Technical Assignment #2

Roosevelt Island Southtown Building No. 5, NY, NY

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Structural Option

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EXECUTIVE SUMMARY

Typically, most new high-rise construction in New York City uses reinforced concrete as the main building component. This is due to concrete's ability to use fast-track construction processes that have little to no lead time. With a typical flat plate floor slab only taking two days to construct, many of the advantages of other materials are often overlooked. This second technical report consists of four proposed alternate floor systems for Roosevelt Island Southtown Building No. 5. The existing floor system, a two-way reinforced concrete flat plate, was re-analyzed and compared to the four new options.

The four systems that were explored were as follows:

1) Composite Steel Floor System
2) Precast Girder-Slab System
3) Two-way Waffle Slab System
4) Post-tensioned Two-way Flat Plate

To keep the report consistent, an internal section of the typical floor was analyzed. From the previous report, it was determined that a 24'-0" x 26'-0" bay was the largest span in the building to eliminate any architectural problems. Therefore, all of the analyses were performed over a span similar to that of the existing condition. Some systems that allowed for a larger span were cross checked with the architectural drawings to determine if the span was necessary. This concept is further detailed later in the report.

After this study of four different floor systems, it has been concluded that the next best alternative to a reinforced concrete two-way flat plate is the use of the precast girder-slab floor system. Although the overall building height would slightly increase with the addition of a larger floor thickness, the building would still be able to be constructed in a reasonably similar amount of time. The advantages of this system will be highlighted more thoroughly in the upcoming pages.

Figure 1: Panoramic View of Roosevelt Island as seen from

STRUCTURAL SYSTEMS

Foundations

Three types of foundation systems are used for Southtown Building No. 5. Individual footings are used for interior columns of the building. These footings range from 3'-0" x 3'-0" to 4'-6" x 4'-6". A mat footing is used at the base of the later force resisting shear walls. The mat is typically 42" thick with some step downs required for the elevator, boiler, and sump pits. Finally, a 12" thick foundation wall is used around the perimeter of the cellar. This system incorporates exterior concrete piers into the wall with footings at the base.

Floor System

The floor system of the apartment building is typically a 8" two-way normal weight concrete flat plate with varying size bays. At the cellar floor, a 6" concrete slab is used with W2.0 x W2.0 welded wire fabric. At the first floor, a 9" concrete slab is used to accommodate for higher occupancy loads. Typical reinforcement for the floor system is #4 @ 14" bottom steel and #4 @ 14" top steel. Middle strip reinforcement is used in the floor slab in some areas of higher stress. Due to the party room and lounge area on the main roof, a 10" concrete floor slab with #5 @ 12" top and bottom steel is used.

Columns

The columns in this New York building are typically rectangular reinforced concrete with varying sizes and reinforcement. The most common column size is 14" x 24" with 8 #6 bars as structural reinforcement. Column loads vary greatly within the building, especially as the elevation rises. The largest loads at the foundation level is 1056 kips of dead load and 139 kips of live load.

Lateral System

Reinforced concrete shear walls make up the lateral force resisting system of the building. The elevator and stairwell core in the center of the building have been assigned as the location of these shear walls. The shear walls rise from the cellar level of the building all the way to the elevator mechanical room. A 12" typical shear wall section consists of #4 @ 12" horizontal reinforcement and #5 @ 12" vertical reinforcement. Openings in the shear walls require link beams in order to transfer high shear forces from one side of the opening to the other.

CODES AND DESIGN REFERENCES

Codes and References

1. "Building Code of the City of New York"
2. "The New York City Seismic Code: Local Law 17/95"
3. "Code Requirements for Structural Concrete" (ACI 318-99), American Concrete Institute as modified by subchapter 10, article 5 of the N.Y.C Building Code
4. "Building Code Requirements for Masonry Structures" (ACI 530-99), American Concrete Institute as modified by reference standard RS 10 -18 of the N.Y.C. Building Code

MATERIALS

Cast in Place Concrete

Foundations: 4 ksi
Foundation Walls: 5 ksi
Slabs on Ground: 4 ksi
Formed Slabs: 5 ksi
Shear Walls:
    Cellar – 2nd Floor: 7 ksi
    3rd Floor – 8th Floor: 6 ksi
    9th Floor – EMR Roof: 5 ksi
Columns
    Cellar: 4 ksi for Buttress, 7 ksi for Columns
    1st Floor – 2nd Floor: 7 ksi
    3rd Floor – 8th Floor: 6 ksi
    9th Floor – EMR Roof: 5 ksi

Reinforcement

ASTM A615, Grade 60
Welded Wire Fabric: ASTM A185
Welded Deformed Wire Fabric: ASTM A467, Grade 60

Structural Steel

All Rolled Shapes: ASTM A572 or A992, Grade 50
All Plates and Connection Material: ASTM A36
All Tubular Sections: ASTM A500, Grade B
All Pipe Sections: A53, Grade B
Anchor Bolts: ASTM F1554

Welding Electrodes: E70XX Low Hydrogen

Bolting Materials: ASTM A325 or A490

Light Gage Framing:
16 Gage and Heavier: ASTM A653, Grade 50
18 Gage and Lighter: ASTM A653, Grade 33

GRAVITY LOADS

Dead Loads

1. Construction Dead Load
    a. Cellar Floor: 75 psf
    b. 1st Floor: 113 psf
    c. 2nd -16th Floor: 100 psf
    d. Main Roof: 113 psf
    e. Mechanical Room Floor: 100 psf
    f. E.M.R Floor: 100 psf
    g. E.M.R Roof: 100 psf
2. Superimposed Dead Load
    a. Cellar Floor: 25 psf
    b. 1st Floor: 30 psf
    c. 2nd -16th Floor: 20 psf
    d. Main Roof: 50psf
    e. Mechanical Room Floor: 25 psf
    f. E.M.R Floor: 25 psf
    g. E.M.R Roof: 25 psf

Live Loads

1. Cellar Floor:
    a. Equipment Rooms: 100 psf
    b. Offices: 50 psf
2. 1st Floor:
    a. Public Area: 100 psf
    b. Residential: 40psf
3. 2nd – 16th Floor: 40 psf
4. Main Roof: 100 psf – Public Area, Mechanical, Storage
5. Mechanical Room Floor: 100 psf
6. E.M.R. Floor: 100 psf – Mechanical + Machine Weight
7. E.M.R. Roof: 30 psf

EXISTING STRUCTURAL SYSTEM

Two-way reinforced concrete flat plate

The existing floor system for Roosevelt Island Southtown Building No. 5 is typically an 8” two-way normal weight reinforced concrete flat plate. Due to the building’s primary use as a residential apartment building, there is no typical bay size to allow for different apartment layouts. The largest bay size, however, is 24’-0” x 26’-0”. At the cellar floor, a 6” concrete slab is used with W2.0 x W2.0 welded wire fabric. At the first floor, a 9” concrete slab is used to accommodate for higher occupancy loads. Typical reinforcement for the 8” floor system is #4 @ 14” bottom steel and #4 @ 14” top steel. Middle strip reinforcement is used in the floor slab in some areas of higher stress. Additionally, several extra bars are required at the faces of the columns to resist the higher moments. The columns in the residential high rise are typically reinforced concrete. The most standard size for these is 14” x 24” with 8 #6 bars as structural reinforcement.

Flat plate floor construction is a perfect flooring system for a place like New York City. Workers are able to construct these floors very fast with little complication, averaging only two days per floor. The flat plate allows for easy coordination with other trades which can save the owner and construction team time and money. Also, the thin floor thickness allows for a maximum floor to ceiling height in these buildings which means additional leasable area for the owners. This is a very important fact since most buildings in New York City have a limit on the overall building height. However, there are some drawbacks to this floor system. Different size reinforcement in a floor’s column strips and middle strips can cause increased cost during construction. This can also take additional time and become very labor intensive. Cast-in-place concrete is also very weather sensitive. Construction can be delayed or even stop in the event of any wet weather. Also, cast-in-place concrete can add more weight to a building than some other materials. This can cause an increase in column and slab sizes, foundation loads, and seismic loads, thus increasing the cost of the building.

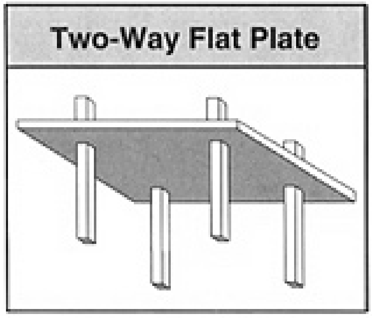


Figure 2: Two-Way Flat Plate Diagram

TYPICAL FLOOR FRAMING PLAN

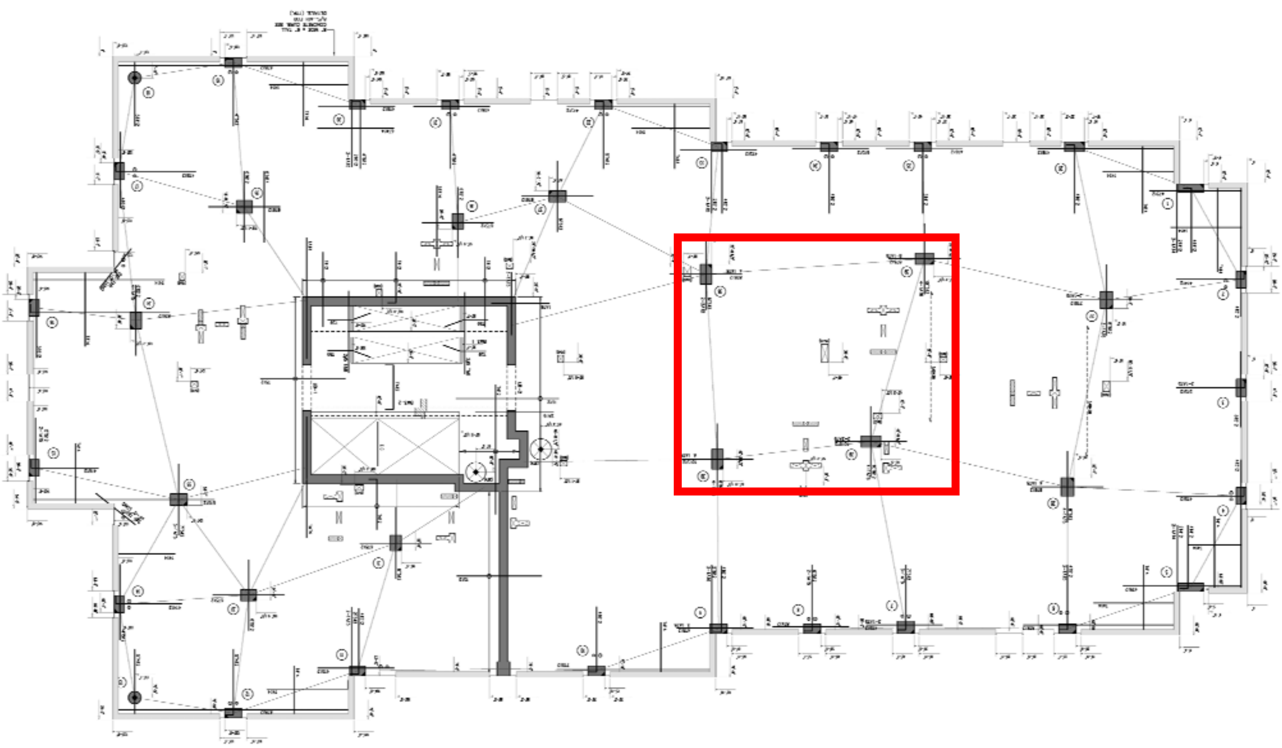


Figure 3: Floor plan of Southtown Building No. 5. Red box indicates interior bay used for analysis of existing two-way reinforced concrete flat plate floor system

COMPOSITE STEEL FLOOR SYSTEM

The composite steel floor system utilizes many advantageous characteristics of both concrete and steel. By combining these two materials, the compressive strength of concrete can increase the load capacity of a steel wide flange shape. Since steel has the highest strength-to-weight ratio of any current building material, it is very useful in today's construction industry. Additionally, composite construction is widely used in the building industry. Being that it is rather familiar amongst contractors, steel is erected in quicker time than most concrete and other flooring systems. It alleviates any formwork that may be required to construct the floor system by employing metal deck over top of steel beams. This can ultimately result in a shorter construction schedule.

However, there are some drawbacks to composite construction. First, additional lead time is required to fabricate steel, especially unique shapes. Without advance planning, this can delay any time that was saved in the erection process. Additionally, composite construction cannot achieve the small floor thicknesses of flat plate concrete slabs. This increases the total building height and may not allow for the same amount of leasable floor space. With the added space, however, one can save some room by integrating plumbing standpipes, HVAC ducts, and electrical conduits into the areas between the steel beams.

In Southtown Building No. 5, the use of steel construction resulted in a set of standard bays throughout the floor plan. The most common bay size is now a 17'-7" x 11'-6" exterior bay and a 22'-6" x 11'-6" interior bay. These bay sizes were strategically placed to alleviate any problems with the architect's interior design. The beam and column placement can be seen below in the typical floor framing plan.

For a typical interior bay, it was determined that a W16x26 steel beam would be able to span the long direction between girders in order to resist the typical floor loads. A W16 x 31 girder spans the short direction of the bay which results in a 22-1/8" total floor depth, almost a 14" difference.

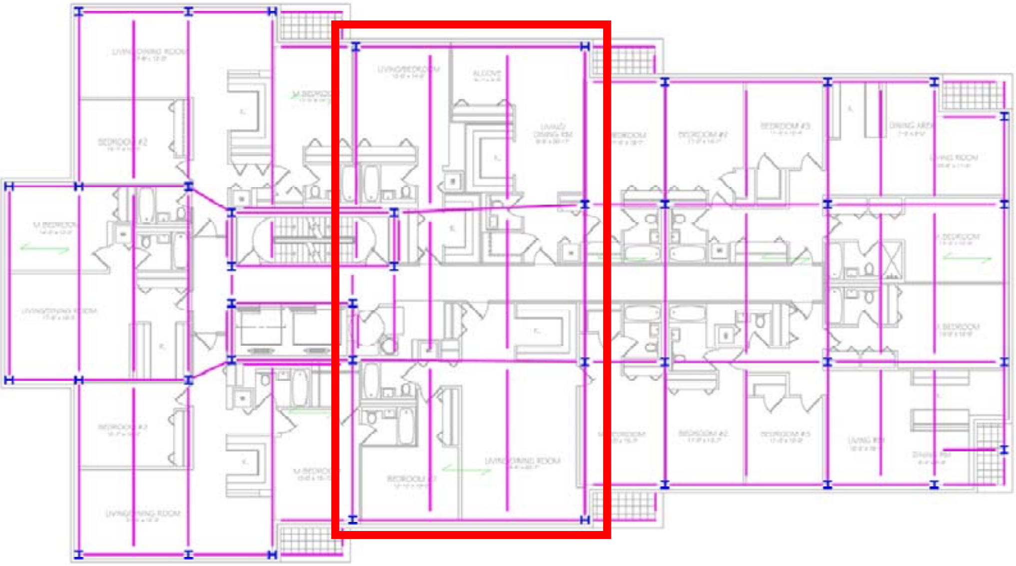
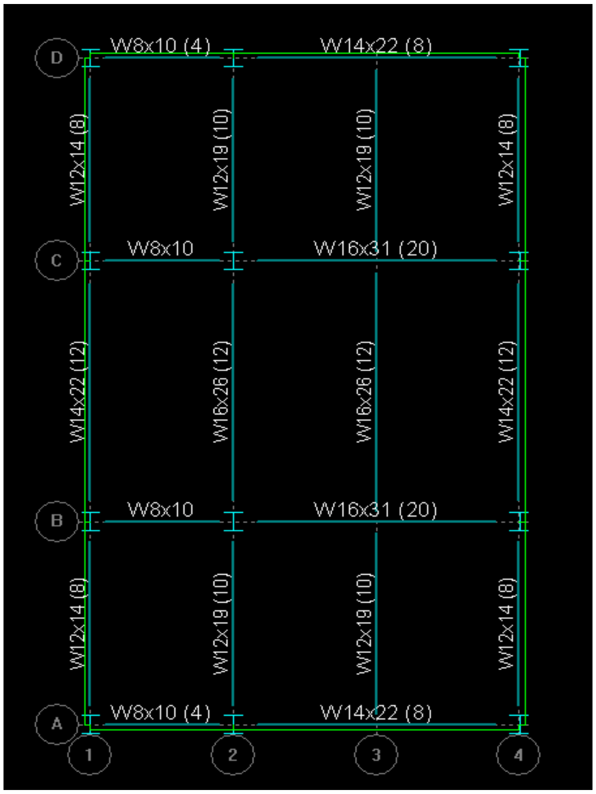


Figure 4: Framing Plan for Composite Steel Flooring System. Red box indicates the bays analyzed



PRECAST GIRDER-SLAB FLOOR SYSTEM

The Girder-Slab floor system is a patented, but non-proprietary, framing system for mid and high-rise residential construction. Specifically targeting apartments, condos, retirement communities, hotels, and other multi-story residential buildings, this unique flooring system has integrated the advantages of short span steel girders and long-span precast conclete slabs. The system is comprised of an interior girder known as an open-web dissymmetric beam, known as a D-Beam, which supports precast, prestressed hollow-core slabs on its bottom flange. The D-Beams also have openings in some of the web to allow for grouting of the hollow core planks. Upon grouting, the system develops composite action, allowing it to support residential live loads.

With this combination of materials, comes many advantages. The Girder-Slab system is able to achieve a comparable floor-to-floor depth with that of a flat plate floor system since the precast planks are connected to the bottom flange of the steel girder. Additionally, the hollow core planks result in a relatively low overall weight of the structure. This is a positive atribute when designing the foundation system. A lighter building can result in smaller foundations and less additional costs. For lateral resistance, a system of “goosenecks” can be used. The goosenecks are prefabricated columns with an 8” deep beam moment connection to the columns as seen in Figure .

Although this system reduces erection time and cost and eliminates weather issues, there are still some drawbacks. The system requires a lot of lead time to assemble the D-beams. Also, a lot of coordination between the steel and precast concrete contractors must be performed in order for the construction to remain on time and on budget. Additional fireproofing must also be added to the steel beams that are exposed.

To efficiently use the Girder-Slab system, the building must be broken into typical bays. As seen in Figure , the layout of Southtown Building No. 5 was redesigned to accommodate for the Girder-Slab system. An interior bay of 27’-0” x 16’-0” was used to analyze this flooring system. Precast planks span the long direction of the bays and the D-Beam girders span the short direction. To adequately span the 27’ direction, an 8” x 4’-0” hollow core plank was used with a 2” topping to further improve the stiffness and acoustics of the floor system. An overall depth of approximately 10” can be achieved which is very close to the 8” floor of the existing flat plate floor slab.

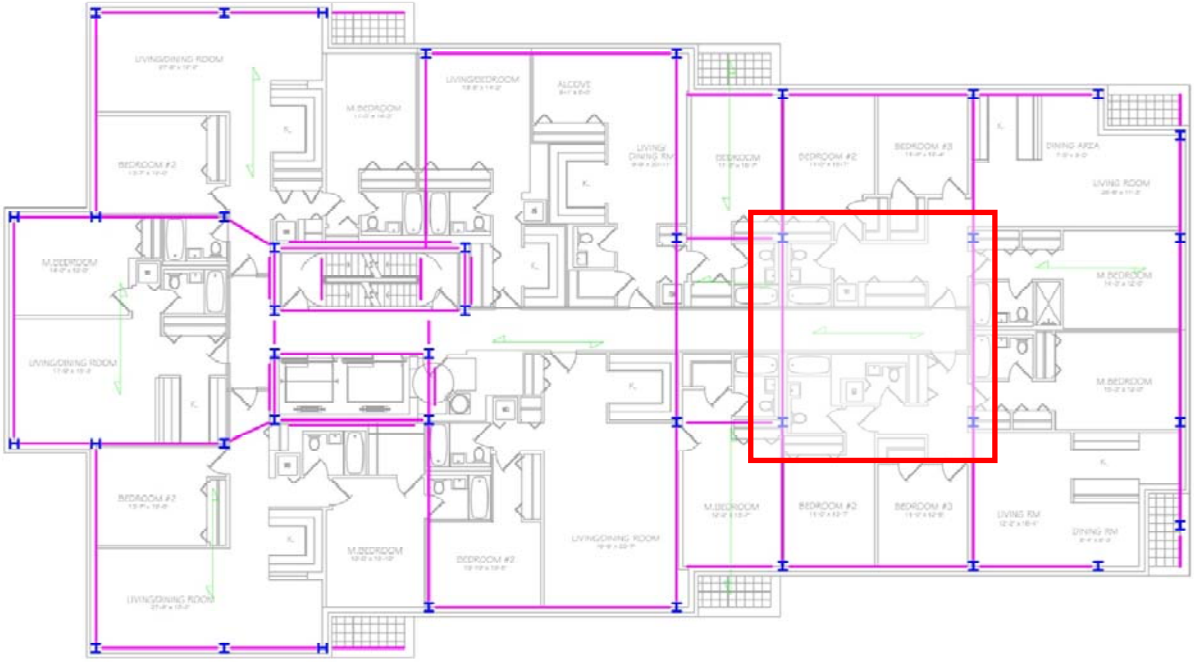


Figure 6: Framing Plan for Girder-Slab flooring system. Red box indicates the interior bay that was analyzed.

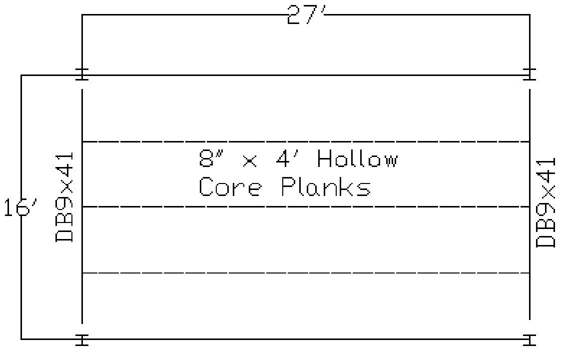


Figure 7: Typical Bay used for Girder-Slab floor system

Figure 8: Gooseneck column used in Girder-Slab construction

TWO-WAY WAFFLE SLAB FLOOR SYSTEM

The waffle slab is a monolithic-poured concrete slab with a flat top surface and an under-surface made of a rectangular grid of deep concrete beams running at right angles. Waffle slabs become economical over flat slabs for long spans and heavy loads. Being that the waffle slab incorporates a square bay system; the floor plan was changed to allow for normal 24’-0” x 24’-0” bays. This spacing was used to accommodate the architect’s apartment layout.

Although this floor system is used seldom used in residential construction, there are some advantages to this system for mid to high-rises. The waffle slab provides adequate fireproofing without having to add any additional material to achieve the necessary rating. There is almost no lead time required for this system since cast-in-place concrete is readily available. Also, the waffle slab offers a very rigid floor system and provides little vibration. Finally, the waffle slab provides for a small floor thickness. With a 3” slab and an 8” rib, the overall floor thickness is 11”, only 3” thicker than the original 8” slab.

Consequently, waffle slabs do have some drawbacks as well. The waffle slab requires a lot of formwork and therefore can be overly expensive when compared to other systems. This can slow up the construction process significantly. Also, since the underside of the slab is irregularly shaped, it is tough to coordinate and hang the mechanical and electrical equipment. This can increase the floor-to-floor heights of the building, decreasing the total amount of leasable space. The use of the waffle slab floor system also increases the total weight of the building. The two-way concrete joists will add additional weight to the building which may result in an increased foundation size. This can also cause seismic loading to control for the building’s design. This would possibly result in a change in design of the lateral resisting system.

The design of the waffle slab was computed using the Concrete Reinforcing Steel Institute Handbook. After the design was finished, it was determined that 30” x 30” voids were necessary to provide adequate capacity for the 230 psf superimposed load. In order to span the 24’ span, a 3” slab and 8” rib was used. The reinforcement provided from the table for an interior panel consisted of a series of #5 bars in both direction for the Column Strip and Middle Strip. A more complete analysis of this floor system can be seen in the Appendix D.

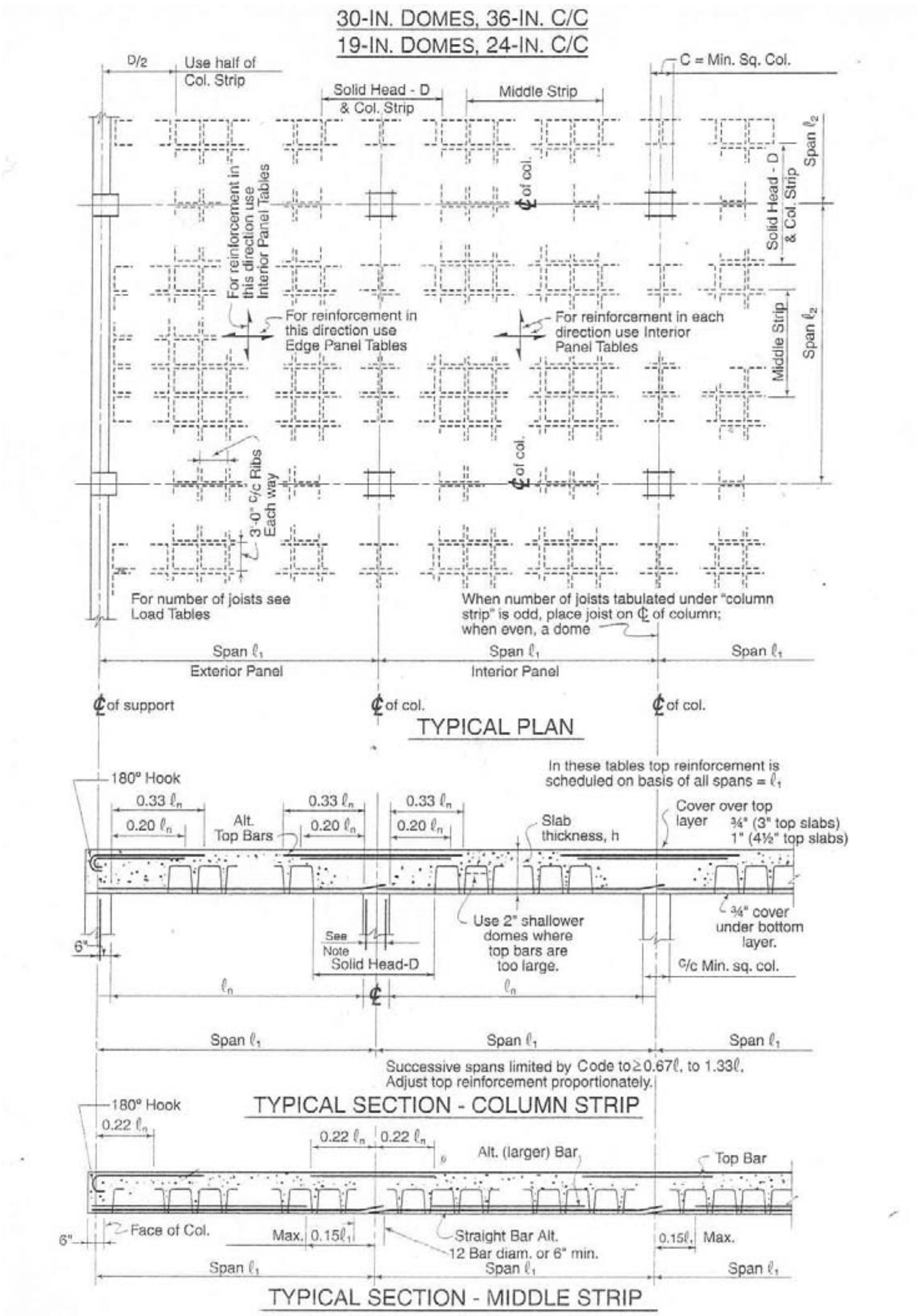


Figure 9: Typical Plan and Sections for Waffle Slab Construction

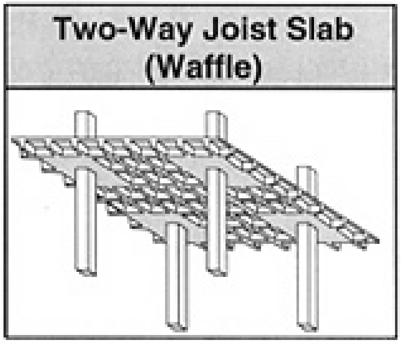


Figure 10: Waffle Slab Diagram

POST-TENSIONED FLAT PLATE FLOOR SYSTEM

Post-tensioned concrete seeks to ease concrete’s natural weakness in tension by imposing a permanent compression load on the structural members. With post-tensioned concrete, high-strength steel tendons, in combination with reinforcing steel bars, are embedded and anchored in the concrete. After the concrete has reached adequate strength, the tendons are tensioned which imposes a compression force on the concrete. These tendons, in addition to the rebar, enable thinner structural members with increased load capacities and less deflection.

Post-tensioned concrete has several noteworthy advantages. Low material costs and availability of concrete are evident in today’s market. With steel becoming very expensive, many residential buildings are being constructed with concrete. Post-tensioned concrete is typically used for moderate to long spans with moderate floor loads. This usually results in shallower depths than that of steel framing. Construction schedule is also a great benefit with post-tensioned floor slabs. Due to the quick strengthening properties of the material, the construction of successive floors can be built within a week of concrete placement. In conventional reinforced slabs, the construction of successive floors must often be delayed until the concrete has gained enough strength to support its own weight. There is essentially no lead time with post-tensioned concrete floor systems.

However, some disadvantages do occur for this floor system. For many buildings, such as residential, the frequency of remodeling and tenant improvements is a big issue. Walls may be moved and holes may be cut into the floor slab to accommodate for these revisions. For these reasons, post-tensioned concrete buildings are more difficult to remodel because of the reinforcing steel embedded in the concrete. It would take time and effort to locate the tendons and rebar to make sure none of it is accidentally pierced. However, sleeves for future penetrations can be cast into the slab prior to pouring the concrete. Finally, the increased load of the cast-in-place concrete can add additional load to the building and possibly result in larger foundation sizes.

For Southtown Building No. 5, a 27’-0” x 24’-0” bay was analyzed using 5000 psi concrete. This resulted in a 7-1/2“ floor slab depth. Banded tendons will span the short direction at the column strips with an effective compressive force of 394 kips. Evenly distributed tendons will span the long direction and have an effective compressive force of 18.5 kips per foot. Uniform #4 bars span the top and bottom direction each way with additional #4 bars at the columns. A more detailed analysis of this can be seen in the Appendix E.

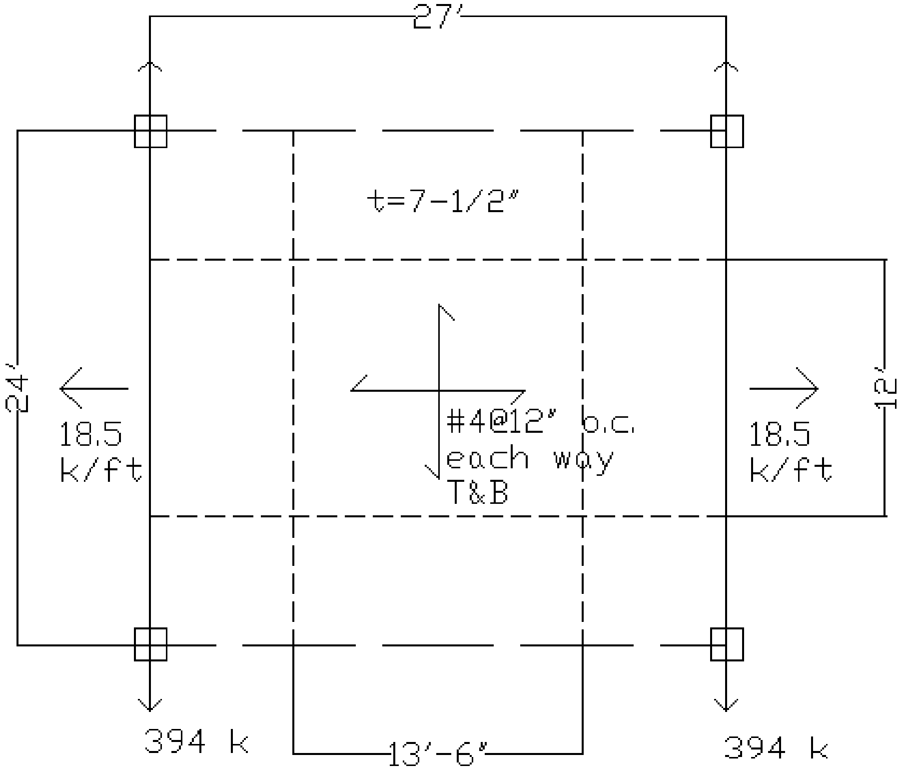


Figure 11: Typical Bay used to analyze Post-Tensioned floor system (Shown with results)

CONCLUSION

After all four alternative designs were analyzed, it was concluded that best alternative to the existing system is the Girder-Slab system. This system allows for a comparable floor thickness with that of the existing two-way flat plate. With the floor-to-floor height being one of the most important aspects in New York City buildings, this system can achieve a 10” floor thickness. Other strong points of this system are its cost and ease of constructability. Only costing $10.05 a square foot, this system is reasonably cheaper than any other flooring system. The reduced weight of the girder slab system would also have a positive impact on the building. This could potentially reduce column sizes at the lower levels and lessen the total weight on the foundation. The lateral system would need to be analyzed further for this system but the use of braced frames around the center core would further reduce the overall weight of the system. Additionally, the 2” topping on this rigid floor system would allow for good sound attenuation between adjacent rooms, something that the flat plate floor system might lack.

However, there are some characteristics of this system that might be a drawback to the existing floor system. The addition of structural steel to the project may slow down the construction process if the coordination between contractors is not handled properly. Also, the lead time with this system might offset any schedule gains that may be achieved. Additionally, the column grid would need to be altered. This would not necessarily affect the architect’s original apartment layout, but it may affect some other mechanical or electrical layouts. These things would need to be analyzed further to make a clear cut decision on which floor system to use. Given the fact that the building is located in New York City and the fast-track construction process of flat plate concrete floors, I believe that the existing floor system is the best choice.

Table 1: System Comparison

System	Existing Two-Way Flat Plate	Composite Floor System	Girder-Slab Floor System	Two-Way Waffle Slab	Post-Tensioned Floor System
Weight (psf)	210	125	115		195
Depth (in.)	8	22.13	10	11	7.5
Vibration	NO	YES	NO	NO	NO
Constructability	MEDIUM	MEDIUM	EASY	MEDIUM	HARD
Lead Time	SHORT	LONG	MEDIUM	SHORT	SHORT
Formwork	YES	NO	NO	YES	YES
Fireproofing	NO	YES	YES	NO	NO
Cost ($/SF)	11.15	12.8	10.05	13.95	13.9
Column Grid Changes	N/A	YES	YES	NO	YES
Lateral System Effects	N/A	YES	YES	NO	NO

APPENDIX

APPENDIX A: EXISTING FLOOR SYSTEM

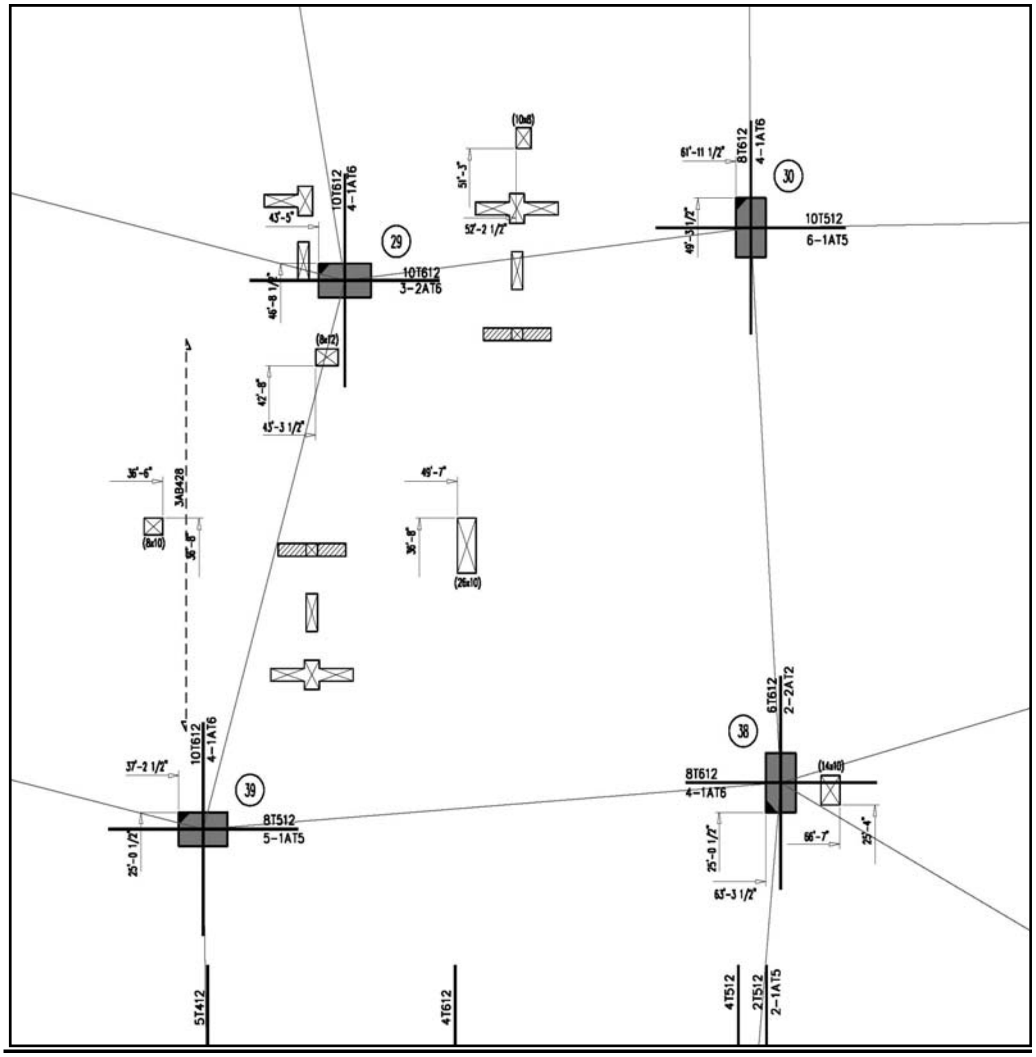


Figure 12: Typical Bay Used for Gravity Load Check

2- Way Flat Plate Floor System Spot Check

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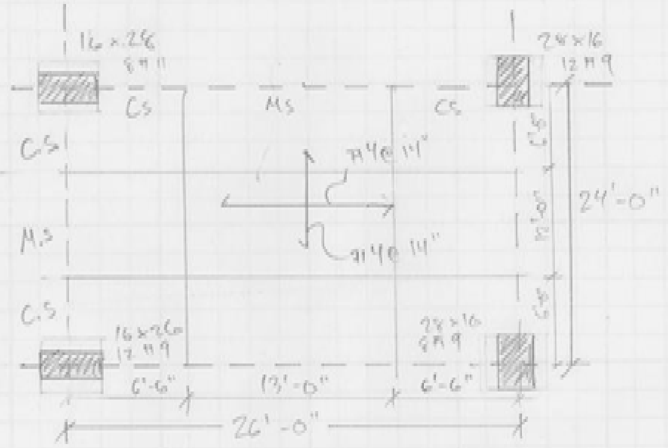
FLAT PLATE FLOOR SLAB SPOT CHECK

WORST CASE SCENARIO : 26'-0" x 24'-0" w/ 8" SLAB INT. SPAN



SLAB: t = 8"
f'_c = 5ksi

REINF: BOTTOM; #4 @ 14"
TOP: #4 @ 14"

COLS: f'_c = 6 ksi
10 ADD'L #C BARS @ 12" (BOTH WAYS) @ SUPPORT

* #4 @ 14 MIDSTRIP BARS BETWEEN COLUMN SUPPORTS.

LOADS

DEAD	
8" SLAB	100 psf
COLUMNS	10 psf
MEP	10 psf
PARTITIONS	20 psf
FIN & MISC	5 psf
	145 psf

LIVE
RESIDENTIAL 40 psf

$$W_u = \text{Max}\begin{cases} = 1.2(145\text{ psf}) + 1.6(40\text{ psf}) = \underline{238\text{ psf}} \\ = 1.4(145\text{ psf}) = 203\text{ psf} \end{cases}$$

MIN REINFORCEMENT = $0.0018\,A_g = 0.0018(12")(8") = 0.1728$ in²/ft
$= 0.204$ in²/14"

#4 = 0.2 in²

#4 @ 14" BOTH WAYS ∴ $A_s = 0.4$ in² > 0.204 in²

$\underline{\text{OK}}$

2/4

FRAMING SYSTEM IS FLATE PLATE w/ OUT EDGE BEAMS

DISTRIBUTION OF M_o — INTERIOR NESTED SPAN

M_{int}^- 0.65

M^+ 0.35

$$M_o = \frac{w_u l_2 l_n^2}{8}$$

① $$M_o = \frac{(238)(24')(24' - 28''/12''/')^2}{8} = 335.2^{'k}$$

② $$M_o = \frac{(238)(24')(24' - 16''/12''/')^2}{8} = 366.8^{'k}$$

③ $$M_o = \frac{(238)(26)(26 - 21''/12''/')^2}{8} = \underline{455^{'k}}$$ ← CONTROL

LOCATION	STRIP	MOMENT	WIDTH	Mu/WIDTH
SUPPORT .65M_o	C_s 75%	$222^{'k}$	12'-0"	$18.5\ ^{'k}/ft$
	M_s 25%	$74^{'k}$	12'-0"	$6.2\ ^{'k}/ft$
MIDSPAN .35M_o	C_s 60%	$95.5^{'k}$	13'-0"	$7.3\ ^{'k}/ft$
	M_s 40%	$63.7^{'k}$	13'-0"	$4.9\ ^{'k}/ft$

COLUMN STRIP

SUPPORT: $$M_n = \frac{M_u}{\phi} = \frac{222^{'k}}{0.9} = 247^{'k}$$

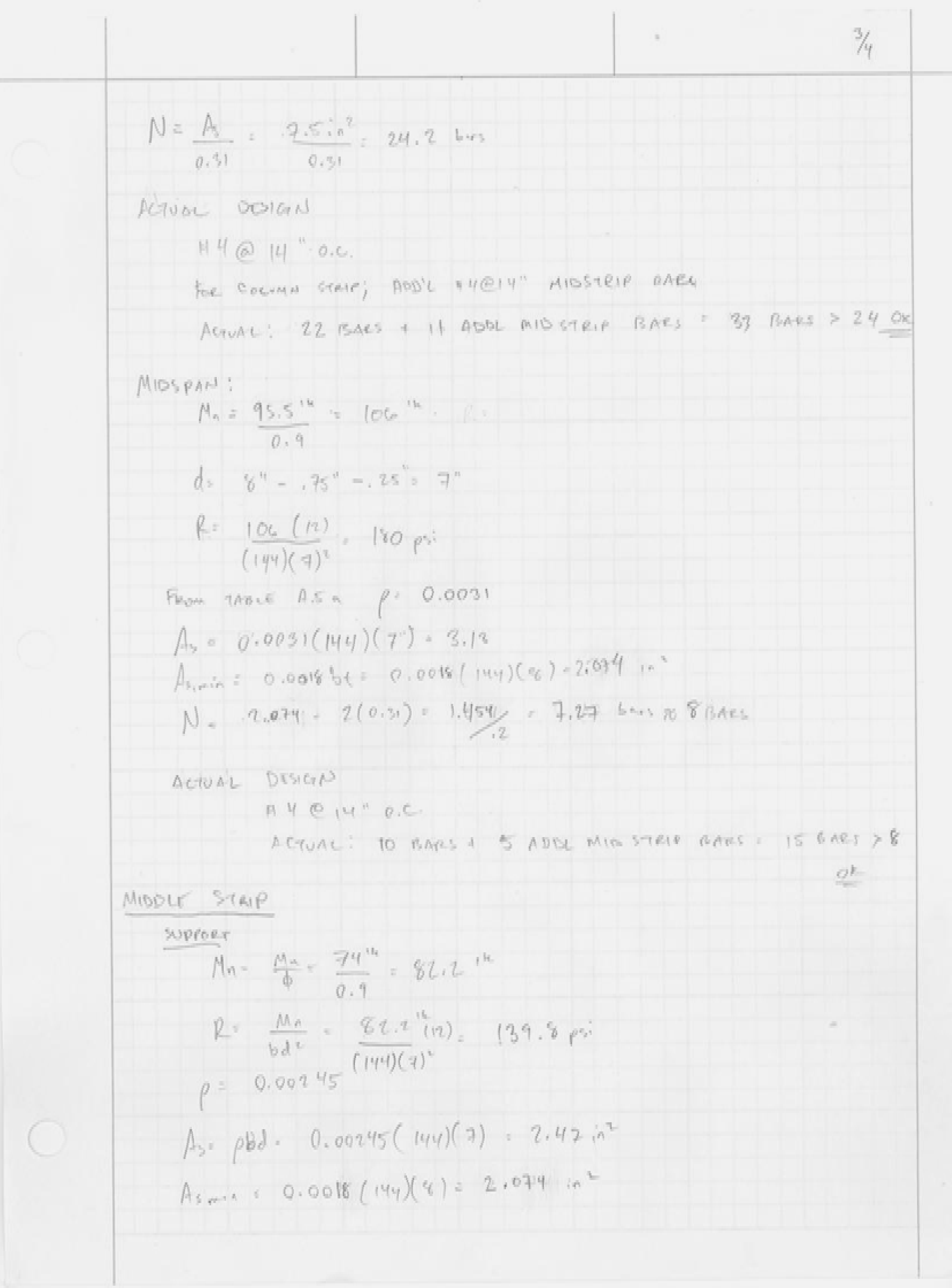
$$d = 8'' - \overset{\text{clear cover}}{0.75''} - 0.25'' = 7''$$

$$R = \frac{M_n}{bd^2} = \frac{247^{'k}(12'')}{144''(7'')^2} = 420\,psi$$

FROM TABLE A.5 A $\rho = 0.0074$

$A_s = \rho b d = 0.0074(144)(7'') = 7.5\,in^2$

$A_{s\,min} = 0.0018\,bt = 0.0018(144'')(8'') = 2.074\,in^2$



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$N = \frac{A_s}{0.31} = \frac{7.5\,in^2}{0.31} = 24.2$ bars

ACTUAL DESIGN

#4 @ 14" O.C.

FOR COLUMN STRIP; ADD'L #4@14" MIDSTRIP BARS

ACTUAL: 22 BARS + 11 ADDL MIDSTRIP BARS = 33 BARS > 24 OK

MIDSPAN:

$M_n = \frac{95.5^{'k}}{0.9} = 106^{'k}$

$d = 8" - .75" - .25" = 7"$

$R = \frac{106\,(12)}{(144)(7)^2} = 180$ psi

FROM TABLE A.5 a $\rho = 0.0031$

$A_s = 0.0031(144)(7") = 3.13$

$A_{s,min} = 0.0018\,bt = 0.0018(144)(8) = 2.074$ in²

$N = 2.074 - 2(0.31) = \frac{1.454}{.2} = 7.27$ bars → 8 BARS

ACTUAL DESIGN

#4 @ 14" O.C.

ACTUAL: 10 BARS + 5 ADDL MID STRIP BARS = 15 BARS > 8 OK

MIDDLE STRIP

SUPPORT

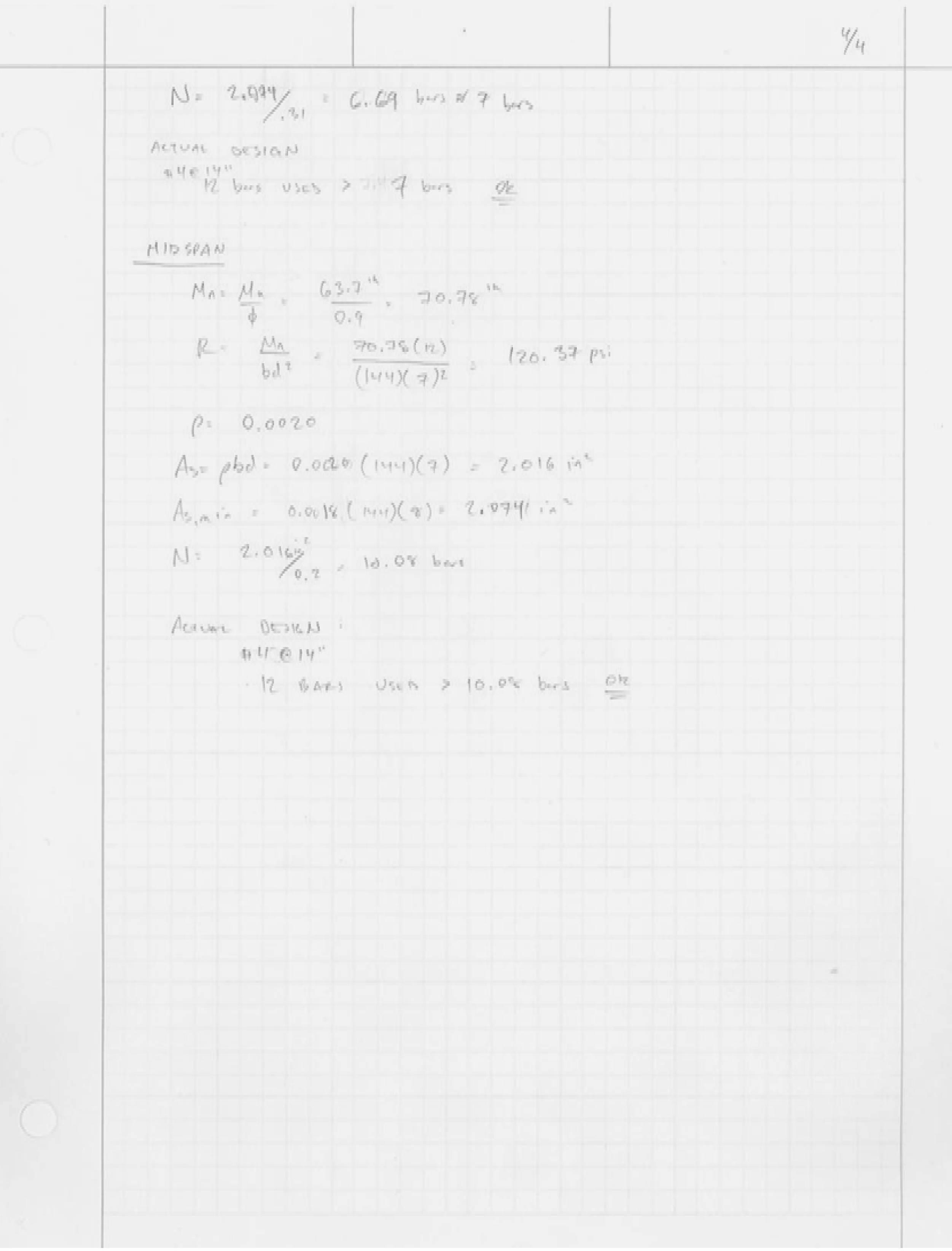
$M_n = \frac{M_u}{\phi} = \frac{74^{'k}}{0.9} = 82.2^{'k}$

$R = \frac{M_n}{bd^2} = \frac{82.2^{'k}(12)}{(144)(7)^2} = 139.8$ psi

$\rho = 0.00245$

$A_s = \rho bd = 0.00245(144)(7) = 2.47\,in^2$

$A_{s,min} = 0.0018(144)(8) = 2.074\,in^2$



4/4

$N = 2.074 / .31 = 6.69$ bars ≈ 7 bars

ACTUAL DESIGN
#4 @ 14"
12 bars uses > 7 bars $\underline{\underline{ok}}$

<u>MIDSPAN</u>

$M_n = \frac{M_u}{\phi} = \frac{63.7^{k}}{0.9} = 70.78^{k}$

$R = \frac{M_n}{bd^2} = \frac{70.78(12)}{(144)(7)^2} = 120.37$ psi

$\rho = 0.0020$

$A_s = \rho bd = 0.0020(144)(7) = 2.016$ in²

$A_{s,min} = 0.0018(144)(8) = 2.0741$ in²

$N = 2.016\text{in}^2 / 0.2 = 10.08$ bars

ACTUAL DESIGN :
#4 @ 14"
12 BARS USED > 10.08 bars $\underline{\underline{ok}}$

APPENDIX B: COMPOSITE STEEL

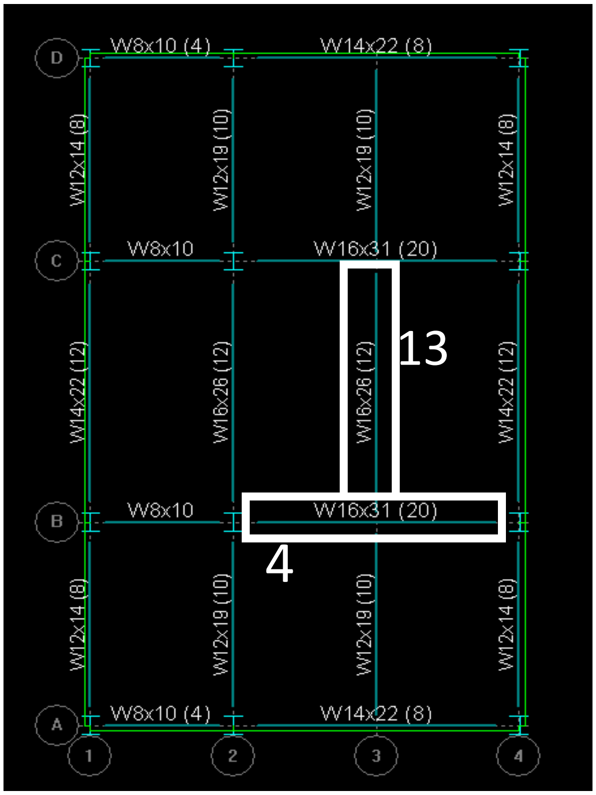
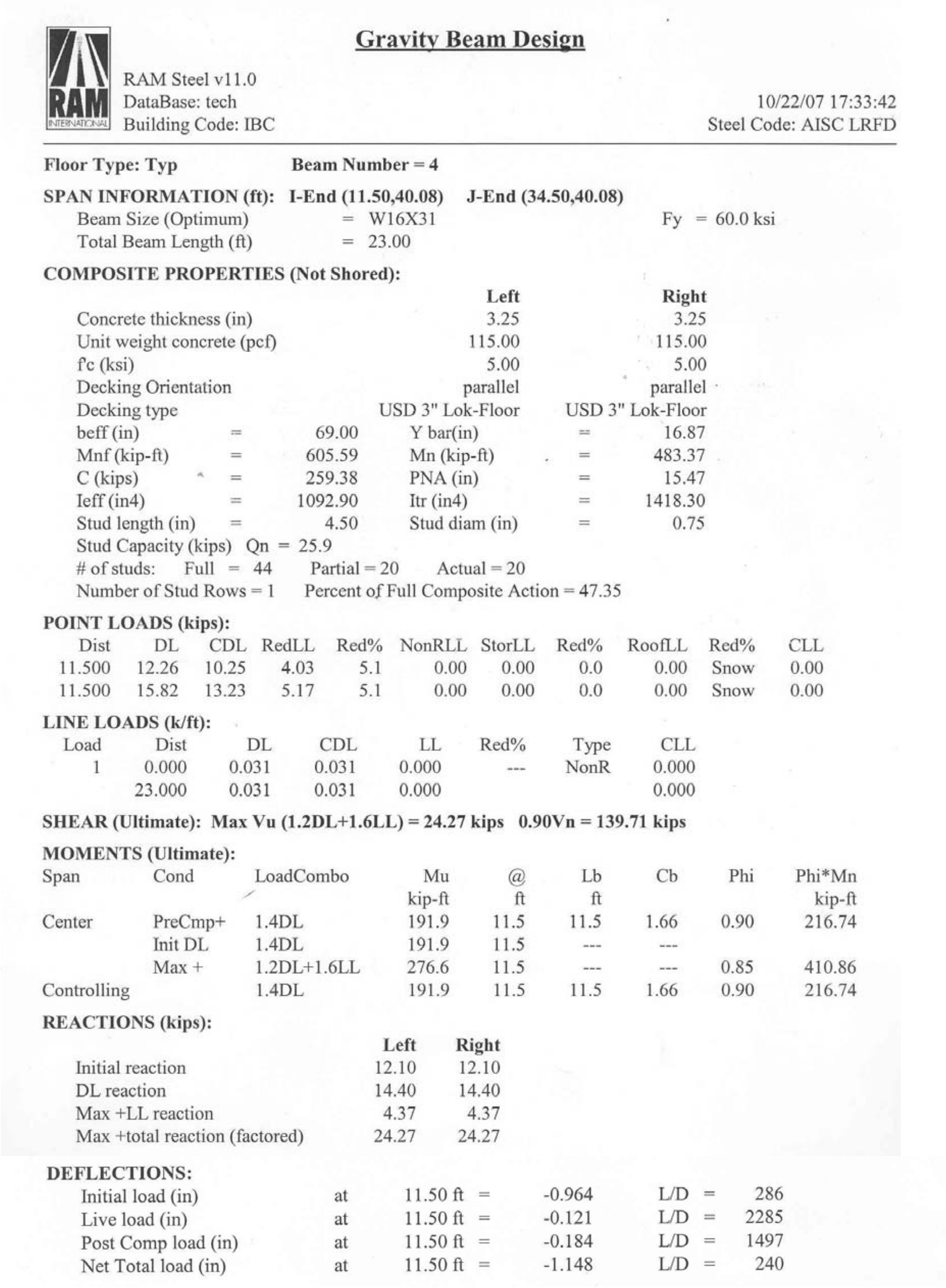
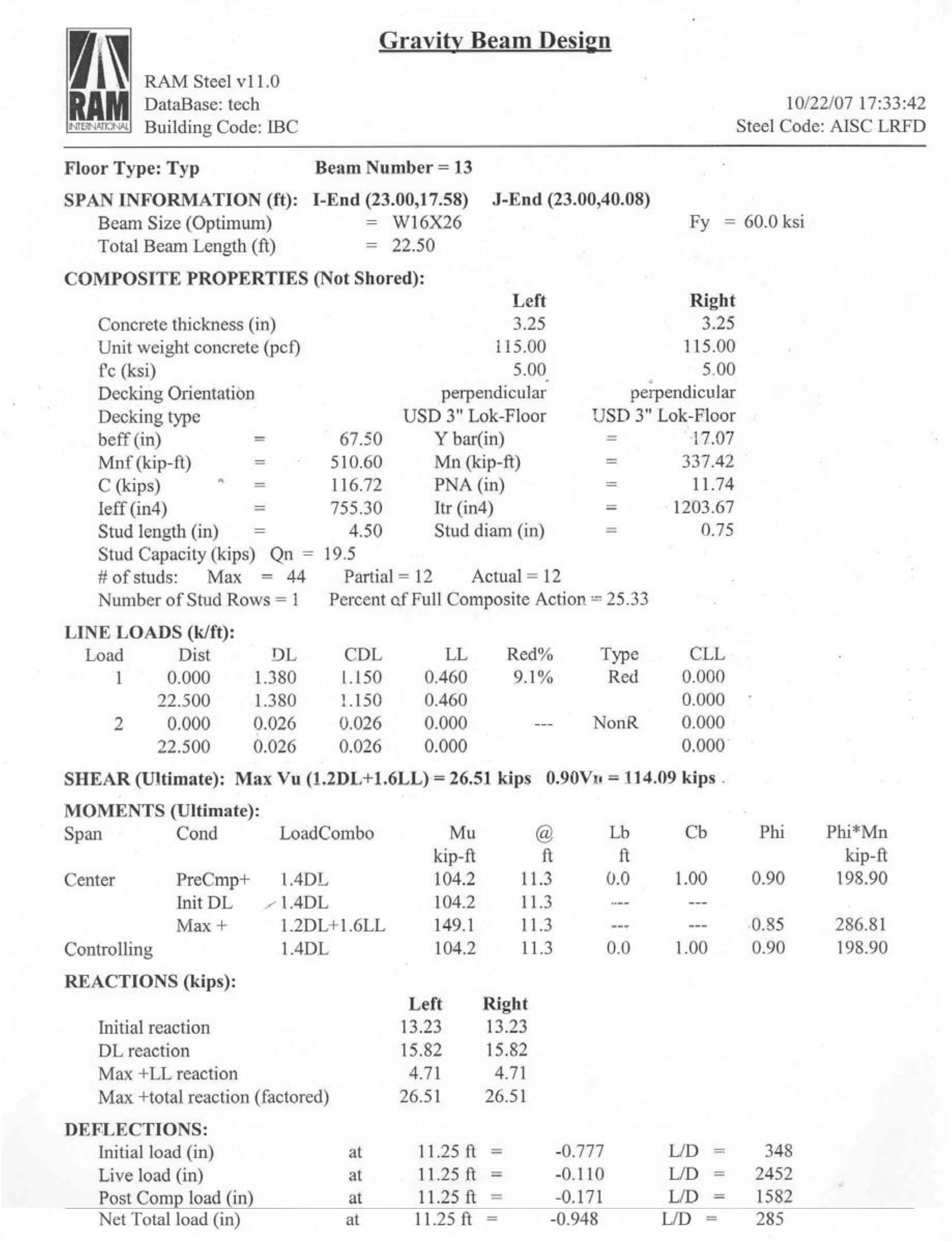


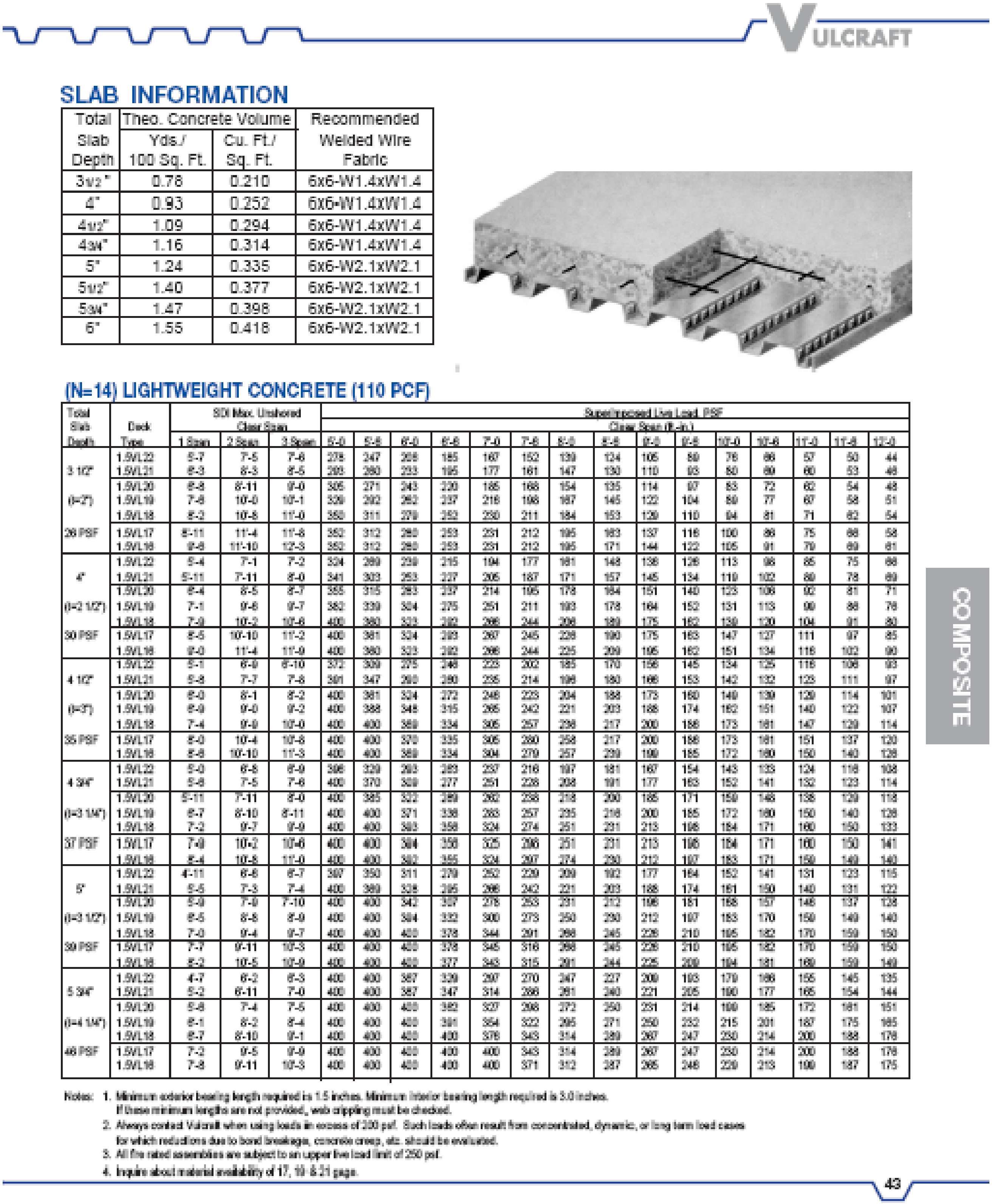
Figure 13: Typical bays analyzed in Ram Structural System. Beams with white boxes are beams with report on next page



```
                              Gravity Beam Design
RAM Steel v11.0
DataBase: tech                                                    10/22/07 17:33:42
Building Code: IBC                                           Steel Code: AISC LRFD

Floor Type: Typ              Beam Number = 4
SPAN INFORMATION (ft):  I-End (11.50,40.08)   J-End (34.50,40.08)
   Beam Size (Optimum)             =  W16X31                          Fy  =  60.0 ksi
   Total Beam Length (ft)          =  23.00
COMPOSITE PROPERTIES (Not Shored):
                                                   Left                Right
   Concrete thickness (in)                         3.25                 3.25
   Unit weight concrete (pcf)                    115.00               115.00
   f'c (ksi)                                       5.00                 5.00
   Decking Orientation                         parallel             parallel
   Decking type                       USD 3" Lok-Floor     USD 3" Lok-Floor
   beff (in)          =        69.00    Y bar(in)          =          16.87
   Mnf (kip-ft)       =       605.59    Mn (kip-ft)        =         483.37
   C (kips)           =       259.38    PNA (in)           =          15.47
   Ieff (in4)         =      1092.90    Itr (in4)          =        1418.30
   Stud length (in)   =         4.50    Stud diam (in)     =           0.75
   Stud Capacity (kips)  Qn  =  25.9
   # of studs:     Full  =  44      Partial = 20      Actual = 20
   Number of Stud Rows = 1      Percent of Full Composite Action = 47.35
POINT LOADS (kips):
    Dist      DL     CDL   RedLL    Red%   NonRLL  StorLL   Red%   RoofLL   Red%    CLL
  11.500   12.26   10.25    4.03     5.1     0.00    0.00    0.0     0.00   Snow   0.00
  11.500   15.82   13.23    5.17     5.1     0.00    0.00    0.0     0.00   Snow   0.00
LINE LOADS (k/ft):
   Load       Dist        DL       CDL        LL     Red%     Type      CLL
      1      0.000     0.031     0.031     0.000      ---     NonR    0.000
            23.000     0.031     0.031     0.000                      0.000
SHEAR (Ultimate):  Max Vu (1.2DL+1.6LL) = 24.27 kips   0.90Vn = 139.71 kips
MOMENTS (Ultimate):
Span         Cond        LoadCombo          Mu        @       Lb       Cb      Phi     Phi*Mn
                                         kip-ft       ft       ft                       kip-ft
Center       PreCmp+     1.4DL            191.9     11.5     11.5     1.66     0.90     216.74
             Init DL     1.4DL            191.9     11.5      ---      ---
             Max +       1.2DL+1.6LL      276.6     11.5      ---      ---     0.85     410.86
Controlling              1.4DL            191.9     11.5     11.5     1.66     0.90     216.74
REACTIONS (kips):
                                        Left      Right
   Initial reaction                    12.10      12.10
   DL reaction                         14.40      14.40
   Max +LL reaction                     4.37       4.37
   Max +total reaction (factored)      24.27      24.27
DEFLECTIONS:
   Initial load (in)          at     11.50 ft  =     -0.964     L/D  =     286
   Live load (in)             at     11.50 ft  =     -0.121     L/D  =    2285
   Post Comp load (in)        at     11.50 ft  =     -0.184     L/D  =    1497
   Net Total load (in)        at     11.50 ft  =     -1.148     L/D  =     240
```



```
                                 Gravity Beam Design

RAM Steel v11.0
DataBase: tech                                                        10/22/07 17:33:42
Building Code: IBC                                                Steel Code: AISC LRFD

Floor Type: Typ               Beam Number = 13
SPAN INFORMATION (ft):  I-End (23.00,17.58)   J-End (23.00,40.08)
   Beam Size (Optimum)              =   W16X26                         Fy  =  60.0 ksi
   Total Beam Length (ft)           =   22.50
COMPOSITE PROPERTIES (Not Shored):
                                                   Left                 Right
   Concrete thickness (in)                         3.25                  3.25
   Unit weight concrete (pcf)                    115.00                115.00
   f'c (ksi)                                       5.00                  5.00
   Decking Orientation                    perpendicular         perpendicular
   Decking type                        USD 3" Lok-Floor      USD 3" Lok-Floor
   beff (in)          =      67.50     Y bar(in)         =              17.07
   Mnf (kip-ft)       =     510.60     Mn (kip-ft)       =             337.42
   C (kips)           =     116.72     PNA (in)          =              11.74
   Ieff (in4)         =     755.30     Itr (in4)         =            1203.67
   Stud length (in)   =       4.50     Stud diam (in)    =               0.75
   Stud Capacity (kips)   Qn  =  19.5
   # of studs:     Max    =  44      Partial = 12      Actual = 12
   Number of Stud Rows = 1     Percent of Full Composite Action = 25.33
LINE LOADS (k/ft):
  Load      Dist      DL      CDL       LL      Red%      Type       CLL
     1     0.000   1.380    1.150    0.460      9.1%      Red      0.000
          22.500   1.380    1.150    0.460                         0.000
     2     0.000   0.026    0.026    0.000       ---      NonR     0.000
          22.500   0.026    0.026    0.000                         0.000
SHEAR (Ultimate):  Max Vu (1.2DL+1.6LL) = 26.51 kips   0.90Vn = 114.09 kips
MOMENTS (Ultimate):
Span         Cond       LoadCombo          Mu       @       Lb       Cb      Phi     Phi*Mn
                                       kip-ft      ft       ft                        kip-ft
Center       PreCmp+    1.4DL           104.2    11.3      0.0     1.00     0.90     198.90
             Init DL    1.4DL           104.2    11.3      ---      ---
             Max +      1.2DL+1.6LL     149.1    11.3      ---      ---     0.85     286.81
Controlling             1.4DL           104.2    11.3      0.0     1.00     0.90     198.90
REACTIONS (kips):
                                          Left    Right
   Initial reaction                      13.23    13.23
   DL reaction                           15.82    15.82
   Max +LL reaction                       4.71     4.71
   Max +total reaction (factored)        26.51    26.51
DEFLECTIONS:
   Initial load (in)            at      11.25 ft  =     -0.777      L/D  =     348
   Live load (in)               at      11.25 ft  =     -0.110      L/D  =    2452
   Post Comp load (in)          at      11.25 ft  =     -0.171      L/D  =    1582
   Net Total load (in)          at      11.25 ft  =     -0.948      L/D  =     285
```



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

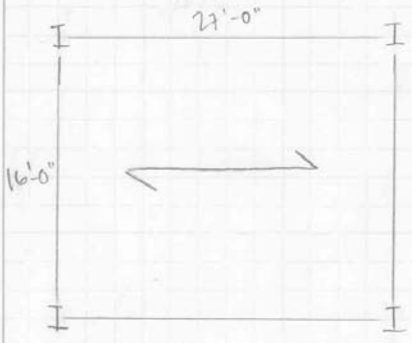
(N=14) LIGHTWEIGHT CONCRETE (110 PCF)

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	208	185	167	152	139	124	105	89	78	68	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
28 PSF	1.5VL17	8-11	11-4	11-8	352	312	280	253	231	212	195	183	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	289	239	215	194	177	161	148	138	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	108	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	381	324	293	267	245	228	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	182	151	134	116	102	90
	1.5VL22	5-1	6-9	6-10	372	309	275	248	223	202	185	170	158	145	134	125	116	108	93
4 1/2"	1.5VL21	5-8	7-7	7-8	391	347	290	260	235	214	196	180	168	153	142	132	123	111	97
	1.5VL20	6-0	8-1	8-2	400	381	324	272	248	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	10-6	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	128
	1.5VL22	5-0	6-8	6-9	398	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	338	283	257	235	216	200	185	172	160	150	140	128
	1.5VL18	7-2	9-7	9-9	400	400	393	358	324	274	251	231	213	198	184	171	160	150	133
37 PSF	1.5VL17	7-9	10-2	10-6	400	400	394	358	325	286	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	389	328	295	268	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	198	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	268	245	228	210	195	182	170	159	150
39 PSF	1.5VL17	7-7	9-11	10-3	400	400	400	378	345	316	268	245	228	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	5-8	7-4	7-5	400	400	400	382	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.

APPENDIX C: GIRDER-SLAB

GIRDER-SLAB SYSTEM



LIVE LOAD = 40 psf

DEAD LOAD =

PLANK DL = 60 psf
PARTITION = 20 psf
MEP = 10 psf
FINISH & MISC = 5 psf

2" LT. WEIGHT TOPPING = 25 psf

$f'_{c\,PLANK} = 5$ ksi
$f'_{c\,GROUT} = 4$ ksi

DB 9x41 PROPERTIES:

STEEL SECTION	TRANSFORMED SECTION
$I_s = 159\ in^4$	$I_t = 332\ in^4$
$S_t = 24.4\ in^3$	$S_t = 62.1\ in^3$
$S_b = 51.0$	$S_b = 77.7\ in^3$
$M_{s\,CAP} = 61.0^{k\text{-}ft}$	$b = 5.25\ in$
$t_w = 0.375\ in$	

ALLOWABLE $\Delta_{LL} = L/240 = (16')(12^{in}/_{ft})/240 = 0.8$

INITIAL LOAD - PRECOMPOSITE

$$M_{DL} = (27')(0.06\ ksf)(16')^2/8 = 51.84 < 61.0\ k\text{-}ft \quad \underline{\underline{ok}}\ \text{USE DB 9x41}$$

$$\Delta_{DL} = \frac{(5)(27')(0.06\ ksf)(16)^4(1728\ in^3/ft^3)}{384(159\ in^4)(29{,}000\ k/in^2)} = 0.52\ in < L/240 = 0.8$$

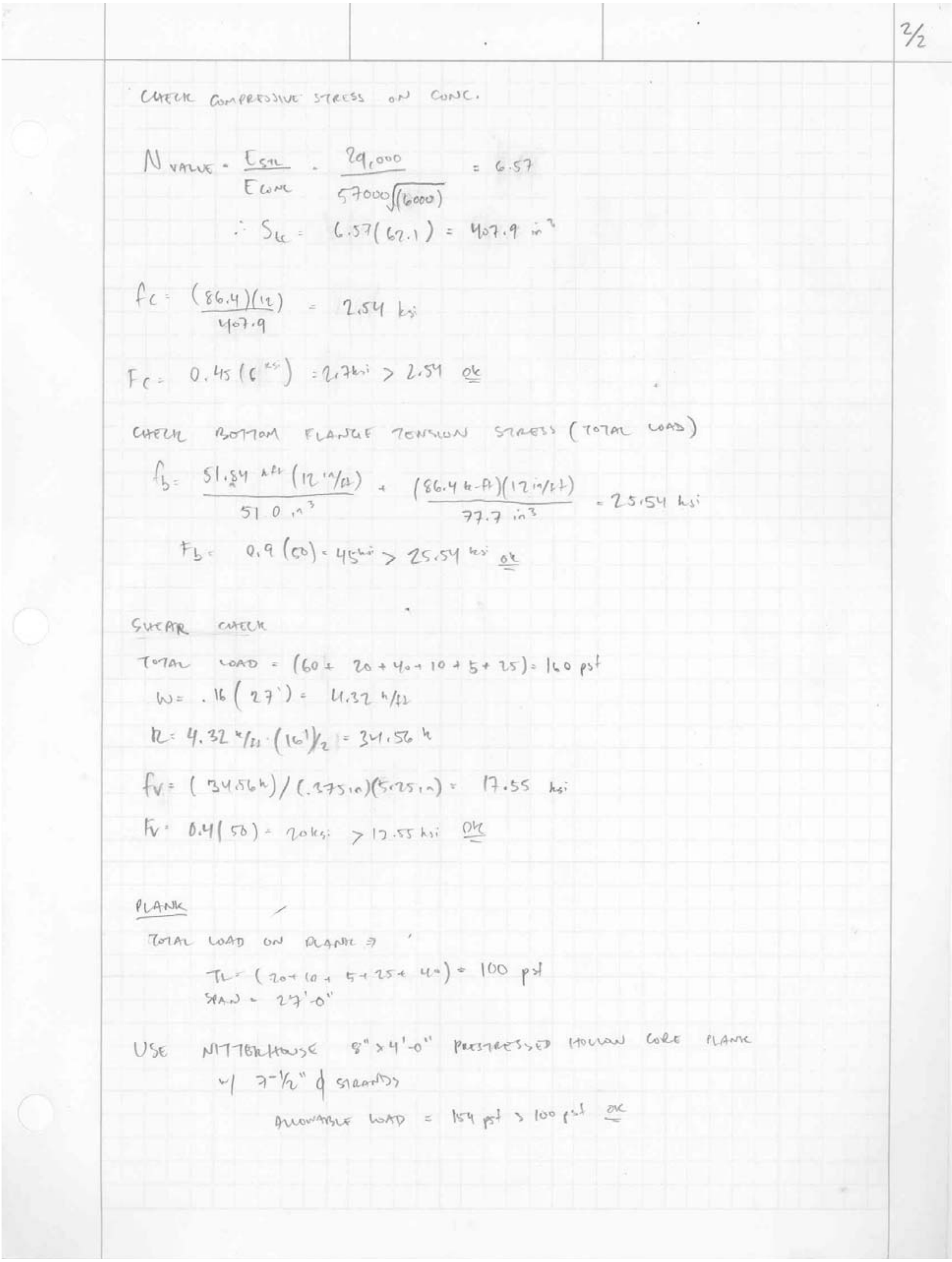
TOTAL LOAD - COMPOSITE

$$M_{SUP} = (27')(0.02 + 0.04 + 0.01 + 0.005 + 0.025)(16)^2/8 = 86.4^{k\text{-}ft}$$

$$M_{TL} = 51.84 + 86.4 = 138.24\ k\text{-}ft$$

$$S_{REQ} = (138.24)(12^{in}/_{ft})/(0.6)(50) = 55.29\ in^3 < 62.1\ in^3$$

$$\Delta_{SUP} = \frac{(5)(27')(0.1)(16)^4(1728\ in^3/ft^3)}{384(332)(29000)} = 0.414 < 0.8\ in \quad \underline{\underline{ok}}$$



2/2

CHECK COMPRESSIVE STRESS ON CONC.

$$N_{VALUE} = \frac{E_{STL}}{E_{CONC}} = \frac{29{,}000}{57000\sqrt{(6000)}} = 6.57$$

$$\therefore S_{tc} = 6.57(62.1) = 407.9 \text{ in}^3$$

$$f_c = \frac{(86.4)(12)}{407.9} = 2.54 \text{ ksi}$$

$$F_c = 0.45(6^{ksi}) = 2.7\text{ksi} > 2.54 \text{ ok}$$

CHECK BOTTOM FLANGE TENSION STRESS (TOTAL LOAD)

$$f_b = \frac{51.84 \text{ k-ft} (12 \text{ in/ft})}{51.0 \text{ in}^3} + \frac{(86.4 \text{ k-ft})(12 \text{ in/ft})}{77.7 \text{ in}^3} = 25.54 \text{ ksi}$$

$$F_b = 0.9(50) = 45^{ksi} > 25.54 \text{ ksi ok}$$

SHEAR CHECK

TOTAL LOAD = (60 + 20 + 40 + 10 + 5 + 25) = 160 psf

$$w = .16(27') = 4.32 \text{ k/ft}$$

$$R = 4.32 \text{ k/ft} \cdot (16')/2 = 34.56 \text{ k}$$

$$f_v = (34.56\text{k})/(.375\text{in})(5.25\text{in}) = 17.55 \text{ ksi}$$

$$F_v = 0.4(50) = 20\text{ksi} > 12.55 \text{ ksi OK}$$

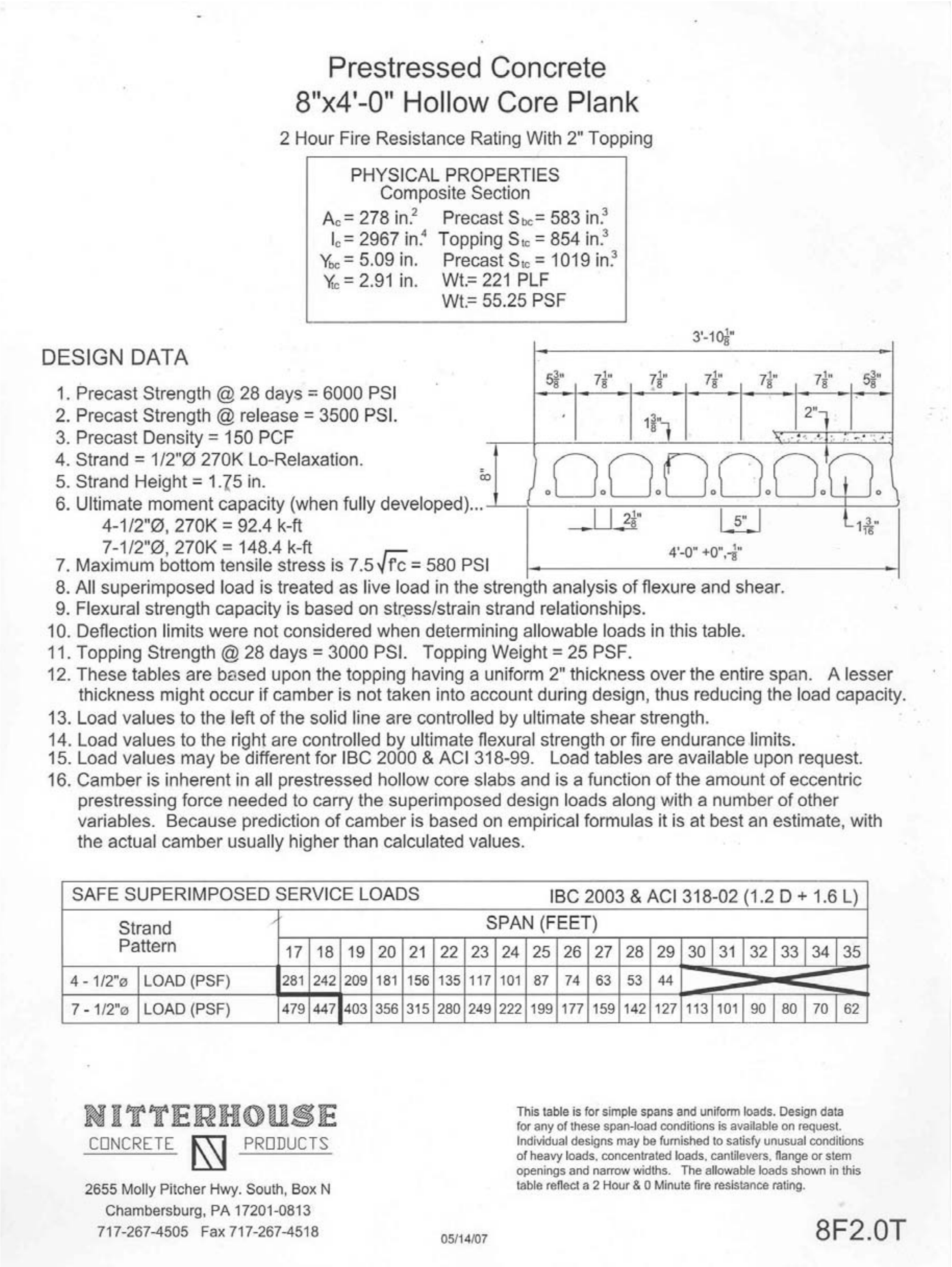
PLANK

TOTAL LOAD ON PLANK ⇒

TL = (20 + 10 + 5 + 25 + 40) = 100 psf
SPAN = 27'-0"

USE NITTERHOUSE 8" x 4'-0" PRESTRESSED HOLLOW CORE PLANK
w/ 7-½" ϕ STRANDS

ALLOWABLE LOAD = 154 psf > 100 psf OK



Prestressed Concrete
8"x4'-0" Hollow Core Plank

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES
Composite Section

A_c = 278 $in.^2$
I_c = 2967 $in.^4$
Y_{bc} = 5.09 in.
Y_{tc} = 2.91 in.
Precast S_{bc} = 583 $in.^3$
Topping S_{tc} = 854 $in.^3$
Precast S_{tc} = 1019 $in.^3$
Wt.= 221 PLF
Wt.= 55.25 PSF

DESIGN DATA

1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI.
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
   - 4-1/2"Ø, 270K = 92.4 k-ft
   - 7-1/2"Ø, 270K = 148.4 k-ft
7. Maximum bottom tensile stress is $7.5\sqrt{f'c}$ = 580 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. Deflection limits were not considered when determining allowable loads in this table.
11. Topping Strength @ 28 days = 3000 PSI. Topping Weight = 25 PSF.
12. These tables are based upon the topping having a uniform 2" thickness over the entire span. A lesser thickness might occur if camber is not taken into account during design, thus reducing the load capacity.
13. Load values to the left of the solid line are controlled by ultimate shear strength.
14. Load values to the right are controlled by ultimate flexural strength or fire endurance limits.
15. Load values may be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
16. Camber is inherent in all prestressed hollow core slabs and is a function of the amount of eccentric prestressing force needed to carry the superimposed design loads along with a number of other variables. Because prediction of camber is based on empirical formulas it is at best an estimate, with the actual camber usually higher than calculated values.

SAFE SUPERIMPOSED SERVICE LOADS — IBC 2003 & ACI 318-02 (1.2 D + 1.6 L)

Strand Pattern		SPAN (FEET)																		
		17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35
4 - 1/2"ø	LOAD (PSF)	281	242	209	181	156	135	117	101	87	74	63	53	44						
7 - 1/2"ø	LOAD (PSF)	479	447	403	356	315	280	249	222	199	177	159	142	127	113	101	90	80	70	62

NITTERHOUSE
CONCRETE PRODUCTS

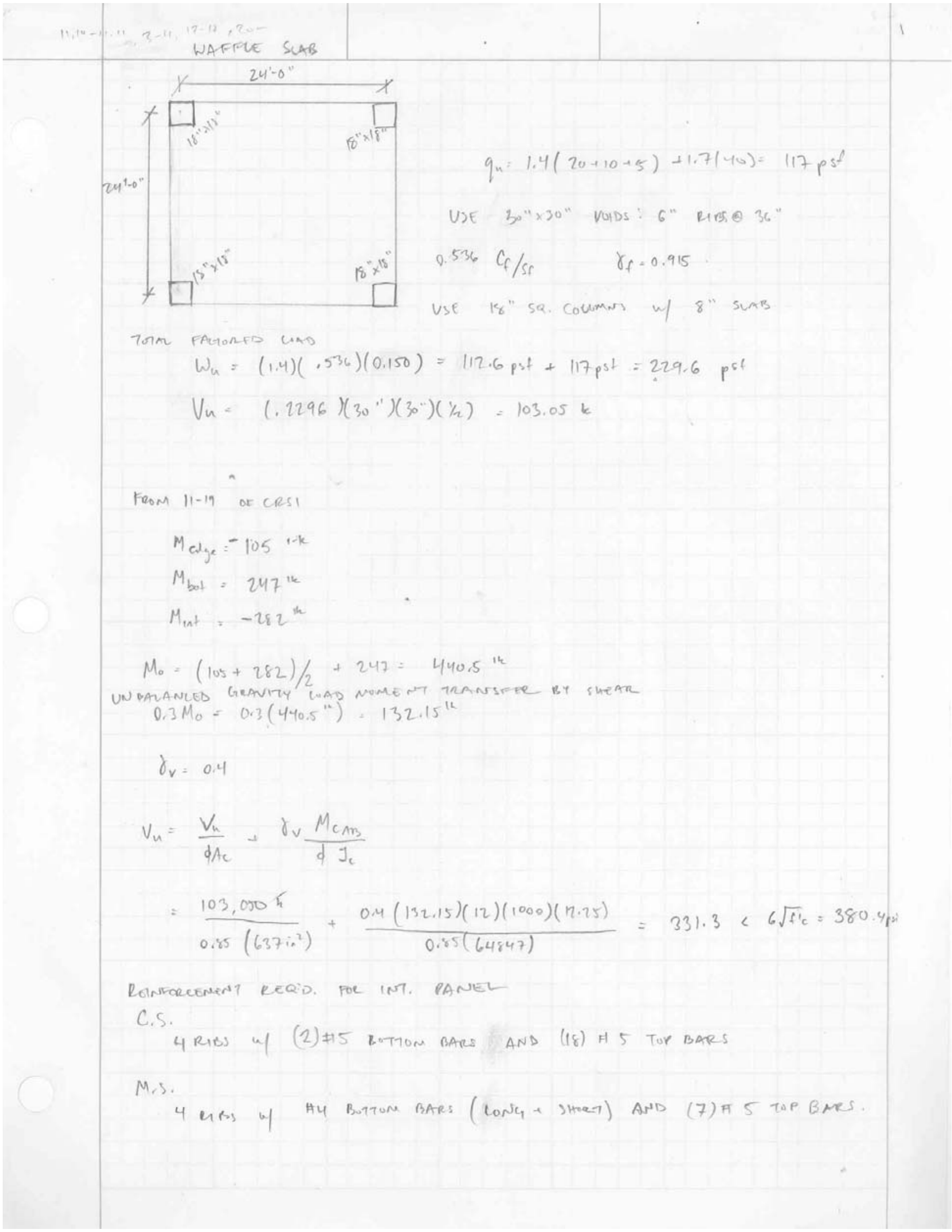
2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17201-0813
717-267-4505 Fax 717-267-4518

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. The allowable loads shown in this table reflect a 2 Hour & 0 Minute fire resistance rating.

05/14/07

8F2.0T

APPENDIX D: WAFFLE SLAB



11,14-11.11, 3-11, 17-17, 20-

WAFFLE SLAB

24'-0"

24'-0"

18"x18"

18"x18"

18"x18"

18"x18"

$q_u = 1.4(20+10+5) + 1.7(40) = 117$ psf

USE 30"x30" VOIDS : 6" RIBS @ 36"

0.536 C_f/S_f $\gamma_f = 0.915$

USE 18" SQ. COLUMNS w/ 8" SLAB

TOTAL FACTORED LOAD

$W_u = (1.4)(.536)(0.150) = 112.6$ psf + 117 psf = 229.6 psf

$V_u = (.2296)(30')(30')(1/2) = 103.05$ k

FROM 11-19 OF CRSI

$M_{edge} = -105$ 'k

$M_{bot} = 247$ 'k

$M_{int} = -282$ 'k

$M_o = (105 + 282)/2 + 247 = 440.5$ 'k

UNBALANCED GRAVITY LOAD MOMENT TRANSFER BY SHEAR

$0.3 M_o = 0.3(440.5') = 132.15$ 'k

$\gamma_v = 0.4$

$$V_u = \frac{V_u}{\phi A_c} + \gamma_v \frac{M_{CAB}}{\phi J_c}$$

$$= \frac{103{,}050 \text{ lb}}{0.85(637 \text{ in}^2)} + \frac{0.4(132.15)(12)(1000)(17.25)}{0.85(64847)} = 331.3 < 6\sqrt{f'_c} = 380.4 \text{ psi}$$

REINFORCEMENT REQ'D. FOR INT. PANEL

C.S.

4 RIBS w/ (2) #5 BOTTOM BARS AND (18) #5 TOP BARS

M.S.

4 RIBS w/ #4 BOTTOM BARS (LONG + SHORT) AND (7) #5 TOP BARS.

2

DEFLECTION CRITERIA

FROM TABLE 11-2

RIB + SLAB DEPTHS

8 IN RIB

3 IN SLAB

$t_e = 8.61''$

MAX SPAN = 30' > 24'

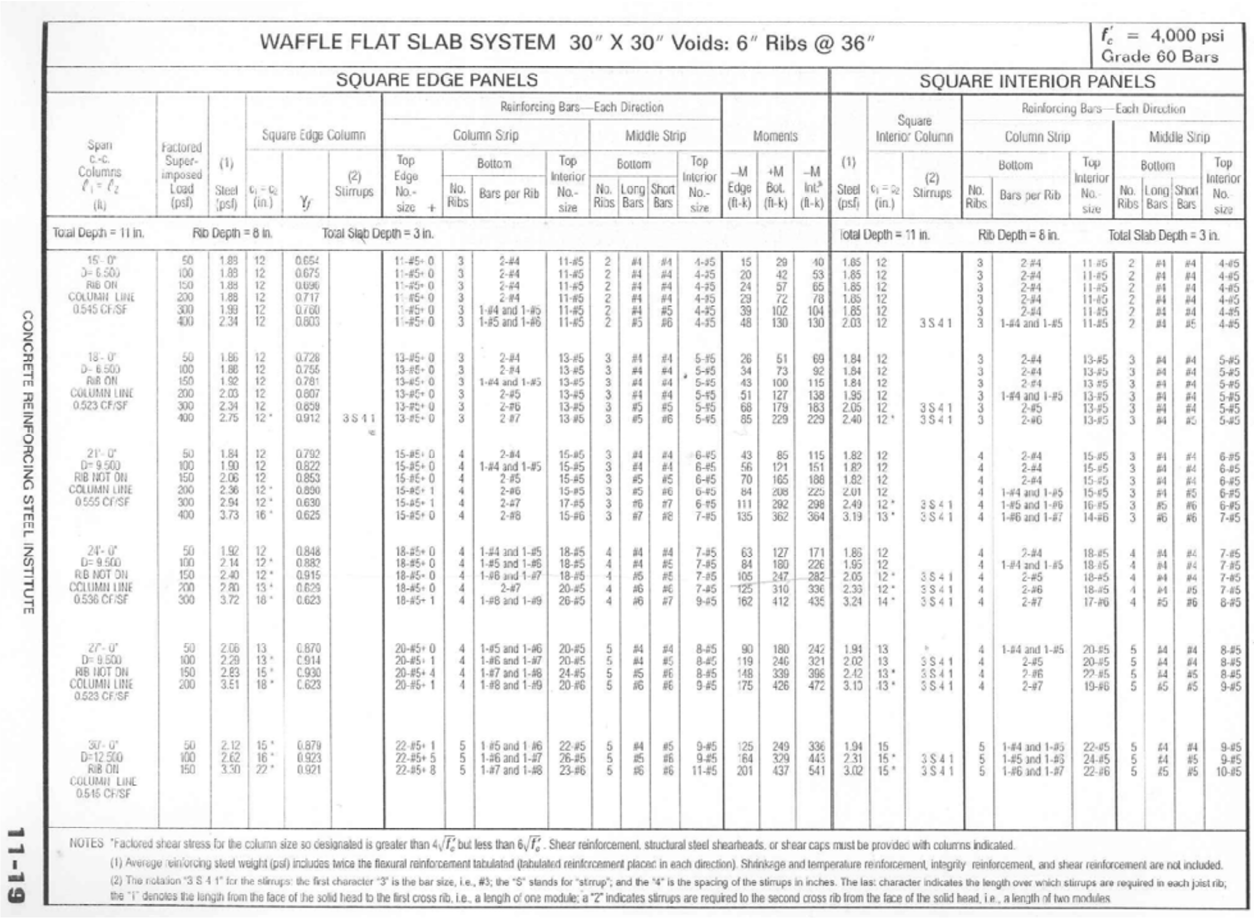
$L/360 = 24(12)/360 = 0.8''$

$L/t_e = 24(12)/8.61 = 33.45 \approx 33$

$$\Delta_{max} = \frac{L}{360} = \frac{kqL^4}{E_c t_e^3} \Rightarrow$$

$$= \frac{0.1028\, w(24)^4}{(3.5\times10^6)(8.61)^3}(144)$$

$$W = 363.9\,\text{psf} > 229.6\,\text{psf} \quad \underline{\underline{\text{OK}}}$$



CONCRETE REINFORCING STEEL INSTITUTE

11-19

WAFFLE FLAT SLAB SYSTEM 30″ X 30″ Voids: 6″ Ribs @ 36″

f'_c = 4,000 psi
Grade 60 Bars

SQUARE EDGE PANELS																	SQUARE INTERIOR PANELS									
			Square Edge Column			Reinforcing Bars—Each Direction								Moments				Square Interior Column		Reinforcing Bars—Each Direction						
						Column Strip				Middle Strip										Column Strip			Middle Strip			
						Top Edge No.-size +	Bottom		Top Interior No.-size	Bottom			Top Interior No.-size							Bottom		Top Interior No.-size	Bottom			Top Interior No. size
Span c.-c. Columns $\ell_1 = \ell_2$ (ft)	Factored Super-imposed Load (psf)	(1) Steel (psf)	$c_1 = c_2$ (in.)	γ_f	(2) Stirrups		No. Ribs	Bars per Rib		No. Ribs	Long Bars	Short Bars		–M Edge (ft-k)	+M Bot. (ft-k)	–M Int.³ (ft-k)	(1) Steel (psf)	$c_1 = c_2$ (in.)	(2) Stirrups	No. Ribs	Bars per Rib		No. Ribs	Long Bars	Short Bars	
Total Depth = 11 in.		Rib Depth = 8 in.			Total Slab Depth = 3 in.												Total Depth = 11 in.			Rib Depth = 8 in.				Total Slab Depth = 3 in.		
15'-0" D= 6.500 RIB ON COLUMN LINE 0.545 CF/SF	50	1.88	12	0.654		11-#5+ 0	3	2-#4	11-#5	2	#4	#4	4-#5	15	29	40	1.85	12		3	2-#4	11-#5	2	#4	#4	4-#5
	100	1.88	12	0.675		11-#5+ 0	3	2-#4	11-#5	2	#4	#4	4-#5	20	42	53	1.85	12		3	2-#4	11-#5	2	#4	#4	4-#5
	150	1.88	12	0.696		11-#5+ 0	3	2-#4	11-#5	2	#4	#4	4-#5	24	57	65	1.85	12		3	2-#4	11-#5	2	#4	#4	4-#5
	200	1.88	12	0.717		11-#5+ 0	3	2-#4	11-#5	2	#4	#4	4-#5	29	72	78	1.85	12		3	2-#4	11-#5	2	#4	#4	4-#5
	300	1.99	12	0.760		11-#5+ 0	3	1-#4 and 1-#5	11-#5	2	#4	#5	4-#5	39	102	104	1.85	12		3	2-#4	11-#5	2	#4	#4	4-#5
	400	2.34	12	0.803		11-#5+ 0	3	1-#5 and 1-#6	11-#5	2	#5	#6	4-#5	48	130	130	2.03	12	3 S 4 1	3	1-#4 and 1-#5	11-#5	2	#4	#5	4-#5
18'-0" D= 6.500 RIB ON COLUMN LINE 0.523 CF/SF	50	1.86	12	0.728		13-#5+ 0	3	2-#4	13-#5	3	#4	#4	5-#5	26	51	69	1.84	12		3	2-#4	13-#5	3	#4	#4	5-#5
	100	1.88	12	0.755		13-#5+ 0	3	2-#4	13-#5	3	#4	#4	5-#5	34	73	92	1.84	12		3	2-#4	13-#5	3	#4	#4	5-#5
	150	1.92	12	0.781		13-#5+ 0	3	1-#4 and 1-#5	13-#5	3	#4	#4	5-#5	43	100	115	1.84	12		3	2-#4	13-#5	3	#4	#4	5-#5
	200	2.03	12	0.807		13-#5+ 0	3	2-#5	13-#5	3	#4	#4	5-#5	51	127	138	1.95	12		3	1-#4 and 1-#5	13-#5	3	#4	#4	5-#5
	300	2.34	12	0.859		13-#5+ 0	3	2-#6	13-#5	3	#5	#5	5-#5	68	179	183	2.05	12	3 S 4 1	3	2-#5	13-#5	3	#4	#4	5-#5
	400	2.75	12 *	0.912	3 S 4 1	13-#5+ 0	3	2-#7	13-#5	3	#5	#6	5-#5	85	229	229	2.40	12 *	3 S 4 1	3	2-#6	13-#5	3	#4	#5	5-#5
21'-0" D= 9.500 RIB NOT ON COLUMN LINE 0.555 CF/SF	50	1.84	12	0.792		15-#5+ 0	4	2-#4	15-#5	3	#4	#4	6-#5	43	85	115	1.82	12		4	2-#4	15-#5	3	#4	#4	6-#5
	100	1.90	12	0.822		15-#5+ 0	4	1-#4 and 1-#5	15-#5	3	#4	#4	6-#5	56	121	151	1.82	12		4	2-#4	15-#5	3	#4	#4	6-#5
	150	2.06	12	0.853		15-#5+ 0	4	2-#5	15-#5	3	#5	#5	6-#5	70	165	188	1.82	12		4	2-#4	15-#5	3	#4	#4	6-#5
	200	2.36	12 *	0.890		15-#5+ 1	4	2-#6	15-#5	3	#5	#6	6-#5	84	208	225	2.01	12		4	1-#4 and 1-#5	15-#5	3	#4	#5	6-#5
	300	2.94	12 *	0.630		15-#5+ 1	4	2-#7	17-#5	3	#6	#7	6-#5	111	292	298	2.49	12 *	3 S 4 1	4	1-#5 and 1-#6	16-#5	3	#5	#6	6-#5
	400	3.73	16 *	0.625		15-#5+ 0	4	2-#8	15-#6	3	#7	#8	7-#5	135	362	364	3.19	13 *	3 S 4 1	4	1-#6 and 1-#7	14-#6	3	#6	#6	7-#5
24'-0" D= 9.500 RIB NOT ON COLUMN LINE 0.536 CF/SF	50	1.92	12	0.848		18-#5+ 0	4	1-#4 and 1-#5	18-#5	4	#4	#4	7-#5	63	127	171	1.86	12		4	2-#4	18-#5	4	#4	#4	7-#5
	100	2.14	12 *	0.882		18-#5+ 0	4	1-#5 and 1-#6	18-#5	4	#4	#5	7-#5	84	180	226	1.95	12		4	1-#4 and 1-#5	18-#5	4	#4	#4	7-#5
	150	2.40	12 *	0.915		18-#5+ 0	4	1-#6 and 1-#7	18-#5	4	#5	#5	7-#5	105	247	282	2.05	12 *	3 S 4 1	4	2-#5	18-#5	4	#4	#4	7-#5
	200	2.80	13 *	0.629		18-#5+ 0	4	2-#7	20-#5	4	#6	#6	7-#5	125	310	336	2.33	12 *	3 S 4 1	4	2-#6	18-#5	4	#4	#5	7-#5
	300	3.72	18 *	0.623		18-#5+ 1	4	1-#8 and 1-#9	26-#5	4	#6	#7	9-#5	162	412	435	3.24	14 *	3 S 4 1	4	2-#7	17-#6	4	#5	#6	8-#5
27'-0" D= 9.500 RIB NOT ON COLUMN LINE 0.523 CF/SF	50	2.06	13	0.870		20-#5+ 0	4	1-#5 and 1-#6	20-#5	5	#4	#4	8-#5	90	180	242	1.94	13		4	1-#4 and 1-#5	20-#5	5	#4	#4	8-#5
	100	2.29	13 *	0.914		20-#5+ 1	4	1-#6 and 1-#7	20-#5	5	#4	#5	8-#5	119	246	321	2.02	13	3 S 4 1	4	2-#5	20-#5	5	#4	#4	8-#5
	150	2.83	15 *	0.930		20-#5+ 4	4	1-#7 and 1-#8	24-#5	5	#5	#6	8-#5	148	339	396	2.42	13 *	3 S 4 1	4	2-#6	22-#5	5	#4	#5	8-#5
	200	3.51	18 *	0.623		20-#5+ 1	4	1-#8 and 1-#9	20-#6	5	#6	#6	9-#5	175	426	472	3.10	13 *	3 S 4 1	4	2-#7	19-#6	5	#5	#5	9-#5
30'-0" D=12.500 RIB ON COLUMN LINE 0.545 CF/SF	50	2.12	15 *	0.879		22-#5+ 1	5	1-#5 and 1-#6	22-#5	5	#4	#5	9-#5	125	249	336	1.94	15		5	1-#4 and 1-#5	22-#5	5	#4	#4	9-#5
	100	2.62	16 *	0.923		22-#5+ 5	5	1-#6 and 1-#7	26-#5	5	#5	#6	9-#5	164	329	443	2.31	15 *	3 S 4 1	5	1-#5 and 1-#6	24-#5	5	#4	#5	9-#5
	150	3.30	22 *	0.921		22-#5+ 8	5	1-#7 and 1-#8	23-#6	5	#6	#6	11-#5	201	437	541	3.02	15 *	3 S 4 1	5	1-#6 and 1-#7	22-#6	5	#5	#5	10-#5

NOTES *Factored shear stress for the column size so designated is greater than $4\sqrt{f'_c}$ but less than $6\sqrt{f'_c}$. Shear reinforcement, structural steel sheatheads, or shear caps must be provided with columns indicated.

(1) Average reinforcing steel weight (psf) includes twice the flexural reinforcement tabulated (tabulated reinforcement placed in each direction). Shrinkage and temperature reinforcement, integrity reinforcement, and shear reinforcement are not included.

(2) The notation "3 S 4 1" for the stirrups: the first character "3" is the bar size, i.e., #3; the "S" stands for "stirrup"; and the "4" is the spacing of the stirrups in inches. The last character indicates the length over which stirrups are required in each joist rib; the "1" denotes the length from the face of the solid head to the first cross rib, i.e., a length of one module; a "2" indicates stirrups are required to the second cross rib from the face of the solid head, i.e., a length of two modules

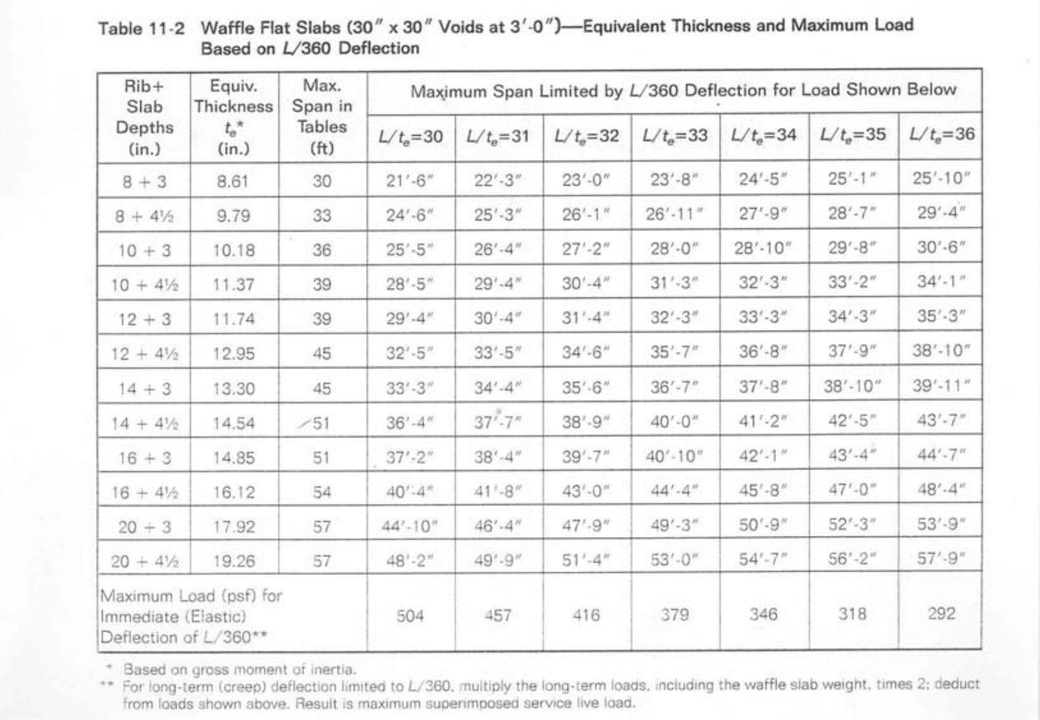


Table 11-2 Waffle Flat Slabs (30″ x 30″ Voids at 3′-0″)—Equivalent Thickness and Maximum Load Based on L/360 Deflection

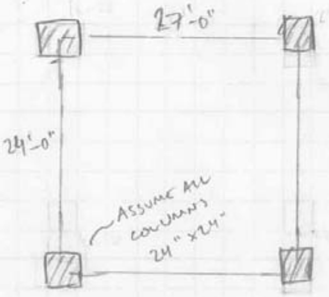
Rib+ Slab Depths (in.)	Equiv. Thickness t_e* (in.)	Max. Span in Tables (ft)	Maximum Span Limited by L/360 Deflection for Load Shown Below						
			L/t_e=30	L/t_e=31	L/t_e=32	L/t_e=33	L/t_e=34	L/t_e=35	L/t_e=36
8 + 3	8.61	30	21′-6″	22′-3″	23′-0″	23′-8″	24′-5″	25′-1″	25′-10″
8 + 4½	9.79	33	24′-6″	25′-3″	26′-1″	26′-11″	27′-9″	28′-7″	29′-4″
10 + 3	10.18	36	25′-5″	26′-4″	27′-2″	28′-0″	28′-10″	29′-8″	30′-6″
10 + 4½	11.37	39	28′-5″	29′-4″	30′-4″	31′-3″	32′-3″	33′-2″	34′-1″
12 + 3	11.74	39	29′-4″	30′-4″	31′-4″	32′-3″	33′-3″	34′-3″	35′-3″
12 + 4½	12.95	45	32′-5″	33′-5″	34′-6″	35′-7″	36′-8″	37′-9″	38′-10″
14 + 3	13.30	45	33′-3″	34′-4″	35′-6″	36′-7″	37′-8″	38′-10″	39′-11″
14 + 4½	14.54	51	36′-4″	37′-7″	38′-9″	40′-0″	41′-2″	42′-5″	43′-7″
16 + 3	14.85	51	37′-2″	38′-4″	39′-7″	40′-10″	42′-1″	43′-4″	44′-7″
16 + 4½	16.12	54	40′-4″	41′-8″	43′-0″	44′-4″	45′-8″	47′-0″	48′-4″
20 + 3	17.92	57	44′-10″	46′-4″	47′-9″	49′-3″	50′-9″	52′-3″	53′-9″
20 + 4½	19.26	57	48′-2″	49′-9″	51′-4″	53′-0″	54′-7″	56′-2″	57′-9″
Maximum Load (psf) for Immediate (Elastic) Deflection of L/360**			504	457	416	379	346	318	292

* Based on gross moment of inertia.

** For long-term (creep) deflection limited to L/360, multiply the long-term loads, including the waffle slab weight, times 2; deduct from loads shown above. Result is maximum superimposed service live load.

APPENDIX E: POST-TENSIONED FLAT PLATE

POST TENSIONED



$f'_c = 5000$ psi

LOADS

SLAB (7½") : 93.75
PARTITIONS : 20 psf
MEP : 10 psf
FINISH + MISC. 5 psf
LIVE : 40 psf

$$\frac{l_n}{42} = \frac{[27' - (\frac{12}{12})](12)}{42} = 7.14 \approx 7.5\text{" SLAB}$$

$$W_{pre} = 0.9(93.75) = 84.4 \text{ psf}$$

FOR 2HR SLAB w/ 1" COVER ON TOP + BOT.
USE 1¼"

LONG SPAN (27'-0")

$W_{pre} = 84.4$ psf

$a = 7.5" - 2(1.25) = 5"$

$$M_{pre} = \frac{(.0844)(27'-0")^2}{8} = 7.69^{'k}$$

$$F = \frac{M_{pre}}{a} = \frac{7.69^{'k}(12)}{5} = 18.46^k$$

$$\frac{F}{A} = \frac{18.46^k(1000)}{(7.5")(12)} = 205.1 \text{ psi}$$

SHORT SPAN (24'-0")

$W_{pre} = 84.4$ psf

$a = 5"$

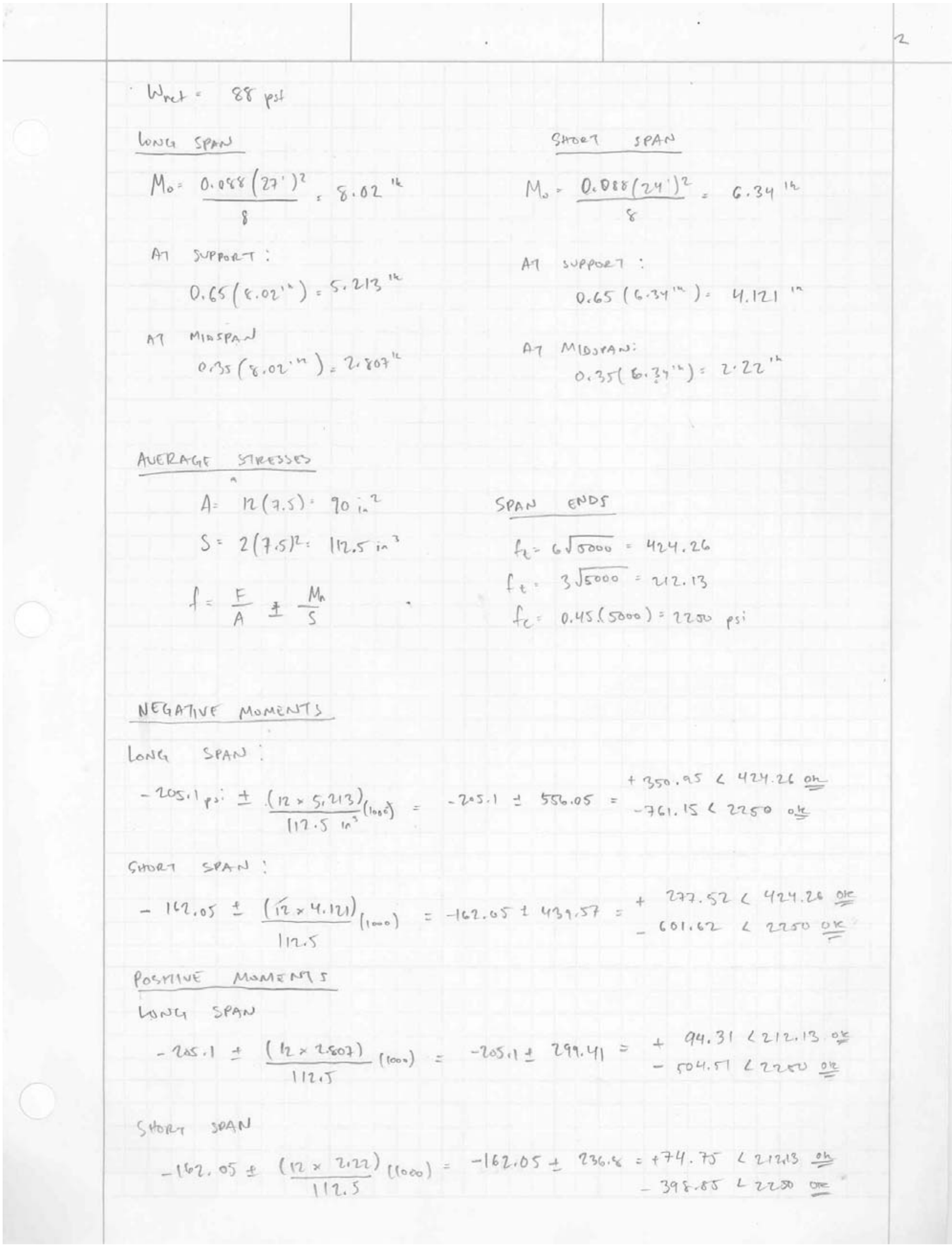
$$M_{pre} = \frac{(.0844)(24'-0")^2}{8} = 6.08^{'k}$$

$$F = \frac{(6.08^{'k})(12)}{5} = 14.58^k$$

$$\frac{F}{A} = \frac{14.58^k(1000)}{(7.5")(12)} = 162.05 \text{ psi}$$

BUNDLES AT STRIPS

$27'\ (14.6\ ^k/_{ft}) = \underline{394.2^k}$



2

$W_{net} = 88$ psf

LONG SPAN

$$M_o = \frac{0.088(27')^2}{8} = 8.02\ 'k$$

AT SUPPORT:

$0.65(8.02\ 'k) = 5.213\ 'k$

AT MIDSPAN

$0.35(8.02\ 'k) = 2.807\ 'k$

SHORT SPAN

$$M_o = \frac{0.088(24')^2}{8} = 6.34\ 'k$$

AT SUPPORT:

$0.65(6.34\ 'k) = 4.121\ 'k$

AT MIDSPAN:

$0.35(6.34\ 'k) = 2.22\ 'k$

AVERAGE STRESSES

$A = 12(7.5) = 90\text{ in}^2$

$S = 2(7.5)^2 = 112.5\text{ in}^3$

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

SPAN ENDS

$f_t = 6\sqrt{5000} = 424.26$

$f_t = 3\sqrt{5000} = 212.13$

$f_c = 0.45(5000) = 2250$ psi

NEGATIVE MOMENTS

LONG SPAN:

$$-205.1\text{ psi} \pm \frac{(12 \times 5.213)}{112.5\text{ in}^3}(1000) = -205.1 \pm 556.05 = \begin{matrix} +350.95 < 424.26 \text{ ok} \\ -761.15 < 2250 \text{ ok} \end{matrix}$$

SHORT SPAN:

$$-162.05 \pm \frac{(12 \times 4.121)}{112.5}(1000) = -162.05 \pm 439.57 = \begin{matrix} +277.52 < 424.26 \text{ OK} \\ -601.62 < 2250 \text{ OK} \end{matrix}$$

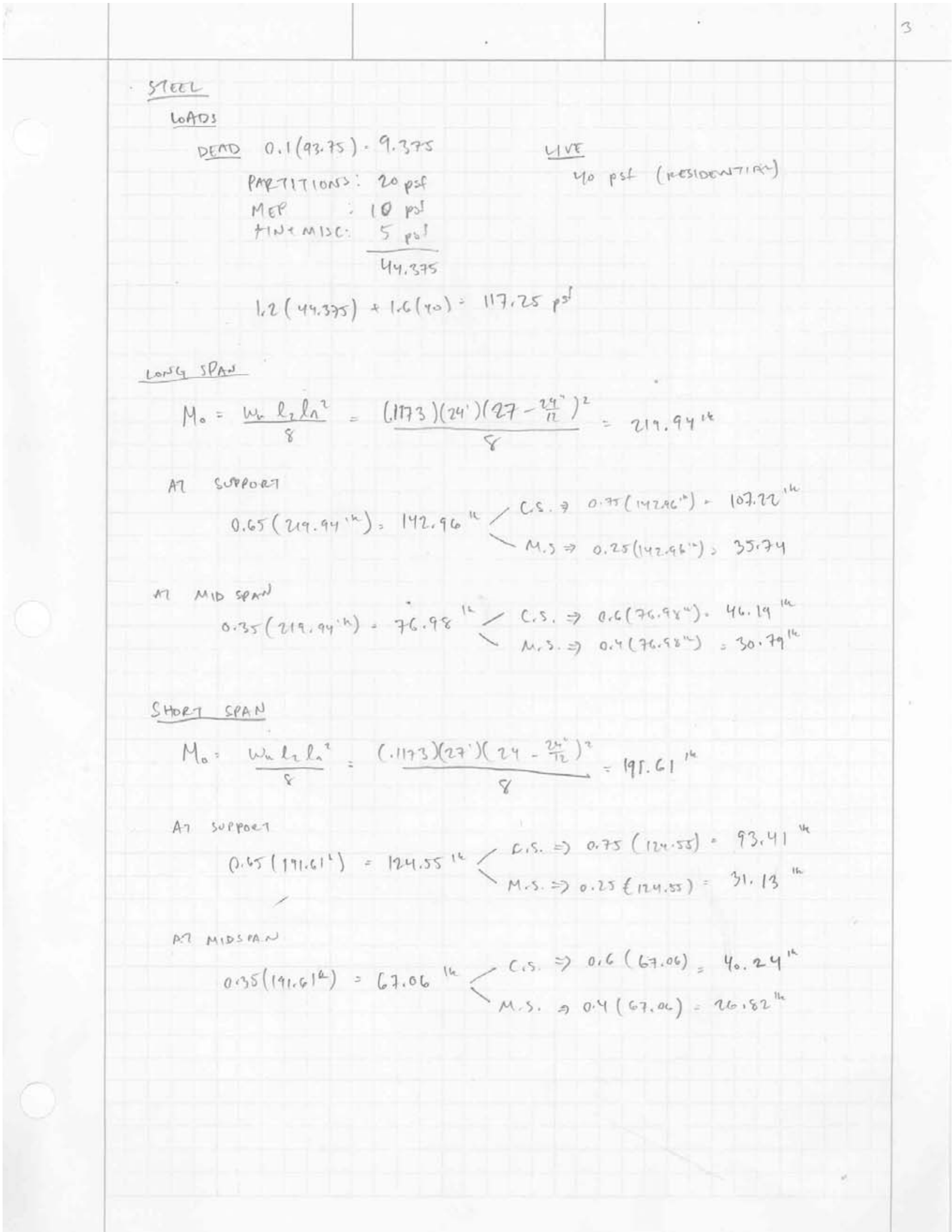
POSITIVE MOMENTS

LONG SPAN

$$-205.1 \pm \frac{(12 \times 2.807)}{112.5}(1000) = -205.1 \pm 299.41 = \begin{matrix} +94.31 < 212.13 \text{ ok} \\ -504.51 < 2250 \text{ ok} \end{matrix}$$

SHORT SPAN

$$-162.05 \pm \frac{(12 \times 2.22)}{112.5}(1000) = -162.05 \pm 236.8 = \begin{matrix} +74.75 < 212.13 \text{ ok} \\ -398.85 < 2250 \text{ OK} \end{matrix}$$



3

STEEL

LOADS

DEAD $0.1(93.75) = 9.375$

PARTITIONS : 20 psf
MEP : 10 psf
FIN + MISC : 5 psf
44.375

LIVE
40 psf (RESIDENTIAL)

$1.2(44.375) + 1.6(40) = 117.25$ psf

LONG SPAN

$$M_o = \frac{w_u l_2 l_n^2}{8} = \frac{(.1173)(24')(27 - \frac{24''}{12})^2}{8} = 219.94^{\prime k}$$

AT SUPPORT

$0.65(219.94^{\prime k}) = 142.96^{\prime k}$ C.S. $\Rightarrow 0.75(142.96^{\prime k}) = 107.22^{\prime k}$
M.S $\Rightarrow 0.25(142.96^{\prime k}) = 35.74$

AT MID SPAN

$0.35(219.94^{\prime k}) = 76.98^{\prime k}$ C.S. $\Rightarrow 0.6(76.98^{\prime k}) = 46.19^{\prime k}$
M.S. $\Rightarrow 0.4(76.98^{\prime k}) = 30.79^{\prime k}$

SHORT SPAN

$$M_o = \frac{w_u l_2 l_n^2}{8} = \frac{(.1173)(27')(24 - \frac{24''}{12})^2}{8} = 191.61^{\prime k}$$

AT SUPPORT

$0.65(191.61^{\prime k}) = 124.55^{\prime k}$ C.S. $\Rightarrow 0.75(124.55) = 93.41^{\prime k}$
M.S. $\Rightarrow 0.25(124.55) = 31.13^{\prime k}$

AT MIDSPAN

$0.35(191.61^{\prime k}) = 67.06^{\prime k}$ C.S. $\Rightarrow 0.6(67.06) = 40.24^{\prime k}$
M.S. $\Rightarrow 0.4(67.06) = 26.82^{\prime k}$

	Long Span				**Short Span**			
	Column Strip		Middle Strip		Column Strip		Middle Strip	
	M+	M-	M+	M-	M+	M-	M+	M-
Moment	46.19	107.22	30.79	35.74	40.24	93.41	26.82	31.13
Width	144.00	144.00	144.00	144.00	162.00	162.00	162.00	162.00
d	6.25	6.25	6.25	6.25	6.25	6.25	6.25	6.25
Mu	51.32	119.13	34.21	39.71	44.71	103.79	29.80	34.59
R	109.49	254.15	72.98	84.72	84.79	196.81	56.51	65.59
rho	0.0020	0.0045	0.0012	0.0015	0.0015	0.0035	0.0009	0.0011
As	1.80	**4.05**	1.08	1.35	1.52	**3.54**	0.91	1.11
As,min	**1.94**	1.94	**1.94**	**1.94**	**2.19**	2.19	**2.19**	**2.19**
N	9	**20**	5	7	8	**18**	5	6
N,min	**10**	10	**10**	**10**	**11**	11	**11**	**11**
Spacing	12"	6"	12"	12"	12"	6"	12"	12"

REFERENCES

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