THE PENNSYLVANIA STATE UNIVERSITY

CROCKER WEST BUILDING

STATE COLLEGE, PA

Senior Thesis Project Tech I:
Structural Concepts and Existing Conditions Report

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Structural Option

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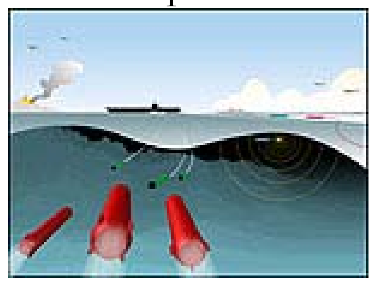
-- EXECUTIVE SUMMARY --

Structural Tech Report I is slated to assess the current structural system of Crocker West in an existing conditions report. Tech I will help us determine and better understand the design methods and selection criterion for the structural plan.



The Crocker West Building 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.

Two lateral analyses, wind and seismic, are included at the end of this report in Appendix B. The wind analysis was performed using the Analytical Method 2 of ASCE 7. Having found a design wind pressure of 16.4 psf at roof level (El. 40'-0 A.F.F.) and comparing it to the 19.6 psf found by the design engineer with the Simplified Method 1 of ASCE 7, I can conclude that the results I found using method 2 are reasonable. The variation of values can ultimately be due to the method used in analyzing and the level of detail required for each method. Similarly, some error can also be seen in the calculated story forces for the structure. Seismic load calculations were completed under the provisions of ASCE 7-05 (Chapters 11 & 12) and IBC 2006 (Section 1613). Using the necessary seismic considerations, I determined the Equivalent Lateral Force Procedure defined in section 12.8 of ASCE 7 was permitted for analysis and thus used. In comparing the results of my calculations versus the designer's spreadsheet, I found our values for V (base shear) to be quite different. I calculated a base shear of 1672 kips, which is nearly double that of the engineer's 883 kips. Having a Cs value of 0.089 versus a Cs value of 0.0607 will not make much difference and thus, I assume the discrepancy lies within the calculated building weight.

It is also important to note that spot checks of various structural components are also included in Appendix B of this report in order to justify other element sizing of the Crocker West Building structure, while Appendix A contains drawings of the project for reference.

-- 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.

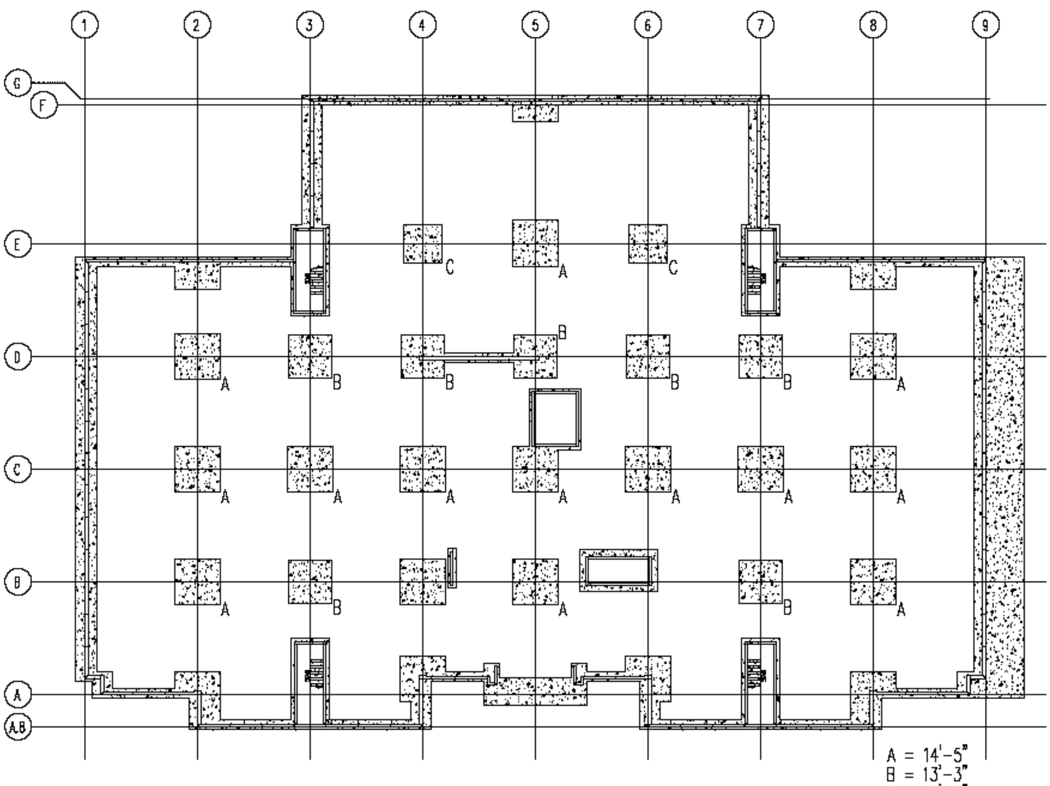


FIGURE #1 – FOUNDATION SYSTEMS

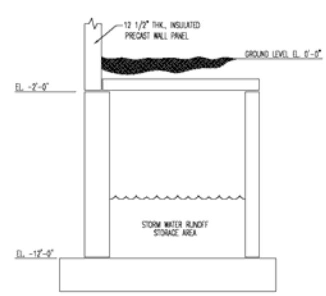


FIGURE #2 – PROPOSED CISTERN SECTION

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.

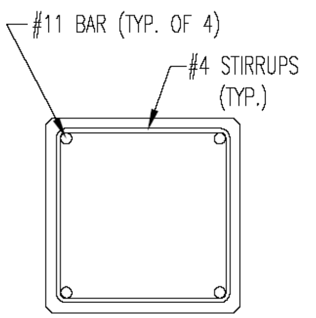


FIGURE #3 – TYP. COLUMN SECTION

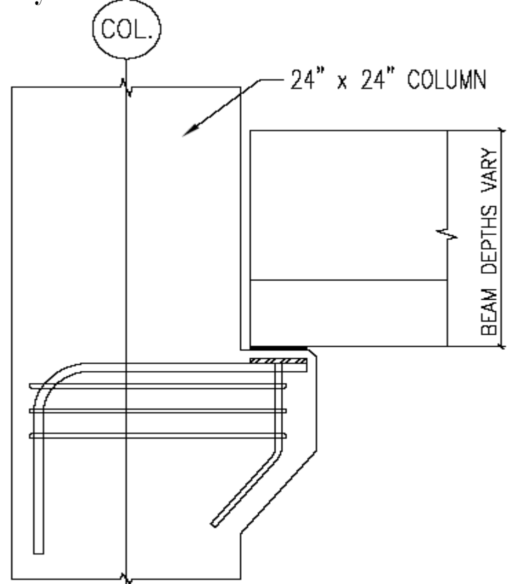


FIGURE #4 – COLUMN w/ HAUNCH

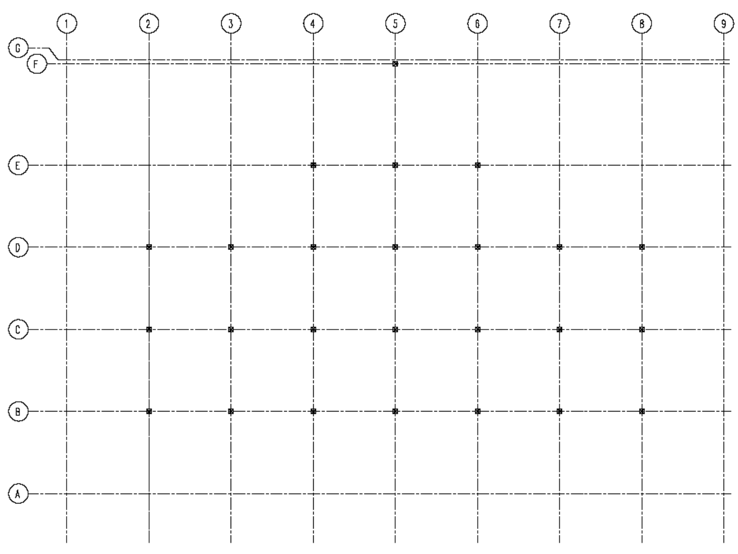


FIGURE #5 – COLUMN GRID

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.

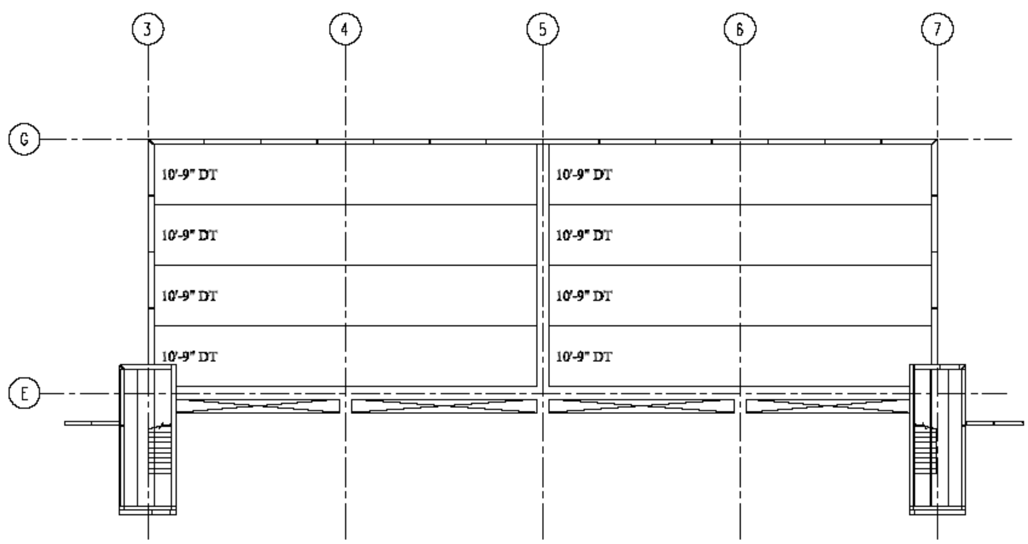


FIGURE #6 – LOW ROOF DT LAYOUT

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.

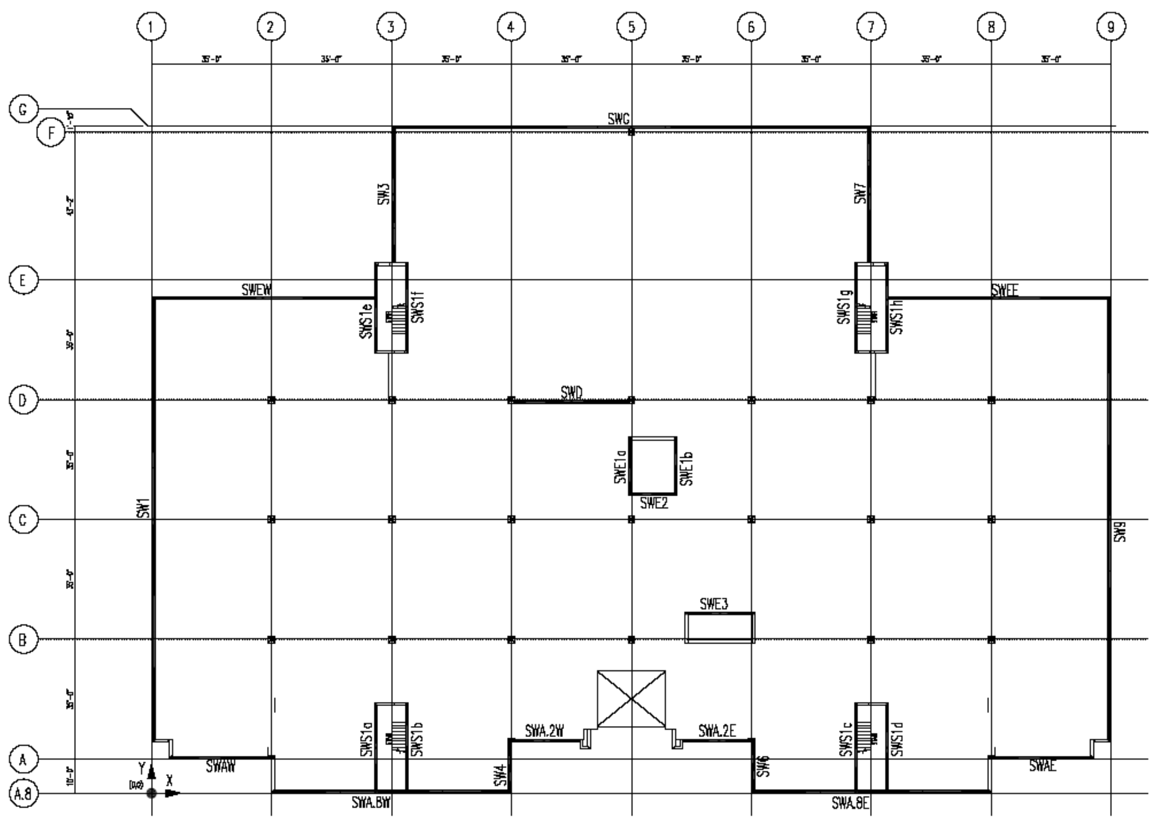


FIGURE #7 – SHEAR WALL LAYOUT

-- STRENGTH OF MATERIALS --

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

-- 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

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 (pf = 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.

-- LATERAL LOADING --
(Wind)

WIND LOADS:

The following wind analysis results were established using the provisions of ASCE 7-05, CH. 6. A complete detailed wind analysis has been included in Appendix B for reference and verification.

Basic Wind Speed	V = 90 mph
	K_d = 0.85
Topographic Factor	*I* = 1.0
	K_h = 1.04

N-S WIND PRESSURES

Height (ft.)	qz (psf)	qh (psf)	$P_{WINDWARD}$ (psf)	$P_{LEEWARD}$ (psf)
0-15	15.0	18.3	13.5	-11.1
20	15.9	18.3	14.1	-11.1
25	16.6	18.3	14.6	-11.1
30	17.3	18.3	15.1	-11.1
40	18.3	18.3	15.7	-11.1
50	19.2	18.3	16.4	-11.1

E-W WIND PRESSURES

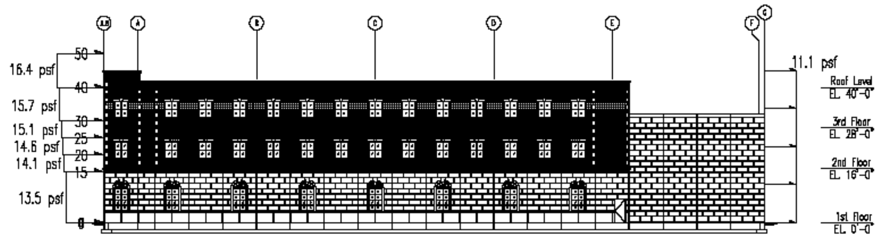
Height (ft.)	qz (psf)	qh (psf)	$P_{WINDWARD}$ (psf)	$P_{LEEWARD}$ (psf)
0-15	15.0	18.3	13.5	-3.13
20	15.9	18.3	14.1	-3.13
25	16.6	18.3	14.6	-3.13
30	17.3	18.3	15.1	-3.13
40	18.3	18.3	15.7	-3.13
50	19.2	18.3	16.4	-3.13

Wind Pressure Diagrams displayed on the following page.

-- WIND PRESSURE DIAGRAMS --

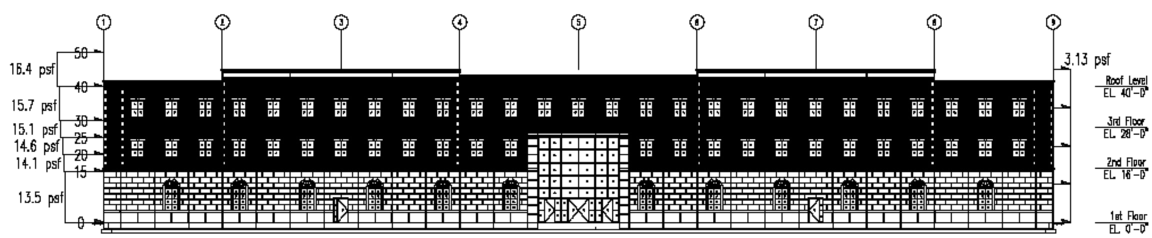
Due to the building symmetry, wind pressures are equal and opposite for reverse wind direction.

→→ Wind Dir. →→



NORTH – SOUTH DIRECTION
(WIND NORMAL TO 280 FT FACE)

→→ Wind Dir. →→



EAST – WEST DIRECTION
(WIND NORMAL TO 195 FT FACE)

N-S WIND FORCES

	Height (ft)	Story Force (kips)	Overturning Moment (ft-k)
1. Low Roof/ Warehouse Area	28	43.05	1205.4
2. Main Structure			
a. 1st Floor	Gnd Lvl.	26.5	0
b. 2nd Floor	16	54.4	870.4
c. 3rd Floor	28	51.0	1428
d. Roof	40	66.1	2645.4

E-W WIND FORCES

	Height (ft)	Story Force (kips)	Overturning Moment (ft-k)
1. Low Roof/ Warehouse Area	28	14.53	406.9
2. Main Structure			
a. 1st Floor	Gnd Lvl.	18.4	0
b. 2nd Floor	16	30.1	481.6
c. 3rd Floor	28	28.2	790.3
d. Roof	40	29.7	1189.5

Floor Tributary Widths

- 1st Floor = 7'-0
- 2nd Floor = 14'-0
- 3rd Floor = 12'-0
- Roof = Varies (see wind calc.'s, Appendix B)

-- LATERAL LOADING --
(Seismic)

SEISMIC LOADS:

The following seismic analysis results were established using the provisions of ASCE 7-05, Chapters 11 & 12 and IBC 2006, Section 1613. A complete detailed seismic analysis has been included in Appendix B for reference and verification.

Seismic Considerations		S_S = 0.17
		S_1 = 0.06
Building Occupancy		Type II
Seismic Design Category		B
Seismic Response Coefficient		C_S = 0.089
Effective Seismic Weight		Wt. (kips)
1. Roof		4336.8
2. 3rd Floor		7244.4
3. 2nd Floor		7194.8
	Total Effective Seismic Wt. =	18,776 kips
Seismic Base Shear		$V = C_SW$ (kips)
1. Roof		V_R = 386
2. 3rd Floor		V_3 = 645
3. 2nd Floor		V_2 = 641
	Total Base Shear (V_T) =	1,672 kips
Overturning Moment		(ft-k)
1. Roof		16,212
2. 3rd Floor		18,060
3. 2nd Floor		10,256
	Total O.T. Moment =	44,528 ft-kips

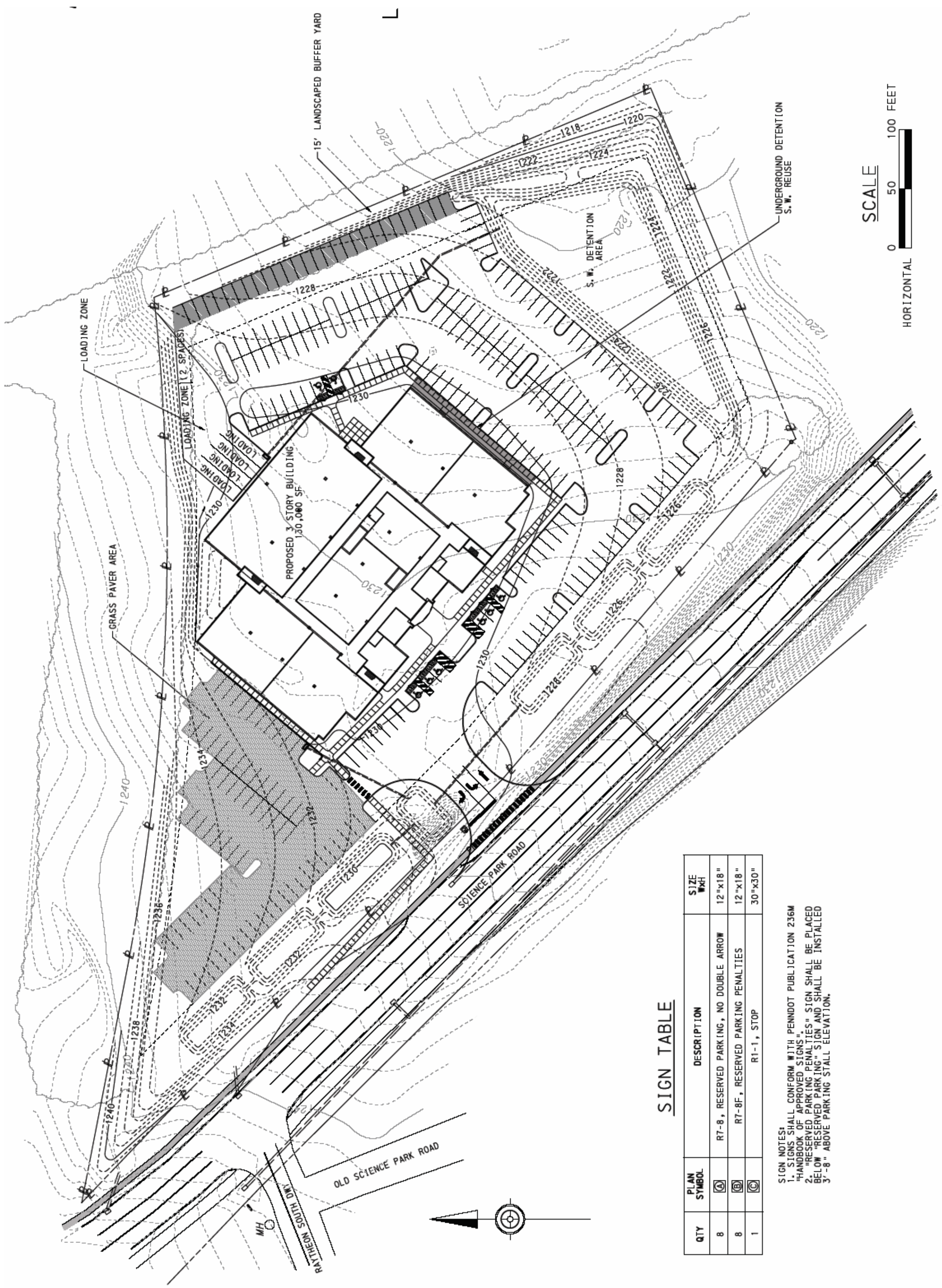
-- CONCLUSIONS --

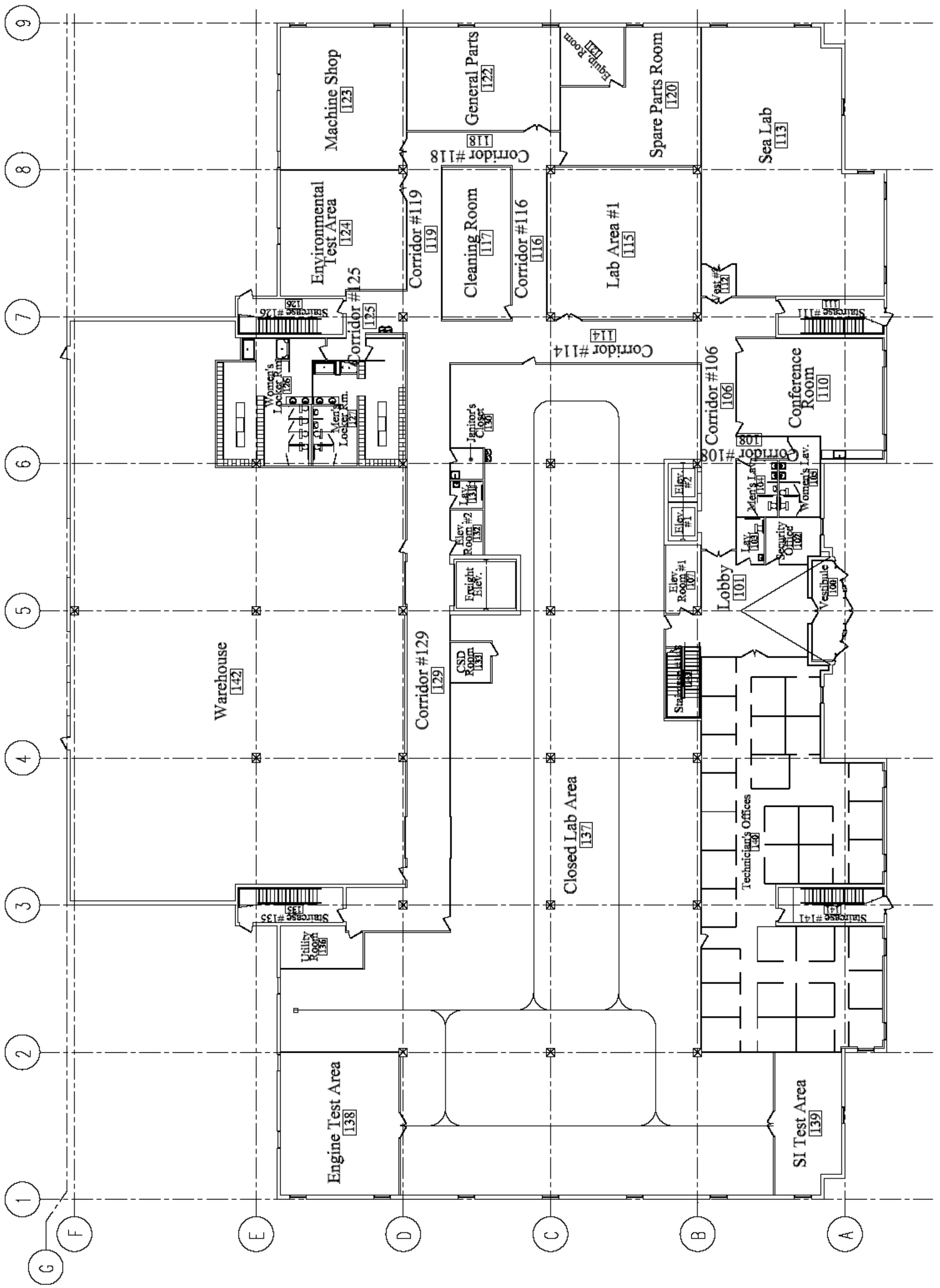
In conclusion, this report presents various types of information to validate the existing structure of Crocker West. I spot checked several different load carrying members and found them to be of similar size to those used in the original design. Likewise, the wind and seismic lateral analyses I performed, seismic controlling, yielded values that are rational to the design engineer's values to prove them practical. The minor discrepancies encountered in the wind analysis I believe are simply due to the fact I used the analytical procedure and not the simplified method. Additionally, I feel the large difference in base shear found between my seismic analysis and the engineers output is caused by our differing seismic weight (W) values. I concluded this based on two situations. First, I found Cs to be 0.089 which is very comparable to Cs = 0.0607 of that determined by the engineer's output. And second, I assumed many of the wall panel lengths when calculating their weights for the effective seismic weight (W) used in determining base shear (V = CsW). This led to me recording a higher seismic weight, thus the higher base shear value. Furthermore, other errors between my calculations and the engineer's could be due to the computer-based design program used and the parameters of that program.

Also note, the spot checks performed are not complete design check. I did not include many checks that would be necessary for proper design of that particular component, nor the entire structure. Disregarding uplift in the wind analysis and ignoring overturning effects on foundation elements are a few examples. However, I would like to add that the concise beam designs included in Appendix B of this report are in fact actual designs for this structure, having completed them myself for the owner of this building.

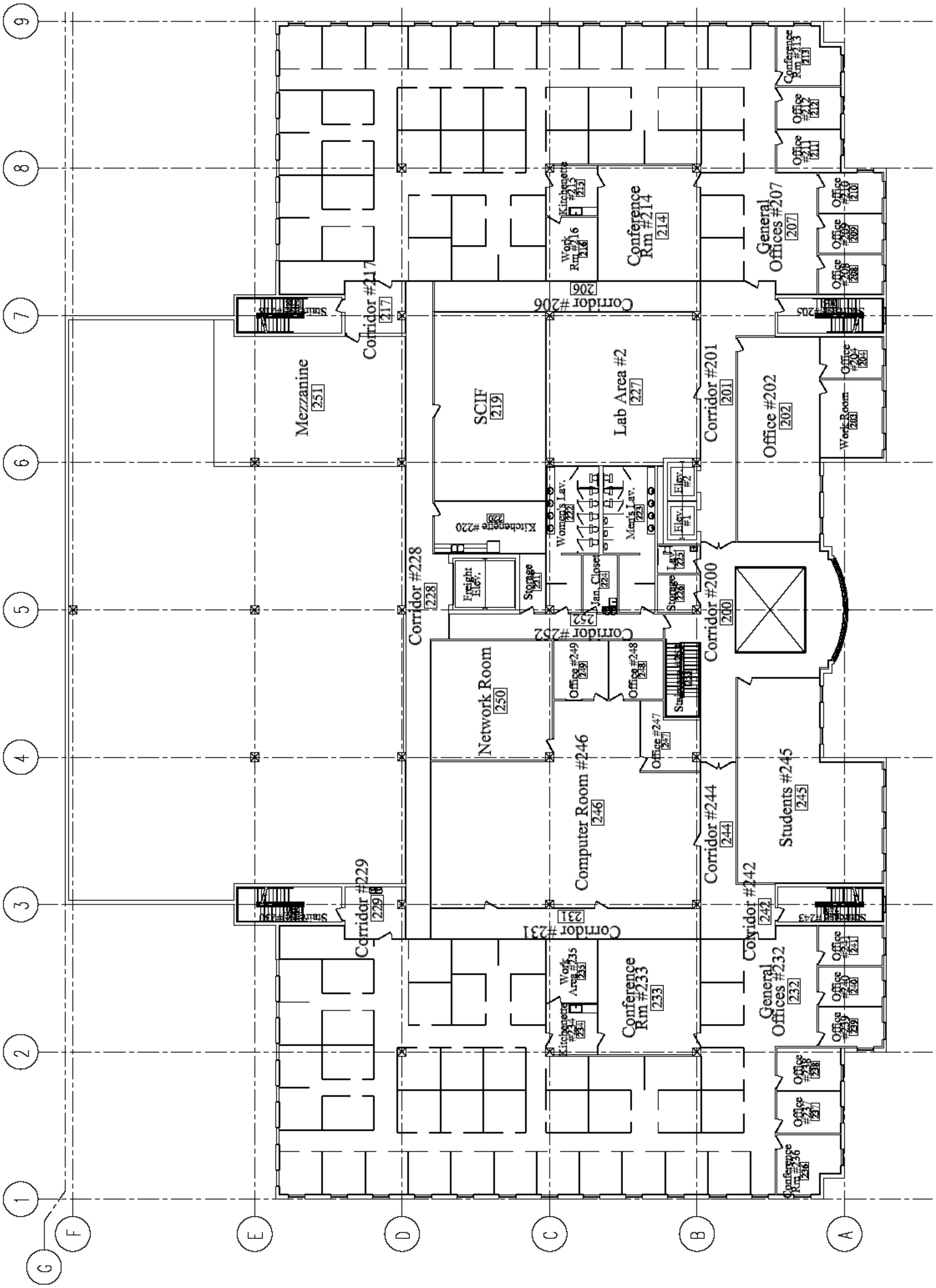
Appendix A

(Project Drawings)

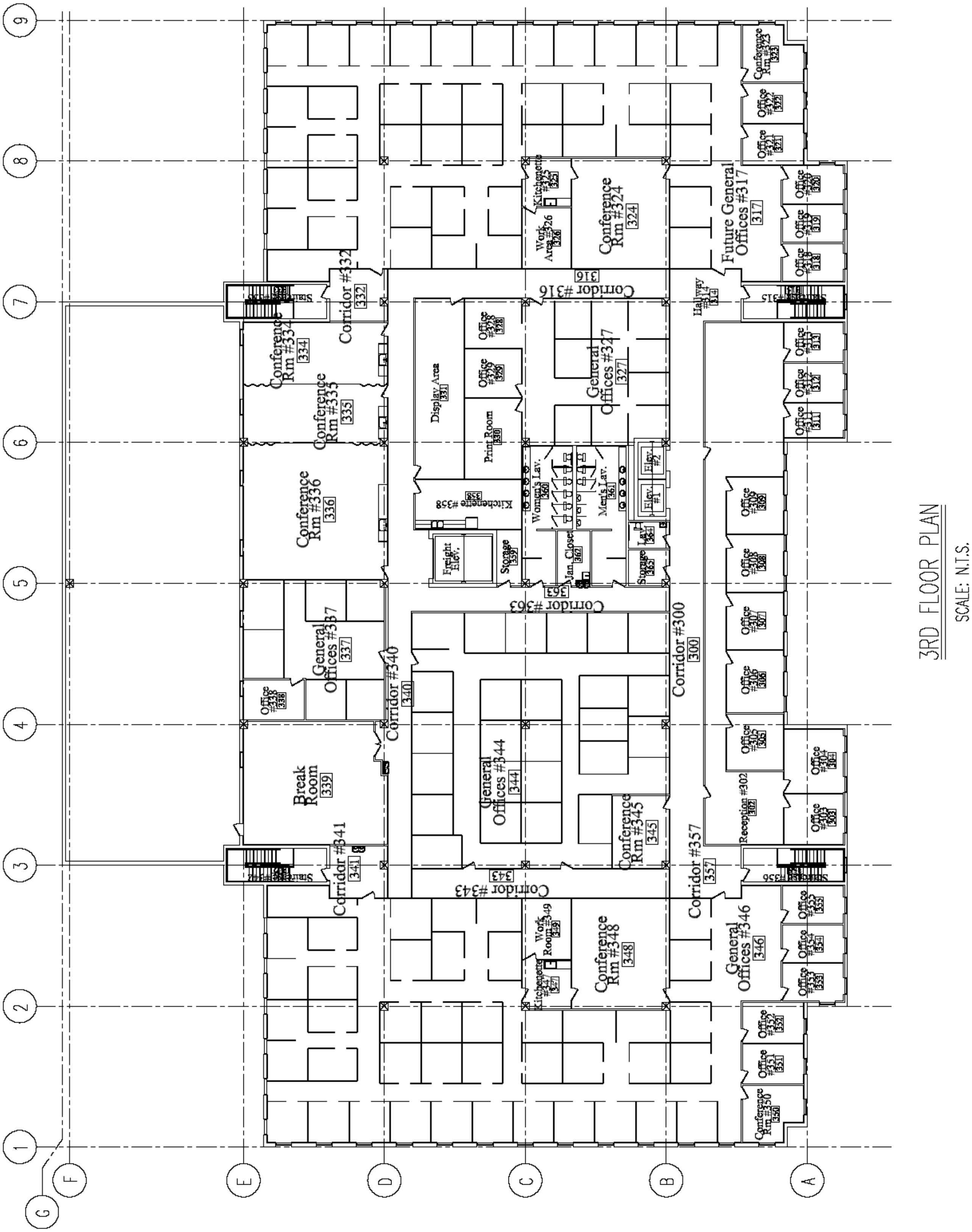

15' LANDSCAPED BUFFER YARD
UNDERGROUND DETENTION S.W. REUSE
SCALE
HORIZONTAL
0
50
100 FEET
S.W. DETENTION AREA
LOADING ZONE
LOADING ZONE (2 SPACES)
LOADING
LOADING
LOADING
PROPOSED 3 STORY BUILDING 30,000 SF
GRASS PAVER AREA
SCIENCE PARK ROAD
OLD SCIENCE PARK ROAD
RAYTHEON SOUTH DWY
MH
SIGN TABLE
QTY | 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.

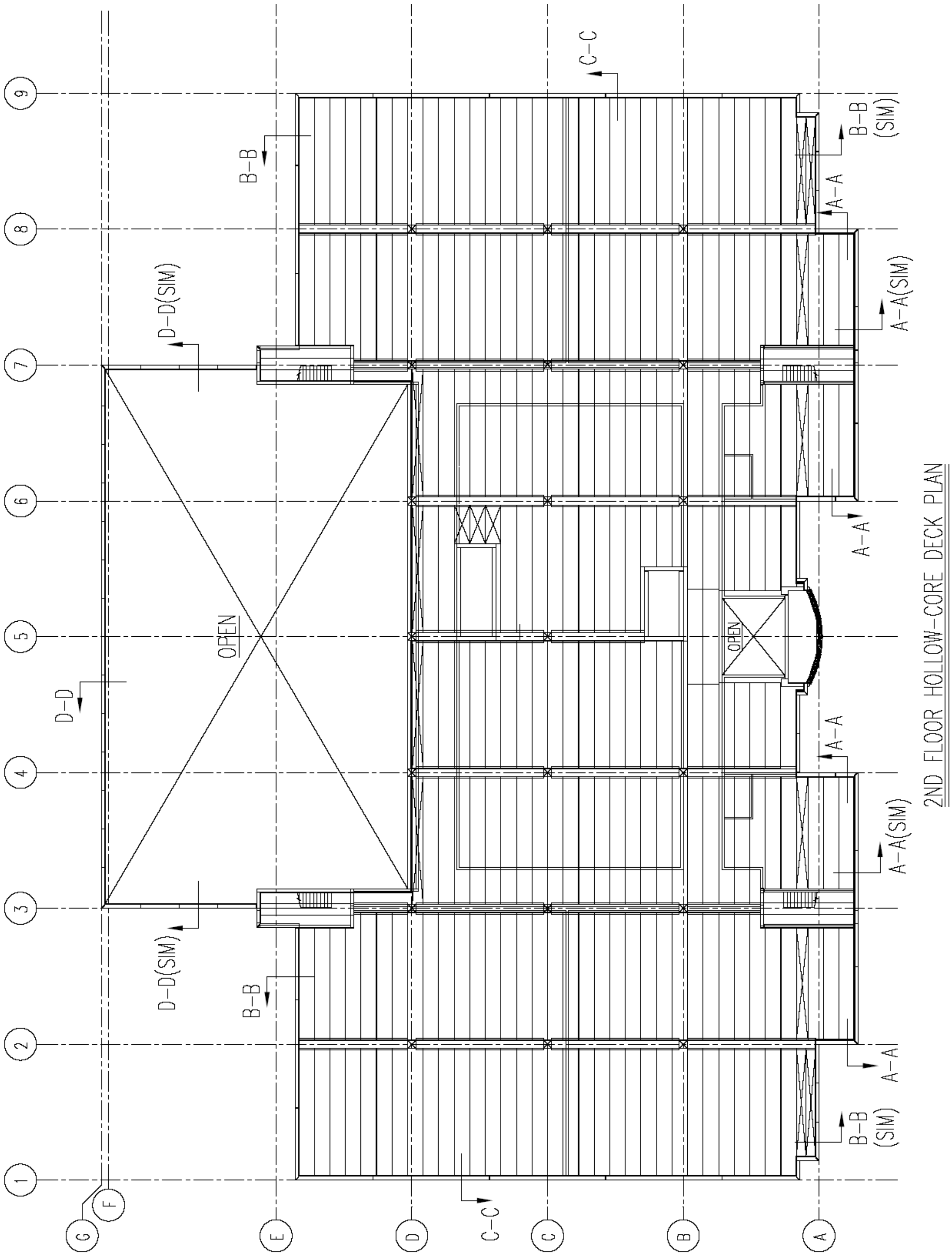


1ST FLOOR PLAN
SCALE: N.T.S.

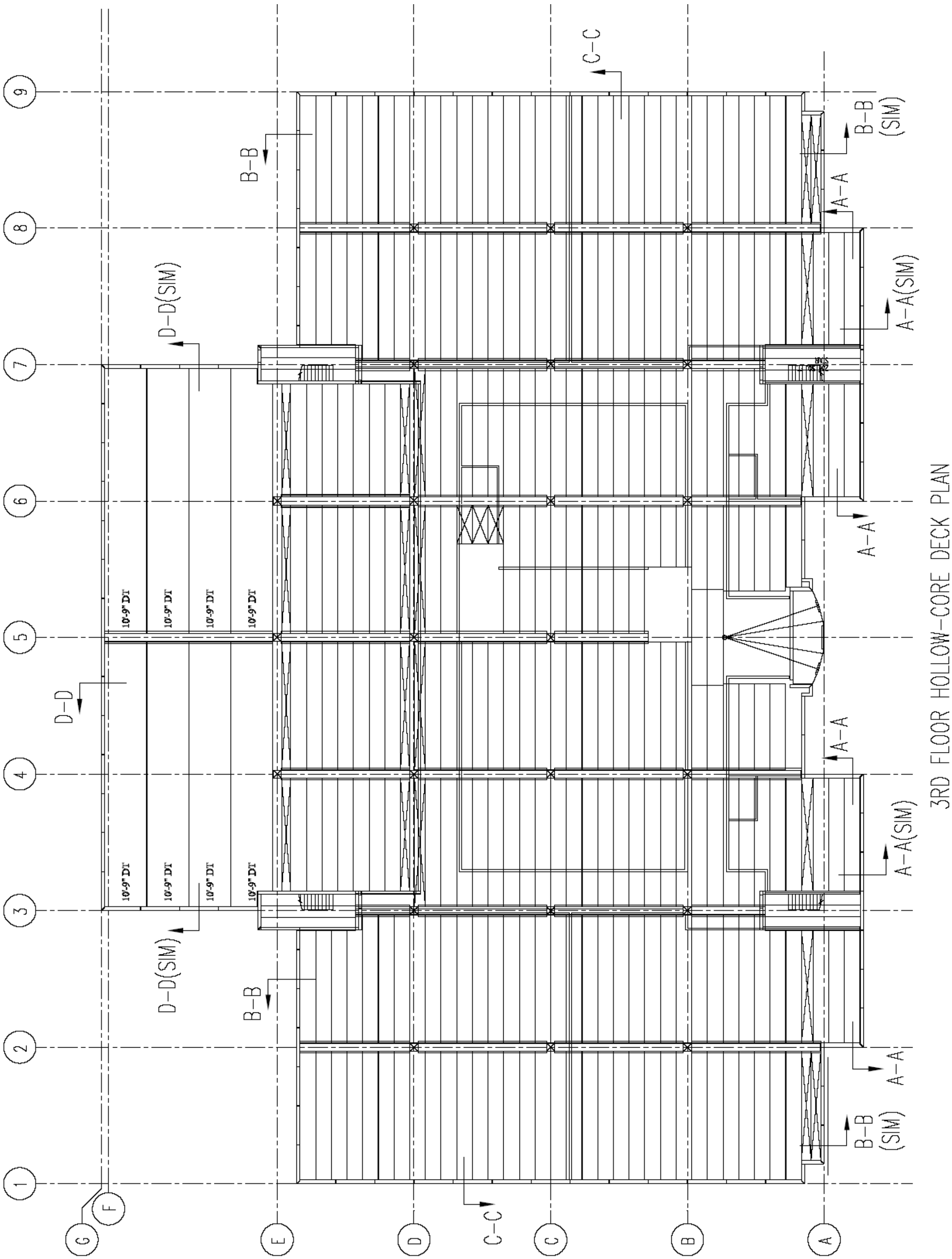


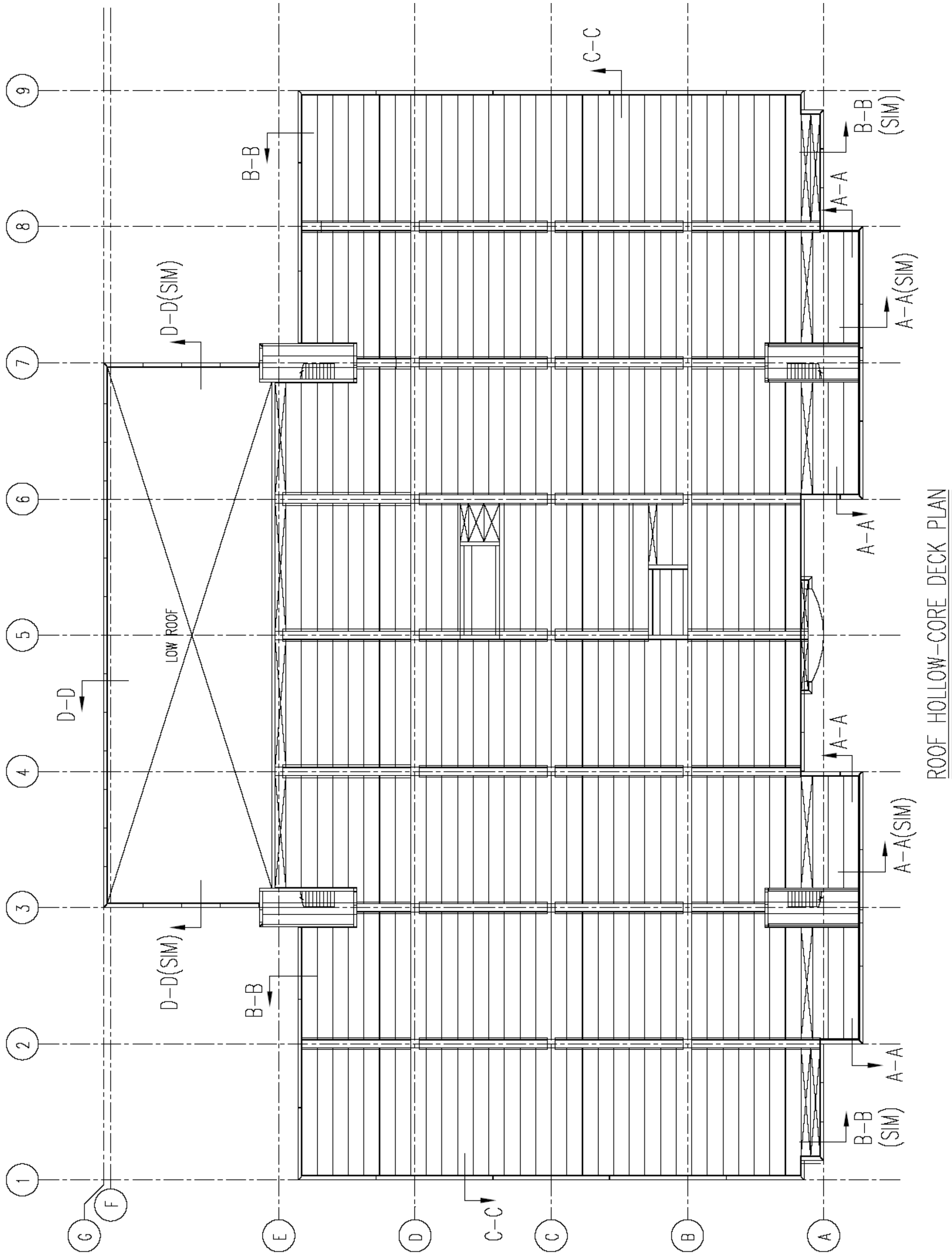
2ND FLOOR PLAN
SCALE: N.T.S.

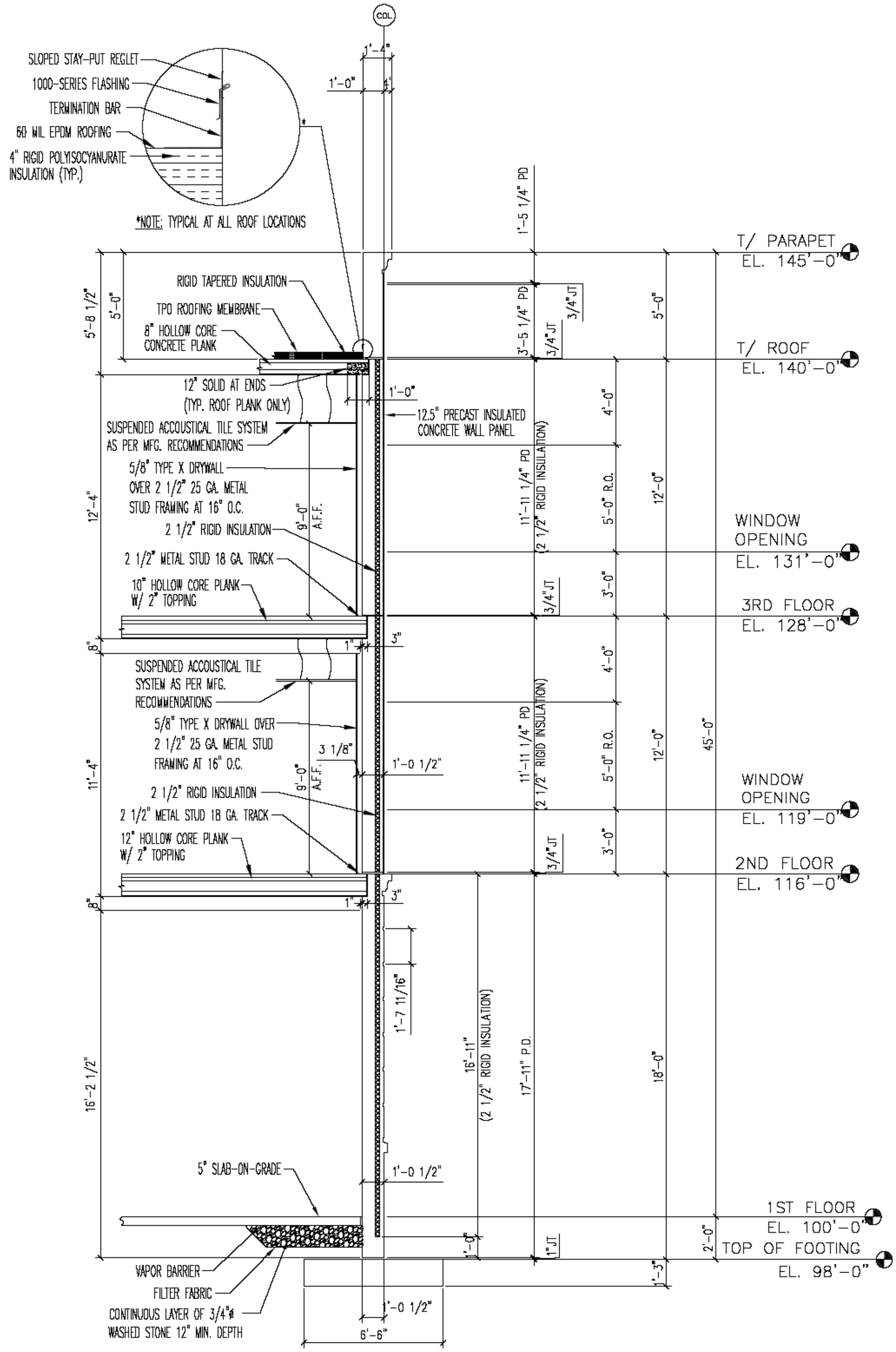

3RD FLOOR PLAN
SCALE: N.T.S.



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

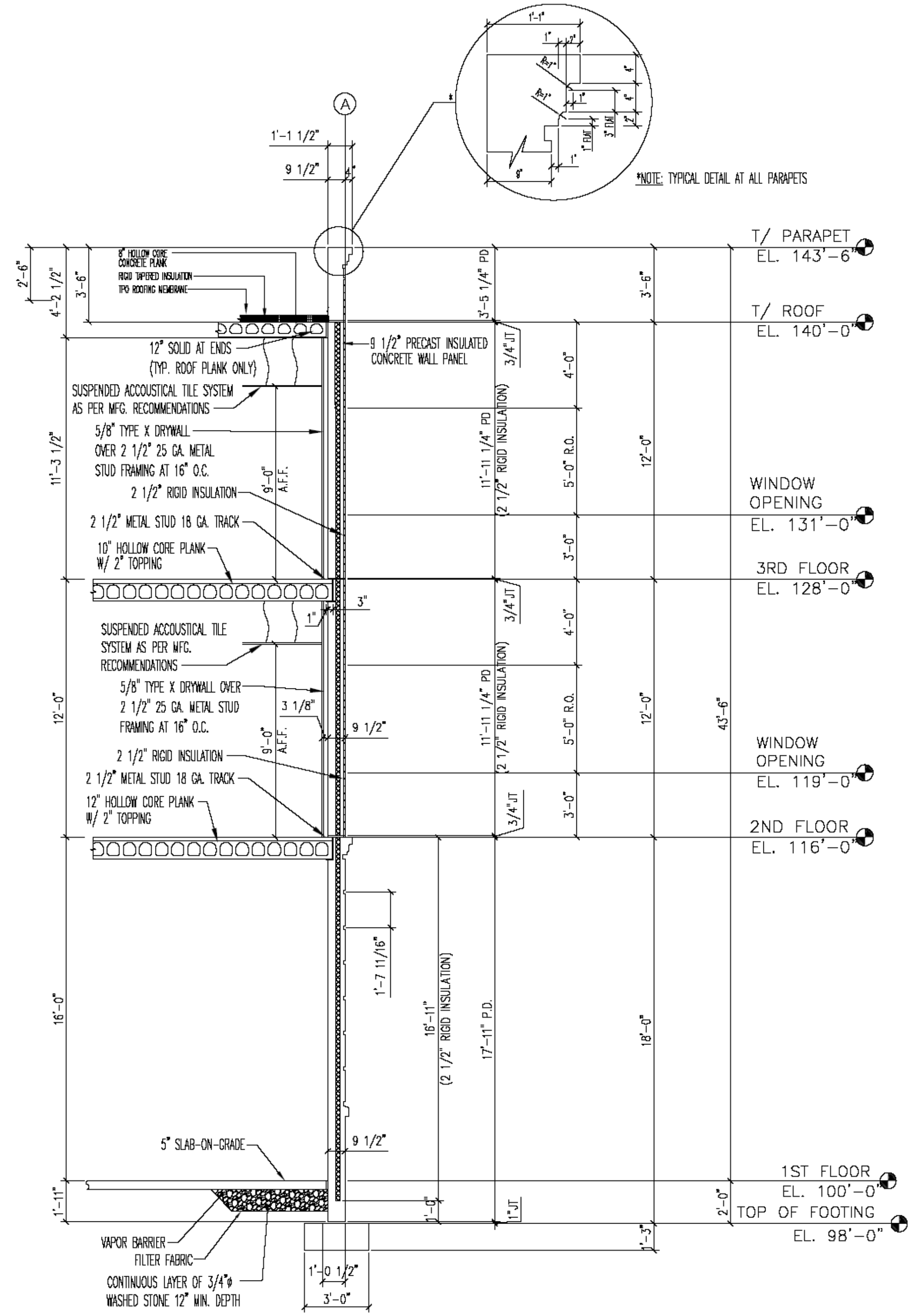

3RD FLOOR HOLLOW-CORE DECK PLAN
SCALE: N.T.S.
A-A
A-A(SIM)
B-B
B-B (SIM)
C-C
D-D
D-D(SIM)
10'-9" DT
1
2
3
4
5
6
7
8
9
A
B
C
D
E
F
G


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



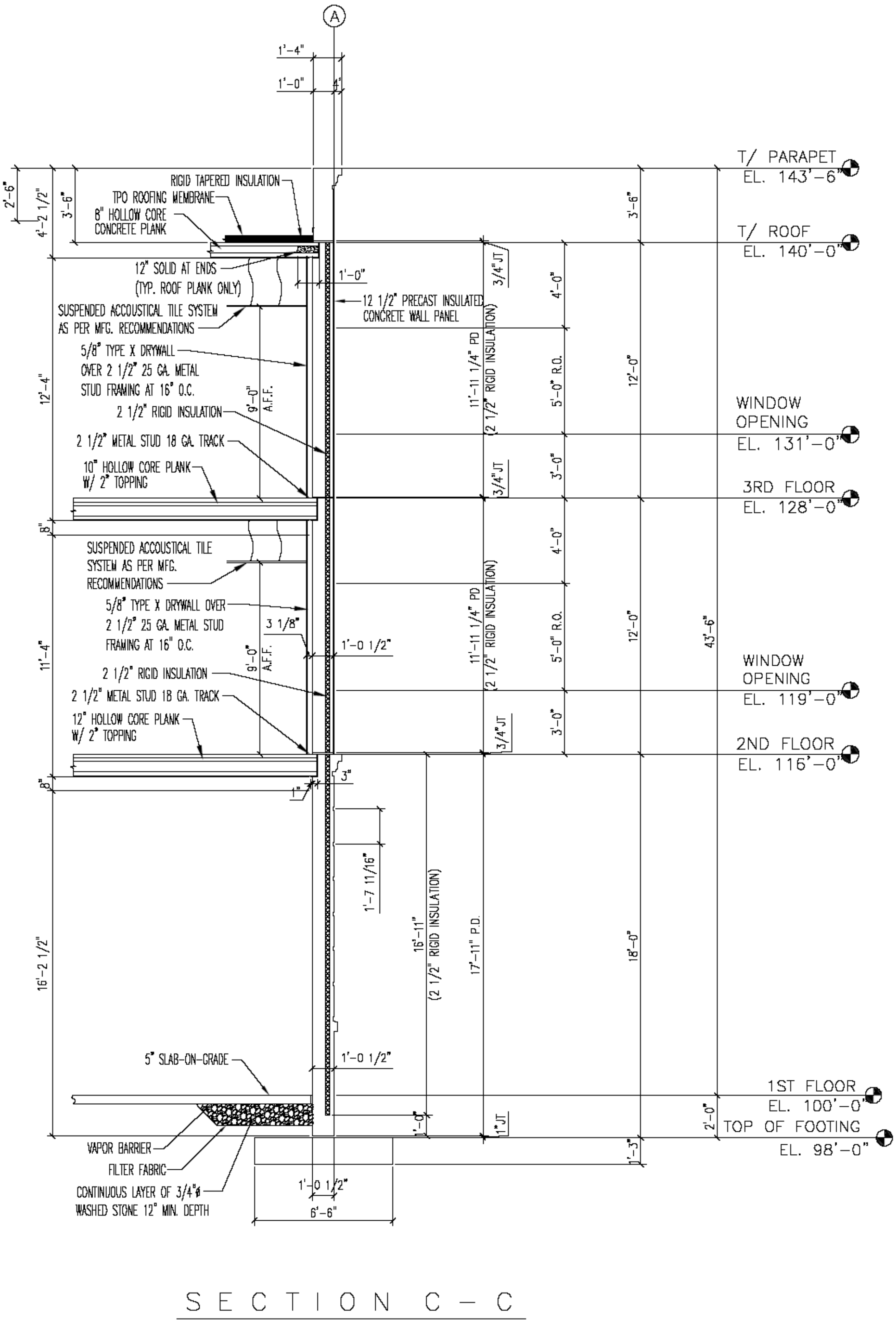
SECTION A – A

SCALE 3/8" = 1'-0"



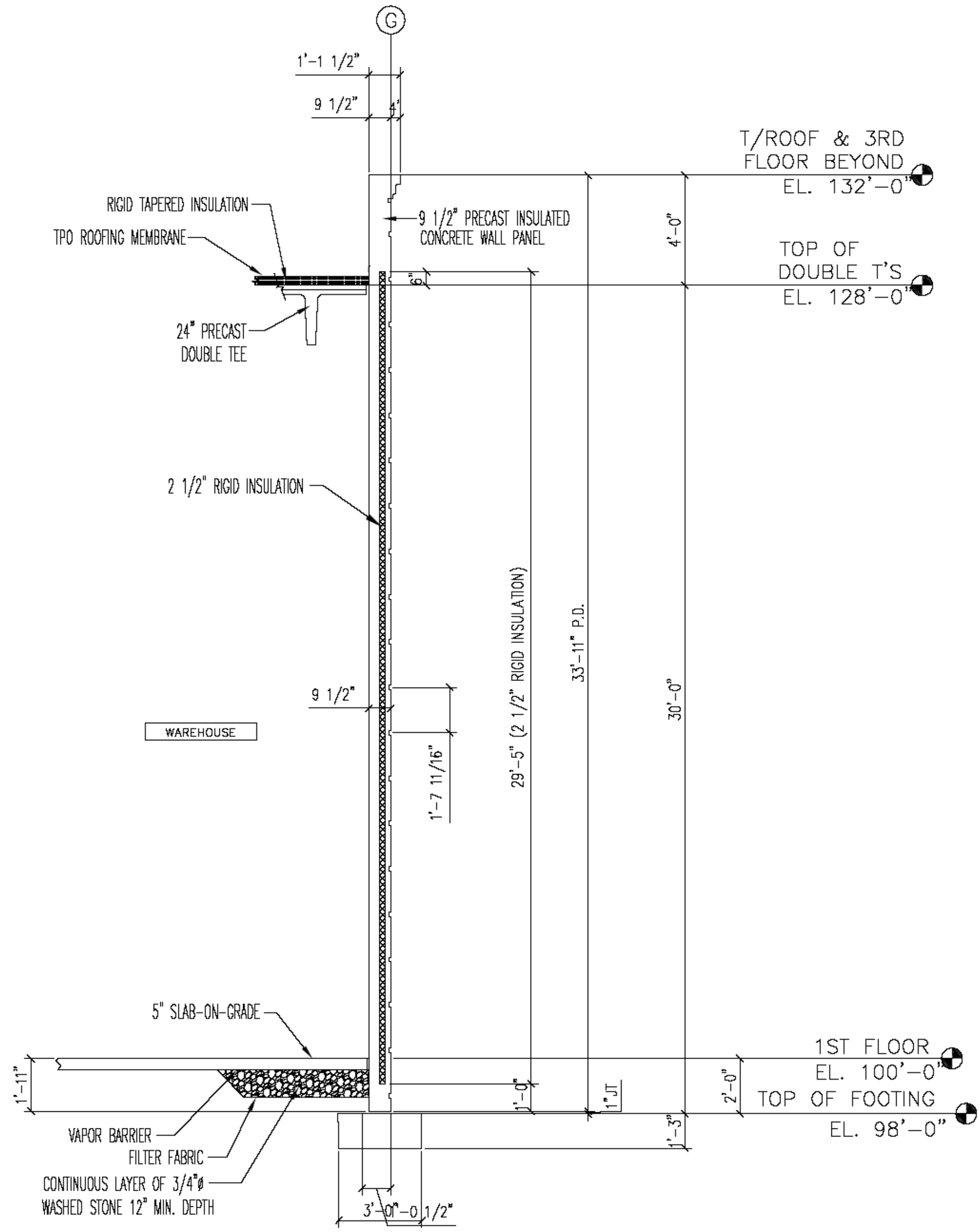
SECTION B – B

SCALE 3/8" = 1'-0"



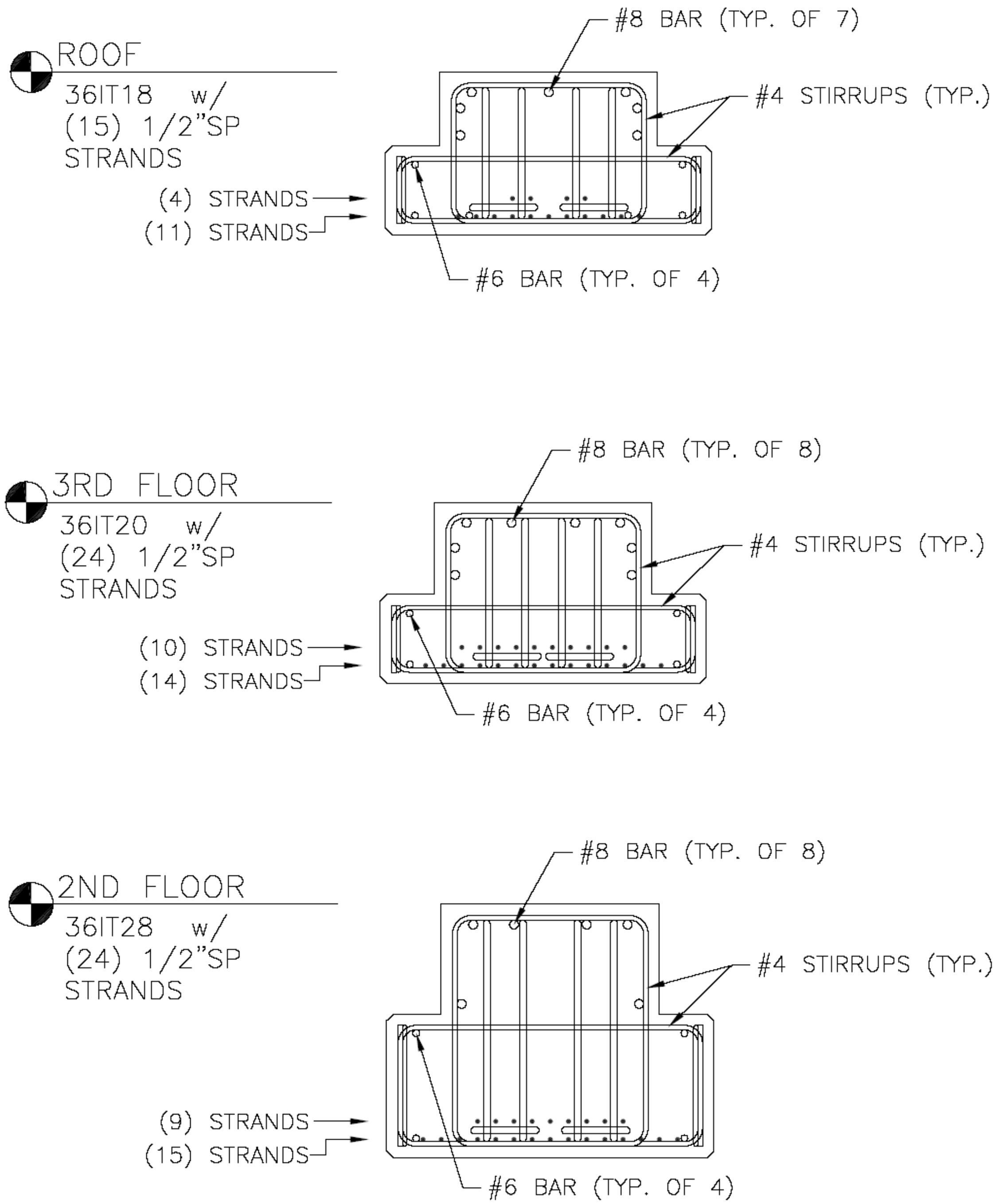
SECTION C – C

SCALE 3/8" = 1'-0"



SECTION D – D

SCALE 3/8" = 1'-0"



TYPICAL IT BEAM SECTIONS

APPENDIX B

(Analyses ~ Spot Checks ~ Designs)

WIND ANALYSIS

(Method 2: Analytical Procedure)

Centre Region Council of Governments
CODE ADMINISTRATION

Home

New Construction

Existing Structures

- Rental Housing
- Fire Safety
- Fire Protection Systems
- FAQs

Administration

Inspections

FAQs

Contact

«back

BASE DESIGN CRITERIA FOR CENTRE REGION CODE COMMERCIAL CONTSTRUCTION

Ground Snow Load (Pg) = 40 PSF

Basic Wind Speed = 90 MPH

Seismic considerations:

- .2 spectral response acceleration for site class B = .17
- 1.0 sec spectral response acceleration for site class B = .06

- Home
- New Construction
- Existing Structures
- Administration
- Inspections
- FAQs
- Contact
- top

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

By ______ Date ______

Page 1 of 11

COMMENTS: WIND ANALYSIS

Project

Ckd By ______ Date ______

[REF. ASCE 7-05 ~ CH. 6]

CHECK TO SEE WHICH METHOD(S) ARE PERMITTED FOR ANALYSIS

- [x] METHOD 1 CHECK → [6.4.1.1]
  1. BLDG. IS A SIMPLE DIAPHRAGM BLDG. ✓OK
  2. MEAN ROOF HEIGHT $(h) \le 60$ FT. ✓OK
  3. BLDG. IS ENCLOSED AND CONFORMS TO SECTION 6.5.9.3 ✓OK
  4. BLDG. IS REGULAR SHAPED ✓OK
  5. BLDG. IS NOT FLEXIBLE ✓OK
  6. BLDG. COMPLIES ✓OK
  7. BLDG. IS SYMMETRICAL W/ FLAT ROOF ✓OK
  8. BLDG. COMPLIES ✓OK

CONCLUSION: PER SECTION 6.4.1.1, METHOD 1 - SIMPLIFIED PROCEDURE MAY BE USED TO ANALYZE STRUCTURE.

- [x] METHOD 2 CHECK → [6.5.1]
  1. BLDG. IS REGULAR SHAPED ✓OK
  2. BLDG. COMPLIES ✓OK

CONCLUSION: PER SECTION 6.5.1, METHOD 2 - ANALYTICAL PROCEDURE PERMITTED AND WILL BE USED FOR ANALYSIS

METHOD 2:

- BASIC WIND SPEED (V) — $V = 90$ MPH
  - THIS VALUE TAKEN FROM CENTRE REGION CODE WEBSITE
  - NOTE: FIG. 6-1 OF ASCE 7-05 YIELDS SAME VALUE
- $K_d = 0.85$ FOR BUILDING STRUCTURES — $K_d = 0.85$
  - DETERMINE FROM TABLE 6-4
- IMPORTANCE FACTOR (I) — $I = 1.00$
  - PER TABLE 6-1 FOR BUILDING CATEGORY II
- TOPOGRAPHIC FACTOR (K_{zt}) — $K_{zt} = 1.0$
  - PER 6.5.7.2, $K_{zt} = 1.0$
- VELOCITY PRESSURE EXPOSURE COEFFICIENT (K_z OR K_h)
  - EXPOSURE C AS PER 6.5.6.3
  - TABLE 6-3
  - K_h DET. AT HEIGHT h = MEAN ROOF HT. (BY LINEAR INTERPOLATION IF NEC.)

$K_{40} \Rightarrow$ 40 1.04; 50 1.09 → $K_h = K_{40} = 1.04$ — $K_h = 1.04$

CONT.

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(CONT.) WIND ANALYSIS[2]

-

HEIGHT (FT.)	K_z
0-15	0.85
20	0.90
25	0.94
30	0.98
40	1.04
50	1.09

* NOTE: EXPOSURE C HAS SAME K_z VALUES FOR MWFRS AND C&C

- VELOCITY PRESSURE (q_z OR q_h)
  - PER 6.5.10,

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I \quad (\text{lb}/\text{ft}^2)$$ [EQ. 6-15]

(EX. CALC.)

(0-15) $q_z = 0.00256(0.85)(1.0)(0.85)(90_{\text{MPH}})^2(1.0)$

$q_{z_{0-15}} = 14.982$ PSF → SAY 15.0 PSF

HEIGHT (FT.)	K_z	q_z (PSF)
0-15	0.85	15.0
20	0.90	15.9
25	0.94	16.6
30	0.98	17.3
*→ 40	1.04	18.3
50	1.09	19.2
43.5 (3'-6 PARAPET)	1.0575	18.6 = q_p
32 (4'-0 PARAPET WAREHOUSE)	0.992	17.5 = q_p
45 (5'-0 PARAPET)	1.075	18.9 = q_p

*→ MEAN ROOF HEIGHT (h) = 40'

∴ $q_h = 18.3$ lb/ft²

$q_h = 18.3$ lb/ft²
$K_h = 1.04$

CONT. →

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COMMENTS
(CONT.) WIND ANALYSIS[3]

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- GUST EFFECT FACTOR (G) → SEE PG. 9 OF WIND ANALYSIS → $G = 0.85$
  - PER 6.5.8.1
- ENCLOSURE CLASSIFICATION — ENCLOSED
  - PER 6.5.9 / 6.2 DEFINITION
- INTERNAL PRESSURE COEFFICIENT (GC_{pi}) — $GC_{pi} = \pm 0.18$
  - FIGURE 6-5, ENCLOSED BLDGS
- EXTERNAL PRESSURE COEFFICIENT(S) (C_p)
  - WINDWARD WALLS — $C_p = 0.8$ (W)
  - LEEWARD WALLS (PER FIG. 6-6)
    - → NORTH-SOUTH DIRECTION (WIND NORMAL TO 280 FT. WALL)
      $L = 195$ FT, $B = 280$ FT, $L/B = 0.696$ → (0-1) — $C_p = -0.5$ ($L_{N\text{-}S}$)
    - → EAST-WEST DIRECTION (WIND NORMAL TO 195 FT. WALL)
      $L = 280$ FT, $B = 195$ FT, $L/B = 1.436$ → (BY L.I.) — $C_p = -0.413$ (L_{EW})
  - SIDE WALLS — $C_p = -0.7$
- DESIGN WIND LOAD(S) (P)
  - PER 6.5.12.2.1

$$P = qGC_p - q_i(GC_{pi}) \quad (\text{lb}/\text{ft}^2) \qquad (\text{EQ. 6-17})$$

  - PER 6.5.12.2.4

(PARAPETS) → $P_p = q_p GC_{pn}$ (lb/ft^2) W: $GC_{pn} = +1.5$ (WINDWARD), $= -1.0$ (LEEWARD) (EQ. 6-20)

(EX. CALC.) ~ DESIGN WIND LOADS FROM 0-15 FT. ON WINDWARD SIDE OF NORTH-FACE (280 FT)

$$P = [(15\text{ PSF})(0.85)(0.8)] - [(18.3\text{ PSF})(\pm 0.18)]$$

$$P = \begin{matrix} 6.906 \text{ PSF} \\ 13.494 \text{ PSF} \end{matrix} \quad (\text{SELECT MAX. VALUE})$$

$$\therefore P_{0\text{-}15} = 13.5 \text{ PSF} \quad (\text{WINDWARD} \sim \text{N-S})$$

$$P = [(18.3\text{ PSF})(0.85)(-0.5)] - [(15.0)(\pm 0.18)] = \begin{matrix} -5.1 \text{ PSF} & (-0.18) \\ -10.5 \text{ PSF} & (+0.18) \end{matrix}$$

$$\therefore P_{0\text{-}15} = -10.5 \text{ PSF} \quad (\text{LEEWARD} \sim \text{N-S})$$

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COMMENTS (CONT.) WIND ANALYSIS[4]

⊥ 280'

NORTH - SOUTH DIRECTION

HEIGHT (FT.)	q_z (PSF)	$P_{WINDWARD}$ (PSF)	$P_{LEEWARD}$ (PSF)
0-15	15.0	13.5	-10.5
20	15.9	14.1	-10.6
25	16.6	14.6	-10.8
30	17.3	15.1	-10.9
40	18.3	15.7	$\boxed{-11.1}$ *
50	19.2	16.4	-11.2
PARAPET(S)			
(3'-6")** 43.5	q_p = 18.6	27.9	-18.6
(4'-0")** 32	q_p = 17.5	26.3	-17.5
(5'-0")** 45	q_p = 18.9	28.4	-18.9

q_h (PSF): $\boxed{18.3}$

G: $\boxed{0.85}$

C_p: W / L: +0.8 / -0.5

GC_{pi}: $\boxed{\pm 0.18}$

GC_{pn}: +1.5 (W), -1.0 (L)

** - IF APPLICABLE

* - LEEWARD P CONSTANT, USE VALUE AT q_h

⊥ 195'

EAST - WEST DIRECTION

HEIGHT (FT.)	q_z (PSF)	$P_{WINDWARD}$ (PSF)	$P_{LEEWARD}$ (PSF)
0-15	15.0	13.5	
20	15.9	14.1	
25	16.6	14.6	
30	17.3	15.1	
40	18.3	15.7	$\boxed{-3.13}$
50	19.2	16.4	
PARAPET(S)			
(3'-6") 43.5	q_p 18.6	27.9	-18.6
(4'-0") 32	q_p 17.5	26.3	-17.5
(5'-0") 45	q_p 18.9	28.4	-18.9

q_h (PSF): $\boxed{18.3}$

G: $\boxed{0.85}$

C_p: W / L: +0.8 / -0.4

GC_{pi}: $\boxed{\pm 0.18}$

GC_{pn}: +1.5 (W), -1.0 (L)

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COMMENTS (CONT.) WIND ANALYSIS 5

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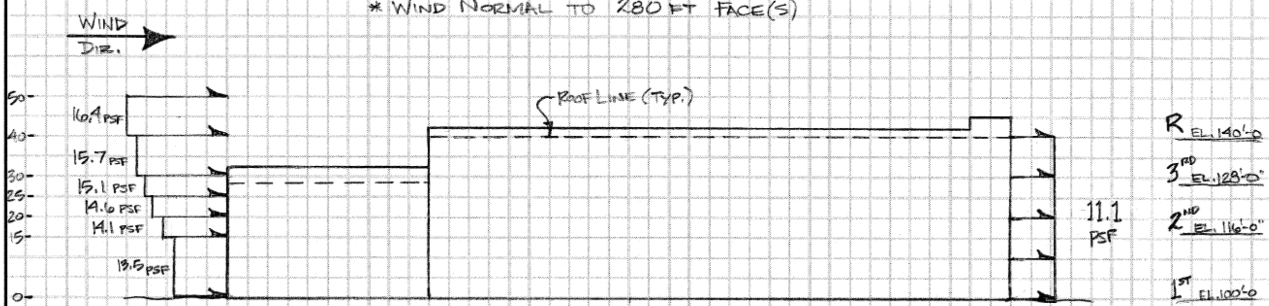
NORTH-SOUTH DIRECTION

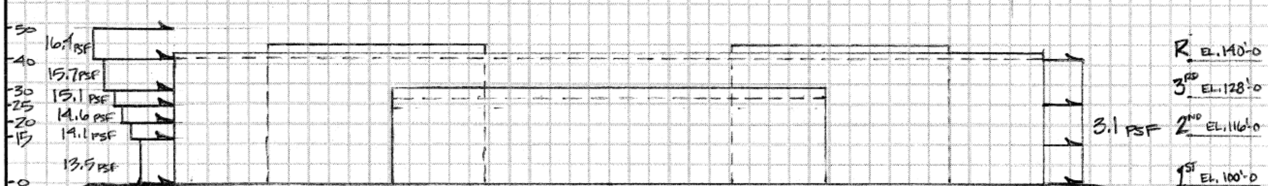
* WIND NORMAL TO 280 FT FACE(S)



NOTE: VALUES ASSUMED FOR EACH WALL 'NORMAL' TO.

EAST-WEST DIRECTION

* WIND NORMAL TO 195 FT FACE(S)



NOTE: VALUES ASSUMED FOR EACH WALL 'NORMAL' TO.

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- Tributary widths for each floor (see Appendix B)

→ 1st Floor (El. 100'-0) $T_W = 7'-0''$

→ 2nd Floor (El. 116'-0) $T_W = 14'-0''$

→ 3rd Floor (El. 128'-0) $T_W = 12'-0$

→ Roof (El. 140'-0) T_W = Varies (6'-0 + parapet)

1. 3'-6" parapet ~ $T_W = 9'-6$
2. 4'-0" parapet ~ $T_W = 10'-0$
3. 5'-0" parapet ~ $T_W = 11'-0$

- Story Force and Overturn Moment

1. Warehouse (Section D-D)

→ Tributary width = 14' + 4' parapet = 18'
* TW → from 14' to 132'-0 elev.

			$\times 140'/1000$	N-S	E-W
14-15	1'-0 × 13.5 psf =	13.5 plf		1.89^k	0.54^k
15-20	5'-0 × 14.1 psf =	70.5 plf		9.87^k	2.82^k
20-25	5'-0 × 14.6 psf =	73.0 plf		10.22^k	2.92^k
25-28	3'-0 × 15.1 psf =	45.3 plf		6.342^k	1.812^k
Parapet →	4'-0 × 26.3 psf =	105.2 plf		14.728^k	4.208^k
					+2.232^k (leeward)

North-South Dir. Story Force = Σ = 43.05^k

East-West Dir. Story Force = Σ = 14.53^k

Overturning moment → $M_{OT} = 43.05^k(28')$

M_{OT} = 1205.4 ft-k

M_{OT} = 406.9 ft-k

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2. North - South Direction

→ 1ST Floor: 7'-0 × 13.5 psf = 94.5 plf × 280'/1000 = 26.46k

→ 2ND Floor:

8-15	7'-0 × 13.5 psf = 94.5 plf × 280'/1000	= 26.46k
15-20	5'-0 × 14.1 psf = 70.5 plf	= 19.74k
20-22	2'-0 × 14.6 psf = 29.2 plf	= 8.176k
~~Leeward~~	14'-0 × 11.1 psf = 155.4 plf	= 43.512k (NOT ADDED)

Story Force = Σ = 54.4k

→ 3RD Floor:

22-25	3'-0 × 14.6 psf = 43.8 × 280'/1000	= 12.26k
25-30	5'-0 × 15.1 psf = 75.5	= 21.14
30-34	4'-0 × 15.7 psf = 62.8	= 17.584

Story Force = Σ = 51.0k

→ Roof:

34-40	6'-0 × 15.7 psf = 94.2 plf × 280'/1000	= 26.38k
40-45	5'-0 parapet × 28.4 psf = 142 plf × 280	= 39.76k

Story Force = Σ = 66.1k

• Overturning Moment (M_{OT})

→ 1ST Floor:
→ 2ND Floor: 54.4k × 16'-0 = 870.4 ft-k
→ 3RD Floor: 51.0k × 28'-0 = 1428 ft-k
→ Roof: 66.1k × 40'-0 = 2645.4 ft-k

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3. EAST - WEST DIRECTION (3'-6 PARAPET)

→ 1ST FLOOR: 7'-0 × 13.5 PSF = 94.5 PLF × 195'/1000 = 18.43^k

→ 2ND FLOOR:

8-15	94.5 PLF	× 155'/1000 =	14.65^k
15-20	70.5 PLF		10.93^k
20-22	29.2 PLF		4.53^k

STORY FORCE = Σ = 30.1^k
(× 16'-0) → 481.6 FT-K = M_{OT}^2

→ 3RD FLOOR:

22-25	43.8 PLF	× 155'/1000 =	6.79^k
25-30	75.5 PLF		11.70^k
30-34	62.8 PLF		9.73^k

STORY FORCE = Σ = 28.2^k
(× 28'-0") → 790.3 FT-K = M_{OT}^3

→ ROOF:

34-40	94.2 PLF	× 155/1000 =	14.60^k
40-43.5	97.7 PLF		15.14^k

STORY FORCE = Σ = 29.74^k
(× 40'-0) → 1189.5 FT-K = M_{OT}^R

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COMMENTS
W.A. ~ CALC. OF GUST EFFECT FACTOR (G)
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6.5.8 GUST EFFECT FACTOR

$$G = 0.925\left[\frac{1+1.7 g_Q I_{\bar{z}} Q}{1+1.7 g_v I_{\bar{z}}}\right]$$

WHERE: AS DEFINED BY 6.5.8.1

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.2\left(\frac{33}{24}\right)^{1/6}$$

$\bar{z} = 0.6h = 0.6(40') = 24$ FT ($\geq \bar{z}_{min} = 15$ FT; TBL. 6-2) OK

$c = 0.2$ FOR EXPOSURE C (TABLE 6-2)

$$\underline{I_{\bar{z}} = 0.2109}$$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{195+40}{469}\right)^{0.63}}} = \sqrt{\frac{1}{1.408}}$$

B = HORIZ. DIM. OF BLDG., NORMAL TO WIND DIR. (USE MINIMUM FOR LARGER G VALUE) B = 195 FT

h = 40 FT (MEAN ROOF HT. ~ MRH)

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 500\left(\frac{24}{33}\right)^{1/5}$$

$\bar{z} = 24$ FT (CALC. ABOVE)

$\ell = 500$ FT, $\bar{\epsilon} = 1/5.0$ } (EXPOSURE C, TABLE 6-2)

$L_{\bar{z}} = 469.148$

$$\underline{Q = 0.843}$$

$$\underline{g_Q = g_v = 3.4}$$

$$G = 0.925\left[\frac{1+1.7(3.4)(0.211)(0.843)}{1+1.7(3.4)(0.211)}\right]$$

$$\boxed{G = 0.845}$$

WILL USE G = 0.85 AS DEFINED BY 6.5.8.1 AND CALC'D HERE

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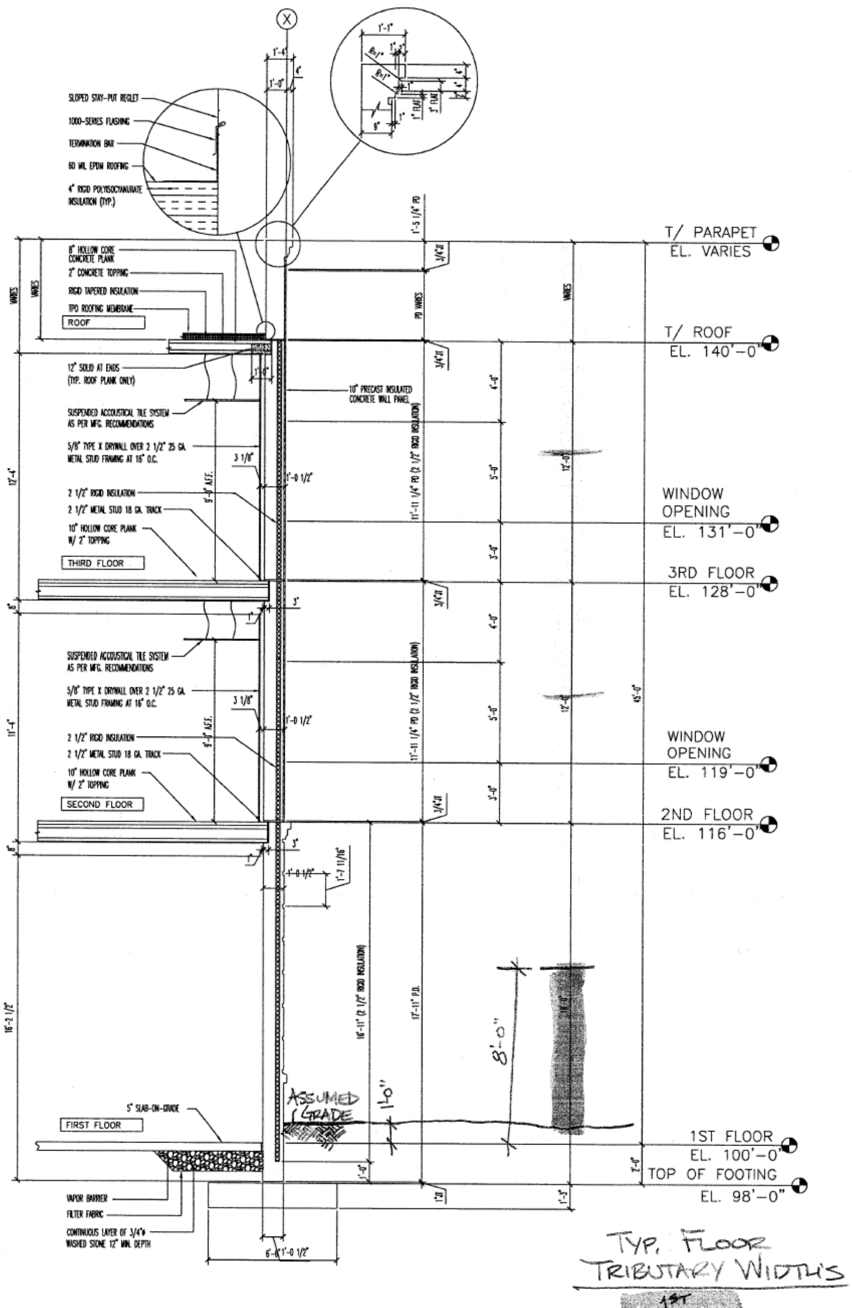
FLOOR TRIB. WIDTHS



WALL SECTION
N.T.S

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Project PSU-ARL BLDG #5

COMMENTS
WIND LOADS (REF ASCE 7-05 CH6)

FROM CENTRE REGION CODE: BASIC WIND SPEED = 90 MPH
IMPORTANCE FACTOR, $I = 1.00$ (TABLE 6-1)
EXPOSURE CATEGORY = C (SECT 6.5.6.3)
ADJUSTMENT FACTOR, $\lambda = 1.53$ (FIG 6-2)

$$p_s = \lambda K_{zt} I p_{s30}$$

$K_{zt} = 1.0$; $p_{s30} = 12.8$ (FIG 6-2)

$p_s = (1.53)(1.0)(1.0)(12.8\text{ PSF}) =$ **19.6**, SAY 20 PSF

ENGINEER'S VALUE (WIND PRESSURE)

X-DIRECTION (EAST-WEST)

WIDTH = 195'
2ND FLOOR TRIB HT = $\frac{1}{2}(18') + \frac{1}{2}(12') = 15'$
3RD FLOOR TRIB HT = $\frac{1}{2}(12') + \frac{1}{2}(12') = 12'$
ROOF TRIB HT = $\frac{1}{2}(12') + 4'$ PARAPET = 10'
LOADS:
2ND FLOOR = 20 PSF × 15' × 195' ÷ 1000 = 58.5 K
3RD FLOOR = 20 PSF × 12' × 195' ÷ 1000 = 46.8 K
ROOF = 20 PSF × 10' × 195' ÷ 1000 = 39.0 K

Y-DIRECTION (NORTH-SOUTH)

WIDTH = 280'
2ND FLOOR TRIB HT = 15'
3RD FLOOR TRIB HT = 12'
ROOF TRIB HT = 10'
LOADS:
2ND FLOOR = 20 PSF × 15' × 280 ÷ 1000 = 84.0 K
3RD FLOOR = 20 PSF × 12' × 280' ÷ 1000 = 67.2 K
ROOF = 20 PSF × 10' × 280' ÷ 1000 = 56.0 K

SEISMIC ANALYSIS

(Equivalent Lateral Force Procedure)

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COMMENTS Seismic Analysis
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Project

[Ref IBC 2006 ~ Section 1613 and ASCE 7-05 ~ Ch. 11 & 12]

Seismic considerations:

- (Per Centre Region Code) → $S_s = 0.17$ $S_1 = 0.06$ ← used for analysis
  - * Compare to ASCE 7-05 → ($S_s \approx 0.14$ Fig. 22-1 $S_1 \approx 0.05$ Fig 22-2)
- Site class → D
  (Per 1613.5.2). Soil properties unknown ∴ use site class D
- $S_{MS} = F_a S_s$ [Eqn 16-37]
  $= 1.6(0.17)$ [Tbl 1613.5.3(1)]
  $\boxed{S_{MS} = 0.272}$ (0.221)
- $S_{M1} = F_v S_1$ [Eqn 16-38]
  $= 2.4(0.06)$ [Tbl 1613.5.3(2)]
  $\boxed{S_{M1} = 0.144}$ (0.12)
- $S_{DS} = \frac{2}{3}(S_{MS})$ [Eqn 16-39]
  $\boxed{S_{DS} = 0.181\bar{3}}$ (0.1493)
- $S_{D1} = \frac{2}{3}(S_{M1})$ [Eqn 16-40]
  $\boxed{S_{D1} = 0.096}$ 0.08
- Seismic Design Category
  - * Building occupancy → Type II
  - Short-period response: [Tbl 1613.5.6(1)]
    $II \Leftrightarrow S_{DS} = 0.181 \Rightarrow \underline{\underline{B}}$
  - 1 sec period response: [Tbl 1613.5.6(2)]
    $II \Leftrightarrow S_{D1} = 0.096 \Rightarrow \underline{\underline{B}}$

∴ Use Design Category B

⇓

Equivalent Lateral Force Procedure permitted and used for analysis

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COMMENTS
(CONT.) SEISMIC ANALYSIS[2]

[REF. ASCE 7-05]

§12.8 EQUIVALENT LATERAL FORCE PROCEDURE $(V = C_s W)$

- $S_s = 0.17$ $S_{DS} = 0.181$ S.D.C. $\boxed{B}$
  $S_1 = 0.06$ $S_{D1} = 0.096$

- RESPONSE MODIFICATION COEFFICIENT (R) [TABLE 12.2-1]
  - USING: ORDINARY PRECAST SHEAR WALLS (AS BEARING WALL SYSTEM) ⇒ $R = 3$

- IMPORTANCE FACTOR (I) [TABLE 11.5-1]
  - OCCUPANCY CAT. II ⇒ $I = 1.0$

- PERIOD DETERMINATION
  - PER §12.8.2 : AS AN ALTERNATIVE TO DETERMINING THE ACTUAL FUNDAMENTAL PERIOD (T); AN APPROX. BUILDING PERIOD (T_a) WILL BE CALC'D. USING §12.8.2.1

  12.8.2.1 ~ APPROX. FUNDAMENTAL PERIOD

  $T_a = C_t h_n^x$ (s) $h_n = 47$ FT. $C_t = 0.02$ $x = 0.75$ [TBL. 12.8-2]

  ↳ $T_a = 0.02(47\text{ FT.})^{0.75}$

  $\underline{T_a = 0.36\text{ s}}$

  → $T_a = 0.36\text{s} < 6.0\text{s} = T_L$ [FIGURE 22-15]

- SEISMIC RESPONSE COEFFICIENT (C_s) PER §12.8.1.1
  - $(T < T_L)$

  $$C_s = \frac{S_{D1}}{T(R/I)}$$ [EQN. 12.8-3]

  $$C_s = \frac{0.096}{0.36(3/1)} = 0.08\bar{9}$$ $\boxed{C_s = 0.089}$

  $$C_s \geq \frac{0.5 S_1}{(R/I)} = \frac{0.5(0.06)}{3} = 0.001 \quad (< 0.089 \text{ OK}✓)$$

  $$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)} = \frac{0.181}{\left(\frac{3}{1}\right)} = 0.060\bar{3}$$

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COMMENTS
(CONT.) SEISMIC ANALYSIS[3]

- EFFECTIVE SEISMIC WEIGHT (W) [PER § 12.7.2]
  → INCLUDES:
  W = TOTAL DEAD LOAD (+) PLUS
  1. MINIMUM OF 25% OF FLOOR LIVE LOAD IN STORAGE AREAS
  2. PARTITION PROVISIONS OF §4.2.2 OR MINIMUM WEIGHT OF 10 PSF OF FLOOR AREA (USE GREATER OF ...)
  3. TOTAL OPERATING WEIGHT OF PERMANENT EQUIP.
  4. WHERE P_f (FLAT ROOF SNOW LOAD) EXCEEDS 30 PSF, 20% OF THE UNIFORM DESIGN SNOW LOAD.
  (NOTE: #4 N/A, P_f < 30 PSF)

1. COLUMNS →
24" × 24" COL'S

WT.: 2 FT × 2 FT × 150 PCF/1000 = 0.6 K/FT — COL. = 0.6 K/FT

TRIB: ROOF = 6' ~ 3RD = 12' ~ 2ND = 15'

2. BEAMS →

A. INVERTED TEE'S (IT):

ROOF (36IT18) = 0.575 K/FT
TOTAL LENGTH OF = 783 FT (782.723 ACTUAL) — R = 450K

3RD (36IT20) = 0.625 K/FT
TOTAL LENGTH OF = 783 FT — 3RD = 490K

2ND (36IT28) = 0.900 K/FT
TOTAL LENGTH OF = 660 FT (656.952) — 2ND = 594K

B. RECTANGULAR (RB):
* ALL RB'S ASSUMED TO BE 12 × 24
ω = 0.3 K/FT

ROOF: 16 FT — R = 4.8K

3RD: 16 FT — 3RD = 4.8K

2ND: 72 FT — 2ND = 21.6K

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COMMENTS
(CONT.) SEISMIC ANALYSIS[4]

Ckd By ______ Date ______ Project ______

3. WALLS → (ESTIMATED)

A. LOW ROOF AREA

9 1/2" − 2 1/2" INSUL. = 7" CONC. (0.583 FT)

(E-W) 0.583 FT × 140 FT (L) × 34 FT (H) × 150 #/FT³ /1000 = 416.5^K (12.24 K/FT)

(N-S) 0.583 FT × 40 FT × 34 FT × 150 #/FT³ /1000 = 119.0^K/WALL (QTY. 2) (3.498 K/FT)

B. E-LINE AREA

(E-W) 0.583 FT × 280 FT × 45.5 FT × 150 PCF/1000 = 1114.75^K (24.486 K/FT)

C. 1-LINE & 9-LINE

(N-S) 12 1/2" − 2 1/2" INSUL. = 10" CONC. (0.833 FT)

0.833 FT × 130 FT × 45.5 FT × 150 PCF/1000 = 739.375/WALL (QTY. 2) (16.2435 K/FT /WALL)

D. A-LINE AREA

0.833 FT × (280 FT + 4(15 FT)) × 47 FT × 150 PCF/1000 = 1997.5^K (42.483 K/FT)

E. MISC. WALLS (9" THICK)

0.75 FT × 195 FT × 44 FT × 150 PCF/1000 = 965.25^K (21.9375 K/FT)

	MAIN	LOW ROOF
TOTAL ⇒	121.5 K/FT	19.24 K/FT

* NOTE: SEE STAIRS FOR STAIR WALL WEIGHTS

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COMMENTS
(CONT.) SEISMIC ANALYSIS 5

4. HOLLOW CORE PLANK (HCP)
- ROOF: 8 IN → 62 PSF × 39,050 SF/1000 = 2421.1^k R
- 3RD: 10 IN → 67.5 PSF × 37,870 SF/1000 = 2556.23^k 3RD
- 2ND: 12 IN → 72.5 PSF × 33,150 SF/1000 = 2403.38^k 2ND

5. MISCELLANEOUS
- 2 IN. TOPPING = (2/12)(150 PCF) = 25 PSF
  - 3RD: 37870 SF (25)/1000 = 946.75 = 947^k 3RD
  - 2ND: 33150 SF (25)/1000 = 828.75 = 829^k 2ND
- ADDITIONAL 12 PSF (MISC.)
  - ROOF: 39050 = 468.6 = 470^k R
  - 3RD: 37870 × 12 PSF/1000 = 454.4 = 454^k 3RD
  - 2ND: 33150 = 397.8 = 398^k 2ND

6. LOW ROOF AREA (APPLY TO 3RD LEVEL)
- IT BEAM (ASSUMING SELF WT ω = 0.900 K/FT)
  41'-0 (0.900 K/FT) = 36.9 → 37^k (LR → 3RD)
- DOUBLE TEE'S (DT):
  47 PSF (5915 SF)/1000 = 278.005^k → 278^k
- 2" TOPPING (25 PSF):
  25 PSF (5915 SF)/1000 = 147.875^k → 148^k

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COMMENTS
(CONT.) SEISMIC ANALYSIS

7. STAIRS →

A. WALLS

9": $(0.75_{FT})[2(26.5\ FT) + 2(10\ FT)](150\ \#/FT^3)/_{1000} = 8.2125\ K/FT \times 4$ SETS OF STAIRS

(32.85 K/FT)

TOTAL WT. OF STAIR WALLS
1545 K

B. STAIRS

VALUES USED FROM EXISTING CALCS. (+ 175 PSF LOAD FOR RISERS & LANDINGS)

▲ 3RD → 420 SF (175 PSF)/1000 = 73.5 K

▲ 2ND → 942 SF (175 PSF)/1000 = 165 K

→ TOTAL LOADS

▲ ROOF: (TRIB = 6 FT)

			(KIP)
COLUMNS	0.6 K/FT (6 FT) × 18 COL.	=	64.8
BEAMS	450 K + 4.8 K	=	454.8
WALLS	121.5 K/FT (6 FT)	=	729.0
8" HC PLANK	2421.1 KIP	=	2421.1
MISC. (12 PSF)	470 KIP	=	470.0
STAIR WALLS	32.85 K/FT (6 FT)	=	197.1

WT. = 4336.8 K @ ROOF LVL.

▲ 3RD: (TRIB. = 12 FT)

COL.	0.6 K/FT (12 FT) × 24 COL.	=	172.8
BM.	490 K + 4.8 K + 37 K	=	531.8
W.	(121.5 K/FT + 19.24 K/FT)(12 FT)	=	1688.88
10" HC		=	2556.23
MISC.	947 K + 454 K + 148	=	1549
S.W.	32.85 (12 FT)	=	394.2
STAIRS		=	73.5 K
DT		=	278

WT. = 7244.4 K @ 3RD FLR. LVL.

▲ 2ND: (TRIB. = 15 FT)

COL.	0.6 (15)(20)	=	180
BM	594 + 21.6	=	615.6
W	(121.5 + 19.24)(15)	=	2111.1
12" HC		=	2403.38
MISC.	829 + 398	=	1227
S.W.	32.85 (15)	=	492.75
STAIRS		=	165

7194.8 K @ 2ND FLR. LVL

TOTAL BLDG. WT. = 18,776 K

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COMMENTS
(CONT.) SEISMIC ANALYSIS

▲ SEISMIC BASE SHEAR [PER § 12.8.1]

★ $V = C_s W$ WHERE $C_s = 0.089$

$V_R = 0.089(4336.8^k)$ ⇒ $V_R = 386^k$ (@ ROOF)

$V_3 = 0.089(7244.4^k)$ $V_3 = 645^k$ (@ 3RD FLR.)

$V_2 = 0.089(7194.8^k)$ $V_2 = 641^k$ (@ 2ND FLR.)

$V_{BASE} = 0.089(18776^k)$ $V = 1672^k$ (BASE SHEAR)

OVERTURNING MOMENTS

ROOF = 16212 FT-K
3RD = 18060 FT-K
2ND = 10256 FT-K
TOTAL = 44528 FT-K

8 OF 8

Seismic Force Distribution Summary for Penn State University- ARL Building #5

Summary of building floor/roof diaphragms and weights:

Diaphragm Level	Diaphragm Elevation (ft)	Diaphragm Weight (kips)
Roof	42.00	3583.7
3rd Flr	30.00	6029.3
2nd Flr	18.00	4920.5
0	0.00	0.0
0	0.00	0.0
0	0.00	0.0
0	0.00	0.0
Ground	0.00	0.0

$\boxed{14533.49}$ **kips -- Total building weight (W)**

Summary of Design Code Requirements and resultant Seismic Force Distribution to diaphragms:

Governing Design Code(s): IBC 2006, ASCE7-05, AND CENTRE REGION CODE

Occupancy Category: I
Seismic Use Group: I
Occupancy Importance Factor: 1.0
Site Classification: D

0.2 second spectral response acceleration (S_s):	0.170	1.0 second spectral response acceleration (S_1):	0.06
F_a:	1.600	F_v:	2.400
S_{MS}:	0.272	S_{M1}:	0.144
S_{DS}:	0.182	S_{D1}:	0.096
Seismic Design Category, based on S_{DS}:	B	Seismic Design Category, based on S_{D1}:	B
		Governing Seismic Design Category:	**B**

Seismic Analysis performed using code perscribed Equivalent Lateral Force Procedure

Calculated Seismic Response Coefficient (C_s): 0.0607
Seismic Base Shear (V): 882.9 kips, distrubuted to diaphragms as shown below:

SPOT CHECKS

(Column ~ Hollow-Core Slab ~ Rect. Beam)

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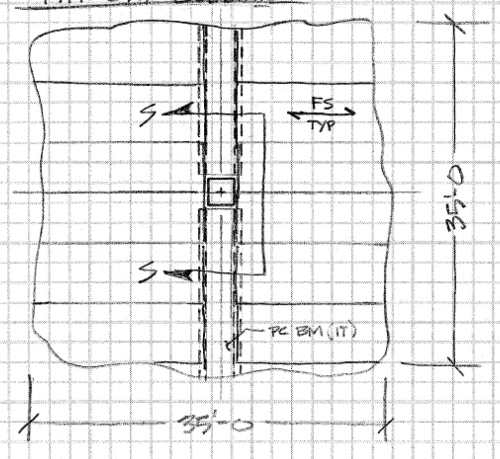
COMMENTS SPOT CHECK (COLUMN)

TYP. BAY COLUMN



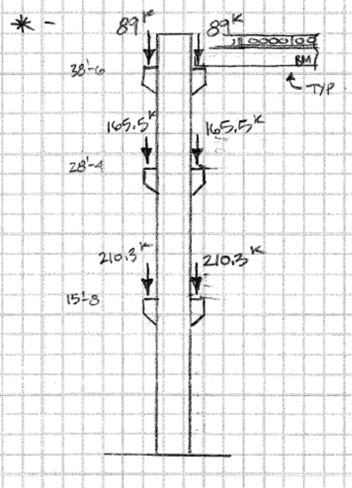
TRIB. AREA = 35 FT × 35 FT => TA = 1225 SF
COL. HT. = 42 FT. (TYP.)

* COLUMN LOADING TAKEN FROM CONCISE BEAM SUMMARY REPORT. LOAD VALUE AT GIVEN FLOOR SET EQUAL TO BEAM END REACTION FOR BEAM DESIGNED AT THAT LEVEL. (SEE PAGE 3 OF SUMMARY REPORT FOR EACH BEAM. IN DESIGN PORTION OF THIS RPT.)
- VALUE HIGHLIGHTED IN GREEN

* ASSUME: NO SIDE SWAY; COLUMN ACTS LIKE SHORT COL. W/ NO SLENDERNESS EFF.

$f'_c = 6000$ PSI $f_y = 60000$ PSI
$E_s = 29000$ KSI

*-



SECTION S-S

MAX. LOAD(S)
@ ERECTION: $P_u = 464.8^K \rightsquigarrow 465^K$
IN-SERVICE: $P_u = 929.6^K \rightsquigarrow 930^K$

* NOTE: IGNORED LOAD EFFECTS ON COLUMN DUE TO ERECTION LOADING.

$\phi P_o = 2132^K > 930^K = P$
∴ ✓OK

CONCLUSION:

FROM THE INTERACTION CURVE ON THE FOLLOWING PAGE WE CAN SEE THAT WHEN THE COLUMN IS UNDER COMPRESSION LOADING ONLY $P_o = 2132$ KIPS WHICH IS MUCH GREATER THAN THE MAX LOAD IN SERVICE OF $P = 930$ KIPS, THUS THE COLUMN HAS SUFFICIENT STRENGTH TO HANDLE THE LOAD(S).

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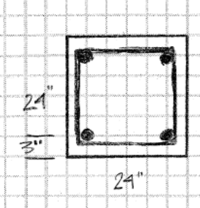
COMMENTS (For Spot Check)
Construction of Interaction Curve for Reinf. Precast Col.

$f'_c = 6000$ psi $\qquad \beta_1 = 0.85 - 2(0.05) = 0.75 \qquad y_t = 12''$

$f_y = 60000$ psi $\qquad d = 24'' - 3'' = 21''$

$E_s = 29000$ ksi $\qquad d' = 3'' \qquad 0.85 f'_c = 0.85(6) = 5.1$ ksi



24" SQ. COL.
(4) #11
#4 TIES @ 22" o.c.
1¼" COVER TYP.

$A_g = 24'' \times 24'' = 576 \text{ in}^2$

$A_s = A'_s = 4(1.56 \text{ in}^2) = 6.24 \text{ in}^2/2 = 3.12 \text{ in}^2$

▲ DET. P_o (NO PRESTRESSING STEEL)

$$P_o = P_n = 0.85 f'_c(A - A'_s - A_s) + (A'_s + A_s) f_y$$
$$= 5.1 \text{ ksi}(576 \text{ in}^2 - 6.24 \text{ in}^2) + (6.24 \text{ in}^2)(60 \text{ ksi})$$
$$= 3280.18 \text{ kips} \rightarrow \qquad P_o = 3280^k$$

$$\phi P_o = \underset{\text{tied}}{0.65}(3280^k) = 2132.11 \text{ kips} \rightarrow \qquad \phi P_o = 2132^k$$

▲ DET. P_{nb} & M_{nb}

$d_t = d = 21'' \qquad a = \beta_1 c = 0.75(12.6'') = 9.45''$

$c = 0.6 d_t = 12.6'' \qquad f'_s = f_y = 60$ ksi $\qquad y' = a/2 = 4.725''$

$A_{comp} = ab = 9.45 \text{ in}(24 \text{ in}) = 226.8 \text{ in}^2$

(w/ No Prestress Steel)

$$P_{nb} = (A_{comp} - A'_s)(0.85 f'_c) + \underbrace{A'_s f'_s - A_s f_s}_{0}$$
$$= (226.8 \text{ in}^2 - 3.12 \text{ in}^2)(5.1)$$
$$= 1140.77^k \rightarrow \qquad P_{nb} = 1141^k$$

$$\phi P_{nb} = 0.65(1141^k) = 741.5^k \rightarrow \qquad \phi P_{nb} = 742^k$$

$$M_{nb} = (A_{comp} - A'_s)(y_t - y')(0.85 f'_c) + A'_s f'_y(y_t - d') + A_s f_s(d - y_t)$$
$$= (226.8 \text{ in}^2 - 3.12 \text{ in}^2)(12'' - 4.73'')(5.1) + 3.12(60)(12-3) + 3.12(60)(21-12)$$
$$= 11668.7 \text{ k-in}$$
$$(972.391 \text{ k-ft})$$

$M_{nb} = 973$ k-ft

$\overset{0.65}{\phi} M_{nb} = 632$ k-ft

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▲ DET. M_o (NEGLECT COMPRESSION REINF, CONSERV.)

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{3.12\text{ in}^2 (60\text{ ksi})}{5.1\,(24'')} = 1.529''$$

$a = 1.53$ in

$$M_o = A_s f_y (d - a/2) = 3.12(60)(21 - 1.53/2) = 3788.05 \text{ k-in} \quad (315.671 \text{ k-ft})$$

$M_o = 316$ k-ft

$$c = a/\beta_1 = 1.53/0.75 = 2.039''$$

$c = 2.04$ in

$$\epsilon_t = \frac{0.003(d-c)}{c} = \frac{0.003(21-2.04)}{2.04} = 0.027894$$

$\epsilon_t = 0.028$
$(\epsilon_t > 0.005 \Rightarrow \phi = 0.9)$

FOR $\phi = 0.9$

$$\phi M_o = 0.9(316 \text{ k-ft}) = 284.104 \quad (3409.24 \text{ k-in})$$

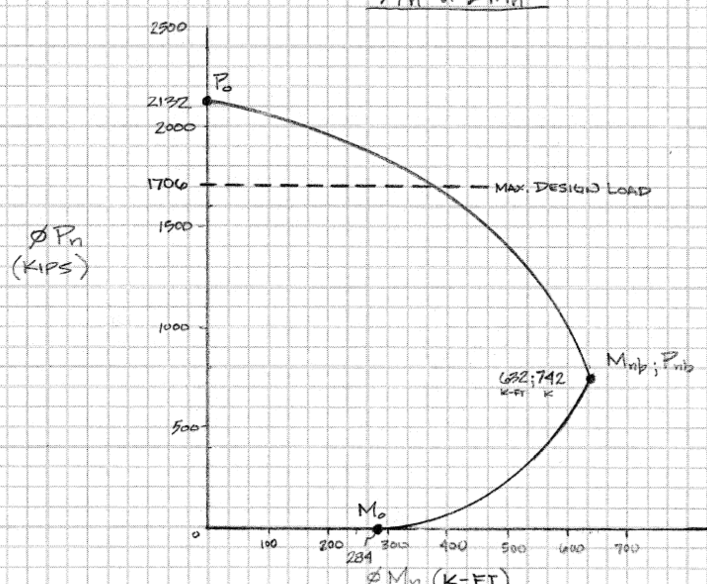
$\phi M_o = 284$ k-ft

▲ MAX DESIGN LOAD

$$= 0.80\,\phi P_o = 0.80(2132^k) = 1705.6$$

$P_n = 1706^k$

ϕP_n vs. ϕM_n



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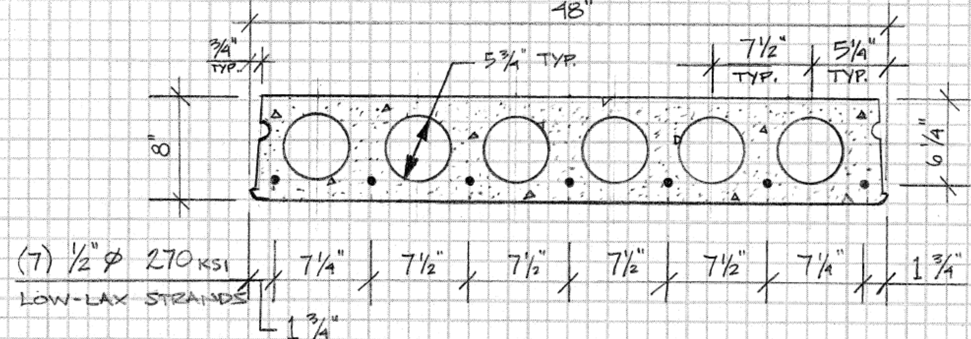
COMMENTS
SPOT CHECK ~ 8" HC SLAB @ ROOF LEVEL

Ckd By
Date
Project

TYPICAL 4' × 32'-10' × 8" HC



SECTION
(8" HOLLOW CORE)

CONCLUSION →

COMPARING THE RESULTS OF MY CALCS WITH THE RESULTS FROM THE CONCISE SUMMARY REPORT (CSR), YOU CAN SEE THAT THE VALUES ARE VERY SIMILAR. DISCREPENCIES MAY HAVE OCCURED DUE TO ROUNDING ERRORS (BOTH W/ MYSELF AND THE PROGRAM) OR DIFFERING VALUES.

* NOTE: SEE CONCISE SUMMARY REPORT FOLLOWING THE END OF THIS SPOT CHECK.

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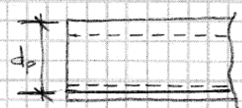
COMMENTS: SPOT CHECK 2

Ckd By ________ Date ________ Project

(7) 1/2" ϕ ~ 270 KSI (f_{pu}) LOW-LAX STRANDS



d_p = 8" − 1.75" = 6.25 IN

INITIAL STRESS = $0.70 f_{pu}$

f'_c = 5000 PSI

f'_{ci} = 3500 PSI

BEAM LENGTH = 32'-10"

→ CLEAR SPAN = 32'-6"

DEAD → MISC. = 12 PSF
SELF = 62 PSF
74 PSF

LIVE → LL_R = 20 PSF
SNOW = 30 PSF
USE 30 PSF (* ROOF PLANK, SNOW GOV.'S LIVE LOAD NOT LIKELY)

SOL. →

$$\phi M_n = \phi A_{ps} f_{ps} (d_p - a/2)$$

$$f_{ps} = f_{pu}\left[1 - \frac{\gamma_p}{\beta_1}\left(\rho_p \frac{f_{pu}}{f'_c}\right)\right]$$

W: 1. γ_p = 0.28 (LOW-LAX. STRANDS)

2. β_1 = 0.80 (f'_c = 5000 PSI)

3. $\rho_p = \dfrac{A_{ps}}{b d_p} = \dfrac{7(0.153 \text{ IN}^2)}{(48")(6.25")}$ — 1.071

$\rho_p = 0.00357$

$$f_{ps} = 270 \text{ KSI}\left[1 - \frac{0.28}{0.80}\left(0.00357 \cdot \frac{270}{5}\right)\right]$$

$\blacktriangle$ f_{ps} = 251.782 KSI

$$\omega_p = \frac{\rho_p f_{ps}}{f'_c} = \frac{0.00357(251.782)}{5} = 0.1797$$

$\blacktriangle$ $\omega_p = 0.180 < 0.288 = 0.36\beta_1$ OK ✓

$$a = \frac{A_{ps} f_{ps}}{0.85 f'_c b} = \frac{1.071(251.8)}{0.85(5)(48)} = 1.32186$$

$\blacktriangle$ a = 1.32"

$$\phi M_n = 0.9(1.071 \text{ IN}^2)(251.8 \text{ KSI})\left[6.25" - \frac{1.32"}{2}\right]$$

$\blacktriangle$ ϕM_n = 1356.4 KIP-IN [113 KIP-FT]

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COMMENTS
(CONT.) SPOT CHECK[3]

$$M_{U,REQ'D} = \frac{\omega l^2}{8} \qquad \text{w: } \omega = 1.2D + 1.6SL \ (\text{DET. MAX.})$$

$$= 1.2(74\text{ PSF}) + 1.6(30\text{ PSF})$$

$$= \frac{(0.1368)(32.5)^2}{8} \qquad \omega = 136.8\text{ PSF } (0.1368\text{ KSF})$$

$$l = 32.5\text{ FT}$$

$$M_{U,REQ'D} = 18.06\text{ FT-K/FT}$$

$\times$ 4 FT WIDE

$$\therefore M_{U,REQ'D} = 72.25\text{ FT-K/SLAB}$$

$$\rightarrow M_u = 72.25\text{ FT-K} < 113\text{ FT-K} = \phi M_n \quad \therefore \underline{\text{OK}} \checkmark$$

- MIN. REINFORCEMENT CHECK ($\phi M_n \geq 1.2 M_{cr}$)

PRESTESS LOSS →

$$A_{ps} f_{pu} = 0.153\text{ IN}^2 \times 270\text{ KSI} = 41.3\text{ K/STRAND}$$

1. ELASTIC SHORTENING:

$$P_{P1} = \overset{70\%}{0.7}(\overset{\#\text{STR}}{7})(41.3^K) = 202.37\text{ KIP} \rightarrow (\text{REF } P_j(\text{JACKING}), \text{ CONCISE RPT. PG. 1})$$

$$M_d = \frac{32.5^2}{8}(0.062 \times 4') = 32.74\text{ FT-KIP } [392.93\text{ IN-KIP}]$$

$$f_{cir} = K_{cir}\left(\frac{P_{P1}}{A} + \frac{P_{P1}e^2}{I}\right) - \frac{M_d e}{I} \qquad \text{w: } A = \text{AREA OF SECTION}, \ e = 8/2 - 1.75" = 2.25"$$

$$= 0.9\left(\frac{202.4}{215} + \frac{202.4(2.25)^2}{16666}\right) - \frac{393(2.25)}{16666}$$

$$f_{cir} = 0.87\text{ KSI}$$

$$E_s = 28000\text{ KSI}$$
$$E_{ci} = 3587\text{ KSI} \qquad (\text{TAKEN FROM C.S.R., PG. 1})$$

$$ES = K_{es} E_s \frac{f_{cir}}{E_{ci}}$$

$$= 1.0\left(\frac{28000}{3587}\right)(0.87) = 6.7914$$

$$\therefore ES = 6.79\text{ KSI}$$

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COMMENTS (CONT.) SPOT CHECK 4

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PRESTRESS LOSS CONT...

2. CONCRETE CREEP:

$$f_{cds} = \frac{M_{sd}\,e}{I} \quad \text{w:} \quad M_{sd} = \frac{32.5^2}{8}(0.012 \times 4') = 6.34 \text{ K-FT } [76.05 \text{ IN-K}]$$

$$= \frac{(76.05)(2.25)}{1666} = 0.1027$$

$$f_{cds} = 0.103 \text{ KSI}$$

$E_c = 4287$ KSI (TAKEN FROM C.S.R., PG. 1)

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

$$= \underset{\text{N.W.C.}}{2.0}\left(\frac{28000}{4287}\right)(0.87 - 0.103) = 10.023$$

$\therefore CR = 10.0$ KSI

3. SHRINKAGE OF CONCRETE

$V/S = 1.92 \text{ IN}^2$ (TAKEN FROM C.S.R., PG. 1)

$R.H. = 70\%$

$$SH = 8.2 \times 10^{-6} K_{sh} E_s \left(1 - 0.06\frac{V}{S}\right) \times (100 - RH)$$

$$= 8.2 \times 10^{-6} \underset{\text{B/C PRESTR.}}{(1.0)}(28000)\left[1 - 0.06(1.92)\right](100 - 70) = 6.0945$$

$SH = 6.09$ KSI

4. STEEL RELAXATION

$K_{re} = 5000$ PSI
$J = 0.040$
} TBL 2.2.3.1 PCI MANUAL: DESIGN HC SLABS

$C = 0.75 \rightarrow$ TBL 2.2.3.2

$f_{si}/f_{pu} = 0.70$ FOR LOW-LAX

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

$$= \left[\frac{5000}{1000}\text{PSI} - 0.040(6.09 + 10 + 6.79)\right] \times 0.75 = 3.06272$$

$RE = 3.06$ KSI

5. TOTAL LOSS AT MIDSPAN

$TL = ES + CR + SH + RE = 25.9719$ KSI $\qquad TL = 13.7\%$

$$\% = \frac{25.9719}{0.7 \times 270} \times 100 = 13.7417$$ (COMPARE TO 13%, C.S.R. PG. 1)

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COMMENTS (CONT.) SPOT CHECK[5]

Ckd By ____ Date ____ Project

(MIN. REINF. CHECK)

→ PRESTRESS LOSS FOUND TO BE 13.7%

$A_{ps} f_{se} = 0.70(7 \times 41.3)(1 - 0.137) = 174.561^{k}$

$$M_{CR} = \frac{I}{Y_b}\left(\frac{P}{A} + \frac{Pe}{S_b} + 7.5\sqrt{f'_c}\right)$$

HT. OF CENTROID → Y_b

$$= \frac{1666}{4}\left(\frac{175^{k}}{215} + \frac{175^{k}(2.25)}{417} + \frac{7.5\sqrt{5000}}{1000}\right) = 951.334 \text{ IN-K/SLAB}$$

$M_{CR} = 951$ IN-K/SLAB

$1.2 M_{CR} = 1.2(951) = 1141.6$ IN-K $<$ 1356.4 IN-K $= \phi M_n$

$\therefore \phi M_n > 1.2 M_{CR}$ ✓ OK

Summary Report

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Concise Beam (TM), Version 4.46c, (c) 2006 Black Mint Software, Inc
Licensed to: 4054021211, Civilsmith Engineering - OK
Project: Applied Research Laboratory V
Problem: Roof 8" HC Plank

SUMMARY REPORT

Design Code Used: ACI318-05

CONCRETE MATERIAL PROPERTIES

			Precast Beam
Concrete Density	Wt	=	150 lb/ft^3 — NWC
Compressive Strength	f'c	=	5.0 ksi
Modulus of Elasticity	Ec	=	4287 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

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'x8"				

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	215.0	1666	4.00	417	417	1.92	12.00	48.00	8.00

* 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.

PRESTRESSING STEEL TENDONS

Prestressing Strand Details

ID	Qty	Material	Section	Offsets x ft	y in	End Offset & Type * Left ft	Right ft	Tendon Area in^2	Jacking Force Pj kip	%fpu
1	7	fpu=270 ksi Es= 28000.0 ksi	SWS#1/2"	0.00 32.83	1.75 1.75	0.00 B	0.00 B	1.071	202.4	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 = 3.3 %, Final = 13.0 %
Maximum Total Prestress Forces: Pj(jacking) = 202.4 kip,
Pi(transfer) = 195.7 kip,
Pe(effective) = 176.1 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	155343	264186	25.86	80.27
1	0.50	RIGHT	0.00	155343	264186	25.86	80.27

BEAM AND TOPPING SELF-WEIGHT

Segment/Length From	To	Linear Weight Beam	Topping

Engineer: EMF
File: HC8-in_Roof_01.con
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Summary Report

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Concise Beam (TM), Version 4.46c, (c) 2006 Black Mint Software, Inc
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Project: Applied Research Laboratory V
Problem: Roof 8" HC Plank

No.	ft	ft	kip/ft	kip/ft
1	0.00	32.83	0.22	

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
Roof Load	SRLr	20 PSF LLr	Line Load	0.08	0.08	0.00	32.83
Roof Load	SRLr	30 PSF SL	Line Load	0.12	0.12	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	-0.1	0.0	0.0
0.17	0.0	0.0	11.1	0.0	0.0
3.42	23.9	14.4	8.9	32.5	0.0
6.67	42.6	25.7	6.7	57.8	0.0
9.92	55.9	33.7	4.4	75.9	0.0
13.17	63.8	38.5	2.2	86.7	0.0
16.42	66.5	40.1	0.0	90.4	0.0
19.83	63.6	38.3	-2.3	86.4	0.0
23.08	55.3	33.4	-4.6	75.2	0.0
26.33	41.8	25.2	-6.8	56.7	0.0
29.58	22.9	13.8	-9.0	31.1	0.0
32.66	0.0	0.0	-11.1	0.0	0.0
32.66	0.0	0.0	0.1	0.0	0.0
32.83	0.0	0.0	0.0	0.0	0.0

SUPPORT REACTIONS (kip)

Unfactored Support Reactions

Engineer: EMF
File: HC8-in_Roof_01.con
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Company: Civilsmith Engineering, Inc.
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Project: Applied Research Laboratory V
Problem: Roof 8" HC Plank

Load Case	Left	Right
Beam Weight	4.2	4.2
SDL BT	0.0	0.0
Topping Wgt	0.0	0.0
SDL AT	0.8	0.8
LL Sustain	0.0	0.0
Live Load	0.0	0.0
Roof Load	3.3	3.3
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	5.0	5.0
Total	8.3	8.3
Factor Max.	11.2	11.2

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	844	2450	0 %	
Bottom of Beam	2.28	1587	2450	0 %	
					Longitudinal Tensile Rebar Needed (in^2) Required Provided Additional
Critical Tension					
Top of Beam	0.00	0	-444	0 %	
Bottom of Beam	0.00	2	-444	0 %	
STRESSES IN SERVICE					
Critical Compression					
Top of Beam	16.42	1786	3000	0 %	
Bottom of Beam	2.12	1141	3000	0 %	
Critical Tension					
Top of Beam	0.17	-10	-849 *	0 %	Class U member - not cracked *
Bottom of Beam	16.42	-172	-849 *	0 %	Class U member - not cracked
STRESSES IN SERVICE (SUSTAINED LOADS ONLY)					
Critical Compression					
Top of Beam	16.42	1027	2250	0 %	
Bottom of Beam	2.12	1307	2250	0 %	

* Bilinear deflection calculation used.

Modulus of Rupture, fr = -530 psi
Transfer Strength Required, f'ci = 2.3 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.

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

	Net Deflection					Change in Deflection			
Location x ft	Net @ * Transfer in	Net @ Erection in	Net @ Complete in	Net DL @ Final in	Net Total @ Final in	DL growth + LL ** in	LL alone in	Span/Deflection DL growth + LL **	Span/Deflection LL alone
0.00	0.000	-0.015	-0.012	-0.008	0.003	0.015	0.012	267	353
0.17	0.008	0.000	0.000	0.000	0.000	0.000	0.000	0	0
3.42	0.140	0.239	0.184	0.103	-0.114	-0.299	-0.217	1305	1799
6.67	0.210	0.365	0.262	0.076	-0.334	-0.596	-0.410	653	951

Engineer: EMF Company: Civilsmith Engineering, Inc.
File: HC8-in_Roof_01.con 3 Wed Oct 08 13:30:11 2008

Summary Report

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Project: Applied Research Laboratory V
Problem: Roof 8" HC Plank

9.92	0.241	0.418	0.278	-0.007	-0.568	-0.846	-0.561	461	694
13.17	0.251	0.436	0.272	-0.081	-0.738	-1.01	-0.657	386	593
16.42	0.254	0.439	0.267	-0.110	-0.800	-1.07	-0.690	365	564
19.83	0.251	0.435	0.272	-0.078	-0.732	-1.00	-0.654	388	596
23.08	0.240	0.417	0.278	-0.002	-0.557	-0.835	-0.555	466	702
26.33	0.208	0.360	0.260	0.079	-0.322	-0.582	-0.401	669	972
29.58	0.135	0.230	0.178	0.101	-0.105	-0.283	-0.206	1376	1890
32.66	0.008	0.000	0.000	0.000	0.000	0.000	0.000	0	0
32.83	0.000	-0.015	-0.012	-0.008	0.003	0.015	0.012	267	353

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
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	0.0	22.2	0.03	0.6314	Tension	0.75	
0.17	0.0	-2.8	-21.7	0.54	0.0067	Tension	0.75	
3.42	32.5	78.9	91.8	1.21	0.0125	Tension	0.83	
6.67	57.8	117.6	93.3	1.80	0.0074	Tension	0.90	
9.92	75.9	117.7	94.4	1.80	0.0074	Tension	0.90	
13.17	86.7	117.7	95.0	1.80	0.0074	Tension	0.90	
16.42	90.4	117.7	95.2	1.80	0.0074	Tension	0.90	
19.83	86.4	117.7	95.0	1.80	0.0074	Tension	0.90	
23.08	75.2	117.7	94.3	1.80	0.0074	Tension	0.90	
26.33	56.7	115.6	93.2	1.76	0.0076	Tension	0.90	
29.58	31.1	77.1	91.7	1.21	0.0125	Tension	0.82	
32.66	0.0	-2.8	-21.7	0.54	0.0067	Tension	0.75	
32.83	0.0	0.0	22.2	0.03	0.6314	Tension	0.75	
Points of Maximum and Minimum Factored Moment								
16.42	90.4	117.7	95.2	1.80	0.0074	Tension	0.90	
0.17	0.0	-2.8	-21.7	0.54	0.0067	Tension	0.75	
Points of Critical Moment Design								
16.42	90.4	117.7	95.2	1.80	0.0074	Tension	0.90	
0.17	0.0	-2.8	-21.7	0.54	0.0067	Tension	0.75	

SHEAR AND TORSION DESIGN CHECK

Design Code Used: ACI318-05

Shear and Torsion Design Forces

x ft	Applied Shear Vu kip	Prestress Component Vp kip	Concrete Strength ØVc kip	Stirrup * Strength ØVs kip	Shear Strength ØVn kip	Applied Torsion Tu kipft	Threshold Torsion ØTcr/4 kipft	Notes & Warnings
0.00	0.0	0.0	-8.1	0.0	-8.1	0.0	5.8	
0.17	-0.1	0.0	-15.3	0.0	-15.3	0.0	6.4	
0.17	10.9	0.0	15.3	0.0	15.3	0.0	6.4	
3.42	8.9	0.0	17.5	0.0	17.5	0.0	11.2	
6.67	6.7	0.0	8.9	0.0	8.9	0.0	11.2	
9.92	4.4	0.0	8.1	0.0	8.1	0.0	11.3	
13.17	2.2	0.0	8.1	0.0	8.1	0.0	11.3	
16.42	0.0	0.0	8.1	0.0	8.1	0.0	11.3	
19.83	-2.3	0.0	-8.1	0.0	-8.1	0.0	11.3	
23.08	-4.6	0.0	-8.1	0.0	-8.1	0.0	11.3	
26.33	-6.8	0.0	-9.1	0.0	-9.1	0.0	11.2	
29.58	-9.0	0.0	-18.3	0.0	-18.3	0.0	11.1	
32.66	-10.9	0.0	-15.3	0.0	-15.3	0.0	6.4	

Summary Report

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Concise Beam (TM), Version 4.46c, (c) 2006 Black Mint Software, Inc
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Project: Applied Research Laboratory V
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32.66	0.1	0.0	15.3	0.0	15.3	0.0	6.4
32.83	0.0	0.0	8.1	0.0	8.1	0.0	5.8

* 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	6.00	0.00	0.00	
0.17	0.00	0.00	N/A	N/A	6.00	0.00	0.00	
0.17	0.00	0.00	N/A	N/A	6.00	0.00	0.00	
3.42	0.00	0.00	N/A	N/A	6.00	0.00	0.00	
6.67	0.00	0.00	N/A	N/A	6.00	0.00	0.00	
9.92	0.00	0.00	N/A	N/A	6.00	0.00	0.00	
13.17	0.00	0.00	N/A	N/A	6.00	0.00	0.00	
16.42	0.00	0.00	N/A	N/A	6.00	0.00	0.00	
19.83	0.00	0.00	N/A	N/A	6.00	0.00	0.00	
23.08	0.00	0.00	N/A	N/A	6.00	0.00	0.00	
26.33	0.00	0.00	N/A	N/A	6.00	0.00	0.00	
29.58	0.00	0.00	N/A	N/A	6.00	0.00	0.00	
32.66	0.00	0.00	N/A	N/A	6.00	0.00	0.00	
32.66	0.00	0.00	N/A	N/A	6.00	0.00	0.00	
32.83	0.00	0.00	N/A	N/A	6.00	0.00	0.00	

* 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.

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COMMENTS
SPOT CHECK ~ 12RB24 BEAM

FLEXURAL DESIGN CHECK OF
2ND FLOOR RECTANGULAR BEAM (12 × 24)

(L = 9'-10" ; C.S. = 9'-6")

LOADS →

DIVIDE BY 1000

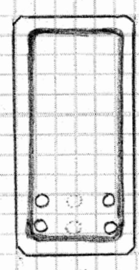
DEAD:	12" HC SLAB	= 72.5 PSF × 32.23'	=	2.337	KLF
	2" TOPPING	= 25 PSF × 34.23'	=	0.856	KLF
	12" HC SOLID	= 77.5 PSF × 2.0'	=	0.155	KLF
	MISC.	= 12 PSF × 34.23'	=	0.411	KLF
	SELF WT.	= 12×24/144 × 150/1000	=	0.300	KLF

$\Sigma = 3.758$ SAY → $\boxed{D = 3.8 \text{ KLF}}$

LIVE: 125 PSF × 34.23/1000 = 4.279 KLF SAY → $\boxed{L = 4.3 \text{ KLF}}$

$\omega = 1.2D + 1.6L = 11.4$ KLF

ASSUME →
1. $f'_c = 6500$ PSI
2. $f_y = 60000$ PSI
3. NO COMPRESSION REINF.
4. #4 STIRRUPS
5. $d = 20.75"$



24" (h)

12" (b)

TRY (6) #8

SOL. → $A_s = (6)(0.79 \text{ in}^2) = 4.74 \text{ in}^2$ $\qquad A_s = 4.74 \text{ in}^2$

$$n = \frac{E_s}{E_c} = \frac{29000 \text{ ksi}(1000)}{57000\sqrt{6500 \text{ psi}}} = 6.31 \qquad n = 6.31$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{4.74 \text{ in}^2 (60 \text{ ksi})}{0.85(6.5)(12 \text{ in})} = 4.289 \rightsquigarrow a = 2.90"$$

$$\rho_{bal} = \frac{A_s}{bd} = \frac{4.74}{(12)(20.75)} = 0.019$$

$$c = a/\beta_1 = 4"$$

$\beta_1 = 0.725$ ($f'_c = 6.5$ ksi)

$$\rho_{MAX} = 0.85\beta_1 \frac{f'_c}{f_y} \frac{\epsilon_u}{\epsilon_u + \epsilon_y} = 0.85(0.725)\left(\frac{6.5}{60}\right)\left(\frac{0.003}{0.003 + \frac{60}{29000}}\right) = 0.0395$$

$$\rho_{MAX} = 0.0395 > \rho_{bal} = 0.019$$

$$\therefore \epsilon_t = \frac{\epsilon_u}{c}(d - c) = \frac{0.003}{4'}(20.75" - 4") = 0.012526$$

$$0.013 > 0.005 \implies \phi = 0.9$$

$$\phi M_n = \phi A_s f_y (d - a/2)$$
$$= 0.9(4.74 \text{ in}^2)(60 \text{ ksi})(20.75" - 2.90"/2) = 4568.886 \text{ IN-K}$$

$\phi M_n = 380.7$ K-FT

$$M_u = \frac{\omega l^2}{8} = \frac{(11.4)(9.5')^2}{8} = 128.606 \text{ K-FT}$$

$\boxed{\phi M_n > M_u}$ ✓OK

COMPARE TO PG. 3 OF THIS SPOT CHECK

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COMMENTS
(CONT.) SPOT CHECK[2]
BEAM

$$A_{s,min} \geq \begin{cases} \dfrac{3\sqrt{f'_c}}{f_y} bd = \dfrac{3\sqrt{6500}}{60000}(12)(20.75) = 1.004 \text{ in}^2 \\ \dfrac{200\,bd}{f_y} = \dfrac{200(12)(20.75)}{60000} = 0.83 \text{ in}^2 \end{cases} \Bigg\} \text{ BOTH} < A_s = 4.74 \text{ in}^2 \quad \text{OK ✓}$$

$$A_{s,max} = 0.0395(12)(20.75) = 9.84 \text{ in}^2 > 4.74 \text{ in}^2 = A_s \quad \text{OK ✓}$$

$$b_{min} = \underset{\text{CLR. CVR.}}{2(1.25")} + \underset{\#4 \text{ STIR.}}{2(0.5")} + 3(1") + 2(1") = 8.5 \text{ in} < 12 \text{ in} \quad \text{OK ✓}$$

CONCLUSION: 12 × 24 RECT. BM W/ (6) #8 REINF. BARS IN TWO (2) LAYERS AND #4 STIRRUPS WILL WORK.

▲ TRY 1 LAYER #8'S (3 TOTAL) → $d = 21.75$ in.

$A_s = 2.37 \text{ in}^2$
$n = 6.31$
$a = 2.145$ in
$\beta_1 = 0.725$
$c = 2.958$ in

$\rho_{bal} = 0.0091 < 0.0395 = \rho_{max}$

$\epsilon_t = 0.0191 > 0.005 \therefore \Rightarrow \phi = 0.9$

$$\phi M_n = 0.9(2.37 \text{ in}^2)(60 \text{ ksi})\left(21.75" - \frac{2.145"}{2}\right) = 2646.3195 \text{ in-k} = \underline{220.527} \text{ ft-k}$$

$$\phi M_n = \begin{cases} 2646 \text{ in-k} \\ 221 \text{ ft-k} \end{cases} \longleftarrow \text{COMPARE TO PG. 3 OF THIS SPOT CHECK}$$

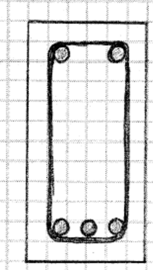
$$\phi M_n = 221 \text{ ft-k} > 129 \text{ ft-k} = M_u \quad \text{OK ✓}$$

$$A_{s,min} \geq \begin{cases} 1.05 \text{ in}^2 \\ 0.87 \text{ in}^2 \end{cases} \quad \underset{(A_s = 2.37 \text{ in}^2)}{\text{OK ✓}} \qquad A_{s,max} = 10.31 \text{ in}^2 \quad \underset{(A_s = 2.37 \text{ in}^2)}{\text{OK ✓}}$$

BEAM WILL WORK

CONCLUSION:
USE A RECT. BM 12 × 24 W/ (3) #8 BARS PLACED A DISTANCE OF 2.75" UP, FROM BOTTOM OF BM. AND #4 STIRRUPS.

NOTE: THIS DESIGN WAS PERFORMED NEGLECTING THE PRESENCE OF TOP COMPRESSION STEEL WHICH WILL MORE THAN LIKELY BE PLACED FOR AIDE IN HOLDING THE STIRRUPS IN PLACE. SECTION SHOULD RESEMBLE... →



Flexural Design Check

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Concise Beam (TM), Version 4.46c, (c) 2006 Black Mint Software, Inc
Licensed to: 4054021211, Civilsmith Engineering - OK
Project: Applied Research Laboratory V
Problem: 9'-10" Non-Prestressed 12RB24

FLEXURAL DESIGN CHECK

Design Code Used: ACI318-05
Beta Used: for precast beam = 0.725
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	220.4	72.6	2.63	0.0218	Tension	0.90	
0.17	-0.2	-88.4	-69.8	1.88	0.0319	Tension	0.90	
0.26	5.1	220.4	72.6	2.63	0.0218	Tension	0.90	
0.50	17.7	220.4	72.6	2.63	0.0218	Tension	0.90	
0.74	29.6	220.4	72.6	2.63	0.0218	Tension	0.90	
1.02	43.1	220.4	72.6	2.63	0.0218	Tension	0.90	
1.26	53.6	220.4	72.6	2.63	0.0218	Tension	0.90	
1.50	63.4	220.4	72.6	2.63	0.0218	Tension	0.90	
1.74	72.6	220.4	72.6	2.63	0.0218	Tension	0.90	
1.97	81.1	220.4	72.6	2.63	0.0218	Tension	0.90	
2.26	90.4	220.4	72.6	2.63	0.0218	Tension	0.90	
2.50	97.5	220.4	72.6	2.63	0.0218	Tension	0.90	
2.73	103.9	220.4	72.6	2.63	0.0218	Tension	0.90	
2.97	109.6	220.4	72.6	2.63	0.0218	Tension	0.90	
3.21	114.7	220.4	72.6	2.63	0.0218	Tension	0.90	
3.45	119.1	220.4	72.6	2.63	0.0218	Tension	0.90	
3.73	123.6	220.4	72.6	2.63	0.0218	Tension	0.90	
3.97	126.5	220.4	72.6	2.63	0.0218	Tension	0.90	
4.20	128.9	220.4	72.6	2.63	0.0218	Tension	0.90	
4.44	130.5	220.4	72.6	2.63	0.0218	Tension	0.90	
4.68	131.5	220.4	72.6	2.63	0.0218	Tension	0.90	
4.92	$\boxed{131.8}$	$\boxed{220.4}$	72.6	2.63	0.0218	Tension	0.90	
5.20	131.3	220.4	72.6	2.63	0.0218	Tension	0.90	
5.44	130.2	220.4	72.6	2.63	0.0218	Tension	0.90	
5.68	128.4	220.4	72.6	2.63	0.0218	Tension	0.90	
5.91	126.0	220.4	72.6	2.63	0.0218	Tension	0.90	
6.15	122.9	220.4	72.6	2.63	0.0218	Tension	0.90	
6.44	118.3	220.4	72.6	2.63	0.0218	Tension	0.90	
6.67	113.8	220.4	72.6	2.63	0.0218	Tension	0.90	
6.91	108.5	220.4	72.6	2.63	0.0218	Tension	0.90	
7.15	102.7	220.4	72.6	2.63	0.0218	Tension	0.90	
7.38	96.1	220.4	72.6	2.63	0.0218	Tension	0.90	
7.62	88.9	220.4	72.6	2.63	0.0218	Tension	0.90	
7.91	79.4	220.4	72.6	2.63	0.0218	Tension	0.90	
8.14	70.8	220.4	72.6	2.63	0.0218	Tension	0.90	
8.38	61.5	220.4	72.6	2.63	0.0218	Tension	0.90	
8.62	51.5	220.4	72.6	2.63	0.0218	Tension	0.90	
8.86	40.9	220.4	72.6	2.63	0.0218	Tension	0.90	
9.14	27.3	220.4	72.6	2.63	0.0218	Tension	0.90	
9.38	15.2	220.4	72.6	2.63	0.0218	Tension	0.90	
9.62	2.5	220.4	72.6	2.63	0.0218	Tension	0.90	
9.66	-0.2	-88.4	-69.8	1.88	0.0319	Tension	0.90	
9.83	0.0	220.4	72.6	2.63	0.0218	Tension	0.90	
Points of Maximum and Minimum Factored Moment								
4.92	131.8	220.4	72.6	2.63	0.0218	Tension	0.90	
0.17	-0.2	-88.4	-69.8	1.88	0.0319	Tension	0.90	
Points of Critical Moment Design								
4.92	131.8	220.4	72.6	2.63	0.0218	Tension	0.90	
0.17	-0.2	-88.4	-69.8	1.88	0.0319	Tension	0.90	

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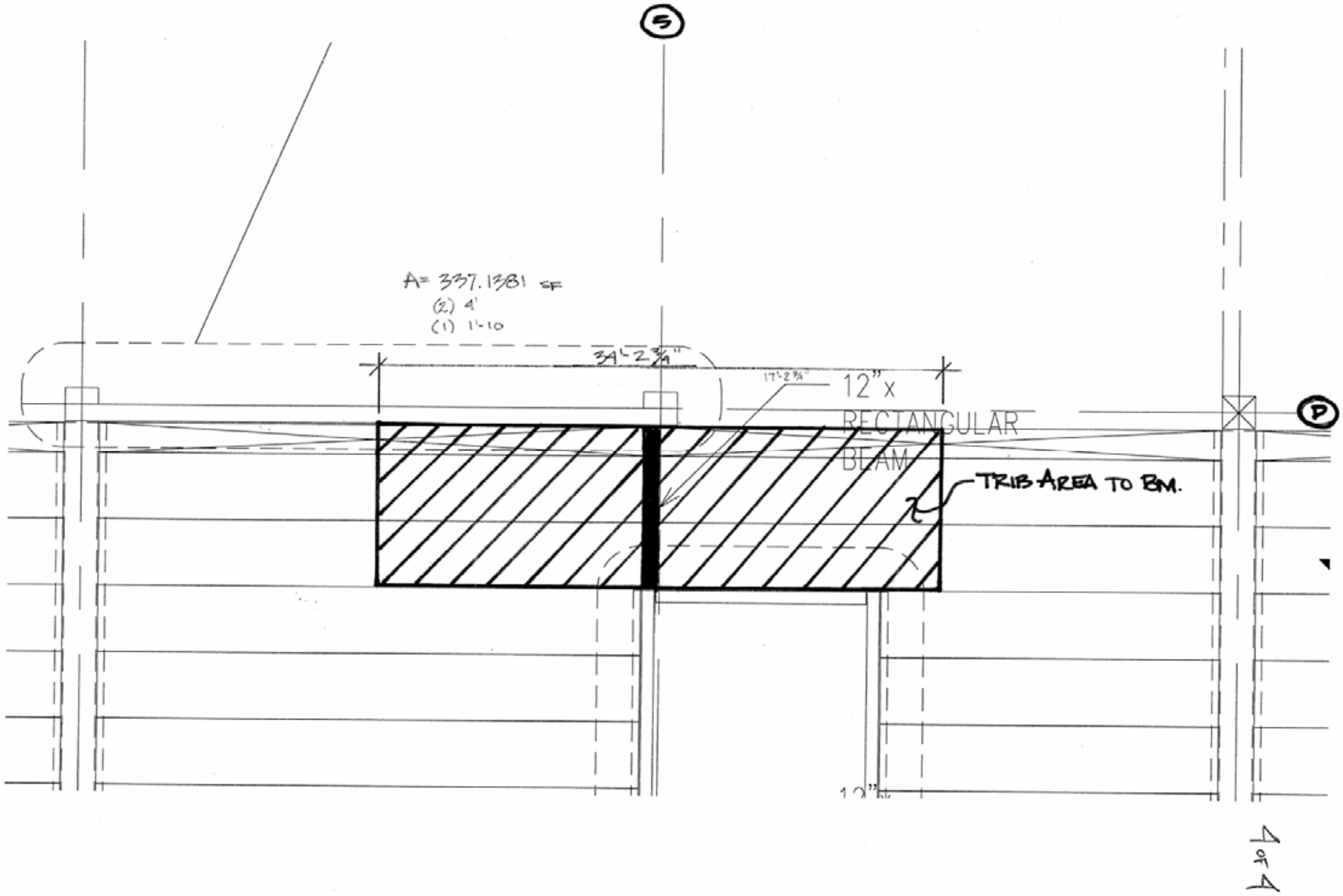
A= 337.1381 SF
(2) 4'
(1) 1'-10
34'-2 3/4"
17'-2 3/4"
12"x
RECTANGULAR
BEAM
TRIB AREA TO BM.
4 OF 4

DESIGNS

(Inverted-Tee Beams)

Summary Report

Concise Beam (TM), Version 4.46c, (c) 2006 Black Mint Software, Inc
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Project: Applied Research Laboratory V
Problem: 2nd Floor 36IT28

SUMMARY REPORT

Design Code Used: ACI318-05

CONCRETE MATERIAL PROPERTIES

			Precast Beam	
Concrete Density	Wt	=	150	lb/ft^3
Compressive Strength	f'c	=	6.5	ksi
Modulus of Elasticity	Ec	=	4888	ksi
Strength at Transfer	f'ci	=	4.0	ksi
Modulus of Elast. at Transfer	Eci	=	3834	ksi

Cement Content = 691 lb/yd^3
Air Content = 5.00 %
Slump = 1.97 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

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	Inverted-Tee	36IT28_12" HC				

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	862.5	53360	12.66	4215	3478	6.74	24.00	36.00	28.00

* 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.

LONGITUDINAL REINFORCING STEEL

Reinforcing Steel Groups

ID	Qty	Grade ksi	Bar Size *	Area in^2	From * ft	To * ft	Offset in	Offset Reference **
1	4	60.0	#8	3.160	0.00 H	32.83 H	25.75	bottom of the beam
2	2	60.0	#6	0.880	0.00 H	32.83 H	2.13	bottom of the beam
3	2	60.0	#8	1.580	0.00 H	32.83 H	17.00	bottom of the beam

notes: * A 'C' or 'H' suffix indicates "epoxy coating" and "hooked end" respectively.
** Offsets are measured up from the bottom or down from the top

Reinforcing Steel Development Lengths (in)

ID	Location Factor	Spacing Factor	Density Factor	Bar Size Factor	Left End of Rebar Tension	Left End of Rebar Compression	Right End of Rebar Tension	Right End of Rebar Compression
1	1.00	1.00	1.00	1.00	0.00	0.00	0.00	0.00
2	1.00	1.00	1.00	1.00	0.00	0.00	0.00	0.00
3	1.00	1.00	1.00	1.00	0.00	0.00	0.00	0.00

note: Product of location and coating factors not taken greater than 1.70.

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

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1	15	fpu=270 ksi	SWS#1/2"SP	0.00	2.00	0.00 B	0.00 B	2.505	473.4	0.70
		Es= 28000.0 ksi		32.83	2.00					
2	9	fpu=270 ksi	SWS#1/2"SP	0.00	4.00	0.00 B	0.00 B	1.503	284.1	0.70
		Es= 28000.0 ksi		32.83	4.00					

notes: * Strand End Types: B - Fully Bonded, D - Debonded, C - Cut, A - Anchored (fully developed)
Prestressing steel is low relaxation strand.
Calculated Losses: Initial = 7.8 %, Final = 13.9 %
Maximum Total Prestress Forces: Pj(jacking) = 757.5 kip,
Pi(transfer) = 698.6 kip,
Pe(effective) = 651.9 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	151740	260550	25.26	79.65
1	0.50	RIGHT	0.00	151740	260550	25.26	79.65
2	0.50	LEFT	0.00	153090	258791	25.48	78.32
2	0.50	RIGHT	0.00	153090	258791	25.48	78.32

BEAM AND TOPPING SELF-WEIGHT

No.	Segment/Length From ft	To ft	Linear Weight Beam kip/ft	Topping kip/ft
1	0.00	32.83	0.90	

EXTERNALLY APPLIED LOADS

Load Case		Load Label	Load Type	Load Intensity (*) Left	Right	Offset (ft) Left	Right
SDL BT	D	12" HC Slabe	Line Load	2.55	2.55	0.00	32.83
SDL AT	D	2" Topping	Line Load	0.78	0.78	0.00	32.83
SDL AT	D	12 PSF	Line Load	0.42	0.42	0.00	32.83
Live Load	L	125 PSF	Line Load	4.38	4.38	0.00	32.83
Live Load	L	5 kip	Point Load	5.00	-	16.42	-

* 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	#4	2	0	0.40	0.00	6.00	0.00

TORSION PARAMETERS

Seg. No.	Torsion Parameters Aoh in^2	Ph in
1	679.50	116.00

Aoh is the area enclosed by the centerline of the outermost closed transverse torsional reinforcement.

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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	-0.1	0.0	-2.1	-0.2	0.0
0.17	-0.1	0.0	208.2	-0.2	0.0
3.29	420.7	212.5	169.0	588.1	0.0
6.67	777.6	391.9	126.5	1087.4	0.0
9.92	1023.7	514.4	85.7	1432.1	0.0
13.17	1174.6	587.8	44.8	1644.2	0.0
16.42	1230.3	612.3	4.0	1723.5	0.0
16.42	1230.3	612.3	4.0	1723.5	0.0
19.80	1170.4	585.8	-46.5	1638.2	0.0
23.05	1015.7	510.4	-87.3	1420.9	0.0
26.29	765.8	386.0	-128.2	1070.8	0.0
29.67	404.9	204.6	-170.6	566.0	0.0
32.66	-0.1	0.0	-208.2	-0.2	0.0
32.66	-0.1	0.0	2.1	-0.2	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	14.7	14.7
SDL BT	41.8	41.8
Topping Wgt	0.0	0.0
SDL AT	19.6	19.6
LL Sustain	0.0	0.0
Live Load	74.3	74.3
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	76.2	76.2
Total	150.5	150.5
Factor Max.	210.3	210.3

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

Location	x ft	Stress psi	Limit psi		Overstress Notice	
STRESSES AT TRANSFER						
Critical Compression						
Top of Beam	0.00	-1	2800		0 %	
Bottom of Beam	30.71	2182	2800		0 %	
Critical Tension						
Top of Beam	30.71	-986	-474		108 %	
Bottom of Beam	0.00	2	-474		0 %	
STRESSES IN SERVICE						
Critical Compression						
Top of Beam	16.42	2981	3900		0 %	
Bottom of Beam	2.12	1256	3900		0 %	
Critical Tension						
Top of Beam	0.17	-80	-967	*	0 %	Class U member - not cracked
Bottom of Beam	16.42	-1174	-967	*	21 %	Class C member - check cracking and cover
STRESSES IN SERVICE (SUSTAINED LOADS ONLY)						

Longitudinal Tensile Rebar Needed (in^2)

Required	Provided	Additional
3.0	3.2	0.0

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Critical Compression				
Top of Beam	16.42	970	2925	0 %
Bottom of Beam	2.12	1619	2925	0 %

* Bilinear deflection calculation used.

Modulus of Rupture, fr = -605 psi
Transfer Strength Required, f'ci = 3.1 ksi
Transfer Strength Specified, f'ci = 4.0 ksi

DISTRIBUTION OF FLEXURAL STEEL & CRACKING

	Bottom of Beam	Top of Beam		
Maximum Crack Width Estimate				
Cracked?	Yes	No		
w =	0.006	0.000	in	- Estimated maximum crack width *
x =	16.42	0.00	ft	- Location of maximum crack width from left end of beam
c =	15.18	0.00	in	- Concrete depth in compression
Ms =	1230.3	0.0	kipft	- External service moment (DL + LL)
Pdc =	-632.9	0.0	kip	- Decompression force at cracked centroid
Mint=	321.3	0.0	kipft	- Internal moment about cracked centroid
Steel Type	mixed	rebar		- Type of steel in tension
	Suri&Dilger	-		- Equation used **
k1 =	2.1	0.0		- (x 10^-5) Coefficient used for equation
fs =	14.2	0.0	psi	- Stress in steel nearest to tension face (after decompression)
h1 =	-	0.00	in	- Distance from the neutral axis to the extreme rebar
h2 =	-	0.00	in	- Distance from the neutral axis to the edge of concrete in tension
dc =	2.00	0.00	in	- Concrete cover to center of steel closest to tension face
A =	-	0.0	in^2	- Area of concrete in tension around each bar/strand
At =	461.5	-	in^2	- Area of concrete in tension
Ast =	4.9	-	in^2	- Area of steel in tension
Maximum Flexural Steel Spacing				
cc =	1.75	0.00	in	- Clear concrete cover to steel closest to tension face
s =	33.84	0.00	in	- Maximum centre-to-centre spacing of steel closest to tension face
fc =	-3472	-235	psi	- Maximum concrete compressive stress
limit=	-3900	-3900	psi	- Allowable concrete compressive stress

Recommended Crack Width (in) and Equivalent z (lb/in) Values (from PCI Design Handbook)

	Critical Exposure w	Critical Exposure z	Prestressed Concrete w	Prestressed Concrete z	Reinforced Concrete w	Reinforced Concrete z
Exterior Exposure	0.007	80000	0.008	90000	0.013	145000
Interior Exposure	0.010	105000	0.011	115000	0.016	175000

* Note: actual crack widths can vary by as much as 50% from this predicted value.
Control of cracking is accomplished by proper steel detailing as specified in the design code.
** Note: Gergely & Lutz equation: w = k1 x fs x h2 / h1 x CubicRoot(dc * A)
Suri & Dilger equation: w = k1 x fs x dc x Sqrt(At / Ast)

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

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	-0.006	-0.005	-0.005	0.005	0.009	0.009	449	449
0.17	0.010	0.000	0.000	0.000	0.000	0.000	0.000	0	0
3.29	0.182	0.164	0.132	0.100	-0.064	-0.196	-0.164	1990	2371
6.67	0.326	0.292	0.228	0.160	-0.169	-0.396	-0.328	983	1188
9.92	0.422	0.372	0.284	0.185	-0.275	-0.559	-0.460	697	847
13.17	0.479	0.418	0.315	0.195	-0.360	-0.675	-0.555	577	703
16.42	0.498	0.432	0.324	0.197	-0.393	-0.717	-0.590	543	660
16.42	0.498	0.432	0.324	0.197	-0.393	-0.717	-0.590	543	660

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19.80	0.477	0.417	0.314	0.194	-0.358	-0.671	-0.552	580	706
23.05	0.419	0.370	0.283	0.184	-0.271	-0.553	-0.455	704	856
26.29	0.321	0.288	0.225	0.158	-0.164	-0.389	-0.322	1001	1210
29.67	0.176	0.158	0.127	0.097	-0.061	-0.188	-0.158	2075	2472
32.66	0.010	0.000	0.000	0.000	0.000	0.000	0.000	0	0
32.83	0.000	-0.006	-0.005	-0.005	0.005	0.009	0.009	449	449

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 = 0.725
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	148.8	257.5	1.96	0.0368	Tension	0.75	
0.17	-0.2	-398.5	-192.6	2.20	0.0322	Tension	0.75	
3.29	588.1	1263.7	1169.1	6.96	0.0082	Tension	0.82	
6.67	1087.4	1816.2	1192.9	9.60	0.0051	Tension	0.90	
9.92	1432.1	1818.1	1209.1	9.64	0.0051	Tension	0.90	
13.17	1644.2	1818.8	1218.8	9.64	0.0051	Tension	0.90	
16.42	1723.5	1819.0	1222.0	9.64	0.0051	Tension	0.90	
16.42	1723.5	1819.0	1222.0	9.64	0.0051	Tension	0.90	
19.80	1638.2	1818.8	1218.5	9.64	0.0051	Tension	0.90	
23.05	1420.9	1818.0	1208.6	9.64	0.0051	Tension	0.90	
26.29	1070.8	1804.8	1192.1	9.56	0.0052	Tension	0.90	
29.67	566.0	1242.9	1168.1	6.88	0.0083	Tension	0.82	
32.66	-0.2	-398.5	-192.6	2.20	0.0322	Tension	0.75	
32.83	0.0	148.8	257.5	1.96	0.0368	Tension	0.75	
Points of Maximum and Minimum Factored Moment								
16.42	1723.5	1819.0	1222.0	9.64	0.0051	Tension	0.90	
0.17	-0.2	-398.5	-192.6	2.20	0.0322	Tension	0.75	
Points of Critical Moment Design								
16.42	1723.5	1819.0	1222.0	9.64	0.0051	Tension	0.90	
0.17	-0.2	-398.5	-192.6	2.20	0.0322	Tension	0.75	

SHEAR AND TORSION DESIGN CHECK

Design Code Used: ACI318-05

Shear and Torsion Design Forces

x ft	Applied Shear Vu kip	Prestress Component Vp kip	Concrete Strength ØVc kip	Stirrup * Strength ØVs kip	Shear Strength ØVn kip	Applied Torsion Tu kipft	Threshold Torsion ØTcr/4 kipft	Notes & Warnings
0.00	0.0	0.0	-63.8	-77.6	-141.4	0.0	29.3	
0.17	-2.1	0.0	-122.5	-68.5	-191.0	0.0	31.5	
0.17	193.5	0.0	122.5	68.5	191.0	0.0	31.5	2
3.29	169.0	0.0	221.1	75.9	297.0	0.0	50.1	
6.67	126.5	0.0	105.9	75.8	181.7	0.0	50.1	
9.92	85.7	0.0	73.4	75.8	149.2	0.0	50.1	
13.17	44.8	0.0	73.4	75.8	149.2	0.0	50.1	
16.42	4.0	0.0	73.4	75.8	149.2	0.0	50.1	
16.42	4.0	0.0	73.4	75.8	149.2	0.0	50.1	
19.80	-46.5	0.0	-73.4	-75.8	-149.2	0.0	50.1	
23.05	-87.3	0.0	-73.4	-75.8	-149.2	0.0	50.1	
26.29	-128.2	0.0	-108.2	-75.8	-184.1	0.0	50.1	
29.67	-170.6	0.0	-221.0	-75.9	-296.9	0.0	50.1	
32.66	-193.5	0.0	-122.5	-68.5	-191.0	0.0	31.5	2
32.66	2.1	0.0	122.5	68.5	191.0	0.0	31.5	

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32.83	0.0	0.0	63.8	77.6	141.4	0.0	29.3

Notes & Warnings
2 - Warning: The applied factored shear, Vu, is greater than the shear strength, ØVn.
* Stirrup resistance based on stirrup area provided.

Transverse Steel (Stirrup) Design for Shear

x ft	Required Shear Steel Total (Av+2At)/s in^2/ft	Torsion* At/s in^2/ft	Stirrup Provided Av+2At in^2	Stirrup Spacing Provided s in	Required s in	Long. Torsion Steel, Al Total Required in^2	Allowable Reduction** in^2	Notes & Warnings
0.00	0.00	0.00	0.40	6.00	21.00	0.00	0.00	
0.17	0.00	0.00	0.40	6.00	21.00	0.00	0.00	
0.17	0.83	0.00	0.40	6.00	5.79	0.00	0.00	1
3.29	0.07	0.00	0.40	6.00	21.00	0.00	0.00	2
6.67	0.22	0.00	0.40	6.00	21.00	0.00	0.00	
9.92	0.13	0.00	0.40	6.00	21.00	0.00	0.00	
13.17	0.11	0.00	0.40	6.00	21.00	0.00	0.00	2
16.42	0.00	0.00	0.40	6.00	21.00	0.00	0.00	
16.42	0.00	0.00	0.40	6.00	21.00	0.00	0.00	
19.80	0.11	0.00	0.40	6.00	21.00	0.00	0.00	2
23.05	0.15	0.00	0.40	6.00	21.00	0.00	0.00	
26.29	0.21	0.00	0.40	6.00	21.00	0.00	0.00	
29.67	0.07	0.00	0.40	6.00	21.00	0.00	0.00	2
32.66	0.83	0.00	0.40	6.00	5.79	0.00	0.00	1
32.66	0.00	0.00	0.40	6.00	21.00	0.00	0.00	
32.83	0.00	0.00	0.40	6.00	21.00	0.00	0.00	

Notes & Warnings
1 - Warning: The shear stirrup spacing is too wide.
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.

Summary Report

Concise Beam (TM), Version 4.46c, (c) 2006 Black Mint Software, Inc
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Project: Applied Research Laboratory V
Problem: 3rd Floor 36IT20

SUMMARY REPORT

Design Code Used: ACI318-05

CONCRETE MATERIAL PROPERTIES

			Precast Beam
Concrete Density	Wt	=	150 lb/ft^3
Compressive Strength	f'c	=	6.0 ksi
Modulus of Elasticity	Ec	=	4696 ksi
Strength at Transfer	f'ci	=	4.0 ksi
Modulus of Elast. at Transfer	Eci	=	3834 ksi

Cement Content = 691 lb/yd^3
Air Content = 5.00 %
Slump = 1.97 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

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	Inverted-Tee	36IT20_10" HC				

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	598.5	19399	9.00	2155	1764	5.34	24.00	36.00	20.00

* 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.

LONGITUDINAL REINFORCING STEEL

Reinforcing Steel Groups

ID	Qty	Grade ksi	Bar Size *	Area in^2	From * ft	To * ft	Offset in	Offset Reference **
1	4	60.0	#8	3.160	0.00 H	32.83 H	17.75	bottom of the beam
2	2	60.0	#6	0.880	0.00 H	32.83 H	2.13	bottom of the beam
3	2	60.0	#8	1.580	0.00 H	32.83 H	12.00	bottom of the beam
4	2	60.0	#8	1.580	0.00 H	32.83 H	15.00	bottom of the beam

notes: * A 'C' or 'H' suffix indicates "epoxy coating" and "hooked end" respectively.
** Offsets are measured up from the bottom or down from the top

Reinforcing Steel Development Lengths (in)

ID	Location Factor	Spacing Factor	Density Factor	Bar Size Factor	Left End of Rebar Tension	Left End of Rebar Compression	Right End of Rebar Tension	Right End of Rebar Compression
1	1.00	1.00	1.00	1.00	0.00	0.00	0.00	0.00
2	1.00	1.00	1.00	1.00	0.00	0.00	0.00	0.00
3	1.00	1.00	1.00	1.00	0.00	0.00	0.00	0.00
4	1.00	1.00	1.00	1.00	0.00	0.00	0.00	0.00

note: Product of location and coating factors not taken greater than 1.70.

PRESTRESSING STEEL TENDONS

Prestressing Strand Details

Offsets   End Offset & Type *   Tendon   Jacking Force

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ID	Qty	Material	Section	x ft	y in	Left ft	Right ft	Area in^2	Pj kip	%fpu
1	12	fpu=270 ksi	SWS#1/2"SP	0.00	2.00	0.00 B	0.00 B	2.004	378.8	0.70
		Es= 28000.0 ksi		32.83	2.00					
2	12	fpu=270 ksi	SWS#1/2"SP	0.00	4.00	0.00 B	0.00 B	2.004	378.8	0.70
		Es= 28000.0 ksi		32.83	4.00					
3	8	fpu=270 ksi	SWS#1/2"SP	0.00	6.00	0.00 B	0.00 B	1.336	252.5	0.70
		Es= 28000.0 ksi		32.83	6.00					

notes: * Strand End Types: B - Fully Bonded, D - Debonded, C - Cut, A - Anchored (fully developed)
Prestressing steel is low relaxation strand.
Calculated Losses: Initial = 10.5 %, Final = 16.7 %
Maximum Total Prestress Forces: Pj(jacking) = 1010.0 kip,
Pi(transfer) = 903.8 kip,
Pe(effective) = 841.3 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	141839	234458	23.61	69.91
1	0.50	RIGHT	0.00	141839	234458	23.61	69.91
2	0.50	LEFT	0.00	142992	213762	23.80	59.18
2	0.50	RIGHT	0.00	142992	213762	23.80	59.18
3	0.50	LEFT	0.00	145468	194305	24.21	48.63
3	0.50	RIGHT	0.00	145468	194305	24.21	48.63

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	0.62	

EXTERNALLY APPLIED LOADS

Load Case		Load Label	Load Type	Load Intensity (*) Left	Right	Offset (ft) Left	Right
SDL BT	D	10" HC Slabs	Line Load	2.48	2.48	0.00	32.83
SDL AT	D	2" Topping	Line Load	0.78	0.78	0.00	32.83
SDL AT	D	12 PSF	Line Load	0.42	0.42	0.00	32.83
Live Load	L	80 PSF	Line Load	3.08	3.08	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	#3	2	0	0.22	0.00	6.00	0.00

TORSION PARAMETERS

Seg. No. Torsion Parameters Aoh Ph

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	in^2	in
1	439.50	100.00

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	-0.1	0.0	-1.7	-0.1	0.0
0.17	-0.1	0.0	163.8	-0.1	0.0
3.29	337.8	196.7	132.4	461.8	0.0
6.67	622.9	362.8	98.3	851.5	0.0
9.92	817.6	476.2	65.5	1117.6	0.0
13.17	934.4	544.2	32.8	1277.3	0.0
16.42	973.3	566.9	0.0	1330.6	0.0
19.80	931.2	542.4	-34.1	1273.0	0.0
23.05	811.3	472.5	-66.8	1109.0	0.0
26.29	613.5	357.3	-99.6	838.6	0.0
29.67	325.2	189.4	-133.7	444.5	0.0
32.66	-0.1	0.0	-163.8	-0.1	0.0
32.66	-0.1	0.0	1.7	-0.1	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	10.2	10.2
SDL BT	40.7	40.7
Topping Wgt	0.0	0.0
SDL AT	19.6	19.6
LL Sustain	0.0	0.0
Live Load	50.6	50.6
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	70.5	70.5
Total	121.1	121.1
Factor Max.	165.5	165.5

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

Location	x ft	Stress psi	Limit psi	Overstress Notice
STRESSES AT TRANSFER				
Critical Compression				
Top of Beam	0.00	-1	2800	0 %
Bottom of Beam	1.99	3246	2800	16 %
Critical Tension				
Top of Beam	1.99	-997	-474	110 %
Bottom of Beam	0.00	4	-474	0 %
STRESSES IN SERVICE				
Critical Compression				
Top of Beam	16.42	5013	3600	39 %
Bottom of Beam	1.99	1791	3600	0 %
Critical Tension				

Longitudinal Tensile Rebar Needed (in^2)

Required	Provided	Additional
1.6	3.2	0.0

Summary Report

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Top of Beam	0.17	-85	-930	*	0 %	Class U member - not cracked
Bottom of Beam	16.42	-1889	-930	*	103 %	Class C member - check cracking and cover

STRESSES IN SERVICE (SUSTAINED LOADS ONLY)
Critical Compression

Top of Beam	16.42	2480	2700	0 %
Bottom of Beam	1.99	2236	2700	0 %

* Bilinear deflection calculation used.

Modulus of Rupture, fr = -581 psi
Transfer Strength Required, f'ci = 4.6 ksi
Transfer Strength Specified, f'ci = 4.0 ksi

DISTRIBUTION OF FLEXURAL STEEL & CRACKING

Maximum Crack Width Estimate

	Bottom of Beam	Top of Beam		
Cracked?	Yes	No		
w =	0.005	0.000	in	- Estimated maximum crack width *
x =	16.42	0.00	ft	- Location of maximum crack width from left end of beam
c =	11.91	0.00	in	- Concrete depth in compression
Ms =	973.3	0.0	kipft	- External service moment (DL + LL)
Pdc =	-796.1	0.0	kip	- Decompression force at cracked centroid
Mint=	291.2	0.0	kipft	- Internal moment about cracked centroid
Steel Type	mixed	rebar		- Type of steel in tension
	Suri&Dilger	-		- Equation used **
k1 =	2.1	0.0		- (x 10^-5) Coefficient used for equation
fs =	16.1	0.0	psi	- Stress in steel nearest to tension face (after decompression)
h1 =	-	0.00	in	- Distance from the neutral axis to the extreme rebar
h2 =	-	0.00	in	- Distance from the neutral axis to the edge of concrete in tension
dc =	2.00	0.00	in	- Concrete cover to center of steel closest to tension face
A =	-	0.0	in^2	- Area of concrete in tension around each bar/strand
At =	291.2	-	in^2	- Area of concrete in tension
Ast =	6.3	-	in^2	- Area of steel in tension

Maximum Flexural Steel Spacing

cc =	1.75	0.00	in	- Clear concrete cover to steel closest to tension face
s =	29.85	0.00	in	- Maximum centre-to-centre spacing of steel closest to tension face
fc =	-5275	-512	psi	- Maximum concrete compressive stress
limit=	-3600	-3600	psi	- Allowable concrete compressive stress

Recommended Crack Width (in) and Equivalent z (lb/in) Values (from PCI Design Handbook)

	Critical Exposure w	Critical Exposure z	Prestressed Concrete w	Prestressed Concrete z	Reinforced Concrete w	Reinforced Concrete z
Exterior Exposure	0.007	80000	0.008	90000	0.013	145000
Interior Exposure	0.010	105000	0.011	115000	0.016	175000

* Note: actual crack widths can vary by as much as 50% from this predicted value.
Control of cracking is accomplished by proper steel detailing as specified in the design code.
** Note: Gergely & Lutz equation: w = k1 x fs x h2 / h1 x CubicRoot(dc * A)
Suri & Dilger equation: w = k1 x fs x dc x Sqrt(At / Ast)

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

	Net Deflection					Change in Deflection			
Location x ft	Net @ * Transfer in	Net @ Erection in	Net @ Complete in	Net DL @ Final in	Net Total @ Final in	DL growth + LL ** in	LL alone in	Span/Deflection DL growth + LL **	Span/Deflection LL alone
0.00	0.000	-0.009	-0.004	-0.004	0.015	0.019	0.019	213	213
0.17	0.019	0.000	0.000	0.000	0.000	0.000	0.000	0	0
3.29	0.339	0.250	0.158	0.029	-0.317	-0.475	-0.346	820	1127
6.67	0.606	0.433	0.253	-0.014	-0.707	-0.960	-0.693	406	562

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9.92	0.785	0.540	0.293	-0.082	-1.06	-1.35	-0.978	287	398
13.17	0.891	0.598	0.309	-0.138	-1.32	-1.63	-1.18	239	329
16.42	0.926	0.616	0.313	-0.160	-1.42	-1.73	-1.26	225	310
19.80	0.888	0.596	0.309	-0.137	-1.31	-1.62	-1.18	240	331
23.05	0.779	0.537	0.292	-0.080	-1.05	-1.34	-0.968	290	402
26.29	0.597	0.427	0.250	-0.012	-0.692	-0.942	-0.681	413	572
29.67	0.327	0.241	0.153	0.030	-0.302	-0.456	-0.332	855	1175
32.66	0.019	0.000	0.000	0.000	0.000	0.000	0.000	0	0
32.83	0.000	-0.009	-0.004	-0.004	0.015	0.019	0.019	213	213

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 = 0.750
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	121.2	127.4	2.41	0.0194	Tension	0.75	
0.17	-0.1	-354.8	-95.7	2.77	0.0163	Tension	0.75	
3.29	461.8	914.0	809.3	8.08	0.0037	Tension	0.83	
6.67	851.5	1155.0	834.1	10.05	0.0024	Tension	0.90	
9.92	1117.6	1163.5	851.0	10.17	0.0023	Tension	0.90	
13.17	1277.3	1168.7	861.1	10.25	0.0023	Tension	0.90	1
16.42	1330.6	1170.7	864.4	10.25	0.0023	Tension	0.90	1
19.80	1273.0	1168.5	860.8	10.25	0.0023	Tension	0.90	1
23.05	1109.0	1163.2	850.4	10.17	0.0023	Tension	0.90	
26.29	838.6	1154.5	833.3	10.05	0.0024	Tension	0.90	
29.67	444.5	898.5	808.2	7.92	0.0038	Tension	0.83	
32.66	-0.1	-354.8	-95.7	2.77	0.0163	Tension	0.75	
32.83	0.0	121.2	127.4	2.41	0.0194	Tension	0.75	
Points of Maximum and Minimum Factored Moment								
16.42	1330.6	1170.7	864.4	10.25	0.0023	Tension	0.90	1
0.17	-0.1	-354.8	-95.7	2.77	0.0163	Tension	0.75	
Points of Critical Moment Design								
16.42	1330.6	1170.7	864.4	10.25	0.0023	Tension	0.90	1
0.17	-0.1	-354.8	-95.7	2.77	0.0163	Tension	0.75	

Notes & Warnings
1 - Warning: clause 9.1.1, ØMn < Mu, design strength is not greater than factored moment

SHEAR AND TORSION DESIGN CHECK

Design Code Used: ACI318-05

Shear and Torsion Design Forces

x ft	Applied Shear Vu kip	Prestress Component Vp kip	Concrete Strength ØVc kip	Stirrup * Strength ØVs kip	Shear Strength ØVn kip	Applied Torsion Tu kipft	Threshold Torsion ØTcr/4 kipft	Notes & Warnings
0.00	0.0	0.0	-42.3	-29.5	-71.8	0.0	15.5	
0.17	-1.7	0.0	-86.5	-26.4	-112.9	0.0	17.5	
0.17	155.4	0.0	86.5	26.4	112.9	0.0	17.5	2
3.29	132.4	0.0	156.1	26.9	183.1	0.0	31.7	
6.67	98.3	0.0	72.9	27.1	100.0	0.0	31.6	
9.92	65.5	0.0	45.8	27.1	72.9	0.0	31.5	
13.17	32.8	0.0	45.8	27.1	72.9	0.0	31.4	
16.42	0.0	0.0	45.8	27.1	72.9	0.0	31.4	
19.80	-34.1	0.0	-45.8	-27.1	-72.9	0.0	31.4	
23.05	-66.8	0.0	-45.8	-27.1	-72.9	0.0	31.5	
26.29	-99.6	0.0	-74.5	-27.1	-101.6	0.0	31.6	

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29.67	-133.7	0.0	-162.9	-26.9	-189.9	0.0	31.7	
32.66	-155.4	0.0	-86.5	-26.4	-112.9	0.0	17.5	2
32.66	1.7	0.0	86.5	26.4	112.9	0.0	17.5	
32.83	0.0	0.0	42.3	29.5	71.8	0.0	15.5	

Notes & Warnings
2 - Warning: The applied factored shear, Vu, is greater than the shear strength, ØVn.
* Stirrup resistance based on stirrup area provided.

Transverse Steel (Stirrup) Design for Shear

	Required Shear Steel		Stirrup	Stirrup Spacing		Long. Torsion Steel, Al		Notes &
	Total	Torsion*	Provided	Provided	Required	Total	Allowable	Warnings
x	(Av+2At)/s	At/s	Av+2At	s	s	Required	Reduction**	
ft	in^2/ft	in^2/ft	in^2	in	in	in^2	in^2	
0.00	0.00	0.00	0.22	6.00	15.00	0.00	0.00	
0.17	0.00	0.00	0.22	6.00	15.00	0.00	0.00	
0.17	1.15	0.00	0.22	6.00	2.30	0.00	0.00	1
3.29	0.12	0.00	0.22	6.00	15.00	0.00	0.00	2
6.67	0.41	0.00	0.22	6.00	6.40	0.00	0.00	
9.92	0.32	0.00	0.22	6.00	8.26	0.00	0.00	
13.17	0.18	0.00	0.22	6.00	14.54	0.00	0.00	2
16.42	0.00	0.00	0.22	6.00	15.00	0.00	0.00	
19.80	0.18	0.00	0.22	6.00	14.54	0.00	0.00	2
23.05	0.34	0.00	0.22	6.00	7.74	0.00	0.00	
26.29	0.41	0.00	0.22	6.00	6.49	0.00	0.00	
29.67	0.12	0.00	0.22	6.00	15.00	0.00	0.00	2
32.66	1.15	0.00	0.22	6.00	2.30	0.00	0.00	1
32.66	0.00	0.00	0.22	6.00	15.00	0.00	0.00	
32.83	0.00	0.00	0.22	6.00	15.00	0.00	0.00	

Notes & Warnings
1 - Warning: The shear stirrup spacing is too wide.
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.

Summary Report

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Project: Applied Research Laboratory V
Problem: Roof 36IT18

SUMMARY REPORT

Design Code Used: ACI318-05

CONCRETE MATERIAL PROPERTIES

			Precast Beam	
Concrete Density	Wt	=	150	lb/ft^3
Compressive Strength	f'c	=	6.5	ksi
Modulus of Elasticity	Ec	=	4888	ksi
Strength at Transfer	f'ci	=	4.0	ksi
Modulus of Elast. at Transfer	Eci	=	3834	ksi

Cement Content = 691 lb/yd^3
Air Content = 5.00 %
Slump = 1.97 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

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	Inverted-Tee	36IT18_8" HC				

Total Beam Length = 32.83 ft, Left Support @ 0.33 ft, Right Support @ 32.50 ft, Span = 32.17 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	550.5	14162	8.13	1742	1435	5.10	24.00	36.00	18.00

* 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.

LONGITUDINAL REINFORCING STEEL

Reinforcing Steel Groups

ID	Qty	Grade ksi	Bar Size *	Area in^2	From * ft	To * ft	Offset in	Offset Reference **
1	3	60.0	#8	2.370	0.00 H	32.83 H	17.50	bottom of the beam
2	2	60.0	#6	0.880	0.00 H	32.83 H	2.13	bottom of the beam
3	2	60.0	#5	0.620	0.00 H	32.83 H	11.00	bottom of the beam
4	2	60.0	#5	0.620	0.00 H	32.83 H	14.00	bottom of the beam

notes: * A 'C' or 'H' suffix indicates "epoxy coating" and "hooked end" respectively.
** Offsets are measured up from the bottom or down from the top

Reinforcing Steel Development Lengths (in)

ID	Location Factor	Spacing Factor	Density Factor	Bar Size Factor	Left End of Rebar Tension	Left End of Rebar Compression	Right End of Rebar Tension	Right End of Rebar Compression
1	1.00	1.00	1.00	1.00	0.00	0.00	0.00	0.00
2	1.00	1.00	1.00	1.00	0.00	0.00	0.00	0.00
3	1.00	1.00	1.00	1.00	0.00	0.00	0.00	0.00
4	1.00	1.00	1.00	1.00	0.00	0.00	0.00	0.00

note: Product of location and coating factors not taken greater than 1.70.

PRESTRESSING STEEL TENDONS

Prestressing Strand Details

Offsets End Offset & Type * Tendon Jacking Force

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ID	Qty	Material	Section	x ft	y in	Left ft	Right ft	Area in^2	Pj kip	%fpu
1	11	fpu=270 ksi	SWS#1/2"SP	0.00	2.00	0.00 B	0.00 B	1.837	347.2	0.70
		Es= 28000.0 ksi		32.83	2.00					
2	4	fpu=270 ksi	SWS#1/2"SP	0.00	4.00	0.00 B	0.00 B	0.668	126.3	0.70
		Es= 28000.0 ksi		32.83	4.00					

notes: * Strand End Types: B - Fully Bonded, D - Debonded, C - Cut, A - Anchored (fully developed)
Prestressing steel is low relaxation strand.
Calculated Losses: Initial = 6.2 %, Final = 9.3 %
Maximum Total Prestress Forces: Pj(jacking) = 473.4 kip,
Pi(transfer) = 444.1 kip,
Pe(effective) = 429.4 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	156030	261304	25.97	78.60
1	0.50	RIGHT	0.00	156030	261304	25.97	78.60
2	0.50	LEFT	0.00	157071	258363	26.15	76.78
2	0.50	RIGHT	0.00	157071	258363	26.15	76.78

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	0.57	

EXTERNALLY APPLIED LOADS

Load Case		Load Label	Load Type	Load Intensity (*) Left	Right	Offset (ft) Left	Right
SDL BT	D	8 " HC Slab	Line Load	2.12	2.12	0.00	32.83
SDL BT	D	12 psf misc	Line Load	0.42	0.42	0.00	32.83
Roof Load	SRLr	30 PSF SL	Line Load	1.05	1.05	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	#3	2	0	0.22	0.00	6.00	0.00

TORSION PARAMETERS

Seg. No.	Torsion Parameters Aoh in^2	Ph in
1	397.50	96.00

Aoh is the area enclosed by the centerline of the outermost closed transverse torsional reinforcement.

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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
0.33	-0.2	-0.2	-1.8	-0.3	0.0
0.33	-0.2	-0.2	87.1	-0.3	0.0
3.29	179.8	134.5	71.1	233.9	0.0
6.64	339.4	253.8	53.0	441.5	0.0
9.85	448.8	335.7	35.6	583.9	0.0
13.20	517.0	386.7	17.4	672.5	0.0
16.42	538.5	402.8	0.0	700.5	0.0
19.76	515.2	385.3	-18.1	670.2	0.0
23.11	445.3	333.0	-36.3	579.2	0.0
26.32	334.1	249.9	-53.7	434.6	0.0
29.67	172.7	129.2	-71.8	224.7	0.0
32.50	-0.2	-0.2	-87.1	-0.3	0.0
32.50	-0.2	-0.2	1.8	-0.3	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	9.4	9.4
SDL BT	41.7	41.7
Topping Wgt	0.0	0.0
SDL AT	0.0	0.0
LL Sustain	0.0	0.0
Live Load	0.0	0.0
Roof Load	17.2	17.2
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	51.1	51.1
Total	68.4	68.4
Factor Max.	89.0	89.0

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

Location	x ft	Stress psi	Limit psi	Overstress Notice	
STRESSES AT TRANSFER					
Critical Compression					
Top of Beam	0.00	0	2800	0 %	
Bottom of Beam	30.57	1949	2800	0 %	
Critical Tension					
Top of Beam	30.57	-701	-474	48 %	
Bottom of Beam	0.00	2	-474	0 %	
STRESSES IN SERVICE					
Critical Compression					
Top of Beam	16.42	3353	3900	0 %	
Bottom of Beam	30.70	1081	3900	0 %	
Critical Tension					
Top of Beam	32.50	-121	-967 *	0 %	Class U member - not cracked
Bottom of Beam	16.42	-1480	-967 *	53 %	Class C member - check cracking and cover

Longitudinal Tensile Rebar Needed (in^2)

Required	Provided	Additional
1.1	2.4	0.0

STRESSES IN SERVICE (SUSTAINED LOADS ONLY)
Critical Compression

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Top of Beam	16.42	2294	2925	0 %
Bottom of Beam	30.70	1268	2925	0 %

* Bilinear deflection calculation used.

Modulus of Rupture,	fr =	-605	psi
Transfer Strength Required,	f'ci =	2.8	ksi
Transfer Strength Specified,	f'ci =	4.0	ksi

DISTRIBUTION OF FLEXURAL STEEL & CRACKING

Maximum Crack Width Estimate

	Bottom of Beam	Top of Beam		
Cracked?	Yes	No		
w =	0.010	0.000	in	- Estimated maximum crack width *
x =	16.42	0.00	ft	- Location of maximum crack width from left end of beam
c =	8.12	0.00	in	- Concrete depth in compression
Ms =	538.5	0.0	kipft	- External service moment (DL + LL)
Pdc =	-377.2	0.0	kip	- Decompression force at cracked centroid
Mint=	154.7	0.0	kipft	- Internal moment about cracked centroid
Steel Type	mixed	rebar		- Type of steel in tension
	Suri&Dilger	-		- Equation used **
k1 =	2.1	0.0		- (x 10^-5) Coefficient used for equation
fs =	22.7	0.0	psi	- Stress in steel nearest to tension face (after decompression)
h1 =	-	0.00	in	- Distance from the neutral axis to the extreme rebar
h2 =	-	0.00	in	- Distance from the neutral axis to the edge of concrete in tension
dc =	2.00	0.00	in	- Concrete cover to center of steel closest to tension face
A =	-	0.0	in^2	- Area of concrete in tension around each bar/strand
At =	355.6	-	in^2	- Area of concrete in tension
Ast =	3.4	-	in^2	- Area of steel in tension

Maximum Flexural Steel Spacing

cc =	1.75	0.00	in	- Clear concrete cover to steel closest to tension face
s =	17.63	0.00	in	- Maximum centre-to-centre spacing of steel closest to tension face
fc =	-4082	-177	psi	- Maximum concrete compressive stress
limit=	-3900	-3900	psi	- Allowable concrete compressive stress

Recommended Crack Width (in) and Equivalent z (lb/in) Values (from PCI Design Handbook)

	Critical Exposure		Prestressed Concrete		Reinforced Concrete	
	w	z	w	z	w	z
Exterior Exposure	0.007	80000	0.008	90000	0.013	145000
Interior Exposure	0.010	105000	0.011	115000	0.016	175000

* Note: actual crack widths can vary by as much as 50% from this predicted value.
Control of cracking is accomplished by proper steel detailing as specified in the design code.
** Note: Gergely & Lutz equation: w = k1 x fs x h2 / h1 x CubicRoot(dc * A)
Suri & Dilger equation: w = k1 x fs x dc x Sqrt(At / Ast)

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

	Net Deflection					Change in Deflection			
Location x ft	Net @ * Transfer in	Net @ Erection in	Net @ Complete in	Net DL @ Final in	Net Total @ Final in	DL growth + LL ** in	LL alone in	Span/Deflection DL growth + LL **	LL alone
0.00	0.000	0.003	0.003	0.003	0.031	0.027	0.027	291	291
0.33	0.023	0.000	0.000	0.000	0.000	0.000	0.000	0	0
3.29	0.214	0.014	0.014	-0.091	-0.333	-0.347	-0.242	1111	1594
6.64	0.378	-0.012	-0.012	-0.231	-0.736	-0.723	-0.504	533	765
9.85	0.485	-0.053	-0.053	-0.362	-1.10	-1.04	-0.735	369	525
13.20	0.550	-0.088	-0.088	-0.458	-1.37	-1.29	-0.916	300	421
16.42	0.571	-0.101	-0.101	-0.490	-1.47	-1.37	-0.981	281	393
19.76	0.549	-0.087	-0.087	-0.455	-1.37	-1.28	-0.911	301	423
23.11	0.482	-0.051	-0.051	-0.357	-1.08	-1.03	-0.726	374	531

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26.32	0.372	-0.011	-0.011	-0.226	-0.720	-0.710	-0.495	543	780
29.67	0.207	0.014	0.014	-0.086	-0.318	-0.332	-0.232	1162	1666
32.50	0.023	0.000	0.000	0.000	0.000	0.000	0.000	0	0
32.83	0.000	0.003	0.003	0.003	0.031	0.027	0.027	291	291

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 = 0.725
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	75.4	107.1	0.72	0.0638	Tension	0.75	
0.33	-0.3	-213.7	-75.3	1.98	0.0235	Tension	0.76	
3.29	233.9	515.6	456.7	4.18	0.0085	Tension	0.83	
6.64	441.5	727.4	469.5	5.80	0.0053	Tension	0.90	
9.85	583.9	728.0	478.3	5.80	0.0053	Tension	0.90	
13.20	672.5	728.3	483.7	5.80	0.0053	Tension	0.90	
16.42	700.5	728.5	485.4	5.80	0.0053	Tension	0.90	
19.76	670.2	728.3	483.6	5.80	0.0053	Tension	0.90	
23.11	579.2	728.0	478.0	5.80	0.0053	Tension	0.90	
26.32	434.6	725.0	469.1	5.76	0.0053	Tension	0.90	
29.67	224.7	507.7	456.2	4.15	0.0086	Tension	0.82	
32.50	-0.3	-213.7	-75.3	1.98	0.0235	Tension	0.76	
32.83	0.0	75.4	107.1	0.72	0.0638	Tension	0.75	
Points of Maximum and Minimum Factored Moment								
16.42	700.5	728.5	485.4	5.80	0.0053	Tension	0.90	
32.50	-0.3	-213.7	-75.3	1.98	0.0235	Tension	0.76	
Points of Critical Moment Design								
16.42	700.5	728.5	485.4	5.80	0.0053	Tension	0.90	
32.50	-0.3	-213.7	-75.3	1.98	0.0235	Tension	0.76	

SHEAR AND TORSION DESIGN CHECK

Design Code Used: ACI318-05

Shear and Torsion Design Forces

x ft	Applied Shear Vu kip	Prestress Component Vp kip	Concrete Strength ØVc kip	Stirrup * Strength ØVs kip	Shear Strength ØVn kip	Applied Torsion Tu kipft	Threshold Torsion ØTcr/4 kipft	Notes & Warnings
0.00	0.0	0.0	-38.9	-26.0	-65.0	0.0	14.2	
0.33	-1.8	0.0	-88.8	-26.0	-114.8	0.0	16.0	
0.33	83.1	0.0	88.8	26.0	114.8	0.0	16.0	
3.29	71.1	0.0	97.0	25.6	122.6	0.0	23.7	
6.64	53.0	0.0	47.4	25.6	73.0	0.0	23.5	
9.85	35.6	0.0	45.0	25.6	70.6	0.0	23.4	
13.20	17.4	0.0	45.0	25.6	70.6	0.0	23.3	
16.42	0.0	0.0	45.0	25.6	70.6	0.0	23.3	
19.76	-18.1	0.0	-45.0	-25.6	-70.6	0.0	23.3	
23.11	-36.3	0.0	-45.0	-25.6	-70.6	0.0	23.4	
26.32	-53.7	0.0	-48.3	-25.6	-73.9	0.0	23.5	
29.67	-71.8	0.0	-101.1	-25.6	-126.7	0.0	23.7	
32.50	-83.1	0.0	-88.8	-26.0	-114.8	0.0	16.0	
32.50	1.8	0.0	88.8	26.0	114.8	0.0	16.0	
32.83	0.0	0.0	38.9	26.0	65.0	0.0	14.2	

* Stirrup resistance based on stirrup area provided.

End of Report