THE PENNSYLVANIA STATE UNIVERSITY

Crocker West Building

State College, PA

Senior Thesis Project Tech II:
Pro-Con Structural Study of Alternate Floor Systems

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-- EXECUTIVE SUMMARY --

Structural Tech Report II is geared towards a comparison report between the existing floor system of the Crocker West Building and three selected alternative floor systems. All floor systems will be assessed using, but not limited to, criteria such as: cost, serviceability, fire-rating and deflection in order to better understand the existing floor elements and establish realistic alternative floor system designs.

Tech II consists of a preliminary study for a 2nd Floor Level, 35' x 35' typical interior bay floor system. The interior bay was chosen based on the original framing plan of the structure containing no columns along the exterior perimeter, and the 2nd Floor Level was selected for design purposes due to the relatively high loading conditions. The existing floor system is designed using prestressed, precast hollow-core slabs with a typical two inch topping supported by inverted-tee (IT) beams that bear on column corbels, see Figure #4 in Structural Systems section below. Although specific height restrictions for floor levels were not required during preliminary design, the existing system still utilized minimum floor depths in order to achieve maximum floor-to-finished ceiling height. Due to this, it may be valuable to investigate alternative floor framing systems. Tech II will examine the following types of alternative solutions:

i. Two-Way Flat Slab Systems

ii. Post-Tensioned Two-Way Slab System

iii. Composite Beams with Metal Decking Systems(s)

Concluding the results found in preliminary analyses of the alternative floor systems, it appears as though the composite beams with metal deck and post-tensioned (PT) slab would be the most practical selections for advanced research. A composite floor system allows for quick erection time and limits design criteria such as deflection and vibration over the long span. The PT slab will be further researched simply due to its ability to allow for shallower floor cavities with respect to longer spans. The thinner floor diaphragms will reduce the total weight of the building and lower the overall height of the structure. Based on the geotechnical report for this project, the weight of the structure is of little interest due to the soil capacity of the site and the amount of rock beneath. However, using the PT slab to aide the height of the structure could be of great importance due to a 45' height restriction implemented by Ferguson Township in State College, Pa. An in-depth design analysis of each floor system would be necessary to fully compare the impacts each design would have on other systems such as the foundation and lateral systems.

**Please note: Beam designs for the existing structure are not included within this report. Please reference Tech Report I (Appendix B) for Concise Summary Reports citing the IT-beam designs. Also, available upon request.

-- BUILDING INTRODUCTION --

Crocker West will be used as a highly classified research facility, specializing in the development and testing of underwater weapons for the U.S. Department of Defense. Located in State College, Pa, the structure will be a 3-story low-rise building with areas classified as office, light industrial, and warehouse totaling nearly 120,000 square feet. The first floor of the CWB will consist mainly of 'closed' lab area, along with technician offices, locker rooms and special test areas. The second floor will include office space, another lab area, computer lab, student room and a room designated to SCIF (Sensitive Compartmented Information Facility), while the third floor will be devoted mostly to office space. The entire building will be constructed of precast systems, including: columns, beams, walls, floor & roof diaphragms. Lateral loads applied to the structure will be collectively distributed throughout the building to specially designed shear walls.

Please note that Appendix A at the end of this report contains drawings of the project for reference, while Appendix B consists of hand calculations and other data used in designing and comparing the alternative floor systems for the Crocker West Building. The following page consists of a plan drawing of a typical bay (designated by hatch) analyzed throughout this report.

-- DESIGN CRITERIA --

DEFLECTION:	Limit
Live Load Deflection	L / 360
Total Deflection	L / 240

**Please note that vibration criteria were not considered in the analysis and design of the alternative (or existing) floor systems. This is partially due to the amount of employees that will be employed in the building, approximately 180, thus high levels of vibration assumed to not occur. Also, the occupant did not address any concern about this issue.

-- STRUCTURAL SYSTEM --

As stated above, CWB is a total precast building. The following are detailed explanations of the individual precast members and systems.

FOUNDATION(S):

The foundation system(s) being implemented consists of typical cast-in-place (CIP) strip and pad footings, as well as a standard CIP slab-on-grade. Fifteen inch deep strip footings ranging from 3'-3" to 6'-6" wide are used along the perimeter of the structure. These footings help distribute wall panel loads to the ground. Additionally, the East walls strip footing of the structure will also be used as a part of the underground water cistern that will be used to collect treatable storm water runoff for reuse. Spread (or Pad) footings will be used throughout the interior portion of the building and will be used to pick up loads from columns and stair-towers. Pads used under columns vary in size from 12' square to 14'-5 square, while pads under the four typical stair-towers are 12'-0 x 25'-6. All pad footings are 2 foot thick unless noted otherwise. A six inch thick slab-on-grade reinforced with W4.0 x W4.0 WWF will complete the foundation system(s) and will be used as the ground floor level of the building. See Figures #1 and #2 below for a plan view of the foundation systems and proposed cistern detail, respectively. Please note, the width of the cistern was unavailable at this time.

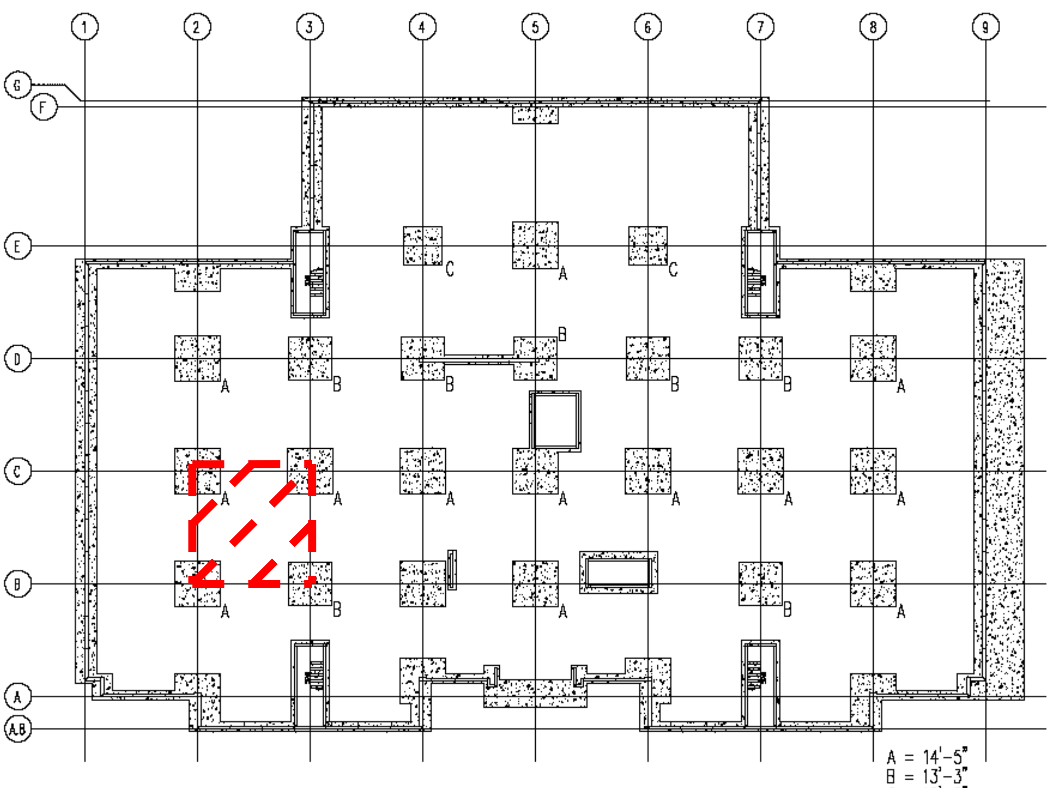


FIGURE #1 – FOUNDATION SYSTEMS

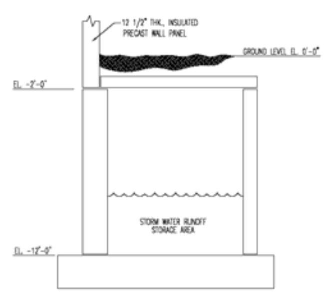


FIGURE #2 – PROPOSED CISTERN SECTION

COLUMNS:

The vertical supporting members for the entire structure are reinforced, precast concrete columns. All columns are 24" x 24" square columns with four (4) #11 longitudinal reinforcing bars and #4 stirrups spaced accordingly (See Figure #3). Columns will be cast for lengths up to 42 feet. Each column will contain haunches and haunch reinforcing (Figure #4) cast monolithically at each floor level, and in the required position for beam bearing and load transfer. The columns are spaced on a 35'-0 x 35'-0 typical bay grid and are connected to the pad footings with four (4) 1 ¼" dia. ASTM A193 threaded rods. See Figure #5 for column grid layout.

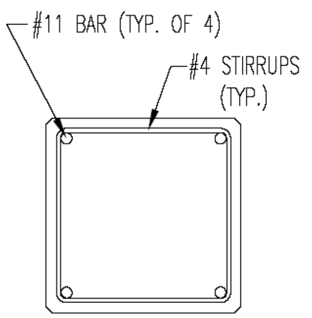


FIGURE #3 – TYP. COLUMN SECTION

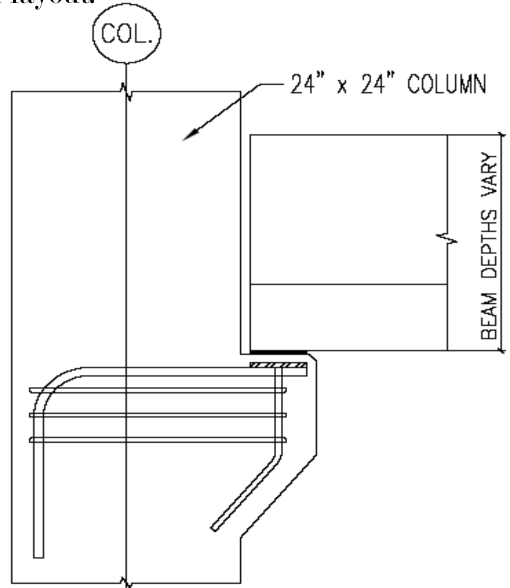


FIGURE #4 – COLUMN w/ HAUNCH

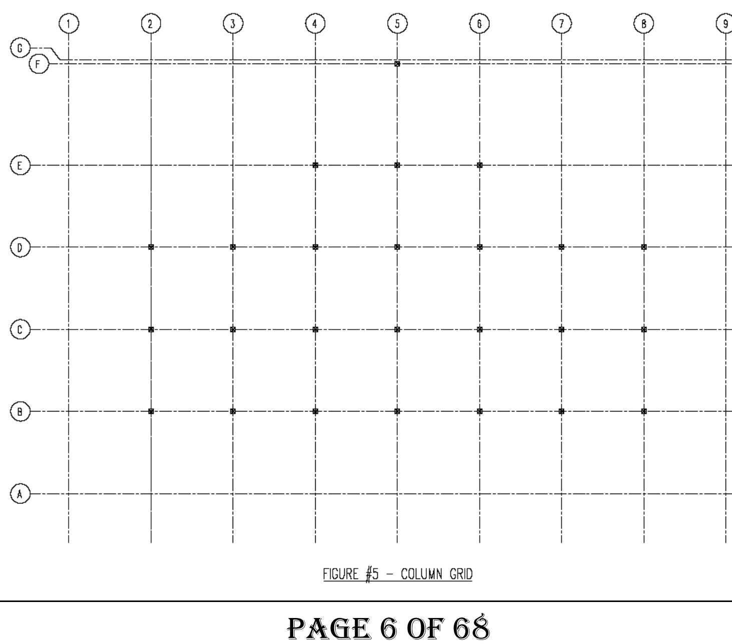


FIGURE #5 – COLUMN GRID

FLOOR SYSTEM:

As previously stated, the 1st Floor (or Ground Level) floor system is a 6" thick slab-on-grade with W4.0 x W4.0 WWF reinforcing. The remaining floor levels are constructed of precast, prestressed hollow-core flat slabs. The 2nd Floor Level will consist of 12 inch and the 3rd Floor Level will be comprised of 10 inch hollow-core flat slabs, each with six (6) 7-wire, ½" dia. 270 ksi low-relaxation prestressing strands and a typical 2" topping. Some of the hollow-core floor system clear spans are nearly 33'-0, with individual panels running in an East-West direction. See drawings in Appendix A for hollow-core panel layout.

Furthermore, these hollow-core slabs are supported by one of two methods. If the floor slab is to bear at an exterior wall panel location, a specially designed bearing ledge will be cast into the precast wall panel with proper reinforcing. For interior bay supports, the hollow-core slabs will be supported by precast, prestressed concrete inverted-tee (IT) beams. IT beams for the 2nd Floor were designed to be 28" deep, while 3rd Floor beams are 20" deep due to dissimilar live loads. See Appendix A for typical IT Beam sections.

ROOF SYSTEM:

The roofing system for the Crocker West Building main roof will be constructed by means of similar materials used in erecting floors two and three. The main roof will consist of 8" hollow-core flat slabs with (7) 7-wire, ½" dia. 270 ksi low-relaxation strands supported by 18" deep inverted-tee beams. The low roof, located in the rear storage area of the building, will be constructed of 10'-9 wide x 24" deep precast concrete double-tees (See Figure #6). In addition, each roof will receive a layer of 4" tapered rigid insulation and a 60 mil EPDM roofing membrane rather than a 2" topping which is not needed on the roof.

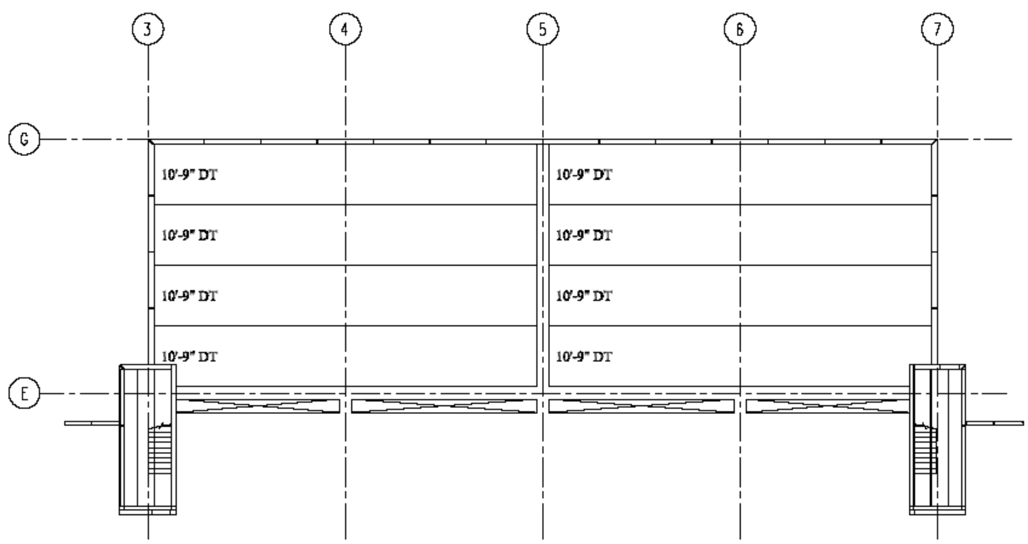


FIGURE #6 – LOW ROOF DT LAYOUT

LATERAL SYSTEM:

One of the key design issues of a total precast structure is the make up of the lateral force resistance system. Crocker West is no different; its lateral system was designed using a compilation of precast shear walls positioned around the perimeter and throughout the building. These precast shear walls are constructed with several different thicknesses of insulated and non-insulated precast panels. Exterior wall panels (all insulated) acting as shear walls in the N-S direction are 12 ½" thick, while E-W direction walls are 9 ½" thick. Shear walls located on the interior of the structure and around stair-towers are 9" thick and non-insulated. Due to the fact that every panel is individually erected, specially designed connections are required for each piece. These connections, not specified in this tech report, are designed to ensure the applied load is safely distributed to the lateral system. Figure #7 below illustrates the layout of the shear walls; each represented by a solid line with a SW designation. Also, typical Wall Sections may be found in Appendix A.

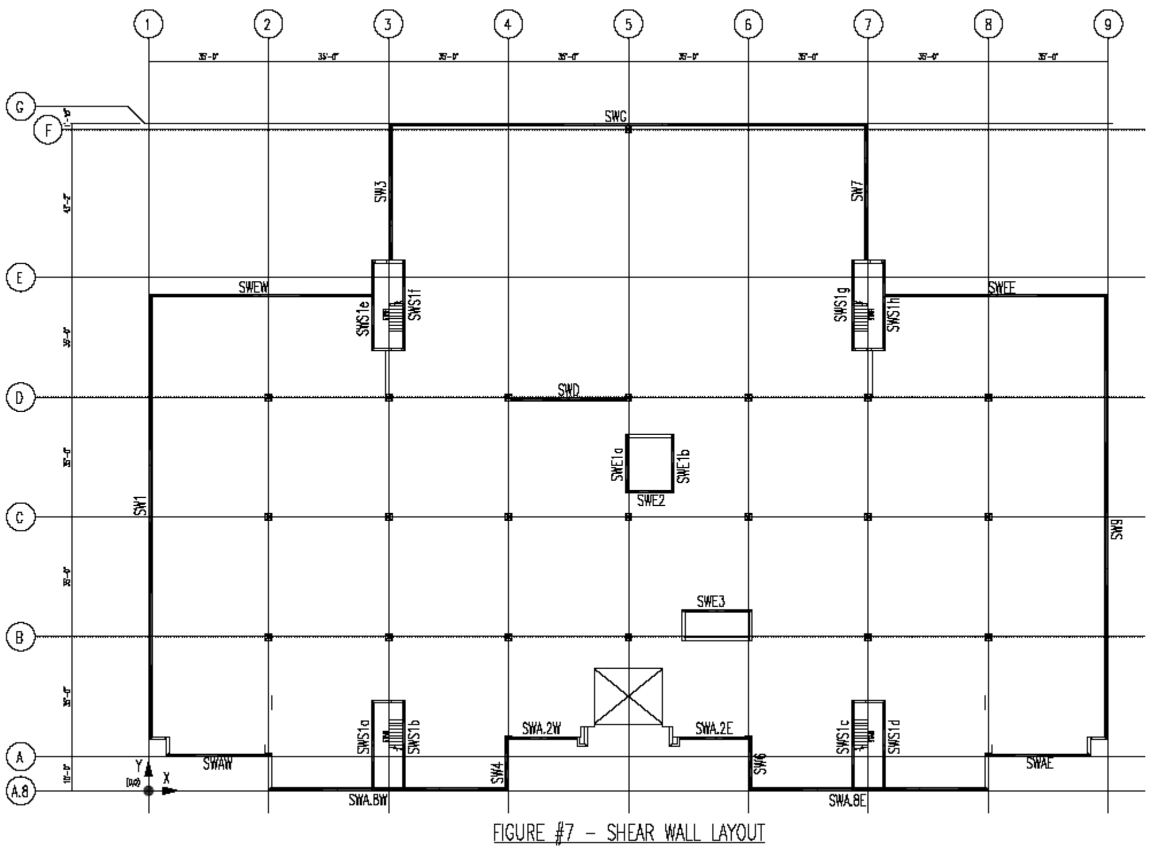


FIGURE #7 – SHEAR WALL LAYOUT

-- STRENGTH OF MATERIALS --

EXISTING:

CAST-IN-PLACE CONCRETE:	f'_c
Slab-on-Grade	4000 psi

PRECAST CONCRETE:	f'_c	f'_{ci}
Columns	6000 psi	3500 psi
Beams	6000 psi	for
Hollow-Core Slabs	6000 psi	ALL
Wall Panels	6000 psi	

REINFORCING STEEL:	f_y
Reinforcing Bars	60000 psi
Stirrups	60000 psi
WWF	60000 psi

PRESTRESSING STRANDS:	f_{ps}	E_s
½" Special (7-Wire) strands	270 ksi	28000 psi

ALTERNATIVE(S):

**Please see individual floor design calculations included in Appendix B of this report.

-- MODEL CODES --

The following codes listed were used in the original design, as well as any and all analysis performed for this tech report.

BUILDING CODES:	
International Building Code (IBC)	IBC 2006
CONCRETE CODES:	
American Concrete Institute (ACI) - Building Code Requirements for Structural Concrete	ACI 318-05
Precast/Prestressed Concrete Institute (PCI) - PCI Design Handbook, Precast and Prestressed Concrete	6th Edition
LATERAL LOADS & DESIGN LOADS:	
American Society of Civil Engineers (ASCE) - Minimum Design Loads for Buildings and Other Structures	ASCE 7-05
IBC	IBC 2006
STEEL DESIGN:	
American Institute of Steel Construction Manual (AISC)	13th Edition

DESIGN LOADS:

LIVE LOADS

	DESIGN	ASCE 7-05
Lobby / 1st Floor Corridors	*a	100 psf
Corridors above 1st Floor	80-125 psf *b	80 psf
Offices	80-125 psf *b	50 psf
Ordinary Flat Roof	20 psf	20 psf
Stairs / Exits	175 psf	100 psf
Snow (p_f = 0.7*40psf = 28 psf)	40 psf	40 psf *c

*Notes:

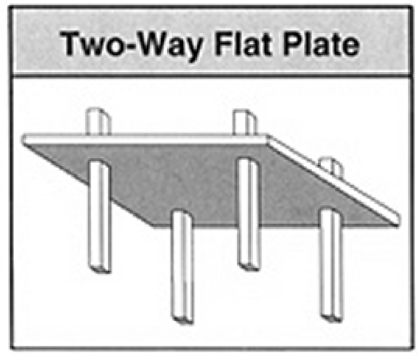
a. Lobby and 1st Floor located at ground level which exceeds 100 psf.
b. Design live loads differ from floor to floor.
   2nd Floor = 125 psf   3rd Floor = 80 psf
c. 40 psf Snow Load specified by Centre Region Code (See Appendix B)

DEAD LOADS

Dead load for structure includes self weight of individual precast members. See seismic analysis in Appendix B for detailed loads.

-- ALTERNATIVE FLOOR SYSTEMS --

i. TWO-WAY FLAT PLATE:



Material Properties:

f'c = 5000 psi (NWC)

fy = 60000 psi

Loading:

Live Load = 125 psf (2nd Floor)

Dead Load = self wt. + 25 psf (superimposed)

Synopsis:

A two-way flat plate system is composed of a uniformly thick, concrete slab that is reinforced in both directions using conventional reinforcement. Flat plates are considerably economical in terms of reinforcement and formwork due to the simplicity of the system. Also, a flat plate system optimizes the depth of the floor plenum, ultimately resulting in minimum story heights.

Hand calculations were performed in the design of the flat plate system. A minimum slab thickness of one foot (t=12") was determined and found to be sufficient for strength and shear. Maintaining the original square column sizes of 24" x 24" it was determined that the column strip reinforcing required (29) #8's in the top and (12) #8's on the bottom, while the middle strip reinforcing required (10) #8's in the top and (8) #8's on the bottom. This reinforcing is required for each strip in each direction. See Flat Plate System hand calculations in Appendix B of this report for a schematic diagram of the reinforcing details (pg. 46).

PCA-Slab, a computer-based design program, was used to compare the design and estimate deflections. Appendix B contains output diagrams for two separate systems. The 1st & 2nd diagrams, on pages 48-49, depict estimated deflection and required reinforcing, respectively, of a two-way flat plate system equal to that designed by hand (max Δ = 1.3"). The 3rd diagram, on page 50, represents the deflection of a similar two-way system with beams (max Δ = 0.8"). Input for both PCA examples is available upon request.

Considerations:

<u>Structural</u> – The flat plate system allows for relatively long spans and a substantial lack of restrictions around columns and walls. Also, because the minimum thickness of the slab is 12", a fire-rating of two hours is over-achieved and no additional fire-proofing is required. However, due to the extensive live load assigned to this level of the structure and a clear span of nearly 35 feet, further analysis may inhibit the use of column drop panels and/or a thicker slab in order to reduce deflection in the diaphragm.

Architectural – The two-way system provides minimal floor plenum thickness and in turn can be used to reduce the overall height of the structure, or provide maximum floor-to-ceiling heights. Either face may be finished to combat a variety of floor and ceiling materials. In addition to the above, a flat plate system allows for a spacious column grid and thus a vast amount of open floor space.

Construction/M.E.P. – Crocker West utilizes a typical 35' x 35' bay throughout the entire structure, this will prove cost effective for things like formwork and reinforcing. However, a detailed analysis would have to be performed to extensively compare the costs associated with the large amount of reinforcing required to that of the repeating forms. The flat plate system also provides an abundance of space for the M.E.P. trades, allowing the individual systems to be smoothly coordinated and flexible. Conversely, a flat plate is constructed with cast-in-place (CIP) concrete which means longer construction schedule due to proper curing of the concrete.

PRO-CON TABLE

Pro	Con
2-hr fire-rating (w/o add'l. fire-proofing)	heavily reinf. sections add weight & cost
long spans with thinner slab thickness	slowed erection time
open plan layout	deflections due to long span
	temp. & shrinkage issues while curing

Conclusion – Although the two-way flat plate system provides the opportunity for a thinner floor plenum, I feel the additional weight of the structure and large deflections rule out this particular design for further investigation. The PCA example with beams may be a viable alternative, however the example was not as detailed and thus will not be pursued.

ii. POST-TENSIONED TWO-WAY SLAB:

Material Properties:

f'c = 5000 psi (NWC)

fy = 60000 psi (Rebar)

0.6"Ø, 7-wire strands (f_{pu} = 270 ksi)

Loading:

Live Load = 125 psf (2nd Floor)

Dead Load = self wt. + 25 psf (superimposed)
+ 13 psf (add'l. 1" concrete)

Synopsis:

A post-tensioned (PT) floor system is constructed using the same methodology and materials as a flat plate system. A PT slab is also composed of a uniformly thick, concrete slab; however the reinforcing differs to that of a flat plate. A PT slab employs long strands of post-tensioning tendons spaced throughout the slab. After the slab is placed and allowed to cure to a required strength, the tendons are then pulled to a desired tensile limit. Once the concrete is fully cured, the tendons are cut, or released, and they induce a substantial compressive force into the slab. When loaded, these compression forces will work against the applied tensile forces distributed throughout the structure.

Based upon the preliminary analysis, it can be concluded that the PT slab system would be considered a practical solution to examine as an alternative. Due to limited experience designing PT slab systems, hand calculations halter in Appendix B after determining some of the service stresses are greater than the allowable. Assuming a span length to slab thickness ratio of 45 (L/h = 45), required a minimum slab thickness of 10". However, as previously stated, the 10" slab proved to be unsafe. The level of error associated with these stresses is sporadic. Some barely beyond the allowable, and others nearly doubled. A variety of construction options are available to avoid this setback. A thicker slab may be re-analyzed to try and increase the amount of stresses it may contain. Also, I feel that the addition of column capitals or drop panels to this system would greatly improve its performance.

As stated, Appendix B of this report contains hand calculations used for the PT slab design. Due to the error found, deflections where not taken into account for this particular design. Nevertheless, the two-way PT slab system is ideal for large spans and higher loads comparatively, and deflections would be minimal due to the compressive forces brought on by the tendons.

Considerations:

Structural – The post-tensioned system can be considered to be a 2-hr fire-rated system when the clear cover to the tendons is no less than 1.75 inches. Moreover, the 24" x 24" square concrete columns incorporated from the original design do not require any additional fire-proofing. Drop panels can be added to reduce slab thickness and column size if desired.

Architectural – The PT two-way system ultimately provides the least slab thickness required for the long spans. Shallower floor cavities will allow the structure to maintain the maximum 45' building height set by the township. *See previous flat plate system for more architectural considerations relating to a PT system.

Construction/M.E.P. – Similar to the Flat Plate system described earlier, the PT slab system will also be able to take advantage of the typical, repetitive bay layout. Even though its slightly more expensive due to the tendons and experienced construction team needed to place, a meticulous study of the system as a whole could prove to be more economical due to the symmetry & simplicity of the building. Worker safety is always an issue with a PT system as well. The high jacking force used to tension the tendons can have catastrophic circumstances if one of the tendons should slip or fail. Thus, a highly trained crew is needed for this potentially hazardous duty.

PRO-CON TABLE

Pro	Con
2-hr fire-rating (w/o add'l. fire-proofing)	worker safety
least slab thickness	labor intensive tendon layout
open plan layout	temp. & shrinkage issues
capable of long spans / high loads	

Conclusion – Although I did not completely design the post-tensioned system in terms of deflections and ordinary reinforcement for shear, I feel this alternate floor system could prove to be very worthy with more research. Additional information would need to be gathered and analyzed in order to determine the effects this system would have on the lateral system.

iii. COMPOSITE BEAMS w/ METAL DECK:

Material Properties:

4 ½ " NWC Concrete Cover

f'c = 3000 psi (NWC)

3" USD Lok Floor Metal Deck (16 Gage)

fy = 60000 psi

Total slab depth of 7 ½"

A992 Beams & Girders (fy = 50 ksi)

Loading:

Live Load = 125 psf (2nd Floor)

Dead Load = self wt. + 25 psf (superimposed)

Synopsis:

The composite floor system analyzed for Tech II consists of a 4 ½" concrete slab placed on 3" metal deck for a total slab depth of 7 ½". Wide flange infill beams and girders support the 7 ½" slab while shear studs connected to the beams and girders help form a composite action between the steel and concrete, resulting in higher design strength. And finally, not included in this report, the entire floor diaphragm would be held in place by A992, W-shaped steel columns.

Preliminary analyses I performed, provided in Appendix B, will show that a typical interior steel bay framed with W30x90(120) girders and W21x55(48) infill beams provides adequate strength. The previous numbers in parenthesis represent the number of shear studs required for each beam. An online steel floor framing design program available on AISC's website (www.aisc.org) was then used to evaluate the preliminary design using similar conditions. The first study (pg. 66) was calculated using the same number of infill beams, resulting in a comparable bay design; differing slightly in the number of shear studs required. For the second analysis an additional infill beam was added (pg. 67), however it did not significantly affect the size of the members.

Further investigation into this particular system may be able to utilize partial composite action. See Appendix B for all calculations, results and references.

Considerations:

Structural – Composite floor systems are relatively effective for control of deflections over long spans with heavy loads, however this typically results in larger, unwanted diaphragm depths. Steel framing is very versatile and can effortlessly be integrated into the existing column grid. The slab and metal deck combination provide a 2-hr fire-rating, but additional fire-proofing would be required for the beams and girders.

Architectural – Being able to sustain considerable spans, the composite system makes available an open floor plan that is greatly welcomed in lab areas and research facilities like CWB. Proving effective structurally, the composite system still results in a floor plenum analogous to the existing system or slightly over. In the end, this could mean adjusting story heights and an overall taller building.

Construction/M.E.P. – Construction time allotted for composite floor systems is significantly less than concrete designs. Erection time for the steel is typically a fast and efficient process by means of proper delivery and erection sequencing. Also, formwork is not necessary for the slab due to the supporting metal deck. This will allow for a greater area of concrete to be poured continuously.

PRO-CON TABLE

Pro	Con
allots for construction sequencing	heavy steel sections
fast erection time (no formwork)	total floor plenum depth
cost effective / construction sequencing	fire-proofing req'd. for beam & girders
capable of long spans / high loads	

Conclusion – The composite steel framing system with slab on metal deck designed for this technical report resulted in a floor depth thicker than the other alternative floor systems. Still, I feel this system is of value based on the time of construction required and no floor-to-floor height restrictions. Additional information would need to be gathered and analyzed in order to determine the effects this system would have on the lateral system.

-- PRO-CON SUMMARY --

	Existing System	Alternate No. 1	Alternate No. 2	Alternate No. 3
	Precast Hollow-Core Slabw/IT Beams	Two-Way Flat Plate System	Post-Tensioned Two-Way Slab	Composite Steel Framing w/ Slab on Metal Deck
Grid Adjustment	(existing)	Reduction in spans may be necessary	Drop panels required	None
Slab Depth	12" Precast Hollow-Core Plank	12" or greater	8" - 10" (Estimated)	4 1/2"
Plenum Depth for 2nd Floor Level	30"	12" or greater	N/A	28" - 36" plus
Maximum Total Deflections	0.093"	0.8" - 1.3"	Low	1.7"
Vibration	N/A	N/A	N/A	N/A
Additional Fire-Proofing	None Required	None Required	None Required	Apply to beams & girders
2-HR Fire-Rating	Achieved	Achieved	Achieved	Achieved pending inspection of additional fire-proofing
Formwork	No	Yes	Yes	No
Ease of Construction	Easy	Difficult	Difficult (Supervised)	Easy
Relative Cost	Very High	Medium to High	Medium to High	Low to Medium
Validity	(Existing)	Not at this time	Additional info. will be gathered & analyzed	Additional info. will be gathered & analyzed

-- CONCLUSIONS --

The purpose of Tech II was to generate ideas and concerns related to preliminary schematic design of possible alternative floor systems for Crocker West. Three alternative systems were selected, analyzed, and then compared amongst each other and the existing system to determine the feasibility of each system within the structure. Prestressed, precast hollow-core slabs and IT-beams make up the existing floor system; while a two-way flat plate system, post-tensioned two-way slab system, and composite steel framing with slab on metal deck system were the selected alternatives to be examined.

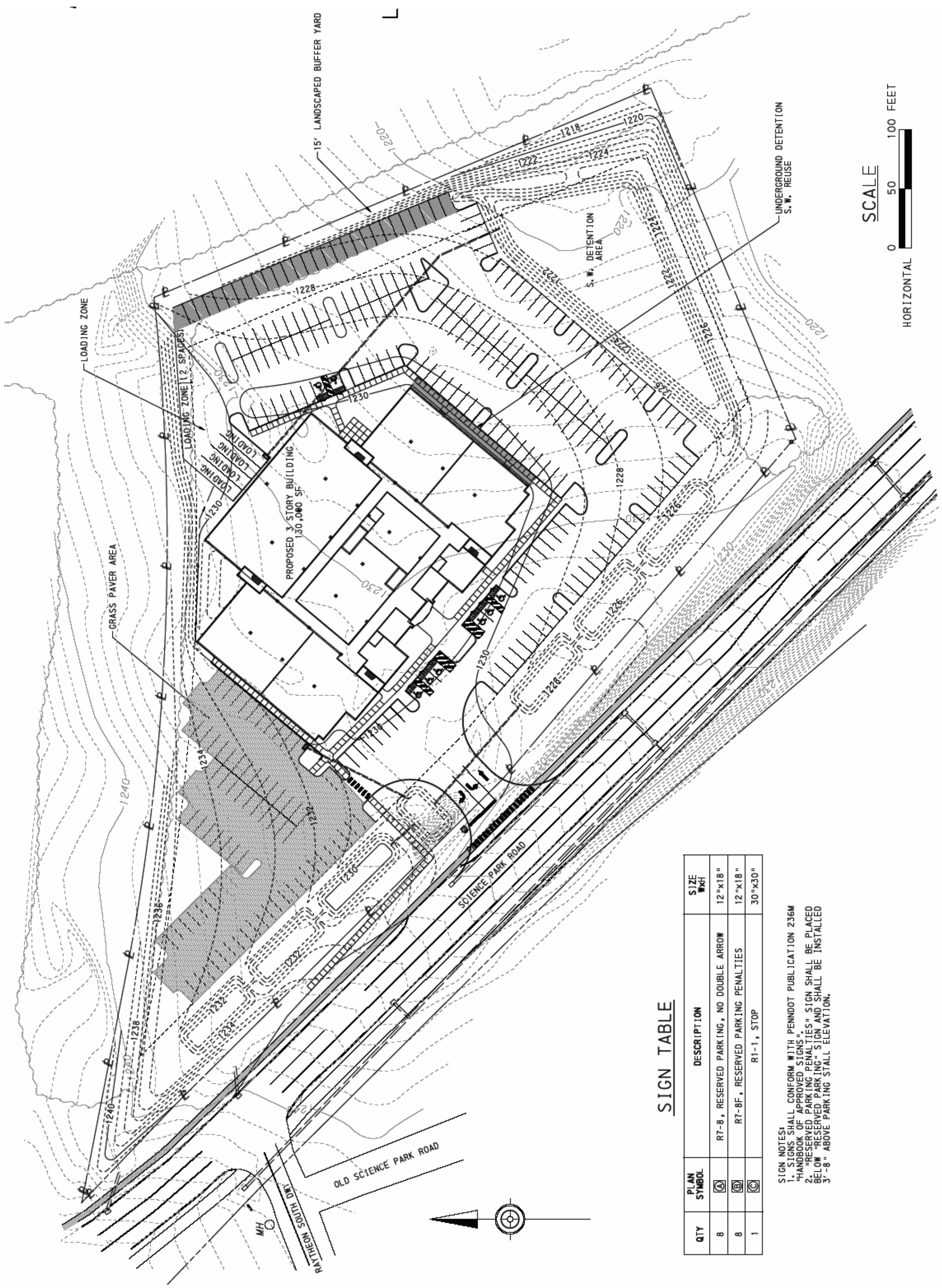
Preliminary analyses of the alternate systems do not strictly rule out any particular system; however, several prove to be very viable and worthy of future research. It appears as though the PT slab and composite systems have the most potential of being an effective alternative. The PT slab will be examined to greater detail based on the fact that this system is capable of carrying higher loads distributed over long spans with respect to plenum depth. PT slabs allow for thinner, lighter floor diaphragms while creating open space throughout the floor plan. The composite floor system generates a much deeper floor cavity than any other system, yet will be considered based on the speed of erection and ability to fast track the project. Similar to the existing system, the framing members are manufactured off-site, shipped to the site, and then erected in a reasonably fast, sequential manner. Cost is another factor for consideration of the composite system. Making use of the two materials (steel & concrete) for tensile and compressive forces makes this system economical and efficient.

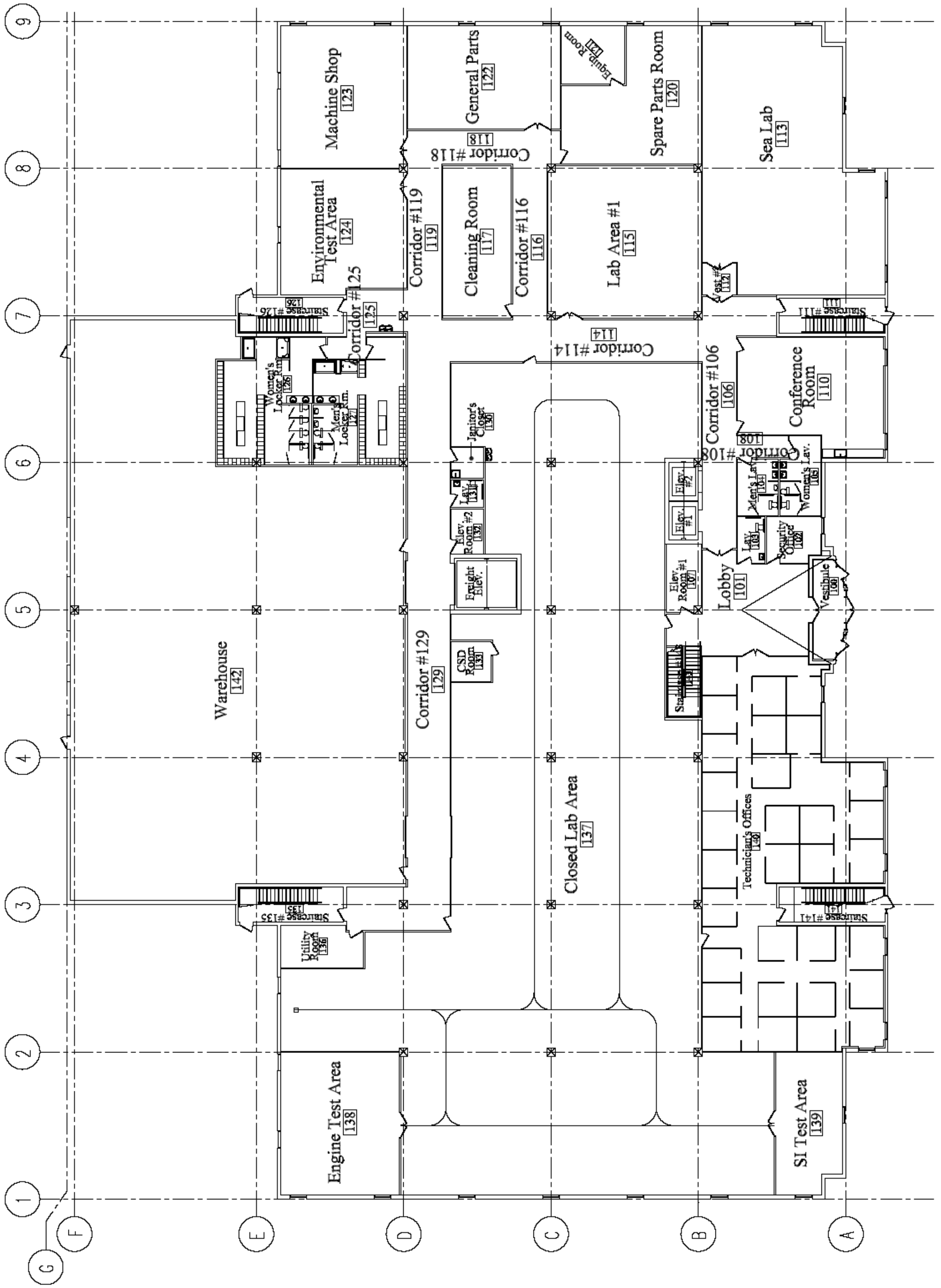
Although the two-way flat plate system was not chosen for in-depth study due to the extensive amount of reinforcing steel required, I feel a system similar to this could be validated and designed to work with this structure with a few manipulations of the existing plan. The required 35-foot spans lend themselves to concern of high deflections and unsafe conditions. Altering the column grid in order to reduce the clear spans is an option; however, this will reduce the amount of open space of the building.

Overall, I feel the existing prestressed system is one of the best choices for this project. Even though this is one of the most expensive systems, the hollow-core slabs and beams are very capable of spanning great distances with minimal deflection due to the majority of the members being cambered from the prestressing strands.

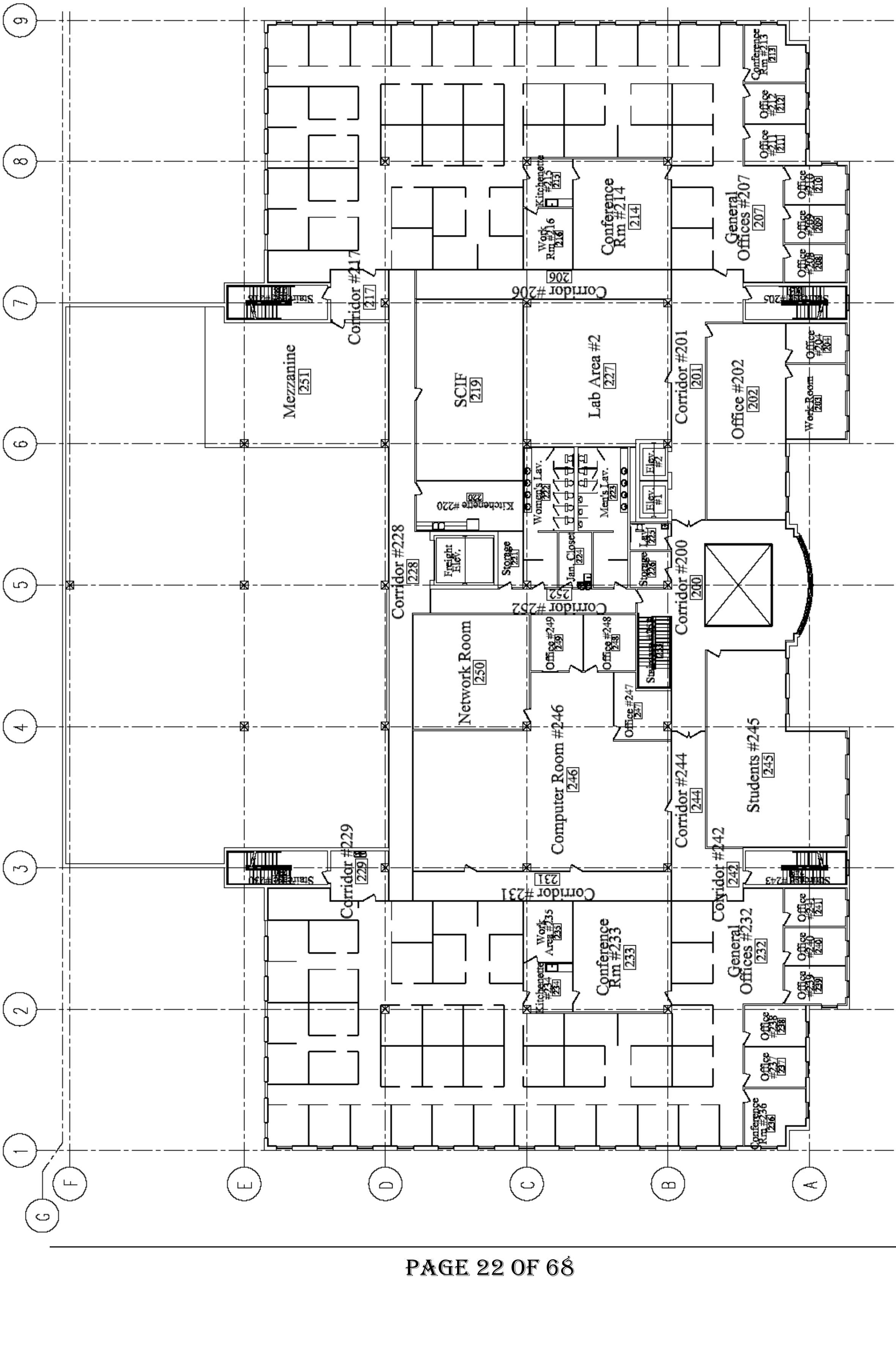
APPENDIX A

(Project Drawings)


15' LANDSCAPED BUFFER YARD
UNDERGROUND DETENTION S.W. REUSE
SCALE
0
50
100 FEET
HORIZONTAL
S.W. DETENTION AREA
LOADING ZONE
LOADING ZONE (2 SPACES)
LOADING
LOADING
PROPOSED 3 STORY BUILDING 30,000 SF
GRASS PAVER AREA
SCIENCE PARK ROAD
OLD SCIENCE PARK ROAD
RAYTHEON SOUTH DWY
MH
SIGN TABLE
QTY
PLAN SYMBOL
DESCRIPTION
SIZE WxH
8
A
R7-8, RESERVED PARKING, NO DOUBLE ARROW
12"x18"
8
B
R7-8F, RESERVED PARKING PENALTIES
12"x18"
1
C
R1-1, STOP
30"x30"
SIGN NOTES:
1. SIGNS SHALL CONFORM WITH PENNDOT PUBLICATION 236M "HANDBOOK OF APPROVED SIGNS".
2. "RESERVED PARKING PENALTIES" SIGN SHALL BE PLACED BELOW "RESERVED PARKING" SIGN AND SHALL BE INSTALLED 3'-8" ABOVE PARKING STALL ELEVATION.

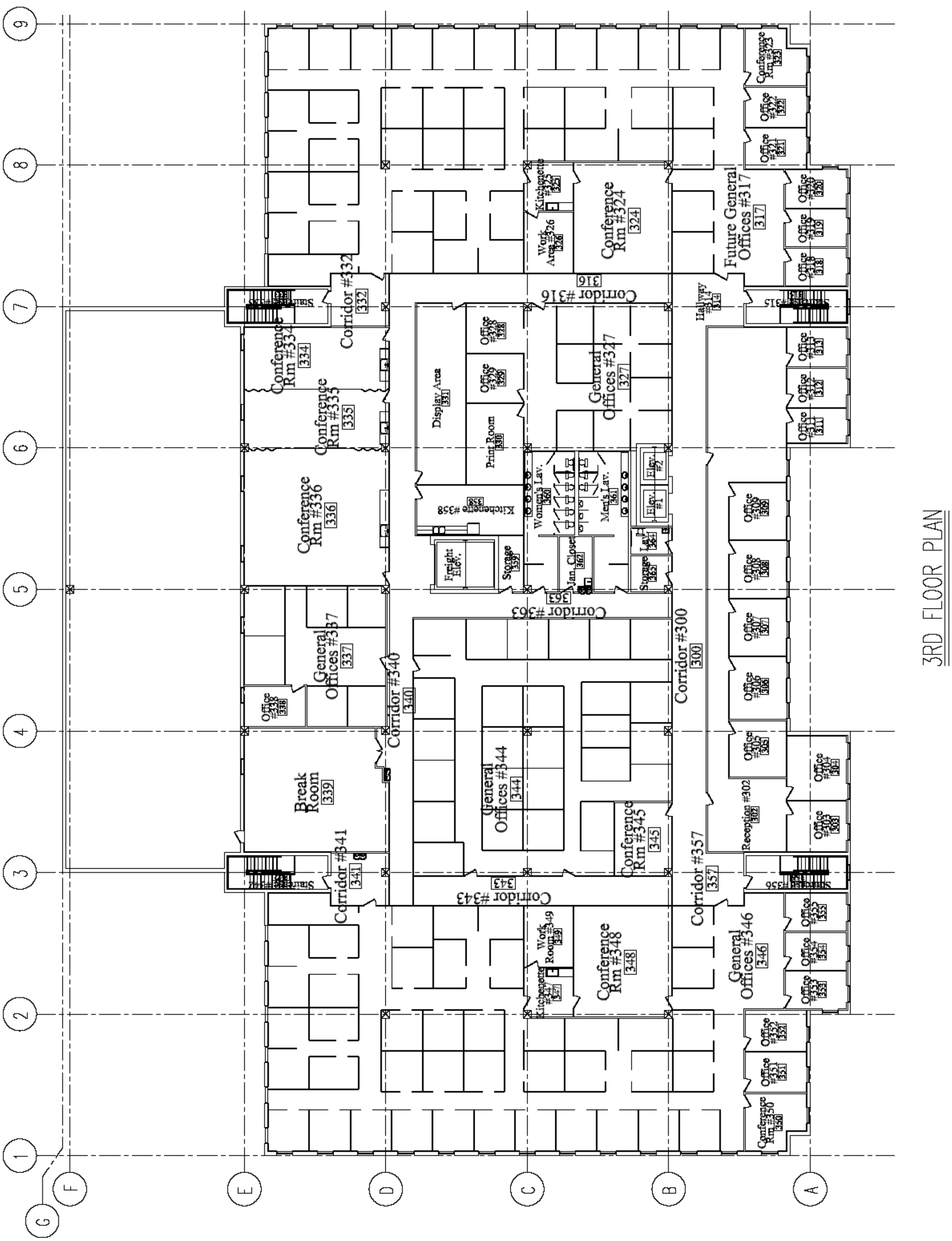


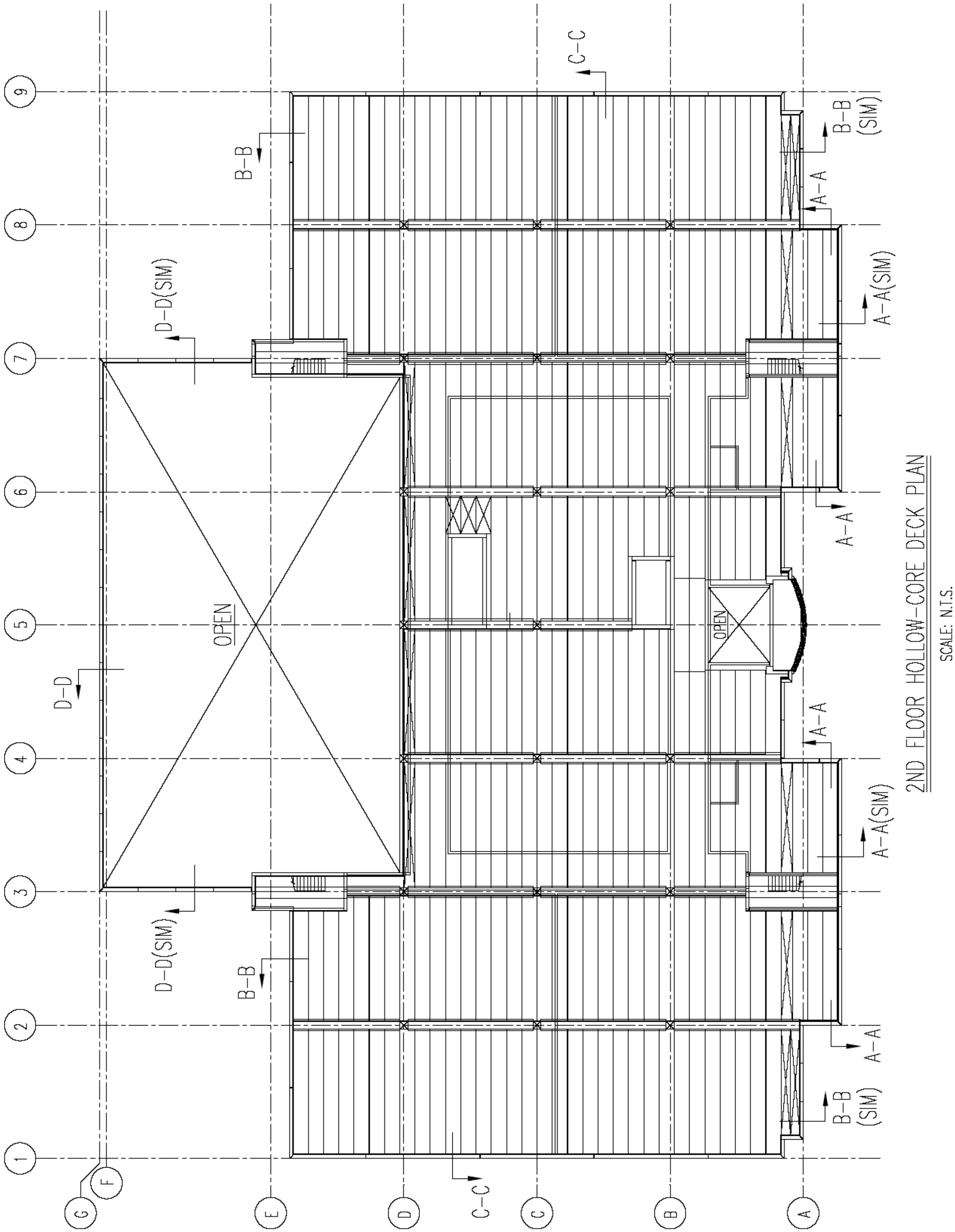
1ST FLOOR PLAN
SCALE: N.T.S.



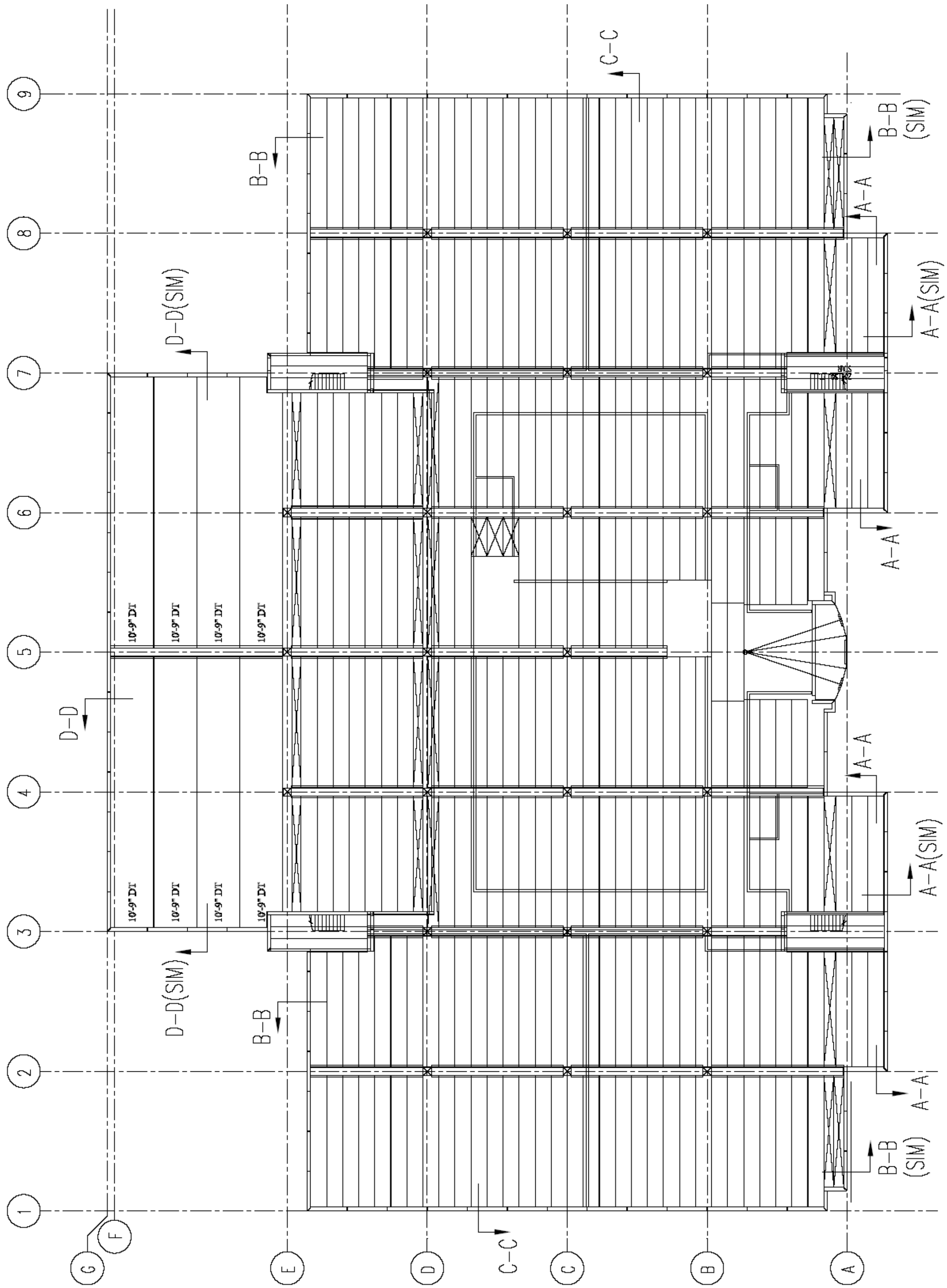
2ND FLOOR PLAN
SCALE: N.T.S.

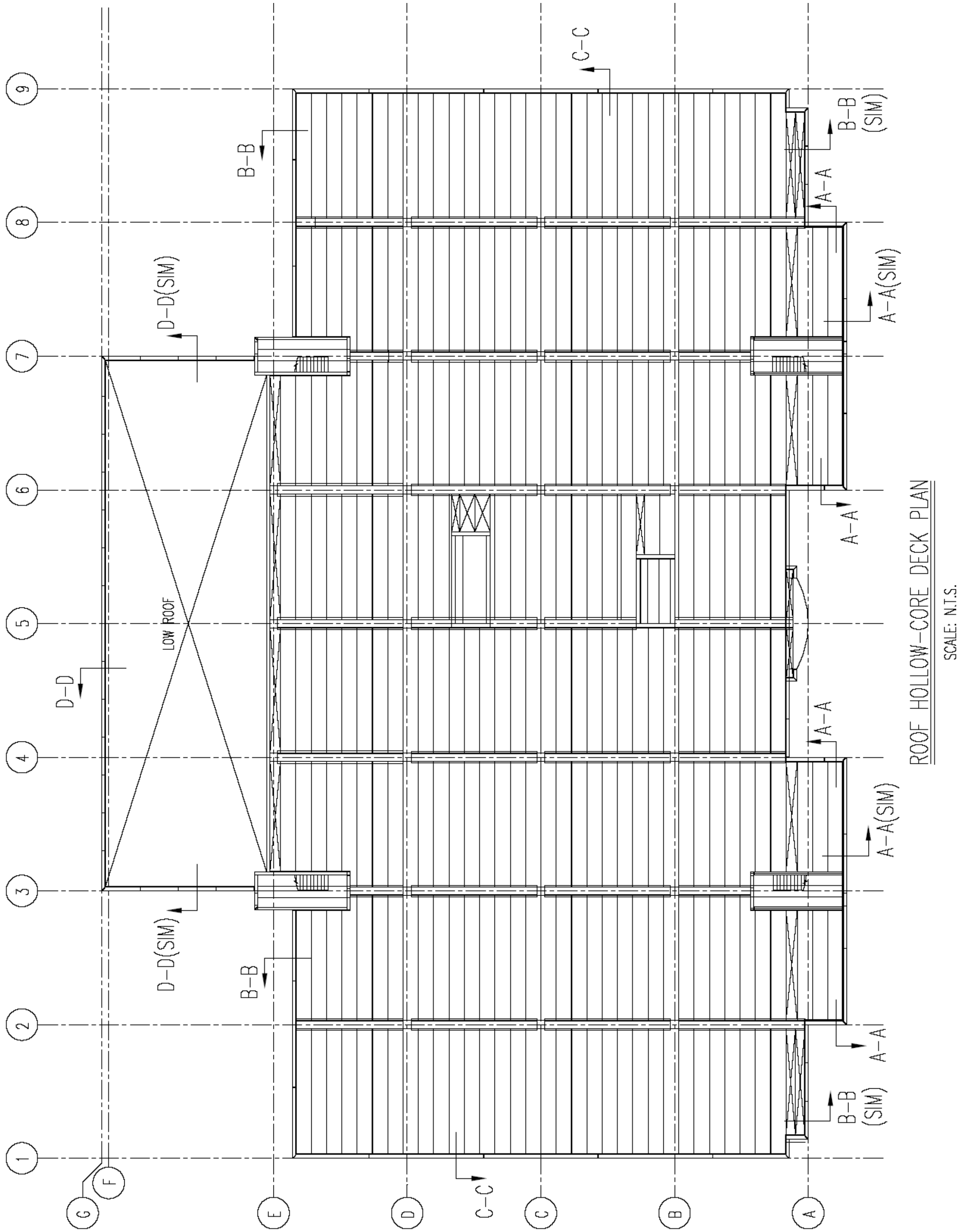
TECH REPORT II

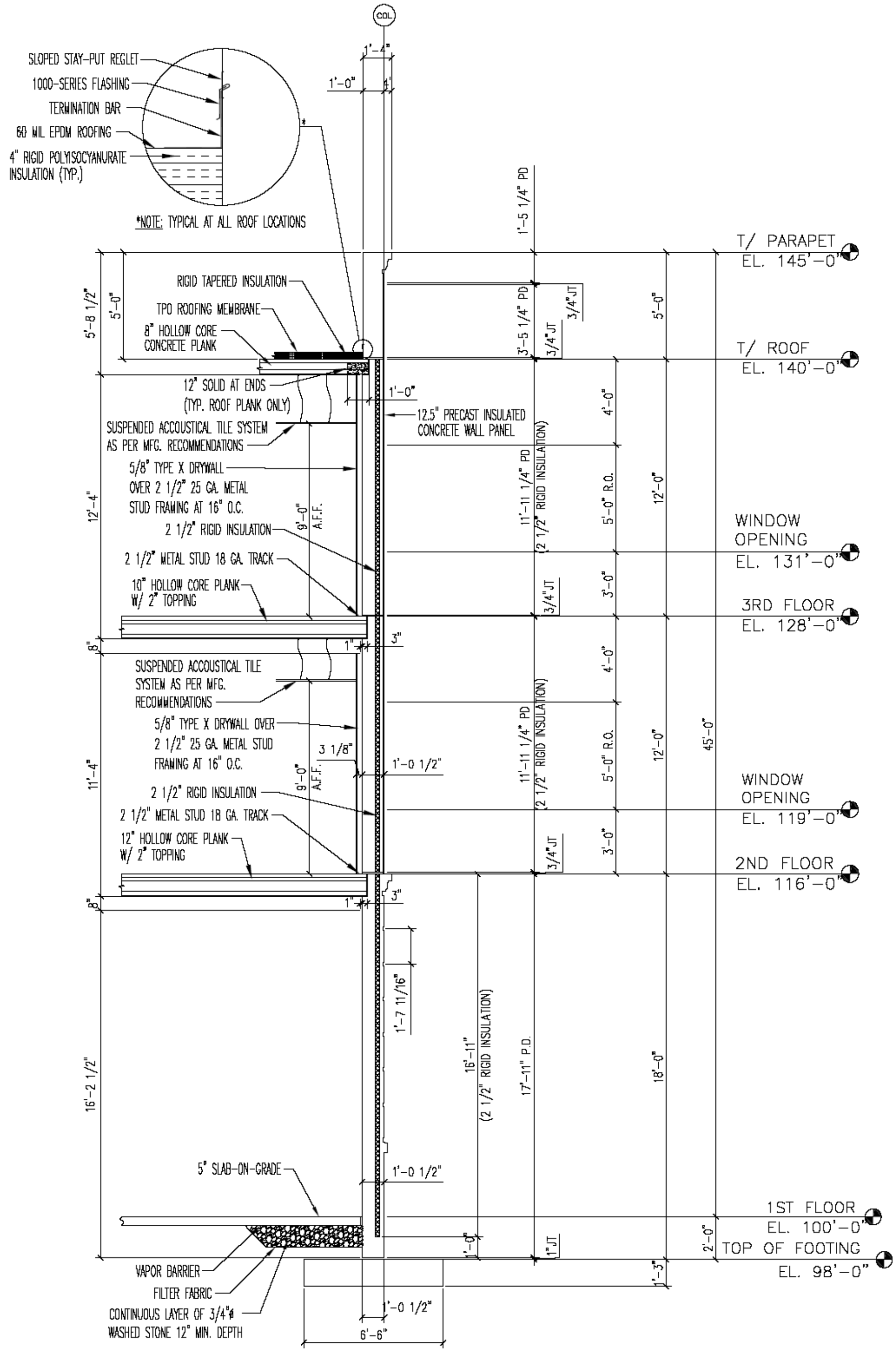



2ND FLOOR HOLLOW-CORE DECK PLAN
SCALE: N.T.S.
OPEN
OPEN
A–A
A–A(SIM)
B–B
B–B (SIM)
C–C
D–D
D–D(SIM)
1
2
3
4
5
6
7
8
9
A
B
C
D
E
F
G

TECH REPORT II



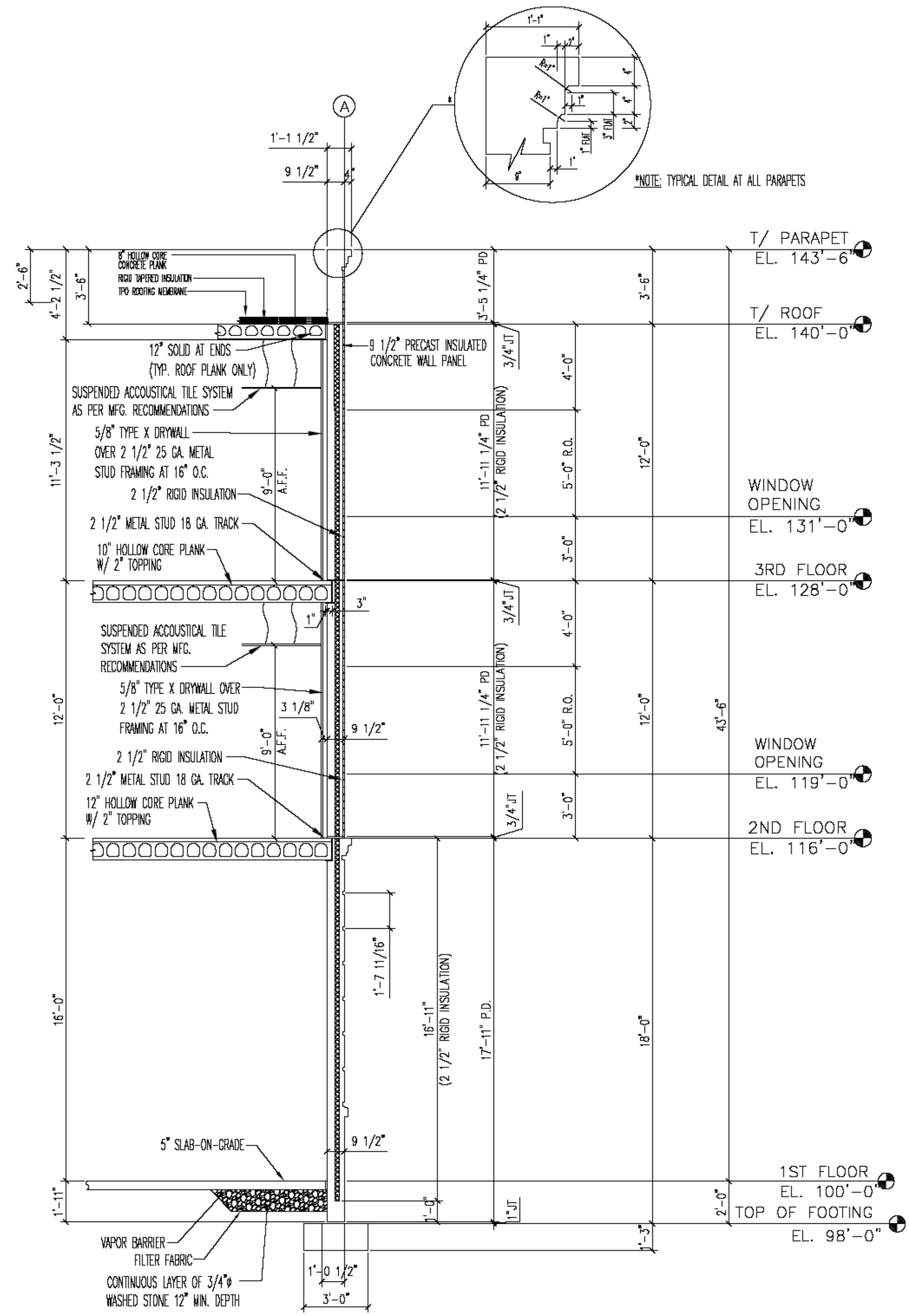

ROOF HOLLOW-CORE DECK PLAN
SCALE: N.T.S.
LOW ROOF
A-A
A-A(SIM)
B-B
B-B (SIM)
C-C
D-D
D-D(SIM)
1
2
3
4
5
6
7
8
9
A
B
C
D
E
F
G



SECTION A – A

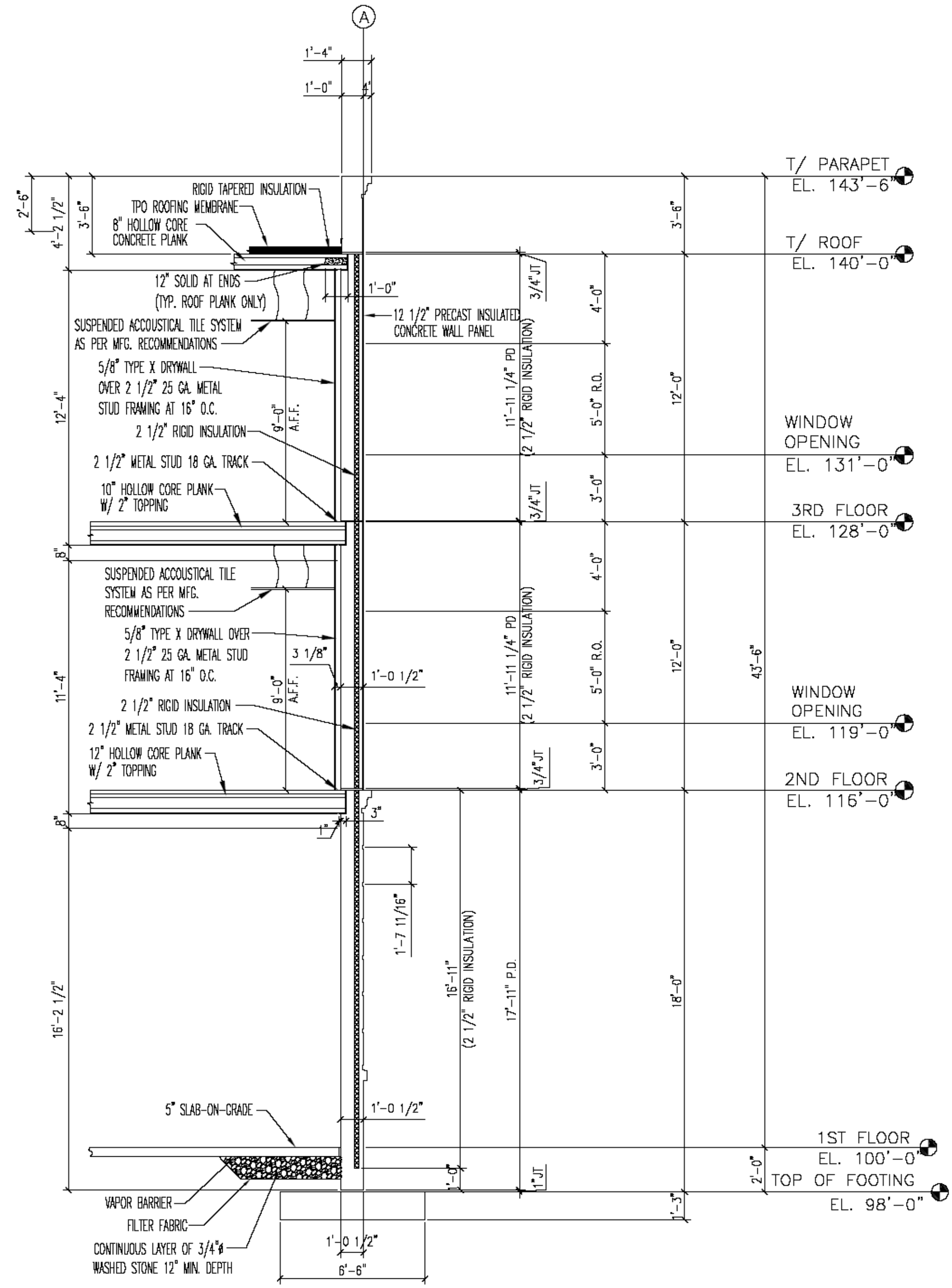
SCALE 3/8" = 1'-0"

TECH REPORT II



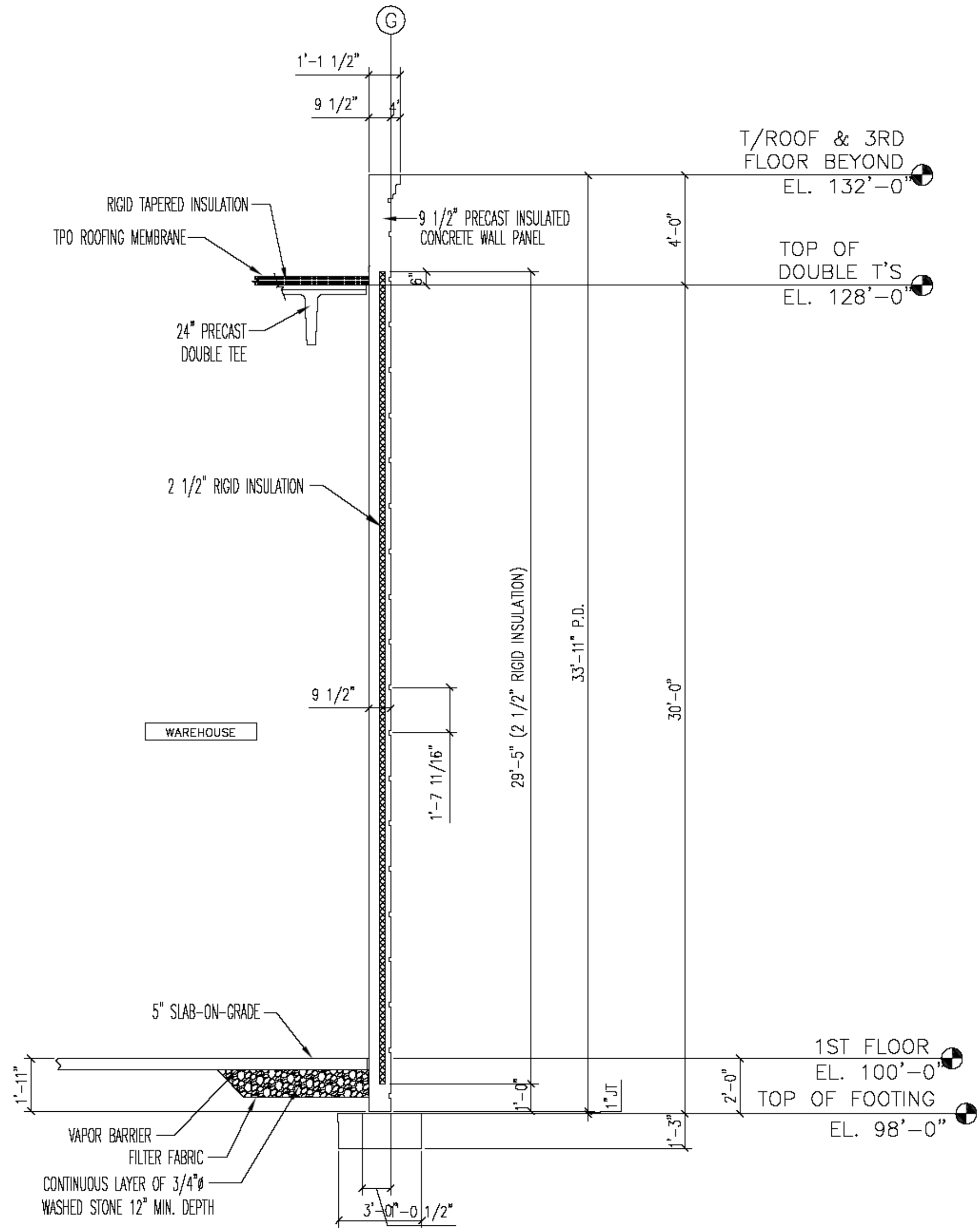
SECTION B – B

SCALE 3/8" = 1'-0"



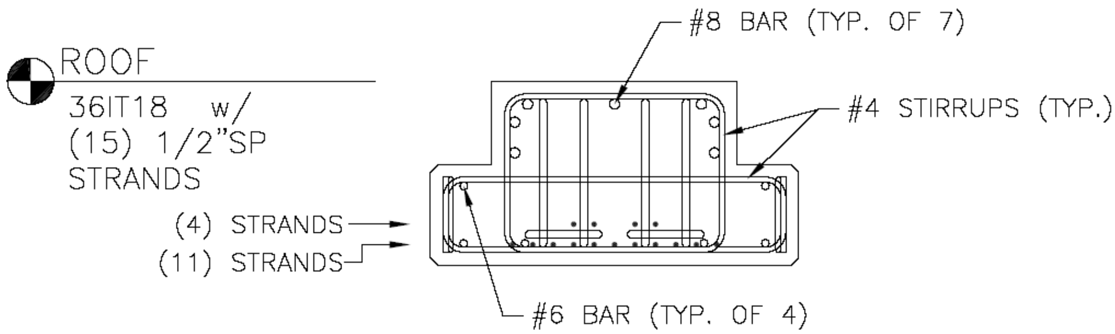
SECTION C – C

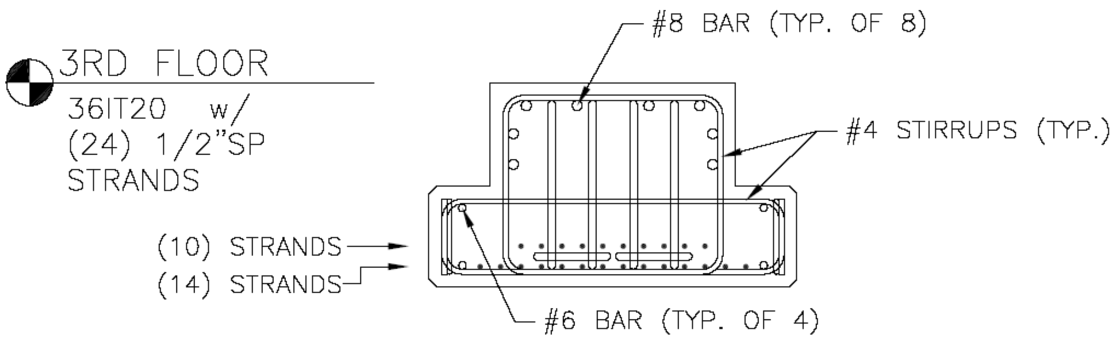
SCALE 3/8" = 1'-0"

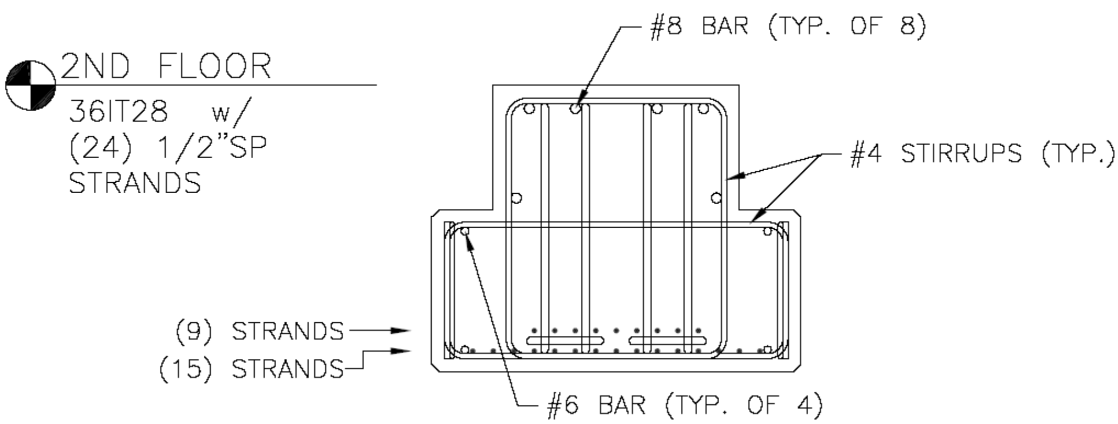


SECTION D – D

SCALE 3/8" = 1'-0"







TYPICAL IT BEAM SECTIONS

Appendix B

(Calculations, Results, & References)

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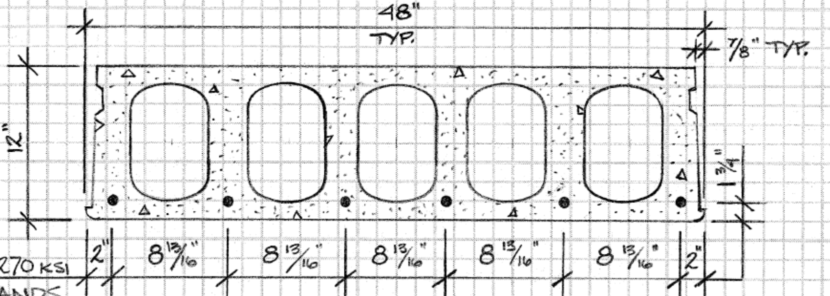
Ckd By ______
Date ______
Project TECH II

COMMENTS
EXISTING : 12" PRECAST, HC PLANK



(6) 1/2" SP, 270 KSI
LOW-LAX STRANDS

12 IN. HC SECTION
N.T.S.

∗ SLAB WT. = 72.5 PSF (UNGROUTED)

(6) 1/2" SP, 270 KSI LOW-LAX STRANDS
INITIAL STRESS = $0.70\, f_{pu}$

$f'_c = 5000$ PSI
$f'_{ci} = 3500$ PSI

TYP. SLAB LENGTH = 32'-10" ⇒ ASSUME:
MIN. 2" BEARING = 32'-6" CLR. SPAN

LOADING →

DEAD: 72.5 PSF (UNGROUTED SELF WT.)
25 PSF (2" TOPPING)
12 PSF (MISC.)
109.5 PSF

LIVE: 125 PSF
125 PSF

$\boxed{\begin{array}{l} DL = 110 \text{ PSF} \\ LL = 125 \text{ PSF} \end{array}}$

◇ FLEX. STRENGTH DESIGN

$\gamma_p = 0.28$ (LOW-LAX STRANDS)

$\beta_1 = 0.80$ ($f'_c = 5$ KSI)

$$\rho_p = \frac{A_{ps}}{b d_p} = \frac{6(0.167 \text{ in}^2)}{(48")(12"-1.75")} = 0.002037$$

$$f_{ps} = f_{pu}\left[1 - \frac{\gamma_p}{\beta_1}\left(\rho_p \frac{f_{pu}}{f'_c}\right)\right] = 270\left[1 - \frac{0.28}{0.80}\left(0.002\, \frac{270}{5}\right)\right]$$

▲ $f_{ps} = 259.607$ KSI

$$\omega_p = \frac{\rho_p f_{ps}}{f'_c} = \frac{0.002(259.6)}{5} = 0.1057$$

▲ $\omega_p = 0.106$ → (< 0.288 = $0.36\beta_1$ ∴ OK✓)

→

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$$a = \frac{A_{ps} f_{ps}}{0.85 f'_c b} = \frac{(6 \times 0.167)(259.6)}{0.85(5)(48")} = 1.27513$$

$\blacktriangle$ $a = 1.28"$

FIND ϕM_n (FLEX. STR.)

$$\phi M_n = \phi A_{ps} f_{ps} (d_p - a/2)$$
$$= 0.9(1.002)(259.6)(10.25 - 1.28/2)$$

$\blacktriangle$ $\phi M_n = 2250.4$ IN-K [187.5 FT-K]

ACTUAL

$$M_u = \frac{\omega L^2}{8}$$

1. $\omega = 1.2(110) + 1.6(125)$
$\omega = 332$ PSF
2. $L = 32.5$ FT (CLR. SPAN)

$$M_u = \frac{0.332(32.5)^2}{8}$$

$M_u = 43.8$ FT-K/FT (× 4 FT WIDE) = 175.3 FT-K/SLAB

→ $\blacktriangle$ ACTUAL $M_u = 175.3$ FT-K PER SLAB < 187.5 FT-K PER SLAB = ϕM_n FLEX. STR.

$\therefore$ OK ✓

◇ MIN. REINF. CHECK ($\phi M_n > 1.2 M_{cr}$)

PRESTRESS LOSSES → $A_{ps} f_{pu} = 0.167(270) = 45.09$ K/STRAND

1. ELASTIC SHORTENING (ES)

$P_{pi} = 0.7(6)(45.1) = 189.378^K$ → $P_{pi} = 189.4^K$ ← C.S.R. (PG. 1)

COMPARE TO 11.7% IN THE FOLLOWING

$M_d = \frac{32.5^2}{8}(0.0725 \times 4') = 38.289$ FT-K (459.469 IN-K) → $M_d = 38.3$ FT-K [459.5 IN-K]

$$f_{cir} = K_{cir}\left(\frac{P_{pi}}{A} + \frac{P_{pi} e^2}{I}\right) - \frac{M_d e}{I}$$

W: A = AREA = 279 IN²
I = 5190 IN⁴
e = 4.25 IN

$$= 0.9\left(\frac{189.4^K}{279 \text{ IN}^2} + \frac{189.4^K (4.25 \text{ IN})^2}{5190 \text{ IN}^4}\right) - \frac{459.5 \text{ IN-K}(4.25 \text{ IN})}{5190 \text{ IN}^4} = 0.827935 \text{ KSI}$$

→

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$f_{cir} = 0.83$ ksi

$ES = K_{es} E_s \frac{f_{cir}}{E_{ci}}$ W: $E_s = 28000$ ksi, $E_{ci} = 3587$ ksi } BOTH TAKEN FROM PG. 1 OF C.S.R.

$= 1.0 \left(\frac{28000}{3587}\right)(0.83) = 6.46283$

ES → ▲ $ES = 6.46$ ksi

2. CONCRETE CREEP (CR)

$f_{cds} = \frac{M_{sd}\, e}{I}$ W: $M_{sd} = \frac{32.5^2}{8}(0.012 \times 4') = 6.3375$ FT-K [76.05 IN-K]

$= \frac{(76.05 \text{ in-k})(4.25 \text{ in})}{5190 \text{ in}^4} = 0.0622276$

$f_{cds} = 0.0623$ $E_c = 4287$ ksi (REF. PG. 1 C.S.R.)

$CR = K_{cr} \frac{E_s}{E_c}(f_{cir} - f_{cds})$

$= 2.0 \left(\frac{28000}{4287}\right)(0.83 - 0.0623) = 10.0016$

CR → ▲ $CR = 10.0$ ksi

3. CONCRETE SHRINKAGE (SH)

$V/S = 2.35$ in (REF. PG. 1 C.S.R.)

R.H. = 70%

$SH = 8.2 \times 10^{-6} K_{sh} E_s (1 - 0.06(V/S)) \times (100 - R.H.)$

$= 8.2 \times 10^{-6}(1.0)(28000)[1 - 0.06(2.35)](100 - 70) = 5.91679$

SH → ▲ $SH = 5.92$ ksi

4. STEEL RELAXATION (RE)

$K_{re} = 5000$ psi → $J = 0.040$ (TBL 2.2.3.1 ~ PCI MAN.: DESIGN OF HC SLABS)

$C = 0.75$ (TBL 2.2.3.2 ~ ")

$RE = [K_{re} - J(SH + CR + ES)]C$

$= 0.75[5 \text{ ksi} - 0.040(5.92 + 10 + 6.46)] = 3.0786$

RE → ▲ $RE = 3.08$ ksi

→

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5. TOTAL LOSS AT MIDSPAN (TL)

$$TL = ES + CR + SH + RE = 25.46 \text{ ksi}$$

$$\frac{25.46 \text{ ksi}}{0.7\,(270 \text{ ksi})} = 0.1347\ (\times 100) = 13.5\ \% \quad \text{FINAL LOSSES}$$

COMPARE TO 11.7% ON PG. 1 OF C.S.R.

(CHECK $\phi M_n > 1.2 M_{cr}$)

$$A_{ps} f_{se} = 0.70(6 \times 45.09)(1 - 0.135) = 163.867^k$$

$$M_{cr} = \frac{I}{y_b}\left(\frac{P}{A} + \frac{Pe}{S_b} + 7.5\sqrt{f_c'}\right)$$

$$= \frac{5190}{5.92}\left(\frac{164}{279} + \frac{164(4.25)}{877} + \frac{7.5\sqrt{5000}}{1000}\right) = 1676.04 \text{ in-k}$$

$$M_{cr} = 1676 \text{ in-k/slab}$$

A. $1.2\,M_{cr} = 2011.2$ in-k/slab 3. $\phi M_n = 2250.4$ in-k/slab

$$\therefore \phi M_n > 1.2\,M_{cr} \quad \text{OK}\checkmark$$

◇ DEFLECTION

$$D+L \rightarrow \Delta = L/240 = \frac{(32'\text{-}10)(12)}{240} = 1.64''$$

$$L \rightarrow \Delta = L/360 = 1.09''$$

⇒ FROM PG. 4 OF THE C.S.R. THE MAX. DEFL. $\Delta_{max} = 0.554''$ AND $\Delta_{final} = -0.092''$ AT FINAL LOADING.

$$\text{ALLOWABLE}\begin{cases}\Delta_{DL} = 1.64'' \\ \Delta_L = 1.04\end{cases} > \begin{matrix} 0.554'' \text{ (CAMBER)} \\ -0.092'' \text{ (DEFL. @ FINAL)}\end{matrix}$$

∴ OK✓

END OF SECTION

Summary Report

Concise Beam (TM), Version 4.46f, (c) 2006 Black Mint Software, Inc
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Project: Applied Research Laboratory V
Problem: 2nd Floor 12" HC Plank

SUMMARY REPORT

Design Code Used: ACI318-05

CONCRETE MATERIAL PROPERTIES

	Precast Beam			Topping		
Concrete Density	Wt =	150	lb/ft^3	Wt =	150	lb/ft^3
Compressive Strength	f'c =	5.0	ksi	f'c =	3.0	ksi
Modulus of Elasticity	Ec =	4287	ksi	Ec =	3320	ksi
Strength at Transfer	f'ci =	3.5	ksi			
Modulus of Elast. at Transfer	Eci =	3587	ksi			

Cement Content = 0 lb/yd^3
Air Content = 5.00 %
Slump = 0.00 in
Aggregate Mix = 0.40 (ratio fine to total aggregate)
Aggregate Size = 0.00 in
Curing Method = Moist
Humidity = 70 %
Basic Shrinkage Strain = 780E-6

Construction Schedule *
Age at Transfer = 0.75 days
Age at Erection = 40 days
Age at Topping Placement = 50 days
Age Topping is Composite = 53 days
* for loss calculations only)

BEAM LAYOUT

Beam is UNSHORED during topping placement and superimposed dead load.

Segment/Length No	From ft	To ft	Offset Z in	Offset Y in	Section Identification Folder	Section	Topping Parameters t1 in	b1 in	t2 in	b2 in
1	0.00	32.83	0.00	0.00	HollowCore	HC4'x12"	0.00	0.00	2.00	48.00

Total Beam Length = 32.83 ft, Left Support @ 0.17 ft, Right Support @ 32.66 ft, Span = 32.49 ft

PRECAST SECTION PROPERTIES (NON-COMPOSITE) *

Seg. No.	A in^2	I in^4	yb in	Sb in^3	St in^3	V/S in	bw in	width in	height in
1	279.0	5190	5.92	877	854	2.35	10.38	48.00	12.00

* These properties do not include the transformed area of any reinforcing or prestressing steel. See the Transformed Section Properties text report for properties that include the area of steel.

COMPOSITE SECTION PROPERTIES *

Seg. No.	Ac in^2	Ic in^4	yb in	yt in	ytt in	Sb in^3	St in^3	Stt in^3	hc in
1	353.4	8158	7.41	4.59	6.59	1101	1777	1238	14.00

* These properties do not include the transformed area of any reinforcing or prestressing steel. See the Transformed Section Properties text report for properties that include the area of steel.

Note: yt & St refer to the top of the precast beam, and ytt & Stt refer to the top of the topping.

PRESTRESSING STEEL TENDONS

Prestressing Strand Details

ID	Qty	Material	Section	Offsets x ft	Offsets y in	End Offset & Type * Left ft	Right ft	Tendon Area in^2	Jacking Force Pj kip	%fpu
1	6	fpu=270 ksi Es= 28000.0 ksi	SWS#1/2"SP	0.00 32.83	1.75 1.75	0.00 B	0.00 B	1.002	189.4	0.70

notes: * Strand End Types: B - Fully Bonded, D - Debonded, C - Cut, A - Anchored (fully developed)
Prestressing steel is low relaxation strand.
Calculated Losses: Initial = 3.0 %, Final = 11.7 % COMPARE TO 13.5%
Maximum Total Prestress Forces: Pj(jacking) = 189.4 kip, COMPARE TO 189.4k
Pi(transfer) = 183.7 kip,

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Problem: 2nd Floor 12" HC Plank

Pe(effective) = 167.3 kip @ x = 16.42 ft

Prestressing Strand Transfer and Development Lengths

ID	Diameter in	End	Debond Length ft	fse psi	fps psi	Transfer in	Development in
1	0.50	LEFT	0.00	159286	264600	26.51	79.16
1	0.50	RIGHT	0.00	159286	264600	26.51	79.16

BEAM AND TOPPING SELF-WEIGHT

Segment/Length No.	From ft	To ft	Linear Weight Beam kip/ft	Topping kip/ft
1	0.00	32.83	0.29	0.10

EXTERNALLY APPLIED LOADS

Load Case		Load Label	Load Type	Load Intensity (*) Left	Right	Offset (ft) Left	Right
Beam Weight	D	Addt'l Self-Wt.	Line Load	0.03	0.03	0.00	32.83
SDL AT	D	12 PSF	Line Load	0.05	0.05	0.00	32.83
Live Load	L	125 PSF	Line Load	0.50	0.50	0.00	32.83

* point loads = kip, line loads = kip/ft, point moment/torsion = kipft, line torsion = kipft/ft

Load Combinations
Factored Combination 1 = 1.40D + 1.40F
Factored Combination 2 = 1.20D + 1.60L + 0.50SRLr + 1.20F + 1.20T
Factored Combination 3 = 1.20D + 0.50L* + 1.60SRLr
Factored Combination 4 = 1.20D + 1.60SRLr + 0.80WE
Factored Combination 5 = 1.20D + 0.50L* + 0.50SRLr + 1.60WE
Factored Combination 6 = 0.90D + 1.60WE
* Load factor reduced from 1.0 to 0.5 for low live loading (garage, public assembly, < 100 lb/ft2)
(The use of T is not yet implemented)

SHEAR STIRRUPS

From ft	To ft	Stirrup Grade ksi	Stirrup Size	Number of Legs Stirrup in Beam	Interface Ties	Total Stirrup Area Stirrup in^2	Interface in^2	Stirrup Spacing Stirrup in	Interface in
0.00	32.83	60.0		0	0	0.00	0.00	0.00	0.00

TORSION PARAMETERS

Seg. No.	Torsion Parameters Aoh in^2	Ph in
1	0.00	0.00

Aoh is the area enclosed by the centerline of the outermost closed transverse torsional reinforcement.
Ph is the perimeter of the area defined as Aoh.

ANALYSIS RESULTS SUMMARY

x (ft)	Total Unfactored Effects Moment (kipft) Total	Sustained	Total Factored Effects Shear (kip)	Moment (kipft)	Torsion (kipft)
0.00	0.0	0.0	0.0	0.0	0.0
0.17	0.0	0.0	-0.2	0.0	0.0
0.17	0.0	0.0	22.2	0.0	0.0
3.42	46.1	22.4	17.7	64.8	0.0
6.67	82.0	39.7	13.3	115.2	0.0
9.92	107.6	52.2	8.9	151.3	0.0

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13.17	123.0	59.6	4.4	172.9	0.0
16.42	128.1	62.1	0.0	180.1	0.0
19.83	122.4	59.4	-4.7	172.1	0.0
23.08	106.5	51.7	-9.1	149.8	0.0
26.33	80.4	39.0	-13.5	113.1	0.0
29.58	44.0	21.4	-18.0	61.9	0.0
32.66	0.0	0.0	-22.2	0.0	0.0
32.66	0.0	0.0	0.2	0.0	0.0
32.83	0.0	0.0	0.0	0.0	0.0

SUPPORT REACTIONS (kip)

Load Case	Unfactored Support Reactions Left	Right
Beam Weight	5.3	5.3
SDL BT	0.0	0.0
Topping Wgt	1.6	1.6
SDL AT	0.8	0.8
LL Sustain	0.0	0.0
Live Load	8.2	8.2
Roof Load	0.0	0.0
Fluid Wgt	0.0	0.0
VWind or EQ	0.0	0.0
Strain Load	0.0	0.0

Load Combo.	Left	Right
Sust. Total	7.7	7.7
Total	15.9	15.9
Factor Max.	22.4	22.4

CONCRETE STRESS RESULTS
(+ve = compression, -ve = tension)

Location	x ft	Stress psi	Limit psi	Overstress Notice	
STRESSES AT TRANSFER					
Critical Compression					
Top of Beam	16.42	376	2450	0 %	
Bottom of Beam	2.28	1297	2450	0 %	Longitudinal Tensile Rebar Needed (in^2)
Critical Tension					Required Provided Additional
Top of Beam	2.12	-70	-444	0 %	
Bottom of Beam	0.00	2	-444	0 %	
STRESSES IN SERVICE					
Critical Compression					
Top of Beam	16.42	1057	3000	0 %	
Bottom of Beam	2.28	907	3000	0 %	
Top of Topping	16.42	541	1800	0 %	
Critical Tension					
Top of Beam	0.17	-16	-849 *	0 %	Class U member - not cracked *
Bottom of Beam	16.42	-165	-849 *	0 %	Class U member - not cracked
Top of Topping	0.17	0	-657 *	0 %	
STRESSES IN SERVICE (SUSTAINED LOADS ONLY)					
Critical Compression					
Top of Beam	16.42	613	2250	0 %	
Bottom of Beam	2.28	1076	2250	0 %	

* Bilinear deflection calculation used.

Modulus of Rupture, fr =	-530	psi
Transfer Strength Required, f'ci =	1.9	ksi
Transfer Strength Specified, f'ci =	3.5	ksi

DISTRIBUTION OF FLEXURAL STEEL & CRACKING
Beam not cracked or crack depth is less than concrete cover.

Engineer: EMF — Company: Civilsmith Engineering, Inc.
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Summary Report

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NET DEFLECTION ESTIMATE AT ALL STAGES
(-ve = deflection down, +ve = camber up)
Deflection growth estimated by use of PCI suggested multipliers - see multiplier report
Design Code Used: ACI318-05

MIDSPAN

Location x ft	Net Deflection: Net @ * Transfer in	Net @ Erection in	Net @ Complete in	Net DL @ Final in	Net Total @ Final in	Change in Deflection: DL growth + LL ** in	LL alone in	Span/Deflection: DL growth + LL **	LL alone
0.00	0.000	-0.013	-0.011	-0.009	-0.003	0.007	0.006	545	694
0.17	0.007	0.000	0.000	0.000	0.000	0.000	0.000	0	0
3.42	0.135	0.231	0.185	0.151	0.040	-0.145	-0.110	2696	3538
6.67	0.222	0.388	0.301	0.225	0.017	-0.284	-0.208	1370	1870
9.92	0.276	0.485	0.367	0.253	-0.033	-0.400	-0.285	975	1366
13.17	0.306	0.538	0.399	0.259	-0.076	-0.475	-0.334	821	1166
16.42	0.315	0.554	0.409	0.259	$\boxed{-0.092}$	-0.501	-0.351	778	1110
19.83	0.305	0.536	0.398	0.259	-0.074	-0.472	-0.333	825	1172
23.08	0.274	0.482	0.364	0.252	-0.030	-0.395	-0.282	988	1381
26.33	0.219	0.382	0.297	0.223	0.019	-0.278	-0.204	1403	1911
29.58	0.130	0.221	0.178	0.145	0.040	-0.137	-0.105	2839	3717
32.66	0.007	0.000	0.000	0.000	0.000	0.000	0.000	0	0
32.83	0.000	-0.013	-0.011	-0.009	-0.003	0.007	0.006	545	694

Span/Deflection Limits: DL growth + LL * = L / 480 for non-structural attachments
L / 240 otherwise
LL alone = L / 360 for floors
L / 180 for roofs

* on temporary supports at transfer ** after completion, including placement of all DL

FLEXURAL DESIGN CHECK

Design Code Used: ACI318-05
Beta Used: for precast beam = 0.800 , for topping = 0.850
The maximum value for fps is limited to 0.98 fpu.

x ft	Factored Moment Mu kipft	Design Strength ØMn kipft	Minimum Strength 1.2Mcr kipft	Depth in Compression c in	Net Tensile Strain	Flexure Class	Ø	Notes & Warnings
0.00	0.0	0.1	58.5	0.03	1.2412	Tension	0.75	
0.17	0.0	-1.3	-44.0	0.07	0.0732	Tension	0.75	
3.42	64.8	149.4	200.1	1.80	0.0174	Tension	0.83	
6.67	115.2	222.1	198.0	2.47	0.0119	Tension	0.90	
9.92	151.3	222.1	196.5	2.47	0.0119	Tension	0.90	
13.17	172.9	222.1	195.6	2.47	0.0119	Tension	0.90	
16.42	180.1	222.1	195.3	2.47	0.0119	Tension	0.90	
19.83	172.1	222.1	195.6	2.47	0.0119	Tension	0.90	
23.08	149.8	222.1	196.5	2.47	0.0119	Tension	0.90	
26.33	113.1	220.0	198.1	2.47	0.0119	Tension	0.90	
29.58	61.9	145.8	200.2	1.76	0.0179	Tension	0.82	
32.66	0.0	-1.3	-44.0	0.07	0.0732	Tension	0.75	
32.83	0.0	0.1	58.5	0.03	1.2412	Tension	0.75	
Points of Maximum and Minimum Factored Moment								
16.42	180.1	222.1	195.3	2.47	0.0119	Tension	0.90	
0.17	0.0	-1.3	-44.0	0.07	0.0732	Tension	0.75	
Points of Critical Moment Design								
16.42	180.1	222.1	195.3	2.47	0.0119	Tension	0.90	
0.17	0.0	-1.3	-44.0	0.07	0.0732	Tension	0.75	

SHEAR AND TORSION DESIGN CHECK

Design Code Used: ACI318-05

Shear and Torsion Design Forces

Applied Shear	Prestress Component	Concrete Strength	Stirrup * Strength	Shear Strength	Applied Torsion	Threshold Torsion	Notes & Warnings

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Summary Report

Concise Beam (TM), Version 4.46f, (c) 2006 Black Mint Software, Inc
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Project: Applied Research Laboratory V
Problem: 2nd Floor 12" HC Plank

x ft	Vu kip	Vp kip	ØVc kip	ØVs kip	ØVn kip	Tu kipft	ØTcr/4 kipft
0.00	0.0	0.0	-13.5	0.0	-13.5	0.0	15.4
0.17	-0.2	0.0	-24.6	0.0	-24.6	0.0	16.5
0.17	21.4	0.0	24.6	0.1	24.7	0.0	16.5
3.42	17.7	0.0	36.4	0.0	36.4	0.0	26.6
6.67	13.3	0.0	17.5	2.7	20.2	0.0	26.7
9.92	8.9	0.0	13.5	2.8	16.2	0.0	26.8
13.17	4.4	0.0	13.5	0.0	13.5	0.0	26.9
16.42	0.0	0.0	13.5	0.0	13.5	0.0	26.9
19.83	-4.7	0.0	-13.5	0.0	-13.5	0.0	26.9
23.08	-9.1	0.0	-13.5	-2.8	-16.2	0.0	26.8
26.33	-13.5	0.0	-18.0	-2.7	-20.7	0.0	26.7
29.58	-18.0	0.0	-37.6	0.0	-37.6	0.0	26.6
32.66	-21.4	0.0	-24.6	-0.1	-24.7	0.0	16.5
32.66	0.2	0.0	24.6	0.0	24.6	0.0	16.5
32.83	0.0	0.0	13.5	0.0	13.5	0.0	15.4

* Stirrup resistance based on required stirrup area.

Transverse Steel (Stirrup) Design for Shear

x ft	Required Shear Steel Total (Av+2At)/s in^2/ft	Required Shear Steel Torsion* At/s in^2/ft	Stirrup Provided Av+2At in^2	Stirrup Spacing Provided s in	Stirrup Spacing Required s in	Long. Torsion Steel, Al Total Required in^2	Long. Torsion Steel, Al Allowable Reduction** in^2	Notes & Warnings
0.00	0.00	0.00	N/A	N/A	10.50	0.00	0.00	
0.17	0.00	0.00	N/A	N/A	10.50	0.00	0.00	
0.17	0.00	0.00	N/A	N/A	10.50	0.00	0.00	2
3.42	0.00	0.00	N/A	N/A	10.50	0.00	0.00	
6.67	0.06	0.00	N/A	N/A	10.50	0.00	0.00	2
9.92	0.06	0.00	N/A	N/A	10.50	0.00	0.00	2
13.17	0.00	0.00	N/A	N/A	10.50	0.00	0.00	
16.42	0.00	0.00	N/A	N/A	10.50	0.00	0.00	
19.83	0.00	0.00	N/A	N/A	10.50	0.00	0.00	
23.08	0.06	0.00	N/A	N/A	10.50	0.00	0.00	2
26.33	0.06	0.00	N/A	N/A	10.50	0.00	0.00	2
29.58	0.00	0.00	N/A	N/A	10.50	0.00	0.00	
32.66	0.00	0.00	N/A	N/A	10.50	0.00	0.00	2
32.66	0.00	0.00	N/A	N/A	10.50	0.00	0.00	
32.83	0.00	0.00	N/A	N/A	10.50	0.00	0.00	

Notes & Warnings
2 - Note: Amount of shear steel required represents minimum requirements.
* Portion of the total stirrup area required to resist torsional shear flow (one leg around periphery).
** Allowable reduction in the additional longitudinal steel in the compression portion of the section.

HORIZONTAL INTERFACE SHEAR TRANSFER CHECK - BY SECTION

Design Code Used: ACI318-05

x ft	Horizontal Shear Force Vu kip	Hor. Shear Strength (no ties) kip	Required Ties Av/s in^2/ft	Tie Size Provided Av in^2	Spacing Required s in	Spacing Provided s in	Hor. Shear Strength ØVnh kip	Notes & Warnings
0.00	0.0	34.0	0.00	0.00	0.00	0.00	34.0	3
0.17	-0.2	-34.0	0.00	0.00	0.00	0.00	-34.0	3
0.17	21.4	34.0	0.00	0.00	0.00	0.00	34.0	3
3.42	17.7	34.0	0.00	0.00	0.00	0.00	34.0	3
6.67	13.3	34.0	0.00	0.00	0.00	0.00	34.0	3
9.92	8.9	34.0	0.00	0.00	0.00	0.00	34.0	3
13.17	4.4	34.0	0.00	0.00	0.00	0.00	34.0	3
16.42	0.0	34.0	0.00	0.00	0.00	0.00	34.0	3
19.83	-4.7	-34.0	0.00	0.00	0.00	0.00	-34.0	3
23.08	-9.1	-34.0	0.00	0.00	0.00	0.00	-34.0	3
26.33	-13.5	-34.0	0.00	0.00	0.00	0.00	-34.0	3
29.58	-18.0	-34.0	0.00	0.00	0.00	0.00	-34.0	3
32.66	-21.4	-34.0	0.00	0.00	0.00	0.00	-34.0	3
32.66	0.2	34.0	0.00	0.00	0.00	0.00	34.0	3
32.83	0.0	34.0	0.00	0.00	0.00	0.00	34.0	3

Notes & Warnings
3 - Note: No Ties Required

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Summary Report

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Project: Applied Research Laboratory V
Problem: 2nd Floor 12" HC Plank

* Shear resistance based on tie area provided.

HORIZONTAL INTERFACE SHEAR TRANSFER CHECK - BY MOMENT DEVELOPMENT REGION

Region of Beam x ft	Shear Length lv ft	Horizontal Shear Force Fh kip	Hor. Shear Resistance - No Ties kip	Total Tie * Area Req'd Acs in^2	Total Area Provided Acs in^2	Maximum Spacing s in	Notes & Warnings
0.00 to 0.17	0.17	0.0	5.7	No Ties Required			
0.09 to 16.42	16.33	204.5	544.0	No Ties Required			
16.42 to 32.66	16.25	204.5	544.0	No Ties Required			
32.66 to 32.66	0.00	0.0	0.0	No Ties Required			
32.66 to 32.83	0.17	0.0	5.7	No Ties Required			

* Required ties should be distributed in proportion to distribution of shear force (or stirrups).

MATERIAL PROPERTIES
PRESTRESSING STEEL

Design Aid 11.2.3 Properties and Design Strengths of Prestressing Strand and Wire

Seven-Wire Strand, f_{pu} = 270 ksi						
Nominal Diameter, in.	3/8	7/16	1/2	$\boxed{\text{1/2 Special}^a}$	9/16	3/5
Area, sq in.	0.085	0.115	0.153	0.167	0.192	0.217
Weight, plf	0.29	0.40	0.52	0.53	0.65	0.74
$0.7f_{pu}A_{ps}$, kips	16.1	21.7	28.9	31.6	36.3	41.0
$0.75f_{pu}A_{ps}$, kips	17.2	23.3	31.0	33.8	38.9	43.0
$0.8f_{pu}A_{ps}$, kips	18.4	24.8	33.0	36.1	41.5	46.9
$f_{pu}A_{ps}$, kips	23.0	31.1	41.3	45.1	51.8	58.6

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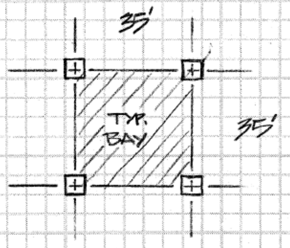
COMMENTS ALTERNATE: FLAT PLATE SYSTEM		Ckd By Date	Project TECH II

REF. ACI 318-05



FLAT PLATE SYSTEM
- NO BEAM
- NO DROP PANELS
- NO COLUMN CAPITALS

24" x 24" COLUMNS TYP.
$f'_c = 5000$ PSI (SLAB) $L_1 = L_2 = 35'$
$f_y = 60000$ PSI (STEEL)

DESIGN TYP. INTERIOR PANEL FOR 2^{ND} FLOOR OF ARL ~ BUILDING V
LL = 125 PSF DL = SELF WT. + 25 PSF (S.I.)

A. INITIAL SLAB THICKNESS (ACI 9.5.3.2)
TBL 9.5(c) ~ MIN. SLAB THK. (t) w/o INTERIOR BEAMS

$\left.\begin{array}{l} f_y = 60000 \text{ PSI} \\ \text{INT. PANEL} \end{array}\right\} t \geq \ell_n/33$ w: $\ell_n = 35' - 24" = 33'$ (396 IN)

$t_{MIN} \geq 396"/33 = 12$

$\underline{t_{MIN} = 12"}$ (SIM. TO ORIGINAL 12" HC)

B. TOTAL STATIC MOMENT (AT FACTORED LOAD)

$\omega_u = 1.2(\text{DEAD}) + 1.6(\text{LIVE})$
$= 1.2(175) + 1.6(125)$
$\omega_u = 410$ PSF

DEAD
SELF WT. = 150 PSF (t = 12")
S.I. = 25 PSF

LIVE
2^{ND} FL = 125 PSF

$$M_o = \frac{\omega_u L_2 L_n^2}{8}$$
$$= \frac{(0.410)(35)(33)^2}{8}$$

$M_o = 1953.4$ FT-K →

C. MOMENTS AT CRIT. SECTION (INT. SPANS)
$M_u^- = 0.65 M_o = 1269.7$ FT-K
$M_u^+ = 0.35 M_o = 683.7$ FT-K

D. LATERAL DISTRIBUTION OF MOMENTS ($\alpha = 0$, B/C NO BEAMS)
% TO COL. STRIP (INT. SPANS) → 75% M_u^- & 60% M_u^+

COL. STRIP → $M_u^- = 0.75(1269.7) = \underline{952.3 \text{ FT-K}}$ ‖ $M_u^+ = 0.60(683.7) = \underline{410.2 \text{ FT-K}}$

MID. STRIP → $M_u^- = 0.25(1269.7) = \underline{317.4 \text{ FT-K}}$ ‖ $M_u^+ = 0.40(683.7) = \underline{273.5 \text{ FT-K}}$

→

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(CONT.)

E. FLEXURAL REINF. FOR SLAB

		COL. STRIP −	COL. STRIP +	MID. STRIP −	MID. STRIP +
1. MOMENT (M_u)	FT-K	952.3	410.2	317.4	273.5
2. STRIP WIDTH (b)	IN	210	210	105	105
3. EFF. DEPTH (d_{eff})[a,c]	IN	10.25	10.25	10.25	10.25
4. $M_n = M_u/\phi$	FT-K	1058.1	455.8	352.7	303.9
5. $R = M_n/bd^2 \times 12000$	PSI	576	248	384	331
6. $(\rho)^b$		0.01036	0.004268	0.006722	0.00575
7. $A_s = \rho bd$	IN^2	22.3	9.2	7.2	6.2
8. $A_{s,MIN} = 0.0018bt$	IN^2	4.5 OK✓	4.5 OK✓	2.3 OK✓	2.3 OK✓
9. $N = A_s/A_{BAR}$[c]		29	12	10	8	★
10. $N_{MIN} = \frac{\text{STRIP WIDTH}}{2t}$		9	9	5	5

a. $d_{eff} = 12'' - \underset{CVR}{0.75''} - \underset{\#8}{1''} = 10.25$ IN

b. $\rho \rightarrow$ (REF. TABLE A.5a AT END OF THIS SECTION)

$R = 576$ PSI $\xrightarrow{\begin{matrix}558 \sim 0.0100 \\ 583 \sim 0.0105\end{matrix}}$ $\rho = 0.01036$

$R = 248$ PSI $\qquad \rho = 0.004268$
$R = 384$ PSI $\qquad \rho = 0.006722$
$R = 331$ PSI $\qquad \rho = 0.00575$

c. ASSUME #8 BARS ($A_{BAR} = 0.79$ IN²)

★ → N GOVERNS (ALL)

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F. SHEAR STRENGTH IN SLAB

$V_u = \omega_u A$

$= (0.410)\left[35'\left(\frac{35'}{2} - 1' - \frac{10.25}{12}\right)\right]$

$V_u = 224.5^k$

WIDE BEAM → $V_c = 2\sqrt{f'_c}\, b_w d$

$V_c = 2\sqrt{5000}\,(35' \cdot 12)(10.25)/1000$

$V_c = 608.8^k$ ($\phi = 0.75$)

$\phi V_c = 0.75(608.8) = 456.6$

$\phi V_c = 457^k$

▲ $\phi V_c = 457^k > 225^k = V_u$ → ∴ OK ✓

PUNCHING → ($B_c = 1.0$ FOR SQ. COL., $\alpha_s = 40$ FOR INT. COL.)

$$\frac{V_c}{\sqrt{f'_c}\, b_o d} = \begin{cases} 4 & \leftarrow \text{GOVERNS} \\ 2 + 4/B_c = 6 \\ \frac{\alpha_s d}{b_o} + 2 = \frac{(40)(10.25)}{4(24 + 2(10.25))} + 2 = 4.30 \end{cases}$$

$V_c = 4\sqrt{f'_c}\, b_o d = 4\sqrt{5000}\,(178)(10.25)/1000 = 516.047$

$V_c = 516^k$

$\phi V_c = 0.75(516) = 387.035$

$\phi V_c = 387^k$

▲ $\phi V_c = 387^k > 225^k = V_u$ → ∴ OK ✓

∴ SLAB DEPTH OF 12 IN OK FOR SHEAR.

G. BAR SPACING

- ALL BARS ARE #8's

MAX SPAC.

COL. STRIP → POSITIVE REINF. = 210"/29 BARS = 7 1/4" SPACING
NEGATIVE REINF. = 210/12 BARS = 17 1/2" SPACING

MID. STRIP → POSITIVE REINF = 105"/10 BARS = 10 1/2" SPACING
NEGATIVE REINF = 105"/8 BARS = 13 1/8" SPACING

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H. BAR DETAILING & CUT-OFF POINTS

MAX. BAR SPAC = 2t = 24" (S_{MAX} = 17.5 IN < 24 IN ∴ OK ✓)

1. COL. STRIP TOP REINF. → 50% CUT-OFF AT 0.30 L_n FROM FACE OF INT. SUPPORT
REMAINER CUT AT 0.20 Ln " " " "

#BARS = 29 (#8)

(15 BARS) 0.30 Ln = 0.30(396") = 118.8" (9'-10 13/16")
(14 BARS) 0.20 Ln = = 79.2" (6'-7 3/16")

COL. STRIP BTM. REINF. → ALL BARS MUST BE CONTINUOUS
#BARS = 12 (#8) MIN. OF 2 BARS MUST PASS THRU COL. CORE

2. MID. STRIP TOP REINF → 100% CUT-OFF AT 0.22 Ln

#BARS = 10 (#8)

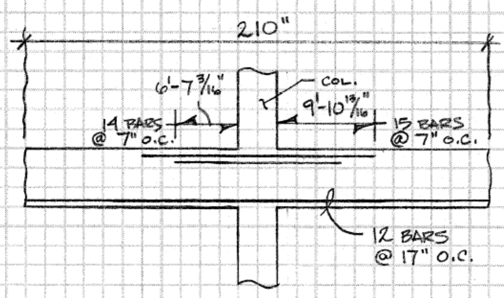
(10 BARS) 0.22 Ln = 0.22(396") = 87.12" (7'-3 1/8")

MID. STRIP BTM. REINF. → 50% LAPPED 6" @ ℄ OF SUPPORT
(8 BARS) REMAINDER CUT, LEAVING MAX. SPAC. OF 0.15 Ln

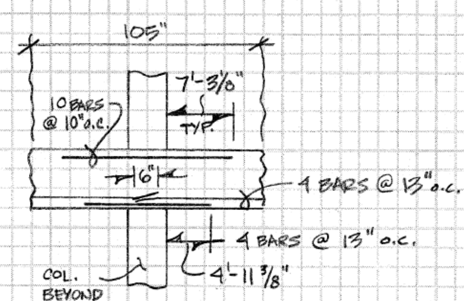
0.15 Ln = 0.15(396") = 59.4" (4'-11 3/8")

▲ REF. ACI 318, FIG. 13.3.8 AT END OF THIS SECTION

THE FOLLOWING DIAGRAMS ARE FOR EACH DIRECTION DUE TO $L_1 = L_2$.



COL. STRIP
N.T.S.

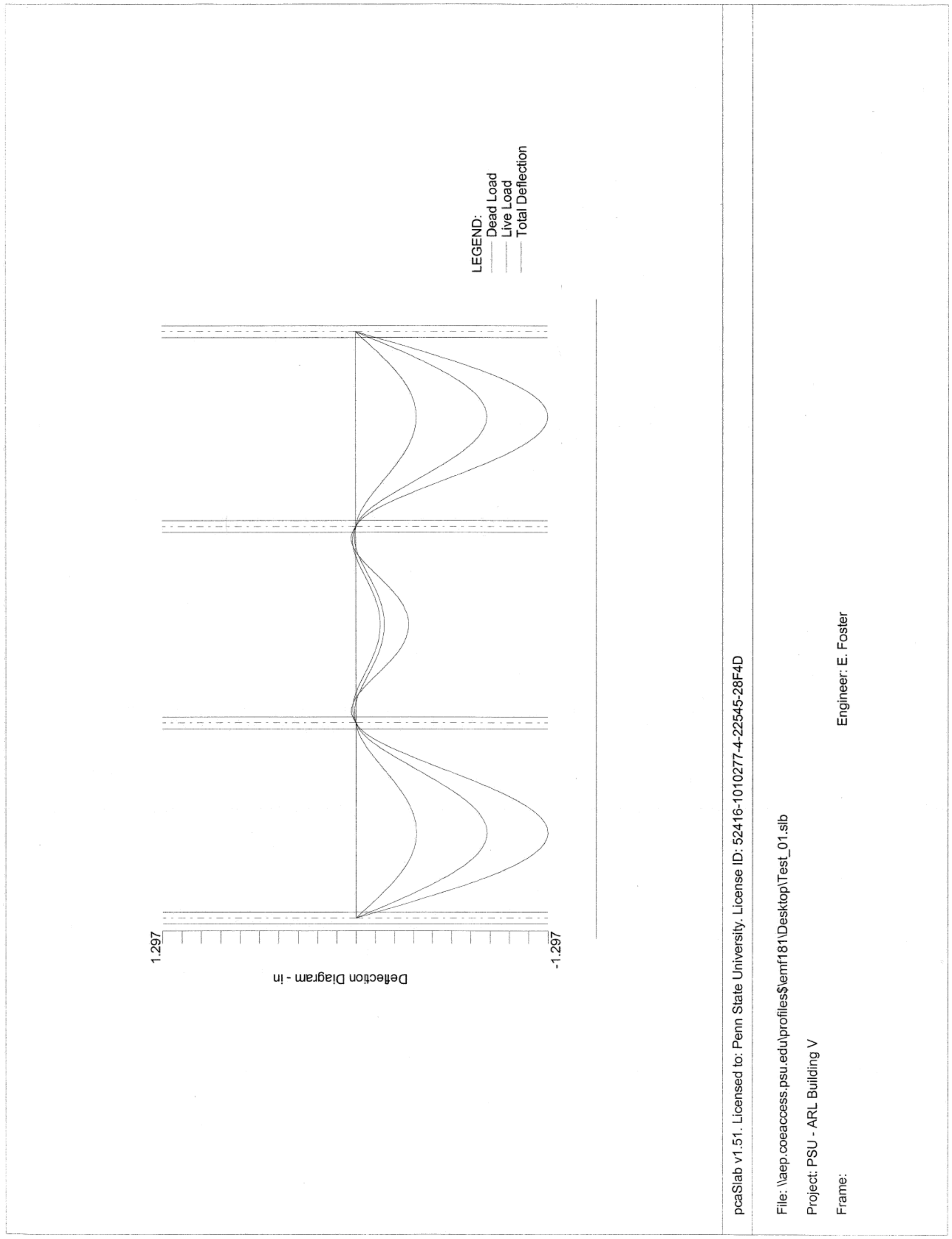


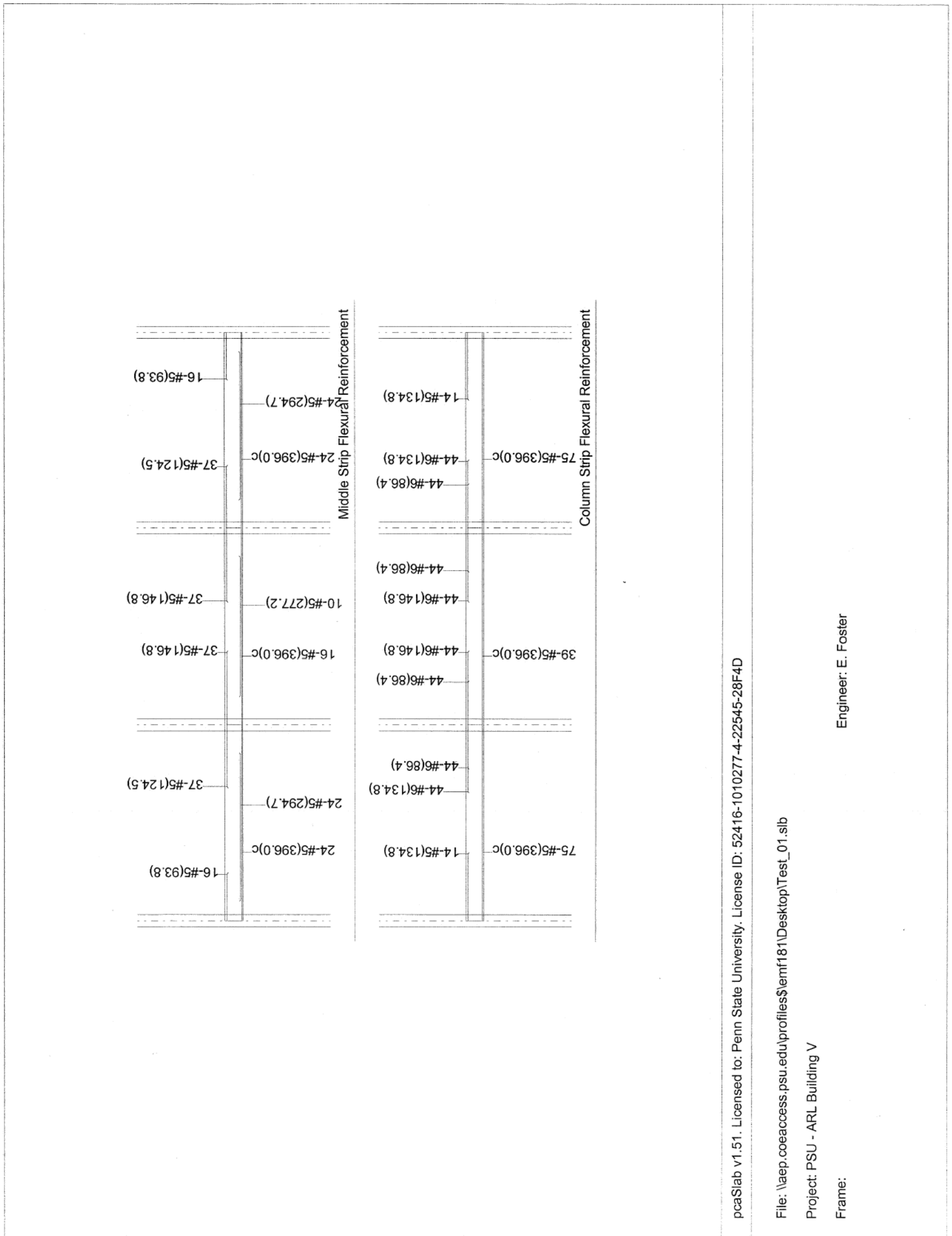
MID. STRIP
N.T.S.

TABLE A.5a
Flexural resistance factor: $R = \rho f_y \left(1 - 0.588 \frac{\rho f_y}{f'_c}\right)$ **psi**

	f_y = 40,000 psi				f_y = 60,000 psi			
	f'_c, psi				f'_c, psi			
ρ	3000	4000	5000	6000	3000	4000	5000	6000
0.0005	20	20	20	20	30	30	30	30
0.0010	40	40	40	40	59	59	60	60
0.0015	59	59	60	60	88	89	89	89
0.0020	79	79	79	79	117	118	118	119
0.0025	98	99	99	99	146	147	147	148
0.0030	117	118	118	119	174	175	176	177
0.0035	136	137	138	138	201	204	205	206
0.0040	155	156	157	157	229	232	233	234
0.0045	174	175	176	177	256	259	261	263
0.0050	192	194	195	196	282	287	289	291
0.0055	211	213	214	215	309	314	317	319
0.0060	229	232	233	234	335	341	345	347
0.0065	247	250	252	253	360	368	372	375
0.0070	265	268	271	272	385	394	399	403
0.0075	282	287	289	291	410	420	426	430
0.0080	300	305	308	310	435	446	453	457
0.0085	317	323	326	329	459	472	479	485
0.0090	335	341	345	347	483	497	506	511
0.0095	352	359	363	366	506	522	532	538
0.0100	369	376	381	384	529	547	558	565
0.0105	385	394	399	403	552	572	583	591
0.0110	402	412	417	421	575	596	609	617
0.0115	419	429	435	439	597	620	634	643
0.0120	435	446	453	457	618	644	659	669
0.0125	451	463	471	476	640	667	684	695
0.0130	467	480	488	494	661	691	708	720
0.0135	483	497	506	511	681	714	733	746
0.0140	499	514	523	529	702	736	757	771
0.0145	514	531	540	547	722	759	781	796
0.0150	529	547	558	565	741	781	805	821
0.0155	545	563	575	582	760	803	828	845
0.0160	560	580	592	600		825	852	870
0.0165	575	596	609	617		846	875	894
0.0170	589	612	626	635		867	898	918
0.0175	604	628	642	652		888	920	942
0.0180	618	644	659	669		909	943	966
0.0185	633	660	676	686		929	965	989
0.0190	647	675	692	703		949	987	1013
0.0195	661	691	708	720		969	1009	1036
0.0200	675	706	725	737		988	1031	1059

REF. ~ NILSON, DARWIN, DOLAN. DESIGN OF CONCRETE STRUCTURES. 13TH Ed.


LEGEND:
Dead Load
Live Load
Total Deflection
1.297
-1.297
Deflection Diagram - in
pcaSlab v1.51. Licensed to: Penn State University. License ID: 52416-1010277-4-22545-28F4D
File: \\aep.coeaccess.psu.edu\profiles$\emf181\Desktop\Test_01.slb
Project: PSU - ARL Building V
Frame:
Engineer: E. Foster


Middle Strip Flexural Reinforcement
16-#5(93.8)
24-#5(294.7)
37-#5(124.5)
24-#5(396.0)c
37-#5(146.8)
10-#5(277.2)
37-#5(146.8)
16-#5(396.0)c
37-#5(124.5)
24-#5(294.7)
24-#5(396.0)c
16-#5(93.8)
Column Strip Flexural Reinforcement
14-#5(134.8)
44-#6(134.8)
75-#5(396.0)c
44-#6(86.4)
44-#6(86.4)
44-#6(146.8)
44-#6(146.8)
39-#5(396.0)c
44-#6(86.4)
44-#6(86.4)
44-#6(134.8)
14-#5(134.8)
75-#5(396.0)c
pcaSlab v1.51. Licensed to: Penn State University. License ID: 52416-1010277-4-22545-28F4D
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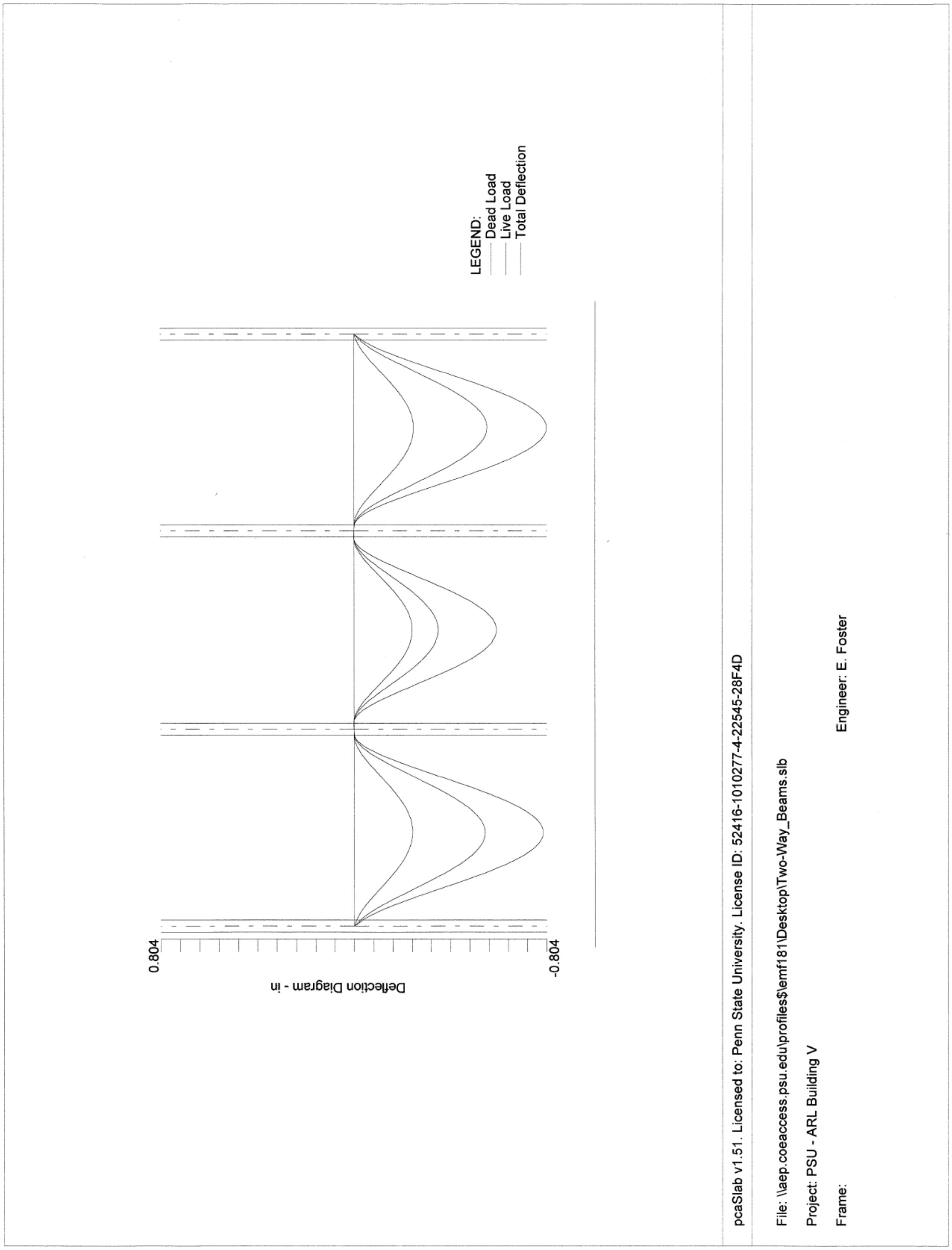
LEGEND:
Dead Load
Live Load
Total Deflection
0.804
-0.804
Deflection Diagram - in
pcaSlab v1.51. Licensed to: Penn State University. License ID: 52416-1010277-4-22545-28F4D
File: \\aep.coeaccess.psu.edu\profiles$\emf181\Desktop\Two-Way_Beams.slb
Project: PSU - ARL Building V
Frame:
Engineer: E. Foster

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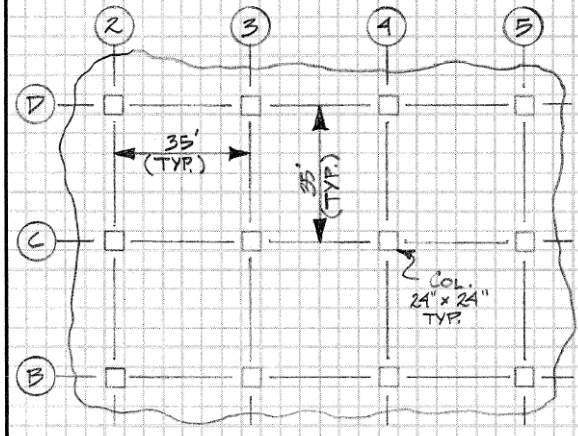
(REF: ACI 318-05 ~ IBC 2006)

TWO-WAY PT FLOOR DESIGN FOR 2ND FLOOR OF ARL'S BUILDING V

A. LOADS →

DEAD: SELF WT. (SLAB)
25 PSF (SI)
13 PSF (ADD'L. 1" NWC DURING CONSTR.)

LIVE: 125 PSF (2ND FLOOR ~ SEE TECH I)



B. MATERIALS →

- $f'_c = 5000$ PSI (NWC ≃ 150 PCF)
  $f'_{ci} = 3500$ PSI
- $f_y = 60$ KSI (REBAR)
- 0.6" ⌀, 7-WIRE STRANDS
  ↳ $A = 0.217$ IN^2 ↳ $f_{pu} = 270$ KSI
  (UNBONDED)

ASSUME PRESTRESS LOSSES APPROX. 13 KSI (ACI 18.6)

(ACI 18.5.1C) $0.7 f_{pu} = 189$ KSI

$f_{se} = 189 - 13 = 176$ KSI

$P_{eff} = f_{se} A = (176 \text{ KSI})(0.217 \text{ IN}^2) = 38.192^K$

$P_{eff} = 38.1^K$/STRAND

C. SLAB → (ASSUME $L/h = 45$)

- ALL COLUMN TO COLUMN DIM.'S = 35'-0

$\frac{35(12)}{45} = 9.33$ → TRY $\boxed{h = 10"}$ SLAB THK.

D. LOADING →

DEAD: S.W. = $(10"/12) \cdot 150$ PCF = 125 PSF
SI = 25 PSF
+1" NWC = 13 PSF
163 PSF = DL

LIVE: 125 PSF
125 PSF = LL
↳ LL > 100 PSF
∴ NO LIVE LOAD REDUCTION PERMITTED

(ACI 13.7.6) $\frac{LL}{DL} = 0.767$ ∴ BECAUSE $\frac{LL}{DL} > 0.75$, PATTERN LOADING MUST BE CONSIDERED! HOWEVER, TECH II NEGLECTS THESE CONSIDERATIONS FOR SIMPLICITY DUE THE RANGE OF ERROR (i.e. 0.767 ≈ 0.75) →

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DESIGN OF INTERIOR FRAME (SAME IN BOTH DIRECTIONS)

E. SECTION PROPERTIES →

- PER ACI 18.3.3: TWO-WAY SLABS ⇒ CLASS U ($f_t \leq 6\sqrt{f'_c}$)
- PER ACI 18.3.4: CLASS U ⇒ UNCRACKED (GROSS) SECTION PROP. ALLOWED

$A = bh = 35'(12)\cdot(10") = 4200 \text{ in}^2$ $\qquad$ $\underline{A = 4200 \text{ in}^2}$

$S = \frac{bh^2}{6} = \frac{(420")(10")^2}{6} = 7000 \text{ in}^3$ $\qquad$ $\underline{S = 7000 \text{ in}^3}$

F. ALLOWABLE STRESSES → $\qquad$ C → COMPRESSION $\quad$ T → TENSION

(ACI 18.3.3) CLASS U:

1. AT JACKING (ACI 18.4.1)
   $f'_{ci} = 3500$ PSI ⇝ $C \leq 0.60 f'_{ci} = 2100$ PSI
   $T \leq 3\sqrt{f'_{ci}} = 177$ PSI
2. AT SERVICE LOADS (ACI 18.4.2(a) & 18.3.3)
   $f'_c = 5000$ PSI ⇝ $C \leq 0.45 f'_c = 2250$ PSI
   $T \leq 6\sqrt{f'_c} = 424$ PSI

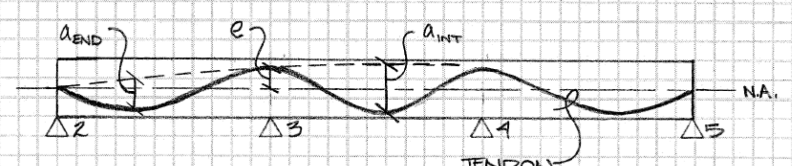
(PER ACI 18.12.4)
AVG. PRECOMPRESSION LIMITS: $\left[125 \text{ psi} \leq P/A \leq 300 \text{ psi}\right]$

ASSUME LOAD BALANCE TARGET = 70% OF SLAB SELF WT.
$0.70(125 \text{ psf}) = 87.5$ PSF

ASSUME 2-HR FIRE RATING w/ CARBONATE AGGREGATE
COVER ≥ 3/4" MIN / 1½ MAX. (SEE IBC 2006 ~ CH. 7)

G. TENDON PROFILE →

- ASSUME PARABOLIC SHAPE: (PROVIDE MAX. DRAPE FOR LOAD BALANCING)



* ECCENTRICITY (e) VARIES ALONG SPAN.

→

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USING TENDON PROFILE (PREV. PG.)

LOCATION	TENDON (CG) LOCATION* (CENTER OF GRAVITY)
EXT. SUPPORT - ANCHOR (10"/2)	5"
INT. SUPPORT - TOP (10" - 3/4" - 0.6"/2)	8.95"
INT. SPAN - BOTTOM (3/4" + 0.6"/2)	1.05"
EXT. SPAN - BOTTOM (1 1/2 + 0.6"/2)	1.8"

* - MEASURED FROM BTM. OF SLAB

$$a_{END} = \frac{(5" + 8.95")}{2} - 1.8" = 5.175"$$

$$a_{INT} = 8.95" - 1.05" = 7.9"$$

H. FORCE REQUIRED TO BALANCE 70% SLAB SELF WT.

- LESS DRAPE IN EXT. SPAN USUALLY GOVERNS (ASSUME TRUE.)

$$\omega_{BL} = 87.5 \text{ PSF} \times 35 \text{ FT}/_{1000} = 3.0625 \text{ KLF}$$

$$\omega_{BL} = 3.06 \text{ K/FT}$$

$$\omega_{BL} = \frac{8Fa}{L^2} \Rightarrow F_E = \frac{\omega_{BL} L^2}{8a}$$ = FORCE NEEDED TO COUNTERACT ω_{BL} IN EXT. SPAN

$$F_E = \frac{(3.06)(35')^2}{8\left(5.175"/_{12}\right)} = 1087.41^K$$

$$F = 1088 \text{ KIPS}$$

I. ✓ PRECOMPRESSION ALLOWANCES →

- NUMBER TENDONS REQ'D. FOR 1088^K

$$\frac{1088 \text{ KIPS}}{38.1 \text{ K/TENDON}} = 28.556 \rightarrow \text{USE } 28 \text{ TENDONS}$$

- $P_{ACTUAL} = 28(38.1) = 1066.8^K$ $\quad P_{ACT} = 1066^K$

$$\omega_{BL'} = \left(\frac{1066}{1088}\right)(3.06) = 3.00 \text{ K/FT} \quad \omega_{BL'} = 3.0 \text{ K/FT}$$

ACTUAL PC STRESS $\frac{P}{A} = \frac{1066^K \cdot 1000}{4200 \text{ IN}^2} = 253.81^K$

$$[125 \le 254 \le 300] \therefore \text{OK} ✓$$

J. ✓ INTERIOR SPAN FORCE →

- $F_I = \frac{3.06(35')^2}{8\left(7.9"/_{12}\right)} = 712.322^K < 1066^K$

- $\omega_{BL} = \frac{8(1066^K)(7.9"/_{12})}{(35')^2} = 4.583 \text{ K/FT}$

$$\frac{\omega_{BL}}{\omega_{DL}} = \frac{4.583 \text{ K/FT}}{0.125 \text{ KSF}(35')}(\times 100) = 105\% \Rightarrow$$

BECAUSE GREATER THAN 100%, MUST RE-ANALYZE. FOR THE PURPOSE OF THIS REPORT I WILL ASSUME ENGINEERING JUDGEMENT AND ASSUME FULL LOADING NEVER HAPPENS DUE TO FACILITY USE.

THUS ⇐ $P_{eff} = 1066^K$

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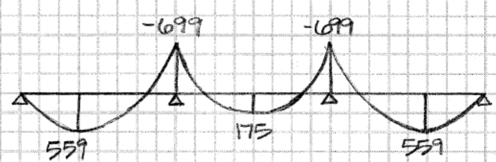
K. SLAB STRESSES →

- DEAD LOAD MOMENTS $\left[\omega_{DL} = \frac{163(35')}{1000} = 5.705 \text{ KLF} \right]$

$\omega L^2 = 6988.63$ K-FT

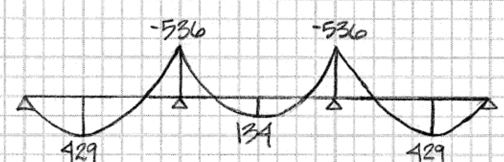
$0.08\,\omega L^2 = 559.09$
$-0.10\,\omega L^2 = 698.86$
$0.025\,\omega L^2 = 174.716$



M (K-FT)

- LIVE LOAD MOMENTS $\left[\omega_{LL} = \frac{125(35)}{1000} = 4.375 \text{ KLF} \right]$

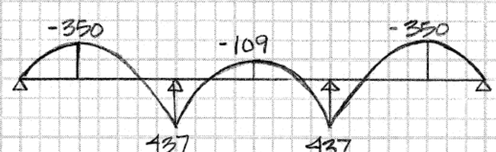
$\omega L^2 = 5359.38$ K-FT



M (K-FT)

- BALANCING MOMENTS $\left[\omega_B = -3.569 \text{ K/FT} \right]$

$-\omega L^2 = -4372.43$



M (K-FT)

L. STRESS STAGES →

- STAGE 1 – IMMEDIATELY AFTER JACKING (ACI 18.4.1)

@ MIDSPAN

$$f_{TOP} = \frac{(-M_{DL} + M_{BAL})}{S} - \frac{P}{A} \qquad f_{BTM} = \frac{(+M_{DL} - M_{BAL})}{S} - \frac{P}{A}$$

@ SUPPORTS

$$f_{TOP} = \frac{(+M_{DL} - M_{BAL})}{S} - \frac{P}{A} \qquad f_{BTM} = \frac{(-M_{DL} + M_{BAL})}{S} - \frac{P}{A}$$

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• STAGE 2 – AT SERVICE LOADS (ACI 18.3.3 & 18.4.2)

@ MIDSPAN

$$f_{TOP} = \frac{(-M_{DL} - M_{LL} + M_{BAL})}{S} - \frac{P}{A} \qquad f_{BTM} = \frac{(M_{DL} + M_{LL} - M_{BAL})}{S} - \frac{P}{A}$$

@ SUPPORTS

$$f_{TOP} = \frac{(M_{DL} + M_{LL} - M_{BAL})}{S} - \frac{P}{A} \qquad f_{BTM} = \frac{(-M_{DL} - M_{LL} + M_{BAL})}{S} - \frac{P}{A}$$

SUMMARY OF STRESSES:

STAGE 1:

MIDSPAN
• INT. SPAN → $f_{TOP} = -367$ PSI $f_{BTM} = -141$ PSI
• EXT. SPAN → $f_{TOP} = -612$ PSI $f_{BTM} = 104$ PSI

SUPPORTS
$f_{TOP} = 195$ PSI $f_{BTM} = -703$ PSI

STAGE 2:

MIDSPAN
• INT. SPAN → $f_{TOP} = -597$ PSI $f_{BTM} = 89$ PSI
• EXT. SPAN → $f_{TOP} = -1348$ PSI $f_{BTM} = 840$ PSI

SUPPORTS
$f_{TOP} = 1114$ PSI $f_{BTM} = -1622$ PSI

ALLOW:

		M	S
S1.	$C \leq 2100$ PSI	> -367 / -612 / -141 OK ✓	> -703 OK ✓
	$T \leq 177$ PSI	> 104 OK ✓	< 195 NG!
S2.	$C \leq 2250$ PSI	> -597 / -1348 OK ✓	> -1622 OK ✓
	$T \leq 424$ PSI	> 89 OK ✓ < 840 NG!	< 1141 NG!

* NOTE: BECAUSE STRESSES EXCEED ALLOWABLE STRESSES, CALC'S ABANDONED AND USE OF DROP PANELS OR COLUMN CAPITALS SHOULD BE CONSIDERED.

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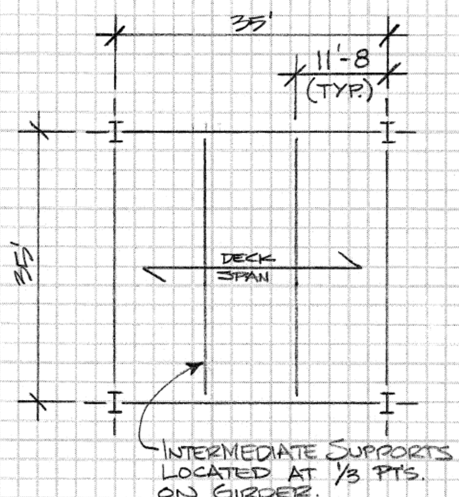
COMMENTS
ALTERNATE: COMPOSITE BEAMS W/ METAL DECK

(REF: AISC STEEL MANUEL, 13TH ED.)

2ND FLOOR
TYPICAL INTERIOR BAY DESIGN (35'×35')



ASSUME → UNSHORED CONSTRUCTION
$f'_c = 3000$ PSI OF N.W.C. (150 #/FT³)
A992 STEEL BEAMS & GIRDERS ($F_y = 50$ KSI)
3/4"ø × 5" STUDS ($F_u = 65$ KSI)

USING USD CATALOGUE SHEETS INCLUDED AT THE END OF THIS SECTION:
→ 3" LOK FLOOR BY USD, PLUS A 4½" N.W.C. COVER YIELD A 7½" TOTAL SLAB THICKNESS & A 2-HR. FIRE RATING, REQUIRING NO ADDITIONAL FIRE-PROOFING. (t = 7.5")

⇒ MAX. UNSHORED SPAN (MS) = 11.67 FT. (3 SPAN)

3" LOK FLOOR → 7.5" SLAB DEPTH → MS = 11.67' (3 SPAN)

⇒ SELECT 16 GAGE DECKING
→ MS = 12.44' > 11.67' ∴ OK ✓
→ MAX. UNIF. LL (12' SPAN) = 310 PSF
ACTUAL = 1.6(125) = 200 PSF < 310 ∴ OK ✓

BEAM DESIGN

2ND FLOOR LOADING →

→ LIVE: 125 PSF × 11.67'/1000 = 1.4583 → LL = 1.5 KLF

→ DEAD: 25 PSF (SUPERIMPOSED)
76.5 PSF (DECK + SLAB)
13 PSF (ADD'L. 1" OF CONCRETE DURING CONSTRUCTION LEVELING)
Σ: 114.5 PSF × 11.67'/1000 = 1.33583 KLF
+ 0.060 KLF (BM. SELF WT. ASSUMED DUE TO HIGH LOADS)
1.39583 KLF

DL = 1.4 KLF

FACTORED → $1.2(1.4) + 1.6(1.5) = \omega_u$

$\omega_u = 4.08$ KLF

$$M_u = \frac{\omega L^2}{8} = \frac{(4.08\ \text{KLF})(35')^2}{8} = 624.75\ '^{K} \rightsquigarrow M_u = 625\ \text{FT-K}$$

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IT HAS BEEN DETERMINED THRU PRIOR DESIGN TRIALS THAT DEFLECTION IS THE CONTROLLING FACTOR, THUS

TRY → **W21 × 55**

1. CHECK FLEXURAL STRENGTH BEFORE CONCRETE CURES (NO LL / NO SIDL)

$\omega_u = 1.2(1.04 + 0.055) = 1.319$

$\omega_u = 1.32$ KLF $\rightsquigarrow$ $M_u = \dfrac{(1.32)(35')^2}{8} = 202.125'^k$

$M_u = 202$ FT-K

(W21×55)
$\phi M_p = 473$ FT-K $>$ 202 FT-K

∴ OK ✓

2. AFTER CONCRETE CURES

$\omega_u = 1.2(1.34 + 0.055) + 1.6(1.5) = 4.069$

$\omega_u = 4.07$ KLF $\rightsquigarrow$ $M_u = \dfrac{(4.07)(35')^2}{8} = 623.066'^k$

$\boxed{M_u = 623 \text{ FT-K}}$

$b_{eff} = \min \left| \begin{array}{l} \frac{1}{4}\text{ SPAN} = \frac{35(12)}{4} = 105'' \\ \text{BM. SPAC.} = 11.67'(12) = 140'' \end{array} \right.$ $\Rightarrow$ $b_{eff} = 105''$

FOR FULL COMPOSITE

$C = \min \left| \begin{array}{l} A_sF_y = 16.2(50) = 810^k \\ 0.85 f'_c A_c = 0.85(3)[105''(7.5-3)] = 1205^k \end{array} \right.$ $\Rightarrow$ $C = 810^k$

$a = \dfrac{C}{0.85 f'_c b} = \dfrac{810^k}{0.85(3)(105)} = 3.025''$

$y = \dfrac{d}{2} + t - \dfrac{a}{2} = \dfrac{20.75''}{2} + 7.5'' - \dfrac{3.025''}{2} = 16.3624''$

DESIGN STRENGTH IS THEN

$\phi M_n = 0.90(810^k)(16.36''/12) = 994.016'^k$

$\phi M_n = 994$ FT-K $>$ 623 FT-K $= M_u$ ∴ OK ✓

3. CHECK SHEAR

$V_u = \dfrac{\omega L}{2} = \dfrac{4.07(35')}{2} = 71.225^k$

$V_u = 71.2^k < 234^k = \phi V_n$ ∴ OK ✓

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SHEAR CONNECTORS

$C = V_h = 810^k$

$\text{MAX DIA.} = 2t_f = 2(0.522'') = 1.044''$ OR $\frac{3}{4}''$

$$\text{REDUCTION FACTOR} = \frac{0.85}{\sqrt{N_r}}\left(\frac{W_r}{h_r}\right)\left[\left(\frac{H_s}{h_r}\right) - 1.0\right] \leq 1.0$$

↳ ASSUME: $N_r = 1.0$ (ONE STUD PER SECTION)
$W_r = 6.0$ IN (RIB WIDTH)
$h_r = 3.0$ IN (DECK HEIGHT)
$H_s = 5.0$ IN (STUD LENGTH)

$$= \frac{0.85}{\sqrt{1.0}}\left(\frac{6.0}{3.0}\right)\left[\left(\frac{5.0}{3.0}\right) - 1.0\right]$$

$= 1.1\overline{3} > 1.0$ → ∴ NO REDUCTION

$$Q_n = \min \begin{cases} 0.5 A_{sc}\sqrt{f'_c E_c} = 0.5(0.4418)\sqrt{3(3321)} = 22.047'' \\ A_{sc} F_u = 0.4418(65) = 28.7161^k \end{cases}$$

$Q_n = 22.05^k$

STUDS REQ'D. FOR ½ SPAN

$$N_1 = \frac{V_h}{Q_n} = \frac{810^k}{22.05^k} = 36.7398$$ → 38 STUDS FOR HALF BEAM (76 TOTAL)

PARTIAL COMPOSITE MAY BE ADEQUATE DUE TO EXCESS FLEXURAL STRENGTH. (939 & 623)

TRY 48 STUDS TOTAL ⇒ $N_1 = \frac{48}{2} = 24$

$\Sigma Q_n = 24(22.05^k) = 529.128^k < 810^k$ & ∴ $C = V_h = 529^k$

$C < A_s F_y$ → FIND P.N.A.

$P_{yf} = b_f t_f F_y = (8.22)(0.522)(50) = 214.542$

$T - C_s = 810^k - 2(214.5^k) = 380.916^k < 529^k$ SO TOP FLANGE NOT ENTIRELY IN COMPRESSION

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FIND t' → DEPTH OF COMPRESSION IN FLANGE

$$810^k - 2[8.22(t')(50)] = 529^k$$

$$t' = 0.341693''$$

$$a = \frac{C}{0.85 f'_c b} = \frac{529^k}{0.85(3)(105)}$$

$$a = 1.9762''$$

FIND $\bar{y}$ →

	A	Y	Ay
W21 × 55	16.2 IN	20.75/2 = 10.375	168.075
FLANGE	−0.3417(8.22) = −2.80872	0.3417/2 = 0.170847	−0.479861
	13.3913		167.595

$$\bar{y} = \frac{168}{13.4} = 12.5152''$$

$$\bar{y} = 12.52''$$

MOMENT ARM FOR CONCRETE COMPRESSIVE FORCE

$$= \bar{y} + t - \frac{a}{2} = 12.52 + 7.5 - \frac{1.98}{2} = 19.0271''$$

MOMENT ARM FOR STEEL COMPRESSIVE FORCE

$$= \bar{y} - \frac{t'}{2} = 12.52'' - \frac{0.3417''}{2} = 12.3\bar{4}''$$

NOMINAL MOMENT

$$M_n = C(19.03'') + C_s(12.3\bar{4}'')$$

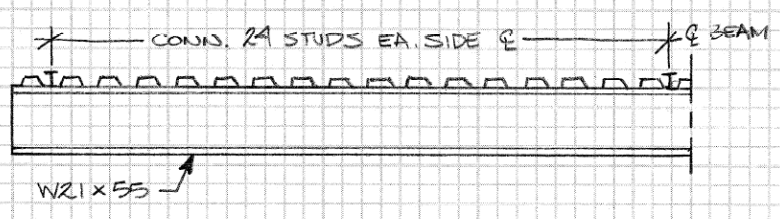
$$= 529(19.03'') + 0.3417(8.22)(50)(12.3\bar{4}'') = 11801.4 \text{ IN-K}$$

$$983.449 \text{ FT-K}$$

DESIGN MOMENT

$$\phi M_n = 0.9(983.45) = 885.104^k > 625^{1k} \therefore \text{OK}$$

24 STUDS ON EA. SIDE OF ℄ OK



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(CONT.)

W21 × 55 : $I = 1140 \text{ in}^3$

$\frac{L}{360} = \frac{35(12)}{360} = 1.17''$ (LIVE)

$\frac{L}{240} = 1.75''$ (D+L)

DEFLECTION(S)

1. BEFORE CONCRETE CURES

$$\omega_D = 1.04 + 0.055 = 1.099 / 1 \text{ KLF}$$

$$\Delta_1 = \frac{5\omega_D L^4}{384 EI_s} = \frac{5(1.10)(35')^4 \cdot 1728}{384(29000)(1140)} = 1.12''$$

2. AFTER CONCRETE CURES

NEED TWO TRANSFORMED SECTIONS

$I_{tr} \Rightarrow$ TRANSF. SLAB WIDTH $= b/n$ & $I_{tr} \Rightarrow$ TRANSF. SLAB WIDTH $= b/2n$

$$n = \frac{E_s}{E_c} = \frac{29000}{3321} = 8.73 \rightarrow \text{USE } 8$$

$$\frac{b}{n} = \frac{105''}{8} = 13.125 \text{ IN.}$$

$$I_{eff} = I_s + \sqrt{\Sigma Q_n / C_f}\,(I_{tr} - I_s)$$

$$= 1140 + \sqrt{529/810}\,(4343 - 1140) = 3728.81$$

$\Rightarrow I_{eff} = 3729 \text{ in}^4$

LIVE LOAD DEFLECTION

$$\Delta_2 = \frac{5 w_L L^4}{384 E I_{eff}} = \frac{5(1.5)(35')^4 \cdot 1728}{384(29000)(3729)} = 0.47''$$

DEFL. DUE TO DEAD LOAD APPLIED AFTER CONCRETE CURES

$$\frac{b}{2n} = \frac{105''}{2(8)} = 6.5625$$

$$I'_{eff} = I_s + \sqrt{\Sigma Q_n / C_f}\,(I'_{tr} - I_s)$$

$$= 1140 + \sqrt{529/810}\,(3743.85 - 1140) = 3244.53$$

$\Rightarrow I'_{eff} = 3245 \text{ in}^4$

DEAD LOAD DEFLECTION

$$\Delta_3 = \frac{5(0.29 \text{ KLF})(35')^4 \cdot 1728}{384(29000)(3245)} = 0.105''$$

TOTAL DEFLECTION $= \Delta_1 + \Delta_2 + \Delta_3 = 1.12'' + 0.47'' + 0.11'' = 1.695''$

$$\Delta_T = 1.70'' < 1.75'' = \Delta_{D+L}$$

∴ OK ✓

Civilsmith Engineering, Inc.
2160 Sandy Drive, Suite C, State College, PA 16803
Phone: (814) 867-9150
Fax: (814) 867-9151

By FOSTER
Date ______
Page ____ of ____
Ckd By ______
Date ______
Project TECH II

COMMENTS
ALTERNATE: COMPOSITE BEAMS W/ METAL DECK 6

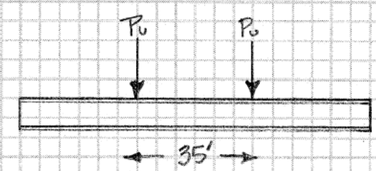
(CONT.)

GIRDER DESIGN (COMPOSITE)

3" M. DECK (PARALLEL (//) TO GIRDER)
4.5" SLAB (f'_c = 3000 PSI)
P_u @ 1/3 PTS

LOADING → LL = 125 PSF

DL = 25 PSF (SUPERIMPOSED)
55 PLF (W21 × 55)
76.5 PSF (DECK + SLAB)
13 PSF (ADD'TL 1" NWC)



BM TRIB = 11.67'

LIVE: $125\text{ PSF}(35' \times 11.67')/1000 = 51.0417$

LIVE = 51^k

DEAD: $(25+76.5+13)(35' \times 11.67')/1000 = 46.7542$
$55\text{ PLF} \times 35'/1000 = 1.925$
} 48.6792

DEAD = 49^k

$P_u = 1.2(49^k) + 1.6(51^k) = 140.082^k$

$\boxed{P_u = 140^k}$

$M_u = Pa = 140^k(11'-8") = 1634.29$ FT-K

$\boxed{M_u = 1635^{1k}}$

← REF. AISC STEEL MANUEL 13TH ED., TBL 3-23 FOR DETERMINATION OF MAX M.

(AISC) TBL 3-19

ASSUME $a = 2" \Rightarrow Y2 = 7.5" - 2"/2 = 6.5"$

TRY → W30 × 90 @ LOCATION 1 (TFL)
- M_u = 2100 FT-K
- ΣQ_n = 1320 KIPS

* CHOSE W30 TO TRY & LIMIT DEFLECTION.

$a = \frac{1320}{0.85(3)(35 \cdot 12)} = 1.23249" < 2"$ ∴ OK ✓

$\frac{1320}{22.05} = 59.86$ → 60 STUDS PER HALF SPAN (120 TOTAL)

USE → W30 × 90 W/ (120) 3/4" φ × 5" LONG

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By FOSTER
Date
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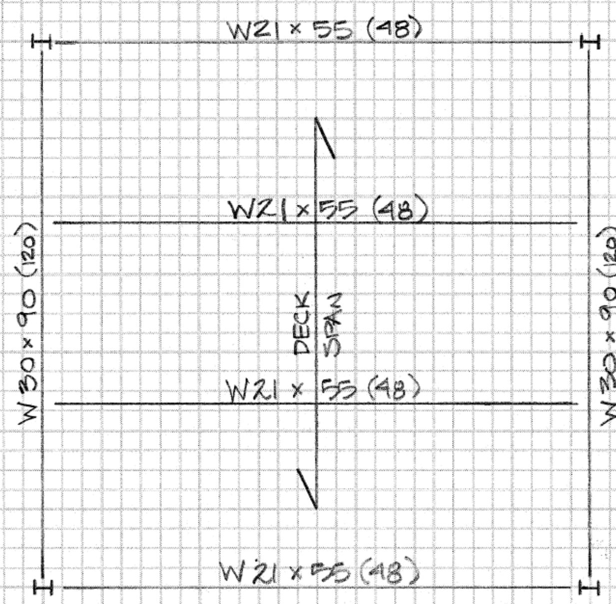
COMMENTS ALTERNATE: COMPOSITE BEAM W/ METAL DECK[7]

Ckd By
Date
Project TECH II

(CONT.)

SUMMARY → TYPICAL 35' × 35' INTERIOR BAY



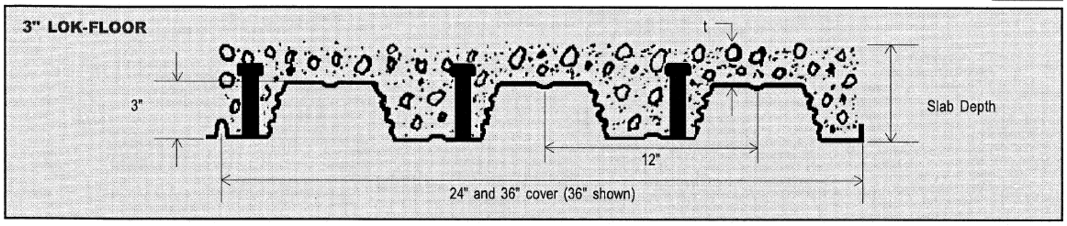
DECK → 3" USD LOK FLOOR W/ 4½" SLAB (f'_c = 3000 PSI)

END OF SECTION

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

USD United Steel Deck, Inc.

LRFD



The **Deck Section Properties** are per foot of width. The **I** value is for positive bending (in.[4]); **t** is the gage thickness in inches; **w** is the weight in pounds per square foot; S_p and S_n are the section moduli for positive and negative bending (in.[3]); R_b and ϕV_n are the interior reaction and the shear in pounds (per foot of width); studs is the number of studs required per foot in order to obtain the full resisting moment, ϕM_{nf}.

DECK PROPERTIES									
Gage	t	w	As	I	S_p	S_n	R_b	ϕV_n	studs
22	0.0295	1.7	0.505	0.797	0.454	0.500	718	2190	0.41
20	0.0358	2.1	0.610	0.993	0.583	0.620	1020	3220	0.50
19	0.0418	2.4	0.710	1.158	0.708	0.726	1350	4310	0.58
18	0.0474	2.8	0.810	1.324	0.832	0.832	1720	4880	0.66
16	0.0598	3.5	1.020	1.666	1.045	1.045	2540	6130	0.83

The **Composite Properties** are a list of values for the composite slab. The **slab depth** is the distance from the bottom of the steel deck to the top of the slab in inches as shown on the sketch. U.L. ratings generally refer to the cover over the top of the deck so it is important to be aware of the difference in names. ϕM_{nf} is the factored resisting moment provided by the composite slab when the "full" number of studs as shown in the upper table are in place; inch kips (per foot of width). A_c is the area of concrete available to resist shear, in.[2] per foot of width. **Vol.** is the volume of concrete in ft.[3] per ft.[2] needed to make up the slab; no allowance for frame or deck deflection is included. **W** is the concrete weight in pounds per ft.[2]. S_c is the section modulus of the "cracked" concrete composite slab; in.[3] per foot of width. I_{av} is the average of the "cracked" and "uncracked" moments of inertia of the transformed composite slab; in.[4] per foot of width. The I_{av} transformed section analysis is based on steel; therefore, to calculate deflections the appropriate modulus of elasticity to use is 29.5 x 10^6 psi. ϕM_{no} is the factored resisting moment of the composite slab if there are no studs on the beams (the deck is attached to the beams or walls on which it is resting) inch kips (per foot of width). ϕV_{nt} is the factored vertical shear resistance of the composite system; it is the sum of the shear resistances of the steel deck and the concrete but is not allowed to exceed $\phi 4(f'_c)^{1/2} A_c$; pounds (per foot of width). The next three columns list the **maximum unshored spans** in feet; these values are obtained by using the construction loading requirements of the SDI; combined bending and shear, deflection, and interior reactions are considered in calculating these values. A_{wwf} is the minimum area of welded wire fabric recommended for temperature reinforcing in the composite slab; square inches per foot.

	COMPOSITE PROPERTIES									Max. unshored spans, ft.			
	Slab Depth	ϕM_{nf} in.k	A_c in²	Vol. ft³/ft²	W psf	S_c in³	I_{av} in³	ϕM_{no} in.k	ϕV_{nt} lbs.	1span	2span	3span	A_{wwf}
22 gage	5.50	52.80	37.6	0.333	48	1.33	10.1	37.18	5690	7.36	9.64	9.96	0.023
	6.00	59.89	42.0	0.375	54	1.52	13.0	42.70	6100	7.02	9.22	9.52	0.027
	6.25	63.43	44.3	0.396	57	1.62	14.6	45.55	6310	6.87	9.02	9.32	0.029
	6.50	66.97	46.6	0.417	60	1.73	16.4	48.44	6530	6.74	8.84	9.13	0.032
	7.00	74.05	51.3	0.458	66	1.94	20.3	54.34	6970	6.56	8.50	8.78	0.036
	7.25	77.59	53.8	0.479	69	2.04	22.5	57.34	7200	6.48	8.35	8.62	0.038
	7.50	81.13	56.3	0.500	73	2.15	24.8	60.37	7430	6.40	8.13	8.47	0.041
	8.00	88.22	61.3	0.542	79	2.37	29.9	66.51	7900	6.25	7.64	8.18	0.045
	8.25	91.76	63.9	0.563	82	2.48	32.7	69.61	8140	6.18	7.41	8.05	0.047
	8.50	95.30	66.6	0.583	85	2.59	35.7	72.73	8390	6.11	7.20	7.92	0.050
20 gage	5.50	62.81	37.6	0.333	48	1.58	10.8	44.32	6720	8.52	10.82	11.18	0.023
	6.00	71.37	42.0	0.375	54	1.81	13.8	50.89	7130	8.12	10.35	10.70	0.027
	6.25	75.65	44.3	0.396	57	1.94	15.5	54.28	7340	7.94	10.14	10.48	0.029
	6.50	79.92	46.6	0.417	60	2.06	17.4	57.73	7560	7.78	9.94	10.27	0.032
	7.00	88.48	51.3	0.458	66	2.31	21.5	64.77	8000	7.58	9.57	9.89	0.036
	7.25	92.76	53.8	0.479	69	2.44	23.8	68.35	8230	7.48	9.40	9.71	0.038
	7.50	97.03	56.3	0.500	73	2.57	26.2	71.97	8460	7.39	9.24	~~9.54~~	0.041
	8.00	105.59	61.3	0.542	79	2.83	31.6	79.32	8930	7.21	8.94	9.23	0.045
	8.25	109.87	63.9	0.563	82	2.96	34.6	83.03	9170	7.13	8.80	9.09	0.047
	8.50	114.15	66.6	0.583	85	3.09	37.8	86.77	9420	7.05	8.66	8.95	0.050
19 gage	5.50	72.04	37.6	0.333	48	1.81	11.4	50.82	7000	9.53	11.74	12.13	0.023
	6.00	82.00	42.0	0.375	54	2.08	14.6	58.38	7820	9.07	11.24	11.61	0.027
	6.25	86.97	44.3	0.396	57	2.22	16.3	62.28	8240	8.87	11.01	11.38	0.029
	6.50	91.95	46.6	0.417	60	2.36	18.3	66.25	8650	8.69	10.80	11.16	0.032
	7.00	101.91	51.3	0.458	66	2.65	22.6	74.37	9090	8.46	10.40	10.75	0.036
	7.25	106.89	53.8	0.479	69	2.80	25.0	78.51	9320	8.35	10.22	10.56	0.038
	7.50	111.87	56.3	0.500	73	2.95	27.5	82.69	9550	8.24	10.04	10.38	0.041
	8.00	121.83	61.3	0.542	79	3.25	33.2	91.18	10020	8.04	9.72	10.05	0.045
	8.25	126.81	63.9	0.563	82	3.40	36.3	95.48	10260	7.95	9.57	9.89	0.047
	8.50	131.78	66.6	0.583	85	3.56	39.6	99.80	10510	7.86	9.43	9.74	0.050
18 gage	5.50	80.96	37.6	0.333	48	2.04	11.9	57.20	7000	10.44	12.55	12.97	0.023
	6.00	92.32	42.0	0.375	54	2.34	15.2	65.72	7820	9.94	12.02	12.42	0.027
	6.25	98.00	44.3	0.396	57	2.50	17.1	70.12	8240	9.71	11.78	12.17	0.029
	6.50	103.68	46.6	0.417	60	2.66	19.1	74.61	8680	9.51	11.55	11.94	0.032
	7.00	115.04	51.3	0.458	66	2.99	23.6	83.80	9560	9.25	11.13	11.50	0.036
	7.25	120.72	53.8	0.479	69	3.15	26.1	88.48	9890	9.13	10.94	11.30	0.038
	7.50	126.40	56.3	0.500	73	3.32	28.8	93.22	10120	9.02	10.75	~~11.11~~	0.041
	8.00	137.76	61.3	0.542	79	3.67	34.7	102.84	10590	8.80	10.41	10.75	0.045
	8.25	143.44	63.9	0.563	82	3.84	37.9	107.71	10830	8.69	10.25	10.59	0.047
	8.50	149.12	66.6	0.583	85	4.01	41.3	112.61	11080	8.59	10.09	10.43	0.050
16 gage	5.50	80.96	37.6	0.333	48	2.50	13.0	57.20	7000	11.85	14.04	14.51	0.023
	6.00	92.32	42.0	0.375	54	2.88	16.6	65.72	7820	11.27	13.45	13.90	0.027
	6.25	98.00	44.3	0.396	57	3.07	18.6	70.12	8240	11.01	13.18	13.62	0.029
	6.50	103.68	46.6	0.417	60	3.27	20.8	74.61	8680	10.78	12.93	13.36	0.032
	7.00	115.04	51.3	0.458	66	3.67	25.7	83.80	9560	10.49	12.46	12.88	0.036
	7.25	120.72	53.8	0.479	69	3.88	28.4	88.48	10010	10.35	12.24	12.65	0.038
	$\boxed{7.50}$	126.40	56.3	0.500	73	4.09	31.3	93.22	10480	10.22	12.04	$\boxed{12.44}$	0.041
	8.00	137.76	61.3	0.542	79	4.52	37.6	102.84	11420	9.96	11.66	12.05	0.045
	8.25	143.44	63.9	0.563	82	4.73	41.1	107.71	11910	9.85	11.48	11.86	0.047
	8.50	149.12	66.6	0.583	85	4.95	44.8	112.61	12330	9.73	11.31	11.69	0.050

3" LOK-FLOOR

30

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

	L, Uniform Live Loads, psf *														
	Slab Depth	φMn in.k	9.00	9.50	10.00	10.50	11.00	11.50	12.00	12.50	13.00	13.50	14.00	14.50	15.00
22 gage	5.50	52.80	235	205	180	160	145	130	115	105	95	85	75	65	60
	6.00	59.89	265	235	205	185	165	145	130	120	105	95	85	75	70
	6.50	66.97	300	265	230	205	185	165	145	130	120	105	95	85	75
	7.00	74.05	330	290	255	230	205	180	165	145	130	120	105	95	85
	7.50	81.13	360	320	280	250	225	200	180	160	145	130	115	105	95
	8.00	88.22	395	345	305	275	245	220	195	175	155	140	125	115	105
	8.25	91.76	400	360	320	285	255	225	205	180	165	145	135	120	105
	8.50	95.30	400	375	330	295	265	235	210	190	170	155	140	125	110
20 gage	5.50	62.81	285	250	225	200	180	160	145	130	115	105	95	85	80
	6.00	71.37	325	285	255	225	205	185	165	150	135	120	110	100	90
	6.50	79.92	365	320	285	255	230	205	185	165	150	135	125	110	100
	7.00	88.48	400	355	315	285	255	225	205	185	165	150	135	125	110
	7.50	97.03	400	390	350	310	280	250	225	205	185	165	150	135	125
	8.00	105.59	400	400	380	340	305	270	245	220	200	180	165	150	135
	8.25	109.87	400	400	395	350	315	285	255	230	210	190	170	155	140
	8.50	114.15	400	400	400	365	330	295	265	240	215	195	180	160	145
19 gage	5.50	72.04	335	295	260	235	210	190	170	155	140	125	115	105	95
	6.00	82.00	380	335	300	265	240	215	195	175	160	145	130	120	110
	6.50	91.95	400	375	335	300	270	245	220	200	180	165	150	135	125
	7.00	101.91	400	400	375	335	300	270	245	220	200	180	165	150	135
	7.50	111.87	400	400	400	365	330	295	270	240	220	200	180	165	150
	8.00	121.83	400	400	400	400	360	325	290	265	240	220	200	180	165
	8.25	126.81	400	400	400	400	375	335	305	275	250	225	205	190	170
	8.50	131.78	400	400	400	400	390	350	315	285	260	235	215	195	180
18 gage	5.50	80.96	380	335	300	270	240	215	195	180	160	145	135	120	110
	6.00	92.32	400	385	340	305	275	250	225	205	185	170	155	140	130
	6.50	103.68	400	400	385	345	310	280	255	230	210	190	175	160	145
	7.00	115.04	400	400	400	385	345	310	280	255	230	210	195	175	160
	7.50	126.40	400	400	400	400	380	340	310	280	255	235	210	195	180
	8.00	137.76	400	400	400	400	400	375	340	305	280	255	230	210	195
	8.25	143.44	400	400	400	400	400	390	350	320	290	265	240	220	200
	8.50	149.12	400	400	400	400	400	400	365	330	300	275	250	230	210
16 gage	5.50	80.96	380	335	300	270	240	215	195	180	160	145	135	120	110
	6.00	92.32	400	385	340	305	275	250	225	205	185	170	155	140	130
	6.50	103.68	400	400	385	345	310	280	255	230	210	190	175	160	145
	7.00	115.04	400	400	400	385	345	310	280	255	230	210	195	175	160
	7.50	126.40	400	400	400	400	380	340	310	280	255	235	210	195	180
	8.00	137.76	400	400	400	400	400	375	340	305	280	255	230	210	195
	8.25	143.44	400	400	400	400	400	390	350	320	290	265	240	220	200
	8.50	149.12	400	400	400	400	400	400	365	330	300	275	250	230	210
22 gage	5.50	37.18	155	135	115	105	90	80	70	60	55	45	40	35	30
	6.00	42.70	180	155	135	120	105	90	80	70	65	55	50	45	35
	6.50	48.44	205	175	155	135	120	105	95	85	75	65	55	50	45
	7.00	54.34	230	200	175	155	135	120	105	95	85	75	65	55	50
	7.50	60.37	255	225	195	170	150	135	120	105	95	80	75	65	55
	8.00	66.51	280	245	215	190	170	150	130	115	105	90	80	70	65
	8.25	69.61	295	260	230	200	175	155	140	125	110	95	85	75	65
	8.50	72.73	310	270	240	210	185	165	145	130	115	100	90	80	70
20 gage	5.50	44.32	190	165	145	130	115	100	90	80	70	65	55	50	45
	6.00	50.89	220	195	170	150	135	120	105	95	85	75	65	60	50
	6.50	57.73	250	220	195	170	150	135	120	105	95	85	75	70	60
	7.00	64.77	280	250	220	195	170	155	135	120	110	95	85	75	70
	7.50	71.97	315	275	245	215	190	170	150	135	120	110	95	85	75
	8.00	79.32	350	305	270	240	215	190	170	150	135	120	110	95	85
	8.25	83.03	365	320	285	250	225	200	180	160	140	125	115	100	90
	8.50	86.77	380	335	295	265	235	210	185	165	150	135	120	105	95
19 gage	5.50	50.82	225	195	175	155	135	120	110	95	85	80	70	65	55
	6.00	58.38	260	225	200	180	160	140	125	115	100	90	80	75	65
	6.50	66.25	295	260	230	205	180	160	145	130	115	105	95	85	75
	7.00	74.37	330	290	260	230	205	185	165	145	130	120	105	95	85
	7.50	82.69	370	325	290	255	230	205	185	165	150	135	120	110	95
	8.00	91.18	400	360	320	285	255	225	205	180	165	150	135	120	110
	8.25	95.48	400	380	335	300	265	240	215	190	170	155	140	125	115
	8.50	99.80	400	395	350	310	280	250	225	200	180	165	145	135	120
18 gage	5.50	57.20	255	225	200	180	160	140	125	115	105	90	85	75	70
	6.00	65.72	295	260	230	205	185	165	145	130	120	105	95	85	80
	6.50	74.61	335	295	265	235	210	190	170	150	135	125	110	100	90
	7.00	83.80	380	335	295	265	235	210	190	170	155	140	125	115	105
	7.50	93.22	400	375	330	295	265	235	215	190	175	155	140	130	115
	8.00	102.84	400	400	370	330	295	265	235	215	195	175	160	145	130
	8.25	107.71	400	400	385	345	310	275	250	225	200	185	165	150	135
	8.50	112.61	400	400	400	360	320	290	260	235	210	190	175	160	145
16 gage	5.50	57.20	255	225	200	180	160	140	125	115	105	90	85	75	70
	6.00	65.72	295	260	230	205	185	165	145	130	120	105	95	85	80
	6.50	74.61	335	295	265	235	210	190	170	150	135	125	110	100	90
	7.00	83.80	380	335	295	265	235	210	190	170	155	140	125	115	105
	7.50	93.22	400	375	330	295	265	235	215	190	175	155	140	130	115
	8.00	102.84	400	400	370	330	295	265	235	215	195	175	160	145	130
	8.25	143.44	400	400	385	345	310	275	250	225	200	185	165	150	135
	8.50	112.61	400	400	400	360	320	290	260	235	210	190	175	160	145

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 l/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.

3" LOK-FLOOR

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USD United Steel Deck, Inc.

U.L. Fire Ratings - Composite Deck, cont'd.

RESTRAINED ASSEMBLY RATINGS (HOURLY): 2

U.L. DES. NO.	F.P.	CONCRETE COVER	USD PRODUCTS
D216	S	2 ½ NW,LW	BL,BLC,LF2,LFC2,LF3,LFC3,NL,NLC
D502	S	2 ½ NW	BL,BLC,LF2,LFC2,LF3,LFC3,NL,NLC
D703	C	2 ½ NW,LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC*
D704	C	2 ½ NW	BL,BLC,LF15,LFC1
D706	C	2 ½ NW	LF3,LFC3
D712	C	2 ½ NW,LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC*
D716	C	2 ½ NW,LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3*
D722	C	2 ½ NW,LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC*
D726	C	2 ½ NW,LW	LF15,LF2,LF3,NL *
D727	C	2 ½ NW	INV.BL,INV. NL *
D730	C	2 ½ NW	LF2,LFC2,LF3,LFC3,NL,NLC*
D733	N	3 ¼ LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC*
D739	C	2 ½ NW,LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC,AWC2,AWC3*
D742	C	2 ½ NW	LF15,LF2,LF3,NL*
D743	C	2 NW,LW	LF2,LFC2,LF3,LFC3*
D745	C	2 ½ NW,LW	LF2,LF3 *
D746	C	2 ½ LW	BL *
D747	C	2 ½ LW	LF2 *
D750	C	2 ½ NW,LW	BL,INV.BL,LF2,LF3,NL *
D752	C	2 ½ LW	BL,BLC,,LF2,LFC2,LF3,LFC3*
D755	C	2 ½ NW,LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC*
D759	C	2 ½ NW,LW	BL,LF15,LF2,LF3,NL*
D760	C	2 ½ NW,LW	LF2,LF3
D767	C	2 ½ NW,LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,AWC2,AWC3
D777	C	2 ½ NW	LF15,LF2,LF3,NL*
D772	C	2 ½ NW,LW	LF2,LF3*
D773	C	2 ½ LW	BL*
D774	C	2 ½ LW	LF2*
D775	C	2 ½ NW,LW	BL,INV. BL,LF2,LF3*
D779	C	2 ½ NW,LW	BL,LF15,LF2,LF3
D822	F	2 ½ NW,LW	LF2,LFC2,LF3,LFC3,NL,NLC*
D824	F	2 ½ NW,LW	BL,BLC,LF15,LFC1
D825	F	2 ½ NW,LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC*
D826	N	3 ¼ LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC*
D831	F	2 ½ NW,LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC*
D832	F	2 ½ NW,LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC*
D833	F	2 ½ NW,LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3*
D837	F	2 ½ NW	BL,BLC,LF15,LFC1*
D840	N	3 ¼ LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC*
D847	F	2 ½ NW,LW	LF2,LFC2,LF3,LFC3,NLC*
D852	F	2 ½ NW,LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3*
D858	F	2 ½ NW,LW	LF2,LFC2,LF3,LFC3,AWC2,AWC3*
D859	F	2 NW,LW	LF2,LFC2,LF3,LFC3*
D860	F	3 ¼ LW	LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC*
D861	F	2 ½ NW,LW	LF2,LF3*
D862	F	2 ½ LW	LF2,LF3*
D870	F	2 ½ NW,LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3*
• D902	N	4 ½ NW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
D902	N	3 ¼ LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
D902	N	3 ½ LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
D906	N	3 ¼ LW	NLC
D907	N	3 ¼ LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3
D908	N	3 ¼ LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
D913	N	3 ¼ LW	BL,LF15,LF2,LFC2,LF3,LFC3
D916	N	4 ½ NW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
D916	N	3 ¼ LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
D916	N	3 ½ LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
D918	N	4 ½ NW	LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
D918	N	3 ¼ LW	LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
D918	N	3 ½ LW	LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
D919	N	3 ¼ LW	LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
D919	N	3 ½ LW	LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
D920	N	3 ¼ LW	LF2,LFC2,LF3,LFC3
D922	N	4 ½ NW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
D922	N	3 ½ LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
D923	N	4 ½ NW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
D923	N	3 ½ LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
D925	N	4 ½ NW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
D925	N	3 ½ LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
D927	N	4 ½ NW	B,BLC,LF2,LF2C,LF3,LF3C,NL,NLC
D927	N	3 ¼ LW	B,BLC,LF2,LF2C,LF3,LF3C,NL,NLC
D929	N	4 ½ NW	B,BLC,LF2,LF2C,LF3,LF3C,NL,NLC
D929	N	3 ¼ LW	B,BLC,LF2,LF2C,LF3,LF3C,NL,NLC

1. **United Steel Deck, Inc.**, is not responsible for the adhesive ability of any spray applied fire protection material, or for any treatment, cleaning, or preparation of the deck surface required for adhesion of fire protection material.

2. The live loads shown in the composite tables may require a reduction if a U.L. fire rating is required. The worst load reduction for any design is 40%. Designs D733, D742, D825, D840, D860, D902, D907, D914, and D916 do not require a reduction if the sidelaps are attached at 24" o.c. as was used in the fire test.

3. Be sure to check the **U.L. Fire Resistance Directory** for all details of construction.

4. Listings marked with * allow the use of phosphatized/painted noncellular deck except LF15. All D9xx listings allow the use of phosphatized/painted noncellular deck.

5. **IN THE F.P. COLUMN:**
   - **S** = suspended ceiling
   - **F** = fibrous fireproofing
   - **C** = cementitious
   - **N** = no fireproofing on the deck.

6. The concrete cover is measured from the top of the deck - add the deck depth to get the total slab thickness.

7. The BSA approvals for use in New York City are 620-76-SM (2 hours) and 621-76-SM (3 hours).

8. **PRODUCT CODES:**
   - **BL** = B-LOK
   - **BLC** = B-LOK cellular
   - **INV. BL** = inverted B-LOK
   - **LF15** = 1½" LOK floor
   - **LFC1** = 1½" LOK floor cellular
   - **LF2** = 2" LOK floor
   - **LFC2** = 2" LOK floor cellular
   - **LF3** = 3" LOK floor
   - **LFC3** = 3" LOK floor cellular
   - **NL** = N LOK
   - **NLC** = N LOK cellular
   - **INV. NL** = inverted N LOK
   - **AWC2**, **AWC3** = three service compact cell sections

FIRE RATINGS, CONT'D

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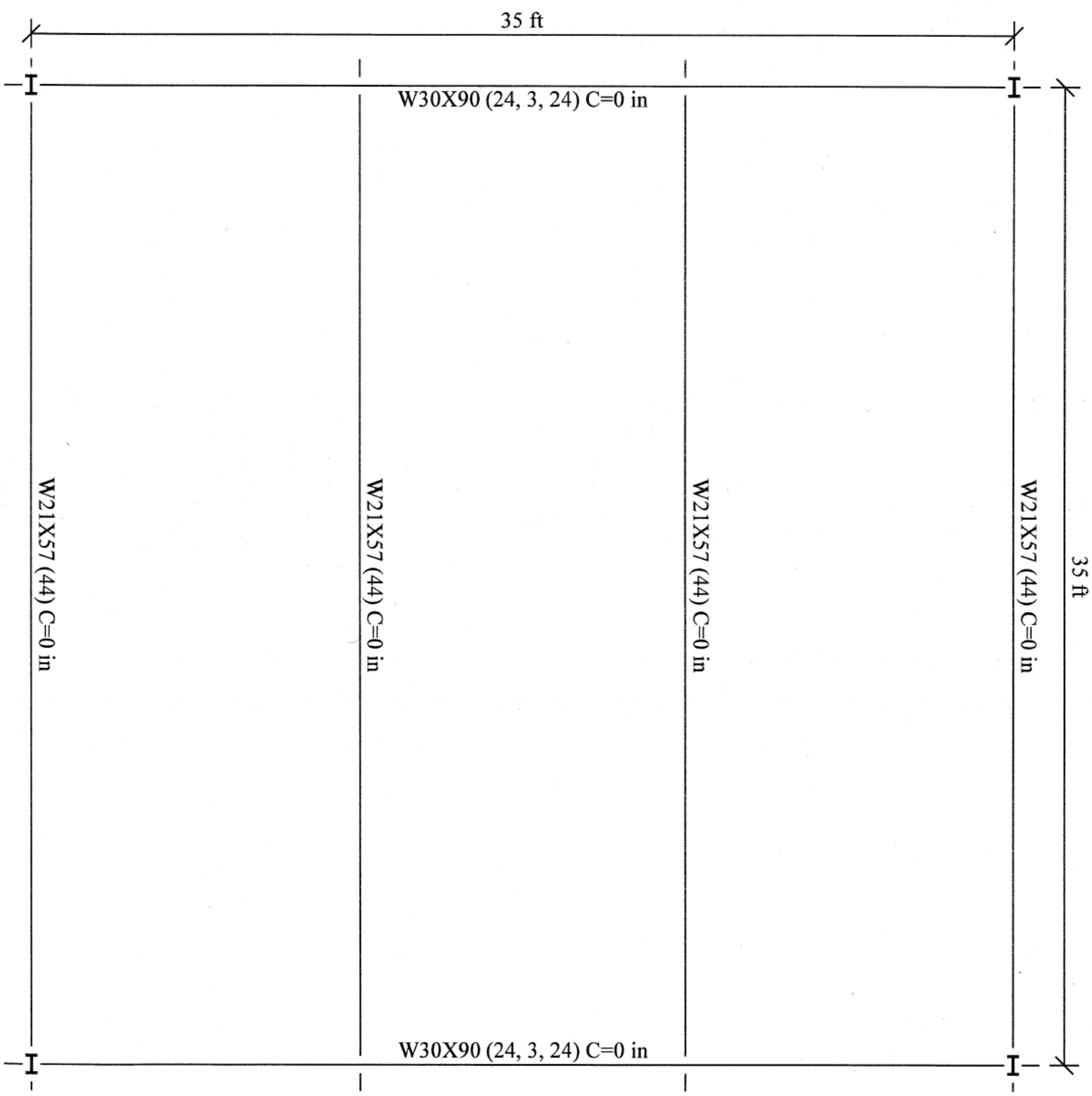
Floor Framing steel tools

Steel SolutionsCenter www.aisc.org/steeltools

Bay Output Summary

$ 2.62/s.f.
7.46 psf

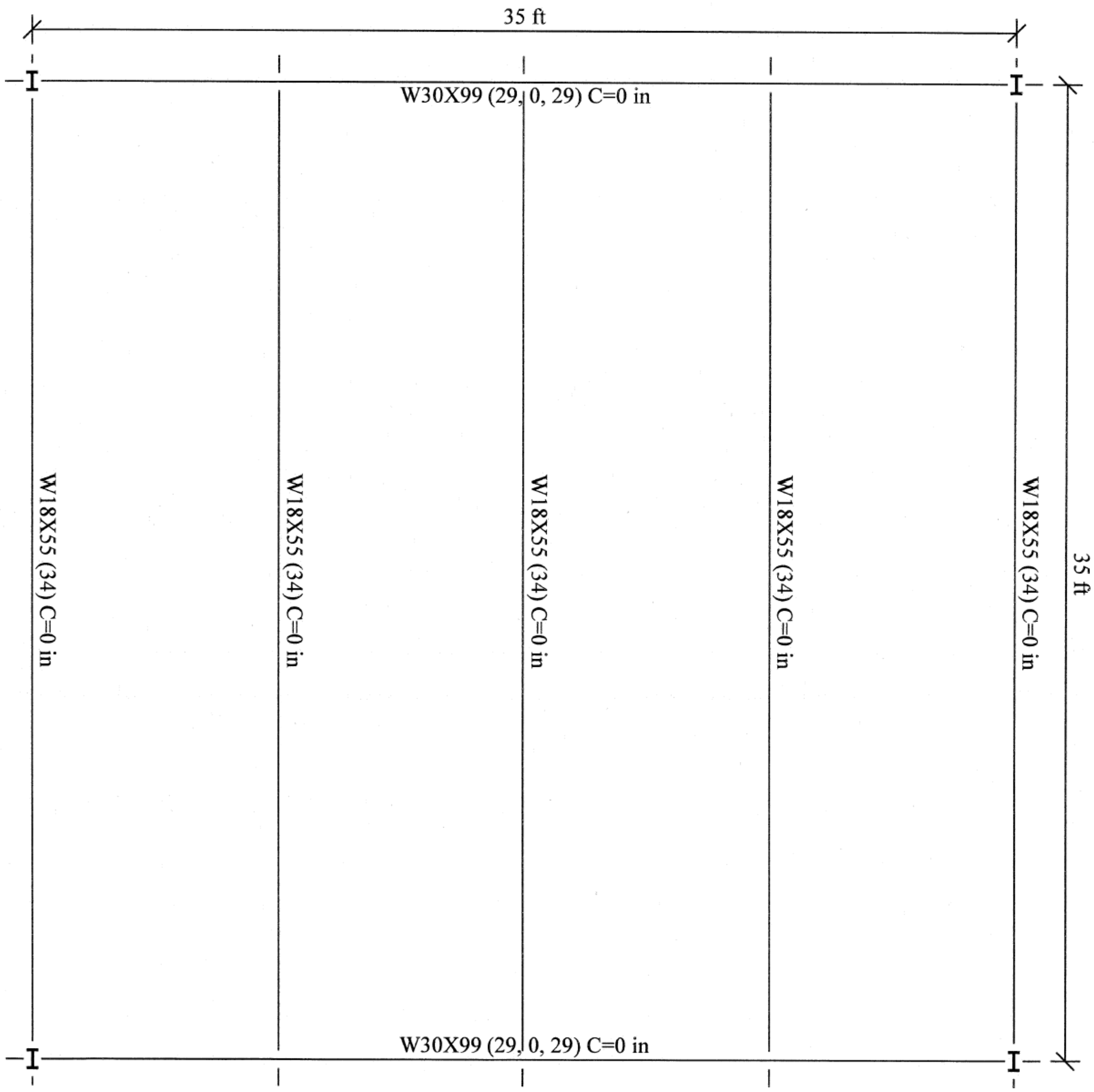


Floor Framing steel tools

Steel SolutionsCenter www.aisc.org/steeltools

Bay Output Summary

$ 3.13/s.f.
9.11 psf



End of Report