HIGH RISE CONDO
SOHO, NEW YORK, NY

TECHNICAL ASSIGNMENT 2

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EXECUTIVE SUMMARY

The second technical report consists of comparative analyses of proposed alternate floor systems for the Soho high rise condo. The existing structural system of the 13 story project consists of a two-way flat plate slab. A description of the system over a typical bay is contained in the following section of this report.

To remain consistent with typical design practice of residential construction, alternate floor systems were analyzed that provided a small floor sandwich dimension. These systems include:

- Redesigned two-way Flat Plate w/ Light weight concrete
- Post tensioned two-way Flat Plate
- Precast Slab-Girder
- Composite Steel

Alternative systems were analyzed using loadings following ASCE 7-05. In some cases existing span condition were modified for alternate systems to remain consistent with typical and economical design/construction practices for these systems. Sizes and properties of these systems have been determined through analyses located in the Appendix. Each section includes a depiction of a typical floor plan as well a summary of analyses. Advantages and disadvantages of each system are described throughout the report with a summary table included in the conclusion. The table includes overall depth, constructability, cost, vibration concerns, column size, lead time, and Fireproofing.

FLOOR LOADS AND DEFLECTION CRITERIA

Typical residential floor levels are submitted to the following loads per engineering judgment and minimum design loads from ASCE 7-05.

Dead Loads

Construction Dead Loads:	
Concrete	150 PCF
Steel Framing	10 PSF
Superimposed Dead Loads:	
¼” Glass and Framing	20 PSF
Partitions	20 PSF
Finishes and Misc.	5 PSF
MEP	10 PSF

Live Loads

Typical Floor	40 PSF

Deflection

- $_{\text{Total}} = L/240$
- $_{\text{Live Load}} = L/360$

EXISTING STRUCTURAL SYSTEM

The Soho high rise consists of residential space floors 2-13. Residential levels consist of roughly 14000 SF for floors 2-5 and 5000 SF for floors 6-13. Column layout and typical bays vary in size to accommodate the variations in apartment layout and architectural floor plan. The flat plate slab construction is ideal for residential construction in Manhattan, due to limited building heights imposed by the city of New York. The overall floor depth of the system is small limiting overall floor to floor height, thereby increasing the number of floors and maximizing rentable floor space. The flat plate allows easy coordination with other trades due to the flat profile of the underside of the slab. The main downfall with flat plate construction is the cost that is associated with differential reinforcement requirements of column strips versus middle strips. Another downfall of any cast in place concrete construction is the increased weight of the building thus increasing column sizes, foundations and seismic forces.

The floor system of the Soho high rise is typically a 10-1/2" two-way normal weight concrete flat plate with bays range in size from 13 feet by 21 feet to 25 feet by 25 feet. Typical reinforcement is #4 @12" bottom steel and #5 @ 16" top steel. Additional reinforcement is required at most of the columns because of the inadequacy of the uniform steel to resist the increased moment. In a number of cases as many as 10 additional # 7 bars are required. The columns in the Soho high rise are primarily standard reinforced concrete with varying sizes, shape and reinforcement depending on their location in the building. The most typical shapes are 20x14 and 12x19, both with 6 #9 bars as reinforcement.

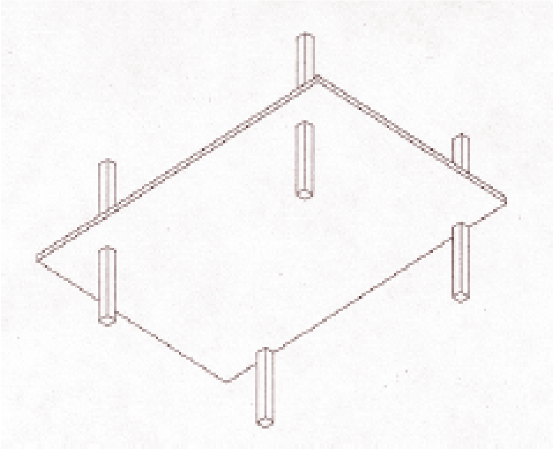


Figure 1:
Typical Flat Plate Construction

ALTERNATE STRUCTURAL SYSTEMS

Redesign of Two-way Flat Plate w/ Light weight Concrete

Two-way flat plate construction is the most used structural system for residential buildings in Manhattan. The existing system uses a 10 ½” thick slab, however when minimum slab thickness is checked for deflection criteria a 9” slab is possible. This increased slab depth is primarily due to the cantilever sections located at the building corners. By adding edge beams in these locations 1 ½” of concrete can be eliminated from each floor level resulting in a significant loss in total building weight and concrete cost. These loads can further be reduced by using light weight concrete for the slabs. The same design strengths can be used and although the material cost is higher a savings in overall cost can be achieved, because of smaller beam, column and foundation sizes. By using concrete containing lightweight aggregate better thermal properties, better fire ratings, reduced autogenous shrinkage, improved contact zone between aggregate and cement matrix, less micro-cracking as a result of better elastic compatibility, more blast resistance, and better shock and sound absorption can be achieved.

As can be seen below in Figure 2 the alternative redesign resulted in significantly less steel for the same bay size used in technical report 1 of 25’x 25’. Two mats of #5 bars @16” each way were used with 9 additional # 5 bars located at columns to resist the larger negative moments. Edge beams were added at the cantilevered corner sections of the building to counteract the increased deflection resulting from the decrease in slab depth. The size of these beams was deflection controlled and resulted in a 12”x 27” section. This will fall within the existing ceiling cavity designed by the architect of 29 ½” and will have no impact on the mechanical systems that are present in the rest of the interstitial space. The full analysis as well as the design assumptions used can be seen in Appendix-A.

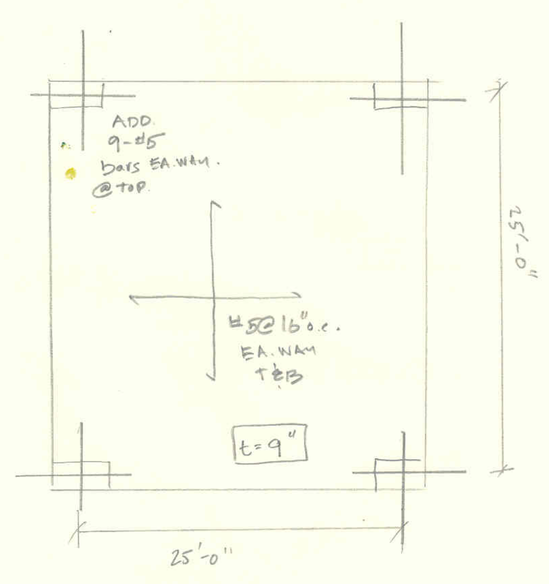


Figure 2
Redesigned Two-way Flat Plate System

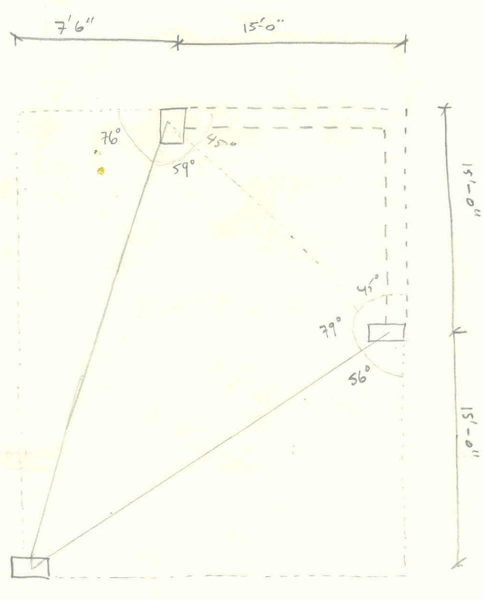


Figure 3
Typical Corner Cantilever

Post Tensioned 2-way Flat Plate

The main advantage of post tensioned concrete design is the thickness of the slab that can be achieved over long spans. By inducing compressive stresses overall moments through the section are decreased, resulting in a thinner slab. This results in lower building weight, smaller member sizes and decrease in construction and material costs. The downfall, however, is that special tendons must be used resulting in added cost from materials and special equipment. This may also have a significant schedule impact. As a result of these additional compressive forces the slab has added resistance to cracking.

The design of a post tensioned system resulted in a 9 ½" slab depth with an increased bay size of 23'4"x33'4". Banded tendons will span the short direction at column strips with an overall effective compression force of 416 kips each. Evenly distributed tendons will span the long direction and have an effective compressive force of 26 kips per foot. Rigid steel reinforcement is used to account for the superimposed dead and live loads on the floor plate. Uniform #4 bars @ 12" o.c. span each way top and bottom with 12 additional #4 bars spanning the long direction being required at the columns. A more detailed analysis can be seen in Appendix-B.

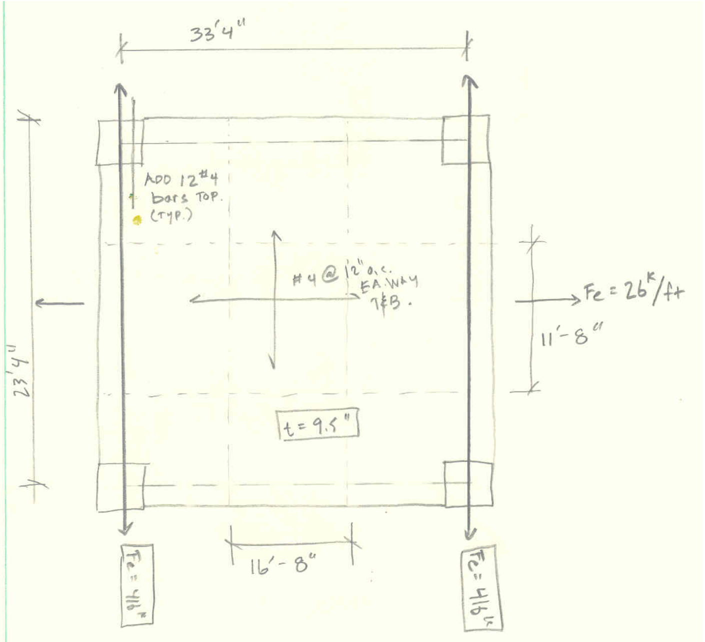


Figure 4
Post Tensioned Flat Plate

Precast Slab-Girder

A precast slab-girder system is a specialized proprietary system designed for residential construction to compete with flat plate concrete construction. A comparative floor depth can be achieved with this system, because the precast planks are at the same elevation as the supporting girders. Bearing connections to the bottom flange of the beam are used to achieve this small floor depth as can be seen in Figure 5. To further limit the floor depth pre-fabricated columns are used with “gooseneck” connections to gain the added girder capacity associated with moment connections as can be seen in Figure 6. Special sections, called “D”-beams, with smaller top flanges must be fabricated for the girder to accommodate the precast planks. These “D” beams also have web knockouts to allow for grouting of the hollow core planks through the steel web achieving a floor system with continuous diaphragmatic action. The weight of this system is low compared to other systems, because the precast planks that are used are hollow core. Slab-girder construction results in a reduction of schedule of up to 25% compared to standard flat plate construction. Additional cost for fireproofing of the steel and coordination of steel and precast contractors may result in added construction cost. An added premium must also be paid for the use of this proprietary system.

Figure 6
Typical "Gooseneck" Connection

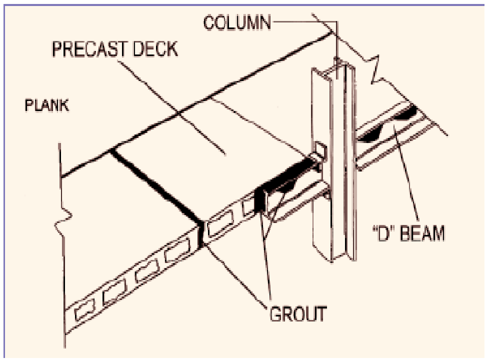


Figure 5
Typical Girder-Slab Construction

To economical use a girder-slab system, as with any other prefabricated unitized system, equal bays sizes are preferred. A typical bay size of 20'x 23'-4" with 3 bays in the buildings short direction and 10 bays in the buildings long direction were chosen. Columns must be added at building corners, as the existing 15' cantilevers cannot be achieved in this system without significant additional detailing and fabrication. Precast planks span the long direction of the bays and the girders span the short direction providing the most efficient use of materials. To resist the typical floor loading 8"x 4' hollow core precast spandeck was chosen with 4 ½" diameter prestressed strands. DB 9x46's were chosen for the steel girders spanning between columns. A 2" non-structural LWC topping was used as recommended by the manufacturer to cover the top flange of the beam as well as improve stiffness, thermal resistance and acoustical performance of the floor system. An overall system depth of 10" can be achieved which is comparative to the existing flat plate system. Section properties as well as design calculations for both the precast spandeck and girders can be seen in Appendix C.

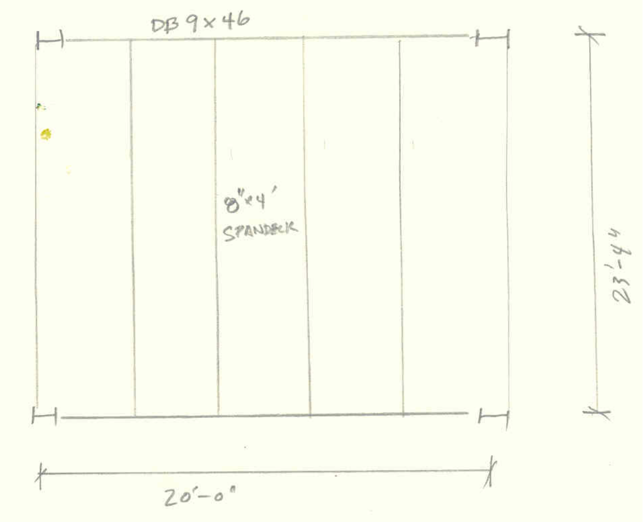


Figure 7
Girder Slab

Composite Steel

Steel has the highest strength to weight ratio of any building material and as such results in the lightest framing system possible. Composite steel construction takes this a step further and uses the floor slabs compressive strength to increase the load capacity of a given wide flange shape. Typical Composite construction can be seen below in Figure 9. The floor depth is shallower than typical steel construction, however a depth as small as concrete construction is not achievable impacting plenum space and total building height. This is a widely used method of construction and the majority of contractors are familiar with the characteristics of composite construction. As a result, there is no learning curve associated with composite construction. Steel construction often has a shorter construction schedule than concrete although lead time for fabrication is higher. Costs for formwork and its impact on schedule are avoided with steel construction by the use of metal deck. Additional cost for fireproofing must be factored into the decision between concrete and steel construction.

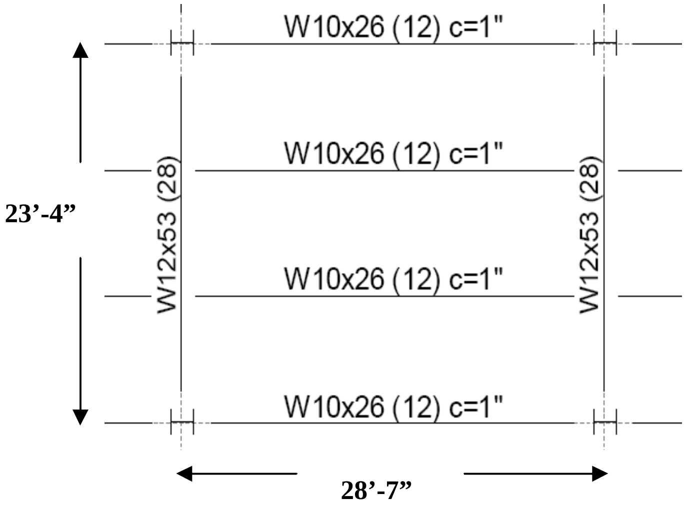


Figure 8
Typical Composite Steel Bay

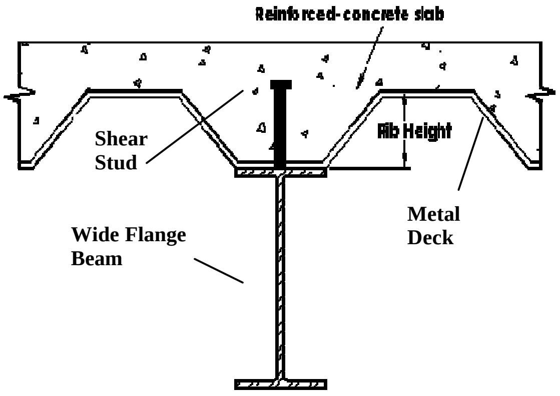


Figure 9
Typical Composite Construction

Composite steel construction benefits from the use of standard bay sizes. A typical bay of 23'4" x 28'7" divides the high rise condo in to 3 bays in the short direction and 7 bays in the long direction. The large increase in bay size and decrease in overall column dimensions will counter the constraints associated with the rigid grid provided by a steel system. Spanning the long direction between girders, evenly spaced W10x26's resist the typical floor loads associated with condo construction. Cambering of beams is an economical method used in steel construction to limit service deflections. In this case a 1" camber has been introduced on the W10's. A larger steel shape would be required to meet the same deflection limitations if camber is not used. W12x53 girder's span the short direction of the bays and result in a total floor depth of 16 ¾". Although this results in roughly a 6" larger floor sandwich than the existing two-way system, HVAC ducts, plumbing and electrical conduit may be integrated into the space between the beams.

CONCLUSION

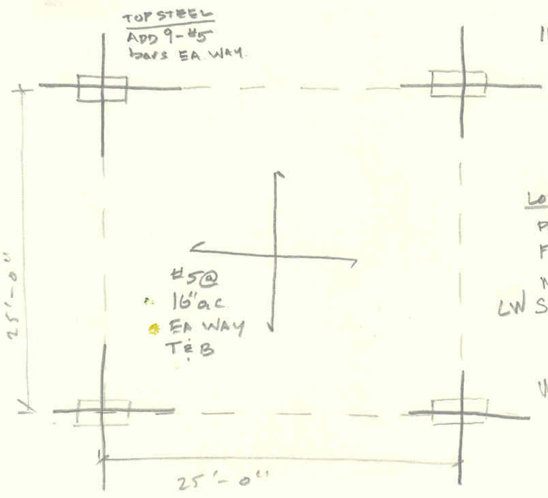
A number of alternative concrete and steel systems have been analyzed for the Soho high rise. Bay sizes were changed in some cases to effectively compare alternate floor systems. The focus of the comparison was to achieve a viable alternative system with a similar floor sandwich depth.

Through the comparison a concrete flat plate remained the ideal system for the high rise. The redesigned flat plate system provided a better solution to the floor framing than the original with a decrease in floor depth and a 29% decrease in service loads. The total cost of the redesigned flat plate was also one dollar cheaper per square foot. On a project of this size significant cost implications will result. The other alternative systems all provided viable options although due to the irregular column layout of the high rise condo the composite steel and slab-girder systems may result in some architectural layout implications. The post tensioned flat plate provided the largest bay sizes with the composite steel providing the second largest, however the cost associated with the PT system is 30% higher than the existing flat plate. The structural depth of the floor sandwich was comparative for all systems excluding the composite steel. Additional investigation would be required to analyze its impact on building systems.

	Flat Plate	Redesigned Flat Plate	PT Flat Plate	Slab-Girder	Composite Steel
Weight (psf)	210	150	195	115	125
Depth (in.)	10 1/2	9	9.5	10	16 3/4
Vibration	N	N	N	N	Y
Constructability	Medium	Medium	Hard	Easy	Medium
Lead Time	Short	Short	Short	Long	Long
Formwork	Yes	Yes	Yes	No	No
Fireproofing	No	No	No	Yes	Yes
Cost ($/SF)					
Material	$4.28	$3.80	$6.12	$6.85	$6.60
Labor	$6.40	$6.05	$7.74	$3.12	$4.10
Total	$10.68	$9.85	$13.86	$9.97	$10.70

APPENDIX

TWO-WAY REDESIGN



INTERIOR BAY

$f'_c = 4^{ksi}$

$f_y = 60^{ksi}$

LOADS

PARTITIONS	20PSF	RESIDENTIAL 40PSF
FIN & MISC	5 PSF	
MEP	10 PSF	
LW SLAB(9")	83 PSF	
	118 PSF	

$W_u = 1.2(118) + 1.6(40) = 206 \text{ PSF}$

THICKNESS

DEFLECTION LIMIT

FLAT PLATE W/O DROP PANELS W/EDGE BEAMS

$$\frac{l_n}{33} = \frac{25-(12''/12)\times 12}{33} = 8.72'' \sim 9'' \text{ SLAB}$$

SLAB WT = 110 PCF × 9/12 = 83 PSF

MIN REINF = $0.0018A_g = 0.0018(9'')(12'') = 0.194\,\text{in}^2/\text{ft}$

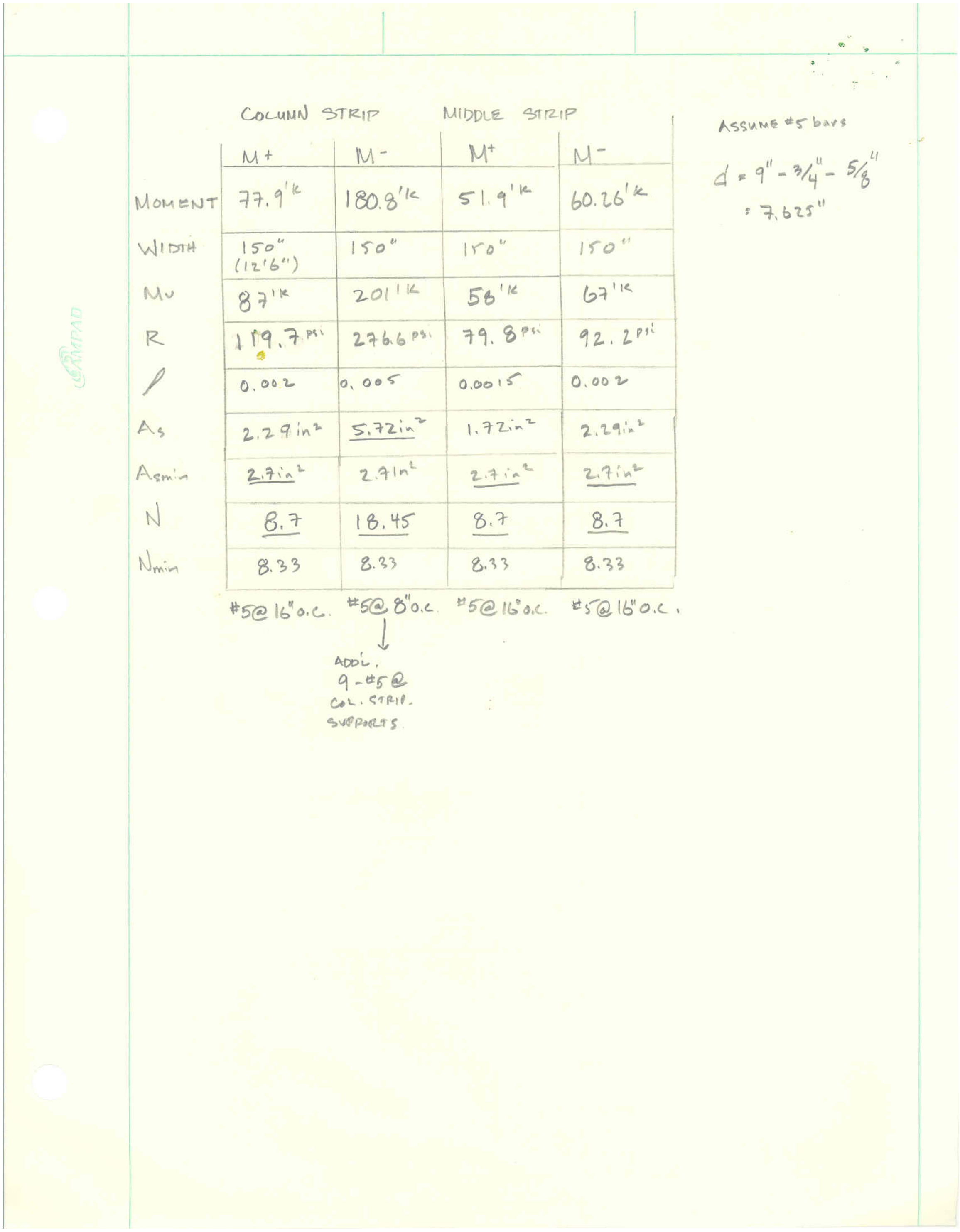
DISTRIBUTION OF M_o

$0.65M_o \rightarrow M^-_{INT}$

$0.35M_o \rightarrow M^+$

$$M_o = \frac{W_u l_2 l_n^2}{8} = \frac{.206(25)(25-12/12)^2}{8} = 370.8'^k$$

LOCATION	STRIP	MOMENT
SUPPORT 0.65 M_o	C.S. 75%	180.8'k
	M.S. 25%	60.26'k
MIDSPAN 0.35 M_o	C.S. 60%	77.89'k
	M.S. 40%	51.9'k



	COLUMN STRIP		MIDDLE STRIP	
	M+	M-	M+	M-
MOMENT	77.9'k	180.8'k	51.9'k	60.26'k
WIDTH	150" (12'6")	150"	150"	150"
M_u	87'k	201'k	58'k	67'k
R	119.7 psi	276.6 psi	79.8 psi	92.2 psi
ρ	0.002	0.005	0.0015	0.002
A_s	$2.29\,in^2$	$5.72\,in^2$	$1.72\,in^2$	$2.29\,in^2$
$A_{s\,min}$	$2.7\,in^2$	$2.71\,n^2$	$2.7\,in^2$	$2.7\,in^2$
N	8.7	18.45	8.7	8.7
N_{min}	8.33	8.33	8.33	8.33
	#5@16" o.c.	#5@8" o.c.	#5@16" o.c.	#5@16" o.c.

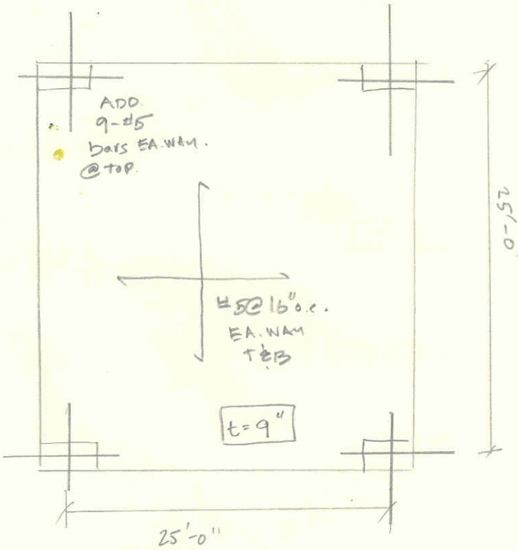
↓

ADD'L.
9-#5@
COL. STRIP.
SUPPORTS.

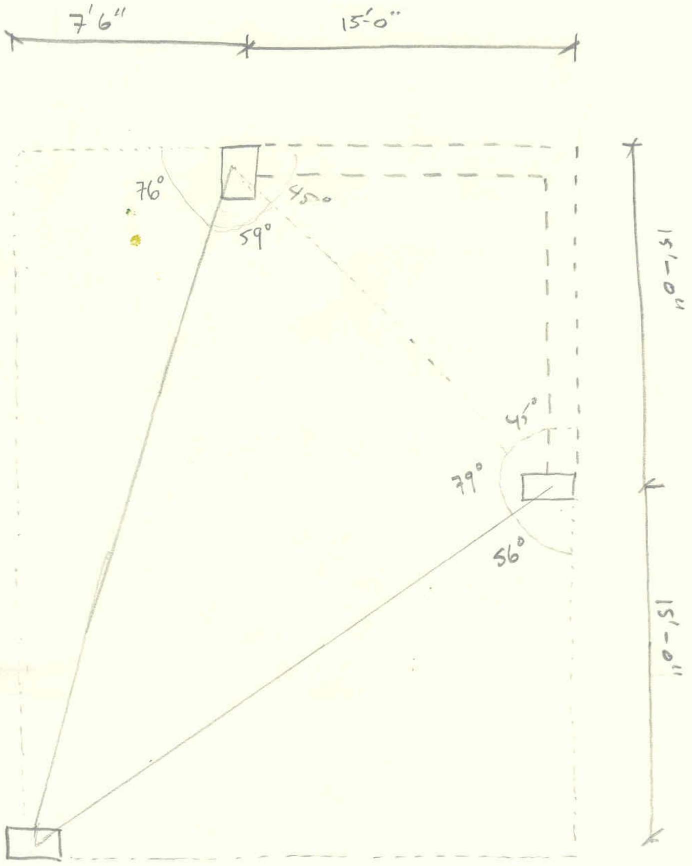
ASSUME #5 bars

$$d = 9'' - 3/4'' - 5/8''$$

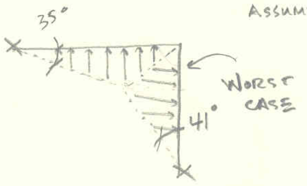
$$= 7.625''$$

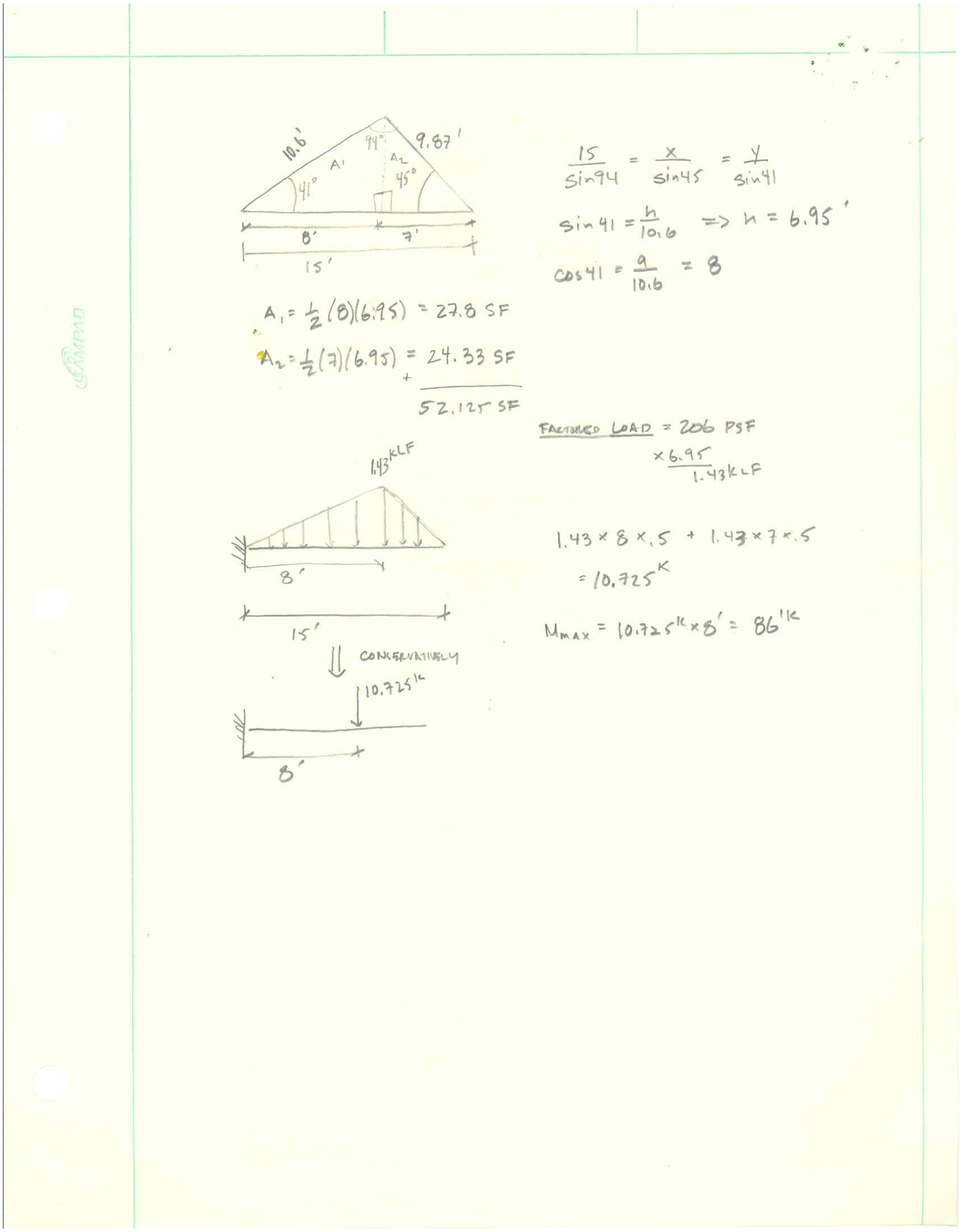

ADD.
9-#5
bars EA.WAY.
@TOP.
#5@16"o.c.
EA.WAY
T&B
t=9"
25'-0"
25'-0"

CANTILEVER BEAMS - RELATIVE STIFFNESS → LOAD PATH



ASSUME 1/3 OF LOAD → CANTILEVER BEAM





$$\frac{15}{\sin 94} = \frac{x}{\sin 45} = \frac{y}{\sin 41}$$

$\sin 41 = \frac{h}{10.6} \Rightarrow h = 6.95'$

$\cos 41 = \frac{a}{10.6} = 8$

$A_1 = \frac{1}{2}(8)(6.95) = 27.8$ SF

$A_2 = \frac{1}{2}(7)(6.95) = 24.33$ SF

$+$ 52.125 SF

FACTORED LOAD = 206 PSF

$\times 6.95$

1.43 KLF

$1.43 \times 8 \times .5 + 1.43 \times 7 \times .5$

$= 10.725^K$

$M_{max} = 10.725^K \times 8' = 86'^K$

$d = \frac{15}{8}(12)(1.65 - 0.005(110)) = 24.75''$

LWC MODIFICATION

$f'_c = 4000\text{ psi}$

$f_y = 60^{ksi}$

ASSUME SW = 300 PLF

$V_{SW} = wl = .3 \times 15 = 4.5^{k} \times 1.2 = 5.4^{k}$

$M_{SW} = \frac{wl^2}{2} = \frac{.3(15)^2}{2} = 33.75^{1k} \times 1.2 = 40.5^{1k}$

$V_{TOT} = 10.725 + 5.4 = 16^{k}$

$M_{TOT} = 40.5 + 86 = 127^{1k}$

$\frac{M_u}{\phi R} = bd^2 \quad R = 667 \; (\rho = 0.0125)$

$\frac{127(12)(1000)}{0.9(667)} = 2538.7\text{in}^3$

$bd^2 = 2538.7\text{in}^3 \quad d = 1.5b$

$2.25b^3 = 2538.7\text{in}^3$

$b = 10.4\text{ in} \Rightarrow 12''$

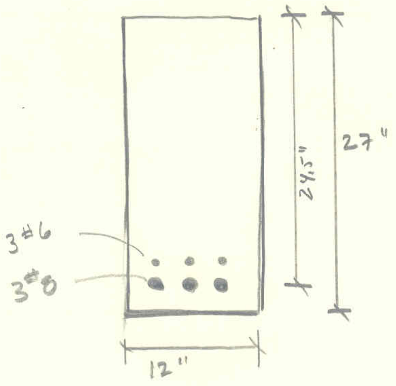
ECONOMIC DESIGN

$\rho = 0.6\rho_{max}$

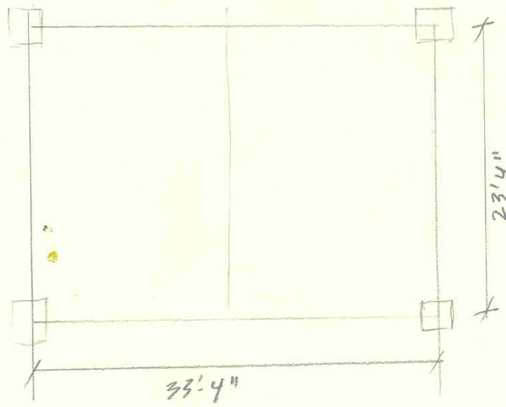
$= 0.6(0.0206) = 0.01236$

$A_s = 0.01236(12)(24.75) = 3.67\text{in}^2$

$3\#8 + 3\#6 = 3.69\text{in}^2$



POST TENSIONED



$f'_c = 4000\,psi$

LOADS

SLAB (9½")	119 PSF
PARTITIONS	20 PSF
MEP	10 PSF
FIN & MISC	5 PSF
LIVE	40 PSF

$W_{pre} = 0.9(119\,PSF) = 107\,PSF$

$W_{NET} = 87\,PSF$

$$\frac{l_n}{42} = \frac{[33'4" - (12")]12}{42} = 9.5"$$

2HR SLAB → 1" COVER T & B

USE 1¼

LONG SPAN

$W_{pre} = 107\,PSF$

$$M_{pre} = \frac{(.107)(33'-4")^2}{8} = 14.86'^k$$

$a = 9.5 - (1.25)2 = 7"$

$$F = \frac{M_{pre}}{a} = \frac{14.86'^k(12)}{7} = 25.5^k$$

$$F/A = \frac{25.5(1000)}{(9.5)(12)} = 223.7\,psi$$

SHORT SPAN

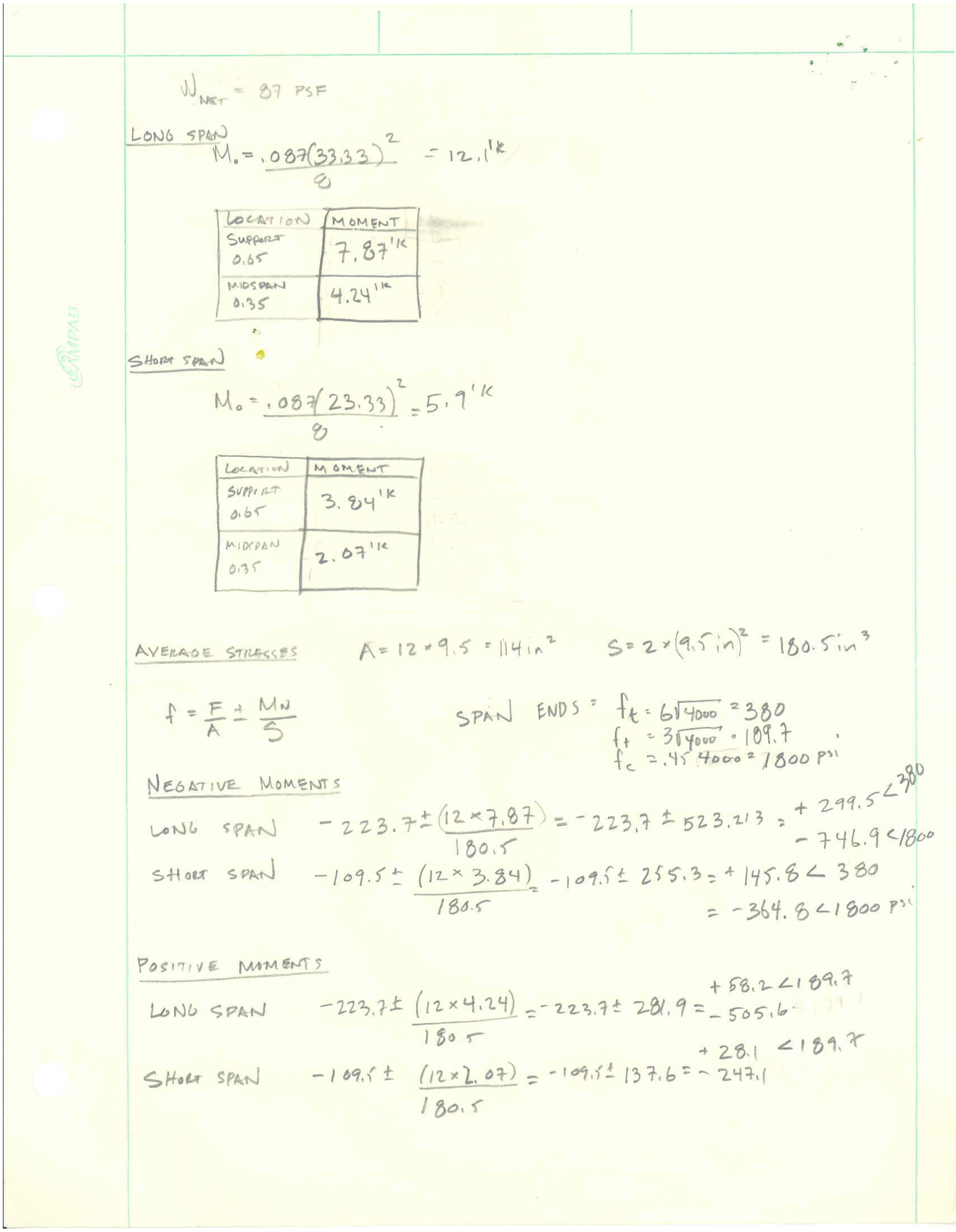
$$M_{pre} = \frac{(.107)(23'-4")^2}{8} = 7.28'^k$$

$$F = \frac{M_{pre}}{a} = \frac{7.28'^k(12)}{7} = 12.48^k$$

$$F/A = \frac{12.48(1000)}{(9.5)(12)} = 109.5\,psi$$

BUNDLES @ STRIPS.

$33.33' \times 12.5^k/ft = 416^k$



$W_{NET} = 87$ PSF

LONG SPAN

$$M_o = \frac{.087(33.33)^2}{8} = 12.1^{'k}$$

LOCATION	MOMENT
SUPPORT 0.65	$7.87^{'k}$
MIDSPAN 0.35	$4.24^{'k}$

SHORT SPAN

$$M_o = \frac{.087(23.33)^2}{8} = 5.9^{'k}$$

LOCATION	MOMENT
SUPPORT 0.65	$3.84^{'k}$
MIDSPAN 0.35	$2.07^{'k}$

AVERAGE STRESSES $A = 12 \times 9.5 = 114 \text{ in}^2$ $S = 2 \times (9.5 \text{ in})^2 = 180.5 \text{ in}^3$

$$f = \frac{F}{A} \pm \frac{M_N}{S}$$

SPAN ENDS: $f_t = 6\sqrt{4000} = 380$
$f_t = 3\sqrt{4000} = 189.7$
$f_c = .45\ 4000 = 1800$ psi

NEGATIVE MOMENTS

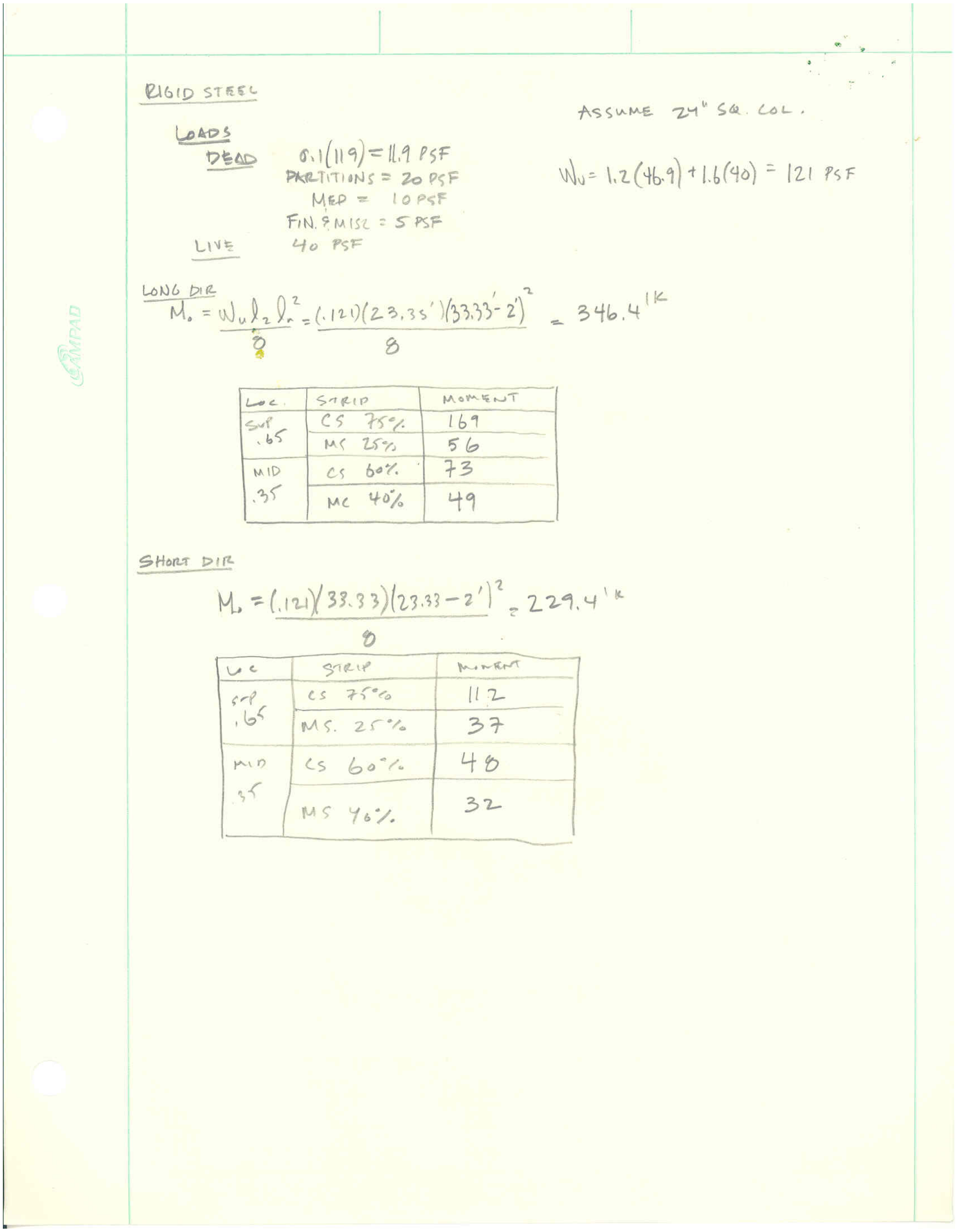
LONG SPAN $-223.7 \pm \frac{(12 \times 7.87)}{180.5} = -223.7 \pm 523.213 = +299.5 < 380$
$-746.9 < 1800$

SHORT SPAN $-109.5 \pm \frac{(12 \times 3.84)}{180.5} = -109.5 \pm 255.3 = +145.8 < 380$
$= -364.8 < 1800$ psi

POSITIVE MOMENTS

LONG SPAN $-223.7 \pm \frac{(12 \times 4.24)}{180.5} = -223.7 \pm 281.9 =$ $+58.2 < 189.7$
-505.6

SHORT SPAN $-109.5 \pm \frac{(12 \times 2.07)}{180.5} = -109.5 \pm 137.6 =$ $+28.1 < 189.7$
-247.1



RIGID STEEL

ASSUME 24" SQ. COL.

LOADS

DEAD 0.1(119) = 11.9 PSF
PARTITIONS = 20 PSF
MEP = 10 PSF
FIN. & MISC = 5 PSF
LIVE 40 PSF

$W_u = 1.2(46.9) + 1.6(40) = 121$ PSF

LONG DIR

$$M_o = \frac{W_u l_2 l_n^2}{8} = \frac{(.121)(23.35')(33.33'-2')^2}{8} = 346.4'^{K}$$

LOC.	STRIP	MOMENT
SUP .65	CS 75%	169
	MS 25%	56
MID .35	CS 60%	73
	MC 40%	49

SHORT DIR

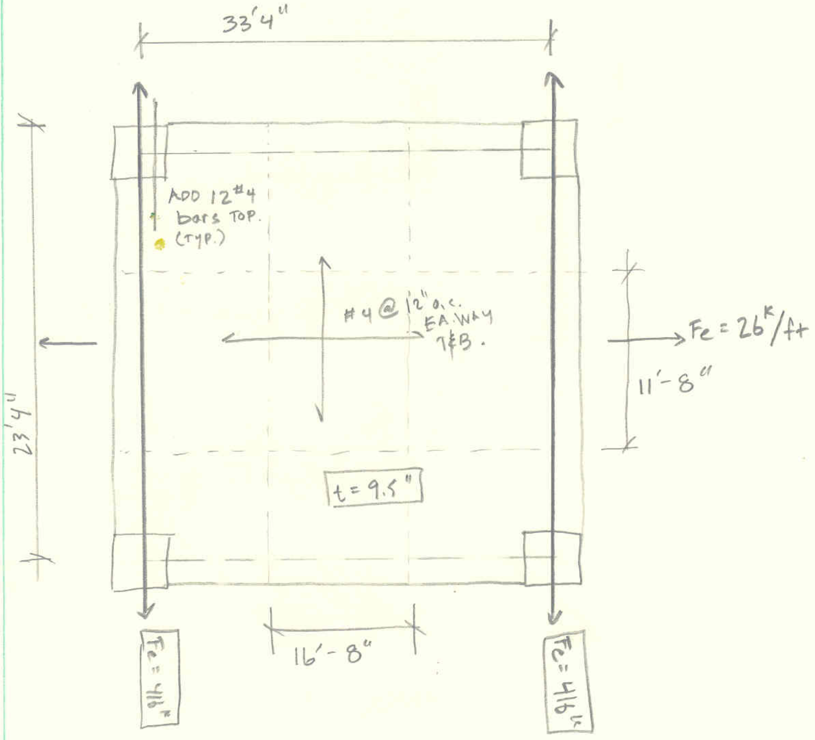
$$M_o = \frac{(.121)(33.33)(23.33-2')^2}{8} = 229.4'^{K}$$

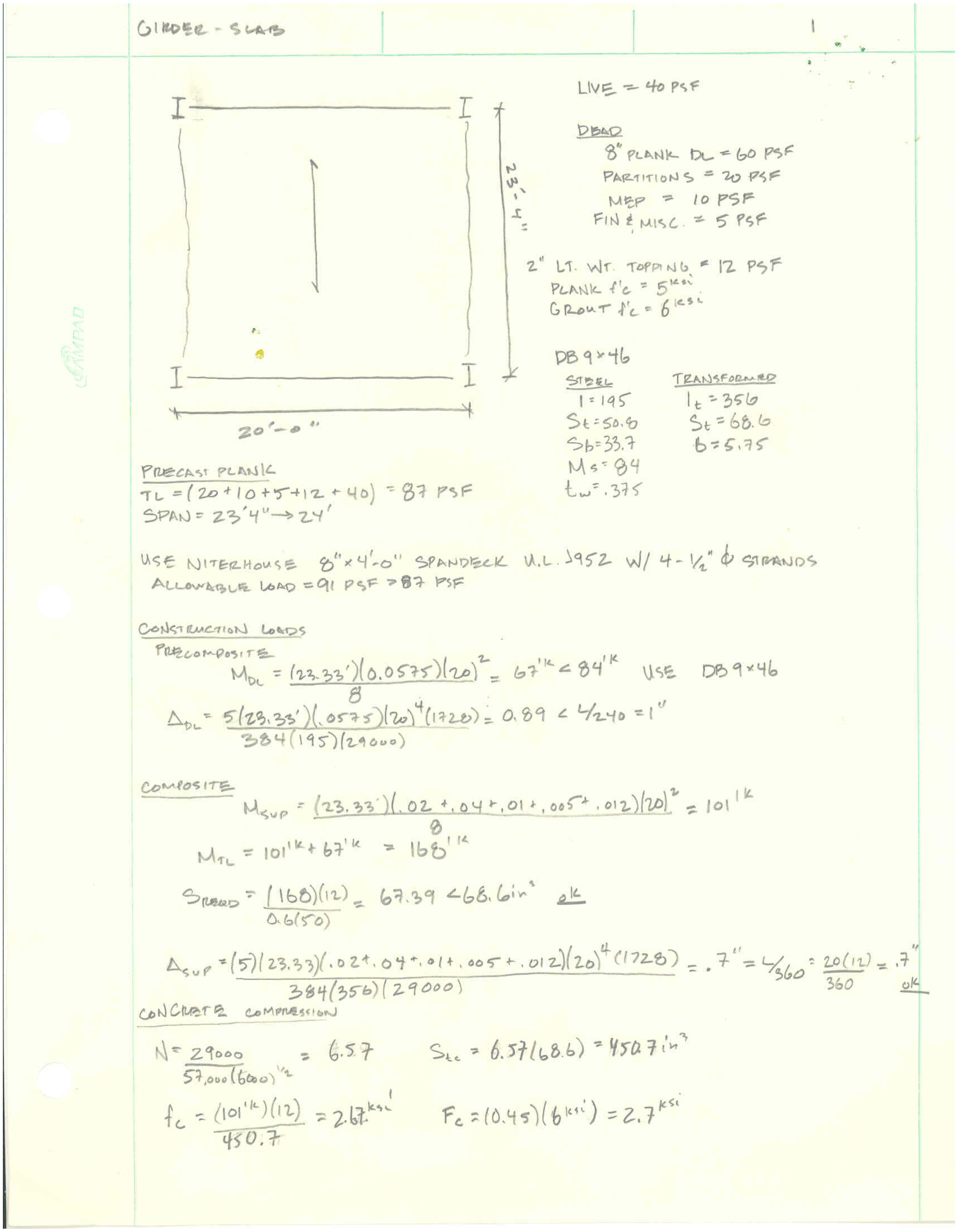
LOC	STRIP	MOMENT
SUP .65	CS 75%	112
	MS. 25%	37
MID .35	CS 60%	48
	MS 40%	32

	LONG DIR				SHORT DIR			
	COL STRIP.		MID STRIP		COL STRIP		MID STRIP.	
	M+	M−	M+	M−	M+	M−	M+	M−
MOMENT	73	169	49	56	48	112	32	37
WIDTH	140"	140"	140"	140"	200"	200"	200"	200"
M_u	81	188	54	62	53	124	36	41
R	102	237	68	78	47	109	32	36
ρ	.002	.004	.00125	.0015	.001	.002	.0005	.0005
A_s	2.31	4.62	1.44	1.73	1.65	3.3	.825	.825
$A_{s\,min}$	2.31	2.31	2.31	2.31	3.3	3.3	3.3	3.3
N	12	24	12	12	16.5	16.5	11.5	16.5
N_{min}	8	8	8	8	11	11	11	11
	@12" o.c.	@6" o.c.	@12" o.c.	@12" o.c.	@12" o.c.	@12" o.c.	@12" o.c.	@12" o.c.

ASSUME #4's

$d = 9.5 - 3/4 - 1/2$
$= 8.25"$


33'4"
ADD 12 #4 bars TOP. (TYP.)
#4 @ 12" o.c. EA. WAY T&B.
Fe = 26k/ft
11'-8"
23'4"
t = 9.5"
16'-8"
Fe = 416k
Fe = 416k



GIRDER - SLAB

1

23'-4"

20'-0"

LIVE = 40 PSF

DEAD

8" PLANK DL = 60 PSF
PARTITIONS = 20 PSF
MEP = 10 PSF
FIN & MISC. = 5 PSF

2" LT. WT. TOPPING = 12 PSF
PLANK $f'_c = 5^{ksi}$
GROUT $f'_c = 6^{ksi}$

DB 9×46

STEEL	TRANSFORMED
$I = 195$	$I_t = 356$
$S_t = 50.8$	$S_t = 68.6$
$S_b = 33.7$	$b = 5.75$
$M_s = 84$	
$t_w = .375$	

PRECAST PLANK

TL = (20 + 10 + 5 + 12 + 40) = 87 PSF
SPAN = 23'4" → 24'

USE NITERHOUSE 8"×4'-0" SPANDECK U.L. J952 W/ 4 - ½" ϕ STRANDS
ALLOWABLE LOAD = 91 PSF > 87 PSF

CONSTRUCTION LOADS

PRECOMPOSITE

$$M_{DL} = \frac{(23.33')(0.0575)(20)^2}{8} = 67'^k < 84'^k \quad \text{USE DB 9×46}$$

$$\Delta_{DL} = \frac{5(23.33')(.0575)(20)^4(1728)}{384(195)(29000)} = 0.89 < L/240 = 1''$$

COMPOSITE

$$M_{SUP} = \frac{(23.33')(.02 + .04 + .01 + .005 + .012)(20)^2}{8} = 101'^k$$

$$M_{TL} = 101'^k + 67'^k = 168'^k$$

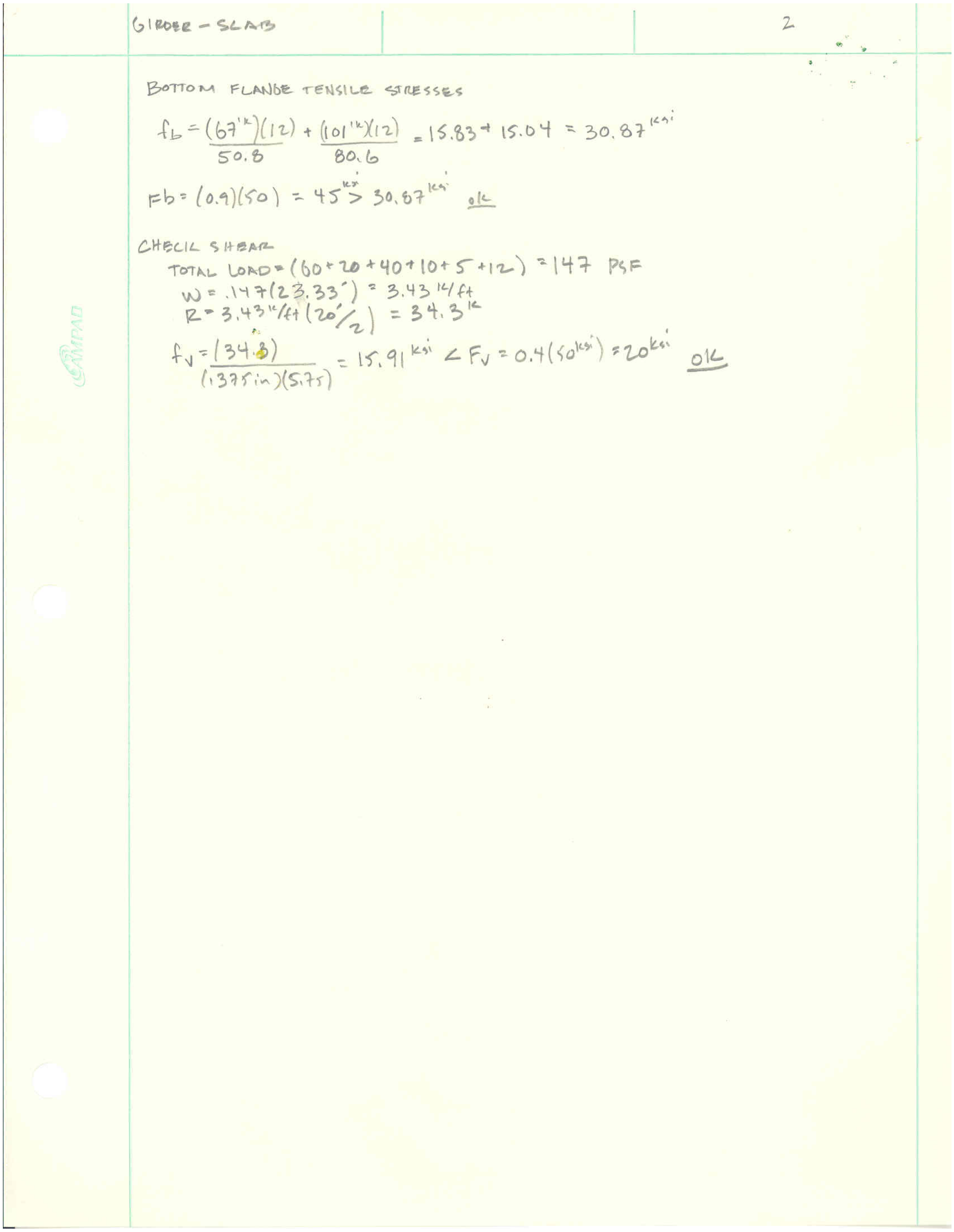
$$S_{REQD} = \frac{(168)(12)}{0.6(50)} = 67.39 < 68.6 \text{ in}^3 \quad \underline{ok}$$

$$\Delta_{SUP} = \frac{(5)(23.33)(.02 + .04 + .01 + .005 + .012)(20)^4(1728)}{384(356)(29000)} = .7'' = L/360 = \frac{20(12)}{360} = .7'' \quad \underline{ok}$$

CONCRETE COMPRESSION

$$N = \frac{29000}{57{,}000(6000)^{1/2}} = 6.57 \qquad S_{tc} = 6.57(68.6) = 450.7 \text{ in}^3$$

$$f_c = \frac{(101'^k)(12)}{450.7} = 2.67^{ksi} \qquad F_c = (0.45)(6^{ksi}) = 2.7^{ksi}$$



GIRDER – SLAB 2

BOTTOM FLANGE TENSILE STRESSES

$$f_b = \frac{(67'^k)(12)}{50.8} + \frac{(101'^k)(12)}{80.6} = 15.83 + 15.04 = 30.87^{ksi}$$

$$F_b = (0.9)(50) = 45^{ksi} > 30.87^{ksi} \quad \underline{ok}$$

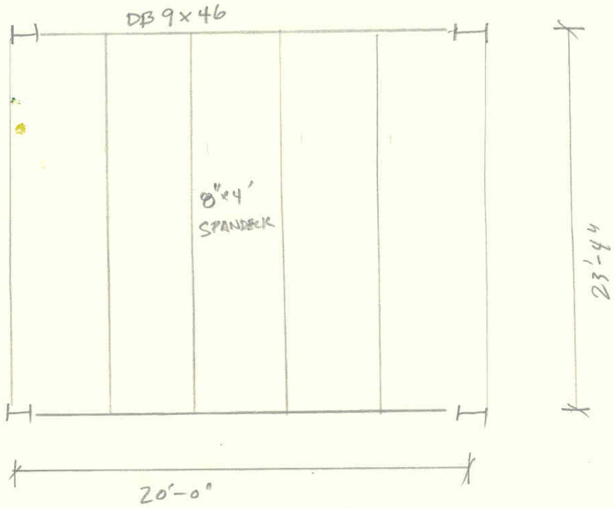
CHECK SHEAR

TOTAL LOAD = (60 + 20 + 40 + 10 + 5 + 12) = 147 PSF

$$w = .147(23.33') = 3.43^{k}/ft$$

$$R = 3.43^{k}/ft\,(20'/2) = 34.3^{k}$$

$$f_v = \frac{(34.3)}{(.375\,in)(5.75)} = 15.91^{ksi} < F_v = 0.4(50^{ksi}) = 20^{ksi} \quad \underline{OK}$$

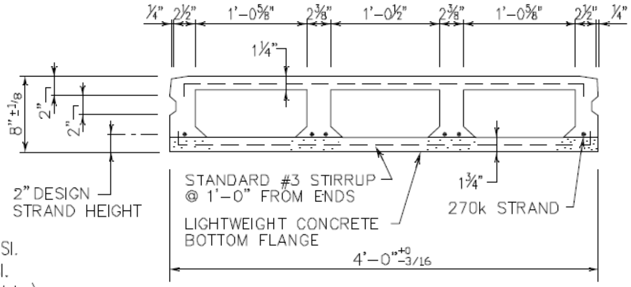


DB 9x46
8"x4'
SPANDECK
23'-4"
20'-0"

Prestressed Concrete
8"x4' SpanDeck—U.L.—J952
(NO TOPPING)

PHYSICAL PROPERTIES
Precast

A = 199 $in.^2$ S_b = 332 $in.^3$
I = 1370 $in.^4$ S_t = 354 $in.^3$
Y_b = 4.13 in. Wt. = 230 PLF
Y_t = 3.87 in. Wt. = 57.5 PSF
e = 2.13 in.



8" SPANDECK CROSS SECTION

UL FIRE RATED J952

DESIGN DATA
1. Precast Strength @ 28 days = 5000 PSI.
2. Precast Strength @ release = 3000 PSI.
3. Precast Density = 150 PCF (Top and Webs)
   = 115 PCF (Soffit)
4. Strand = 1/2"ø, 270 K Lo-Relaxation.
5. Strand Height = 2.00 in.
6. Ultimate moment capacities (when fully developed)...
   4 – 1/2"ø, 270K = 68.0'K
   6 – 1/2"ø, 270K = 96.3'K
7. Maximum bottom tensile stress is $6\sqrt{f'c}$ =424 PSI.
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Shear values are the maximum allowable before shear reinforcement is required.
11. Deflection limits were not considered when determing allowable loads in this table.
12. All values in this table are based on ultimate strength and are not governed by service stress.

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	565	510	460	420	361	309	265	231	199	175	152	133	118	105	91	80	71	62	54	47	41	35	☒
Shear 4 – 1/2"ø	429	383	345	313	286	263	241	225	209	195	177	159	143	130	118	110	104	95	88	79	72	66	
Flexure 6 – 1/2"ø	825	745	675	619	531	456	395	344	302	265	236	209	186	167	149	134	120	107	96	86	78	70	63
Shear 6 – 1/2"ø	446	398	359	326	298	274	253	234	218	204	191	179	169	159	150	138	126	115	106	97	89	82	76

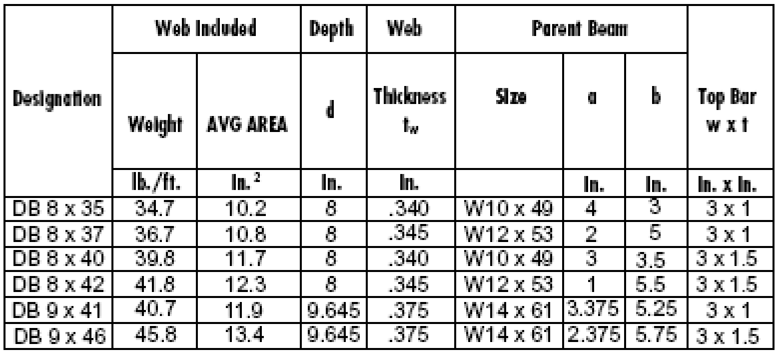
NITTERHOUSE
CONCRETE PRODUCTS

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

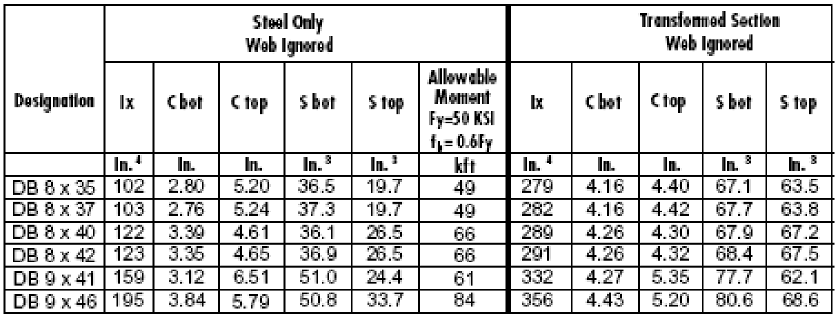
REVISED 12/93

D-Beam Dimensions Table

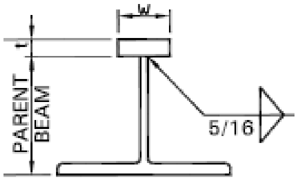


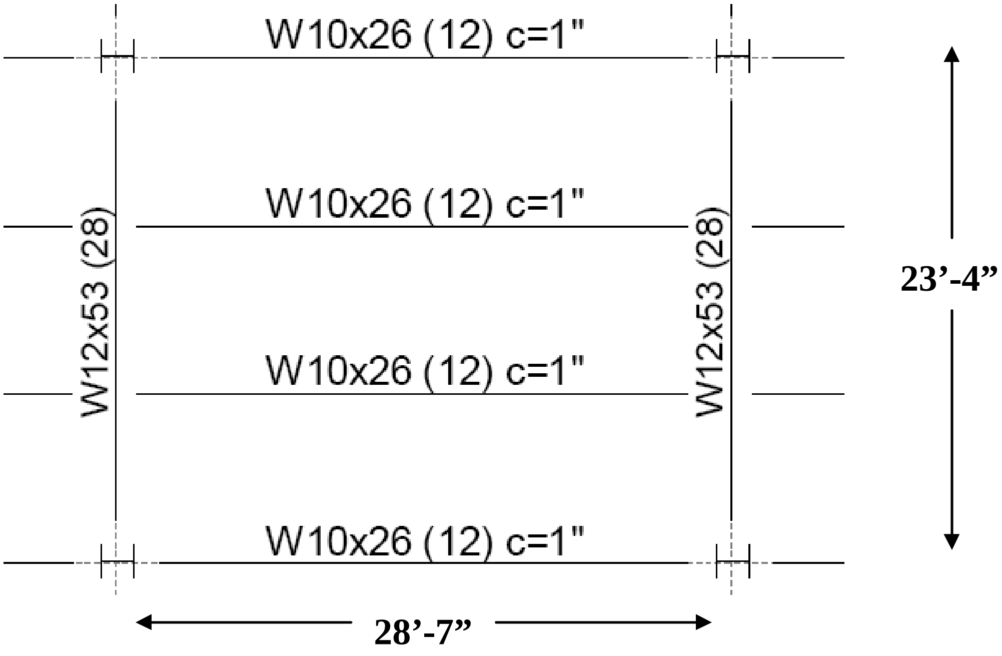
Designation	Web Included		Depth	Web	Parent Beam			Top Bar
	Weight	AVG AREA	d	Thickness t_w	Size	a	b	w x t
	lb./ft.	In.^{2}	In.	In.		In.	In.	In. x In.
DB 8 x 35	34.7	10.2	8	.340	W10 x 49	4	3	3 x 1
DB 8 x 37	36.7	10.8	8	.345	W12 x 53	2	5	3 x 1
DB 8 x 40	39.8	11.7	8	.340	W10 x 49	3	3.5	3 x 1.5
DB 8 x 42	41.8	12.3	8	.345	W12 x 53	1	5.5	3 x 1.5
DB 9 x 41	40.7	11.9	9.645	.375	W14 x 61	3.375	5.25	3 x 1
DB 9 x 46	45.8	13.4	9.645	.375	W14 x 61	2.375	5.75	3 x 1.5

D-Beam Properties Table



Designation	Steel Only Web Ignored						Transformed Section Web Ignored				
	Ix	C bot	C top	S bot	S top	Allowable Moment Fy=50 KSI f_b= 0.6Fy	Ix	C bot	C top	S bot	S top
	In.^{4}	In.	In.	In.^{3}	In.^{3}	kft	In.^{4}	In.	In.	In.^{3}	In.^{3}
DB 8 x 35	102	2.80	5.20	36.5	19.7	49	279	4.16	4.40	67.1	63.5
DB 8 x 37	103	2.76	5.24	37.3	19.7	49	282	4.16	4.42	67.7	63.8
DB 8 x 40	122	3.39	4.61	36.1	26.5	66	289	4.26	4.30	67.9	67.2
DB 8 x 42	123	3.35	4.65	36.9	26.5	66	291	4.26	4.32	68.4	67.5
DB 9 x 41	159	3.12	6.51	51.0	24.4	61	332	4.27	5.35	77.7	62.1
DB 9 x 46	195	3.84	5.79	50.8	33.7	84	356	4.43	5.20	80.6	68.6





Typical Composite Steel Bay

Gravity Beam Design

```
RAM Steel v10.0
DataBase: composite floor compare                                    10/25/06 23:38:06
Building Code: IBC                                              Steel Code: ASD 9th Ed.

Floor Type: floor              Beam Number = 30
SPAN INFORMATION (ft):   I-End (28.58,38.87)   J-End (57.08,38.87)
   Beam Size (User Selected)        =  W10X26                        Fy  =  50.0 ksi
   Total Beam Length (ft)           =  28.50
COMPOSITE PROPERTIES (Not Shored):
                                                     Left                Right
   Concrete thickness (in)                           3.25                 3.25
   Unit weight concrete (pcf)                      110.00               110.00
   f'c (ksi)                                         4.00                 4.00
   Decking Orientation                        perpendicular        perpendicular
   Decking type                               VULCRAFT 1.5VL       VULCRAFT 1.5VL
   beff (in)           =        85.50    Y bar(in)         =          11.37
   Seff (in3)          =        39.45    Str (in3)         =          50.45
   Ieff (in4)          =       355.03    Itr (in4)         =         556.16
   Stud length (in)    =         4.00    Stud diam (in)    =           0.75
   Stud Capacity (kips)   q  =  8.3
   # of studs:     Full  =  46     Partial = 12     Actual = 12
   Number of Stud Rows = 1     Percent of Full Composite Action = 26.21
LINE LOADS (k/ft):
  Load      Dist       DL       CDL       LL     Red%     Type     CLL
    1      0.000     0.638    0.389    0.311    3.8%      Red     0.311
          28.500     0.638    0.389    0.311                      0.311
SHEAR:  Max V (DL+LL) = 13.36 kips   fv = 4.99 ksi   Fv = 20.00 ksi
MOMENTS:
Span        Cond        Moment      @        Lb       Cb      Tension Flange     Compr Flange
                        kip-ft      ft       ft                 fb       Fb        fb      Fb
Center      PreCmp+      71.1      14.3     0.0      1.00     30.58    33.00     30.58   33.00
            Max +        95.2      14.3     ---      ---
          Mmax/Seff                                           28.95    33.00      ---     ---
          Mconst/Sx+Mpost/Seff                                33.93    45.00      ---     ---
Controlling              71.1      14.3     0.0      1.00     30.58    33.00      ---     ---
fc (ksi)  =  0.43      Fc  =  1.80
REACTIONS (kips):
                                          Left      Right
   Initial reaction                       9.98       9.98
   DL reaction                            9.09       9.09
   Max +LL reaction                       4.27       4.27
   Max +total reaction                   13.36      13.36
DEFLECTIONS:  (Camber = 1)
   Initial load (in)          at      14.25 ft  =      -1.383       L/D  =     247
   Live load (in)             at      14.25 ft  =      -0.432       L/D  =     792
   Post Comp load (in)        at      14.25 ft  =      -0.791       L/D  =     433
   Net Total load (in)        at      14.25 ft  =      -1.173       L/D  =     291
```

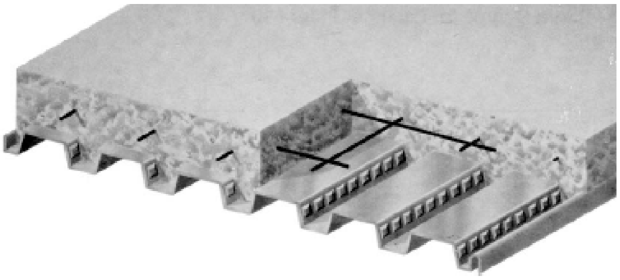
Gravity Beam Design

```
RAM Steel v10.0
DataBase: composite floor compare                                        10/25/06 23:40:20
Building Code: IBC                                                  Steel Code: ASD 9th Ed.

Floor Type: floor            Beam Number = 18
SPAN INFORMATION (ft):  I-End (28.58,23.33)   J-End (28.58,46.66)
    Beam Size (User Selected)       =   W12X53                         Fy  =  50.0 ksi
    Total Beam Length (ft)          =   23.33
COMPOSITE PROPERTIES (Not Shored):
                                                 Left              Right
    Concrete thickness (in)                      3.25               3.25
    Unit weight concrete (pcf)                 110.00             110.00
    f'c (ksi)                                    4.00               4.00
    Decking Orientation                      parallel           parallel
    Decking type                         VULCRAFT 1.5VL     VULCRAFT 1.5VL
    beff (in)          =        69.99    Y bar(in)        =        11.22
    Seff (in3)         =        89.49    Str (in3)        =       107.95
    Ieff (in4)         =       801.87    Itr (in4)        =      1170.19
    Stud length (in)   =         4.00    Stud diam (in)   =         0.75
    Stud Capacity (kips)  q  =  11.1
    # of studs:     Full  =  108     Partial = 28      Actual = 28
    Number of Stud Rows = 1     Percent of Full Composite Action = 25.58
POINT LOADS (kips):
      Dist     DL     CDL   RedLL    Red%   NonRLL  StorLL   Red%   RoofLL   Red%    CLL
     7.770  18.18   11.09    8.87    24.7     0.00    0.00    0.0     0.00   Snow    8.87
    15.540  18.21   11.10    8.88    24.7     0.00    0.00    0.0     0.00   Snow    8.88
SHEAR:  Max V (DL+LL) = 24.90 kips   fv = 5.96 ksi   Fv = 20.00 ksi
MOMENTS:
Span        Cond        Moment      @       Lb      Cb      Tension Flange     Compr Flange
                         kip-ft     ft      ft                fb       Fb        fb       Fb
Center      PreCmp+      155.5    15.5     7.8    1.75     26.43    33.00     26.43    33.00
            Max +        193.7    15.5     ---     ---
          Mmax/Seff                                        25.98    33.00       ---      ---
          Mconst/Sx+Mpost/Seff                             29.07    45.00       ---      ---
Controlling              155.5    15.5     7.8    1.75     26.43    33.00       ---      ---
fc (ksi)  =  0.59       Fc  =  1.80
REACTIONS (kips):
                                     Left     Right
    Initial reaction                19.98     19.96
    DL reaction                     18.21     18.18
    Max +LL reaction                 6.69      6.68
    Max +total reaction             24.90     24.87
DEFLECTIONS:
    Initial load (in)        at     11.66 ft  =     -0.701      L/D  =     399
    Live load (in)           at     11.66 ft  =     -0.224      L/D  =    1250
    Post Comp load (in)      at     11.66 ft  =     -0.462      L/D  =     606
    Net Total load (in)      at     11.66 ft  =     -1.163      L/D  =     241
```

SLAB INFORMATION

Total Slab Depth	Theo. Concrete Volume Yds./ 100 Sq. Ft.	Cu. Ft./ Sq. Ft.	Recommended Welded Wire Fabric
3 1/2"	0.78	0.210	6x6-W1.4xW1.4
4"	0.93	0.252	6x6-W1.4xW1.4
4 1/2"	1.09	0.294	6x6-W1.4xW1.4
4 3/4"	1.16	0.314	6x6-W1.4xW1.4
5"	1.24	0.335	6x6-W2.1xW2.1
5 1/2"	1.40	0.377	6x6-W2.1xW2.1
5 3/4"	1.47	0.398	6x6-W2.1xW2.1
6"	1.55	0.418	6x6-W2.1xW2.1



(N=14) LIGHTWEIGHT CONCRETE (110 PCF)

Total Slab Depth	Deck Type	SDI Max. Unshored Clear Span			Superimposed Live Load, PSF														
					Clear Span (ft.-in.)														
		1 Span	2 Span	3 Span	5'-0	5'-6	6'-0	6'-6	7'-0	7'-6	8'-0	8'-6	9'-0	9'-6	10'-0	10'-6	11'-0	11'-6	12'-0
	1.5VL22	5'-7	7'-5	7'-6	278	247	206	185	167	152	139	124	105	89	76	66	57	50	44
3 1/2"	1.5VL21	6'-3	8'-3	8'-5	293	260	233	195	177	161	147	130	110	93	80	69	60	53	46
	1.5VL20	6'-8	8'-11	9'-0	305	271	243	220	185	168	154	135	114	97	83	72	62	54	48
(t=2")	1.5VL19	7'-6	10'-0	10'-1	329	292	262	237	216	198	167	145	122	104	89	77	67	58	51
	1.5VL18	8'-2	10'-8	11'-0	350	311	279	252	230	211	184	153	129	110	94	81	71	62	54
26 PSF	1.5VL17	8'-11	11'-4	11'-8	352	312	280	253	231	212	195	163	137	116	100	86	75	66	58
	1.5VL16	9'-6	11'-10	12'-3	352	312	280	253	231	212	195	171	144	122	105	91	79	69	61
	1.5VL22	5'-4	7'-1	7'-2	324	269	239	215	194	177	161	148	136	126	113	98	85	75	66
4"	1.5VL21	5'-11	7'-11	8'-0	341	303	253	227	205	187	171	157	145	134	119	102	89	78	69
	1.5VL20	6'-4	8'-5	8'-7	355	315	283	237	214	195	178	164	151	140	123	106	92	81	71
(t=2 1/2")	1.5VL19	7'-1	9'-6	9'-7	382	339	304	275	251	211	193	178	164	152	131	113	99	86	76
	1.5VL18	7'-9	10'-2	10'-6	400	360	323	292	266	244	206	189	175	162	139	120	104	91	80
30 PSF	1.5VL17	8'-5	10'-10	11'-2	400	361	324	293	267	245	226	190	175	163	147	127	111	97	85
	1.5VL16	9'-0	11'-4	11'-9	400	360	323	292	266	244	225	209	195	162	151	134	116	102	90
	1.5VL22	5'-1	6'-9	6'-10	372	309	275	246	223	202	185	170	156	145	134	125	116	106	93
4 1/2"	1.5VL21	5'-8	7'-7	7'-8	391	347	290	260	235	214	196	180	166	153	142	132	123	111	97
	1.5VL20	6'-0	8'-1	8'-2	400	361	324	272	246	223	204	188	173	160	149	139	129	114	101
(t=3")	1.5VL19	6'-9	9'-0	9'-2	400	388	348	315	265	242	221	203	188	174	162	151	140	122	107
	1.5VL18	7'-4	9'-9	10'-0	400	400	369	334	305	257	236	217	200	186	173	161	147	129	114
35 PSF	1.5VL17	8'-0	10'-4	10'-8	400	400	370	335	305	280	258	217	200	186	173	161	151	137	120
	1.5VL16	8'-6	10'-10	11'-3	400	400	369	334	304	279	257	239	199	185	172	160	150	140	126
	1.5VL22	5'-0	6'-8	6'-9	396	329	293	263	237	216	197	181	167	154	143	133	124	116	108
4 3/4"	1.5VL21	5'-6	7'-5	7'-6	400	370	309	277	251	228	208	191	177	163	152	141	132	123	114
	1.5VL20	5'-11	7'-11	8'-0	400	385	322	289	262	238	218	200	185	171	159	148	138	129	118
(t=3 1/4")	1.5VL19	6'-7	8'-10	8'-11	400	400	371	336	283	257	235	216	200	185	172	160	150	140	126
	1.5VL18	7'-2	9'-7	9'-9	400	400	393	356	324	274	251	231	213	198	184	171	160	150	133
37 PSF	1.5VL17	7'-9	10'-2	10'-6	400	400	394	356	325	298	251	231	213	198	184	171	160	150	141
	1.5VL16	8'-4	10'-8	11'-0	400	400	392	355	324	297	274	230	212	197	183	171	159	149	140
	1.5VL22	4'-11	6'-6	6'-7	397	350	311	279	252	229	209	192	177	164	152	141	131	123	115
5"	1.5VL21	5'-5	7'-3	7'-4	400	369	328	295	266	242	221	203	188	174	161	150	140	131	122
	1.5VL20	5'-9	7'-9	7'-10	400	400	342	307	278	253	231	212	196	181	168	157	146	137	128
(t=3 1/2")	1.5VL19	6'-5	8'-8	8'-9	400	400	394	332	300	273	250	230	212	197	183	170	159	149	140
	1.5VL18	7'-0	9'-4	9'-7	400	400	400	378	344	291	266	245	226	210	195	182	170	159	150
39 PSF	1.5VL17	7'-7	9'-11	10'-3	400	400	400	378	345	316	266	245	226	210	195	182	170	159	150
	1.5VL16	8'-2	10'-5	10'-9	400	400	400	377	343	315	291	244	225	209	194	181	169	159	149
	1.5VL22	4'-7	6'-2	6'-3	400	400	367	329	297	270	247	227	209	193	179	166	155	145	135
5 3/4"	1.5VL21	5'-2	6'-11	7'-0	400	400	387	347	314	286	261	240	221	205	190	177	165	154	144
	1.5VL20	5'-6	7'-4	7'-5	400	400	400	362	327	298	272	250	231	214	199	185	172	161	151
(t=4 1/4")	1.5VL19	6'-1	8'-2	8'-4	400	400	400	391	354	322	295	271	250	232	215	201	187	175	165
	1.5VL18	6'-7	8'-10	9'-1	400	400	400	400	376	343	314	289	267	247	230	214	200	188	176
46 PSF	1.5VL17	7'-2	9'-5	9'-9	400	400	400	400	400	343	314	289	267	247	230	214	200	188	176
	1.5VL16	7'-8	9'-11	10'-3	400	400	400	400	400	371	312	287	265	246	229	213	199	187	175

Notes:
1. Minimum exterior bearing length required is 1.5 inches. Minimum interior bearing length required is 3.0 inches. If these minimum lengths are not provided, web crippling must be checked.
2. Always contact Vulcraft when using loads in excess of 200 psf. Such loads often result from concentrated, dynamic, or long term load cases for which reductions due to bond breakage, concrete creep, etc. should be evaluated.
3. All fire rated assemblies are subject to an upper live load limit of 250 psf.
4. Inquire about material availability of 17, 19 & 21 gage.

References

LRFD Manual of Steel Construction 3rd Edition 2001

ACI 318-05

Nitterhouse Concrete Products Precast Catalog

Girder-Slab Systems Design Catalog

Vulcraft Deck Catalog

RS Means Assemblies Cost Data 2002