THE PENNSYLVANIA STATE UNIVERSITY

Crocker West Building

State College, PA

Senior Thesis Project Tech III:
Lateral System Analysis and Confirmation Design

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-- EXECUTIVE SUMMARY --

The purpose of Tech Report III is to complete an in-depth analysis of the existing lateral force resisting system implemented into the Crocker West Building. CWB is a 3-story, 42' tall office building and research facility being constructed in State College, Pa. The entire structure will consist of precast systems including: columns, prestressed beams & diaphragms, and walls. Built-up, precast concrete shear walls are the basis of the lateral system used to resist wind and seismic forces.

Tech III contains a detailed analysis of the main lateral force resisting system (MLFRS) utilized in Crocker West. A preliminary study performed for Tech I clearly showed that loading caused by seismic forces governed; thus, the lateral analysis for this tech report only considers seismic loading. The lateral study performed demonstrates how loads caused by seismic forces are distributed through each individual precast element, eventually leading to the amount of load required to be distributed through each panel-to-panel connection.

In addition to performing a lateral analysis for CWB, other design factors such as drift, story drift, and torsion were also taken into consideration for the purpose of this report. Because the structure is only 3-stories high in a relatively low wind area and constructed entirely of concrete, these factors proved to be of little concern. However, these factors may prove to be more relevant in future studies of a higher structure.

Finally, Appendices A & B following the conclusion of Tech III contain project drawings and supporting design information, respectively. The supporting design information consists of detailed calculations of the seismic load distribution through the structural elements, as well as a spot check of shear wall SWD evaluated as a plain concrete shear wall and diaphragm checks to confirm structural integrity of the structure.

**Please note: Seismic analysis and shear wall calculations included in Appendix B of this report are actual designs used in the design and construction of Crocker West. Using a previous in-office project as a design guide, I only incorporated several for verification of load path and design procedure. Other designs available upon request.

-- BUILDING INTRODUCTION --

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, 42' low-rise building with typical 35' square bays broken into areas classified as office, light industrial, and warehouse totaling nearly 120,000 square feet. The first floor of 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. Crocker West utilizes a 16'-0" floor-to-floor height for the ground level, while the remaining two floors have a typical floor-to-floor height of 12'-0". Lateral loads applied to the structure will be collectively distributed throughout the building to specially designed shear walls.

Please note that Appendix A at the end of this report contains drawings of the project for reference, while Appendix B consists of hand calculations and supplementary data used in designing the lateral force resisting system for the Crocker West Building.

-- 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 into 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.

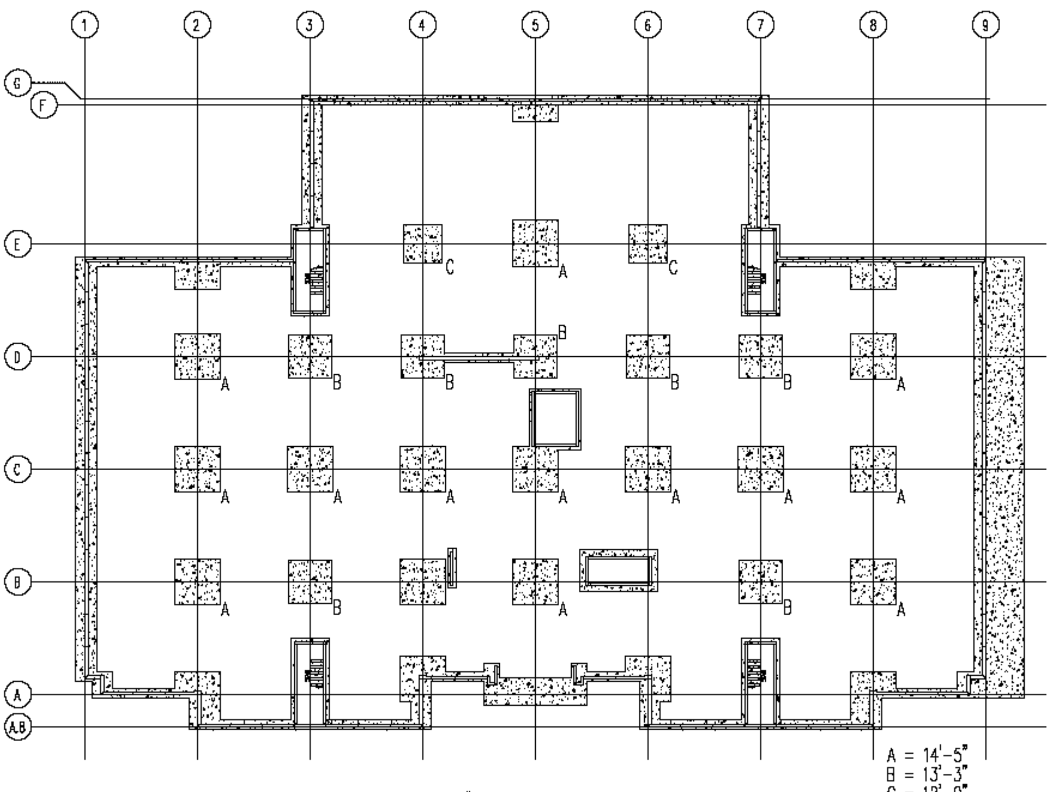


FIGURE #1 – FOUNDATION SYSTEMS

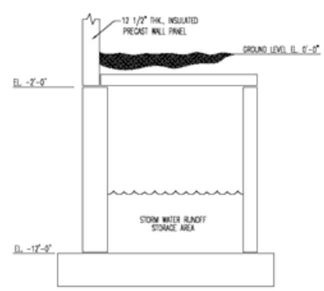


FIGURE #2 – PROPOSED CISTERN SECTION

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.

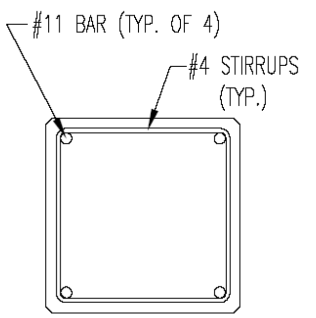


FIGURE #3 – TYP. COLUMN SECTION

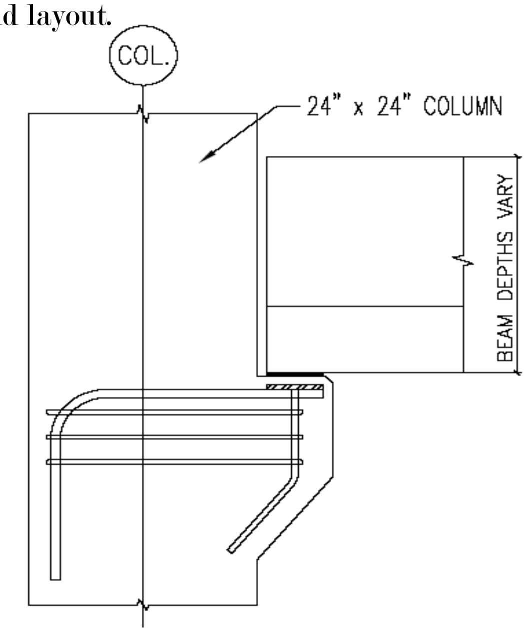


FIGURE #4 – COLUMN w/ HAUNCH

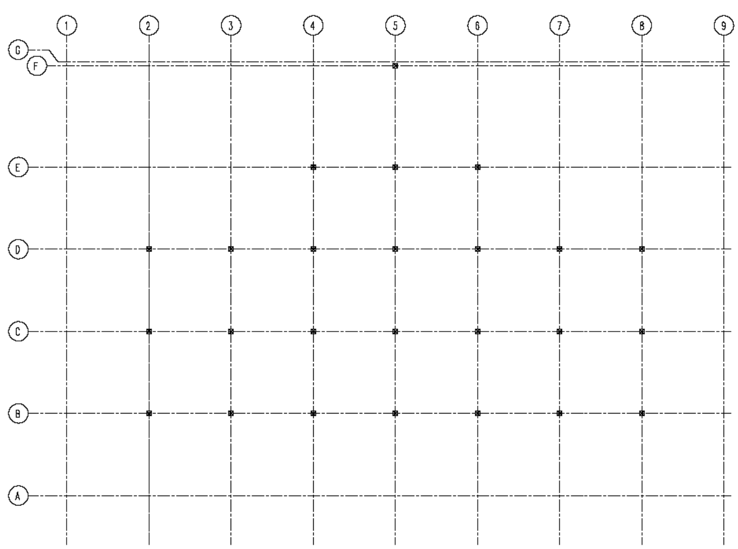


FIGURE #5 – COLUMN GRID

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.

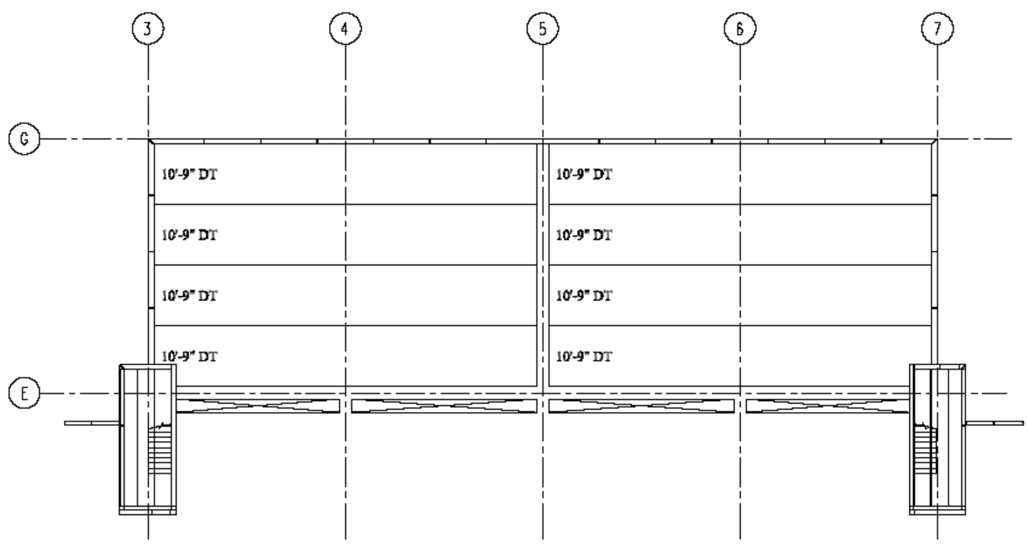


FIGURE #6 – LOW ROOF DT LAYOUT

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.

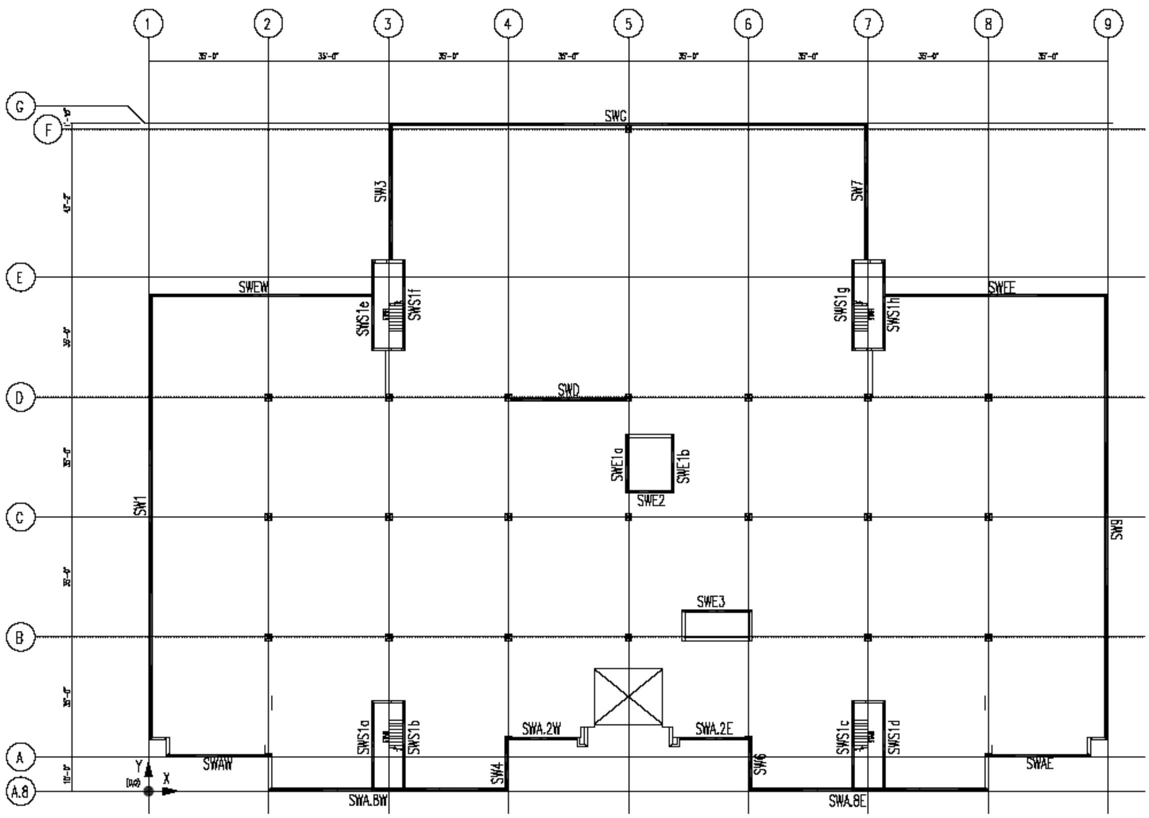


FIGURE #7 – SHEAR WALL LAYOUT

-- 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)	ACI 318-05
- Building Code Requirements for Structural Concrete	
Precast/Prestressed Concrete Institute (PCI)	6th Edition
- PCI Design Handbook, Precast and Prestressed Concrete	
LATERAL LOADS & DESIGN LOADS:	
American Society of Civil Engineers (ASCE)	ASCE 7-05
- Minimum Design Loads for Buildings and Other Structures	
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 (p_f = 0.7*40psf = 28 psf)	40 psf	40 psf *c

*Notes:

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

DEAD LOADS

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

-- SEISMIC LOAD DISTRIBUTION --

In order to determine the distribution of lateral loads throughout the structure, a more detailed seismic analysis was performed using the provisions set-forth in chapter's 11 & 12 of ASCE 7-05, and IBC 2006. Results of this analysis differ from that presented in Tech I partly due to an error in seismic weight calculations and consideration of the low roof area as a flexible diaphragm. The following results act as actual design values for the Crocker West Building.

Seismic Considerations	S_S = 0.17
	S_1 = 0.06
Building Occupancy	Type II
Seismic Design Category	B
Seismic Response Coefficient	C_S = 0.0607
Response Modification Coefficient	R = 3
Deflection Amplification Factor	C_d = 3

Effective Seismic Weight (W)

i. Roof	3908.6 kips
ii. 3rd Floor	6101.2 kips
iii. 2nd Floor	5226.1 kips
Total Effective Seismic Wt. =	15,235.8 kips

Seismic Diaphragm Shear (V = CsW)

i. Roof	V_R = 344.3 kips
ii. 3rd Floor	V_3 = 383.9 kips
iii. 2nd Floor	V_2 = 197.3 kips
Total Base Shear (V) =	925.5 kips

Additional Load Combinations Considered (per 12.4.2.3 – ASCE 7-05)

5. $(1.2 + 0.2S_{DS})D + \rho Q_E + L + 0.2S$

7. $(0.9 - 0.2S_{DS})D + \rho Q_E + 1.6H$ (Governs - 0.86D)**

**These load combinations were used in order to determine the percentage of dead load that may be considered to resist overturning at each wall. See Seismic Load Combinations on page 31 for further detail.

PROCEDURE:

Similar to page 49 in Tech Report I, the seismic analysis performed for this report utilizes an equivalent lateral force procedure excel spreadsheet (available upon request) set-up by the owner of Civilsmith Engineering, Inc. (Reference Appendix B). The spreadsheet is used to determine overall seismic loads, distribution of lateral loads, base shear, seismic weight, and other information pertinent in the lateral system design. The following is an overview & description of the lateral analysis procedure used at Civilsmith Engineering, Inc. to establish viable results for actual design.

a. Determine seismic weight (W) of the structure, breaking down the weights of individual elements per floor levels. (See Tech I for similar calculations of this step. Actual spreadsheet used for the purpose of design available upon request.)
b. In no particular order, find the Center of Mass & Center of Rigidity for each floor level of the structure.
    i. Center of Mass: determined by entering individual element masses and their centroid locations into an in-office spreadsheet that calculates the C.o.M. location for each floor.
    ii. Center of Rigidity: the relative stiffness (based on EI) of each wall was calculated using WinBeam. For each wall designed as a shear wall, the modulus of elasticity (E) & moment of inertia (I) was entered into the WinBeam design program. Next a typical **100,000** kip load was applied to each wall. The resulting deflections of each analysis were then used as a basis for the relative stiffness of each wall. Centroid and rel. stiff. of each wall then analyzed to determine C.o.R. of each floor.
c. Seismic analyses conducted via excel. Base shear (V), diaphragm weights & shears displayed to summary page. (See 'Seismic Force Distribution Summary' in Appendix B, pg. 30) In addition to base shear and diaphragm shears, this page also contains a summary of the lateral load resisting elements and loads distributed to each element.
d. Upon completion of the lateral analysis, the assigned percentage of the load to each individual shear wall is then used to further breakdown the wall component and design each individual panel separately. See Appendix B, starting on pg. 33, for verification of this procedure. As you can see, loads taken from the excel spreadsheet are analyzed and distributed accordingly to each floor level for each built-up shear wall. Next, the loads distributed to the 'built-up' shear wall are then segregated amongst the individual wall panels that make up that shear wall. (Ref. "Built-up Shear Wall" on page 34 in Appendix B).
e. The process of 'breaking-down' the load distributed to each component is continued throughout analysis of every load resisting element. By doing this, one can track the load path(s) used to distribute the load(s) through particular elements, eventually reaching the point of individual connection design.

**Please note: Design of <u>every</u> shear wall is not included within this report. However, calculations for particular members/walls/procedures are available upon request.

DISTRIBUTION OF LOADS:

As described above, the lateral loads are distributed to individual load resisting shear walls at each floor level. There are a total of 28 walls designated as shear walls for the Crocker West project. The majority of the lateral forces in the North-South direction (Normal to the 280' face) are resisted by shear walls SW1 & SW9, while 14 other shear walls support the remainder of the load. Similarly, East-West lateral forces are resisted by 12 built-up shear walls, with a great percentage of this load being distributed to wall SWD. Shear wall SWD, consisting of 3 solid, precast wall panels (designed as shear walls) and utilizing the columns at the ends as piers, proves to be the most rigid wall in the structure. For this reason, wall SWD provides the greatest resistance to lateral loading and was spot checked to confirm.

In addition to the specially designed shear walls used in Crocker West, the designed floor systems call for a 2" concrete topping to be placed over the precast, hollow-core plank. Per 12.3.1.2 (ASCE 7-05), this allows CWB's floor diaphragms to be classified as a 'rigid' diaphragm. The rigid diaphragm created by the combination of hollow-core plank with longitudinal reinforcing in the grout key and the 2" concrete topping, provide a logical load path for lateral loading to be distributed.

DRIFT:

Seismic story drift considerations for Crocker West were not integrated into the actual design based on the assumption that the shear walls provided in each direction will limit drift concern by restraining large deflections. Also, based on the high seismic weight of the structure in relation to the height and relative stiffness of the structure, one can use good engineering judgment to assume the building will not deflect to a level of concern.

For the purpose of this report, I calculated the individual story drifts for the Crocker West Building. WinBeam again was used as a design aide for this analysis. Entering the modulus of elasticity (E) and moment of inertia (I) for wall SWD (between grids 4 & 5) allows the program to determine design factors such as shear and deflection. The deflections resulting from the applied loads per story were then used to calculate individual story drifts.

per Table 12.12-1 (ASCE 7-05)

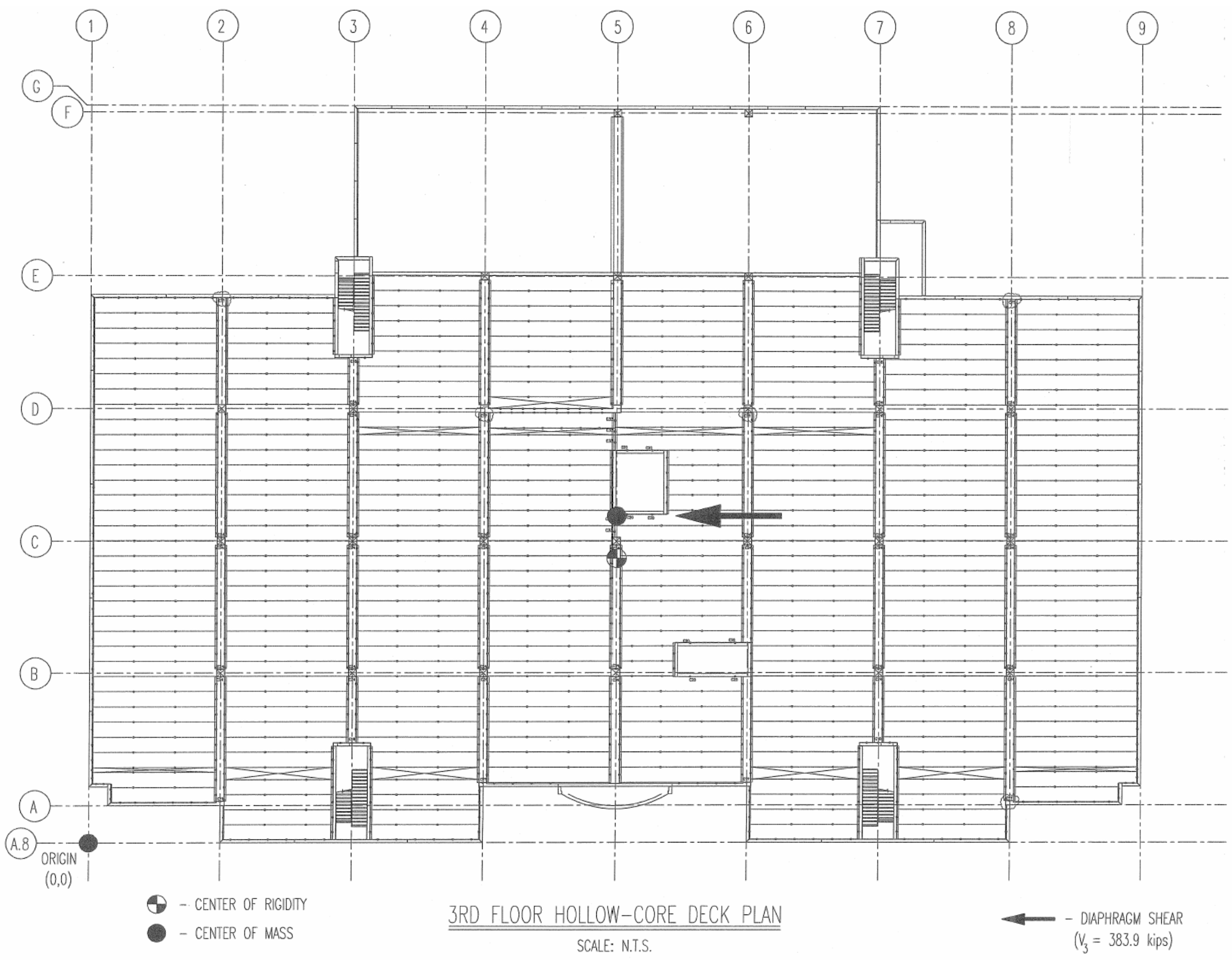
- Allowable Story Drift $\leq$ **0.020h$_{sx}$**	=	**9.6" @ Roof**
		6.7" @ 3rd Floor
		3.8" @ 2nd Floor
- Calculated Story Drifts	-	**0.064" @ Roof**
		0.038" @ 3rd Floor
		0.015" @ 2nd Floor

As can be seen, drift is very limited thus proving the assumption stated above. Concluding, seismic story drift is not an issue.

ACCIDENTAL TORSION:

Analogous to drift, concern of torsional forces that may occur due to lateral loading for the office building were neglected. As you can see from the floor plans in Appendix A, the building's design is symmetrical in shape. Also, from the lateral analysis portion of this report, the Center of Mass (COM) & Center of Rigidity (COR) differs slightly from floor-to-floor. Simply because of this fact, a great amount of torsion will not be induced upon the building and may be neglected.

Torsion was determined for Tech Report III by using COM and COR values obtained in previous sections to determine the eccentricity (e) between the two. It was also found, per 12.8.4.2 (ASCE 7-05) that the minimum moment arm to be considered for accidental torsion shall include five-percent (5%) of the dimension of the structure normal to the applied force. After determination of each floors eccentricity, the diaphragm shear at each level was applied to the COM to obtain the torsional moment per floor. Maximum torsional moment occurs at the 3rd level in both directions. The figure below illustrates the worst case of eccentricity at the 3rd floor level, thus leading to high torsion. See Torsion calculations in Appendix B for detailed calculations.



3RD FLOOR HOLLOW-CORE DECK PLAN
SCALE: N.T.S.

-- CONCLUSIONS --

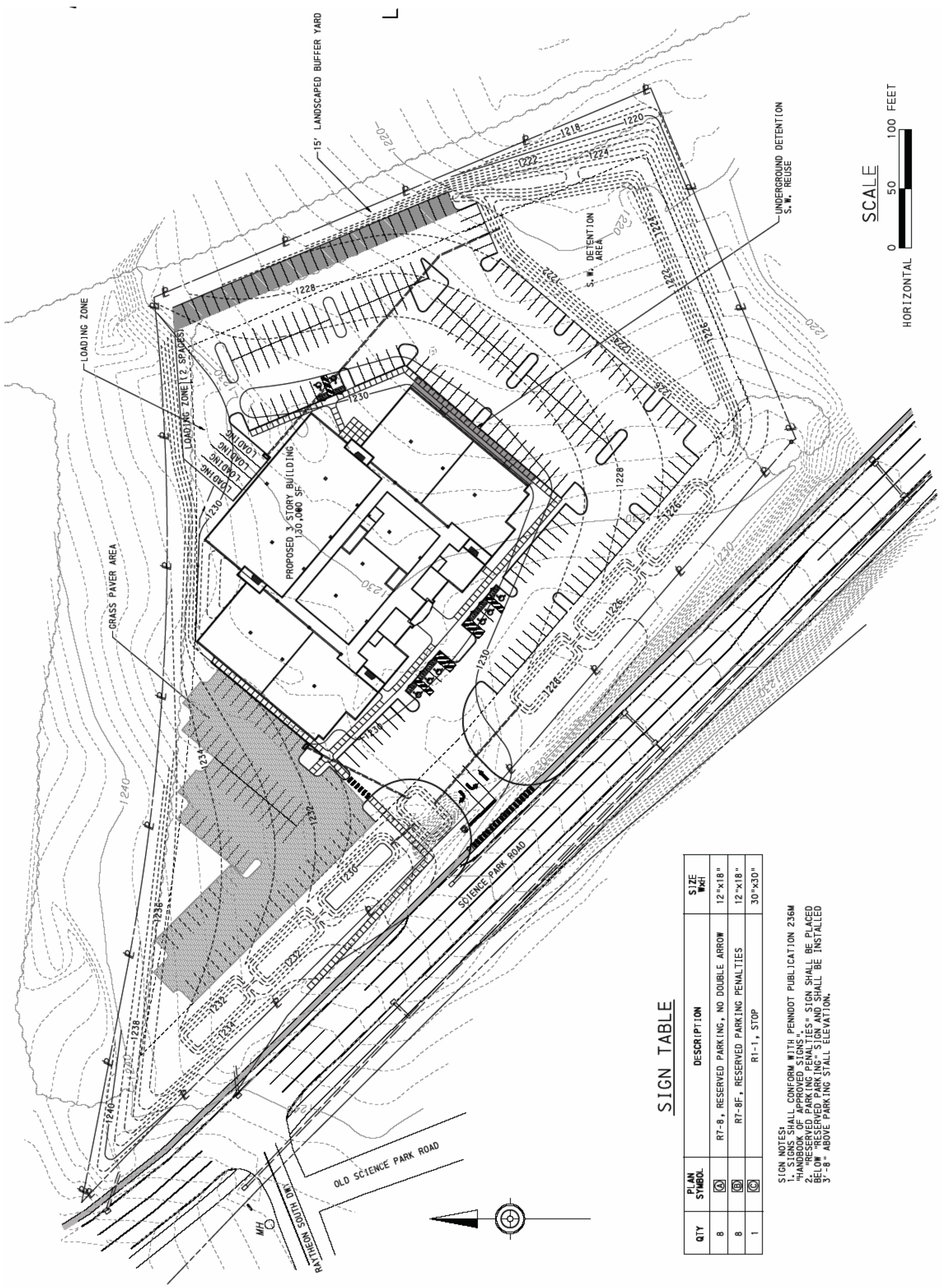
In addition to actually performing a portion of the lateral analysis for CWB, Tech Report III helped gain a better understanding of how lateral loads are distributed and resisted within a structure.

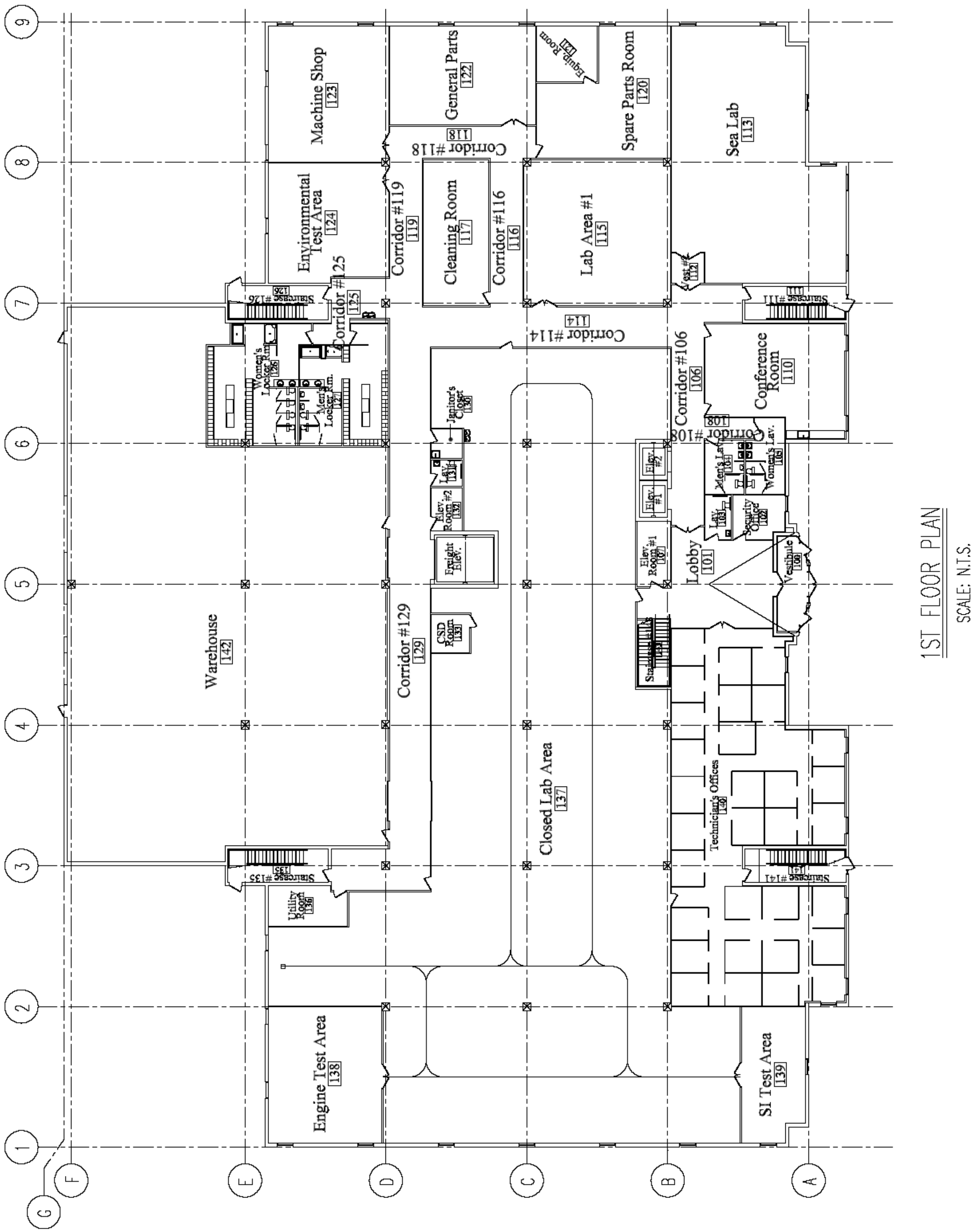
Lateral loading applied to the building, whether wind or seismic, will be collectively distributed throughout the entire building. The seismic forces that governed the design will be distributed either through the floor diaphragms, or directly to the nearly 30 shear walls designed for the Crocker West project. Having personally designed several of the designated shear walls, I can conclude that a logical load path for the calculated loads does exist and can be considered a valid design.

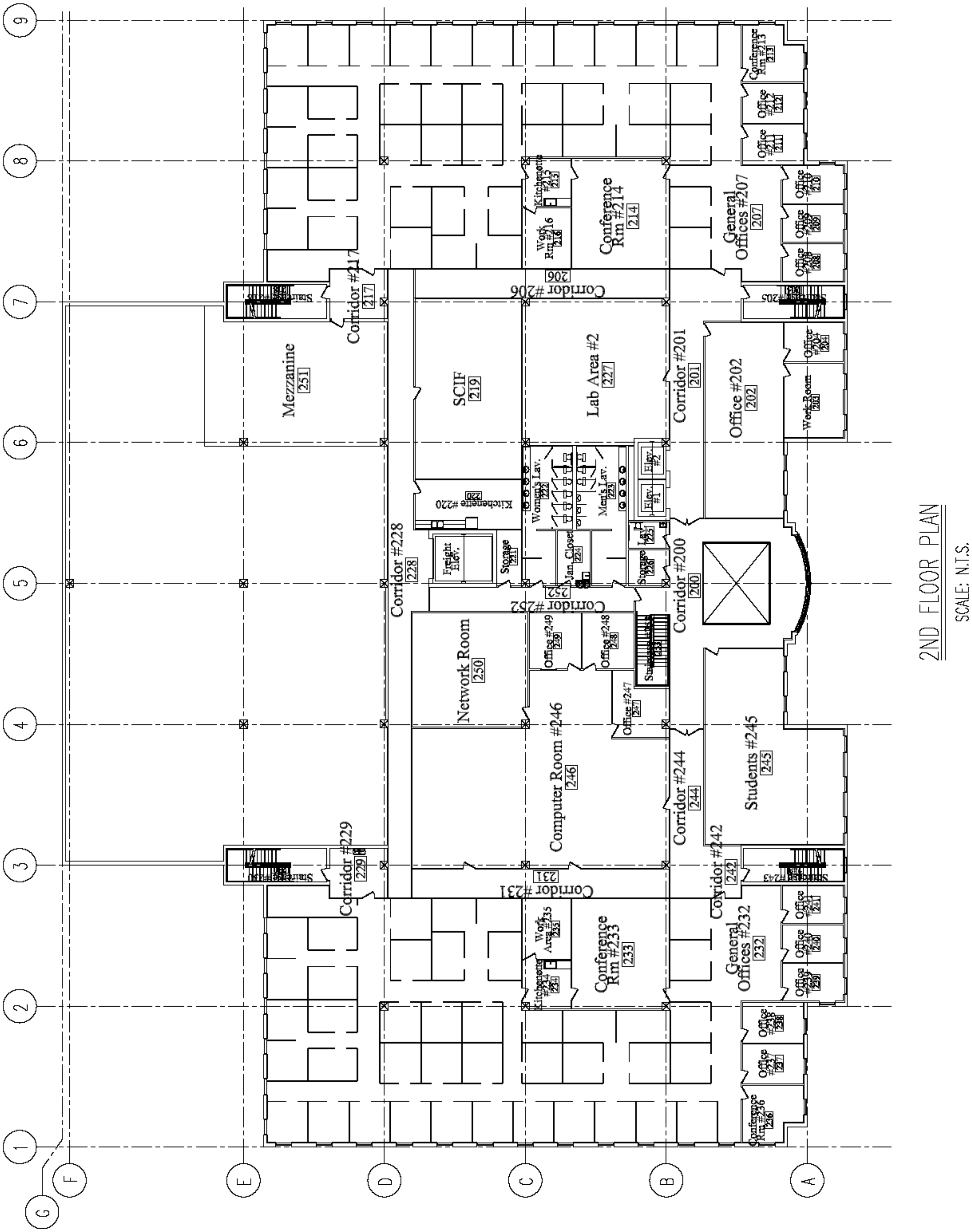
Although drift and torsion did not play a key factor in this particular design, I feel this criterion will be of greater importance in future research; particularly when a redesign of the existing structure is considered for the second semester portion of the senior thesis project.

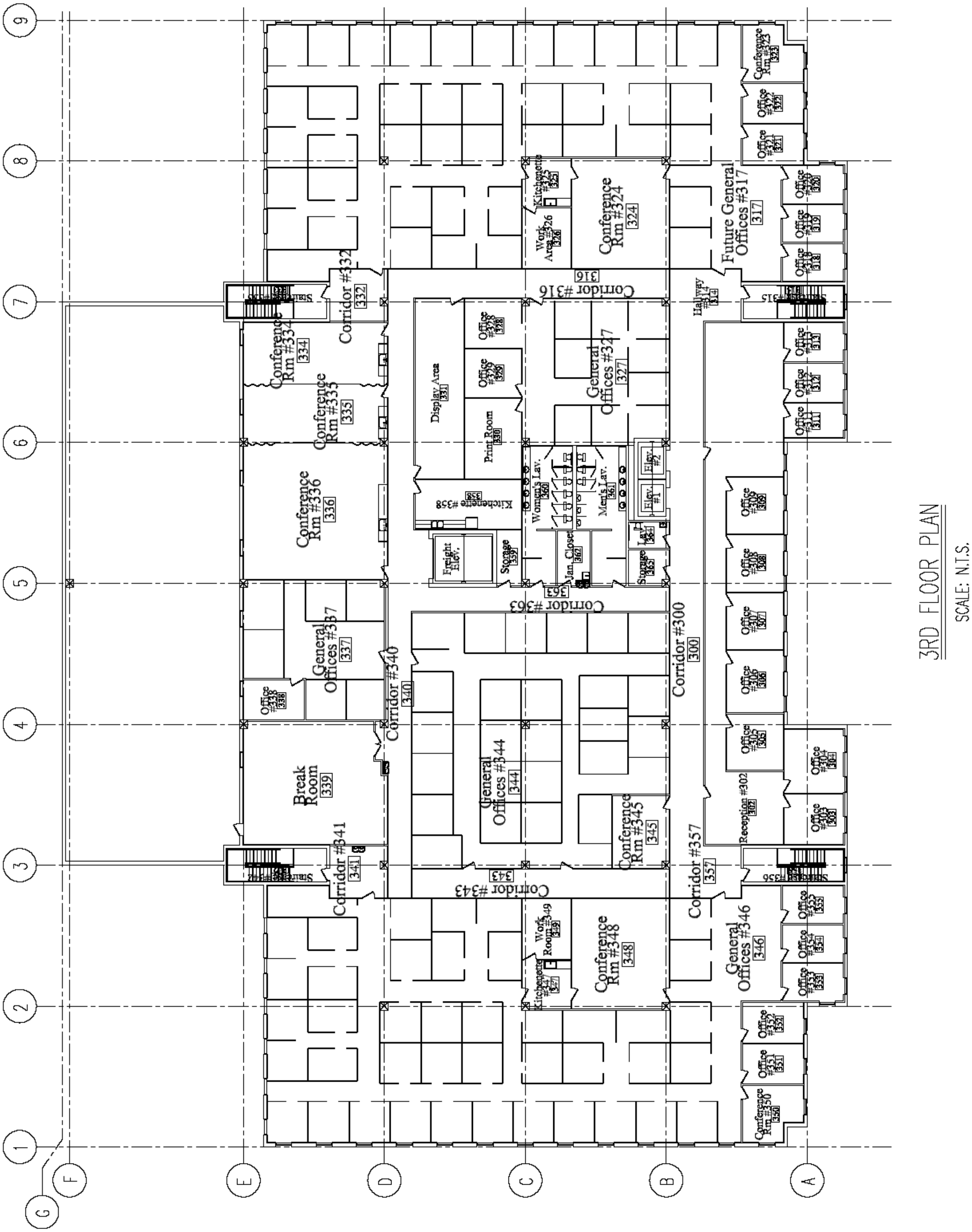
APPENDIX A

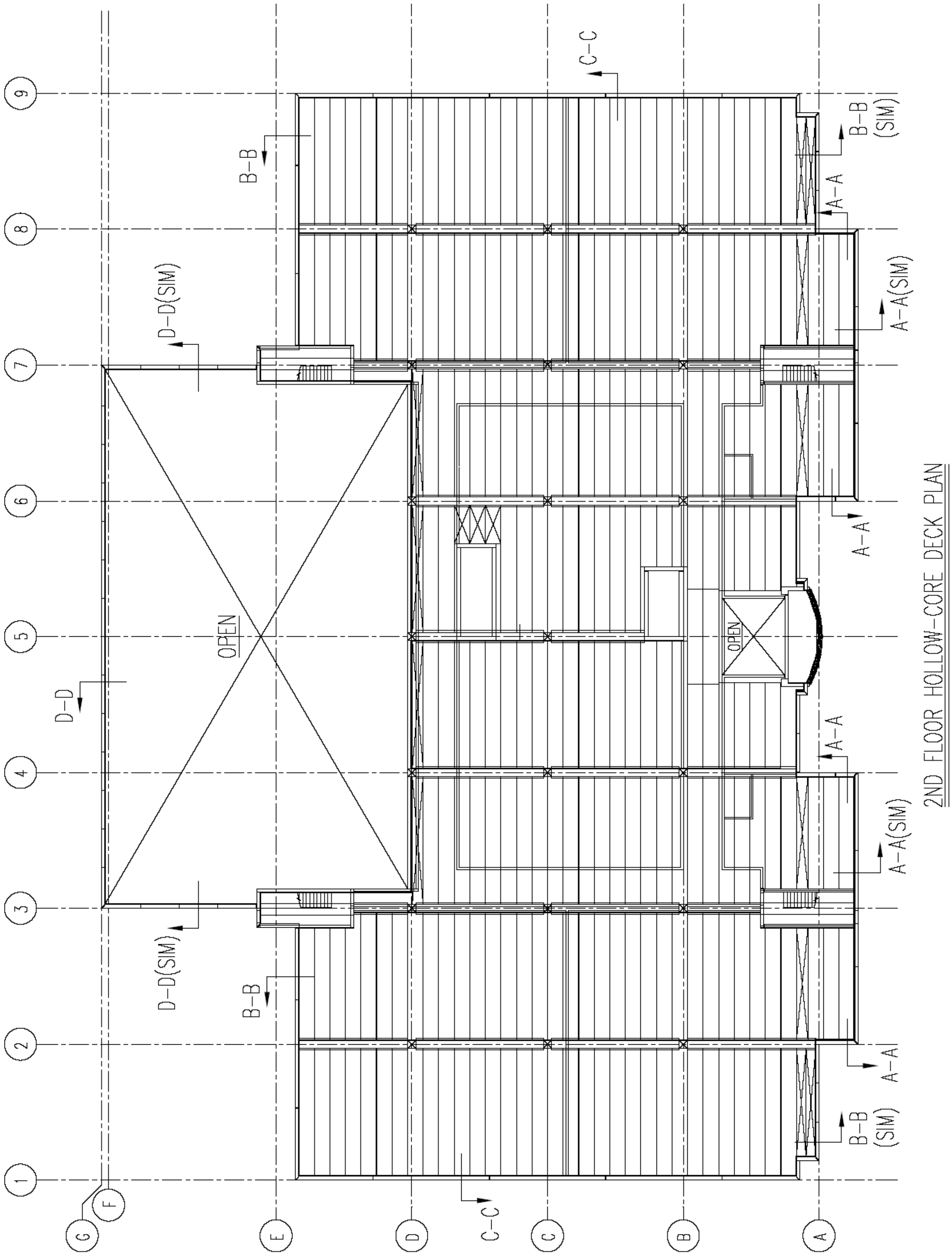
(Project Drawings)

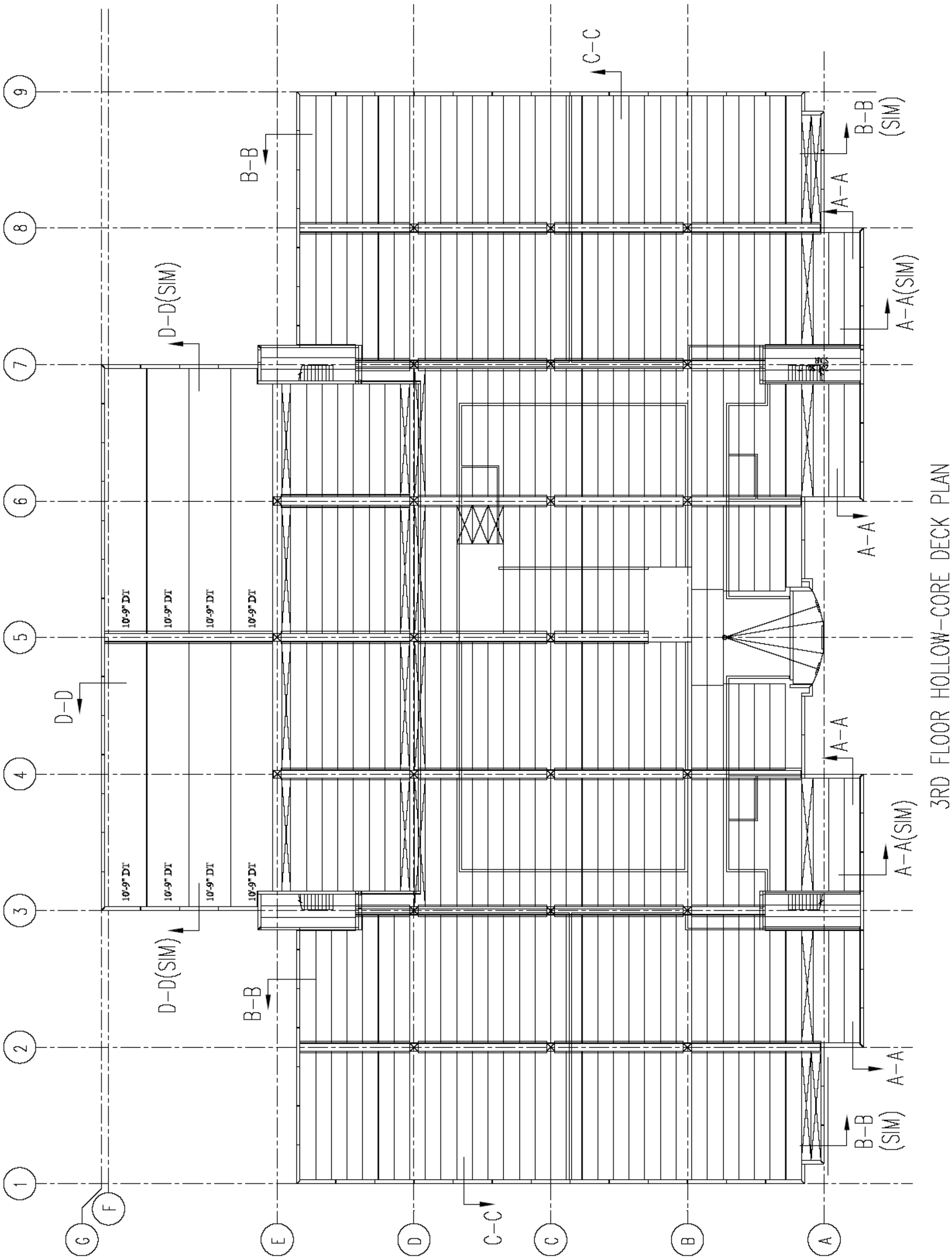

15' LANDSCAPED BUFFER YARD
UNDERGROUND DETENTION S.W. REUSE
S.W. DETENTION AREA
SCALE
HORIZONTAL
0 50 100 FEET
LOADING ZONE
LOADING ZONE (2 SPACES)
LOADING
LOADING
PROPOSED 3 STORY BUILDING 30,000 SF
GRASS PAVER AREA
SCIENCE PARK ROAD
OLD SCIENCE PARK ROAD
RAYTHEON SOUTH DWY
MH
SIGN TABLE
QTY | PLAN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

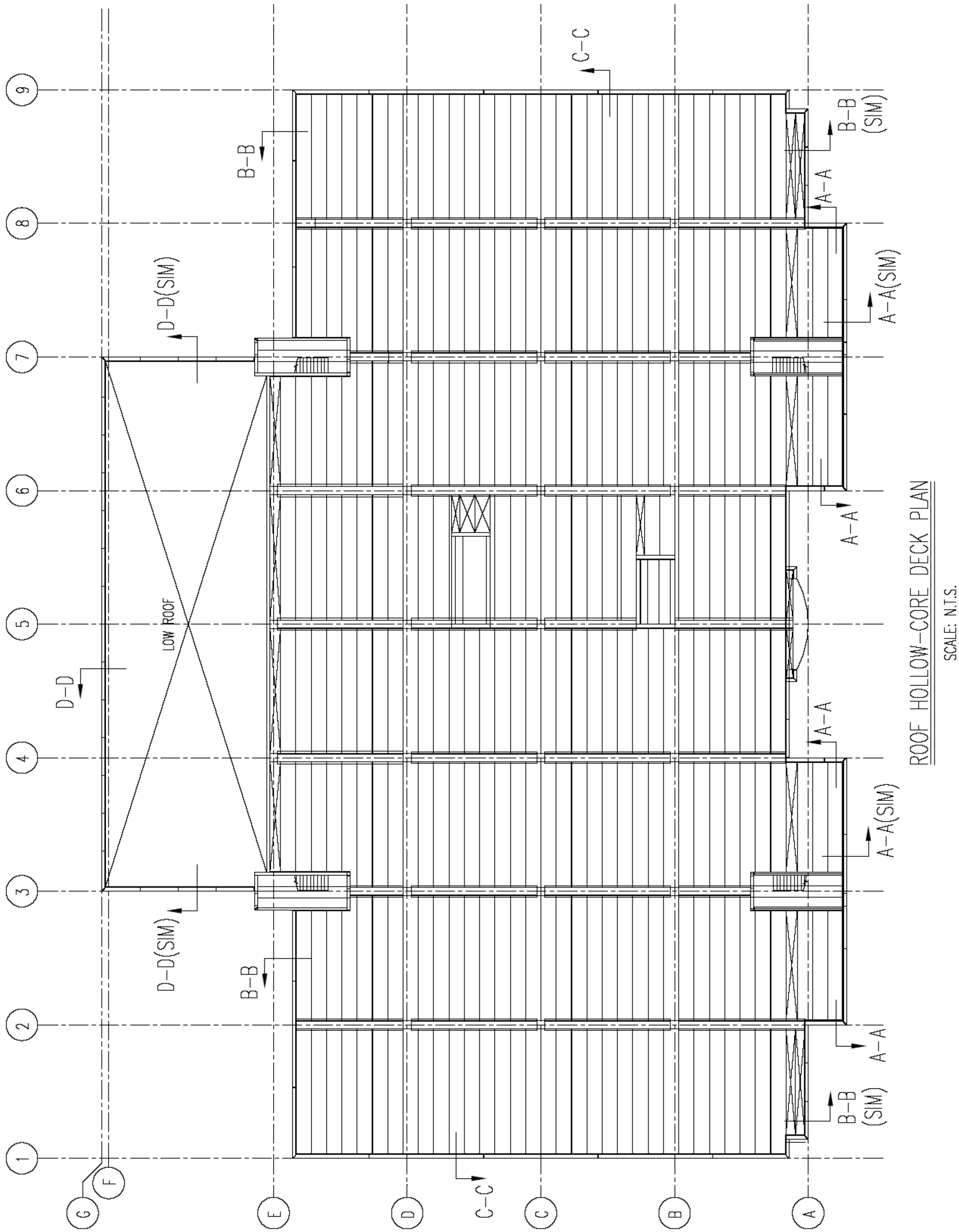

Machine Shop 123
General Parts 122
Equip. Room 121
Spare Parts Room 120
Sea Lab 113
Corridor #118 118
Environmental Test Area 124
Corridor #119 119
Cleaning Room 117
Corridor #116 116
Lab Area #1 115
Vest # 112
Corridor #125 125
Staircase #126 126
Staircase #111 111
Corridor #114 114
Corridor #106 106
Conference Room 110
Women's Locker Rm. 128
Men's Locker Rm. 127
Janitor's Closet 130
Corridor #108 108
Men's Lav. 104
Women's Lav. 105
Elev. #2
Elev. #1
Lav. 131
Elev. Room #2 132
Security Office 102
Lav. 103
Freight Elev.
Elev. Room #1 107
Lobby 101
Vestibule 100
Warehouse 142
Corridor #129 129
CSD Room 133
Closed Lab Area 137
Technician's Offices 140
Staircase #135 135
Staircase #141 141
Utility Room 136
Engine Test Area 138
SI Test Area 139
1ST FLOOR PLAN
SCALE: N.T.S.

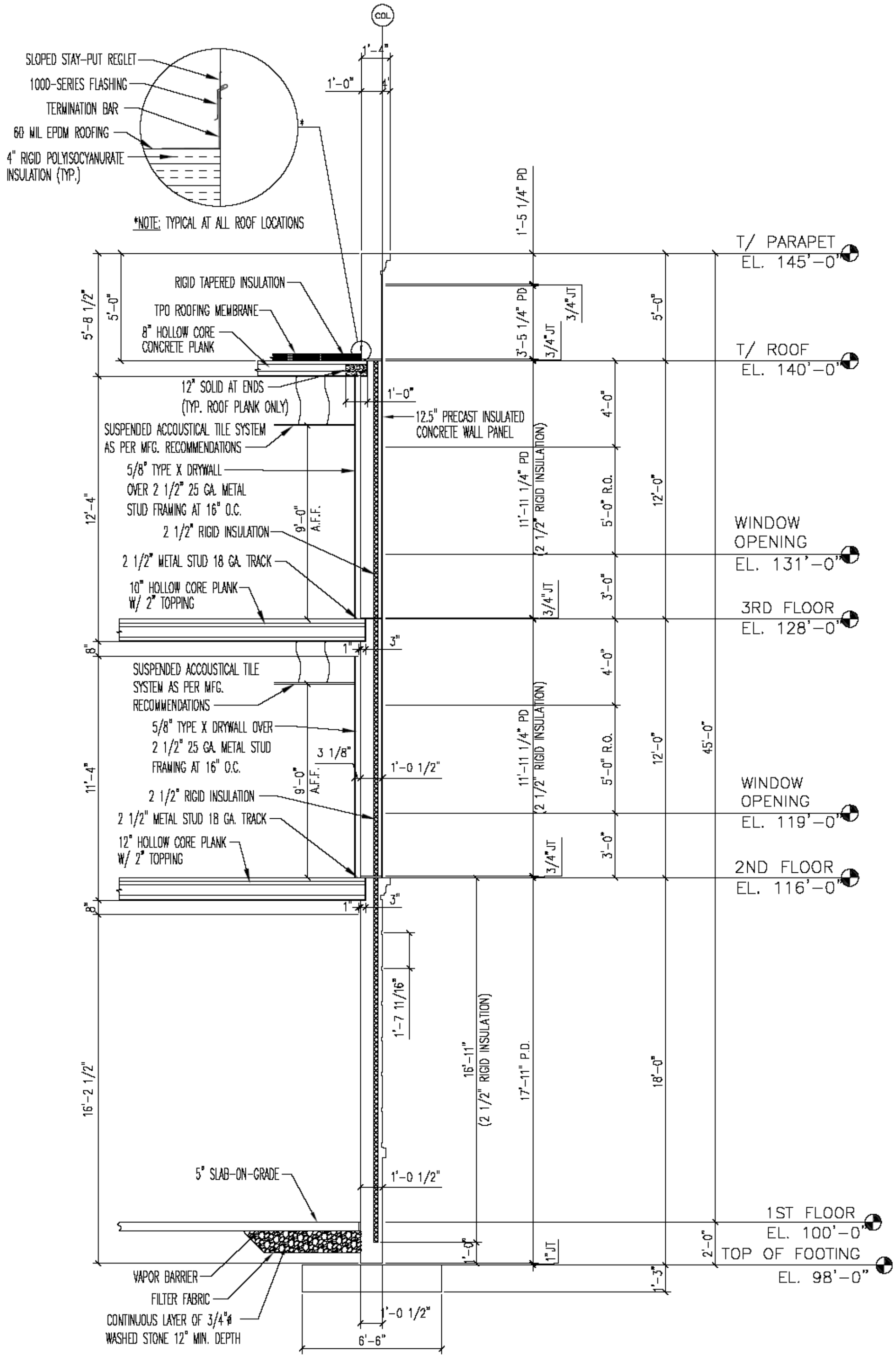

2ND FLOOR PLAN
SCALE: N.T.S.


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)

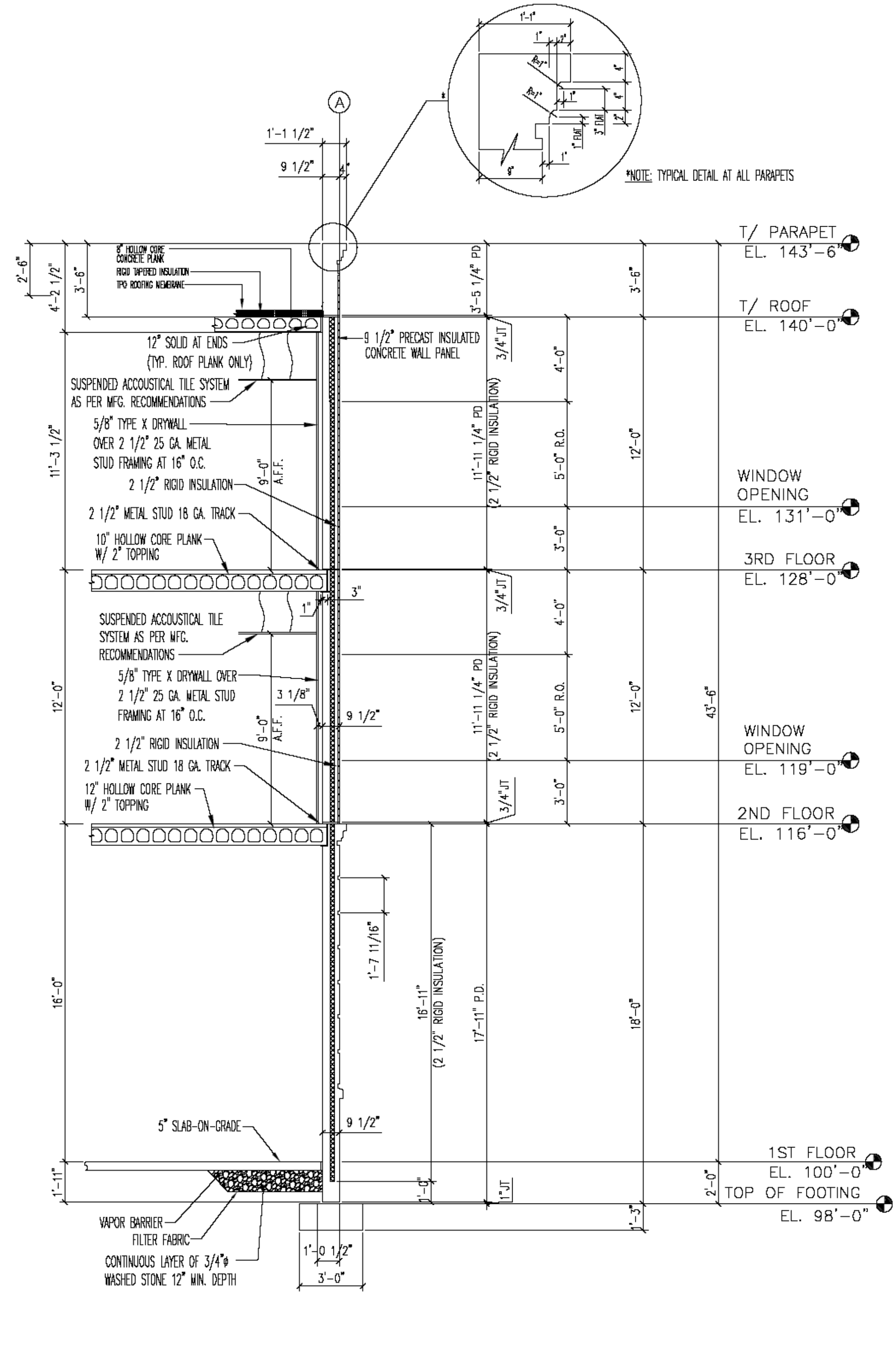

3RD FLOOR HOLLOW-CORE DECK PLAN
SCALE: N.T.S.
A-A
A-A(SIM)
B-B
B-B (SIM)
C-C
D-D
D-D(SIM)
10'-9" DT


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



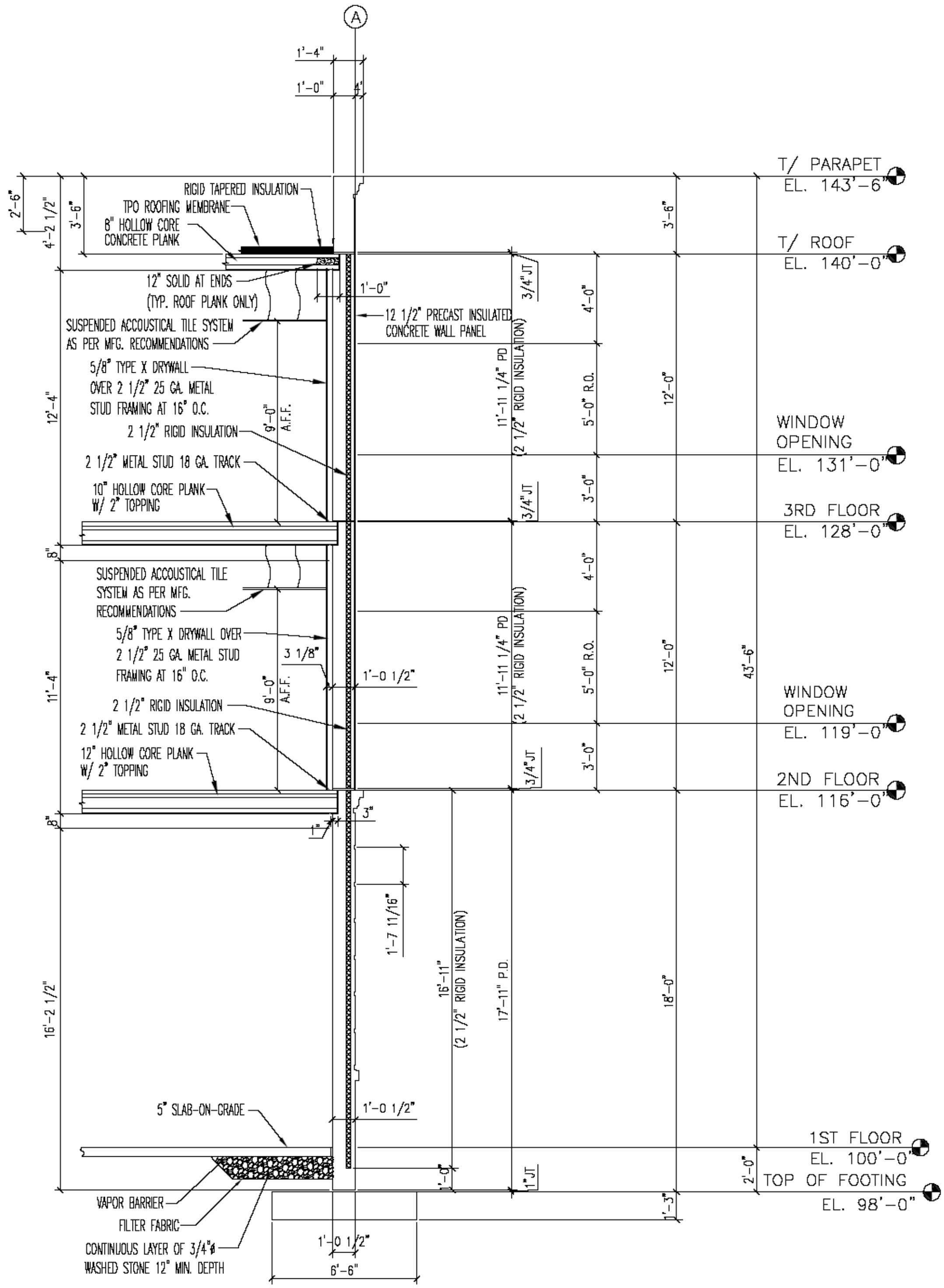
S E C T I O N   A — A

SCALE 3/8" = 1'-0"



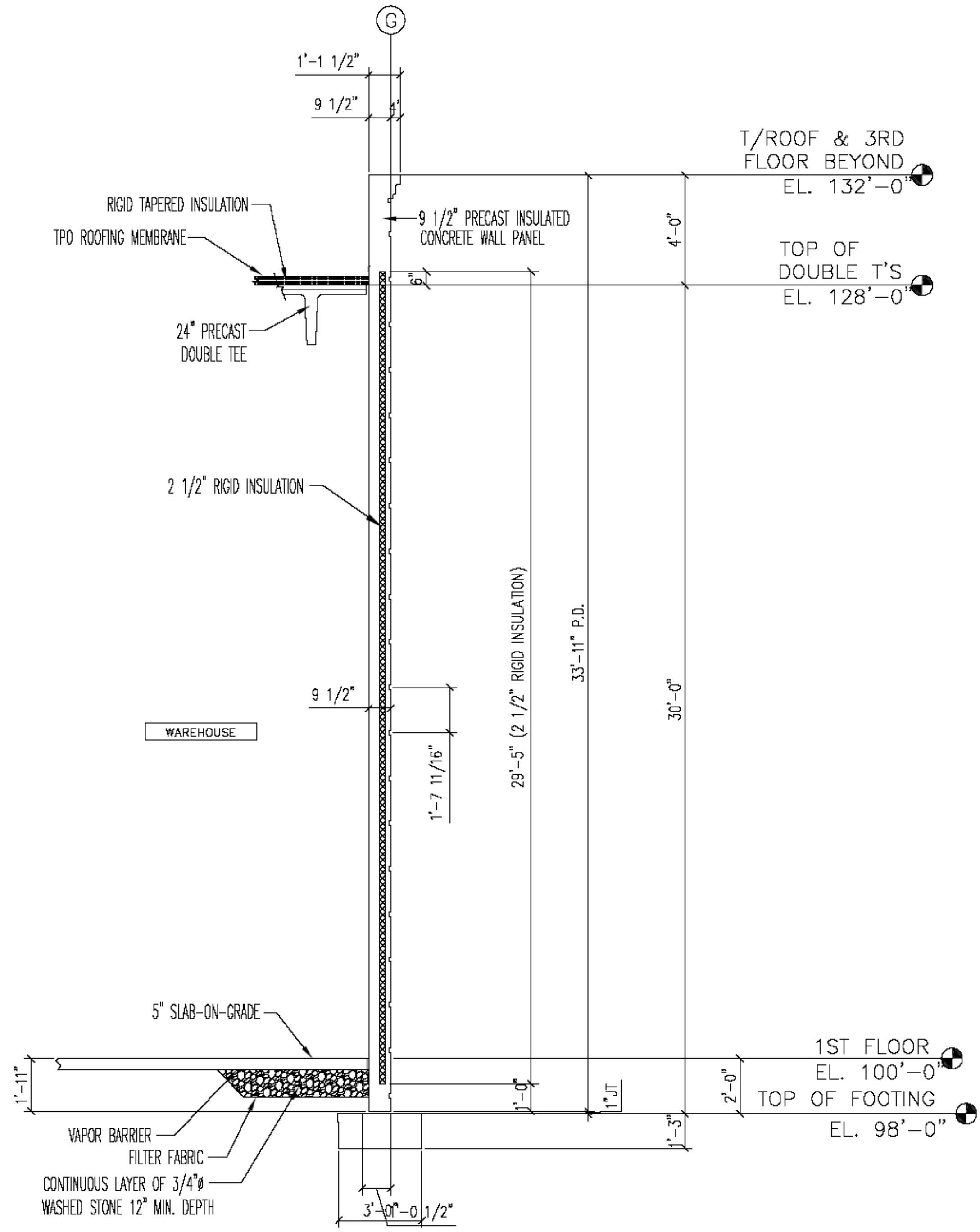
S E C T I O N B – B

SCALE 3/8" = 1'–0"



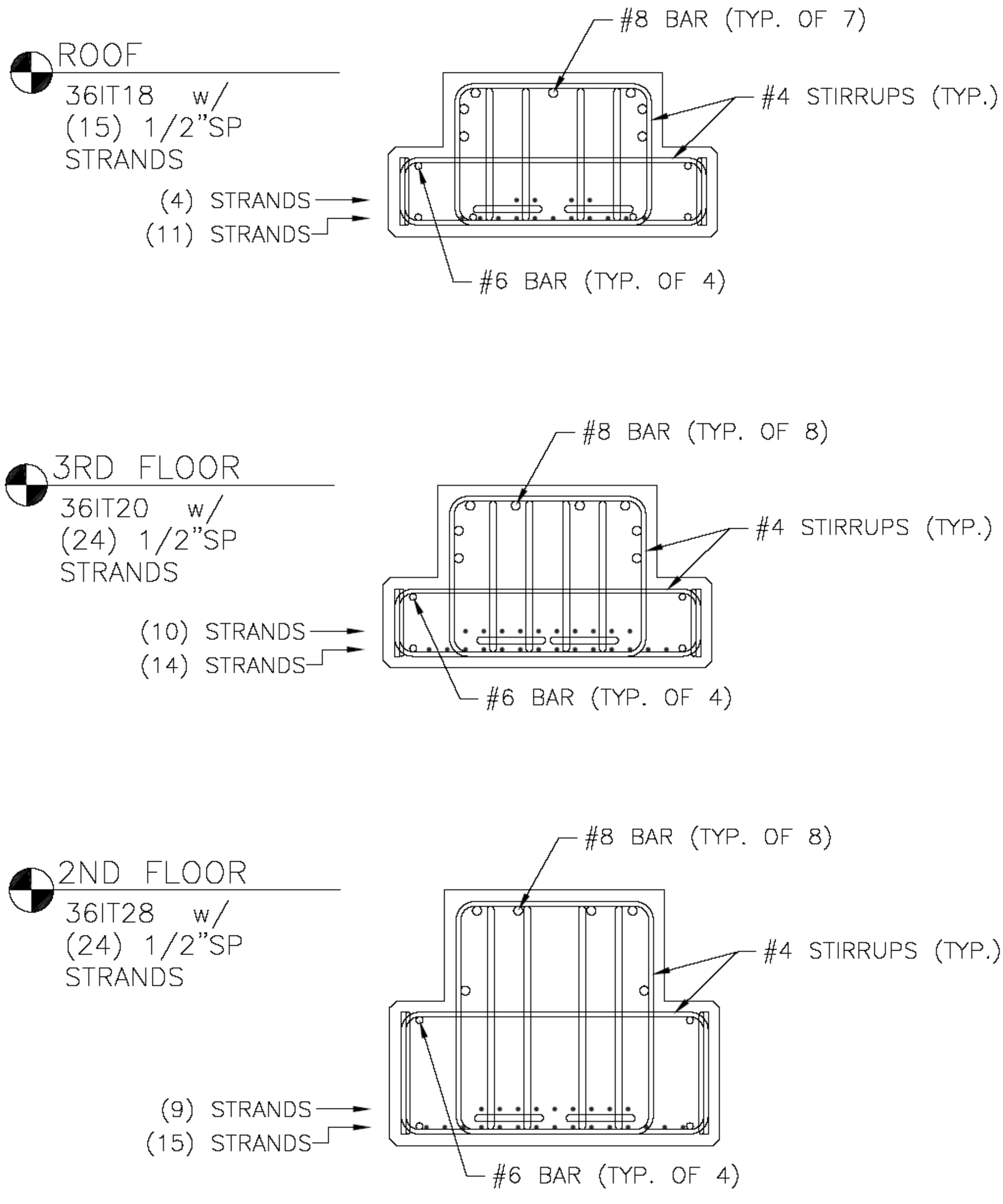
SECTION C-C

SCALE 3/8" = 1'-0"



SECTION D-D

SCALE 3/8" = 1'-0"



TYPICAL IT BEAM SECTIONS

Appendix B

(Seismic Load Distribution Calculations)

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	3908.6
3rd Flr	30.00	6101.2
2nd Flr	18.00	5226.1
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

15235.83 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: II
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): 925.5 kips, distrubuted to diaphragms as shown below:

Diaphragm Level	Diaphragm Elevation (ft)	Diaphragm Shear (kips)
Roof	42.00	344.3
3rd Flr	30.00	383.9
2nd Flr	18.00	197.3
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

Summary of Lateral Load Resisting Elements and the Distribution of the Diaphragm Shears to them:

*Tabulated forces are in kips

	Diaphragm Level							
	Roof	3rd Flr	2nd Flr	0	0	0	0	Ground
East-West Elements								
SWAW	26.4	29.6	15.2	0.0	0.0	0.0	0.0	0.0
SWAE	26.4	29.6	15.2	0.0	0.0	0.0	0.0	0.0
SWA.2W	8.2	9.1	4.7	0.0	0.0	0.0	0.0	0.0
SWA.2E	8.2	9.1	4.7	0.0	0.0	0.0	0.0	0.0
SWA.8W	38.7	43.5	22.2	0.0	0.0	0.0	0.0	0.0
SWA.8E	38.7	43.5	22.2	0.0	0.0	0.0	0.0	0.0
SWD	125.7	139.4	72.0	0.0	0.0	0.0	0.0	0.0
SWEW	34.5	38.6	19.8	0.0	0.0	0.0	0.0	0.0
SWEE	34.5	38.6	19.8	0.0	0.0	0.0	0.0	0.0
SWE2	3.6	3.9	2.0	0.0	0.0	0.0	0.0	0.0
SWE3	11.4	12.6	6.6	0.0	0.0	0.0	0.0	0.0
SWG	0.0	7.0	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
North-South Elements								
SW1	105.8	115.1	61.4	0.0	0.0	0.0	0.0	0.0
SW9	105.8	115.1	61.4	0.0	0.0	0.0	0.0	0.0
SW4	3.2	3.5	1.8	0.0	0.0	0.0	0.0	0.0
SW6	3.2	3.5	1.8	0.0	0.0	0.0	0.0	0.0
SWS1a	19.0	20.7	11.0	0.0	0.0	0.0	0.0	0.0
SWS1b	18.9	20.6	10.9	0.0	0.0	0.0	0.0	0.0
SWS1c	18.9	20.5	10.9	0.0	0.0	0.0	0.0	0.0
SWS1d	19.0	20.7	11.0	0.0	0.0	0.0	0.0	0.0
SWS1e	17.7	19.3	10.2	0.0	0.0	0.0	0.0	0.0
SWS1f	18.9	20.6	10.9	0.0	0.0	0.0	0.0	0.0
SWS1g	18.9	20.5	10.9	0.0	0.0	0.0	0.0	0.0
SWS1h	17.7	19.3	10.2	0.0	0.0	0.0	0.0	0.0
SWE1a	5.6	6.1	3.2	0.0	0.0	0.0	0.0	0.0
SWE1b	5.7	6.2	3.3	0.0	0.0	0.0	0.0	0.0
SW3	0.0	5.0	0.0	0.0	0.0	0.0	0.0	0.0
SW7	0.0	5.0	0.0	0.0	0.0	0.0	0.0	0.0

Forces summarized in the table above are based on the following Centers of Mass and Centers of Rigidity for each floor:

Center of Mass								
X Direction	140.2	140.2	141.8	0.0	0.0	0.0	0.0	0.0
Y Direction	77.2	86.6	69.7	0.0	0.0	0.0	0.0	0.0
Center of Rigidity								
X Direction	140.2	140.2	140.2	76.8	76.8	76.8	76.8	76.8
Y Direction	73.5	75.5	73.5	41.3	41.3	41.3	41.3	41.3

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COMMENTS
Seismic Loads

[Reference(s) → ASCE 7-05 (Ch. 11 & 12) & IBC 2006 (Section 1613)]

Seismic Considerations

- From Tech I and per Centre Region Code : $S_s = 0.17$
  $S_1 = 0.06$

Site Class: D (Assumed due to sink-holes throughout site)

[Table 1613.5.3 (1 & 2)]

$S_{MS} = F_a S_s = 1.6(0.17) = 0.272$

$S_{M1} = F_v S_1 = 2.4(0.06) = 0.144$

$S_{DS} = 2/3\, S_{MS} = 0.67(0.272) = 0.1813$

$S_{D1} = 2/3\, S_{D1} = 0.67(0.144) = 0.096$

Using Occupancy Category II Ref Table 1604.5

Short - Period Design Cat. = B [Table 1613.5.6(1)]

1 - Second Period Design Cat. = B [Table 1613.5.6(2)]

∴ Use Design Category B

From Table 12.2.1 (ASCE 7-05)

Response Modification Coeff. → R = 3
Deflection Amplification Factor → $C_d = 3$

} #6. Ordinary Precast Shear Walls (Bearing Wall Systems)

From Spreadsheet

$\boxed{V = 925.5^k}$ (Seismic Base Shear)

344.3^k	383.9^k	197.3^k
Roof	3rd	2nd

Determine Controlling Seismic Load Combination (ASCE 7-05) (2.3.2 and 12.4.2.3)

(5) $1.2D + 1.0E + 1.0L + 0.2S$

$(1.2 + 0.2\underbrace{S_{DS}}_{(0.182)})D + \rho Q_E + 1.0L + 0.2S$

$= 1.24D + 1.0E + 1.0L + 0.2S$

(7) $0.9D + 1.0E + \cancel{1.6H}$

$(0.9 - 0.2S_{DS})D + \rho Q_E$

$= 0.86D + 1.0E$ ★ - Controls

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COMMENTS
SEISMIC LOAD DISTRIBUTION

LOW ROOF AREA → DISTRIBUTE LOADS TO SHEAR WALLS
SWG (E-W DIRECTION)
SW3/SW7 (N-S DIRECTION)
} CONSIDERED AS **FLEXIBLE DIAPHRAGM**

■ WALL SWG

1. ASSUME 75% OF LOW ROOF LOAD APPLIED TO SWG

DT'S = 291.3^K
HC'S = 233^K
IT = 37^K
561.3^K

$561.3(0.75) = 421^K$

$V = C_s W$
$= 0.0607(421)$
$V = 25.6^K$ (APPLIED AT LOW ROOF ELEV.)

2. APPLY WT. OF "LEANING WALLS" ALONG GRID LINE G
→ USING (3) PANELS AS SW'S AT EACH END [6 TOTAL]
LEAVES THE REMAINING (8) PANELS AS "LEANING"

$(8 \text{ PNL'S}) \times (10' \text{ WIDE} \times 34' \text{ HIGH}) \times (87.5 \text{ PSF}) / 1000 = 238^K$
PANEL DIM'S; 9 1/2" WALL W/ 2 1/2" INSUL.

$V = 0.0607(238) = 14.446$
$V = 14.4^K$ (APPLIED AT WALL CENTROID HT.)

3. LOAD AT EACH WALL PANEL

$V_R = 25.6^K / 6 \text{ PNL'S} = 4.267$ $\quad V_R = 4.3^K/\text{PANEL}$ (APPLIED AT LOW ROOF ELEV.)

$V_M = 14.4^K/6 = 2.4$ $\quad V_M = 2.4^K/\text{PNL}$ + SELF WT. (APPLIED AT WALL CENTROID HT.)

■ WALL(S) SW3 & SW7

1. ASSUME 60% OF LOW ROOF LOAD TO SW3 (SAME FOR SW7)
$561.3^K(0.60) = 336.78$
$V = 20.4^K$ (APPLIED AT LOW ROOF ELEV.)

2. USING ALL (4) PANELS AS SW'S (NO "LEANING WALLS")

3. $V_R = 20.4^K/4 = 5.1^K/\text{PANEL}$ (APPLIED AT LOW ROOF ELEV.)

V_M = PANEL SELF WT. (APPLIED AT WALL CENTROID HT.)

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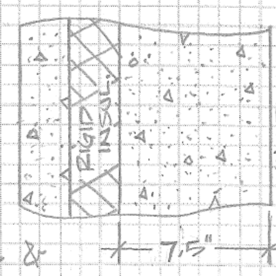
COMMENTS
SEISMIC LOAD DISTRIBUTION

■ WALL SW1 (WALL SW9 SAME) → BOTH 12½" THICK



FROM EXCEL SS → LOADS: @ ROOF = 105.8^k
3RD = 115.1^k
2ND = 61.4^k

DETERMINE AMOUNT OF LOAD APPLIED TO 30' LONG WALLS & 19.83' LONG WALLS

$$I_{30} = \frac{bh^3}{12} = \frac{(7.5")(30' \cdot 12)^3}{12} = 29{,}160{,}000 \text{ in}^4$$

$$I_{19} = 8{,}425{,}790 \text{ in}^4$$

$$I_T = 104{,}331{,}580 \text{ in}^4$$

$\frac{I_{30}}{I_T} = 0.279 \rightarrow$ 28% OF LOAD TO EACH 30' WALL

$\frac{I_{19}}{I_T} = 0.081 \rightarrow$ 8% " " " " 19.83' WALL

LOADS AT 30' WALL (BEARING WALLS)

ROOF = 105.8 × 28% = 29.6^k
3RD = 115.1 × 28% = 32.2^k
2ND = 61.4 × 28% = 17.2^k

LOADS AT 19.83' WALL (BEARING WALLS)

ROOF = 105.8 × 8% = 8.5^k
3RD = 115.1 × 8% = 9.2^k
2ND = 61.4 × 8% = 4.9^k

■ WALL SWAE (WALL SWAW SAME)

FROM EXCEL SS → ROOF = 26.4^k
3RD = 29.6^k
2ND = 15.2^k

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WALL SWA.8E (WALL SWA.8W SAME) → 9½" NLB WALL
- 2½" FACE
- 2½" RIGID INSUL.
- 4½" INTERIOR CONC.

FROM EXCEL SS → ROOF = 38.7^K
3RD = 43.5^K
2ND = 22.2^K

(1) 30' WALL ~ (2) 20' WALLS

$I_{30} = \frac{(4\frac{1}{2}")(30' \cdot 12)^3}{12} = 17,496,000 \text{ in}^4$ $I_{20} = 5,184,000 \text{ in}^4$

$I_T = 27,864,000 \text{ in}^4$

63% OF LOAD TO 30' WALL ~ 19% OF LOAD TO EACH 20' WALL

ROOF = 24.4^K ROOF = 7.4^K
3RD = 27.4^K 3RD = 8.3^K
2ND = 14.0^K 2ND = 4.2^K

WALL SWEE (WALL SWEW SAME) → 9½" NLB

FROM EXCEL SS → ROOF = 34.5^K
3RD = 38.6^K
2ND = 19.8^K

(1) 30' WALL ~ (1) 20' WALL ~ (1) 15' WALL

$I_{30} = 17,496,000 \text{ in}^4$
$I_{20} = 5,184,000 \text{ in}^4$
$I_{15} = 2,187,000 \text{ in}^4$

$I_T = 24,867,000 \text{ in}^4$

30 (70%)
20 (21%)
15 (9%)

30' WALL LOADS → (NLB) ROOF = 24.2^K
3RD = 27.0^K
2ND = 13.9^K

20' WALL LOADS → (NLB) ROOF = 7.2^K
3RD = 8.1^K
2ND = 4.2^K

15' WALL LOADS → (NLB) ROOF = 3.1^K
3RD = 3.5^K
2ND = 1.8^K

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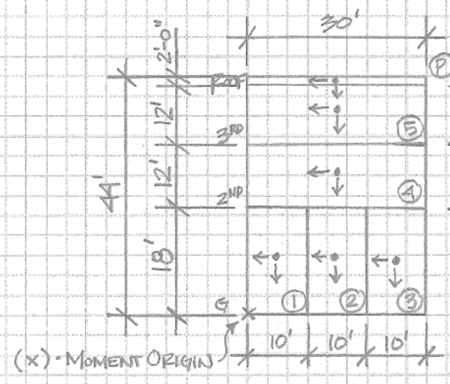
SEISMIC LOAD DISTRIBUTION

30'-0" BUILT-UP SHEAR WALL (BEARING WALL)

* DISTRIBUTION OF LOAD TO INDIVIDUAL PANELS ALONG SW1 & SW9



$V_{E,ROOF} = 29.6^K$

$V_{E,3RD} = 32.2^K$

$V_{E,2ND} = 17.2^K$

FROM PG. 3 OF THIS SECTION

∴ 12½" WALL THK. → 10" NWC = 125 PSF
2½" R. INSUL.

SELF WT.'S →

PANELS ① ② ③ : $(10 \times 18)(125)/_{1000} = 22.5^K$ PER PANEL

④, ⑤ : $(30 \times 12)(125)/_{1000} = 45^K$ " "

SOLID PARAPET Ⓟ : $(30 \times 2)(156)/_{1000} = 9.4^K$

SEISMIC WT.'S →

C_s

PANELS ① ② ③ : $0.0607(22.5) = 1.4^K$

④, ⑤ : $0.0607(45) = 2.7^K$

Ⓟ : $0.0607(9.4) = 0.6^K$

DEAD LOADS →

ROOF $= 74$ PSF $(35/2)/_{1000} = 1.3$ KLF $\times 30' = 39^K$

3RD $= 101$ PSF $(35/2)/_{1000} = 1.77$ KLF $\times 30' = 53^K$

2ND $= 106$ PSF $(35/2)/_{1000} = 1.86$ KLF $\times 30' = 55.7^K$

AT BASE

OVERTURNING → $M_0 = (29.6 \times 42') + (32.2 \times 30') + (17.2 \times 18') + (0.6 \times 43') + (2.7 \times 36') + (2.7 \times 24') + 3(1.4 \times 9') = 2744.4$ FT-K

RESISTING → (USING 86% OF DL COMPUTED ON PG. 1 OF THIS SECTION)

$M_R = [(22.5 \times 5') + (22.5 \times 15') + (22.5 \times 25') + 2(45 \times 15') + (9.4 \times 15') + (39 + 53 + 55.7)(15')]\, 0.86$

$M_R = 4058.34$ FT-K

($M_R > M_0$ ∴ NO DESIGN MOMENT @ BASE)

BASE SHEAR → $V_u = 29.6^K + 32.2^K + 17.2^K + 3(1.4) + 2(2.7) + 0.6$

$V_u = 89.2^K$ (@ 30' WALL)

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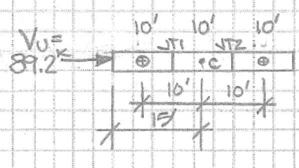
LOADS AT HORIZ. JTS

SHEAR @ ROOF = 0.6^K

" @ 3^{RD} = $0.6^K + 29.6^K + 2.7^K = 32.9^K$

" @ 2^{ND} = $0.6^K + 29.6^K + 2(2.7^K) + 32.2^K = 67.8^K$

LOADS AT VERT. JT'S



VERT. SHEAR @ JOINT(S) 1 & 2

$Q = A\bar{y}_1 = (10' \times 7.5/12)(10') = 62.5\ FT^3$

$I = \dfrac{(7.5/12)(30)^3}{12} = 1406.25\ FT^4$

VERT. SHEAR $q = \dfrac{VQ}{I} = \dfrac{(89.2^K)(62.5\ FT^3)}{1406.25\ FT^4} = 3.96\ ^K/_{FT}$

LOADS AT FOUNDATION CONNECTIONS

* NO DESIGN MOMENT @ BASE → SHEAR ONLY

USING (2) CONN'S PER PANEL →

$V_U = 89.2^K / 6\ CONN'S = 14.9^K/_{CONN.}$

AT 2^{ND} FLOOR

$M_O = (29.6^K \times 24') + (32.2^K \times 12') + (0.6 \times 25') + (2.7^K \times 18') + (2.7 \times 6') = 1176.6\ FT\text{-}K$

$M_R = 0.86\left[2(45^K \times 15') + (9.4^K \times 15') + (39^K \times 15') + (53^K \times 15')\right] = 2469.1\ FT\text{-}K$

$M_R > M_O$ → ∴ NO DESIGN MOMENT

AT 3^{RD} FLOOR

$M_O = (29.6^K \times 12') + (0.6 \times 13') + (2.7 \times 6') = 379.2\ FT\text{-}K$

$M_R = 0.86\left[(45^K \times 15') + (9.4^K \times 15') + (39^K \times 15')\right] = 1204.9\ FT\text{-}K$

$M_R > M_O$ → ∴ NO DESIGN MOMENT

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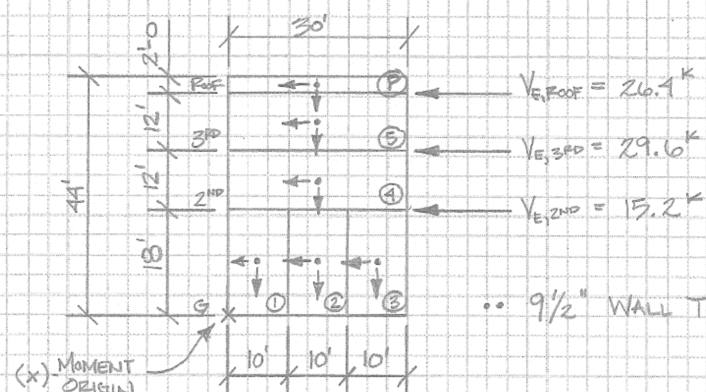
30'-0" BUILT-UP SHEARWALL (NON-LOAD BEARING WALL)

∗ WORST NLB CASE @ SWAE (WALL SWAW SAME)



∴ 9½" WALL THK. → 7" NWC = 87.5 PSF
2½" R. INSUL.

SELF WT.'S →

PANELS ① ② ③ : $87.5\text{ PSF}(10' \times 18')/_{1000} = 15.8^k$ PER PANEL

" ④ ⑤ : $87.5\text{ PSF}(12' \times 30')/_{1000} = 31.5^k$ " "

SOLID PARAPET Ⓟ : $(9\tfrac{1}{2}"/12)(150\text{ PCF})(2' \times 30')/_{1000} = 7.13^k$

SEISMIC WT'S →

PANELS ① ② ③ : $0.0607(15.8^k) = 0.956^k$

" ④ ⑤ : $0.0607(31.5^k) = 1.91^k$

Ⓟ : $0.0607(7.13^k) = 0.432^k$

AT BASE

(OVERTURNING) $M_o = (26.4^k \times 42') + (29.6^k \times 30') + (15.2^k \times 18') + (0.432^k \times 43') + (1.91^k \times 36') + (1.91^k \times 24') + 3(0.956^k \times 9') = 2429.5^{'k}$

(RESISTING) ↳ 86% DL $M_R = 0.86[(7.13^k + 31.5^k + 31.5^k)(15') + 15.8^k(5') + 15.8^k(15') + 15.8^k(25')]$

$M_R = 1514.1^{'k} < M_o$ → ∴ DESIGN MOMENT @ BASE

(DESIGN MOMENT) $M_u = M_o - M_R$ → $\underline{M_u = 915.4^{'k}}$

(BASE SHEAR) $V_u = 26.4^k + 29.6^k + 15.2^k + 0.432^k + 2(1.91^k) + 3(15.8^k)$

$\underline{V_u = 122.7^k}$

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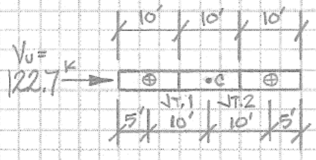
LOADS AT HORIZ. JTS.

SHEAR @ ROOF = 0.432^k

" @ 3RD = $0.432^k + 1.91^k + 26.4^k = 28.7^k$

" @ 2ND = $0.432^k + 2(1.91^k) + 26.4^k + 29.6^k = 60.3^k$

LOADS AT VERT. JT'S



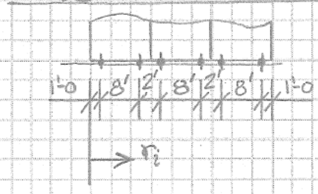
VERT. SHEAR @ JT'S 1 & 2

$$Q = A\bar{y} = \left(10' \times \tfrac{4.5''}{12}\right)(10') = 37.5 \text{ FT}^3$$

$$I = \frac{bh^3}{12} = \frac{(4.5/12)(30')^3}{12} = 843.75 \text{ FT}^4$$

$$q = \frac{VQ}{I} = 5.5^k/\text{FT}$$

LOADS AT FOUNDATION CONN.'S



$M_u = 915.4'^k$ (SEE PG. 6 OF THIS SECTION)

$$T_{u,MAX} = \frac{M_u \times r_{MAX}}{\Sigma r^2}$$

r_{MAX}

$$\Sigma r^2 = 1^2 + 9^2 + 11^2 + 19^2 + 21^2 + 29^2 = 1846 \text{ FT}^2$$

$$V_u = 122.7^k / 6 \text{ CONN.} = 20.5^k/\text{CONN.}$$

$$T_{u,MAX} = \frac{915.4'^k (29')}{1846 \text{ FT}^2} = 14.4^k$$

DESIGN TENSILE STRENGTH PER CONN. TO PREVENT UP-LIFT

AT 2ND FLOOR

$$M_o = (26.4^k \times 24') + (29.6^k \times 12') + (0.432^k \times 25') + (1.91^k \times 18') + (1.91^k \times 6') = 1045.4'^k$$

$$M_R = 0.86\left[(7.13^k + 31.5^k + 31.5^k)(15')\right] = 904.6'^k$$

(DESIGN MOMENT) → $M_u = 140.8'^k$

AT 3RD FLOOR

$$M_o = (26.4^k \times 12') + (0.432^k \times 13') + (1.91^k \times 6') = 333.9'^k$$

$$M_R = 0.86\left[(7.13^k + 31.5^k)(15')\right] = 498.3'^k$$

∴ NO DESIGN MOMENT

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COMMENTS
SEISMIC LOAD DISTRIBUTION

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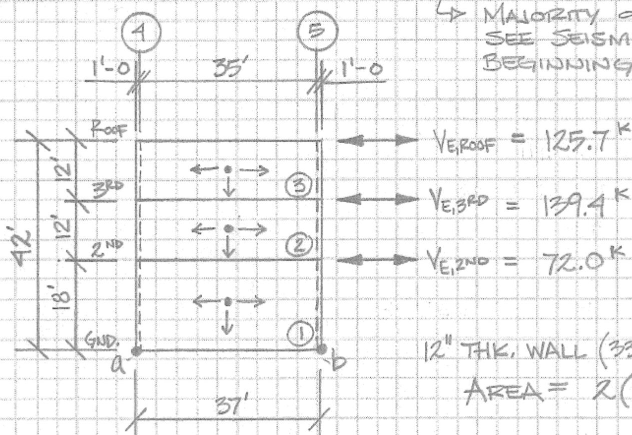
WALL SWD (MAIN SHEARWALL)

↳ MAJORITY OF LOAD DISTRIBUTED TO THIS WALL. SEE SEISMIC LOAD DISTRIBUTION SUMMARY AT THE BEGINNING OF THIS SECTION.



12" THK. WALL (33' LONG) W/ 24" × 24" PIERS @ EA. END

AREA = $2(2' \times 2') + (1' \times 33') = 41\ FT^2$

SELF WT.'S → PANEL ①: $(41\ SF \times 18')(150\ PCF)/_{1000} = 110.7^K$

PANEL ② & ③: $(41\ SF \times 12')(150)/_{1000} = 73.8^K$

SEISMIC WT.'S → PANEL ①: $0.0607(110.7) = 6.72^K$

② & ③: $0.0607(73.8) = 4.48^K$

DEAD LOADS → (AT GRID LINE 4):

ROOF = $\left[\underset{(DECK)}{80\ PSF}(35' \times 35') + \underset{(IT)}{570\ PLF}(33')\right]/_{1000} = 116.8^K$

3RD = $\left[\underset{(DECK)}{121\ PSF}(35' \times 35') + \underset{(IT)}{620\ PLF}(33')\right]/_{1000} = 168.7^K$

2ND = $\left[\underset{(DECK)}{116\ PSF}(35' \times 17.5') + \underset{(IT)}{900\ PLF}(16.5')\right]/_{1000} = 85.9^K$

TOTAL = 371.4^K

(AT GRID LINE 5):

ROOF = $\left[80(35' \times 22.5') + 570(18')\right]/_{1000} = 73.3^K$

3RD = $\left[121(35' \times 22.5') + 620(18')\right]/_{1000} = 106.4^K$

2ND = $\left[116(35' \times 5') + 900(2')\right]/_{1000} = 22.1^K$

TOTAL = 201.8^K

SHEAR → @ 3RD: $V_U = 125.7^K + 4.48^K = 129.5^K$

2ND: $V_U = 125.7^K + 139.4^K + 2(4.48) = 273.4^K$

BASE: $V_U = 125.7^K + 139.4^K + 72^K + 2(4.48^K) + 6.72^K = 352.1^K$

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✓OVERTURNING →

(ΣM_a ~ ABOUT PT. a)

• AT BASE

$M_o = 125.7^k(42') + 139.4^k(30') + 72^k(18') + 4.5^k(36') + 4.5^k(24') + 6.7^k(9')$

$M_o = 11087.7$ FT-K

$M_R = 0.86\left[371.4^k(1') + 201.8^k(36') + 2(73.8^k)(18.5') + 110.7^k(18.5')\right] = 10676.7\ ^{'k}$

→ $M_R < M_o$ ∴ DESIGN MOMENT

(DESIGN MOMENT) $M_U = M_o - M_R = 411$ FT-K

• AT 2ND FLR.

$M_o = 125.7^k(24') + 139.4^k(12') + 4.5^k(18') + 4.5^k(6') = 4797.6$ FT-K

$M_R = 0.86\left[(116.8^k + 168.7^k)(1') + (73.3^k + 106.4^k)(36') + 2(73.8^k)(18.5')\right] = 8157.4$ FT-K

→ $M_R > M_o$ ∴ NO DESIGN MOMENT

• AT 3RD FLR.

$M_o = 125.7^k(12') + 4.5^k(6') = 1535.4$ FT-K

$M_R = 0.86\left[116.8^k(1') + 73.3^k(36') + 73.8^k(18.5')\right] = 3544$ FT-K

→ $M_R > M_o$ ∴ NO DESIGN MOMENT

(ΣM_b ~ ABOUT PT. b)

• AT BASE

$M_o = 11087.7$ FT-K (SEE PREV. CALC., ABOVE)

$M_R = 0.86\left[201.8^k(1') + 371.4^k(36') + 2(73.8^k)(18.5') + 110.7^k(18.5')\right] = 15781.6\ ^{'k}$

→ $M_R > M_o$ ∴ NO DESIGN MOMENT

• AT 2ND

$M_o = 4797.6$ FT-K (SEE PREV. CALC)

$M_R = 0.86\left[(73.3^k + 106.4^k)(1') + (116.8^k + 168.7^k)(36') + 2(73.8^k)(18.5')\right] = 11341.9$ FT-K

→ $M_R > M_o$ ∴ NO DESIGN MOMENT

• AT 3RD

$M_o = 1535.4$ FT-K (SEE PREV. CALC.)

$M_R = 0.86\left[73.3^k(1') + 116.8^k(36') + 73.8^k(18.5')\right] = 4853.3$ FT-K

→ $M_R > M_o$ ∴ NO DESIGN MOMENT

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COMMENTS
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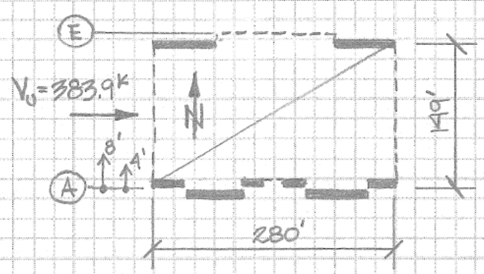
DIAPHRAGM CHECK

CASE #1 →

- E-W LOADS
- ASSUME ONLY WALLS NEAR GRID LINE(S) (A) & (E)
- WORST CASE DIAPHRAGM SHEAR @ 3RD FLOOR, FROM LAT. ANALYSIS:

@ 3RD $\quad V_U = 383.9^K$



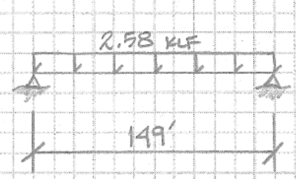
$\omega_U = \frac{383.9^K}{149'} = 2.58\ ^K/_{FT}$

$M_U = \frac{\omega L^2}{8} = \frac{2.58(149)^2}{8} = 7160$ FT-K

$A_{S,REQ'D} = \frac{7160'^K (12''/')}{0.9(60)(275' \times 12''/')} = 0.48\ IN^2$

∗ APPROX. DIAPHRAGM DIMENSIONS

SHEAR $V_U = \frac{\omega L}{2} = \frac{2.58(149)}{2} = 192^K$



@ 4'-0": $V_U = \omega\left(\frac{l}{2} - x\right)$
$= 2.58\left(\frac{149'}{2} - 4'\right) = 182^K$

@ 8'-0": $V_U = 2.58\left(\frac{149}{2} - 8\right) = 172^K$

AT JOINTS

@ 4'-0": $V_U = 182^K$ → $v_U = \frac{182^K}{280'} = 0.65\ ^K/_{FT}$

@ 8'-0": $V_U = 172^K$ → $v_U = \frac{172^K}{280'} = 0.61\ ^K/_{FT}$

AT WALL

BASED ON $V_U = 192^K$ OVER 130' (MIN) @ S.W.'S

$v_U = \frac{192}{130} = 1.48\ ^K/_{FT}$

(ACI 318-05 ~ CH. 21)

$V_n = A_{cv}\left(2\sqrt{f'_c} + \rho_n f_y\right) \leq A_{cv}\rho_n f_y = (2'' \times 12'')(0.001167)(65)$

$= (2'' \times 12'')\left(2\sqrt{5000} + 0.001167(65)\right) \qquad = 2.6\ ^K/_{FT}$ (CONTROLS)

$= 6\ ^K/_{FT}$

$\underline{V_n = 2.6\ ^K/_{FT}}$ $\left(> 1.5\ ^K/_{FT}\right)$ ∴ OK ✓

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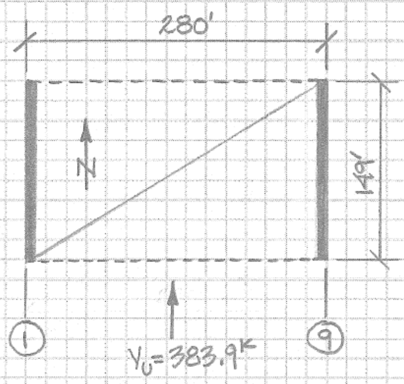
DIAPHRAGM CHECK

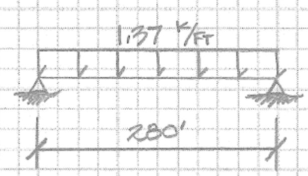
CASE #2 →

- N-S LOADS
- ASSUME ONLY WALLS ALONG GRID LINE(S) 1 & 9
- WORST CASE @ 3RD FLOOR (SEE PREV. CALC.):

$V_U = 383.9^K$



* APPROX. DIAPHRAGM DIMENSIONS



$\omega_U = 383.9/280 = 1.37$ KLF

$M_U = \frac{\omega L^2}{8} = 13436.5$ FT-K

$V_U = \frac{\omega L}{2} = 192^K$

$A_{S,REQ'D.} = \frac{13436.5(12")}{0.9(60)(145' \cdot 12")} = 1.72 \text{ IN}^2$

ALONG WALL (CONNECTION)

$V_U = 192^K$ OVER 130' WALL

$V_U = 192^K / 130' = 1.48$ K/FT

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TECH III

DRIFT / STORY DRIFT

(REF. ASCE 7-05, CH.12)

• (PER TABLE 12.12-1)

ALLOWABLE STORY DRIFT (Δ_a) = $0.020\, h_{sx}$

@ ROOF = 0.020(40' × 12"/') = 9.6" (* ALSO, TOTAL ALLOW. DRIFT)

@ 3RD = 0.020(28' × 12"/') = 6.7"

@ 2ND = 0.020(16' × 12"/') = 3.8"

• (PER 12.8.6)

$$\delta_x = \frac{C_d\,\delta_{xe}}{I}$$

WHERE: $C_d = 3$ (TABLE 12.2-1: A.6. ORDINARY PRECAST SHEAR WALLS)

$I = 1.0$ (TABLE 11.5-1: OCCUP. CAT II)

δ_{xe} = DEFLECTIONS DETERMINED VIA ELASTIC ANALYSIS

* THIS FACTOR VARIES PER FLOOR AND WAS ESTIMATED USING WINBEAM TO MODEL THE SHEARWALL AS A CANTILEVER BEAM. (SEE FOLLOWING PAGES)

STORY DRIFT →

@ 2ND: $\delta_2 = 3.0(0.004869") = 0.015"$

3RD: $\delta_3 = 3.0(0.01258") = 0.038"$

ROOF: $\delta_R = 3.0(0.021472") = 0.064"$ ← TOTAL DRIFT

WinBeam

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Description:

  Tech III Report
  Shear Wall SWD

Units: English

Properties - X = feet, E = ksi, I = in^4
  X = 0; E = 4300; I = 87528384;
  X = 40; E = 4300; I = 87528384;

Moment Releases - X = feet

Supports - X = feet, Displacement = inches, Rotation = radians
  X = 0; Disp = 0; Rotation = 0;

Springs - X = feet, VSpring = kip/inch, RSpring = kip in/rad

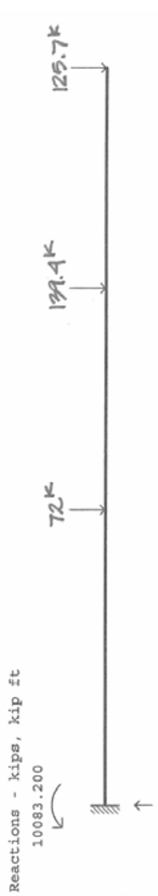
Point Loads - X = feet, PLoad = kips, Moment = kip ft
  X = 16; PLoad = -72;
  X = 28; PLoad = -139.4;
  X = 40; PLoad = -125.7;

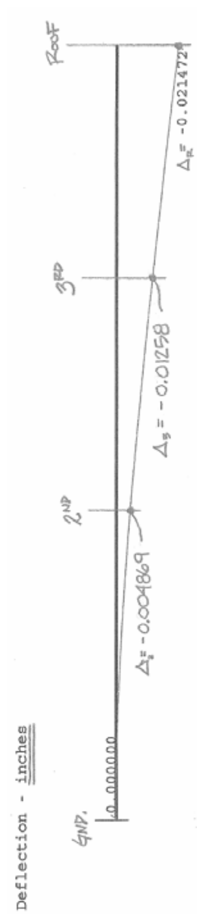
Uniform Loads - XStart & XEnd = feet, UStart & UEnd = kip/ft
```

$$I = \frac{bh^3}{12} = \frac{(12")(37' \times 12"/')^3}{12}$$

$$I = 87{,}528{,}384 \text{ IN}^4$$

FROM EXCEL SPREADSHEET





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TORSION

• (PER 12.8.4.2)

MIN. MOMENT ARM → $e + 5\%(\text{LENGTH NORMAL TO FORCE}) = r$

N-S: V_u, 195′ × 280′

$5\%(280') = 14'$

@ ROOF : $e = 0$ → $r_R = 14'$
@ 3RD : $e = 0$ → $r_3 = 14'$
@ 2ND : $e = 1.6'$ → $r_2 = 15.6'$

E-W: V_u, 195′ × 280′

$5\%(195') = 0.7'$

@ ROOF : $e = 77.2' - 73.5' = 3.7'$ $(+0.7') = 4.4'$
@ 3RD : $e = 86.6 - 75.5 = 11.1'$ $= 11.8'$
@ 2ND : $e = 73.5 - 69.7 = 3.8'$ $= 4.5'$

- APPLY LATERAL LOADS

TORSIONAL MOMENT PER FLOOR =

		N-S FORCE	E-W FORCE
@ ROOF :	$344.3^k(e)$	4820 FT-K	1515 FT-K
@ 3RD :	$383.9^k(e)$	5375 FT-K	4530 FT-K
@ 2ND :	$197.3^k(e)$	3078 FT-K	888 FT-K

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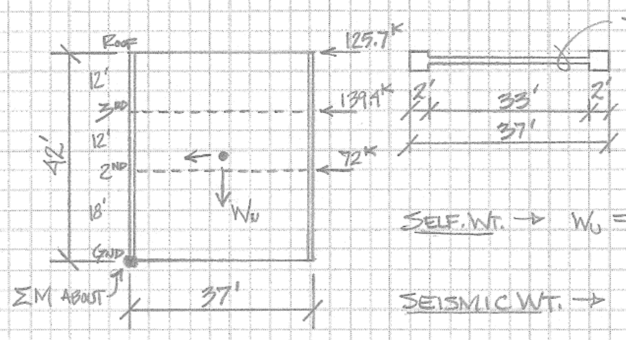
COMMENTS
SEISMIC LOAD DISTRIBUTION (SPOT CHECK)

SPOT CHECK OF SHEARWALL SWD, MODELED AS ORDINARY REINFORCED, CAST-IN-PLACE CONCRETE.

ASSUME: $f_y = 60$ KSI
$f'_c = 6$ KSI



A = 41 FT² (PREV. CALC)

SELF WT. → $W_u = (41\text{ FT}^2 \times 42')(150\text{ PCF})/1000 = 258.3^K$

$W_u = 258.3^K$

SEISMIC WT. → $0.0607(258.3) = 15.7^K$

OVERTURNING MOMENT → $M_o = 125.7^K(42') + 139.4^K(30') + 72^K(18') + 15.7^K(21')$

$M_o = 11087.1$ FT-K

RESISTING MOMENT → $M_R = 0.86\left[371.4^K(1') + 201.8^K(36') + 258.3^K(18.5')\right]$

$M_R = 10676.7$ FT-K

DESIGN MOMENT → $M_u = M_o - M_R = 410.4$ FT-K (AT BASE)

REINFORCING → $V_u = 352.1^K$ (AT BASE) (PREV. CALC.)

✓ (2) CURTAINS REQ'D?: $2A_{cv}\sqrt{f'_c} = 2(12'' \times 35'(12''/')) \sqrt{6000}/1000 = 780.8^K$

(ACI $V_u = 352.1^K < 780.8^K$ ∴ ONLY 1 CURTAIN REQ'D.

REQ'D. ρ_L & ρ_T (ASSUME BOTH ≥ 0.0025)

$A_{cv} = 144$ IN²/FT $\quad A_{s_L} = 0.0025(144)$

$A_{s_L} = 0.36$ IN²/FT

TRY USING #5's @ 10" O.C. MAX:

$\frac{0.31\text{ IN}^2}{10''} = \frac{x}{12''} \rightarrow 0.37\text{ IN}^2/\text{FT} > 0.36$ ∴ OK ✓

USE #5's @ 10" O.C. MAX. (BOTH DIRECTIONS)
(1 CURTAIN ONLY)

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✓ SHEAR CAPACITY → $V_n = A_{cv}(\alpha_c \sqrt{f'_c} + \rho_t f_y)$

$h_w / l_w = \frac{42'}{35'} = 1.2 \rightsquigarrow \alpha_c = 3.0$

$\rho_t = \frac{0.31}{(10'')(12'')} = 0.002583$

$A_{cv} = 12'' \times 35'(12) = 5040 \text{ IN}^2$

$V_n = 5040 \text{ IN}^2 \left[3.0\sqrt{6000} + (2.583 \times 10^{-3})(60000)\right] / 1000$

$V_n = 1737.5^k$

$\overset{0.6}{\phi} V_n = 1042.5^k > 352.1^k \therefore$ OK ✓

For Center of Mass

2nd Floor

		X	Y	X	Y
Floor Deck:	3806.5 k	140	70.18	532910	267140
IT's line 2&8	113.2	140	73.75	15848	8349
IT's line 3&7	84.95	140	73.43	11893	6238
IT's line 4	41.66	105	61.92	4374	2580
IT's line 5	22	140	74	3046	1610
IT's line 6	38.59	175	64.31	6753	2482
(4) Stair Towers:	147.7	140	77.5	20678	11447
columns	162	140	78.06	22680	12646
monumental stair	19.3	135.83	49.42	2622	954
Rect. Beams	36.75	140	37.68	5145	1385
9" Walls	202.5	146.29	81.83	29624	16571
9.5" Walls	32.81	140	15	4593	492
12.5" Walls	60	140	8	8400	480
mezzanine	153.5	189.5	127	29088	19495
	4921			697655	351866
				141.76	71.50

3rd Floor

		X	Y	X	Y
Floor Deck:	4572.6 k	140	78.23	640164	357714
IT's line 2&8	113.2	140	73.75	15848	8349
IT's line 3&7	84.95	140	73.43	11893	6238
IT's line 4	56.76	105	78.92	5960	4479
IT's line 5	67.5	140	77.17	9450	5209
IT's line 6	53.69	175	67.38	9396	3618
Low Roof (DT)	594.4	140	172.67	83216	102635
9" Wall	148.5	148.7	83.81	22082	12446
9.5" Wall	57.8	140	15	8092	867
12" Wall	48	140	8	6720	384
(4) Stair towers	73.5	140	77.5	10290	5696
columns	158.4	140	93.1	22176	14747
	6029			845287	522382
				140.20	86.64

Roof

		X	Y	X	Y
Roof Deck:	3023.2 k	140	78.23	423248	236505
IT's line 2&8	113.2	140	73.75	15848	8349
IT's line 3&7	84.95	140	73.43	11893	6238
IT's line 4	56.76	105	78.92	5960	4479
IT's line 5	49.20	140	77.17	6888	3797
IT's line 6	53.69	175	67.38	9396	3618
9" Wall	74.3	148.7	83.81	11048	6227
9.5" Wall	28.9	140	15	4046	434
12.5" Wall	24	140	8	3360	192
columns	75.6	140	88.33	10584	6678
	3584			502271	276516
				140.15	77.16

END OF REPORT