Executive Summary:

Pro-Con Structural Study of Alternate Floor Systems is an investigation into possible alternative structural systems for Lexington II in Washington, D.C. For this report, several structural systems were designed and analyzed for the existing conditions in Lexington II. The results of the system designs were then compared with various criteria to determine which system is structurally the most feasible redesign for Lexington II.

The existing system for Lexington II is a two-way flat plate slab system. This system is structural the thinnest possible floor slab, an important consideration in Washington, D.C. where zoning requirements restrict height.

Other systems evaluated and designed as alternatives for Lexington II include;

- One-Way Flat Slab
- One-Way Joist System
- Concrete Slab on Steel Deck and Steel Framing
- Composite Slab
- Pre-cast Floor Slab

The first issue to arise was the need to regulate the existing column grid. To design the alternative systems, a new column grid was assumed with larger spans. Other issues looked at were effects of each system on foundation, lateral design, vibrations, and fireproofing. All designs proved to be either lighter or similar to the existing two-way slab in weight, creating no dramatic change in foundation. Also, all of the alternative designs can work well with the existing shear walls as lateral support. Some systems may be able economize the lateral system by redesigning the framing system as either moment or braced frames. Fireproofing and vibrations caused no major issues among any of the floor system analyzed.

The controlling factor in determining feasibility of a new structural system was floor sandwich depth. This found that either a one-way joist system or a composite system were the best choices for a building redesign.

Introduction:

Lexington II at Market Square North is a residential tower located in the historic quarter of Washington, D.C. With a floor area of 72,000 square feet, Lexington II is 12 stories high with three below grade levels of parking and retail.

Although a larger metropolis building, Lexington II is primarily residential and therefore has residential loads most of its levels. The only large loads on Lexington II are those of the public areas such as lobbies and retail spaces. There is also one loading dock for trucks which would carry a larger load.

From ASCE7-02

Dead Load:

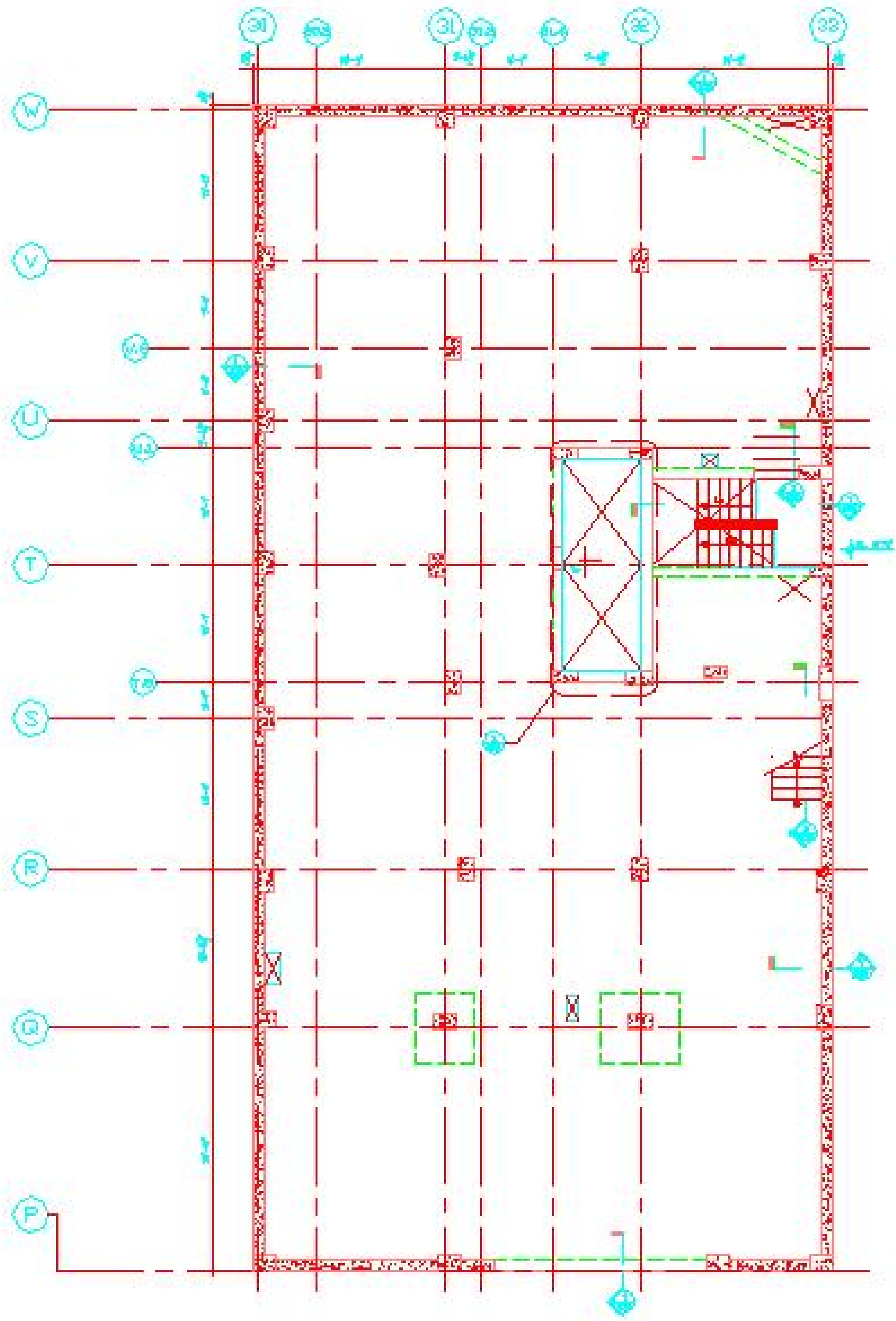
Substructure Slab (10”)…………….125psf
Superstructure Slab (8”)……………100psf
Mechanical/ Lighting…………………5psf
Finishes……………………………...15psf
Partitions……………………………included in live load, see below

Live Load:

Roof………………………………...20psf
Public Levels/ Stairs………………100psf (ASCE7-02)
Mechanical………………………..150psf (Common assumption)
Lobbies……………………………100psf (ASCE&-02)
Residential Levels…………………40psf + 20psf (for partitions)

Washington DC has strict height requirements on all of the buildings constructed there. In the downtown district where Lexington II is located, a height cap of 130’ has been placed on all buildings. Because of this restriction, concrete structural systems which reduce the total amount of floor sandwich in a building are commonly used.

In keeping with DC practice, Lexington II currently uses a two-way flat plate slab across small sized bay to minimize the depth of its floor sandwiches.



Existing Lexington II column plan

Existing Structural System:

The basic structural system of Lexington II is two-way flat plate slab supported by columns. The existing system of Lexington is complicated by offset columns in many locations. Other irregularities of the structural system are the presence of edge beams along the flat-plate slab in a very limited number of locations and drop panels located beneath the truck bay.

FOUNDATON:

The foundation of Lexington II is a MAT foundation with thickness of 3'-6". This foundation is constructed of 5000psi compressive strength concrete and reinforced with grade 60 reinforcing bars. The reinforcing bars are #8 bars located every 9" o.c. with #11 top bars placed where needed. The MAT foundation sits on original soil and structural fill of 8000psf except for the southern wall which on HP 14 x 89 piles every 5' on center. These piles are to avoid costly controls which would be needed to prevent undermining the preexisting building to which Lexington II abuts.

GRAVITY SYSTEM:

The floors of Lexington II are two-way flat plate slab, and in most cases, unsupported by edge beams. The flat plate slab is 8" thick for all residential floors and is increased to 10" where greater loads occur on the lobby and parking levels. All slabs are constructed of 4000psi strength concrete. Typical slab reinforcement is a continuous bottom mat of grade 60 #4 bars every 12" with top reinforcement placed where needed.

LATERAL SYSTEM:

The primary component of the lateral system is a core of shear walls located around the elevator and stair shafts. All shear walls are 12" thick and constructed of 4000psi concrete. Shear wall reinforcement includes #4 bars every 12" on center.

However, the shear wall system alone is not enough to provide for the lateral loads Lexington II sees. The additional lateral strength in Lexington II comes from the framing system of monolithically poured columns connecting into the floor slabs.

ADVANTAGES:

The current gravity system in Lexington II is a sensible choice for many reasons. Due to the small spans and light loads present, the existing floor system can be relatively very thin compared to most common structural systems. Two-way slab can also be made thinner than other systems because beams are not needed to support the floor. The concrete system also eliminates the need for additional fireproofing added to the system.

DISADVANTAGES:

Although two-way slab is thin, it does not provide space for any MEP systems to run in, therefore a suspended ceiling must be run throughout the building adding additional depth to the floor sandwich. Another disadvantage of the current systems of Lexington II is the irregular placement of columns and beams to try and ensure a small floor sandwich. This irregular system prevents future changes in the floor layout. Two-way slab also has a slow construction time. Form work must be placed throughout the building and the concrete must be poured, cured, and finished on time. Many alternative methods provide members produced offsite which must only be fastened together on site.

Design of Alternative Systems:

The following is an investigation into other possible building systems which may have been used in Lexington II. Each system has been evaluated and compared with the existing system and the other alternative systems.

Systems Evaluated:

- One-Way Flat Slab
- One-Way Joist System
- Concrete Slab on Steel Deck and Steel Framing
- Composite Slab
- Pre-cast Floor Slab

The above systems were designed for a common floor with residential loading. The several systems, which could easily be designed for multiple loads, were also designed for the lobby as a brief comparison in the size and weight of members needed to carry the additional live load of the lobby.

Criteria for determining feasible structural systems were based primarily on member size and weight and their effect on the floor sandwich. Due to the Washington, D.C. zoning restrictions, systems which greatly increased the buildings total height are impractical to consider. Other criteria the floor systems were compared with included ease and time of construction, effect on floor vibration, if fireproofing is required, and if the system works well with the existing foundation and lateral systems. Material cost can be estimated based on the weight of each system, remembering that in general steel costs more per pound than concrete. Construction cost can be thought of as greater for concrete due to the need for formwork and additional labor.

One-way slab:

The first system analyzed for the Lexington II redesign was a monolithically poured one way concrete slab. One way slab was chosen for its ability to work with an irregular column grid and short span ranges. Other concrete systems such as one-way joist and waffle slabs were deemed unreasonable for the short span lengths in Lexington II.

Before starting the analysis, the only argument against one way slab was the almost square bays of Lexington II which are usually more characteristic of two-way slabs.

In order to design a one way slab, design aids from the CRSI handbook (2002) were utilized.

LOADS:

Loads were determined in technical assignment #1 using ASCE 7-02. For designing one way slab using the CRSI handbook, loads were adjusted so that self weight was not included. Self weight was already taken into account by CRSI. The values in the CRSI tables are 'total factored load was calculated, and reduced by the prescribed φ-factors, from which 1.4 times the slab weight was deducted using a unit weight of 150 pounds per cubic foot (pcf).'[1]

Live load: 60psf for residential floors
Dead load: 20psf for finishes and MEP

W_u= 1.2*(20) + 1.6*(60) = 120psf
W_u= 1.4*(20) + 1.7*(60) = 130psf (old LRFD factors)

SPANS:

Spans for each bay were determined based on the largest span caused by offset beams. The one way slab was then taken to span the shorter direction in bay. Two spans seemed to prevail in almost every bay, 13' to 13'-9" and 16'-1" to 16'-6". As a conservation measure, I took the lengths which the one way slab spanned to be 13'-9" or 16'-4". To account for bearing, 4" was added to both of these values.

DESIGN:

Repetitiveness is a key to economical one way slab design. For practicality, it was decided that a uniform slab thickness should be used for each floor slab. Using design aids from the CRSI, the minimum slab thickness for the largest span in an exterior bay of Lexington II was found. The results of this design were a 6 ½" slab with ρ= .005. The

[1] CRSI Design Handbook 2002, Chapter 7- One Way Slabs: scope of load table, page 7-1

other bays were then designed for a 6 ½" slab with the minimum possible amount of steel reinforcement. The final design was as follows:

Exterior Bay, 17' (CRSI p. 7-12)
- 6 ½" slab
- ρ=.005
- Top bars = #5 at 11"
- Bottom bars= #5 at 12"
- Temperature/ Shrinkage bars= #4 at 17"
- Wu= 155 > 130psf

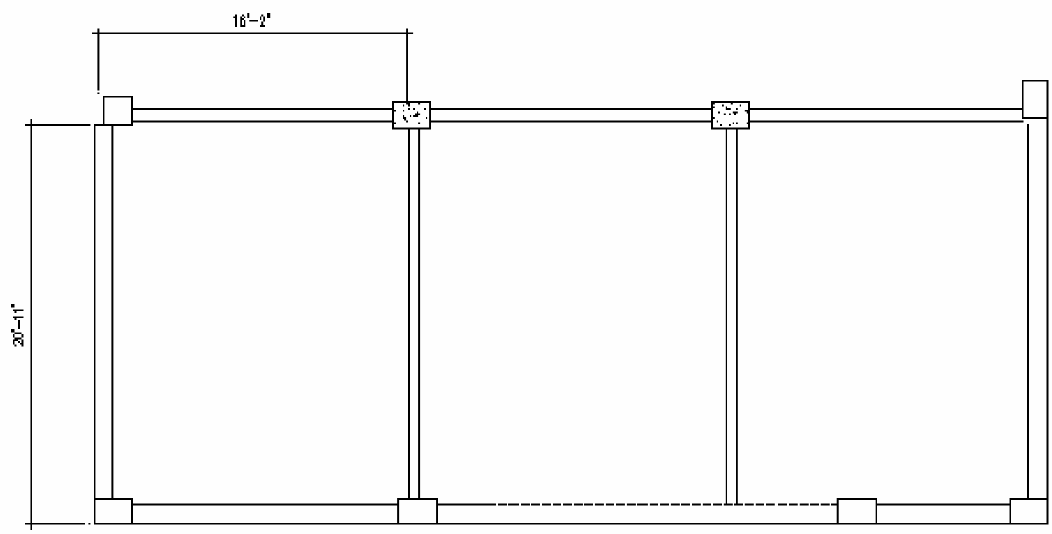
Exterior Bay, 14', 6 ½" slab (CRSI p. 7-17)
- ρ=.005
- Top bars = #5 at 11"
- Bottom bars= #5 at 12"
- Temperature/ Shrinkage bars= #4 at 17"
- Wu= 282 > 130psf

Interior Bay, 17', 6 ½" slab (CRSI p. 7-12)
- ρ=.005
- Top bars = #5 at 11"
- Bottom bars= #4 at 10"
- Temperature/ Shrinkage bars= #4 at 17"
- Wu= 182> 130psf

Interior Bay, 114', 6 ½" slab (CRSI p. 7-14)
- ρ= minimum
- Top bars = #4 at 12"
- Bottom bars= #3 at 9"
- Temperature/ Shrinkage bars= #4 at 17"
- Wu= 154> 130psf

Deflections and ultimate stresses are taken into consideration in the CRSI design aids; therefore no additional checks of these values were required.

Finally, concrete beams are needed to span in the 26' length direction of the bays and to support the floor slabs. The load the beams carry includes the weight of the slab. The design of the beam, based on ultimate moments finds that the concrete cross section must be at least bd^2=6190in^3. This leaves several options such as a square beam 18" x 18", or the common standard of d=2b dictating a beam of 10" x 25". The final beam design was 18" by 18". This beam was chosen to reduce the floor sandwich without creating an overly wide beam. This will add an addition 18" to the floor sandwich.



Typical large bays spanned by beams

The beams were then checked with a deflection criteria of l/240.

l/240 = (26.6*12)/240= 1.27 in.

Actual deflection= 1.244 in < 1.27

This deflection is very close to the allowable, and therefore the beam will be redesigned to 15” x 20”, deflection = 1.08 inches.

ADVANTAGES:

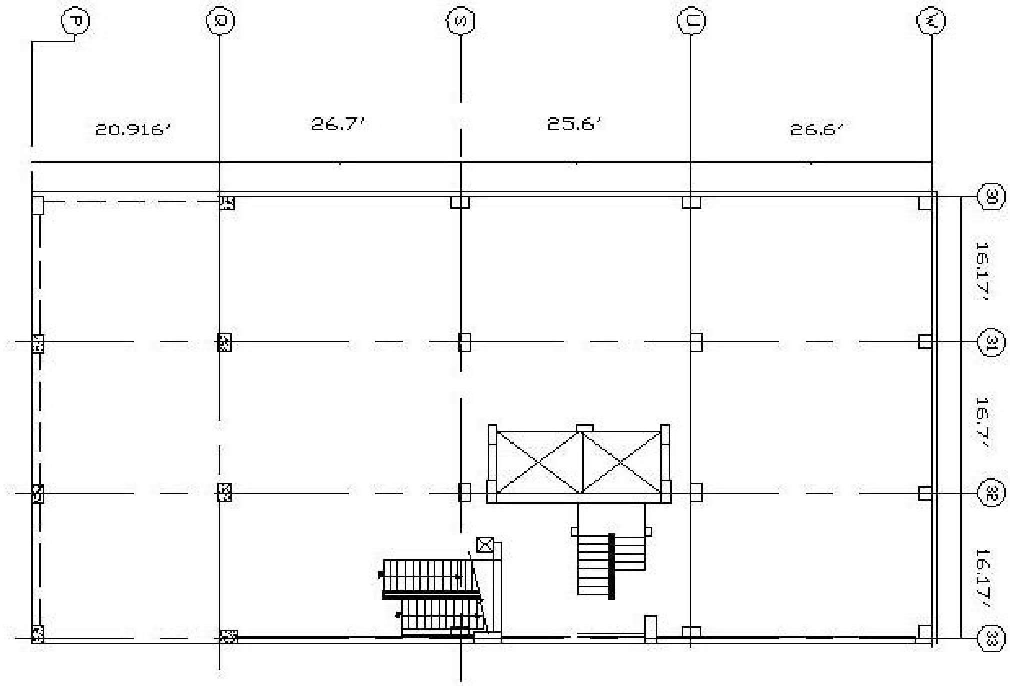
- Works well on the smaller sized bays in Lexington II
- Simple construction and formwork
- No additional fireproofing
- No change in foundation or lateral system necessary

DISADVANTAGES:

- A suspended ceiling is needed to hide MEP systems
- Thin slab means vibration could become a problem
- Floor sandwich= 6.5”slab plus 20” beam
- Total weight=112psf

One-way Joist Floor:

Another concrete system to consider is a one way joist floor system. One way joist floors are known for their ability to reduce dead weight and reinforcement. However, one way joist construction is usually more efficient for longer spans. To remedy this, the column gird of Lexington II was changed for this analysis. Every other row of columns was eliminated, and the remaining columns were moved into an exact grid formation.[2]



New Bay Lay Out

LOADS:

Loads were determined in technical assignment #1 using ASCE 7-02. For designing one way joist floors using the CRSI handbook, loads were adjusted so that self weight was not included. Self weight was already taken into account by CRSI. An additional 2psf is added to the final self weight to account for bridging.

Live load:	60psf for residential floors
Dead load:	20psf for finishes and MEP

[2] All subsequent systems are designed using this column grid

W_u= 1.2*(20) + 1.6*(60) = 120psf
W_u= 1.4*(20+69) + 1.7*(60) = 227psf (old LRFD factors)

SPANS:

As determined above. Although the bays are 16.17' and 16.7', each bay was approximated so that the joists would span 16'. This reduction in span can be accounted for when the area of the columns is subtracted from the center to center distance, thus leaving a face to face span of 16' for each bay.

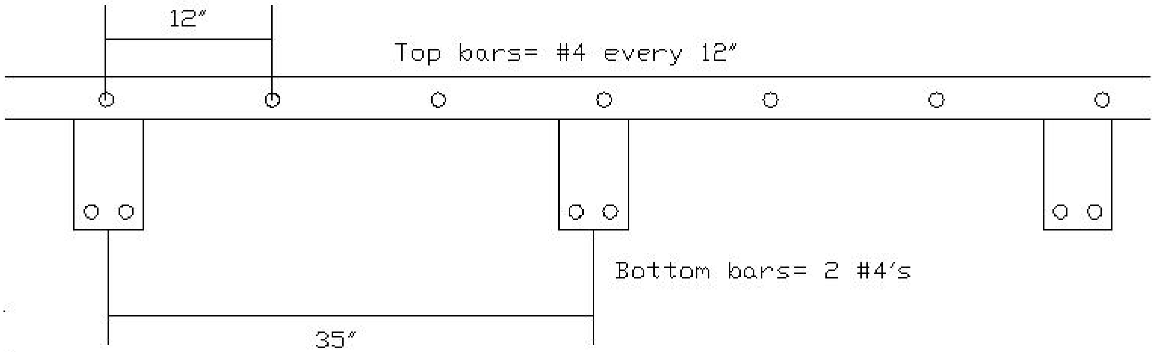
DESIGN:

Design for the one way joist floor was determined from design aids provided in the CRSI handbook. The main criteria while designing the one way joist floor was the depth of the floor sandwich. The current system, two way flat plate, has a very small floor sandwich of 8". Lexington II is located in Washington D.C. where there is a height restriction on buildings. The smallest floor depth in the CSRI handbook is 3" slab + 8" joists, this is the system chosen for the following analysis.

Try 3" slab with 8" deep ribs= 11" total depth
30" forms + 5" Rib @ 35" c-c
Concrete strength= 4000psi
Steel strength= 60,000psi

End Span: (CRSI p. 8-20)
Clear span=16' W= 180psf > Wu=130psf
Top Steel= #4 bars at 12"
Bottom Steel = two #4 bars

Interior Span: (CRSI p. 8-20)
Clear span=16' W= 131psf > Wu=120psf
Top Steel= #4 bars at 12"
Bottom Steel = one #3 bar and one #3 bar



Joist Section for end span

Finally the girders to support the joist system had to be designed. To do this, the previous load plus the weight of the one-way joist system had to be factored. The girder was designed to meet the flexural strength required to support the loads. The girder was designed with a depth equal to that of the joists in order to prevent additional depth to the floor sandwich. The final girder design was 26" base and 8" deep.

Deflection for the girder was checked and compared to l/240.

l/240 = (26.6*12)/240= 1.27 in.

Actual deflection= .8 in < 1.27in

Therefore a 16" x 8" girder will work.

ADVANTAGES:

Reduces vibration
Is fire rated
Additional stiffness
Additional weight (Total weight = 75.0psf)
Works well with current shear wall system

DISADVANTAGES:

3" slab + 8" rib is deeper then the current floor system (however MEP system can be easily integrated into the joist system without the need for additional space provided by a suspended ceiling)

New column grid may change foundation. Reduction in number of columns results in each column carrying more weight and an increase in punching shear.

Steel Beams with Metal Deck and Concrete Slab:

Wide flange steel beams with metal form deck and reinforced concrete slab was another system analyzed for Lexington II. A non-composite system was analyzed to provide incite into how other materials besides concrete would perform in Lexington II. Steel had many benefits such as its strength in tension, its strength to weight ratio, and its long life time. Steel is also easy to fabricate off site and then erect quickly saving on time and labor. Steel, however requires thicker floor sandwiches then the existing flat plate system.

To economize the steel system, the larger bay span option (eliminating every other row of columns) was used. This is done to utilize the strength of the steel. A steel beam system was decided on in place of a steel joist system to help reduce the floor sandwich depth as much as possible. The floor sandwich depth is especially important in Lexington II because of the Washington D.C. building height requirements.

LOADS:

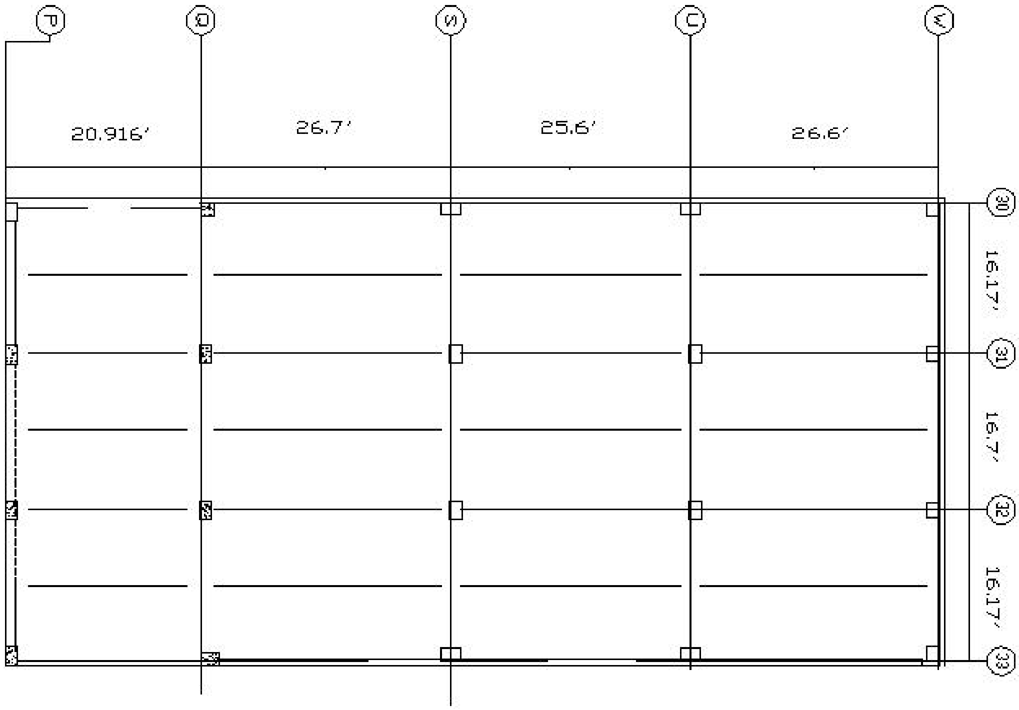
Loads were determined in technical assignment #1 using ASCE 7-02. To design using steel decking catalogs, a self weight for the slab had to be assumed and added to the design weight from technical assignment 1.

Live load: 60psf for residential floors
Dead load: 20psf for finishes and MEP
Slab Weight: Slab + Deck with concrete weight
(4.5”/12)*(150pcf) + (2”/12)*(150pcf)/2 = 69psf

W_u= 1.2*(20+69) + 1.6*(60) = 203psf (current LRFD factors)
W_u= 1.4*(20+69) + 1.7*(60) = 227psf (old LRFD factors)

SPANS:

Spans for a typical bay of 26.6’ by 16.7 were assumed to be divided by two steel beams. An average span of 9’ for the steel decking was used.



Beam Spacing in Bays

DESIGN:

The first step in designing the steel beam system was to determine the steel decking. This was done using the Design Manual and Catalog of Products by United Steel Deck (USD), Inc. USD accounts for deflections due to live load and stresses, therefore no checks were used on the deck. The above load, including the assumed slab and deck weight, was used at a span of 9', in a triple span condition.

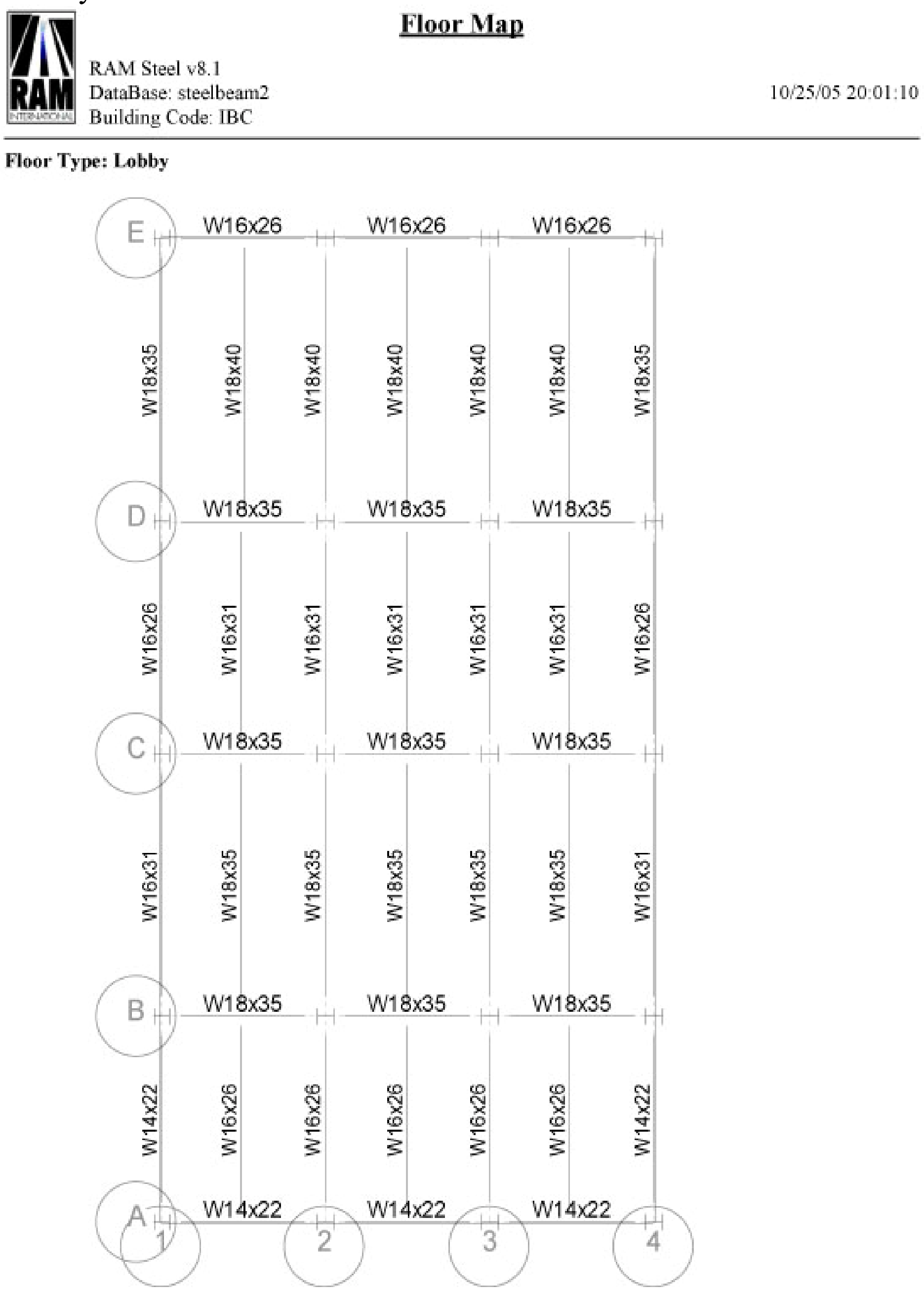
Decking: 18 gauge, UF2X 282psf > 200psf

Next the slab was designed. The slab design was also determined from the USD manual.

Slab: 6" Mesh: 44- W4.0 x 4.0

Before the steel beams could be designed, the weight of the slab and deck had to be compared to the assumed weight of 69psf. According to the USD manual, the weight of the concrete slab and deck system is 60psf. This makes the assumed 69psf conservative and the design can continue.

The last item to be designed for this system is the steel beams. The design for the steel beams was completed in RAM . RAM is written to include sizing members for allowable deflection and stress, therefore no deflection or stress checks of the members were necessary. Results are below.



Lobby Beam Sizes
*Lobby was designed with a live load of 100psf

Floor Map

RAM Steel v8.1
DataBase: steelbeam2
Building Code: IBC

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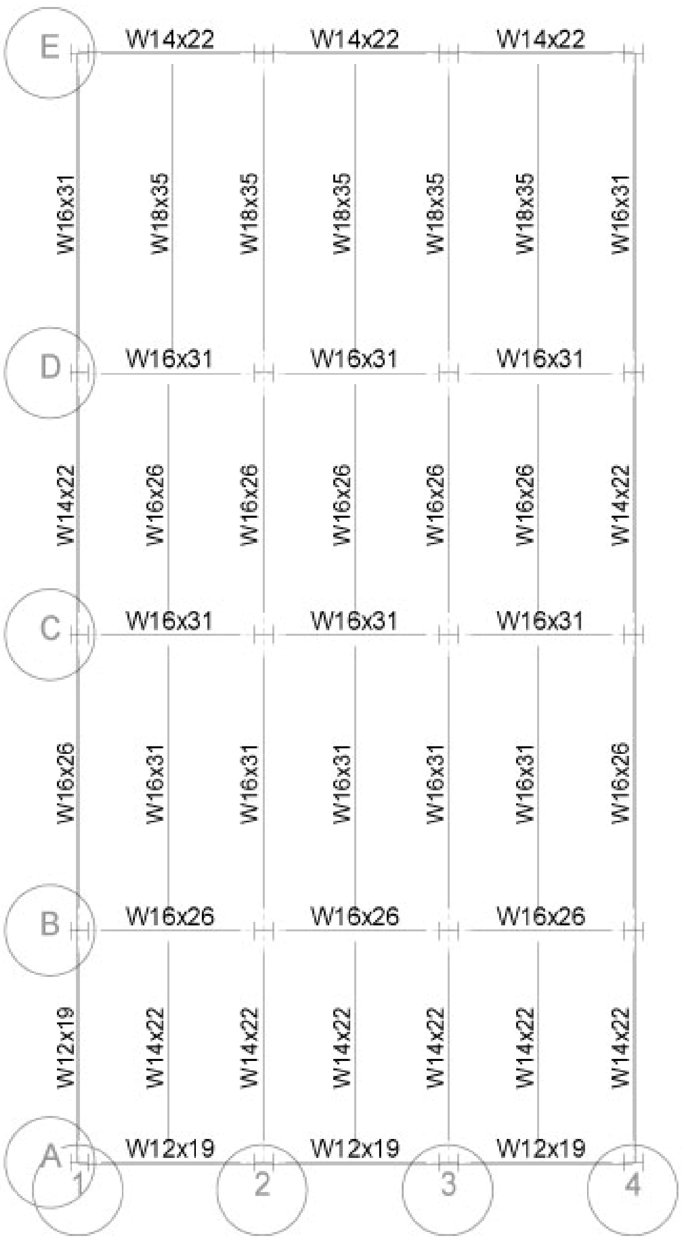
Floor Type: Residental



Residential Beam Sizes

ADVANTAGES:

Fabricated offsite
Rapid erection
Ductile
Good strength in both tension and compression
Long life time
Weight= 67.5psf

Designing a new foundation and lateral systems rather then 3’-6” MAT and concrete shear walls may prove to be more economical

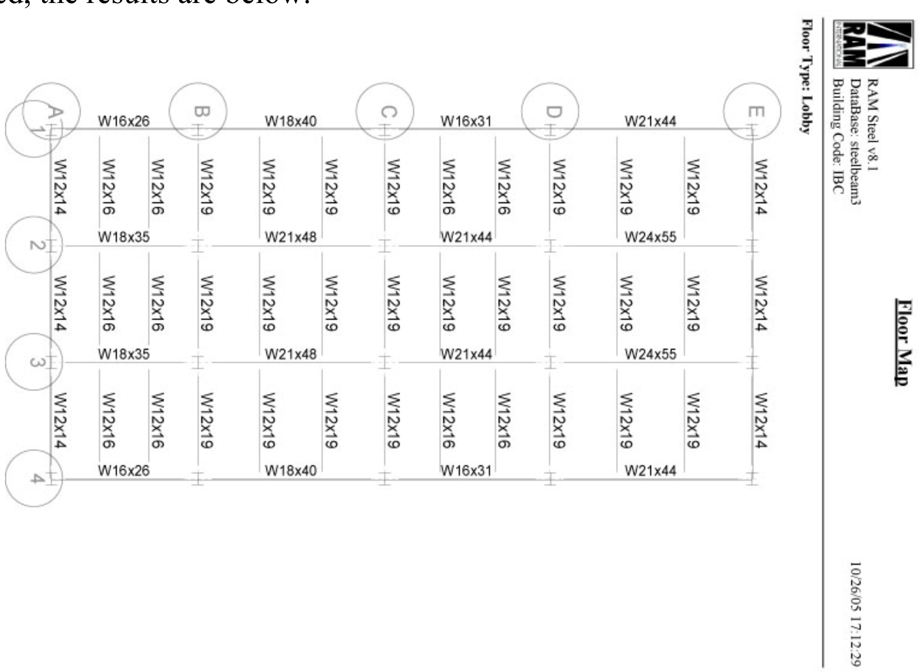
DISADVANTAGES:

Floor sandwich was greatly increased. This can be demonstrated by looking at the depth of almost any steel member. For example, a W16 x 26 is 15.7” deep plus there is a 8” slab and deck assembly on top.

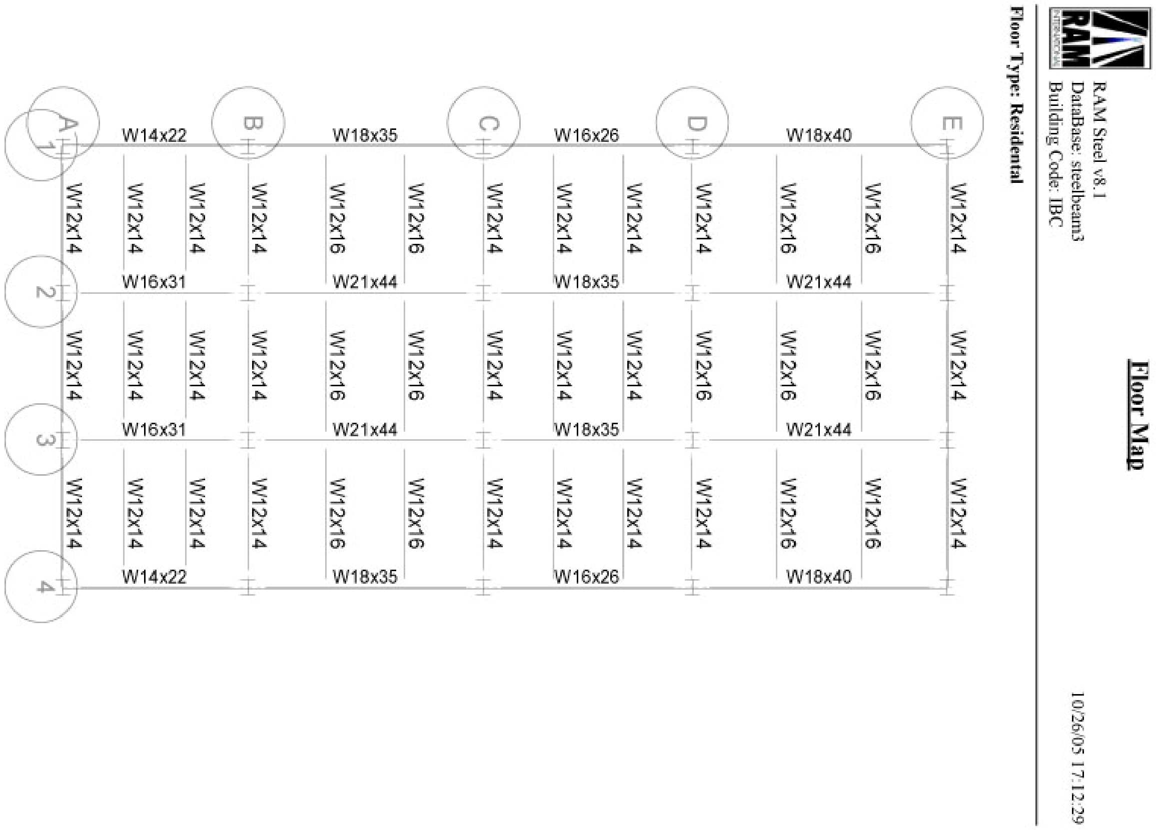
Requires fireproofing
Requires lateral bracing

ALTERNATIVE SPAN:

In an attempt to try and reduce the beam depth, an alternate beam layout was also analyzed, the results are below.



Lobby Beam Sizes



Residential Beam Sizes

As you can see, while most of the beams were reduced to W 12 shapes, the girders increased in size to W21's. This will not help reduce the depth of the floor sandwich as wanted.

Composite Beams and Deck:

Composite wide flange steel beams with composite metal deck and a concrete slab was another investigated system. Using a composite system will hopefully achieve all the benefits of a steel system as well as the strength and stiffness of a concrete system and help to reduce the floor sandwich size.

The spans used to analyze the composite system were the same as used in the steel beam non-composite system design. The only analysis done for the composite system was preformed on the beam spacing which worked the best for the non-composite system.

LOADS:

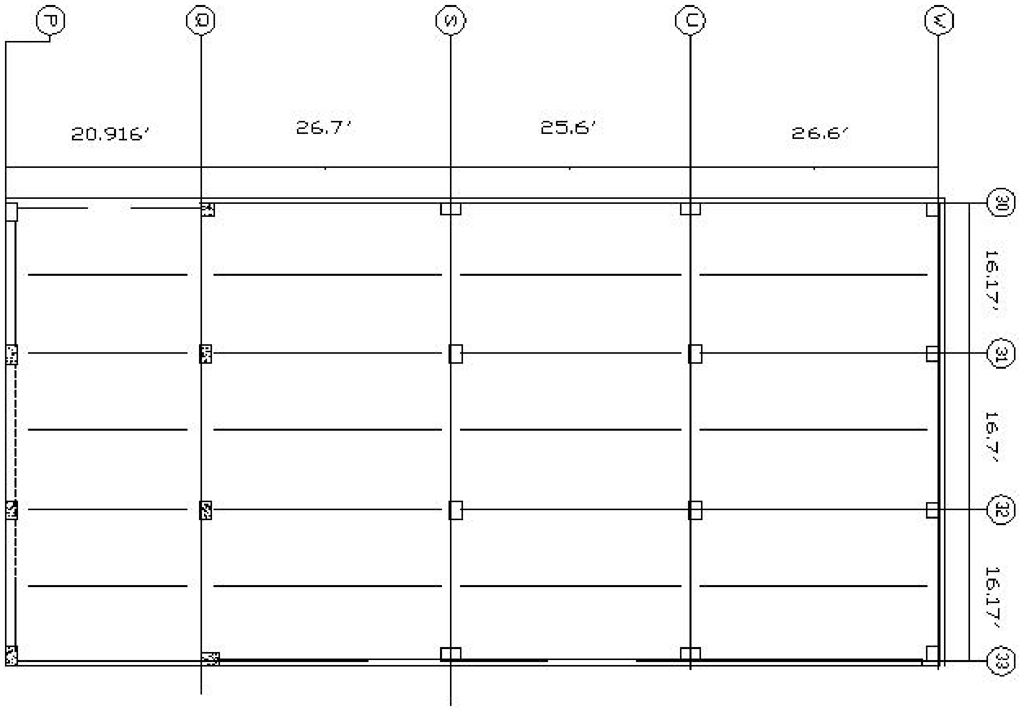
Loads were determined in technical assignment #1 using ASCE 7-02. To design using steel decking catalogs for composite deck, only the uniform live load must be considered.

Live load: 60psf for residential floors

W_u= 1.6*(60) = 96psf

SPANS:

Spans for a typical bay of 26.6' by 16.7 were assumed to be divided by two steel beams. An average span of 9' for the steel decking was used.



Beam Spacing in Bays

DESIGN:

To design the composite system, first the composite deck was picked from the USD catalog. USD accounts for deflection and stresses, therefore no checks were run on this system. To keep shallow floor sandwiches, the 1.5" loc-floor was used with normal weight concrete and a compressive strength of 3ksi.

Before a slab depth was picked, the fire proofing tables were reviewed and it was determined that with many assemblies, 1.5" + 2.5" of cover would be needed to ensure that additional fireproofing on the system was not necessary

Decking: 22 gauge, 1.5" loc, slab=4", one stud per foot 165psf>60psf

The required area of reinforcing steel was A=.023. Therefore 6x6 W4.0x4.0 welded wire mesh was used in the design. The total weight of the composite deck was 31psf, significantly less then non-composite deck.

The composite beams were designed using RAM. Ram designed the beams with shear studs varying in number per beam from 8 to 22. RAM is written to include sizing members for allowable deflection and stress, therefore no deflection or stress checks of the members were necessary. The RAM results are below.

Floor Map

RAM Steel v8.1
DataBase: composite
Building Code: IBC

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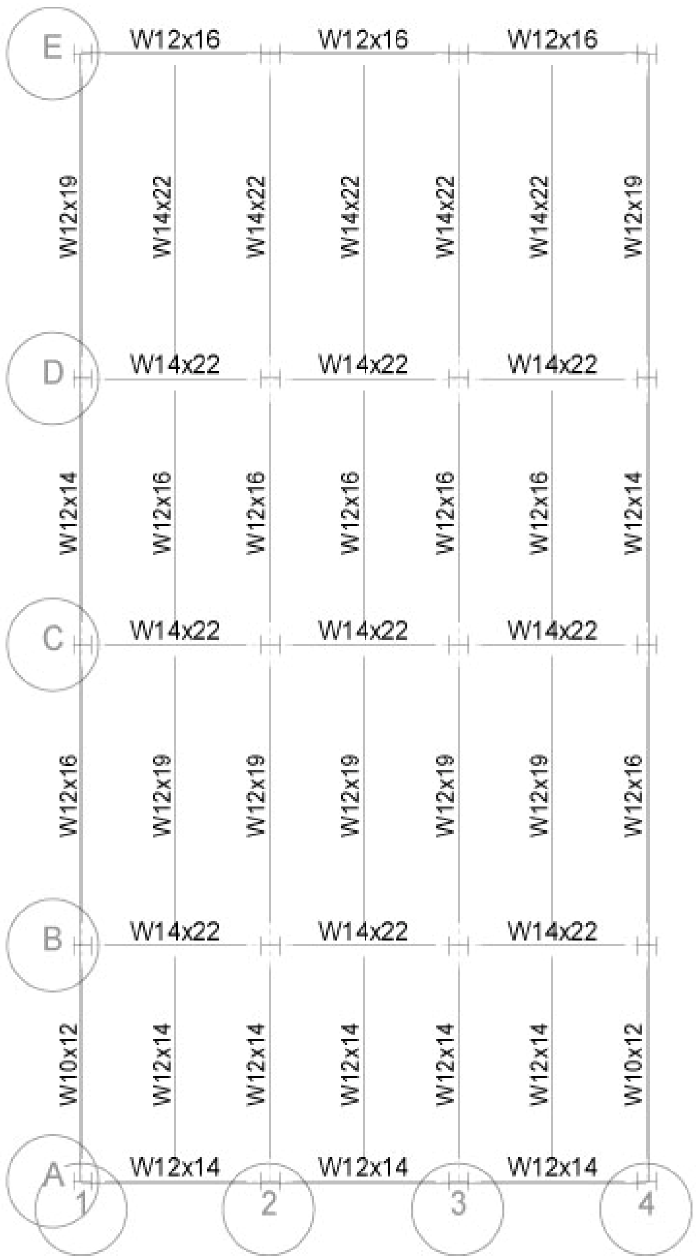
Floor Type: Lobby



Lobby Beam Sizes

Floor Map

RAM Steel v8.1
DataBase: composite
Building Code: IBC

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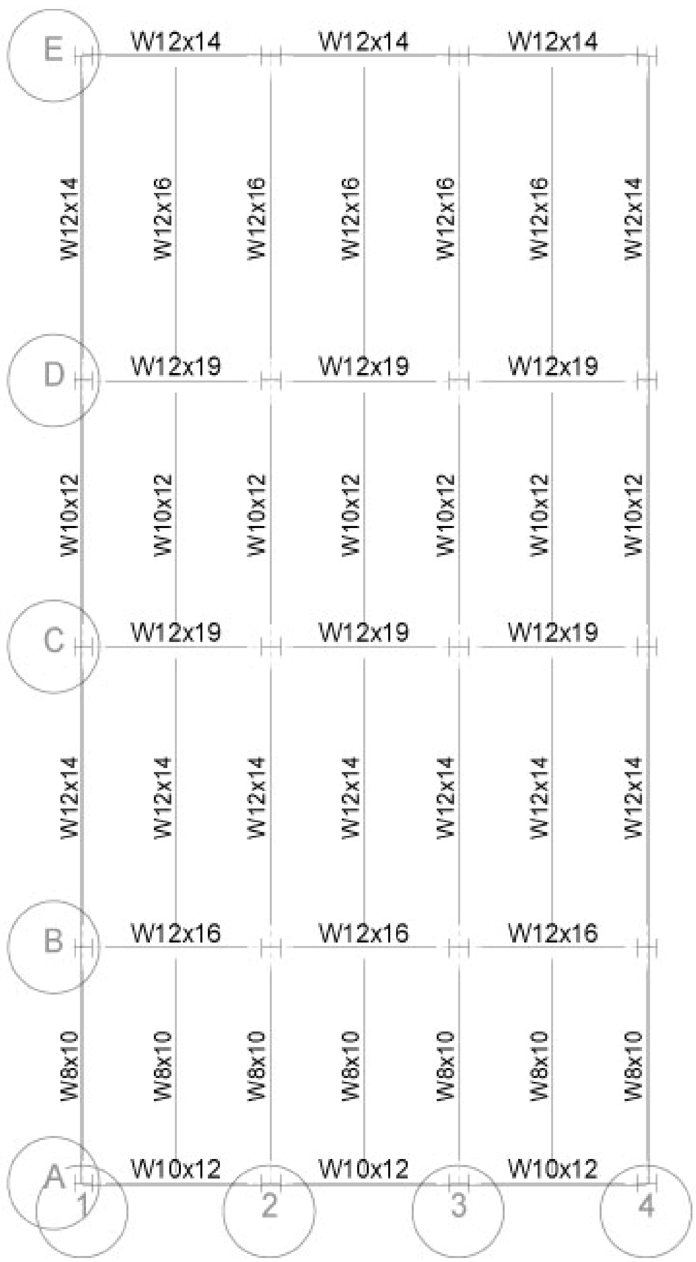
Floor Type: Residental



Residential Beam Sizes

ADVANTAGES:

Smaller floor sandwich then the non-composite steel system
Lighter slab weight then existing system
No shoring is needed
A more economical foundation and lateral system may be possible

DISADVANTAGES:

Additional cost and labor of shear studs

Pre-cast Concrete:

The last structural system considered for Lexington II was pre-cast concrete floor slab and beams. Pre-cast systems can be manufactured to the exact specs needed for the building. For this analysis, common pre-cast shapes as found in the PCI handbook were used.

LOADS:

The PCI handbook directs that 'load tables for stemmed deck member, flat deck members, and beams show the allowable uniform superimposed service load'.

Live load:	60psf for residential floors
Dead load:	20psf for finishes and MEP
Total load:	80psf

SPANS:

Pre-cast floor slabs can be adjusted for any span necessary. The shapes given in PCI are 4' wide and the spans range from 10' to 30'. Lexington's bays are 26.7' by 16.6', therefore the option exists to span the 16' direction or add beams and span a shorter direction.

DESIGN:

The design of the pre-cast concrete system was done using design aids found in the PCI handbook. Due to the building height restrictions in Washington DC, the floor sandwich depth was a critical part of design. To keep the floor sandwich as thin as possible, solid flat slab and hollow-core slabs were investigated instead of T beams.

Another way to keep thinner floor spans was to divide the 16.6' or 20'span directions with additional beams. The more beams carrying the weight, the smaller each beam can be. Another consideration was the amount of reinforcement steel needed in each piece. To reduce costs, the minimum amount of steel should be used.

Possible slabs:	4HC6 with 66-S strains. Meets 80psf at spans of 20 feet and less. 6" thick.
	4HC6+2" with 66-S strains. Meets 80psf at spans of 21 feet and less. 2" topping add depth making the floor sandwich 8" deep.
	FS4 with 66-S strains. Meets 80psf at spans of 14 feet or less. Total depth is 4".

FS4+2 with 66-S strains. Meets 80psf at 15 feet or less.
Total depth is 6".

To design the beams the weight of the pre-cast slab must be included. 4HC6 is 45psf and FS4 is 50psf, when topping is included, those weights increase to 74psf and 75psf. A conservative assumption is to use 75psf as the slab weight.

Live load: 60psf for residential floors
Dead load: 20psf for finishes and MEP
Slab load: 75psf

Total load: 155psf

All of the beams listed in the PCI design tables are more then adequate to meet both the load and the span requirements. Therefore a smaller, custom made pre-cast beam would be more efficient.

Using Microsoft excel, a spreadsheet was designed to find beam section dimensions (b and d) for rectangular beams ranging in spans from 3' to 20'. To help determine a feasible beam dimensions, a quick estimation of the base dimension was run with the assumption that at least 2 #4 reinforcement bars would be need.

b > Cover (with stirrup) + spacing between bars + cover (with stirrup)
b > 2.5 + 1 + 2.6
b > 6in.

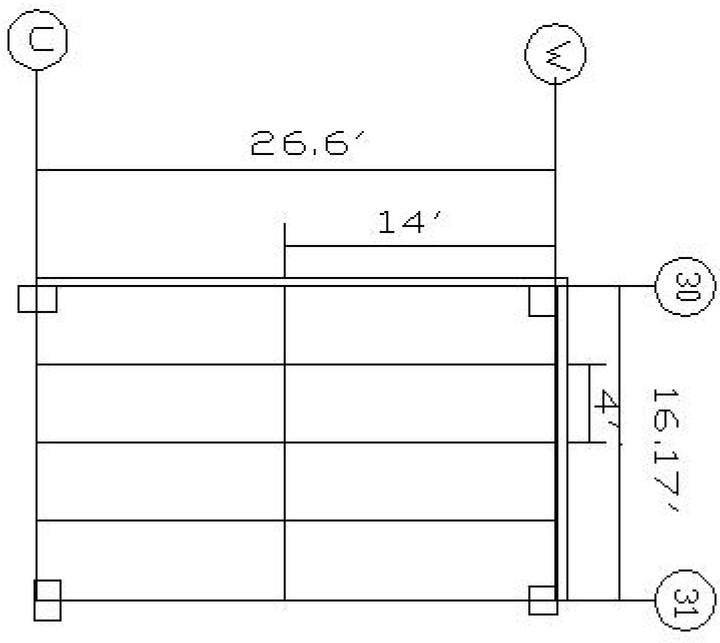
Span (ft)	Wu (plf)	Mu (kip-in)	bd^2 (in^3)
3	630	8.505	10.37195
4	840	20.16	24.58537
5	1050	39.375	48.01829
6	1260	68.04	82.97561
7	1470	108.045	131.7622
8	1680	161.28	196.6829
9	1890	229.635	280.0427
10	2100	315	384.1463
11	2310	419.265	511.2988
12	2520	544.32	663.8049
13	2730	692.055	843.9695
14	2940	864.36	1054.098
15	3150	1063.125	1296.494
16	3360	1290.24	1573.463

if d= 5in	if d=10in	if d=15	if d=20	if d=b	if d=6	d=7	d=8	d=9
b(in)	b(in)	b(in)	b(in)	b(in)	b(in)	b(in)	b(in)	b(in)
0.414878	0.10372	0.046098	0.02593	2.180822	0.28811	0.211672	0.162062	0.128049
0.983415	0.245854	0.109268	0.061463	2.907762	0.682927	0.501742	0.384146	0.303523
1.920732	0.480183	0.213415	0.120046	3.634703	1.333841	0.979965	0.750286	0.592818
3.319024	0.829756	0.36878	0.207439	4.361643	2.304878	1.69338	1.296494	1.02439
5.270488	1.317622	0.58561	0.329405	5.088584	3.660061	2.689024	2.058784	1.626694
7.867317	1.966829	0.874146	0.491707	5.815524	5.463415	4.013937	3.073171	2.428184
11.20171	2.800427	1.244634	0.700107	6.542465	7.778963	5.715157	4.375667	3.457317
15.36585	3.841463	1.707317	0.960366	7.269406	10.67073	7.839721	6.002287	4.742547
20.45195	5.112988	2.272439	1.278247	7.996346	14.20274	10.43467	7.989043	6.312331
26.5522	6.638049	2.950244	1.659512	8.723287	18.43902	13.54704	10.37195	8.195122
33.75878	8.439695	3.750976	2.109924	9.450227	23.4436	17.22387	13.18702	10.41938
42.1639	10.54098	4.684878	2.635244	10.17717	29.28049	21.5122	16.47027	13.01355
51.85976	12.96494	5.762195	3.241235	10.90411	36.01372	26.45906	20.25772	16.0061
62.93854	15.73463	6.993171	3.933659	11.63105	43.70732	32.1115	24.58537	19.42547

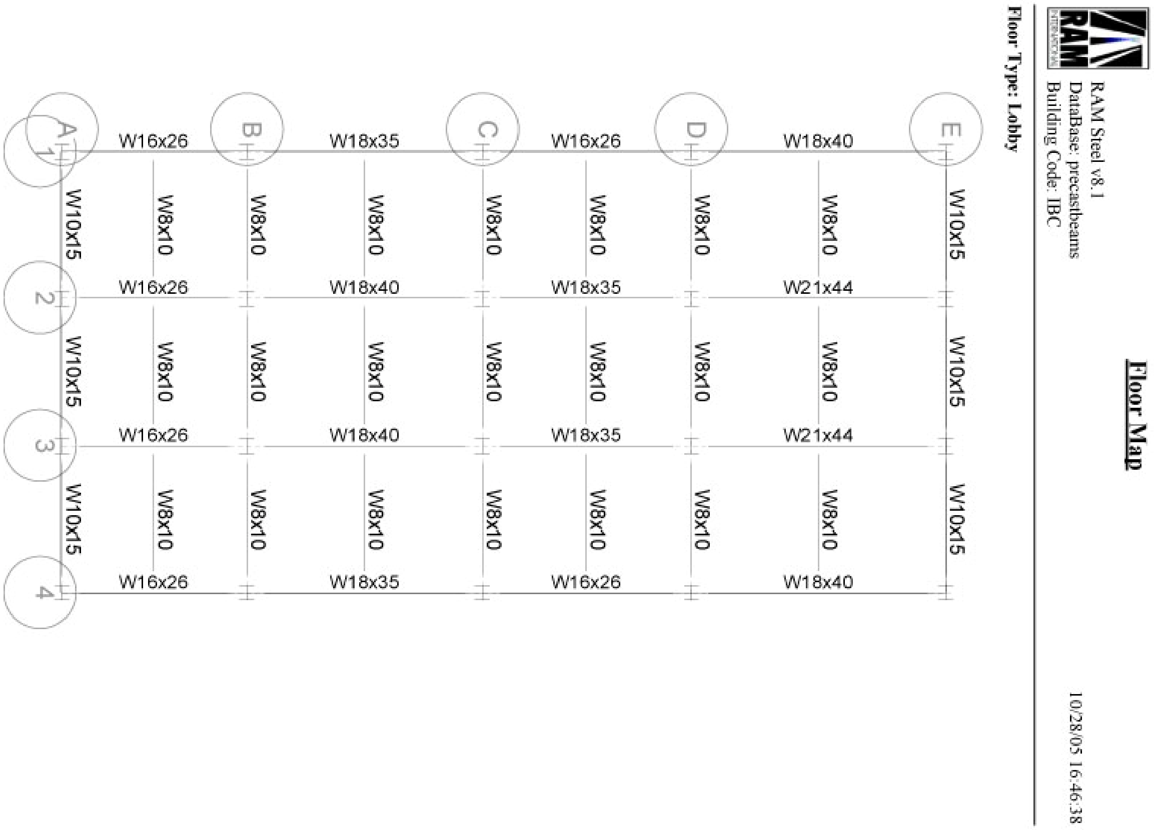
Possible beam sizes with b>6 have been highlighted

The final pre-cast design decided on was to use 4 4’ pre-cast floor slab panels along the 16’ bay width. The panels decided on were FS4 with 66-S strains. These panels were chosen without topping to ensure the minimum slab thickness. The span picked was the slab’s maximum span of 14 feet. 14 feet was picked so that only one beam per bay would be needed. By reviewing the excel chart above, the only possible concrete shape to span the entire 16’ bay weight was a d=15” by b= 7”. The depth of 15” increases the floor depth greatly, and by observation a base of 7” does not appear large enough to fix the reinforcement requirements.

A more feasible beam system is to use a steel beam, which would be smaller, to support the pre-cast panels. The steel beam design was done using RAM. RAM is written to include sizing members for allowable deflection and stress, therefore no deflection or stress checks of the members were necessary.



Location of Pre-Cast Slab Panels in a typical bay



Steel Beam Design for a typical residential floor

ADVANTAGES:

- Fast to construct
- Weight of system = 54psf
- Lighter weight and steel beams may provide for a new foundation

DISADVANTAGES:

- Steel girders require fireproofing
- Lighter system may cause vibration problems
- Difficult connections to connect concrete to steel system

	Floor Depth	Weight[1]	Fireproofing	Vibration	General Comments	Feasibility
Existing System: Two-Way Flat Plate	8" floor slab with suspended ceiling for MEP space	100psf	No additional fireproofing is required			
One-Way Slab	6.5" slab + 20" beam = 26.5"	112psf	No additional fireproofing is required	Heavier then existing system, will dampen vibrations	• Works with existing column layout. • Rearranging bay sizes may help to reduce beam depth, however bay sizes are already very small • Simple formwork and construction	Increased weight and floor depth make more analysis unnecessary without alternating the column gird.
One-Way Joist	3" slab + 8" ribs =11"	75psf	No additional fireproofing is required	Joists add more stiffness	• Form work is easy to erect • Larger columns and punching shear will result	Should be investigated
Steel with Non-Composite Deck	8" slab + 16" beam = 24"	67.5psf	Additional fireproofing is required	Lighter system may cause vibration issues	• Lateral Bracing required • Complex connections • Possible foundation and lateral system redesign	Possible for investigating, however floor sandwich may become a problem
Composite with Composite Deck	4" slab +1.5" deck + 12" beam =17.5"	35psf	Additional fireproofing required on steel beams	Usually no vibration problems with composite	• No shoring required • Extra cost and labor of shear studs • Possible foundation and lateral system redesign	Should be investigated
Pre-Cast Slab with Steel Beams	4" slab + 18" beam = 22"	54psf	Additional fireproofing is required on beams	Lighter system could cause vibration problems	• Fast to construct, all pieces fabricated offsite	Possible for investigating

[1] All weights are determined for the upper right bay.

Conclusion:

Reviewing the benefits and shortcomings of the evaluated system, it is easy to see that any further investigation into the gravity system of Lexington II should begin with the design of a new column gird. The existing column grid only works well with the exiting two-way slab system.

The existing system is also one of the heaviest systems investigated; however it has the closest column spacing. One can assume any increases in foundation size would be due to increased punching shear caused by each column, which now carries a larger floor area. The other consideration for a foundation redesign would involve a reduction in size due to the lighter structural systems.

Another system which would have to be revaluated for any further investigation is that of the lateral loads. The current system of shear walls can be used with any system, but may not be the most economically lateral system for buildings comprised of steel framing.

However, the controlling criteria for a Washington DC building is that of floor sandwich depth. By looking exclusively at floor sandwich depths, the conclusion can be reached that after the existing system the best two alternatives would be to design either a one-way joist floor or a composite system. Both alternatives are lighter in weigh then the existing two-way slab and have no major setbacks as far as fireproofing, vibrations, or other construction issues.

SOLID ONE-WAY SLABS—END SPAN Top Steel for $-M_u$

$f'_c = 3{,}000$ psi Grade 60 Bars $\rho \approx 0.0050$

Thickness (in.)	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10
Top Bars Spacing (in.)	#4 12	#4 12	#4 11	#4 9	#5 12	#5 11	#5 10	#5 10	#5 9	#6 12	#6 11	#6 10	#6 10
Bottom Bars Spacing (in.)	#4 12	#4 11	#4 10	#4 8	#4 8	#5 12	#5 11	#5 11	#5 10	#5 9	#6 12	#6 11	#6 11
Top Bars Free End Spacing (in.)	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12
T-S Bars Spacing (in.)	#3 15	#3 13	#3 12	#3 11	#4 18	#4 17	#4 15	#4 14	#4 13	#4 13	#4 12	#5 18	#5 17
Areas of Steel (in.^{2}/ft) Top Interior Bottom	.200 .200	.200 .218	.218 .240	.267 .300	.310 .300	.338 .310	.372 .338	.377 .338	.413 .372	.440 .413	.480 .440	.528 .480	.528 .480
Slab Wt. (psf)	50	56	63	69	75	81	88	94	100	106	113	119	125
CLEAR SPAN	FACTORED USABLE SUPERIMPOSED LOAD (psf)												
6′-0″	700	906											
6′-6″	586	761	967										
7′-0″	496	645	821										
7′-6″	423	552	704	988									
8′-0″	363	475	608	856	986								
8′-6″	314	412	528	747	861	976							
9′-0″	272	359	462	656	757	858							
9′-6″	237	314	405	579	669	759	916						
10′-0″	207	276	357	513	593	674	814	890					
10′-6″	158	191	248	364	481	591	722	790	957				
11′-0″	138	167	218	323	429	528	647	708	859	987			
11′-6″	120	146	192	287	383	473	582	636	774	890			
12′-0″	105	127	169	256	343	426	524	574	700	806	952		
12′-6″	91	111	149	228	308	383	473	518	634	731	865		
13′-0″	79	97	131	204	277	346	428	469	575	664	787	937	999
13′-6″	68	84	115	182	249	312	388	426	523	605	719	857	914
14′-0″	58	73	101	162	224	282	352	386	477	552	657	785	837
14′-6″	49	62	88	145	202	256	320	351	435	505	602	721	769
15′-0″	42	53	76	129	182	231	291	320	397	462	552	662	707
15′-6″		45	66	115	163	209	264	291	363	423	507	610	651
16′-0″			56	102	147	190	241	265	332	388	466	562	600
16′-6″			48	90	132	171	219	241	304	356	429	519	554
17′-0″			40	79	118	155	199	220	278	327	395	479	511
17′-6″				69	105	140	181	200	255	300	363	442	473
18′-0″				60	94	126	164	182	233	275	335	409	437
18′-6″				51	83	113	149	165	213	253	309	378	405
19′-0″				44	73	101	135	149	195	232	284	350	374
19′-6″					64	90	122	135	178	213	262	324	347
20′-0″					56	80	109	122	162	195	241	300	321

Note: See Fig. 7-1 for reinforcing bar details.

One-way Slab for an end span. Used to determine thickness of one-way slab. From CRSI

7-12 CONCRETE REINFORCING STEEL INSTITUTE

SOLID ONE-WAY SLABS—INTERIOR SPAN — Top Steel for $-M_u$

f'_c = 3,000 psi — Grade 60 Bars — $\rho \approx 0.0050$

Thickness (in.)	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10
Top Bars	#4	#4	#4	#4	#5	#5	#5	#5	#5	#6	#6	#6	#6
Spacing (in.)	12	11	10	9	12	11	10	10	9	12	11	10	10
Bottom Bars	#3	#3	#3	#4	#4	#4	#4	#4	#4	#5	#5	#5	#5
Spacing (in.)	10	9	7	12	11	10	10	9	8	12	11	10	10
T-S Bars	#3	#3	#3	#3	#4	#4	#4	#4	#4	#4	#4	#5	#5
Spacing (in.)	15	13	12	11	18	17	15	14	13	13	12	18	17
Areas of Steel (in.^{2}/ft) Top Interior	.200	.218	.240	.267	.310	.338	.372	.372	.413	.440	.480	.528	.528
Bottom	.132	.147	.189	.200	.218	.240	.240	.267	.300	.310	.338	.372	.372
Slab Wt. (psf)	50	56	63	69	75	81	88	94	100	106	113	119	125
CLEAR SPAN	**FACTORED USABLE SUPERIMPOSED LOAD (psf)**												
6′-0″	703	923											
6′-6″	589	775											
7′-0″	498	657	907										
7′-6″	425	562	778	988									
8′-0″	365	485	673	856									
8′-6″	315	420	586	747	935								
9′-0″	273	367	513	656	822								
9′-6″	238	321	452	579	727	894	980						
10′-0″	208	282	399	513	646	795	872						
10′-6″	181	243	317	410	539	661	779	882					
11′-0″	159	214	281	365	482	592	699	792	964				
11′-6″	139	189	249	326	432	532	629	713	870	994			
12′-0″	122	167	222	291	388	479	568	644	787	901			
12′-6″	107	148	197	261	349	433	514	583	715	819	967		
13′-0″	94	131	176	234	315	392	465	529	650	746	882		
13′-6″	82	116	157	210	285	355	423	481	593	681	806	959	
14′-0″	71	102	139	188	257	322	384	438	541	623	739	880	939
14′-6″	61	90	124	169	233	293	350	400	495	570	678	809	863
15′-0″	53	79	110	151	210	266	319	365	453	523	623	745	795
15′-6″	45	69	97	136	190	242	291	333	416	480	573	688	733
16′-0″		60	86	121	172	220	265	305	381	442	528	635	678
16′-6″		51	76	108	156	200	242	279	350	406	487	587	627
17′-0″		44	66	96	140	182	221	255	322	374	450	543	580
17′-6″			57	86	127	165	201	233	296	345	416	503	538
18′-0″			49	76	114	150	184	213	272	318	384	467	499
18′-6″			42	66	102	136	167	195	250	293	355	433	463
19′-0″				58	91	123	152	178	230	270	329	402	429
19′-6″				50	81	111	138	162	211	249	304	373	399
20′-0″				43	72	100	125	147	194	229	281	346	370

Note: See Fig. 7-1 for reinforcing bar details.

Used to find minimum reinforcement needed in an interior span for the slab thickness previously determined. From CRSI.

SOLID ONE-WAY SLABS—INTERIOR SPAN — f'_c = 3,000 psi — Grade 60 Bars — Recommended Minimum Steel Top and Bottom

Thickness (in.)	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10
Top Bars	#4	#4	#4	#4	#4	#4	#4	#4	#4	#4	#4	#4	#4
Spacing (in.)	12	12	12	12	12	12	12	12	12	12	12	11	11
Bottom Bars	#3	#3	#3	#3	#3	#3	#3	#3	#4	#4	#4	#4	#4
Spacing (in.)	12	12	12	11	10	9	8	7	12	12	12	11	11
T-S Bars	#3	#3	#3	#3	#4	#4	#4	#4	#4	#4	#4	#5	#5
Spacing (in.)	15	13	12	11	18	17	15	14	13	13	12	18	17
Areas of Steel (in.^{2}/ft) Top Interior	.200	.200	.200	.200	.200	.200	.200	.200	.200	.200	.200	.218	.218
Bottom	.110	.110	.110	.120	.132	.147	.165	.189	.200	.200	.200	.218	.218
Slab Wt. (psf)	50	56	63	69	75	81	88	94	100	106	113	119	125
CLEAR SPAN	**FACTORED USABLE SUPERIMPOSED LOAD (psf)**												
6′-0″	579	680	781	969									
6′-6″	483	568	652	811									
7′-0″	407	479	550	686	851								
7′-6″	345	407	468	585	727	903	990						
8′-0″	295	348	400	502	627	780	855	931					
8′-6″	253	299	344	434	543	678	743	810	876	942			
9′-0″	218	259	298	377	473	592	650	708	766	824	881	940	998
9′-6″	189	224	258	328	414	520	571	622	673	725	775	826	878
10′-0″	163	194	224	287	363	458	503	548	594	640	684	729	775
10′-6″	142	169	195	251	319	362	397	434	470	507	542	578	615
11′-0″	123	147	170	220	282	320	351	383	416	448	479	512	544
11′-6″	106	128	148	193	249	283	311	340	369	398	425	454	483
12′-0″	92	111	129	169	220	251	275	301	327	353	378	404	429
12′-6″	79	96	112	148	194	222	244	267	290	314	336	359	382
13′-0″	68	83	96	130	172	197	216	237	258	279	298	319	340
13′-6″	58	71	83	113	152	174	191	210	229	248	265	284	303
14′-0″	49	61	71	99	134	154	169	186	203	220	235	252	269
14′-6″	41	51	60	85	117	136	149	165	180	195	209	224	239
15′-0″		43	50	73	103	119	132	145	159	172	185	198	212
15′-6″			42	63	90	105	115	128	140	152	163	175	187
16′-0″				53	78	91	101	112	122	133	143	154	165
16′-6″				44	67	79	87	97	107	117	125	135	144
17′-0″					57	68	75	84	92	101	108	117	126
17′-6″					47	57	64	72	79	87	93	101	109
18′-0″						48	54	60	67	74	80	87	93
18′-6″							44	50	56	62	67	73	79
19′-0″								41	46	51	55	61	66
19′-6″										41	44	49	54
20′-0″													42

Note: CRSI recommendations for minimum reinforcement are based on practical considerations of rigidity against deplacement under normal construction traffic. In all cases, [illegible]

Used to find minimum reinforcement needed in an interior span for the slab thickness previously determined. From CRSI.

CONCRETE REINFORCING STEEL INSTITUTE

7-14

3

One-Way Slab Beam Design

$M_u = wl^2/8$

$= 3.48(26.6)^2/8$

$= 307.7'^{k} \approx 3693$ in-kips

$\rho = .6\rho_{max}$ ρ_{max} - From Table A-4

$= .6(.0206)$

$= .0124$

$R = 663$ From Table A-5

$M_u = \phi R b d^2$

$bd^2 = 3693 \cdot 1000 / .9 \cdot 663$

$= 6190 \text{ in}^3$

Try 18" x 18" beam

$I = 18 \cdot 18^3 / 12$

$= 8748 \text{ in}^4$

$\Delta = 5wl^4 / 384(E)(I)$

$= 5(290)(26.6 \cdot 12)^4 / 384(3.6 \cdot 10^6)(8748)$

$= 1.24"$

Try 15" x 20" beam

$I = 10{,}000 \text{ in}^4$

$\Delta = 1.08 \text{ in}$

Alexis Pacella –Structural Option
Dr. Schneider
Lexington II, Washington D.C.
Technical Report #2
October 31, 2005

8-20

STANDARD ONE-WAY JOISTS (1) MULTIPLE SPANS	30" Forms + 5" Rib @ 35" c.-c. (2) FACTORED USABLE SUPERIMPOSED LOAD (PSF)											f'_c = 4,000 psi f_y = 60,000 psi
	8" Deep Rib + 3.0" Top Slab = 11.0" Total Depth											
TOP BARS Size	# 4	# 4	# 4	# 4	# 5	End Span Defl. Coeff. (3)	# 4	# 4	# 4	# 4		Int. Span Defl. Coeff. (3)
TOP BARS @	12	12	11	9	11		12	12	10.5	8		
BOTTOM BARS #	# 3	# 4	# 4	# 5	# 5		# 3	# 3	# 4	# 4		
BOTTOM BARS #	# 4	# 4	# 5	# 5	# 6		# 3	# 4	# 4	# 5		
Steel (psf)	.50	.60	.72	.89	1.09		.56	.63	.78	1.00		
CLEAR SPAN	END SPAN						INTERIOR SPAN					
14'-0"	184	258	274*	285*	298*	.450	194	302	312*	322*		.277
	0	0	346	436	464*		0	0	410	538		
15'-0"	150	215	244*	253*	263*	.593	159	253	280*	289*		.365
	0	0	292	370	428*		0	0	347	459		
16'-0"	123	180	219*	226*	235*	.767	131	213	253*	261*		.472
	0	0	247	316	382*		0	0	296	394		
17'-0"	100	151	197*	203*	211*	.978	107	180	230*	237*		.602
	0	0	210	271	339		0	0	254	340		
18'-0"	81	126	178*	184*	190*	1.229	87	152	210*	216*		.756
	0	0	179	233	295		0	0	218	295		
19'-0"	65	105	153	167*	172*	1.525	71	129	188	198*		.939
	0	0	0	202	257		0	0	0	257		
20'-0"	51	88	131	152*	157*	1.873	56	109	162	181*		1.153
	0	0	0	175	224		0	0	0	225		
21'-0"		72	112	139*	143*	2.276	44	92	140	167*		1.401
		0	0	151	196		0	0	0	197		
22'-0"		59	95	128*	131*	2.742		77	121	154*		1.687
		0	0	131	172			0	0	173		
23'-0"		48	80	113	120*	3.276		64	104	143*		2.016
		0	0	0	151			0	0	151		
24'-0"			68	98	110*	3.884		52	89	132*		2.390
			0	0	132			0	0	133		
25'-0"			56	84	102*	4.572		42	76	116		2.814
			0	0	116			0	0	0		
26'-0"			46	72	94*	5.349			65	102		3.292
			0	0	102				0	0		
27'-0"				61	86*	6.221			55	89		3.828
				0	89				0	0		

(1) For gross section properties, see Table 8-1.
(2) First load is for standard square joist ends; second load is for special tapered joist ends.
(3) Computation of deflection is not required above horizonal line (thickness ≥ $\ell_n/18.5$ for end spans, $\ell_n/21$ for interior spans).
(4) Exclusive of bridging joists and tapered ends.
*Controlled by shear capacity. +Capacity at elastic deflection = $\ell_n/360$.

PROPERTIES FOR DESIGN (CONCRETE .36 CF/SF) (4)

NEGATIVE MOMENT												
STEEL AREA (SQ. IN.)	.58	.58	.64	.78	.99		.58	.58	.67	.88		
STEEL % (UNIFORM)	1.03	1.03	1.12	1.37	1.75		1.03	1.03	1.18	1.54		
(TAPERED)	.55	.55	.60	.74	.94		.55	.55	.63	.83		
EFF. DEPTH, IN.	9.8	9.8	9.8	9.8	9.7		9.8	9.8	9.8	9.8		
– ICR/IGR	.208	.208	.222	.256	.298		.208	.208	.230	.278		
POSITIVE MOMENT												
STEEL AREA (SQ. IN.)	.31	.40	.51	.62	.75		.22	.31	.40	.51		
STEEL %	.09	.12	.15	.18	.22		.06	.09	.12	.15		
EFF. DEPTH, IN.	9.8	9.8	9.7	9.7	9.6		9.8	9.8	9.8	9.7		
+ICR/IGR	.164	.207	.254	.303	.353		.121	.164	.207	.254		

CONCRETE REINFORCING STEEL INSTITUTE

From CRSI. Chart used to determine One-way joist system sizing.

Girder Design

$M_u = wl^2/8$ $\qquad w_u \approx 3\text{ klf}$

$= \frac{3(26.6)^2}{8}$

$= 265.3^{'k} = 3183.6 \text{ in-k}$

$bd^2 = M_u / \phi R$

$= 3183.6 \cdot 1000 / .9 \cdot .663$

$= 5335.34 \text{ in}^2$

Let b = 8 - joist

d = 26"

Use 8 × 26" $\qquad w(\text{psi}) = 3000 \text{ klf} \cdot \frac{k}{ft} \cdot \frac{ft}{12 in}$

$I = 11717.33$

$\Delta = 5(250)(26.6 \cdot 12)^4 / 384(3.6 \cdot 10^6)(11717) = .8 \text{ in}$

Total weight of system

54psi → slabs

$8/12 \cdot 26/12 \cdot 150\text{pcf} = 216.66\text{plf}$

$\cdot (16.2 + 26.6) = 9273.3 \text{ lbs}$

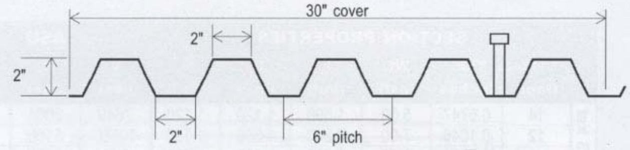
$= 22 \text{ psf}$

$54 + 22 = 75$

Calculations and deflection check for girder needed to support one-way joist system.

SECTION PROPERTIES						ASD			LRFD		
Metal Thickness Gage	Metal Thickness Inches	Wt. (psf)	I_p (in.^{4})	S_p (in.^{3})	S_n (in.3)	V (lbs)	R_1 (lbs)	R_2 (lbs)	ϕV (lbs)	ϕR_1 (lbs)	ϕR_2 (lbs)
24	0.0239	1.50	0.232	0.192	0.200	2360	360	836	3223	532	1156
22	0.0295	2.00	0.300	0.252	0.263	4205	528	1484	5477	736	1992
20	0.0358	2.00	0.379	0.325	0.339	6062	728	2224	8067	1004	3064
18	0.0474	3.00	0.523	0.468	0.485	8796	1204	3948	11182	1648	5388

UF2X



The bottom flange can accept a ¾" shear stud.

approx. scale: 1½" = 1'0"

UNIFORM TOTAL LOAD / Load that Produces l/180 Deflection, psf

	Gage	Span Condition	Span 6'0"	6'6"	7'0"	7'6"	8'0"	8'6"	9'0"	9'6"	10'0"
ASD	24	Single	128 / 94	109 / 74	94 / 59	82 / 48	72 / 40	64 / 33	57 / 28	51 / 24	46 / 20
		Double	130 / 226	111 / 178	96 / 143	84 / 116	74 / 96	66 / 80	59 / 67	53 / 57	48 / 49
		Triple	162 / 177	138 / 139	120 / 112	105 / 91	92 / 75	82 / 62	73 / 52	66 / 45	59 / 38
	22	Single	168 / 122	143 / 96	123 / 77	108 / 62	94 / 51	84 / 43	75 / 36	67 / 31	60 / 26
		Double	173 / 293	148 / 230	128 / 184	111 / 150	98 / 123	87 / 103	78 / 87	70 / 74	63 / 63
		Triple	215 / 229	184 / 180	159 / 144	139 / 117	122 / 97	108 / 81	97 / 68	87 / 58	78 / 49
	20	Single	217 / 154	185 / 121	159 / 97	139 / 79	122 / 65	108 / 54	96 / 46	86 / 39	78 / 33
		Double	224 / 370	191 / 291	165 / 233	144 / 189	126 / 156	112 / 130	100 / 110	90 / 93	81 / 80
		Triple	279 / 289	238 / 228	205 / 182	179 / 148	158 / 122	140 / 102	125 / 86	112 / 73	101 / 63
	18	Single	312 / 212	266 / 167	229 / 133	200 / 109	176 / 89	155 / 75	139 / 63	124 / 53	112 / 46
		Double	320 / 510	273 / 401	236 / 321	206 / 261	181 / 215	160 / 179	143 / 151	128 / 129	116 / 110
		Triple	399 / 399	340 / 314	294 / 252	256 / 204	226 / 168	200 / 140	179 / 118	160 / 101	145 / 86
LRFD	24	Single	177 / 94	164 / 74	149 / 59	130 / 48	114 / 40	101 / 33	90 / 28	81 / 24	73 / 20
		Double	154 / 226	142 / 178	132 / 143	123 / 116	116 / 96	104 / 80	93 / 67	83 / 57	75 / 49
		Triple	175 / 177	162 / 139	150 / 112	140 / 91	131 / 75	124 / 62	115 / 52	103 / 45	94 / 38
	22	Single	245 / 122	226 / 96	195 / 77	170 / 62	150 / 51	133 / 43	118 / 36	106 / 31	96 / 26
		Double	266 / 293	233 / 230	201 / 184	176 / 150	155 / 123	137 / 103	122 / 87	110 / 74	99 / 63
		Triple	302 / 229	279 / 180	250 / 144	218 / 117	192 / 97	171 / 81	152 / 68	137 / 58	124 / 49
	20	Single	335 / 154	292 / 121	252 / 97	220 / 79	193 / 65	171 / 54	152 / 46	137 / 39	124 / 33
		Double	353 / 370	301 / 291	260 / 233	227 / 189	200 / 156	177 / 130	158 / 110	142 / 93	128 / 80
		Triple	418 / 289	375 / 228	324 / 182	283 / 148	249 / 122	221 / 102	197 / 86	177 / 73	160 / 63
	18	Single	494 / 212	421 / 167	363 / 133	316 / 109	278 / 89	246 / 75	220 / 63	197 / 53	178 / 46
		Double	505 / 510	431 / 401	372 / 321	325 / 261	286 / 215	253 / 179	226 / 151	203 / 129	183 / 110
		Triple	627 / 399	536 / 314	463 / 252	404 / 204	356 / 168	316 / 140	282 / 118	253 / 101	229 / 86

NOTES:
Vented deck with 1.5% open area is available for use with insulating fills. Insulating fill manufacturers have determined load capacities of various combinations of fill and deck both with and without foamed plastic insulation boards. Refer to the fill manufacturer's literature for loading limitations.
R_1 is the bearing capacity at an **exterior** condition. R_2 is the bearing capacity at an **interior** condition.

USD chart used to size gauge of metal decking and slab thickness needed for the form deck system.

concrete slabs on UF2X form deck - UNIFORM LOADS, PSF

LRFD

Slab	Mesh	+d	-d	+M	-M	Spans, feet 4'6"	5'0"	5'6"	6'0"	6'6"	7'0"	7'6"	8'0"	8'6"	9'0"
4.0"	66 - W2.0 x 2.0* 66 - W2.9 x 2.9	1.919 1.904	3.007 2.962	4.060 5.785	6.326 8.921	157 224	127 181	105 150	88 126	75 107	65 93	57 81	50 71	44 63	56
4.5"	66 - W4.0 x 4.0 44 - W2.9 x 2.9 44 - W4.0 x 4.0	2.387 2.404 2.387	3.412 3.462 3.412	9.975 10.893 14.708	14.062 15.463 20.585	386 ### ###	313 342 ###	259 282 381	217 237 320	185 202 273	160 174 235	139 152 205	122 133 180	108 118 160	97 105 142
5.0"	66 - W4.0 x 4.0* 44 - W2.9 x 2.9 44 - W4.0 x 4.0	2.887 2.904 2.887	3.912 3.962 3.912	12.135 13.242 17.948	16.222 17.812 23.825	### ### ###	381 ### ###	315 343 ###	264 289 389	225 246 332	194 212 286	169 185 249	149 162 219	132 144 194	117 128 173
5.5"	44 - W2.9 x 2.9* 44 - W4.0 x 4.0	3.404 3.387	4.462 4.412	15.591 21.188	20.161 27.065	### ###	### ###	392 ###	329 ###	281 377	242 325	211 283	185 249	164 220	146 197
6.0"	44 - W4.0 x 4.0	3.887	4.912	24.428	30.305	###	###	###	###	###	364	317	279	247	220
6.5"	44 - W4.0 x 4.0	4.387	5.412	27.668	33.545	###	###	###	###	###	###	351	308	273	244
7.0"	44 - W4.0 x 4.0	4.887	5.912	30.908	36.785	###	###	###	###	###	###	385	338	299	267
						24 gage						22 gage		20 gage	

USD chart used to determine wire mesh needed in form deck and slab.

1.5 x 12" DECK F_y = 33ksi f'_c = 3 ksi 145 pcf concrete

LRFD

L, Uniform Live Service Loads, psf *

Gage	Slab Depth	φMn in.k	5.00	5.50	6.00	6.50	7.00	7.50	8.00	8.50	9.00	9.50	10.00	10.50	11.00
22 gage	4.00	36.40	400	400	390	330	280	240	205	180	155	140	120	105	95
	4.50	42.43	400	400	400	385	325	280	240	210	185	160	140	125	110
	5.00	48.46	400	400	400	400	375	320	275	240	210	185	160	145	125
	5.50	54.50	400	400	400	400	400	360	310	270	235	205	185	160	145
	6.00	60.53	400	400	400	400	400	400	345	300	265	230	205	180	160
	6.50	66.56	400	400	400	400	400	400	380	330	290	255	225	200	175
	6.75	69.57	400	400	400	400	400	400	395	345	300	265	235	205	185
	7.00	72.59	400	400	400	400	400	400	400	360	315	275	245	215	190
20 gage	4.00	43.31	400	400	400	395	340	290	250	220	190	170	150	135	120
	4.50	50.61	400	400	400	400	395	340	295	255	225	200	175	155	140
	5.00	57.90	400	400	400	400	400	390	335	295	260	225	200	180	160
	5.50	65.19	400	400	400	400	400	400	380	330	290	255	225	200	180
	6.00	72.49	400	400	400	400	400	400	400	370	325	285	255	225	200
	6.50	79.78	400	400	400	400	400	400	400	400	355	315	280	250	220
	6.75	83.43	400	400	400	400	400	400	400	400	375	330	290	260	230
	7.00	87.07	400	400	400	400	400	400	400	400	390	345	305	270	240
19 gage	4.00	49.98	400	400	400	400	395	340	295	255	225	200	175	160	140
	4.50	58.54	400	400	400	400	400	400	345	300	265	235	210	185	165
	5.00	67.09	400	400	400	400	400	400	395	345	305	270	240	215	190
	5.50	75.65	400	400	400	400	400	400	400	390	345	305	270	240	215
	6.00	84.20	400	400	400	400	400	400	400	400	385	340	300	270	240
	6.50	92.76	400	400	400	400	400	400	400	400	400	375	335	295	265
	6.75	97.03	400	400	400	400	400	400	400	400	400	390	350	310	280
	7.00	101.31	400	400	400	400	400	400	400	400	400	400	365	325	290
18 gage	4.00	55.70	400	400	400	400	400	380	330	290	255	225	200	180	160
	4.50	65.38	400	400	400	400	400	400	390	340	300	265	235	210	190
	5.00	75.06	400	400	400	400	400	400	400	395	345	305	270	245	220
	5.50	84.73	400	400	400	400	400	400	400	400	390	345	310	275	245
	6.00	94.41	400	400	400	400	400	400	400	400	400	385	345	305	275
	6.50	104.09	400	400	400	400	400	400	400	400	400	400	380	340	305
	6.75	108.93	400	400	400	400	400	400	400	400	400	400	400	355	320
	7.00	113.76	400	400	400	400	400	400	400	400	400	400	400	370	335
16 gage	4.00	55.70	400	400	400	400	400	380	330	290	255	225	200	180	160
	4.50	65.38	400	400	400	400	400	400	390	340	300	265	235	210	190
	5.00	75.06	400	400	400	400	400	400	400	395	345	305	270	245	220
	5.50	84.73	400	400	400	400	400	400	400	400	390	345	310	275	245
	6.00	94.41	400	400	400	400	400	400	400	400	400	385	345	305	275
	6.50	104.09	400	400	400	400	400	400	400	400	400	400	380	340	305
	6.75	108.93	400	400	400	400	400	400	400	400	400	400	400	355	320
	7.00	113.76	400	400	400	400	400	400	400	400	400	400	400	370	335
22 gage	4.00	27.28	400	345	285	240	200	170	145	125	110	95	85	75	65
	4.50	32.47	400	400	340	285	240	205	175	150	130	115	100	90	75
	5.00	37.79	400	400	400	335	280	240	205	180	155	135	120	105	90
	5.50	43.20	400	400	400	380	325	275	235	205	180	155	135	120	105
	6.00	48.68	400	400	400	400	365	310	270	230	200	175	155	135	120
	6.50	54.22	400	400	400	400	400	350	300	260	225	195	175	150	135
	6.75	57.00	400	400	400	400	400	365	315	275	240	210	180	160	140
	7.00	59.79	400	400	400	400	400	385	330	285	250	220	190	170	150
20 gage	4.00	32.48	400	400	345	290	245	210	180	155	135	120	105	90	80
	4.50	38.69	400	400	400	345	295	250	215	190	165	145	125	110	100
	5.00	45.06	400	400	400	400	345	295	255	220	190	170	150	130	115
	5.50	51.55	400	400	400	400	395	335	290	255	220	195	170	150	135
	6.00	58.13	400	400	400	400	400	380	330	285	250	220	195	170	150
	6.50	64.78	400	400	400	400	400	400	370	320	280	245	215	190	170
	6.75	68.12	400	400	400	400	400	400	390	335	295	260	230	200	180
	7.00	71.48	400	400	400	400	400	400	400	355	310	270	240	210	190
19 gage	4.00	37.46	400	400	400	340	290	245	215	185	160	140	125	110	100
	4.50	44.68	400	400	400	400	345	295	255	220	195	170	150	135	120
	5.00	52.10	400	400	400	400	400	345	300	260	230	200	175	155	140
	5.50	59.67	400	400	400	400	400	395	345	300	260	230	205	180	160
	6.00	67.34	400	400	400	400	400	400	390	340	295	260	230	205	185
	6.50	75.10	400	400	400	400	400	400	400	380	335	295	260	230	205
	6.75	79.00	400	400	400	400	400	400	400	400	350	310	275	245	215
	7.00	82.92	400	400	400	400	400	400	400	400	370	325	285	255	225
18 gage	4.00	41.82	400	400	400	380	325	280	240	210	185	160	145	125	115
	4.50	49.93	400	400	400	400	390	335	290	250	220	195	170	155	135
	5.00	58.27	400	400	400	400	400	390	340	295	260	230	200	180	160
	5.50	66.77	400	400	400	400	400	400	390	340	300	265	235	210	185
	6.00	75.41	400	400	400	400	400	400	400	385	340	300	265	235	210
	6.50	84.14	400	400	400	400	400	400	400	400	380	335	295	265	235
	6.75	88.54	400	400	400	400	400	400	400	400	400	355	315	280	250
	7.00	92.95	400	400	400	400	400	400	400	400	400	370	330	295	260
16 gage	4.00	41.82	400	400	400	380	325	280	240	210	185	160	145	125	115
	4.50	49.93	400	400	400	400	390	335	290	250	220	195	170	155	135
	5.00	58.27	400	400	400	400	400	390	340	295	260	230	200	180	160
	5.50	66.77	400	400	400	400	400	400	390	340	300	265	235	210	185
	6.00	75.41	400	400	400	400	400	400	400	385	340	300	265	235	210
	6.50	84.14	400	400	400	400	400	400	400	400	380	335	295	265	235
	6.75	88.54	400	400	400	400	400	400	400	400	400	355	315	280	250
	7.00	92.95	400	400	400	400	400	400	400	400	400	370	330	295	260

1 STUD/FT.

NO STUDS

* The **Uniform Live Loads** are based on the LRFD equation $\phi M_n = (1.6L + 1.2D)l^2/8$. Although there are other load combinations that may require investigation, this will control most of the time. The equation assumes there is no negative bending reinforcement over the beams and therefore each composite slab is a single span. Two sets of values are shown; ϕM_{nf} is used to calculate the uniform load when the full required number of studs is present; ϕM_{no} is used to calculate the load when no studs are present. A straight line interpolation can be done if the average number of studs is between zero and the required number needed to develop the "full" factored moment. The tabulated loads are checked for shear controlling (it seldom does), and also limited to a live load deflection of 1/360 of the span.

An upper limit of 400 psf has been applied to the tabulated loads. This has been done to guard against equating large concentrated to uniform loads. Concentrated loads may require special analysis and design to take care of servicibility requirements not covered by simply using a uniform load value. On the other hand, for any load combination the values provided by the composite properties can be used in the calculations.

Welded wire fabric in the required amount is assumed for the table values. If welded wire fabric is not present, deduct 10% from the listed loads.

Refer to the example problems for the use of the tables.

1.5" LOK-FLOOR

USD chart used to determine gauge size and slab thickness for composite deck.

Strand Pattern Designation

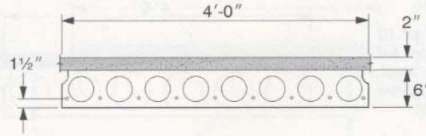
76-S
- S = straight
- Diameter of strand in 16ths
- No. of strand (7)

Safe loads shown include dead load of 10 psf for untopped members and 15 psf for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load.

Capacity of sections of other configurations are similar. For precise values, see local hollow-core manufacturer.

HOLLOW-CORE

4′-0″ x 6″
Normal Weight Concrete



f'_c = 5,000 psi
f'_{ci} = 3,500 psi

Section Properties

		Untopped		Topped	
A	=	187	in²	—	
I	=	763	in⁴	1,640	in⁴
y_b	=	3.00	in.	4.14	in.
y_t	=	3.00	in.	3.86	in.
S_b	=	254	in³	396	in³
S_t	=	254	in³	425	in³
b_w	=	16.00	in.	16.00	in.
wt	=	195	plf	295	plf
		49	psf	74	psf
V/S	=	1.73	in.		

Key
306 — Safe superimposed service load, psf
0.2 — Estimated camber at erection, in.
0.2 — Estimated long-time camber, in.

4HC6

Table of safe superimposed service load (psf) and cambers (in.) — **No Topping**

Strand Designation Code	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
	Span, ft																		
66-S	306	257	217	184	157	135	116	100	87	75	65	56	48	42	36	30			
	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	−0.1	−0.2	−0.4			
	0.2	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.1	0.1	0.0	−0.2	−0.3	−0.5	−0.7	−1.0			
76-S	358	301	254	217	186	160	139	121	105	92	80	70	61	53	47	40	35		
	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0.1	0.0	−0.1	−0.3		
	0.3	0.3	0.3	0.4	0.4	0.4	0.4	0.3	0.3	0.2	0.1	0.0	−0.1	−0.3	−0.5	−0.7	−1.0		
96-S		384	326	279	240	208	182	159	140	123	109	97	86	76	67	60	53	46	41
		0.3	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.1	0.0	−0.1
		0.4	0.5	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.5	0.5	0.4	0.2	0.1	−0.1	−0.4	−0.6	−0.9
87-S			383	331	286	249	218	192	169	150	133	119	106	95	84	76	68	60	54
			0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.7	0.8	0.8	0.7	0.7	0.7	0.6	0.5	0.4	0.3
			0.6	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.8	0.8	0.7	0.7	0.5	0.4	0.2	0.0	−0.3
97-S				364	317	277	243	214	189	168	150	134	120	107	96	87	78	70	62
				0.6	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.9	1.0	1.0	0.9	0.9	0.8	0.8	0.7
				0.8	0.9	0.9	1.0	1.0	1.1	1.1	1.1	1.1	1.0	1.0	0.9	0.8	0.6	0.4	0.2

4HC6+2

Table of safe superimposed service load (psf) and cambers (in.) — **2″ Normal Weight Topping**

Strand Designation Code	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
	Span, ft																
66-S	305	258	220	188	162	139	119	97	78	62	47	35					
	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	−0.1					
	0.2	0.2	0.2	0.1	0.1	0.0	−0.1	−0.2	−0.3	−0.5	−0.7	−0.9					
76-S	358	304	260	224	194	168	146	122	101	82	66	52	39				
	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0.1	0.0				
	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	−0.2	−0.3	−0.5	−0.7	−0.9				
96-S		390	336	291	253	221	194	170	146	123	104	87	72	58	46	35	
		0.4	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.1	0.0	
		0.4	0.4	0.4	0.4	0.4	0.3	0.3	0.2	0.1	−0.1	−0.3	−0.5	−0.7	−1.0	−1.4	
87-S			398	346	302	265	234	206	182	158	136	117	100	85	71	59	47
			0.6	0.6	0.7	0.7	0.7	0.7	0.8	0.8	0.7	0.7	0.7	0.6	0.5	0.4	0.3
			0.5	0.6	0.6	0.6	0.5	0.5	0.4	0.4	0.2	0.1	−0.1	−0.3	−0.5	−0.8	−1.2
97-S				382	335	294	260	231	205	181	157	137	119	102	88	75	63
				0.7	0.8	0.8	0.9	0.9	0.9	1.0	1.0	0.9	0.9	0.9	0.8	0.8	0.7
				0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.5	0.4	0.2	0.0	−0.2	−0.5	−0.8

Strength based on strain compatibility; bottom tension limited to $6\sqrt{f'_c}$; see pages 2-2–2-6 for explanation.

PCI chart used to determine reinforcement needed in hollow-core pre-cast slab.

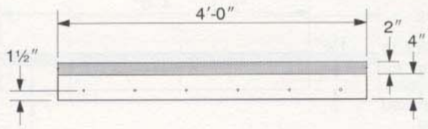
Strand Pattern Designation

76-S
- S = straight
- Diameter of strand in 16ths
- No. of strand (7)

Safe loads shown include dead load of 10 psf for untopped members and 15 psf for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load.

SOLID FLAT SLAB

4″ Thick
Normal Weight Concrete



f'_c = 5,000 psi
f'_{ci} = 3,500 psi

Section Properties

		Untopped		Topped	
A	=	192	in²	—	
I	=	256	in⁴	763	in⁴
y_b	=	2.00	in.	2.84	in.
y_t	=	2.00	in.	3.16	in.
S_b	=	128	in³	269	in³
S_t	=	128	in³	242	in³
b_w	=	48.00	in.	48.00	in.
wt	=	200	plf	300	plf
		50	psf	75	psf
V/S	=	1.85	in.		

Key
- 196 — Safe superimposed service load, psf
- 0.1 — Estimated camber at erection, in.
- 0.1 — Estimated long-time camber, in.

FS4

Table of safe superimposed service load (psf) and cambers — **No Topping**

Strand Designation Code	Span, ft 10	11	12	13	14	15	16	17	18	19	20	21	22
66-S	196	165	132	105	83	66	52	41	31				
	0.1	0.1	0.1	0.0	0.0	−0.1	−0.2	−0.3	−0.4				
	0.1	0.1	0.0	0.0	−0.1	−0.3	−0.4	−0.6	−0.9				
76-S	230	190	151	122	98	79	63	50	40	30			
	0.1	0.1	0.1	0.1	0.0	0.0	−0.1	−0.2	−0.3	−0.5			
	0.1	0.1	0.1	0.0	−0.1	−0.2	−0.3	−0.5	−0.9	−1.0			
58-S	253	212	180	154	127	104	86	70	57	46	37		
	0.2	0.2	0.2	0.2	0.1	0.1	0.0	0.0	−0.1	−0.3	−0.4		
	0.2	0.2	0.2	0.2	0.1	0.0	−0.1	−0.3	−0.5	−0.8	−1.1		
68-S	300	252	214	180	152	127	105	88	73	60	50	40	32
	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.0	−0.1	−0.3	−0.4	−0.7
	0.3	0.3	0.3	0.3	0.2	0.2	0.1	−0.1	−0.3	−0.5	−0.8	−1.2	−1.6

FS4+2

Table of safe superimposed service load (psf) and cambers — **2″ Normal Weight Topping**

Strand Designation Code	Span, ft 10	11	12	13	14	15	16	17	18	19	20	21
66-S	369	296	224	167	123	87	57	33				
	0.1	0.1	0.1	0.0	0.0	−0.1	−0.2	−0.3				
	0.0	0.0	0.0	−0.1	−0.2	−0.3	−0.5	−0.7				
76-S		346	265	203	153	113	80	53	31			
		0.1	0.1	0.1	0.0	0.0	−0.1	−0.2	−0.3			
		0.0	0.0	−0.1	−0.1	−0.3	−0.4	−0.6	−0.9			
58-S		400	342	274	214	166	127	95	67	44		
		0.2	0.2	0.2	0.1	0.1	0.0	0.0	−0.1	−0.3		
		0.1	0.1	0.0	0.0	−0.1	−0.3	−0.4	−0.7	−0.9		
68-S				335	268	213	169	132	101	74	52	32
				0.2	0.2	0.2	0.2	0.1	0.0	−0.1	−0.3	−0.4
				0.1	0.1	0.0	−0.1	−0.3	−0.5	−0.7	−1.0	−1.4

Strength based on strain compatibility; bottom tension limited to $6\sqrt{f'_c}$; see pages 2-2–2-6 for explanation.

PCI chart used to determine reinforcement needed in pre-cast solid flat slab.