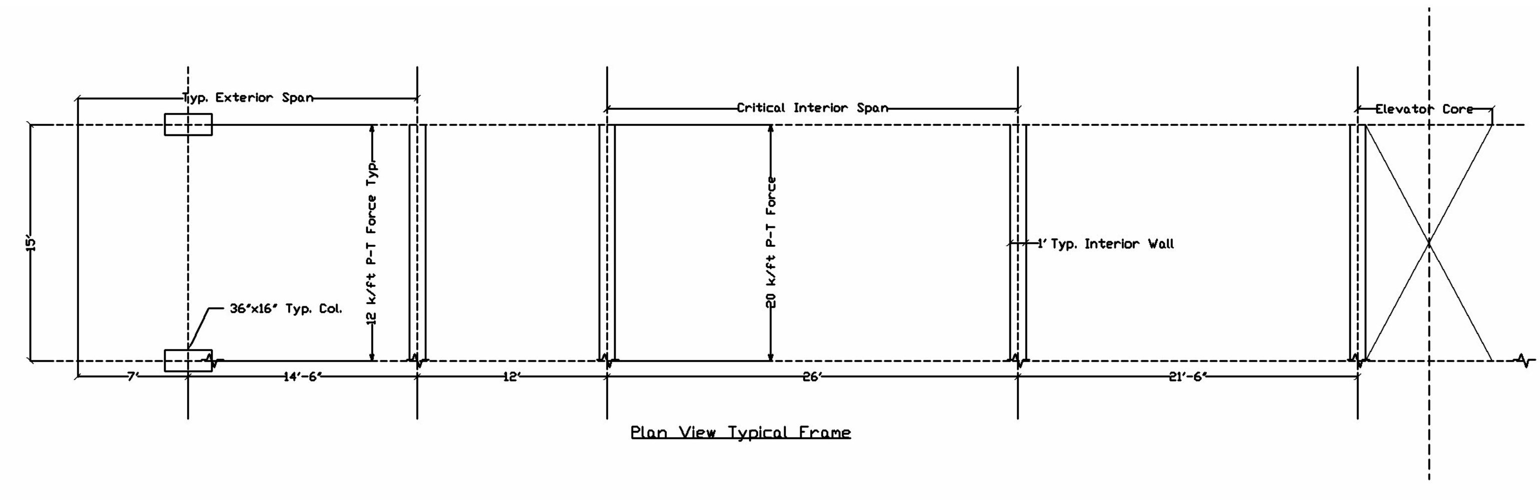
g. Steel

- Yield strength	270,000psi
- Effective stress after losses	171,000psi
- Preliminary long term losses	15,000psi

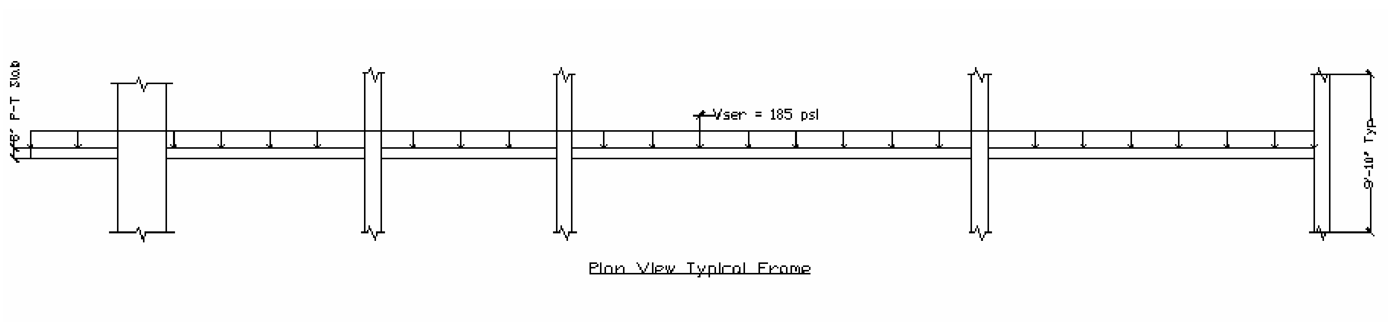
Existing Structural Floor System

The current floor system consists of a one way cast in place post tensioned 8" concrete slab on each floor. The floor slab in supported in the interior bays by 12" wide interior shear walls spanning north to south and by 16 columns around the perimeter. There are 2 columns and 4 shear walls per apartment. The slab spans east to west between shear wall supports. The typical column size is 16" x 36". The shear walls run parallel to each other. The largest interior span in between shear walls is 26'; other interior spans are 22' and 14'. The largest exterior span between column and shear wall is 14.5'.

<u>Frame Layout of Existing System:</u>



Plan View Typical Frame



Plan View Typical Frame

Drawing specifications shows that the slab is designed for a post tensioned effective compressive stress of 12k/ft in both directions. This design value is increased to 20 k/ft at the location of the largest 26’ span. Post-tensioning tendons for this slab are 7 wire. There is post-tensioning of the concrete on both directions, N-S and E-W. However, the primary action of this one way slab is from East to West, which coincides with the short direction between shear wall supports. There is also regular reinforcement in this directions further suggesting the one way action of the slab. In the transverse N-S direction, the tendons are located directly over the shear walls and are used for deflection and crack control.

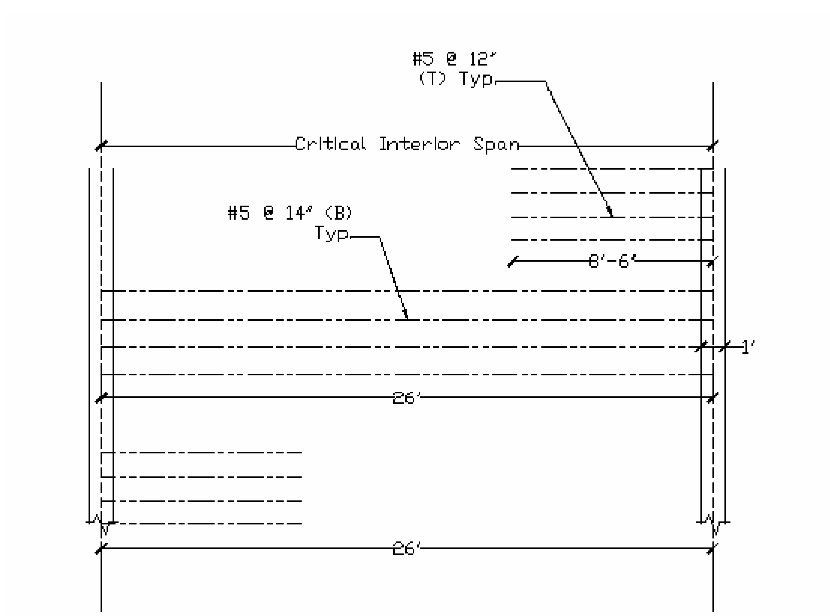
The slab is reinforced in the east to west direction with regular reinforcing bars. The typical bottom reinforcement is #5 bars. Typically:

- Spans < 15’	#5@18”
- 15’-22’ Spans	#5@16”
- Spans > 22’	#5@14”
- Middle core	#5@12”
- North-South core perimeter	#5@10”

Top positive reinforcement is provided over the shear wall supports. Reinforcement extends 1/3 times the span on each side of the span from the centerline of the support.

For the largest span, Lmax = 26’, the typical layout of the reinforcements is: negative reinforcement extends 8.5’ from the centerline of the shear wall support. Typical reinforcement is #5 bars. For spans < 17’, use #5@18”. Larger spans use #5 @ 12.

Critical Reinforcement Layout:



Existing Typical Reinforcement Layout

An analysis of this system was performed by hand. The calculations are based on one foot strip. Calculation includes:

Three permissible stress checks:

1. Stresses at transfer due to self weight
   - Extreme fiber compression: fc < 0.6fci’
   - Extreme fiber in tension: ft < 6• fci’
2. Stresses at service unfactored loads
   - Sustained loads (Dead loads only)
     - § Extreme fiber compression: fc<0.45fc’
     - § Extreme fiber tension for Class U – assumes un-cracked under full service loads: ft<0.75fc’
   - Total Loads (Dead loads and live loads)
3. Flexural Strength check
   - Extreme fiber compression: fc<0.6fc’
   - Extreme fiber tension < 0.75fc’

A summary is provided here, detailed calculations can be found in Appendix B

1. Permissible Stresses at Transfer

Dp = 6.75”
Lmax = 26’
S= 12*8^2/6 = 128 in3
Po= 12 k/ft
A = 12*8 = 96in2
e = 3”
fci’ = 2500 psi
Assume 5% initial losses
Initial Stress:
    Md=25^2(100)/11 = 6.15’-k
    Md/S = 576 psi tension top
        -576 psi compression bottom
Prestress Effect: Po/A ± Po(e)/S
        = -406.25 top compression
          156.25 bottom tension
Net Stresses at transfer:
    Top: 576 – 406.25 = **169.75 psi < 6• fci’ = 300psi** Good
    Bottom: -576 + 156.25 = **-419,75 < 0.6*fci’ = -1500psi** Good

2. Service Stress Check Summary

fc = 4500 psi	Exterior Span	1st Int. Span	2nd Int. Span	3rd Int. Span
Length	14.500	12.000	26.000	21.500
P (kip/ft)	12.000	12.000	20.000	12.000
A (in2)	96.000	96.000	96.000	96.000
S(in3)	128.000	128.000	128.000	128.000
P/A (psi)	125.000	125.000	208.333	125.000
e(in)	3.000	3.000	3.000	3.000
P(e)/S	281.250	281.250	468.750	281.250
Sustained Check	fc-allow (psi)	-2025.000	ft-allow (psi)	402.492
Wsus (psf)	125.000	125.000	125.000	125.000
Msus ('k)	2.628	1.636	7.682	5.253
Msus/S	246.387	153.409	720.170	492.454
fc-actual (psi)	**-90.137**	**2.841**	**-459.754**	**-336.204**
ft(psi)	**-159.863**	**-252.841**	**43.087**	**86.204**
Service Check	fc-allow (psi)	-2700	ft-allow (psi)	402.492
Wser (psf)	185.000	185.000	185.000	185.000
Mser ('k)	3.890	2.422	11.369	7.774
Mser/S	364.652	227.045	1065.852	728.832
fc-actual (psi)	**-208.402**	**-70.795**	**-805.436**	**-572.582**
ft(psi)	**-41.598**	**-179.205**	**388.769**	**322.582**

- compression + tension

3. Flexural Strength – Factor Loads

A) Without Rebar

According to UBC 97 and given live and dead loads:
Wu = 1.4(Wdl) + 1.7(Wll)
Wdl= 150pcf*(8/12) + 25psf superimposed
Wll= 40psf typ floor + 20psf partitions
Wu = 277 psf

Capacity for unbonded tendons
fsu = fse + 1.0f’c/100p + 10ksi
p = Aps/bdp = (12/24.8)(.153)/(12*6.75”) = .000914
fse = 171 ksi
fsu = 230 ksi
Fult = (230/171)*12 = 16.14 k/ft
Mu[1] = 0.9(16.14 k/ft)*(6.57”/12”/ft) = **8.1’k < 11.5 ‘k** à Rebar is needed

B) Strength Calculations including Rebar

As provided at Lmax = #5 @ 14” = 0.265 in2/ft

Fu-reb = 0.265 * 60 = 15.94 k

a = (15.94 + 16.14)/(3.83*12) = 0.7”

jd-p = 8” -0.35”-1.25” = 6.4”

jd-r = 8” – 0.35” – 1.0” = 6.65”

Mu = (.9)(16.14’k(6.4”/12) + 15.94’k(6.65”/12)) = 15.84’k

By limit design: Wu(l^2)/8 = 15.84 ft-k + 8.1 ft-k = 23.94 ft-k

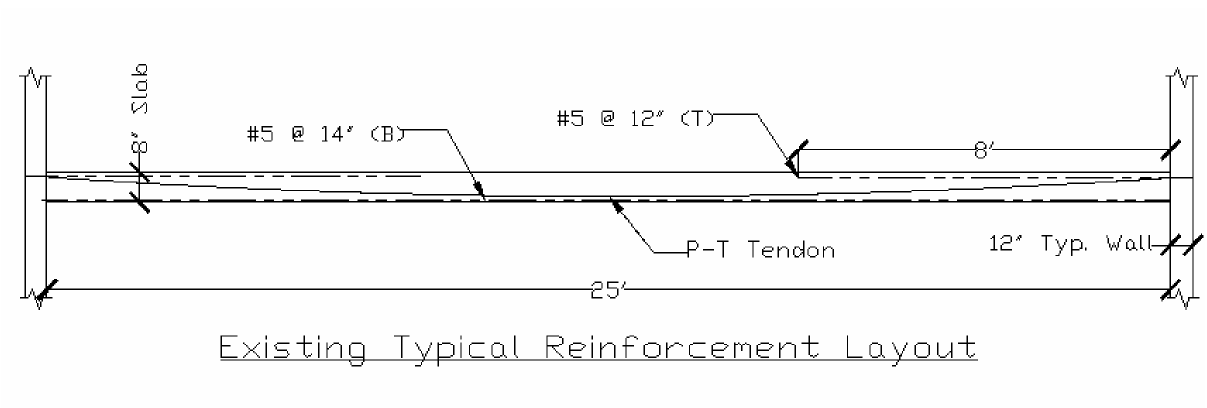
For Lmax = 26’ à Wu = 8(23.94 ft-k)/26^2 = **283 psf > 277 psf** Good

Check minimum reinforcement:

As,min = 0.0015 * 8 * 12 = **0.144 in2/ft < 5 @ 18” = 0.2 in2/ft** Good

Reinforcement was found adequate. The regular reinforcement was found necessary for strength requirements.

Reinforcement Layout Cross Section:



Summary:

Total Weight	100 psf
Reinforcement	1.61 psf + Post Tensioning tendons both directions
Advantages	Code does not limit the depth of the slab Slab depth is only 8” even with a maximum span length of 26’ Rebar placement needed only in one direction Formwork is modular and reusable
Disadvantages	Equipment intensive Post-tensioning is expensive Requires specialized knowledge to fabricate, assemble and install

Study of Alternate Floor Systems

The analysis of the existing floor system was performed and found adequate. This shows that live load of 60 psf and dead loads of 125 psf and all other assumptions made previously about the system span and supports are adequate for analysis and design. The rest of the report will concentrate on the design requirements for alternate floor systems of this multi-story residential building under gravity loads.

The four alternate systems that I will look at are:

1. Two-way flat plate on columns
2. Two-way slab with interior beams
3. One – way hollow core pre-cast slab
4. Composite Steel Deck & Smart Beam System

1. Two Way Flat Plate on Columns

Upon examination of the existing structure, it was observed that the shear walls that make the lateral system of the building are an extension of the columns layout of the parking garage. Sections through the building show that the shear walls (12" wide) have a bearing effective area (labeled BE) at each end of 36" wide. This is the same location and dimensions as the layout of the columns frame forming the parking garage structure: 12" x 36" columns @ 15' c/c north to south.

The spacing in the east-west direction is the same as that of the existing span dimensions for the typical apartment layout outlined earlier, typically 15'-25'. Since span length/width ratio < 2, I wanted to analyze what the effect will be on the structural slab by having the 12" x 36" columns with a two way flat plate. The two-way action could possible eliminate the need for specialized post-tensioning. The limiting factor in this or any other non presstressed system is that the ACI code limits the slab thickness. For a two-way flat plate the slab thickness is restricted by the ACI code (9.5.3.2) to be Ln/33 without drop panels.

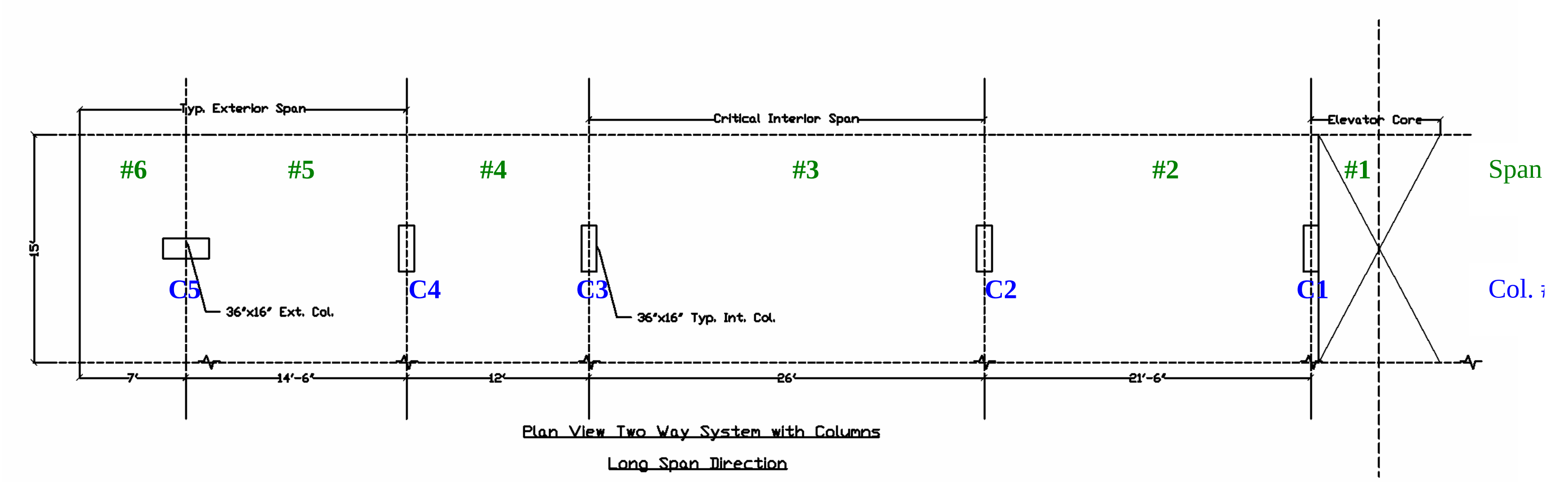
Lmax = 26' à Ln-max = 25' à hmin = (25'*12)/33 = **9" > 8" current slab thickness.**

Analysis for this system was done both by hand and using the PCI program ADOSS. Since this is the first time I learned and used ADOSS, I preformed the hand calculations which can be found in Appendix A for both the exterior and critical interior spans in the long direction to verify the programs output. Calculations include:

- Minimum area of reinforcement
- Strength required reinforcement including unbalanced moments
- Shear checks around effective column perimeter.

Results from hand calculations differ slightly from those obtained by the program. This difference results in the software's ability to redistribute the moments more accurately across spans because it uses the Equivalent Frame Method, adjusting for joint stiffness. My hand calculations where limited to the ACI Table 8.3.3 coefficients as an estimate and the Direct Design Method for analysis. Also, the software calculates worst moment envelopes of 75% partial live loading on alternate spans. Other than the moments and shears used for the design being slightly different, the required reinforcement does not vary greatly. Analysis in ADOSS was also performed on the short span direction. They are not included in the report because they were not found to be critical. In this direction reinforcement was governed by minimum required reinforcement for temperature and shrinkage = 0.0018Ag = 0.19 in2/ft for a 9" slab or #4 @ 12. Because of the lower strength requirements in this direction, this layer was specified to be the inner layer for strength calculations à d=7.0"

Layout:



Plan View Two Way System with Columns

Long Span Direction

Results ADOSS: Input – LONG SPAN

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UNITS                   U.S. in-lb
CODE                    ACI 318-89

SLAB SYSTEM             FLAT PLATE
FRAME LOCATION          INTERIOR

DESIGN METHOD           STRENGTH DESIGN
MOMENTS AND SHEARS      NOT PROPORTIONED

NUMBER OF SPANS   6

CONCRETE FACTORS         SLABS          BEAMS         COLUMNS
  DENSITY(pcf  )         150.0          150.0          150.0
  TYPE                 NORMAL WGT     NORMAL WGT     NORMAL WGT
  f'c     (ksi)            4.5            4.5            4.5
  fct     (psi)          449.4          449.4          449.4
  fr      (psi)          503.1          503.1          503.1


REINFORCEMENT DETAILS: NON-PRESTRESSED
  YIELD STRENGTH Fy  =  60.00 ksi
  DISTANCE TO RF CENTER FROM TENSION FACE:
       AT SLAB TOP    =   1.00 in  OUTER LAYER
       AT SLAB BOTTOM =   1.00 in  OUTER LAYER
  MINIMUM FLEXURAL BAR SIZE:
       AT SLAB TOP    =  # 4
       AT SLAB BOTTOM =  # 4
  MINIMUM SPACING:
       IN SLAB =   6.00 in
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SPAN/LOADING DATA

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SPAN NUMBER	LENGTH L1 (ft)	Tslab (in)	WIDTH L2*** LEFT (ft)	RIGHT (ft)	SLAB SYSTEM	DESIGN STRIP (ft)	COLUMN STRIP** (ft)	UNIFORM LOADS S. DL (psf)	LIVE (psf)
1*	.5	9.0	7.5	7.5	1	15.0	.0	.0	.0
2	21.5	9.0	7.5	7.5	1	15.0	7.5	25.0	60.0
3	26.0	9.0	7.5	7.5	1	15.0	7.5	25.0	60.0
4	12.0	9.0	7.5	7.5	1	15.0	6.0	25.0	60.0
5	14.5	9.0	7.5	7.5	1	15.0	7.3	25.0	60.0
6*	7.0	9.0	7.5	7.5	1	15.0	.0	25.0	60.0

* -Indicates cantilever span information.
** -Strip width used for positive flexure.
***-L2 widths are 1/2 dist. to transverse column.
"E"-Indicates exterior strip.

PARTIAL LOADING DATA

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PARTIAL LOADINGS ARE NOT SPECIFIED

COLUMN/TORSIONAL DATA

****

COLUMN NUMBER	COLUMN ABOVE SLAB C1 (in)	C2 (in)	HGT (ft)	COLUMN BELOW SLAB C1 (in)	C2 (in)	HGT (ft)	CAPITAL** EXTEN. (in)	DEPTH (in)	COLUMN STRIP* (ft)	MIDDLE STRIP* (ft)
1	12.0	36.0	10.0	12.0	36.0	10.0	.0	.0	7.5	7.5
2	12.0	36.0	10.0	12.0	36.0	10.0	.0	.0	7.5	7.5
3	12.0	36.0	10.0	12.0	36.0	10.0	.0	.0	6.0	9.0
4	12.0	36.0	10.0	12.0	36.0	10.0	.0	.0	6.0	9.0
5	36.0	16.0	10.0	36.0	16.0	10.0	.0	.0	7.3	7.8

Output:

DISTRIBUTION OF DESIGN MOMENTS AT SUPPORTS

COL NUM	CROSS SECTN	TOTAL MOMENT (ft-k)	TOTAL-VERT DIFFERENCE (ft-k) (%)	COLUMN STRIP MOMENT (ft-k) (%)	BEAM MOMENT (ft-k) (%)	MIDDLE STRIP MOMENT (ft-k) (%)
1	LEFT TOP	-.2	.0 (0)	-.2 (99)	.0 (0)	.0 (0)
	BOT	.0	.0 (0)	.0 (0)	.0 (0)	.0 (0)
	RGHT TOP	62.1	.0 (0)	61.6 (99)	.0 (0)	.4 (0)
	BOT	.0	.0 (0)	.0 (0)	.0 (0)	.0 (0)
2	LEFT TOP	-212.0	.0 (0)	-159.0 (75)	.0 (0)	-53.0 (25)
	BOT	.0	.0 (0)	.0 (0)	.0 (0)	.0 (0)
	RGHT TOP	232.1	.0 (0)	174.0 (75)	.0 (0)	58.0 (25)
	BOT	.0	.0 (0)	.0 (0)	.0 (0)	.0 (0)
3	LEFT TOP	-184.9	.0 (0)	-138.7 (75)	.0 (0)	-46.2 (25)
	BOT	.0	.0 (0)	.0 (0)	.0 (0)	.0 (0)
	RGHT TOP	133.4	.0 (0)	100.1 (75)	.0 (0)	33.4 (25)
	BOT	.0	.0 (0)	.0 (0)	.0 (0)	.0 (0)
4	LEFT TOP	-31.1	.0 (0)	-23.3 (75)	.0 (0)	-7.8 (25)
	BOT	.0	.0 (0)	.0 (0)	.0 (0)	.0 (0)
	RGHT TOP	40.4	.0 (0)	30.3 (75)	.0 (0)	10.1 (25)
	BOT	.0	.0 (0)	.0 (0)	.0 (0)	.0 (0)
5	LEFT TOP	-50.1	.0 (0)	-48.4 (96)	.0 (0)	-1.7 (3)
	BOT	.0	.0 (0)	.0 (0)	.0 (0)	.0 (0)
	RGHT TOP	73.6	.0 (0)	71.1 (96)	.0 (0)	2.5 (3)
	BOT	.0	.0 (0)	.0 (0)	.0 (0)	.0 (0)

DISTRIBUTION OF DESIGN MOMENTS IN SPANS

SPAN NUM	CROSS SECTN		TOTAL MOMENT (ft-k)	TOTAL-VERT DIFFERENCE (ft-k) (%)	COLUMN STRIP MOMENT (ft-k) (%)	BEAM MOMENT (ft-k) (%)	MIDDLE STRIP MOMENT (ft-k) (%)
2	9.14	TOP	.0	.0 (0)	.0 (0)	.0 (0)	.0 (0)
		BOT	102.4	.0 (0)	61.4 (60)	.0 (0)	41.0 (39)
3	13.65	TOP	.0	.0 (0)	.0 (0)	.0 (0)	.0 (0)
		BOT	137.8	.0 (0)	82.7 (60)	.0 (0)	55.1 (40)
4	7.50	TOP	-11.5	.0 (0)	-6.9 (60)	.0 (0)	-4.6 (39)
		BOT	10.2	.0 (0)	6.1 (60)	.0 (0)	4.1 (39)
5	6.16	TOP	.0	.0 (0)	.0 (0)	.0 (0)	.0 (0)
		BOT	44.3	.0 (0)	26.6 (60)	.0 (0)	17.7 (40)

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N E G A T I V E   R E I N F O R C E M E N T

COLUMN NUMBER	PATT NO.	LOCATION @COL FACE	TOTAL DESIGN (ft-k)	COLUMN STRIP AREA (sq.in)	COLUMN STRIP WIDTH (ft)	MIDDLE STRIP AREA (sq.in)	MIDDLE STRIP WIDTH (ft)
1	3	|| R	62.1	1.75	7.5	1.46	7.5
2	4	|| R	232.1	5.12	7.5	1.64	7.5
3	4	L ||	-184.9	4.08	6.0	1.75	9.0
4	1	|| R	40.4	1.17	6.0	1.75	9.0
5	4	|| R	73.6	2.02	7.3	1.51	7.8

N E G A T I V E   R E I N F O R C E M E N T

COLUMN NUMBER	COLUMN STRIP LONG BARS NO	SIZE	LEFT (ft)	RIGHT (ft)	COLUMN STRIP SHORT BARS NO	SIZE	LEFT (ft)	RIGHT (ft)	MIDDLE STRIP LONG BARS NO	SIZE	LEFT (ft)	RIGHT (ft)
1	5	# 4	.50	6.65	4	# 4	.50	4.60	8	# 4	.50	5.01
2	6	# 6	8.01	8.06	6	# 6	5.50	5.50	9	# 4	8.01	8.06
3	5	# 6	8.00	8.00	5	# 6	5.50	5.50	9	# 4	6.76	6.00
4	3	# 4	4.25	4.25	3	# 4	3.00	3.00	9	# 4	3.78	3.25
5	5	# 4	5.25	7.00	5	# 4	4.00	7.00	8	# 4	4.41	7.00

P O S I T I V E   R E I N F O R C E M E N T

SPAN NUMBER	PATT NO.	LOCATION FROM LEFT (ft)	TOTAL DESIGN (ft-k)	COLUMN STRIP AREA (sq.in)	COLUMN STRIP WIDTH (ft)	MIDDLE STRIP AREA (sq.in)	MIDDLE STRIP WIDTH (ft)
2	4	9.1	102.4	1.74	7.5	1.46	7.5
3	4	13.6	137.8	2.36	7.5	1.56	7.5
4**	3	7.5	10.2	1.17	6.0	1.75	9.0
5	2	6.2	44.3	1.41	7.3	1.51	7.8

P O S I T I V E   R E I N F O R C E M E N T

SPAN NUMBER	COLUMN STRIP LONG BARS NO	SIZE	LENGTH (ft)	COLUMN STRIP SHORT BARS NO	SIZE	LENGTH (ft)	MIDDLE STRIP LONG BARS NO	SIZE	LENGTH (ft)	MIDDLE STRIP SHORT BARS NO	SIZE	LENGTH (ft)
2	5	# 4	21.25	4	# 4	18.81	4	# 4	21.75	4	# 4	18.27
3	6	# 4	25.50	6	# 4	19.50	4	# 4	26.50	4	# 4	18.20
4**	3	# 4	11.50	3	# 4	9.00	5	# 4	12.50	4	# 4	8.40
5	4	# 4	13.25	3	# 4	11.69	4	# 4	13.75	4	# 4	11.32

A D D I T I O N A L   I N F O R M A T I O N   A T   S U P P O R T S

COLUMN NUMBER	REINF. SUMMARY W/O U. MOMENT REQ'D (sq.in)	PROV'D (sq.in)	ADD'L R/F REQ'D DUE TO UNBALANCED (U.) MOMENT TRANSFER: MAX.U. MOMENT (ft-k)	GAMMA -f	FLEXURAL TRANSFER (ft-k)	PATT NO.	CRITICAL SLABW (ft)	SECTION - AREA - (sq.in)	R/F	
1	3.20	3.40	80.3	.71	57.3	3	5.3	1.63	3	# 4
2	6.76	7.08	42.5	.69	29.3	2	5.3	.82	0	# 6
3	5.83	6.20	-63.6	.69	-43.8	2	5.3	1.24	0	# 6
4	2.92	3.00	23.1	.69	15.9	2	5.3	.44	0	# 4
5	3.53	3.60	31.2	.53	16.4	3	3.6	.46	0	# 4

Deflections

SPAN NUMBER	DEAD LOAD Ieff. (in^4)	COLUMN STRIP DEFLECTION DUE TO: DEAD (in)	LIVE (in)	TOTAL (in)	MIDDLE STRIP DEFLECTION DUE TO: DEAD (in)	LIVE (in)	TOTAL (in)
1	10935.	-.004	-.002	-.005	-.004	-.002	-.005
2	10457.	.083	.047	.130	.039	.021	.060
3	10094.	.164	.125	.289	.092	.066	.158
4	10935.	-.004	-.002	-.006	-.009	-.004	-.013
5	10935.	.017	.007	.024	.008	.003	.011
6	10935.	.048	.021	.069	.003	.001	.004

Summary Results: SHORT SPAN

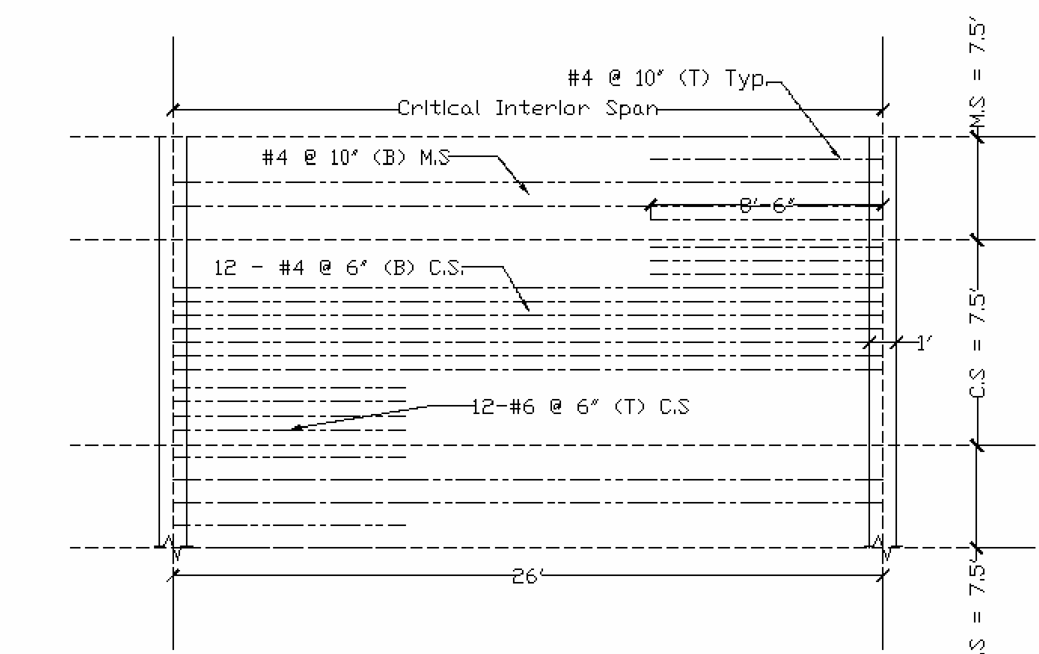
N E G A T I V E   R E I N F O R C E M E N T

COLUMN NUMBER	COLUMN STRIP LONG BARS NO	SIZE	LEFT (ft)	RIGHT (ft)	COLUMN STRIP SHORT BARS NO	SIZE	LEFT (ft)	RIGHT (ft)	MIDDLE STRIP LONG BARS NO	SIZE	LEFT (ft)	RIGHT (ft)
1	4	# 4	1.00	4.75	4	# 4	1.00	3.50	15	# 4	1.00	3.75
2	5	# 4	5.25	5.25	4	# 4	4.00	4.00	15	# 4	4.53	4.53
3	5	# 4	5.10	5.10	4	# 4	3.90	3.90	15	# 4	4.50	4.50
4	4	# 4	5.10	1.50	4	# 4	3.90	1.50	15	# 4	4.14	1.50

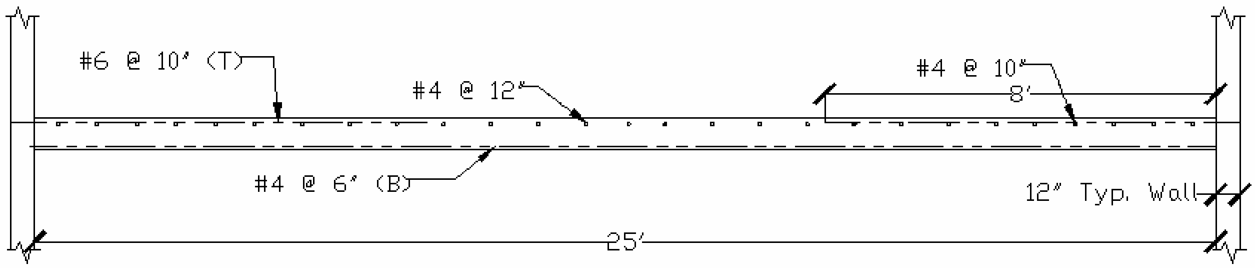
P O S I T I V E   R E I N F O R C E M E N T

SPAN NUMBER	COLUMN STRIP LONG BARS NO	SIZE	LENGTH (ft)	COLUMN STRIP SHORT BARS NO	SIZE	LENGTH (ft)	MIDDLE STRIP LONG BARS NO	SIZE	LENGTH (ft)	MIDDLE STRIP SHORT BARS NO	SIZE	LENGTH (ft)
2	4	# 4	14.25	4	# 4	12.63	8	# 4	14.75	7	# 4	12.25
3	4	# 4	14.50	4	# 4	11.25	8	# 4	15.50	7	# 4	10.50
4	4	# 4	13.75	4	# 4	12.13	8	# 4	14.25	7	# 4	11.75

Summary Critical Span Typ. Reinforcement Layout Long-Span:



Existing Typical Reinforcement Layout



Column Strip Typical Reinforcement Layout

Two – Way Flat Plate Summary:

Total Weight	112.5 psf
Reinforcement	1.35 psf long span + 0.97 psf short span = 2.32 psf
Deflection	0.3" at Lmax = 26' à l/1040
Advantages	Formwork is modular and reusable Doesn't require extensive expertise – known practice Small deflections Fast Construction
Disadvantages	Increased amount of rebar Rebar on both directions can be a labor intensive and time consuming job if it is not modularized properly between C.S, M.S, short span and long span directions. Need to carefully consider punching shear around columns

A critical consideration for this system is to verify that the columns can support the load. In the first apartment floor, the factored load that the column with the largest tributary area will experience is:

A = 15' x 24' = 360 ft2
Wl = 13*0.6*60 psf = 468 psf
Wd = 13*125 psf = 1625 psf
Wu = 1.2(468) + 1.6(1625) = 3161 psf
P = 3161psf*(360ft2) = **1138 kips**
Mu = **21.2 ft –k** à Worst moment obtained from ADOSS output of the moment distribution at the joints.

If the column proves to be ineffective, a higher strength concrete can be used. Alternatively, a steel column encased in concrete for fire-proofing and finishes can be evaluated.

2. Two Way Slab with Longitudinal and/or Transverse Beam

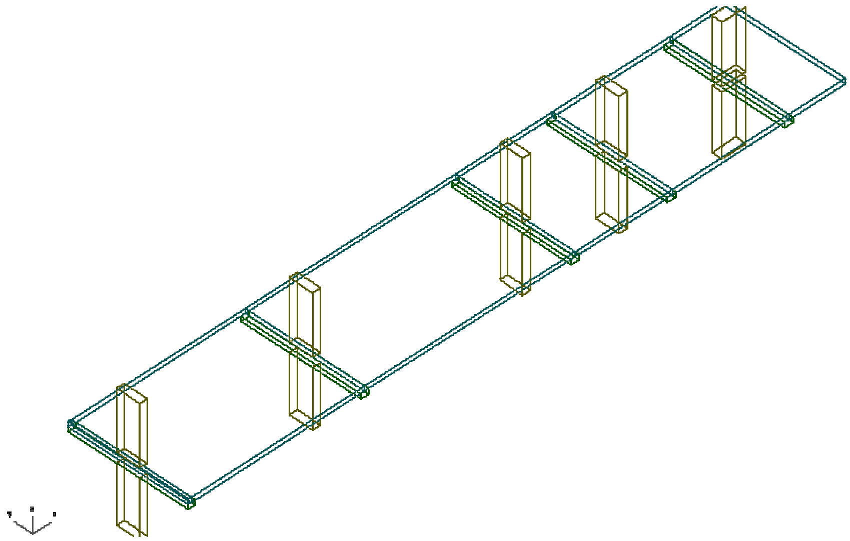
Several designs were performed in ADOSS to compare the different required slab thickness for the critical span, Lmax=26' by comparing multiple options involving the use of transverse and longitudinal beams. The longitudinal beams considered are shallow beams with widths equal to that of the columns, 36". The transverse beams are located from columns to column in the north to south direction, spanning the same direction and location as the current shear wall.

A summary of the different options is listed below:

Option	Transverse Beam	Longitudinal Beam	Req. Slab Depth	Concrete (cub. yards)	Foamwork (sq.ft)
1	None	36" x 8.5"	8.8"	32.4	1462
2	12" x 9"	36" x 9"	8.8"	33.1	1469
3	12 x 12"	36 x 9"	8.7"	33.9	1478
4	12" x 16"	36" x 9"	8.3"	30.9	1478
5	12" x 18"	36" x 9"	8"	30.9	1478
6	**12" x 16"**	**None**	**8.5"**	**32.0**	**1223**

Option # 6 is the one that I will analyze further. It doesn't have the shallowest slab depth but it has the least deflections, 0.3" in 3rd span compared to 0.47" for Option #5, and it also uses the least amount of formwork. It is easier to form, construct and reuse. The transverse beams are located were the shear walls of the existing system are, therefore there is no depth restrictions other than formwork and concrete construction considerations. The system also proves to be the least use in concrete and least use in formwork within its class.

Layout:



Results:

N E G A T I V E   R E I N F O R C E M E N T

COLUMN NUMBER	PATT NO.	LOCATION @COL FACE	TOTAL DESIGN (ft-k)	COLUMN STRIP AREA (sq.in)	COLUMN STRIP WIDTH (ft)	MIDDLE STRIP AREA (sq.in)	MIDDLE STRIP WIDTH (ft)
1	4	|| R	86.1	2.55	7.5	1.38	7.5
2	4	|| R	222.7	5.27	7.5	1.68	7.5
3	4	L ||	-190.0	4.52	6.0	1.65	9.0
4	1	|| R	45.0	1.10	6.0	1.65	9.0
5	4	|| R	71.4	2.08	7.3	1.42	7.8

P O S I T I V E   R E I N F O R C E M E N T

SPAN NUMBER	PATT NO.	LOCATION FROM LEFT (ft)	TOTAL DESIGN (ft-k)	COLUMN STRIP AREA (sq.in)	COLUMN STRIP WIDTH (ft)	MIDDLE STRIP AREA (sq.in)	MIDDLE STRIP WIDTH (ft)
2	4	9.1	90.7	1.64	7.5	1.38	7.5
3	4	13.6	129.5	2.37	7.5	1.56	7.5
4**	3	7.5	12.6	1.10	6.0	1.65	9.0
5	4	6.9	39.6	1.33	7.3	1.42	7.8

NEGATIVE REINFORCEMENT

COLUMN NUMBER	COLUMN STRIP LONG BARS NO	SIZE	LEFT (ft)	RIGHT (ft)	COLUMN STRIP SHORT BARS NO	SIZE	LEFT (ft)	RIGHT (ft)	MIDDLE STRIP LONG BARS NO	SIZE	LEFT (ft)	RIGHT (ft)
1	7	# 4	.50	6.65	6	# 4	.50	4.60	7	# 4	.50	5.01
2	6	# 6	8.01	8.06	6	# 6	5.50	5.50	9	# 4	8.01	8.06
3	4	# 7	8.00	8.00	4	# 7	5.50	5.50	9	# 4	6.76	6.00
4	3	# 4	4.25	4.25	3	# 4	3.00	3.00	9	# 4	3.25	3.68
5	6	# 4	5.25	7.00	5	# 4	4.00	7.00	7	# 4	4.41	7.00

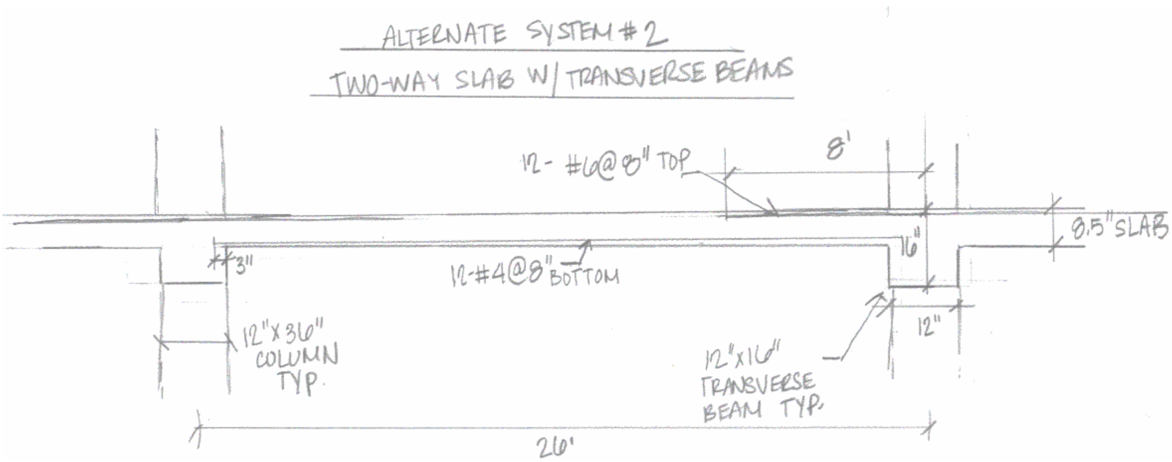
POSITIVE REINFORCEMENT

SPAN NUMBER	COLUMN STRIP LONG BARS NO	SIZE	LENGTH (ft)	COLUMN STRIP SHORT BARS NO	SIZE	LENGTH (ft)	MIDDLE STRIP LONG BARS NO	SIZE	LENGTH (ft)	MIDDLE STRIP SHORT BARS NO	SIZE	LENGTH (ft)
2	5	# 4	21.25	4	# 4	18.81	4	# 4	21.75	3	# 4	18.27
3	6	# 4	25.50	6	# 4	19.50	4	# 4	26.50	4	# 4	18.20
4**	3	# 4	11.50	3	# 4	9.00	5	# 4	12.50	4	# 4	8.40
5	4	# 4	13.25	3	# 4	11.69	4	# 4	13.75	3	# 4	11.32

<u>Deflection:</u>

SPAN NUMBER	DEAD LOAD Ieff. (in^4)	COLUMN STRIP DEFLECTION DUE TO: DEAD (in)	LIVE (in)	TOTAL (in)	MIDDLE STRIP DEFLECTION DUE TO: DEAD (in)	LIVE (in)	TOTAL (in)
1	9212.	-.003	-.001	-.004	-.003	-.001	-.004
2	8849.	.089	.052	.141	.039	.022	.062
3	7945.	.194	.142	.336	.105	.074	.179
4	9212.	-.001	-.001	-.001	-.007	-.003	-.010
5	9212.	.018	.008	.026	.008	.003	.011
6	9212.	.054	.025	.079	.004	.002	.006

<u>Typ. Reinforcement Layout:</u>



3. One-way Hollow Core Pre-Cast Slabs

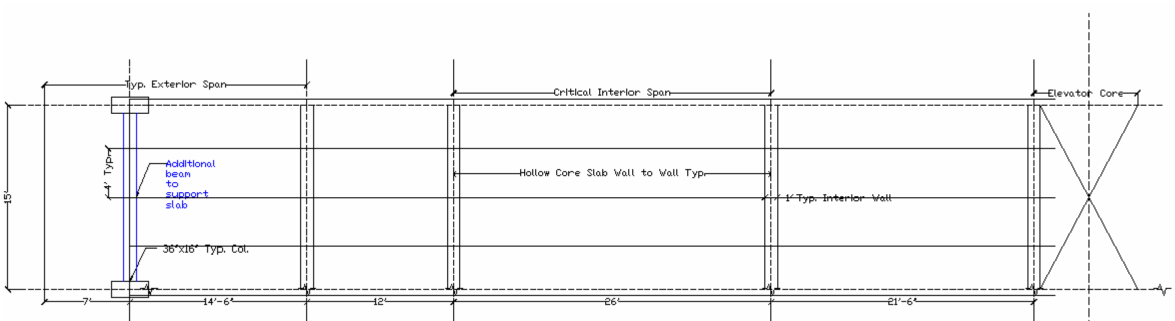
Analysis of the one-way hollow core system is based on Nitterhouse Product Catalog Span Tables found online at www.nitterhouse.com . The frame work for the analysis is based on the one way action of the existing structural slab system. As a result the same spacing and layout was used for analysis with out any modifications. The hollow core slabs would span east to west from shear wall to shear wall. There are two strength considerations for this layout:

I. Maximum span, 26’: occurs under live loads of 60psf

II. Maximum load: for corridor, public spaces and terraces live load = 100 psf. This loading is experiences over a shorter span of 21.5’

Both combinations will be looked at for strength requirements when selecting spans from the tales.

Layout:



Plan View Hollow Core Slab Layout

Selection:

Case I: Maximum Span
Superimposed Loads = 25 psf dead + 20 psf partitions + 40 psf live= 85 psf
Lmax = 26'
From Nitterhouse span table: 8" Hollow Core Slab U.L J917 (no topping)

Will require **6 – ½**" diameter strands à **Wallow = 96 psf @ 26' span > 85 psf**   Good

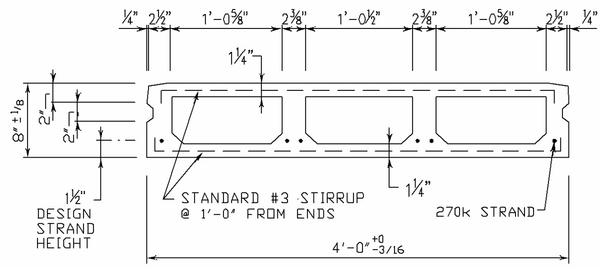
Case II: Maximum Loading
Superimposed Loads = 25 psf dead + 20 psf partition + 100 psf live (terrace & corridors) = 145 psf
Lmax = 21.5'
For the same slab @ 22' span à **Wallow = 174 psf > 145 psf**   Good

Prestressed Concrete
8"x4' SpanDeck–U.L.–J917
(NO TOPPING)

PHYSICAL PROPERTIES

Precast

A	= 180 in.^{2}	S_b	= 397 in.^{3}
I	= 1543 in.^{4}	S_t	= 375 in.^{3}
Y_b	= 3.89 in.	Wt.	= 230 PLF
Y_t	= 4.11 in.	Wt.	= 57.5 PSF
e	= 2.39 in.		



8" SPANDECK CROSS SECTION

UL FIRE RATED J917

DESIGN DATA

1. Precast Strength @ 28 days = 5000 PSI.
2. Precast Density = 150 PCF.
3. Strand = 1/2"ø, 270 K Lo-Relaxation.
4. Strand Height = 1.50 in.
5. Ultimate moment capacities (when fully developed)...
   4 – 1/2"ø, 270K = 74.3'K
   6 – 1/2"ø, 270K = 105.6'K
6. Maximum bottom tensile stress is $6\sqrt{f'c}$ =424 PSI.
7. All superimposed load is treated as live load in the strength analysis of flexure and shear.
8. Flexural strength capacity is based on stress/strain strand relationships.
9. All values in this table are based on ultimate strength and are not governed by service stress.
10. Shear values are the maximum allowable before shear reinforcement is required.
11. Deflection limits were not considered when determing allowable loads in this table.

8" SPANDECK W/O TOPPING		ALLOWABLE SUPERIMPOSED LOAD (PSF)																					
STRAND PATTERN		SPAN (FEET)																					
		10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32
Flexure	4 – 1/2"ø	610	550	499	457	399	341	294	255	222	195	171	151	133	117	103	92	82	72	66	56	49	43	
Shear	4 – 1/2"ø	441	393	354	321	294	270	249	231	215	201	188	177	160	145	132	120	110	101	95	90	82	75	
Flexure	6 – 1/2"ø	885	800	726	667	586	509	437	382	334	296	263	234	208	187	168	151	136	122	111	100	90	81	73
Shear	6 – 1/2"ø	459	411	370	337	308	283	262	243	226	211	197	185	174	164	155	147	139	131	120	111	102	94	87

NITTERHOUSE
CONCRETE PRODUCTS

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.

2655 MOLLY PITCHER HWY. SOUTH, BOX N
CHAMBERSBURG, PA 17201-0813
717-267-4505 • FAX: 717-267-4518

REVISED 12/93

Other design considerations that need to be accounted for when choosing this system and that are not considered in the span tables are:

- § Deflections
- § Fire Rating

§ Noise Control

§ Vibrations

The following deflection and fire rating considerations and design are based on Reference 2, Manual for the Design of Hollow Core Slabs 2nd Edition. Tables referenced are included in Appendix D.

<u>Deflections:</u>

Wdead = 25 psf	Ec = 4300 ksi
Wlive = 60 psf	Wt = 230 psf
Lmax = 26'	e = 2.39"
dp = 8" – 1.5" = 6.5"	I = 1543 in3

Initial Camber:

ApsFpu = 0.153(270ksi) = 41.3 k / strand
Initial Stress = 70% fpu
Initial Losses = 5%
Eci = 3250 ksi
Po = 0.95(0.7)(6)(41.3) = 164.8 kips
Camber = P(e)L^2/(8EI) – 5wL^4/(384EI) = 0.9557 – 0.47 = 0.48"

Erections Camber: (Factors from Table 1, Appendix D)
= 0.9557*1.80 – 0.47*1.85 = 0.85"

Final Camber: (Factors from Table 1, Appendix D without topping)
= 0.9557*2.45 – 0.47*2.7 = 1.1"

Superimposed Dead Load Instant Deflection
= 5(Wd)(L^4)(1728)/(384*EI) = 0.149"
w/ creep = 0.149*(3.0 creep factor) = 0.45"

Superimposed Live Load Instant Deflection
= 5(Wl)(L^4)(1728)/(384*EI) = 0.37"

For slabs with non-strutural elements attached to the slab, Chapter 9 of the ACI Code provisons: deflection < L/480 = 26'*12/480 = 0.65"

Change in camber: 1.1"-0.85" =	0.25"	
Sustained dead Loads:	-0.45"	
Instantaneous live Load:	<u>-0.37</u>	
	<u>-0.57" < 0.65"</u>	Good

<u>Fire Rating</u>: Table 2, Appendix D

Effective Thickness = 180in2/ (4'*12) = 3.75" à 1.25 hour rating Not Enough
For the 2 hour rating required à Effective thickness = 5"
Areq = 5"(4'*12) = 240in2 à **need a 10" slab**

Design Table 3 (Appendix D) allows for a 2 hour fire rating in an 8" slab if ¼" sprayed mineral fiber or vermiculite cementious material is provided. However, this would be expensive, time consuming and would live a rough finish that is not desired for luxury apartments.

There is one more issue to be considered when choosing this system. The floor is not at straight angles around the entire perimeter of every floor, but at 30 – 60 degree angles from each other. This was deliberately done to maximize the glass/viewing towards the ocean. This means that the planks around this area will have to be made with special dimensions. This can considerably affect the price.

<u>4. Composite Steel Deck and Smartbeam System</u>

Picking a steel system is the hardest selection of all because of the usually high ceiling to floor heights that they required as compared to concrete. Aside from that steel does provide some advantages in three main aspects that affect the design of this building considerably:

I. **Weight**: From previous analysis of the lateral system in Technical Assignment #1, it was clear that weight is an important aspect of the loads experienced by the building especially since earthquake loading was the controlling case for strength design. If we reduce the weight of the structure, it will help our lateral system greatly allowing for less shear wall and more open space.
II. **Open Space**: The effectiveness of the steel frame is in the span. Steel is more efficient in bending in comparison to shear. As a result, it performs better on longer spans where bending stresses are larger than shear stresses. Because of this we can have larger spans, providing more space without columns or interruptions. This is very valuable in multi-use, residential and commercial space.
III. **Modularization**: To optimize a steel system thought a reduction in cost and erection time, a modular structure is proffered. If my ceiling to height ratio must

increase using a steel floor system, I must make this system pay back by decreasing either the cost or constructions time, or both is possible. Therefore, maintaining my current framing layout was not a possibility. The spans of the current system are too random and scattered (14.5’, 12’, 26’, 21.5’) for a feasible steel constructions. Therefore, I modified the layout. By removing the second row of concrete columns between the 14.5’ and 12’ spans from previous design I was left with a more modular structure that consists of two 26’ spans and one 22’ span. This layout works out very well since the shorter span, 22’, is that span that experiences the larger loading (100psf corridor and terrace), evening out the design.

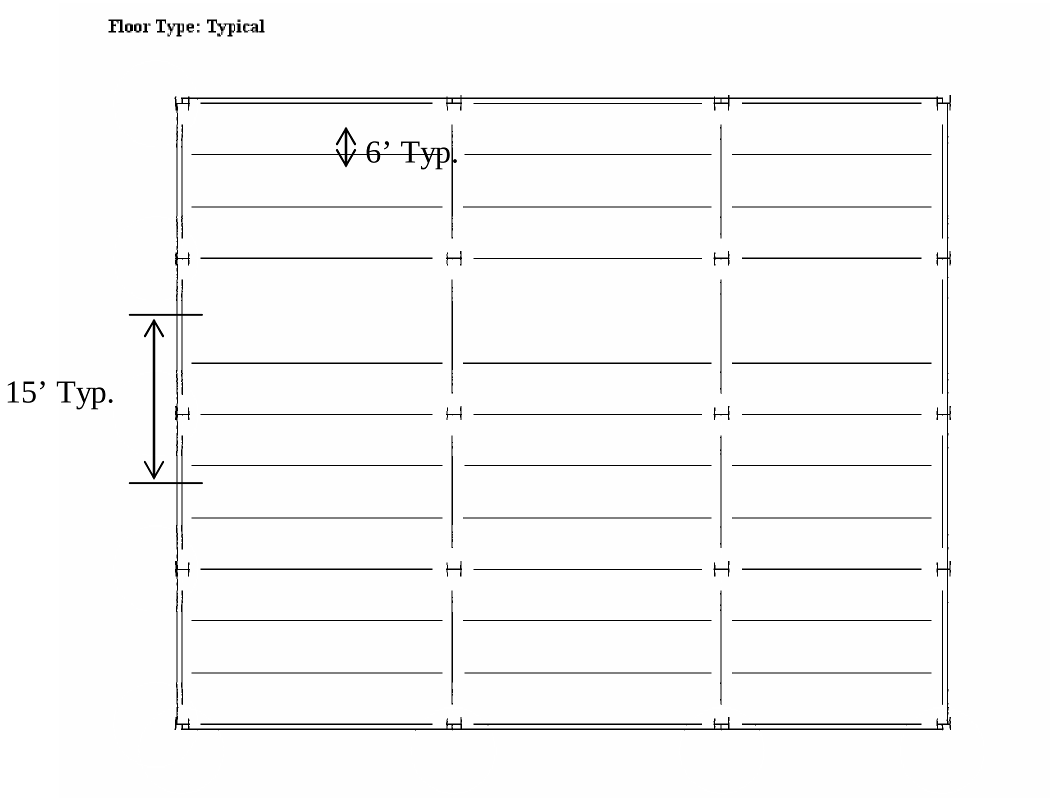
Finally, to minimize the ceiling to floor height I decided to use castellated steel beam or “smart beams”. Smart beams are beams that have holes perforated in the web. The advantages to this are multiple:

§ Reduce weight

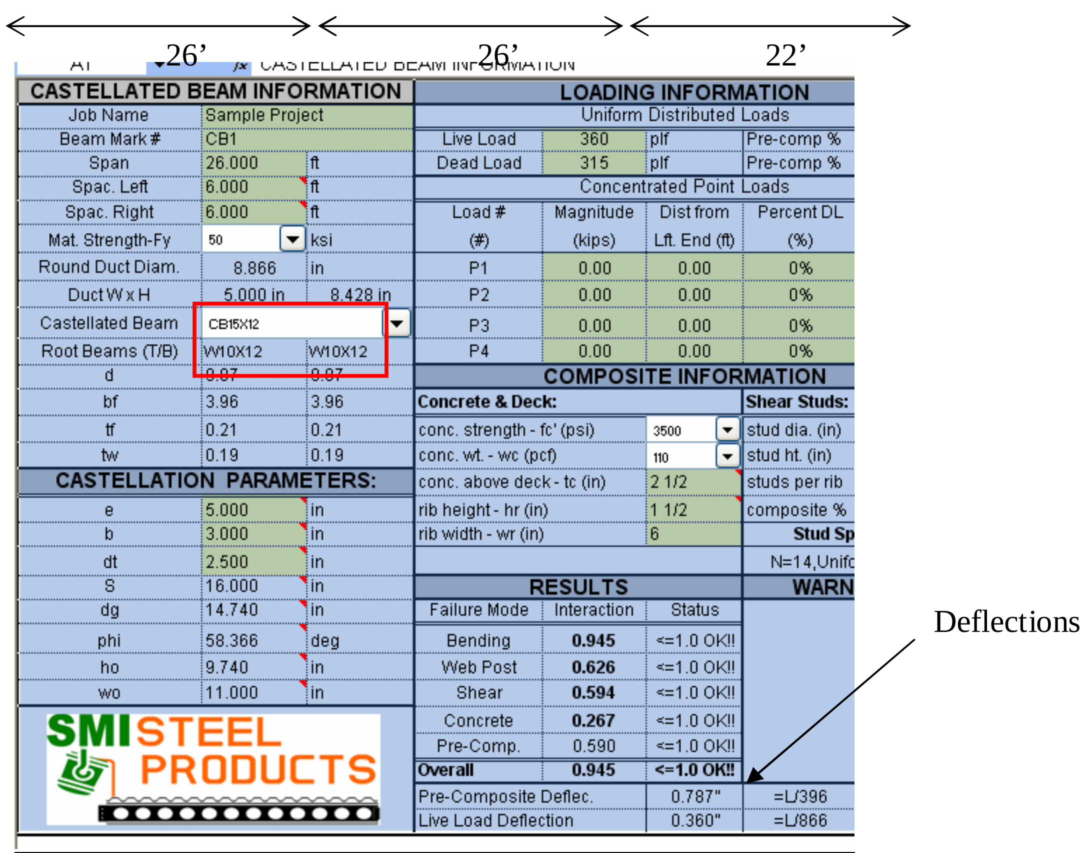
§ Allow for duct work to pass thought the beams minimizing ceiling heights

§ Increased moment of inertia to weight ratio and increased stiffness

I used to programs to design my apartment floor system. I used the program provided by SMI Steel Products, a big distributor of the product and I also modeled the system in RAM. Both programs resulted in the same results.



<u>Results:</u>



CASTELLATED BEAM INFORMATION		
Job Name	Sample Project	
Beam Mark #	CB1	
Span	26.000	ft
Spac. Left	6.000	ft
Spac. Right	6.000	ft
Mat. Strength-Fy	50	ksi
Round Duct Diam.	8.866	in
Duct W x H	5.000 in	8.428 in
Castellated Beam	CB15X12	
Root Beams (T/B)	W10X12	W10X12
d	9.87	9.87
bf	3.96	3.96
tf	0.21	0.21
tw	0.19	0.19

CASTELLATION PARAMETERS:		
e	5.000	in
b	3.000	in
dt	2.500	in
S	16.000	in
dg	14.740	in
phi	58.366	deg
ho	9.740	in
wo	11.000	in

SMI STEEL PRODUCTS

LOADING INFORMATION			
Uniform Distributed Loads			
Live Load	360	plf	Pre-comp %
Dead Load	315	plf	Pre-comp %
Concentrated Point Loads			
Load # (#)	Magnitude (kips)	Dist from Lft. End (ft)	Percent DL (%)
P1	0.00	0.00	0%
P2	0.00	0.00	0%
P3	0.00	0.00	0%
P4	0.00	0.00	0%

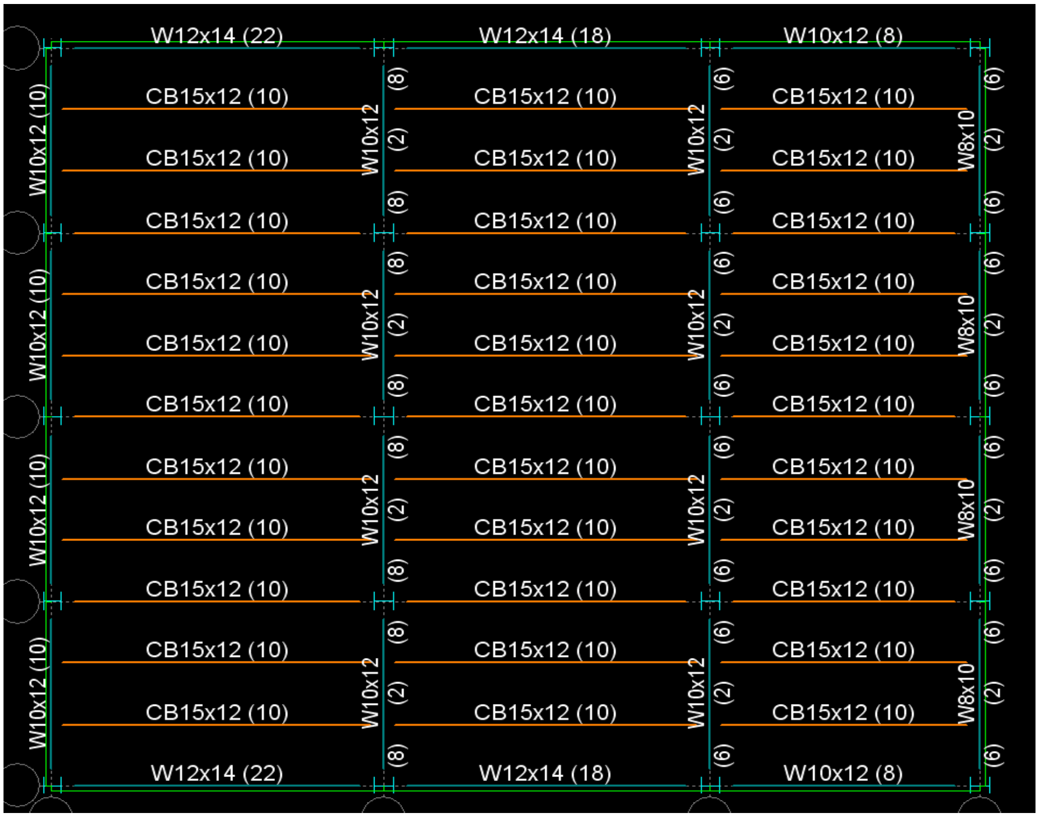
COMPOSITE INFORMATION		
Concrete & Deck:		**Shear Studs:**
conc. strength - fc' (psi)	3500	stud dia. (in)
conc. wt. - wc (pcf)	110	stud ht. (in)
conc. above deck - tc (in)	2 1/2	studs per rib
rib height - hr (in)	1 1/2	composite %
rib width - wr (in)	6	**Stud Sp**
		N=14,Unifo

RESULTS		
Failure Mode	Interaction	Status
Bending	**0.945**	<=1.0 OK!!
Web Post	**0.626**	<=1.0 OK!!
Shear	**0.594**	<=1.0 OK!!
Concrete	**0.267**	<=1.0 OK!!
Pre-Comp.	0.590	<=1.0 OK!!
Overall	**0.945**	**<=1.0 OK!!**

WARN

Pre-Composite Deflec.	0.787"	=L/396
Live Load Deflection	0.360"	=L/866

RAM:



The overall depth for this system would be depth of CB 15X12 = 14.75” + 1.5” deck + 2.5” concrete topping = 18.75”. This is considerably higher than any other other proposed concrete systems. An advantage is that the castellated beam allows for 5” x 8.5” opening for ducts, taking away room needed in the ceiling to floor height.

Finally, deflection results obtained for the manufacturer are larger than those calculated for the concrete systems, 0.8” in the largest Lmax span = 26’ à l/396

Summary Table

Option	**Overall Depth**	**Weight (psf)**	**Rebar 1 à 4**	**Material Cost 1 à 5**	**Construction**	**Deflections (in)**	**Availability**
Existing P-T slab	**8”**	100	**1**	4	Slow Machine Intensive	<<	Ready
Two-way Plate on Columns	9”	112.5	2	**1**	Modular Foamwork Labor Intesive (rebar placement)	0.448”	Ready
Two-way slab on Beams	8.5”	102.5	3	5	Requires more formwork than other options Labor Intesive (rebar placement) Beams require extra detailing	0.336”	Ready
One-way Hollow Slab	10”	57.5	N/A	3	Fast Construction Time	0.57”	Limited
Steel Beams on Composite Floor	18.75”	**40.1**	N/A	**2**	Ordering and procurement of steel might be an issue Tight site location for all material handling and storage	0.78”	X

References:

1. *Manual for the Design of Hollow Core Slabs*, 2nd Edition
2. *Design Handbook for Two Way Systems*, Volume 3. PCI publication

APPENDIX A

1. Two way slab (non prestressed) short direction + long direction

Long direction:

A) minimum thickness required by code:

- Deflections $l_n = 26 \times 12 - 12'' = 300$

$h_{min}: 300/33 = 9.0''$

- shear: $W_d = \left(25 + \frac{9}{12}(150)\right) \times 1.4 = 185.5$ psf

$W_l = 1.7\,(60) = 102$ psf

$W_u = 287.5$ psf

For interior column: (26' critical)

$V_u = \left(15 \times 26 - \left(\frac{12 \times 36}{144}\right)\right) \times 287.5 = 111.26$ kips

$2(C_1 + C_2) = 2(12 + 36) = 96$ in

chart (scan) - D.H.V3 pg 73

$d = 7.5''$

modifications: $\beta_c = 36/12 = 3 > 2$

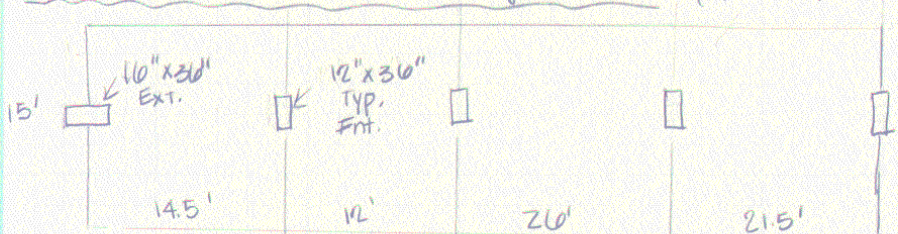
$K_1 = 1.15 \times (0.5 + 1/\beta_c) = 0.96$

new effective slab depth $= d = 9''$

$h_{min} = d + 0.75 + 0.25 = 10''$

new $W_u = 312$ psf

B.) Divide structure into design frames ($l_2 = 15'$)



l_1 =	14.5'	12'	26'	21.5'
l_n =	12.5'	11'	25'	20.5'
M_o =	91.4	70.8	365.6	250

c) Distribution of Moments

Span 1: End span

$-M_e = 0.3 \times 91.4\text{'K} = 27.42\text{'K}$
$+M_e = 0.5 \times 91.4\text{'K} = 45.7\text{'K}$
$-M_{ie} = 0.7 \times 91.4\text{'K} = 63.98\text{'K}$

Interior Spans

Span #	2	3	4
$-M = 0.65 M_o$	46.02 'K	237.64 'K	162.5 'K
$+M = 0.35 M_o$	24.78 'K	127.96 'K	87.5 'K

D) Consider whether columns are adequate for pattern loading

$\beta_a = \frac{W_d}{W_l} = \frac{150}{60} = 2.5 > 2$ pattern loading doesn't govern

E) Distribute moments to column + middle strips

- Exterior Negative:

No Edge Beam $\beta_t = 0$
100% C.S → 27.42'K

- Interior Negative moments

span#	1 ('K)	2 ('K)	3 ('K)	4 ('K)
-M	63.98	46.02	237.64	162.5
(75%) C.S	48	34.5	178.2	121.9
(25%) M.S	15.98	11.52	59.44	40.6

- Positive Moments

Span #	1 ('K)	2 ('K)	3 ('K)	4 ('K)
+M ('K)	45.7'K	24.78	127.96	87.5
C.S (60%)	27.42	14.9	76.78	52.5
M.S (40%)	18.3	9.88	51.18	35

use: 9" slab
d = 7.75

F) Design Required Reinforcement

- Exterior negative: C.S. = 15/2 = 7.5'

$$A_s = \frac{M_u}{\phi f_y j d} = \frac{27.42\text{ k}(12''/\text{FT})(1000\text{ lb/kip})}{0.9(60{,}000)(7.75)} = 0.79\text{ in}^2$$

$$a = \frac{0.79(60000)}{0.85(4500)(7.5\times 12)} = 0.138$$

$j = 1 - a/2d = 0.99 \sim 1$ assumed

Assuming #4 bars $A_s = 0.2\text{ in}^2$

$$n = \frac{0.79\text{ in}^2}{0.2\text{ in}^2/\text{bar}} = 4\text{ bars}$$

$$s = \frac{7.5\times 12}{4} = 22'' > 2(9) = 18''$$

Δ 4 # 4 bars (#4 @ 18")

Calculate unbalanced moment (req. reinforcement)

$$\gamma_f = \frac{1}{1+\frac{2}{3}\sqrt{\frac{c_1 + d/2}{c_2+d}}} = \frac{1}{1+\frac{2}{3}\sqrt{\frac{36 + 7.75/2}{16 + 7.75}}} = 0.7$$

$\gamma_f M_u = 0.7(27.42) = 19.1$ Kip

width of moment transfer section $= c_1 + 2(1.5 t_s) = 36 + 2(1.5(9)) = 63''$

A_s required $\approx 0.7 A_{s_{Mu}} = (0.7)(0.79\text{ in}^2) = 0.55\text{ in}^2$

check $a = \frac{0.55(60{,}000)}{0.85(4500)(63)} = 0.137''$

$j = 1 - 0.137/2(7.75) = 0.99 \sim 1$ assumed

$n = \frac{0.55}{0.2} = 3$ bars $\quad s = \frac{63}{3} = 21'' > 18''$

use 4 #4

$$\phi M_n = \frac{0.8(60)(7.75)(0.9)}{(12)} \quad 27.9\text{ k} > 27.4\text{ k}$$

use 5 #4 @ 18" $\quad \phi M_n = 34.87$ 'K > 27.4 'K

Negative Reinforcement @ interior support

- critical column support: 4 span # 3

$\boxed{\text{Column strip:}}$

- $M = 178.2'k$ @ support

- $A_s = \dfrac{178.2(12)(1000)}{(0.9)(60000)(0.90\times 7.75)} = 5.68 \text{ in}^2$
- check $a = \dfrac{5.68(60)}{0.85(4.5)(7.5\times 12)} = 0.99''$
- $j = 1 - a/2d = 1 - 0.99/2(7.75) = 0.94 > 0.9$ assumed

$A_s = \dfrac{178.2(12)}{0.9(60)(0.94\times 7.75)} = \underline{\underline{5.44 \text{ in}^2}}$

$a = 0.94$

$j = 0.94 = 0.94$ used

- w/ # 6 bars $A_s = 0.44$

$n = \dfrac{5.44}{0.44} = 13$ bars

- spacing $= \dfrac{7.5\times 12}{13} = 7'' \approx 6'' < 2(9) = 18''$

$\underline{\underline{\text{use } 13 \text{ \#6 bars}}}$ C.S

$\phi M_n = \dfrac{5.72(0.9)(60)(0.94)(7.75)}{12} = 187.5'k > 178.2'k$

$\boxed{\text{Middle strip}}$

- $M = 59.44'k$

- $A_s = 1.80 \text{ in}^2$ w/ $j = 0.95$

$a = 0.31 \rightarrow j = 0.98 > 0.95$ ✓

- use: # 4 bars $A_s = 0.4 \text{ in}^2$

$n = \dfrac{1.8}{0.4} = 5$ bars # 4 / strip

$\times 2$ strip $\rightarrow$ $\underline{\underline{10 \text{ \#4 bars}}}$ M.S

check shear

$w_u = 285 psf$
column = 12" x 36" $M_n = 187.5$ 'K
$d = 7.75$
$d/2 = 3.875$
$b_o + (12 + 2(3.875))2 + (16 + 2(3.875))2 = (19.75 + 23.75)2 = 87$ in

$$V_u = 285\left(23.75 \times 15 - \left(\frac{19.75 \times 23.75}{144}\right)\right) = 101.5 \text{ Kips}$$

$V_n = K_1 V_u$

From Ref. # 1 (slab9)
pg 73:
$K_1 = 2.75\left(.5 + \frac{1}{(3)}\right) = 2.2$

$\frac{V_u}{\theta} = 2.2(101) = \underline{\underline{222 \text{ psi}}}$

allowable:

$$V_c = \left(2 + \frac{4}{\beta_c}\right)\sqrt{f_c'} = 357$$

$$\beta_c = \frac{23.75}{19.75} = 1.2$$

$V_c = 4\sqrt{f_c'} = \underline{\underline{268.32}}$

$$V_c = \left(\frac{\alpha_c d}{b_o} + 2\right)\sqrt{f_c'}$$ $\alpha_c = 40$ interior columns

$$= \left(\frac{40(7.75)}{87} + 2\right)\sqrt{f_c'} = 373$$

$\frac{V_u}{\theta} = 222 \text{ psi} < 268.32$ ✓ Good

APPENDIX B

Permissible stresses at transfer @ midspan

- extreme fiber stress in compression : $0.6 f_c' = 2700$ psi
- extreme fiber stress in tension : $3\sqrt{f_c'} = 201.25$ psi
- However, concrete shored until full strength.

$d_p = 6.75''$

$l = 26'$

5% initial loss

initial stress:

$M_d = \frac{26^2 (100)(1)}{11} = 6.15\ 'k$ → due to self-weight

$\frac{M_d}{S} = \frac{6.15 (12)}{(12)(8)^2/6} = \pm 576$ psi < top → tension, bottom → compression

Prestress effect : $P_0 = 12$ k/ft = 1k/in x 12" = 12K

$A = 12 \times 8 = 96 \text{ in}^2$

$e = 3''$

$S = 128 \text{ in}^3$

Prestress effect = $\frac{P_0}{A} \pm \frac{P_0(e)}{S}$

$= \frac{12}{96} \pm \frac{12(3)}{128} = -406.25$ top (compression)

$=$ $= 156.25$ bottom (tension)

Net concrete stress:

Top = $576 - 406.25 = 169.75$ psi $< 6\sqrt{f_{ci}'} = 300$ psi

Bottom = $-576 + 156.25 = -419.75 < 0.6 f_{ci}' = 0.6(2500) = 1500$ psi

$f_{ci}' = 2500$ psi

Close → also: other spans are smaller → take some of the load - 21' redistributed

3. Flexural strength (factored loads)

according to UBC 97 $1.4D + 1.7L$

$w = 1.4(125\,psf) + 1.7(60) = 277\,psf$

Capacities for unbonded tendons

$$f_{su} = f_{se} + \frac{1.0 f'_c}{100\rho} + 10\,ksi$$

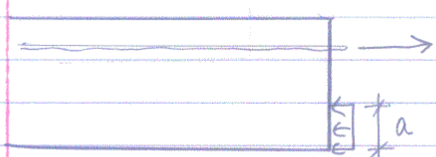
$$\rho \Rightarrow \text{ratio of } \frac{A_{ps}}{b d_p} = \frac{(12/24.8)(0.153)}{12 \times 4.75} = 0.000914$$

15% losses after prestressing

$f_{se} = 0.7(1-0.15)(270) = 160.65\,ksi$

$$f_{su} = 160.65 + \frac{1.0(4.5)}{100(0.000914)} + 10 = 220\,ksi$$

Support & midspan (inverted)



$F_{ult} = 220/160.65 \times 12.0 = 16.43\,K/'$

- $f_{cult} = 0.85(4.5) = 3.83\,K/'$

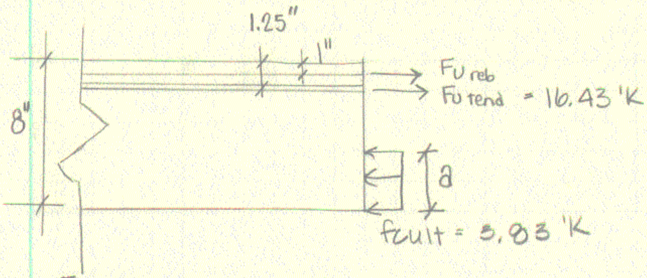
- $a = \dfrac{16.43}{3.83 \times 12} = 0.357''$

- $jd = 8'' - 1.25'' - 0.357''/2 = 6.57''$

- $M_u = 0.9(16.43) \times \dfrac{6.57}{12} = 8.1'K$

☆ Rebar is needed ☆

4. Strength calculations w/ rebar



For max span: 26'

As: #5@ 14" → 0.265 in² /ft

$Fu_{reb} = 0.265 \times 60 = 15.94^{K}$

$a = \frac{15.94 + 16.43}{3.83 \times 12} = 0.7''$

$Jdp = 8'' - 0.35'' - 1.25'' = 6.4''$

$Jd_R = 8'' - 0.35'' - 1.00'' = 6.65''$

$Mu = \frac{0.9}{12}\left[16.43(6.4) + 15.94(6.65)\right] = 15.84\ 'K$

By limit Design: $\frac{wul^2}{8} = 15.84 + 8.1 = 23.94$

For span = 26' → $\frac{8 \times 23.94}{26^2} = \boxed{283 \text{ psf} > 277 \text{ psf}}$

check min reinforcement

- $As_{min} = 0.0015 \times 8 \times 12 = 0.144$ in² /ft

Min reinforcement provided = #5@18 = $\boxed{0.2 \text{ in}^2/\text{ft} > 0.144 \text{ in}^2/\text{ft}}$

<u>APPENDIX D</u>

Table 2.4.1 Long term multipliers[6]

Condition	Without Composite Topping	With Composite Topping
At Erection:		
1. Deflection (downward) component - apply to the elastic deflection due to the member weight at release of prestress	1.85	1.85
2. Camber (upward) component - apply to the elastic camber due to the prestress at the time of release of prestress	1.80	1.80
Final:		
3. Deflection (downward) component - apply to the elastic deflection due to the member weight at release of prestress	2.70	2.40
4. Camber (upward) component - apply to the elastic camber due to prestress at the time of release of prestress	2.45	2.20
5. Deflection (downward) - apply to elastic deflection due to superimposed dead load only	3.00	3.00
6. Deflection (downward) - apply to elastic deflection caused by the composite topping	—	2.30

movement and (–) to indicate downward move- P = 0.95(0.7)(4)(41.3) = 109 ok

Table 1: Camber Multipliers

Table 2.4.2 Maximum Permissible Computed Deflections[1]

Type of member	Deflection to be considered	Deflection limitat
Flat roofs not supporting or attached to non-structural elements likely to be damaged by large deflections	Immediate deflection due to live load L	$\frac{\ell^{*}}{180}$
Floors not supporting or attached to non-structural elements likely to be damaged by large deflections	Immediate deflection due to live load L	$\frac{\ell}{360}$
Roof or floor construction supporting or attached to nonstructural elements likely to be damaged by large deflections	That part of the total deflection occurring after attachment of nonstructural elements (sum of the long-term deflection due to all sustained loads and the immediate deflection due to any additional live load)**	$\frac{\ell^{***}}{480}$
Roof or floor construction supporting or attached to nonstructural elements not likely to be damaged by large deflections		$\frac{\ell^{****}}{240}$

* Limit not intended to safeguard against ponding. Ponding should be checked by suitable calculations of deflection, including added deflections d water, and considering long-term effects of all sustained loads, camber, construction tolerances, and reliability of provisions for drainage.

** Long-term deflection shall be determined in accordance with 9.5.2.5 or 9.5.4.2, but may be reduced by amount of deflection calculated to occur ment of nonstructural elements. This amount shall be determined on basis of accepted engineering data relating to time-deflection characteristics similar to those being considered.

*** Limit may be exceeded if adequate measures are taken to prevent damage to supported or attached elements

Table 2: Maximum Permissible Deflections

Fig. 6.2 Fire endurance (heat transmission) of hollow core units

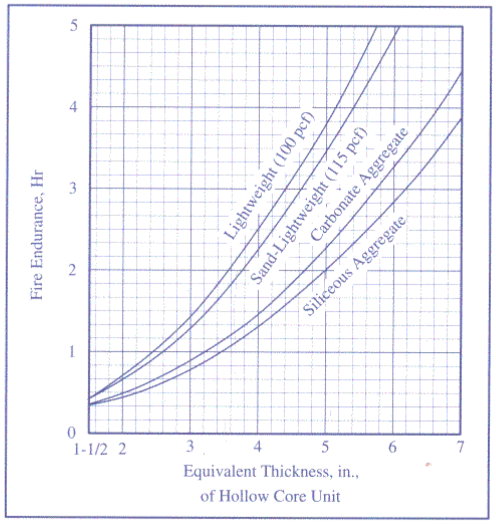


Figure 1: Fire Rating