FINAL REPORT

1000 CONTINENTAL SQUARE

KING OF PRUSSIA, PENNSYLVANIA

Carter Davis Hayes

Structural Option

May 2, 2008

Advisor: Dr. Hanagan

1000 CONTINENTAL SQUARE

KING OF PRUSSIA PENNSYLVANIA

CARTER DAVIS HAYES
STRUCTURAL OPTION
DR. HANAGAN

PROJECT INFORMATION

OWNER : BPG PROPERTIES, LTD.
ARCHITECT : SPG3 ARCHITECTS
STRUCTURAL ENGINEER : THE HARMAN GROUP
MEP ENGINEER : GIOVANETTI SHULMAN ASSOCIATES
LIGHTING DESIGNER : THE LIGHTING PRACTICE
GEOTECHNICAL ENGINEER : PENNONI ASSOCIATES INC.

ARCHITECTURE

- High-end office space
- Featuring large, open floor plans with uninterrupted forty-foot bays along each side of the building.
- This building is located along the prominent intersection of Pennsylvania Routes 202, 76 and 422.
- The interior is finished with top of the line materials and modern fixtures, and the entry features a two level lobby with a dramatic cantilevered walkway and staircase.

MECHANICAL

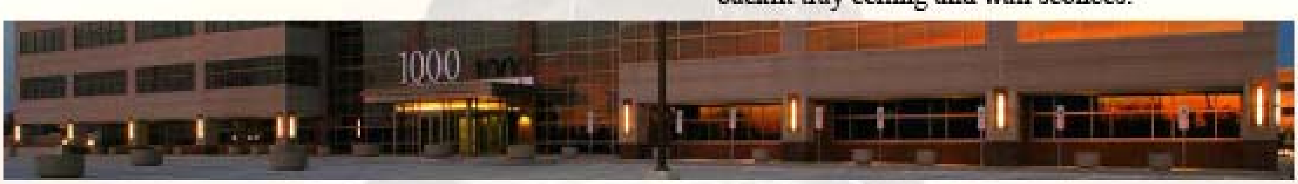
- Two 60,000 CFM air handling units located on the roof provide the air to VAV boxes, two per floor .

STRUCTURAL

- Composite steel structural frame
- 6" long ¾" diameter headed studs on a 3" 20 gage composite metal deck with a 6 ¼" slab.
- Typical framing is standard w shapes, W12's for columns, W18's for beams, and W24's for girders.
- There are two standard bay sizes, 30' x 40' on the exterior and 30' x 35' in the interior.
- In the east - west direction run two moment frames and north - south are two identical brace frames.

ELECTRICAL & LIGHTING

- Power supplied is a 277/480 V three phase four wire system.
- Stepped down by a 150 kVA transformer.
- All floors have four 277/480 V panelboards, four 208/120 V panelboards.
- The building's current design only provides emergency lighting and additional lighting in common areas like lobbies and stairwells.
- The design for the lobby incorporates florescent pendent fixtures and wall washes, while that elevator lobby uses recessed can lights accented by a backlit tray ceiling and wall sconces.



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EXECUTIVE SUMMARY

This paper is the conclusions of a year's worth of analysis and computations based on the design of the office building at 1000 Continental Square in King of Prussia, PA. The purpose of the calculations presented in this report is to explore the redesign of the structural system in concrete. The building is a high-end office space, featuring large, open floor plans with uninterrupted forty-foot bays along each side of the building. This building is located along the prominent intersection of Pennsylvania Routes 202, 76 and 422; and is in close proximity to a Pennsylvania Turnpike interchange and the King of Prussia Mall. The building has a partially sub-grade ground floor mainly for mechanical systems and storage with five floors of leasable space above that. The existing structural frame is steel with composite concrete slabs, and lateral loads are resisted by two moment frames along the long axis of the building and two eccentrically braced frames along the short axis. The concrete redesign incorporates a pan-joist slab supported on wide beams which also act as components of a moment frame in the long direction of the building. The short axis of the building is laterally supported by two reinforced concrete shear walls with take the place of the two original braced frames. The redesign also includes new concrete column and footing designs.

The results of the redesign show that the concrete system is a feasible alternative to the existing steel system. A quick RS Means estimates shows that the concrete system is only $2.50 more per square foot. This is not so bad considering thinner slab depths, smaller deflections, and more rigid structure. The concrete should also have shorter lead times, but a longer overall construction time. Under the conditions at 1000 Continental Square, there is not decisive reason to switch to the concrete system, however if the project had limitations on vibration, overall height, or serviceability the concrete system would be favored.

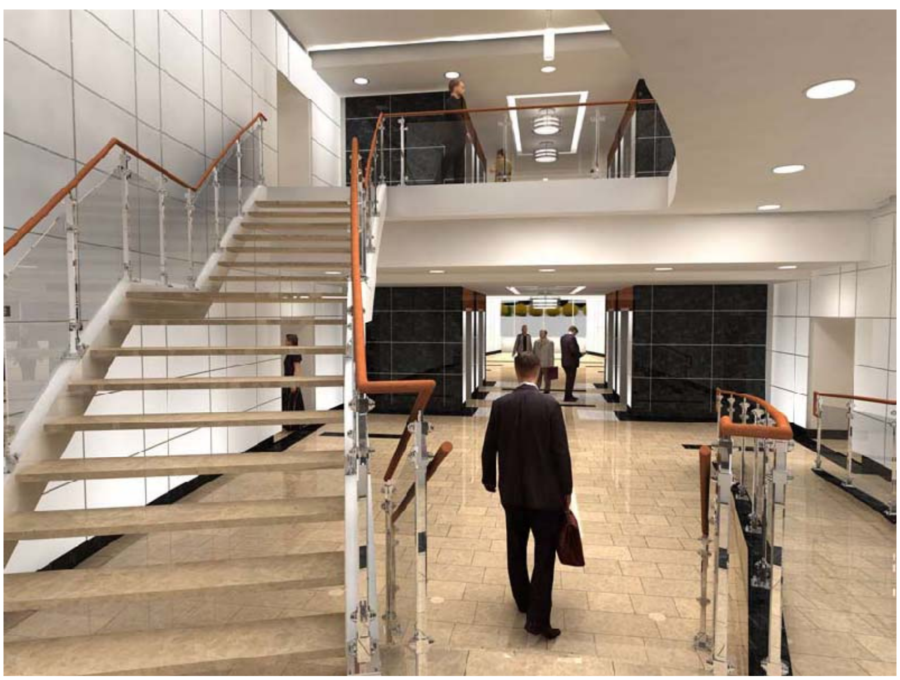
There are two breadth studies in architecture and lighting design also included in this thesis. The architectural study resulted in an amusing free-form floor plan with innovative design features. The plan includes serpentine walls which echo features of the building façade, a concentric elliptical reception area inspired by the building's grand lobby, and new modular cubical system that is rearrangeable and expandable to adapt to changing office needs.

The second breadth study was in using daylighting to reduce the number of kilowatt hours expended by fixtures near the building perimeter. The breadth started with the layout of general lighting throughout the cubicle spaces. Then the effects of daylighting were checked under different weather conditions and times of year.

1000 Continental Square was designed to adhere to the 2004 Pennsylvania Uniform Construction Code which references IBC 2003 and ASCE 7-02. This study used IBC 2006 and ASCE 7-05, along with using some estimations and simplifications of floor areas and loadings, which could account for some discrepancies in my calculations when compared to those of the design engineer. Further findings of this report are located in the Conclusions section.

I. INTRODUCTION

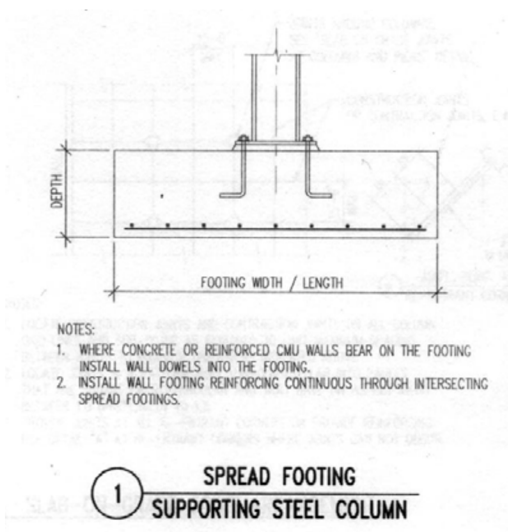
1000 Continental Square is a high-end office building, featuring large, open floor plans with uninterrupted forty-foot bays along each side of the structure. These 40' bays are designed for 100 pound per square foot live loads allowing tenants almost limitless possibilities as far as building use is concerned. This building is located along the prominent intersection of Pennsylvania Routes 202, 76, and 422; and is in close proximity to a Pennsylvania Turnpike interchange and the King of Prussia Mall. The interior is finished with top of the line materials and modern fixtures intended to attract to the wealthier clientele of the region which are already there as a result of other amenities like the mall. The entry features a two level lobby with a dramatic cantilevered walkway and staircase. The building envelope is mainly architectural precast panels highlighted with brick accents. Strip windows are set into the precast on three sides of the building; the fourth side is a giant, convex, reflective glass curtain wall which dominates the facade along the highway. Another glass curtain wall prominently marks the building's main entrance along with six foot tall building numbers above the doors.

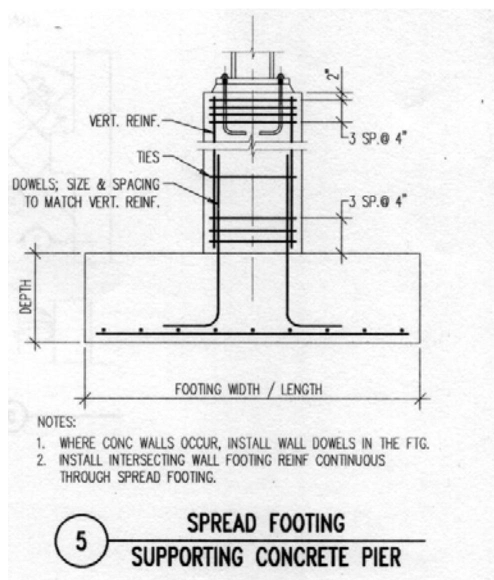


II. EXISTING STRUCTURAL SYSTEMS

FOUNDATIONS

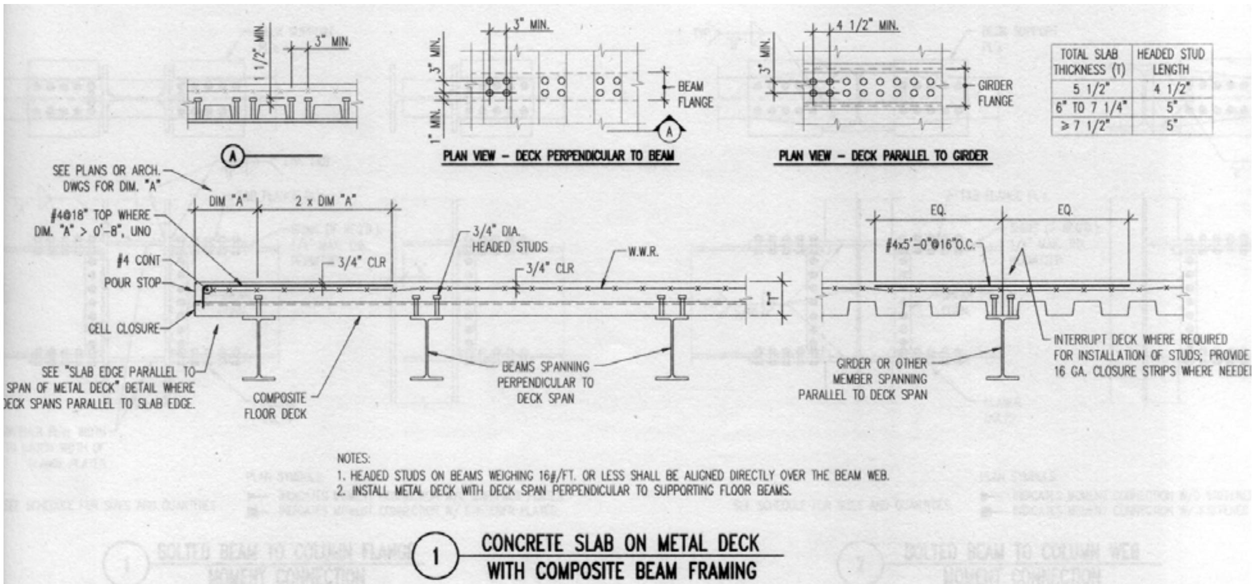
The foundations for 1000 Continental Square are a series of spread footings with continuous wall footings under the retaining walls located on the ground floor. The soils under the footings were found to withstand 4000 psf in most locations, according to the geotechnical report furnished by Pennoni Associates, Inc. on 24 of February 2004. Suitable bearing pressures were attained by deep dynamic compaction or partial soil exchange. Footing dimensions range from 4' x 4' x 1.5' to 20' x 20' x 4'; however, typical footings are approximately 14' x 14' x 3'. Special 55' x 18' x 3.5' spread footings are used under the braced frames. The tops of most footings are located 1.5' below grade, and minimum bearing depth is 3'. Columns either bear directly on footings, or in some atypical situations, concrete piers are placed on top of the footings and columns bear on those. Footings have bottom reinforcement ranging from (7) #4's to (16) #11's with typical reinforcement being approximately (12) #9's. The continuous wall footings are integrated into the spread footings they intersect, and their reinforcement is continuous throughout. Concrete in all footings has a minimum compressive strength, f'c = 3000 psi with a unit weight of 145 pcf. There is a 4" thick slab on grade which acts as the floor system for the ground floor and utilizes 4000 psi compressive strength concrete.





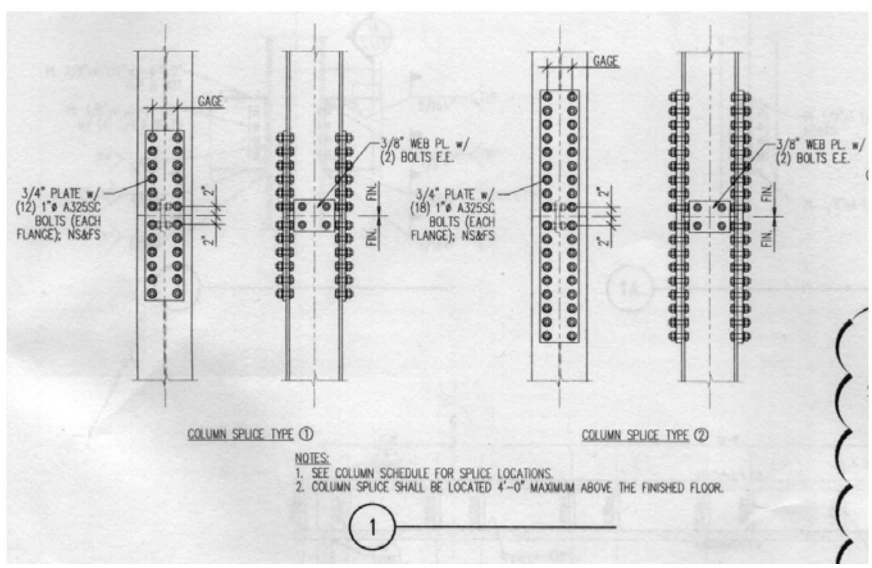
FLOOR FRAMING

All the floor framing above grade in the 1000 Continental Square project are 6¼" composite slabs. They consist of 3¼" lightweight concrete over 3" deep 20 gauge galvanized composite floor deck. The slab is reinforced by one layer of 6 x 6 – W1.4 x W1.4 WWR, and has a weight of 115 pcf and a compressive strength of 3500 psi. This is supported by W 18 x 35's spanning 40' bays which tie into an assortment of girders spanning 30'; W 24 x 55's being the most typical. Composite action is achieved through 6" long, ¾" diameter headed studs, approximately 34, evenly spaced per beam. The W 18's feature a typical camber of 1.5". Variations in design occur at architectural features, the elevator shafts, and intersections with the moment frames; elsewhere, the system is nearly identical on all floors.



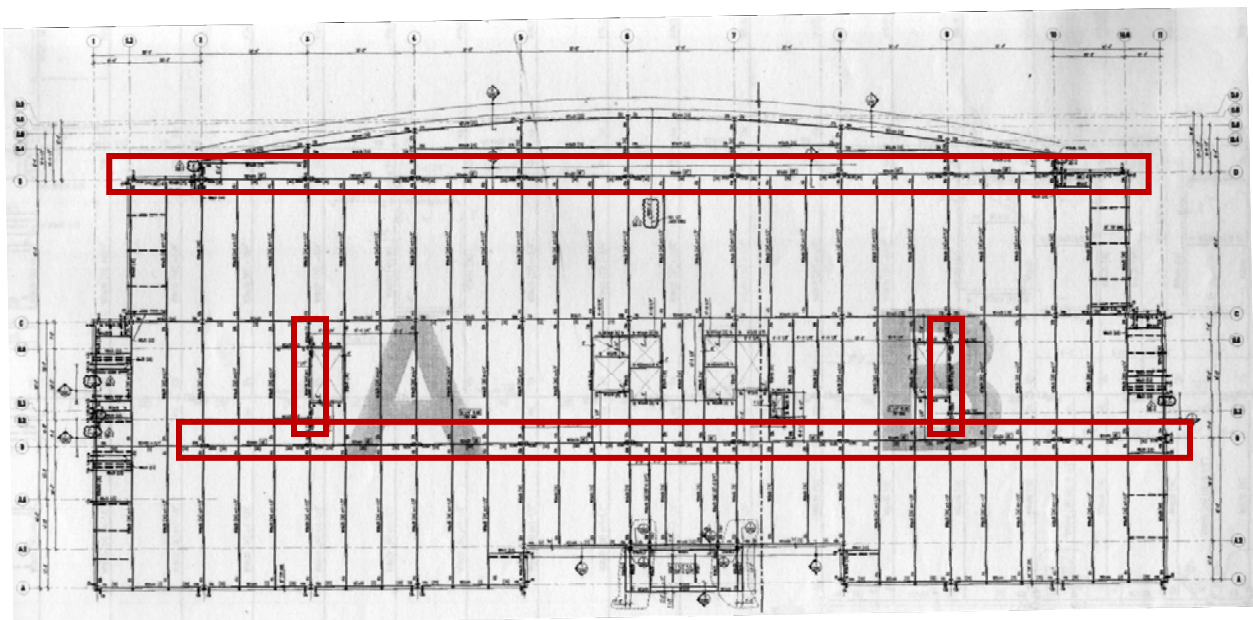
COLUMNS

The column grid for the building is laid out rectilinearly using three spans: 40', 35', 40', in the N-*S* direction and (10) 30' spans in the E-W, thereby creating large, uninterrupted, regular bays to simplify leasing. Column sizes vary between W 12 X 230's on the first floor of the moment frames, to W 12 X 40's for gravity columns on the top floors. Splice levels are located a maximum of 4ft above the second and fourth floors. Typical columns are W 12 x 152's on the bottom floors, W 12 x 96's on the middle floors, and W 12 x 40' on the top levels. Typical columns are fixed to foundations with four ¾" diameter anchor rods with 1' embed depths and 4" hooks.



LATERAL LOAD RESISTING SYSTEMS

1000 Continental Square is reinforced against lateral loads by different systems along its long axis (E-W) and short axis (N-S). In the E-W direction, two moment frames fit into the existing grid along column lines B and D, and act over the full height of the building, and effectively, its full length. In the N-S direction, two full-height eccentrically braced frames fit off-grid, between lines B and C, and along column lines 3 and 9, to provide support for the short axis. These systems act to counter both wind and seismic forces, however, wind loads were found to control the design in this situation. There are two additional types of one story braced frames used in the building, mainly to support architectural elements, which are not analyzed in this report.



III. PROJECT STATEMENT

1000 Continental Square uses a composite steel structural system. This system was found to be the lightest weight and relatively easy to construct making it one of the best options. However, it was found to have some rather serious drawbacks as well. Problems like long lead times and the need for spray on fireproofing drag out the construction process and add cost. Additionally, through the first three technical reports it appeared that many of the members were oversized when checked for strength in order to deal with serviceability issues. These issues arise from the large bay sizes and relatively light structural system. This inefficiency could be minimized with an alternate framing system. The current steel system also uses two moment frames to resist lateral loads along the long axis of the building. This moment frame adds a great deal weight and cost to the building. An alternate system could more efficiently handle these lateral loads.

IV. CODES AND MATERIALS

This section outlines the codes referenced by both the original design engineer in the existing section and the ones used to check the existing design and do the redesign in the proposed section. The materials section lists specifications of all materials used in the original structural design and those assumed to be used in the proposed redesign.

CODES (EXISTING)

Building Code:	2004 Pennsylvania Uniform Construction Code
Building Subcode:	International Building Code (IBC) 2003
Minimum Design Loads:	American Society of Civil Engineers (ASCE), 7-02
Reinforced Concrete:	American Concrete Institute (ACI), 318-02
	Concrete Reinforcing Steel Institute, Manual of Standard Practice, 27th Edition, March 2001
Precast Concrete:	Precast/Prestressed Concrete Institute (PCI), Design Handbook 5th Edition
Steel Construction:	American Institute of Steel Construction (AISC), Manual of Steel Construction, LRFD, 3rd Edition, 2001
Steel Decking:	Steel Deck Institute, Design Manual

CODES (PROPOSED)

Building Code:	International Building Code (IBC) 2006
Minimum Design Loads:	American Society of Civil Engineers (ASCE), 7-05
Reinforced Concrete:	American Concrete Institute (ACI), 318-05

MATERIALS (EXISTING)

Cast in place concrete (normal weight 145 pcf)

Footings		3,000 psi
Topping slabs		3,000 psi
Lightweight slabs on metal deck (115 pcf)		3,500 psi
Normal weight slabs on metal deck		3,500 psi
Slabs on grade		4,000 psi
Walls and piers		4,000 psi
Cast in Place on precast		5,000 psi
Pourable fill		1,000 psi

Precast Concrete (normal weight 145 pcf)

Structural precast		5,000 psi

Reinforcing Steel

All types U.N.O.	ASTM A615	60,000 psi

Structural Steel

W Shapes	ASTM A992	50,000 psi
Channels, angles, and plates	ASTM A36	36,000 psi
Round pipes	ASTM A53 E or S	35,000 psi
Square and Rectangular HSS's	ASTM A500	46,000psi

MATERIALS (PROPOSED)

Cast in place concrete (normal weight 145 pcf)

Footings		3,000 psi
Columns (Floors G & 1)		5,500 psi
Columns (Floors 2 – 6)		4,000 psi
Pan-Joist Slabs and Beams		4,000 psi
Slabs on grade		4,000 psi

Reinforcing Steel

All types U.N.O.	ASTM A615	60,000 psi

V. DESIGN LOADS

LIVE LOADS

All floors	100 psf	Due to the open floor plan, all areas are assumed to be lobby or corridor space
Roof	20 psf	Standard flat roof loading
Snow load	21 psf	From ASCE 7-05 (see below)

$p_f=0.7C_eC_tIp_g$		Equation 7-1
Terrain Category	B	Section 6.5.6.2
Exposure	Partially	Table 7-2 Footnote
C_e	1.0	Table 7-2
C_t	1.0	Table 7-3
I	1.0	Table 7-4
p_g	30psf	Figure 7-1

DEAD LOADS

Floor self weight		
Steel	50 psf	From steel deck manufacturer's design tables
Concrete	113psf	Based on cubic feet of concrete per square foot
Roof self weight		
Steel	5 psf	From steel deck manufacturer's design tables
Concrete	113 psf	Based on cubic feet of concrete per square foot
Arch. Precast Panels	50 psf	Material property
Superimposed DL	22 psf	(see below)

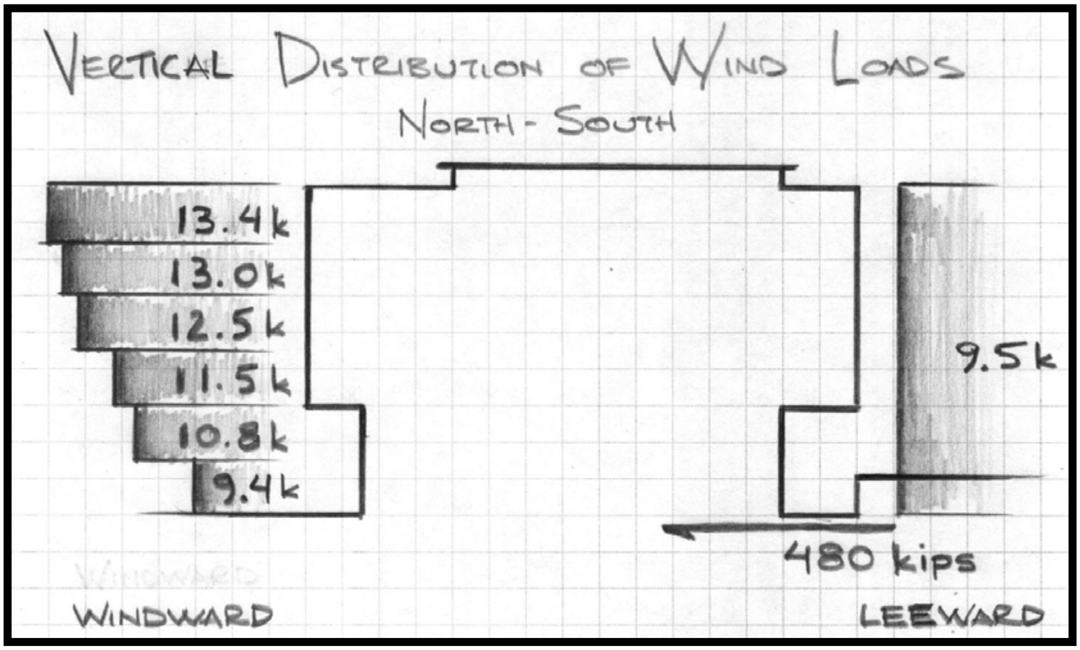
MEP	7 psf
Ceiling Finishes	3 psf
Floor Finishes	12 psf

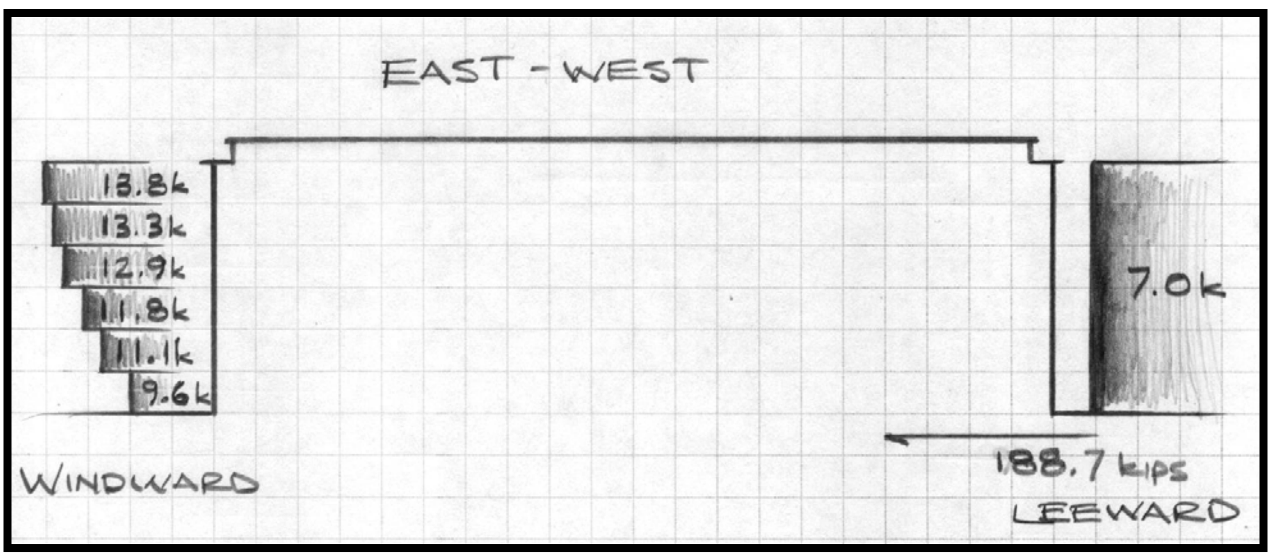
WIND LOADS

Basic Wind Speed	90 mph
Exposure Category	B
Enclosure Category	Enclosed
Wind Directionality Factor (Kd)	0.85
Importance Factor (I)	1.0
Topographic Factor (Kzt)	1.0
Gust Effect Factor (G)	0.828 (E-W) or 0.798 (N-S)
Internal Pressure Coefficient	± 0.18

VERTICAL DISTRIBUTION OF WIND LOADS			
E-W DIRECTION			
Height (ft)	Windward Pressure (psf)	Leeward Pressure (psf)	Total (psf)
13	9.61	7.03	16.64
26	11.12	7.03	18.15
39	11.82	7.03	18.85
52	12.87	7.03	19.90
65	13.34	7.03	20.37
78	13.81	7.03	20.84
N-S DIRECTION			
Height (ft)	Windward Pressure (psf)	Leeward Pressure (psf)	Total (psf)
13	9.36	9.50	18.86
26	10.83	9.50	20.33
39	11.50	9.50	21.00
52	12.51	9.50	22.01
65	12.96	9.50	22.46
78	13.42	9.50	22.92

WIND LOAD SUMMARY		
East - West Direction	Base Shear: 188.68 kips	Overturning Moment: 7,962.16 kip-ft
North - South Direction	Base Shear: 479.33 kips	Overturning Moment: 8,805.83 kip-ft


VERTICAL DISTRIBUTION OF WIND LOADS
NORTH - SOUTH
13.4k
13.0k
12.5k
11.5k
10.8k
9.4k
9.5k
480 kips
WINDWARD
LEEWARD


EAST - WEST
13.8k
13.3k
12.9k
11.8k
11.1k
9.6k
7.0k
188.7 kips
WINDWARD
LEEWARD

SEISMIC LOADS (EXISTING)

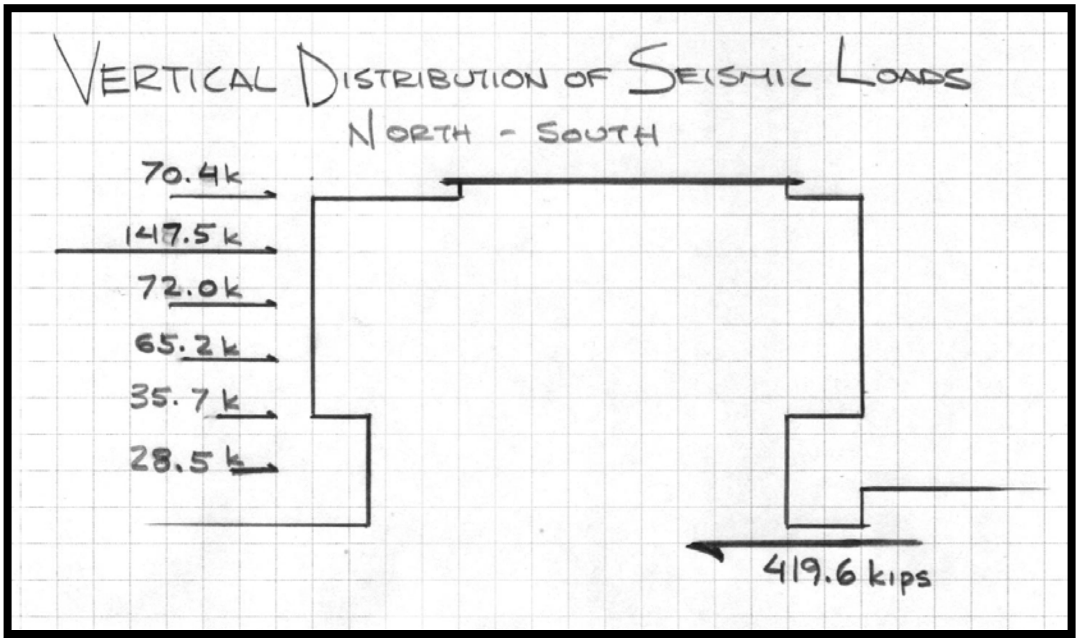
Item	Design Value E-W	Design Value N-S	Code Basis (ASCE 7-05)
Hazard Exposure Group	I		Table 1-1
Performance Catagory	B		Table 11.6-1,2
Importance Factor (I)	1.00		Table 11.5-1
Spectral Acceleration for Short Periods (S_S)	0.278		Figure 22-1
Spectral Acceleration for One Second Periods (S_1)	0.06		Figure 22-2
Damped Design Spec. Resp. Acc. at Short Periods (S_{DS})	0.2224		Section 11.4.4
Damped Design Spec. Resp. Acc. at One Second Periods (S_{D1})	0.068		Section 11.4.4
Seismic Response Coefficient (C_S)	0.0635	0.0278	Section 12.8.1.1
Soil Site Class	C		Section 20.3.3
Basic Structural System	Comp. Steel		
Seismic Resisting System	OSMF	CEBF	
Response Modification Factor (R)	3.5	8	Table 12.2-1
Deflection Modification Factor (C_d)	3	4	Table 12.2-1
Analysis Procedure Utilized	Equiv. Lat. Force		
Design Base Shear	420 kips		

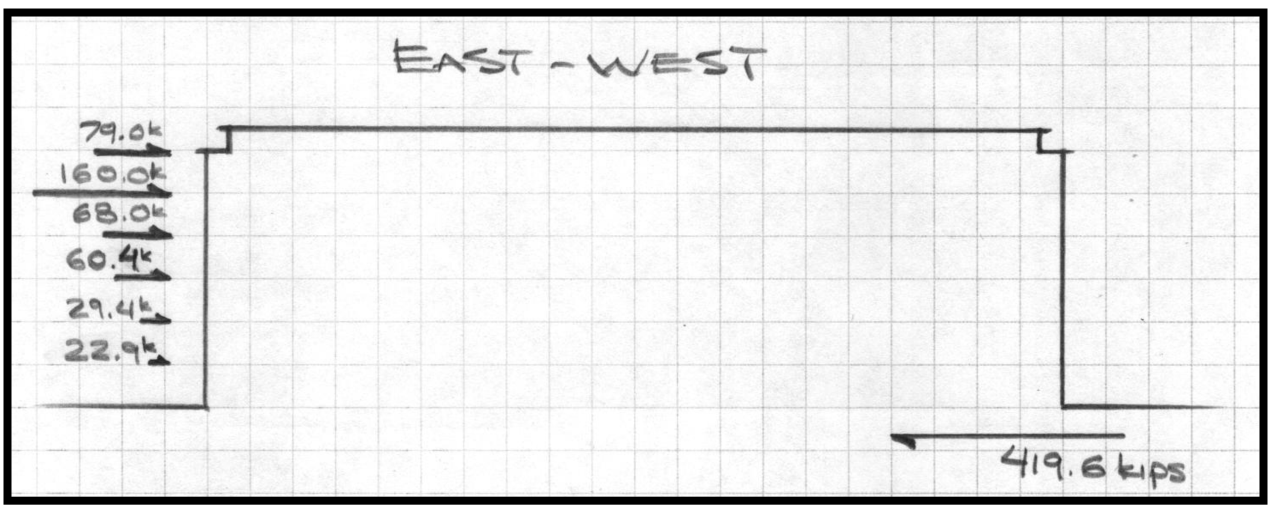
VERTICAL DISTRIBUTION OF SEISMIC FORCES

Height (ft)	E-W DIRECTION Story Shear (kips)	N-S DIRECTION Story Shear (kips)
0	419.60	419.60
13	396.68	390.68
26	367.24	355.00
39	306.88	289.85
52	238.90	217.87
65	79.01	70.36

SEISMIC LOAD SUMMARY

Base Shear: 419.60 kips	Overturning Moment: 42,209.27 kip-ft


VERTICAL DISTRIBUTION OF SEISMIC LOADS
NORTH - SOUTH
70.4k
147.5k
72.0k
65.2k
35.7k
28.5k
419.6 kips


EAST - WEST
79.0k
160.0k
68.0k
60.4k
29.4k
22.9k
419.6 kips

SEISMIC LOADS (PROPOSED)

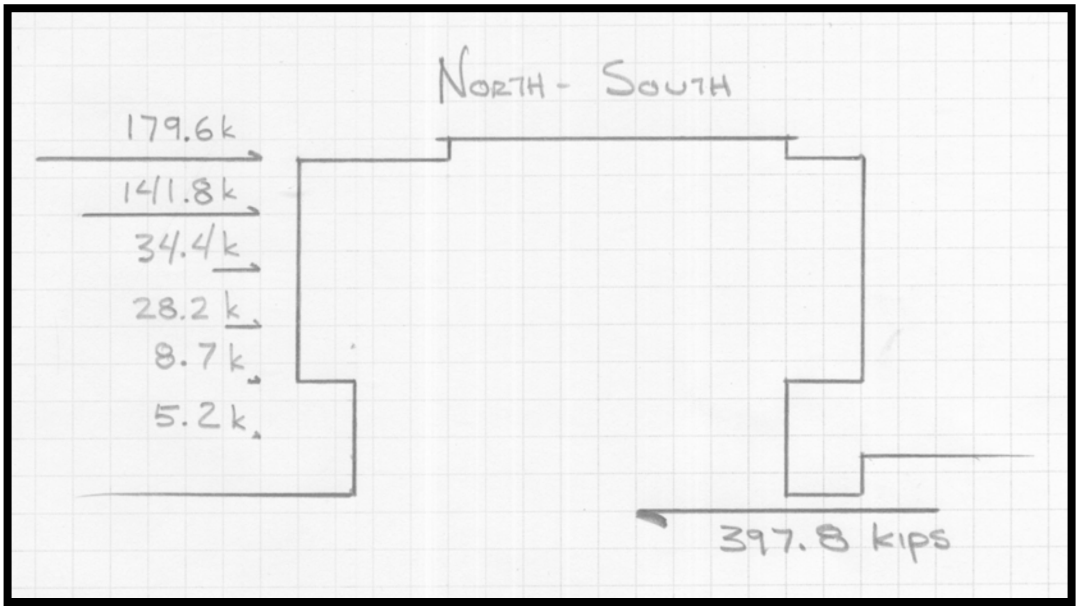
Item	Design Value		Code Basis (ASCE 7-05)
	E-W	N-S	
Hazard Exposure Group	I		Table 1-1
Performance Category	B		Table 11.6-1,2
Importance Factor (I)	1.00		Table 11.5-1
Spectral Acceleration for Short Periods (S_S)	0.278		Figure 22-1
Spectral Acceleration for One Second Periods (S_1)	0.06		Figure 22-2
Damped Design Spec. Resp. Acc. at Short Periods (S_{DS})	0.2224		Section 11.4.4
Damped Design Spec. Resp. Acc. at One Second Periods (S_{D1})	0.068		Section 11.4.4
Seismic Response Coefficient (C_S)	0.0635	0.0278	Section 12.8.1.1
Soil Site Class	C		Section 20.3.3
Basic Structural System	Rein. Concrete		
Seismic Resisting System	SCMF	SCSW	
Response Modification Factor (R)	6	8	Table 12.2-1
Deflection Modification Factor (C_d)	5	5.5	Table 12.2-1
Analysis Procedure Utilized	Equiv. Lat. Force		
Design Base Shear	398 kips		

VERTICAL DISTRIBUTION OF SEISMIC FORCES

Height (ft)	E-W DIRECTION	N-S DIRECTION
	Story Shear (kips)	
0	397.80	397.80
13	392.62	392.62
26	383.94	383.94
39	355.72	355.72
52	321.36	321.36
65	179.60	179.60

SEISMIC LOAD SUMMARY

Base Shear: 397.80 kips	Overturning Moment: 55666.30 kip-ft

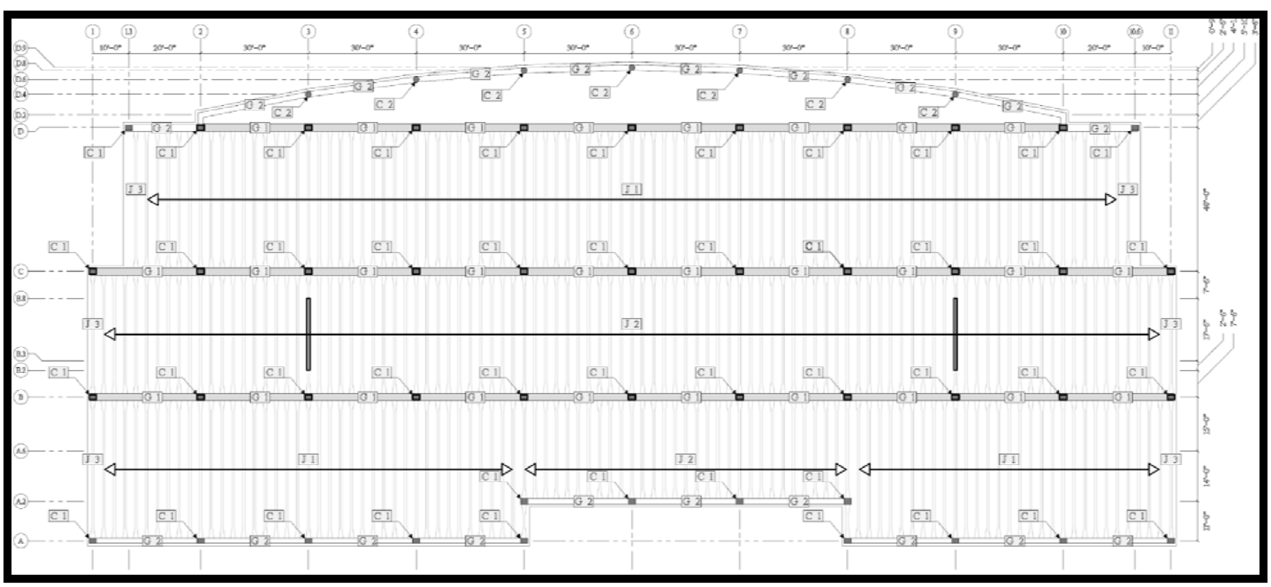

NORTH - SOUTH
179.6k
141.8k
34.4k
28.2 k
8.7 k
5.2 k
397.8 kips


EAST-WEST
179.6k
141.8k
34.4k
28.2k
8.7k
5.2k
397.8 kips

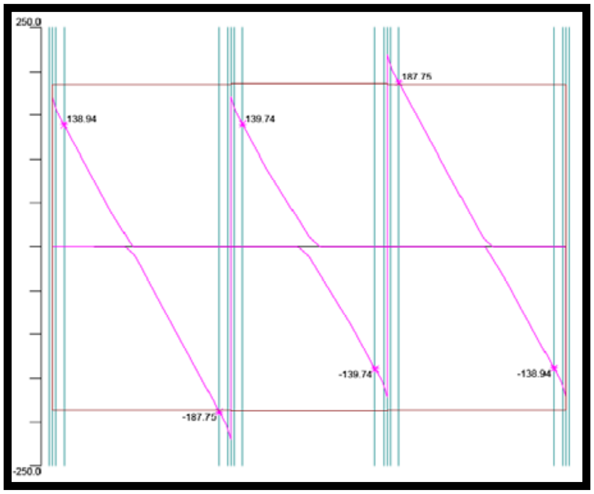
VI. STRUCTURAL REDESIGN

FLOOR SYSTEM

The design of the substitute concrete floor system takes over where technical report three had ended. The conclusion that had been reached that a concrete floor would be thinner and solve many of the problems the lighter composite steel system such as serviceability and fireproofing; however, all the systems explored in that technical report had their respective drawbacks. A different floor system would need to be picked, and after a conversation with the design engineer the best option appeared to be a Filigree slab and beam system. However, since that system is proprietary it was impossible to design that myself it would need to be approximated with a similar common system. This led to the final design choice, pan joists with wide beams. This system is similar to the filigree system in that the weight of the slab is reduced by introducing voids into them with the pans in order to create ribs. The filigree system might have ended up cheaper since no additional form work is need during construction but structurally the designs should be comparable.



The CRSI Design Handbook was used to do the preliminary design. The design was picked based on the length of the span, superimposed load, and moment. This resulted in the selection of 30" wide forms with 6" ribs and a total slab depth of 24.5", 20" ribs with a 4.5" slab depth. This 4.5" slab gave the system its desired two hour fire rating. Rebar was then sized based on a three span layout with lengths of 40’, 35’, and 40’. The varying length bays which run along the north side of the building were ignored for simplicity because the load they contribute varied from zero to six feet of tributary area. Additionally, it was conservative to assume the second 40’ span was an end span because the extra load can only lower the mid span moment which defined



that span. The deflections in the slabs were then checked and found to be well under and code limitation ending up around L/1400 for the 40’ span which is equal to a defection of less than half an inch. A final design was done in PCA Slab where the strength and deflection of the preliminary design was rechecked with the current LRFD load factors. The final design uses (1) #7 and (1) #8 in each rib to resist positive moment and top steel varies from #4’s to #6’s throughout the three spans. For ease of design and in order to reduce construction cost, the same 30” forms where used for the roof slab. A new rebar layout was determined for these spans as well. It was determine through PCA analysis and then verified through hand calculations that the ribs were insufficient to handle shear at the beam interface. As a result the updated design uses tapered ends on the ribs to disperse the shear forces. The ends of the ribs taper out to 16” wide at the beam interface. The strength and shear checks of the loads in PCA Slab of the roof slab showed the preliminary designs of the CRSI Handbook to be adequate in both moment and shear so no additional calculations were performed on these values.

BEAM SCHEDULE

MARK	BASE	HEIGHT	LONGITUDINAL REINFORCEMENT		STIRRUPS	
			TOP BARS	BOTTOM BARS	SIZE	SPACING
J 1	6"-7 ¾"	20"	# 6 @ 9"	(1) # 7, (1) # 8	-	-
J 2	6"-7 ¾"	20"	# 5 @ 9"	(1) # 5, (1) # 6	-	-
J 3	6"-7 ¾"	20"	# 6 @ 9	(4) #8 TWO LAYERS	-	-
G 1	24"	24 ½"	(7) # 11	(4) # 9	4 LEGS # 4	1@2", 15@8", 2@11"
G 2	18"	24 ½"	(6) # 10	(3) # 6, (2) # 8	4 LEGS # 3	1@2", 15@7", 3@11"

COLUMN SCHEDULE

	MARK	C 1	
ROOF	DETAILS		
LEVEL 5	F'c	4000 PSI	
	SIZE	24" X 18"	
	VERT. REINF.	(12) # 6 (4 X 2)	
	TIES	#3 @ 12"	
LEVEL 4	F'c		
	SIZE		
	VERT. REINF.		
	TIES		
LEVEL 3	F'c		
	SIZE		
	VERT. REINF.		
	TIES		
LEVEL 2	F'c		
	SIZE		
	VERT. REINF.		
	TIES		
LEVEL 1	F'c	5500 PSI	
	SIZE	24" X 18"	
	VERT. REINF.	(12) # 6 (4 X 2)	
	TIES	#3 @ 12"	
FOUNDATION	F'c		
	SIZE		
	VERT. REINF.		
	TIES		

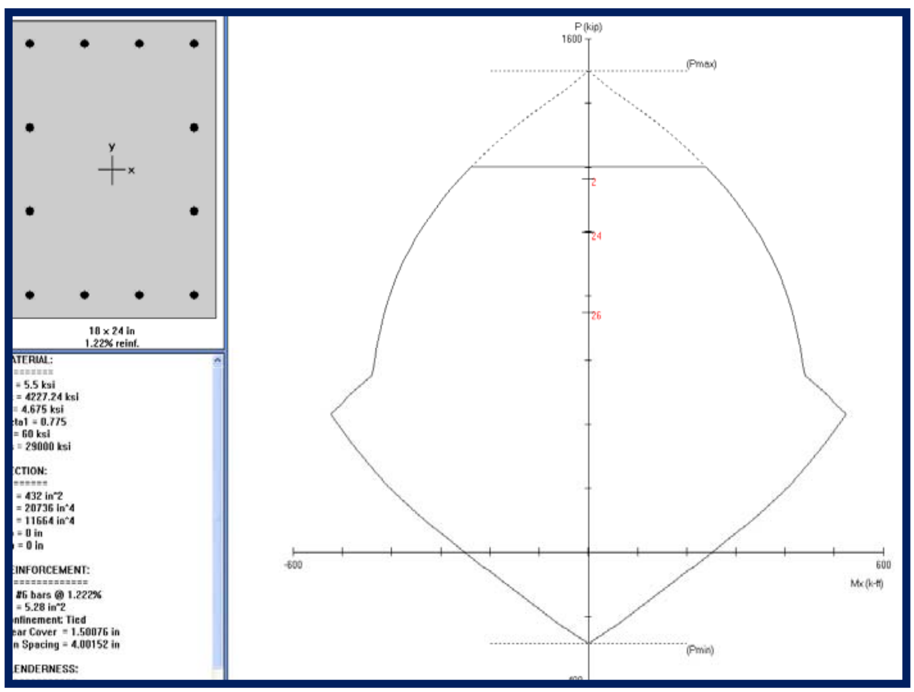
BEAMS

Following the load, the next part of the design were the beams. In order to simplify the framing, the beams were designed to be the same depth as the slab and ribs. As a result the interior beams end up with a width of 24” based on the reinforcement ratio and ultimate moment. The CRSI

Design Handbook was then used to find an appropriate rebar layout to support the given loading. These layouts can be found in Appendix A.4. These preliminary were then rechecked by hand calculations to determine final rebar layout. Interior beams were determined to need (7) #11's at supports for negative moment and (4) #9's to resist the positive moment at mid span. Designs were also checked for exterior beams not part of the lateral resisting system. In the east – west direction the beams were determined to need to be 18" wide with (6) #10's through the columns and (2) #8's and (3) #7's to resist positive moment. The beams to support the façade which run north - south along the exterior were determined to fit into the same dimensions of the other ribs; however, additional reinforcement was necessary at midspan. The required area of steel was too great for the rib width and as a result mandated two layers of (2) #8's.

COLUMNS

Columns were then designed using RAM Structural System. The full building was modeled in RAM and appropriate gravity loads were applied to the floors and lateral loads to the diaphragms. The structure was then run through the column module to come up with preliminary column designs. These were then modified with different dimensions, concrete strengths, and bar layouts until a simple uniform design was found. This resulted in 5500 psi concrete being used on the first two floors and 4000 psi on all the rest. All columns as rectangular 24" x 18" except those along the curved north wall which are circular with a 20" diameter for architectural reasons.



The rebar layout for all columns are (12) #6's arranged with 4 on the 18" faces and 2 in between on each side. Column designs were also spot checked with PCA Column.

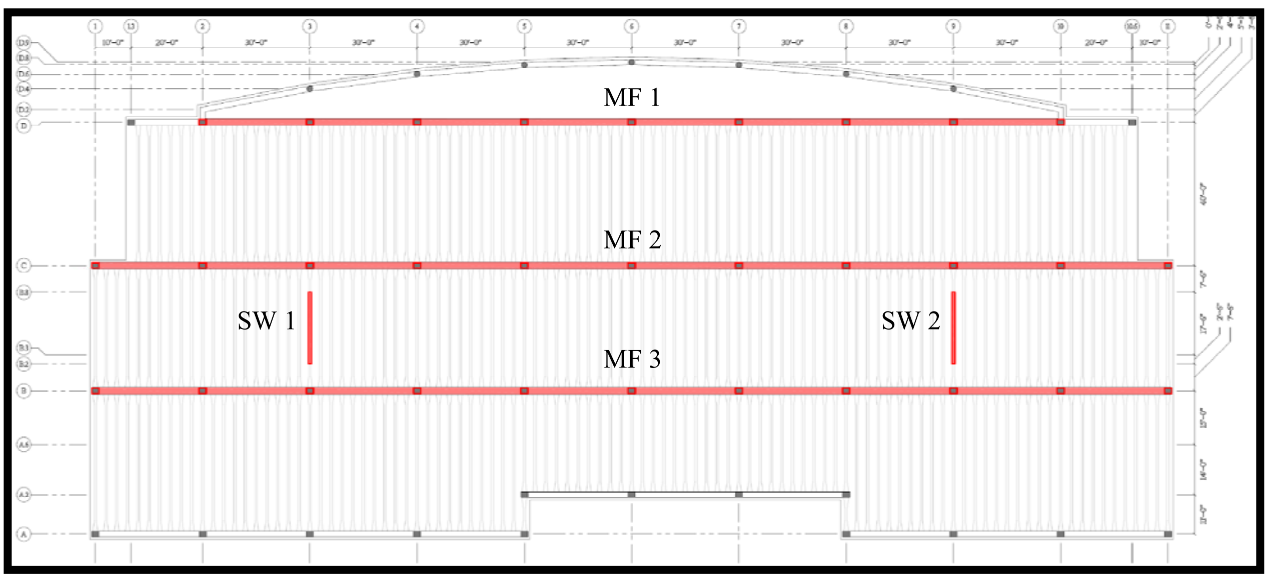
FOUNDATIONS

Foundations were designed similarly to the columns, where the foundation module was run to determine preliminary sizes. The designs were then modified through a series of iterations to simplify and unify the foundation designs. The typical foundation for all interior columns is 12'x12' with (13) # 7 in each direction. All circular columns have 9'x9' foundations with (11)

#6's in each direction. There are a variety of other foundations around the perimeter of the structure which result from various different loadings based on architectural features.

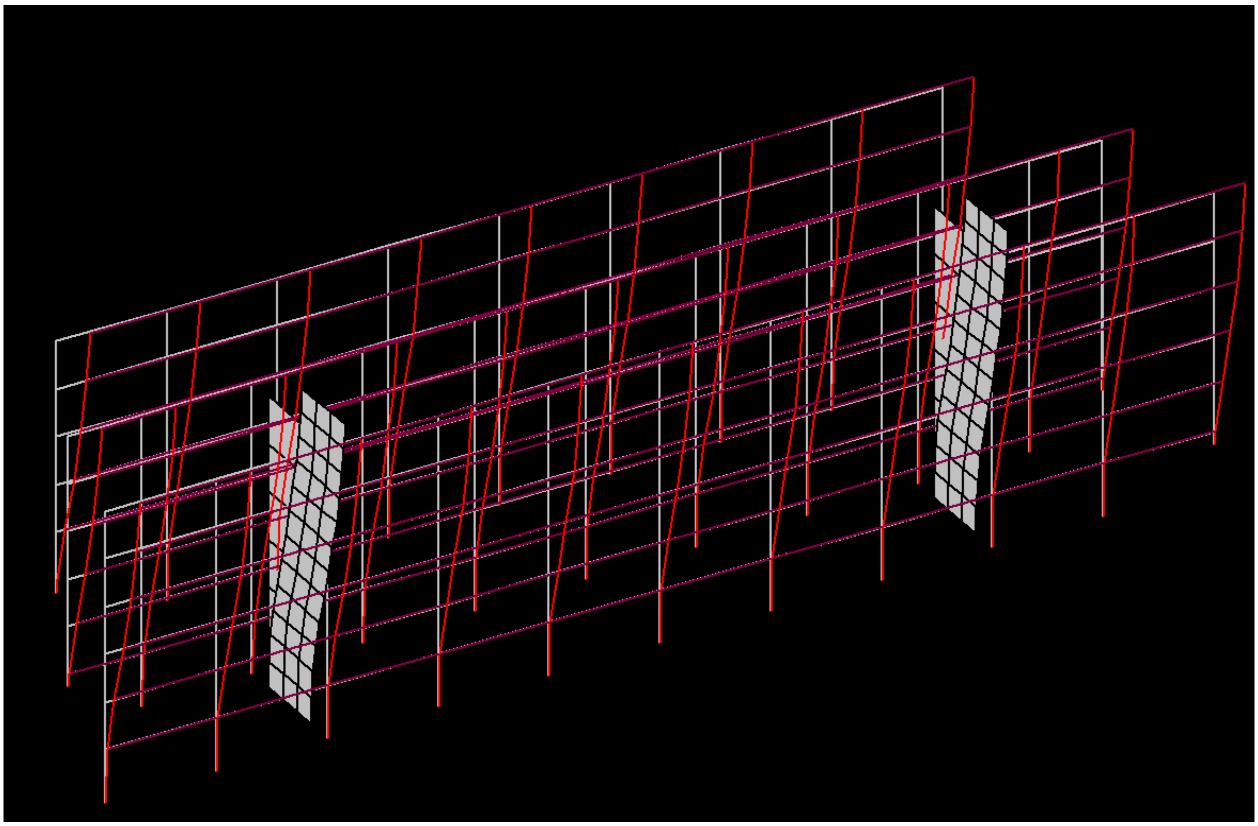
LATERAL LOAD RESISTING SYSTEM

The redesign of the lateral load resisting system is very similar to the system in the original with different systems along its long axis (E-W) and short axis (N-S). The new lateral resisting elements are shown in the picture below. In the E-W direction three moment frames fit into the existing grid along column lines B, C, and D, and act over the full height of the building and effectively its full length. In the N-S direction two full height shear walls fit off grid between lines B and C along column lines 3 and 9 to provide support for the short axis. These systems act to counter both wind and seismic forces which control in the east-west and north-south directions respectively. The moment frames where checked using the moments at the beam column connection from RAM. The shear walls were designed in PCA Wall and the load values can from calculated vertical windload distributions. The wall designs ended up being 10" wide with #5 @10" horizontally and #5 @16" vertically and (8) #9's in the boundary elements.



DRIFT

The analysis of total building drift was completed through the use of the RAM software. I placed the controlling load cases both seismic and wind into the software in their respective directions. Then analyzed drift at each corner of the building as well as the approximate center in order to achieve both the extreme values as well as an average. The computed values for drift were then compared to the code standard for serviceability Δ = H/400 which comes out to 2.34" when computed for 1000 Continental Square. The recorded values, at roof level, at all five points and in both load cases were well under this standard.



STORY DRIFT

Individual story shears were checked in respect to seismic loading. Using the same five control points, the seismic drifts at each level were compared to the allowable story drift, Δ = 0.020 h_{sx}, as given in table 12.12-1 in ASCE 7-05. All story drifts fell below their respective limit values. The exact values can be seen in Appendix A.11.

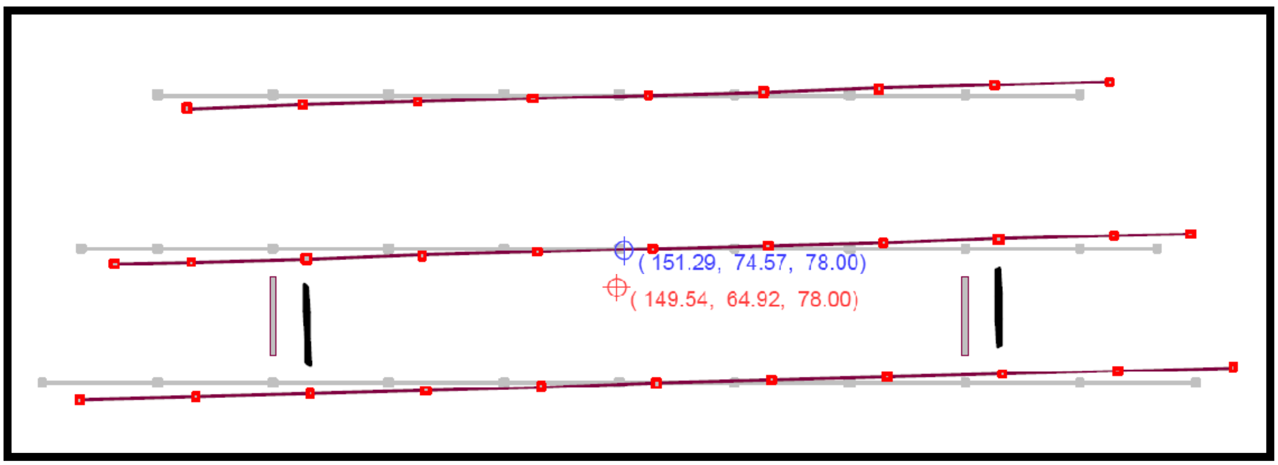
OVERTURNING MOMENT

Overturning moments were calculated by multiplying each seismic story force and wind load (after it had been distributed to its respective floor diaphragm through tributary area) by the height of that diaphragm. The resulting values for wind were 9439 ft-k (E-W) and 10448 ft-k (N-

S), and 55666 ft-k for seismic. When compared to the moment created by the calculated seismic weight times the minimum moment arm from the center of mass to the most extreme member of that direction's respective lateral system, it is found that all values are within an acceptable range. The moment countering overturning is approximately 102,620 ft-k in the north-south direction and over 5 million ft-k in the east-west direction. Obviously these moments would not actually be applied around a single point like they are assumed here but distributed throughout the structure; however, these calculations prove the weight of the building is enough to counter the overturning moment resulting from wind and seismic.

TORSION

Torsion in a building is a result of the eccentricity between the point where lateral loads are applied and the center of rigidity. This is to say the eccentricity between the center of mass and center of rigidity results in torsion from seismic loads, and similarly the eccentricity between the geometric center and center of rigidity results in torsion from wind. It can be assumed torsion has very little effect on the structure in the north-south direction because the centers of mass, rigidity, and geometry are within a foot of each other on every floor except the first and second. However, in the y direction greater eccentricities occur and thus the effect of torsional shears must be checked. This effect can be seen in the deflected shape of the lateral systems at roof level under seismic loads as shown below.



The torsional shear calculations had to be preceded by the calculation of relative stiffness for each lateral resisting frame. This was accomplished using the RAM model by applying unit loads to each frame at each level of the structure and checking their respective deformations. Diaphragms were turned off to prevent interactions between different frames, and all stories below the one being checked were set as below ground to prevent their lateral deflection. The stiffness of each frame was determined by dividing the load by its deformation. Then these were summed for each level so the relative stiffness of each frame on each level to all the others could be determined. The results were that the shear walls are generally much stiffer than the moment frames. As expected since both shear walls are identical they are equally stiff and split the loads evenly between them. The moment frames on the other hand are not identical but still relatively even and take approximately 30, 36, and 34 percent of the lateral forces on each floor.

RELATIVE STIFFNESS					
Floor	N-S		E-W		
	SW 1	SW 2	MF 1	MF 2	MF 3
1	50.0 %	50.0 %	30.7%	35.6%	33.8%
2	50.0 %	50.0 %	30.3%	35.8%	33.9%
3	50.0 %	50.0 %	30.0%	35.9%	34.1%
4	50.0 %	50.0 %	30.1%	35.9%	34.1%
5	50.0 %	50.0 %	30.1%	35.8%	34.1%
Roof	50.0 %	50.0 %	30.2%	36.0%	33.8%

Once the relative stiffness of each frame is computed, torsional effects can be determined. As was stated earlier, due to its symmetry, the north-south direction is ignored. The formula for torsional shear in a direction is $F_i = VeR_iC/\sum RC^2$. Here V is the base shear in the east-west direction, R_i is the relative stiffness of a frame, and C is the perpendicular distance to the centers of geometry or rigidity depending on whether the load is wind or seismic.

TORSION FROM WIND															
				MF1				MF2				MF3			
Floor	V	COG, Y	e	R_i	C	RC^2	F_i	R_i	C	RC^2	F_i	R_i	C	RC^2	F_i
1	29.85	63.00	1.60	30.7%	52.00	828.84	0.72	35.6%	12.00	51.23	0.19	33.8%	23.00	178.64	0.35
2	31.75	66.88	3.02	30.3%	48.12	700.57	1.44	35.8%	8.12	23.63	0.29	33.9%	26.88	245.01	0.90
3	33.25	65.88	2.02	30.0%	49.12	723.03	1.01	35.9%	9.12	29.88	0.22	34.1%	25.88	228.48	0.60
4	34.56	65.88	2.02	30.1%	49.12	725.84	1.05	35.9%	9.12	29.83	0.23	34.1%	25.88	228.10	0.63
5	35.36	65.88	1.95	30.1%	49.12	726.29	1.04	35.8%	9.12	29.81	0.23	34.1%	25.88	228.08	0.62
Roof	17.88	65.88	1.06	30.2%	49.12	729.02	0.29	36.0%	9.12	29.95	0.06	33.8%	25.88	226.25	0.17

TORSION FROM SEISMIC															
				MF1				MF2				MF3			
Floor	V	COR, Y	e	Ri	C	RC2	Fi	Ri	C	RC2	Fi	Ri	C	RC2	Fi
1	5.18	74.97	12.81	30.7%	40.03	491.17	0.90	35.6%	0.03	0.00	0.00	33.8%	34.97	412.97	0.87
2	8.68	75.30	11.43	30.3%	39.70	476.85	1.33	35.8%	0.30	0.03	0.01	33.9%	35.30	422.55	1.32
3	28.22	75.32	11.45	30.0%	39.68	471.83	4.28	35.9%	0.32	0.04	0.04	34.1%	35.32	425.56	4.34
4	34.36	75.20	11.33	30.1%	39.80	476.53	5.19	35.9%	0.20	0.01	0.03	34.1%	35.20	421.97	5.19
5	141.76	75.03	11.09	30.1%	39.97	480.91	21.05	35.8%	0.03	0.00	0.02	34.1%	35.03	417.86	20.87
Roof	179.60	74.85	9.94	30.2%	40.15	487.08	24.14	36.0%	0.15	0.01	0.11	33.8%	34.85	410.26	23.43

The effects of torsional shear are greater with seismic loading than in wind loading, which understandable since seismic is the controlling load case anyway. The increase in shear as a result of torsion is around 30% on each floor. These values only result in a few kips in each moment frame, however one must account for their effects. As a result of torsion moment frames one and three end up with higher shear loads even though frame two has the greatest stiffness and there for takes the most direct shear. The updated story shears in each moment frame are given in the table below.

DESIGN SHEAR IN EAST -WEST DIRECTION				
Floor	Direct Shear	Total MF1	Total MF2	Total MF3
1	5.18	2.49	1.84	2.62
2	8.68	3.95	3.12	4.26
3	28.22	12.74	10.18	13.96
4	34.36	15.52	12.35	16.90
5	141.76	63.73	50.83	69.15
Roof	179.60	78.41	64.77	84.10

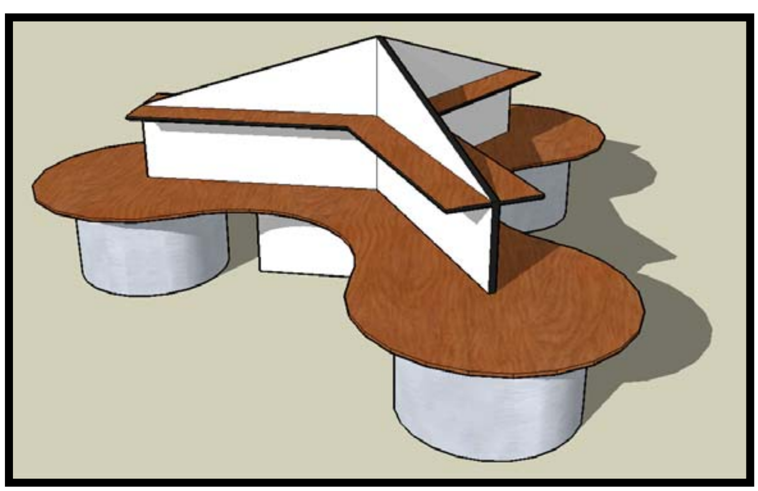
VII. BREADTH STUDIES

ARCHITECTURE FLOOR LAYOUT

The first breadth study for this thesis was an architectural layout of a typical office floor. An architectural engineering firm was chosen as the tenant since currently there are no companies leasing the space, and there is an obvious familiarity with the needs of such an office. The first step in the process was to set up a schedule of required spaces and approximate square footages. Research also had to be done on the amount of desk space needed per worker and how many additional spaces each employee needs such as conference rooms and common space. General ratios of managers to engineers to drafts men, etc. were also estimated. Thornton Tomasetti was gracious enough to supply floor plans of their New York office for me to approximate such values in addition to drawing off of experience from summer internships.

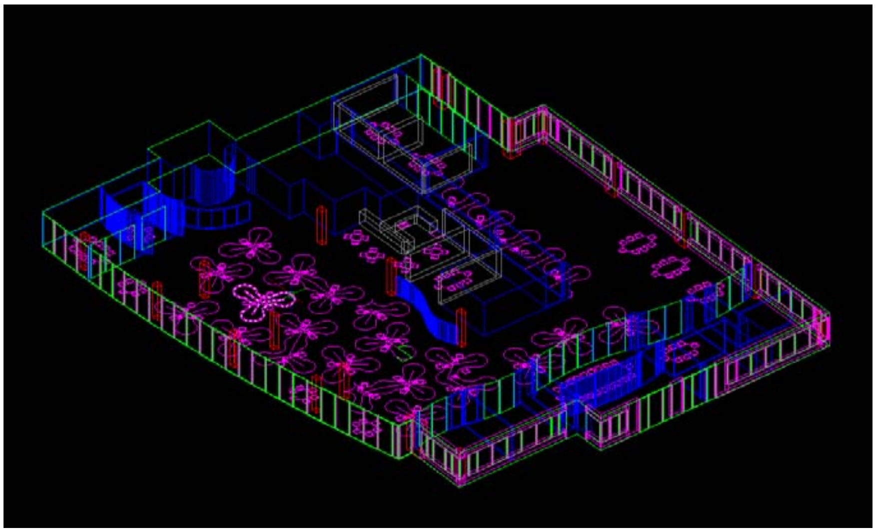
Use	Percent Area at TT	Resulting Area	Percent Area at 1000	Actual Area of Design	Percent Difference
Cubicles	44.76%	6644	45.50%	6382	-3.94%
Offices	22.40%	3325	13.17%	1847	-44.45%
Conference rooms	13.69%	2032	19.74%	2769	36.29%
Kitchens	4.43%	657	4.56%	640	-2.56%
Libraries	7.24%	1074	9.04%	1268	18.04%
Drafting areas	4.90%	727	5.49%	770	5.94%
Waiting areas	2.59%	384	2.49%	349	-9.08%

Average areas are within 10 % of those of the Thornton Tomasetti office with the exception of offices, conference rooms, and library space. Everyone who was consulted said there is never enough conference room and open table space which is why offices were sacrificed for it. However, if the need for those office spaces arises there are several conference rooms which are a comparable size to offices and could be converted which would bring both values closer to those of Thornton Tomasetti.



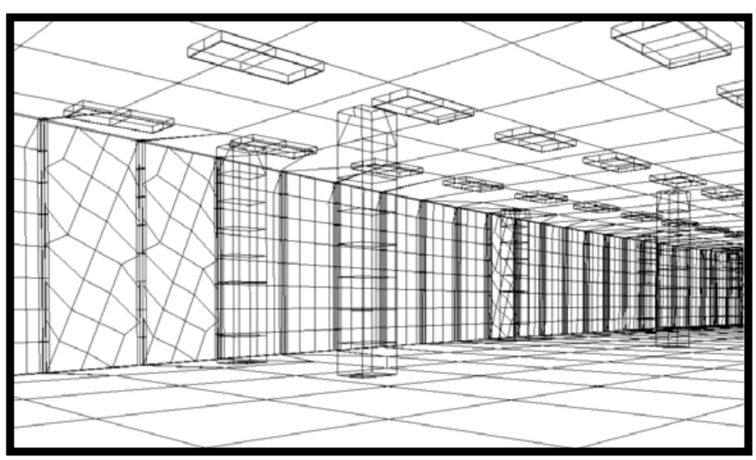
The next topic which was confronted in this breadth study is the cubicle work space. In Thornton Tomasetti's office the average cubicle is approximately 45 square feet with 27.5 square feet of desk space. However, workers who were contacted said there is almost never enough desk space because of the amount of space drawings and papers take up. Additionally, traditional square cubicles, although efficient, seem out of place in an AE office where the idea of modern edgy designs is trying to be sold. To remedy these problems a new modular type of cubicle was developed with gives the worker a more desk space, more of which is within arm's reach, while giving the floor plan a little more creativity.

The final architectural detail of the floor plan is taken from the curving line of the north face of the building, elliptical entry lobby, and the freeform shape of the cubicle system. These curvalinear shapes are carried through to the concentric ellipses of the lobby and reception desk, and the surpentine wall at the west end of the office and the divider between the cubicle space and kitchen. Just as the north face of the building breaks the strict rectangular form of the building and adds a much needed architectural intrest to the façade of the building, these curving features break the monotony of a linear floor plan, soften its harshness and add some focal interest.



DAYLIGHTING CALCULATIONS

The purpose of the second breadth study of this report is to look into the effects of day lighting on the luminance of the main office area in the cubicles. With the expansive glass on the convex curtain wall of the building there appears to be the potential to save money by using the diffused northern light to illuminate part of the cubicle space. This would require the design of the lighting system to be on multiple zones which could be shut off or put on light sensors to vary the intensity of their output.

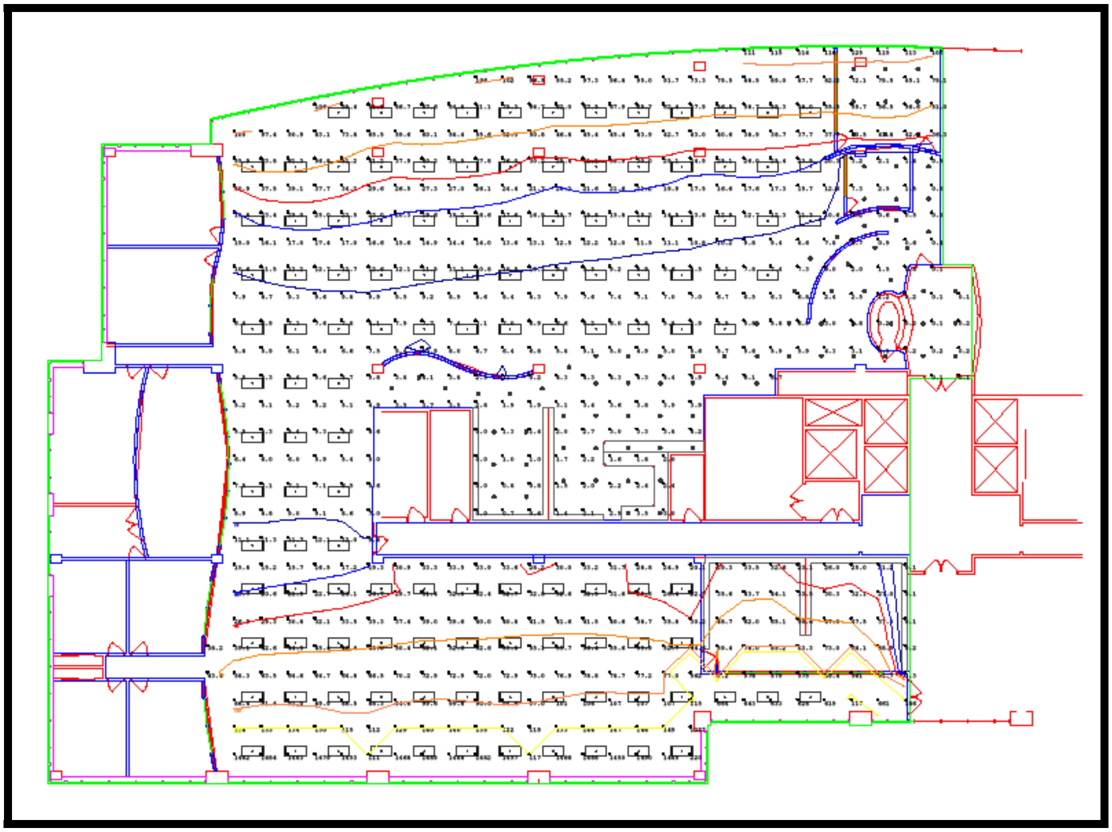


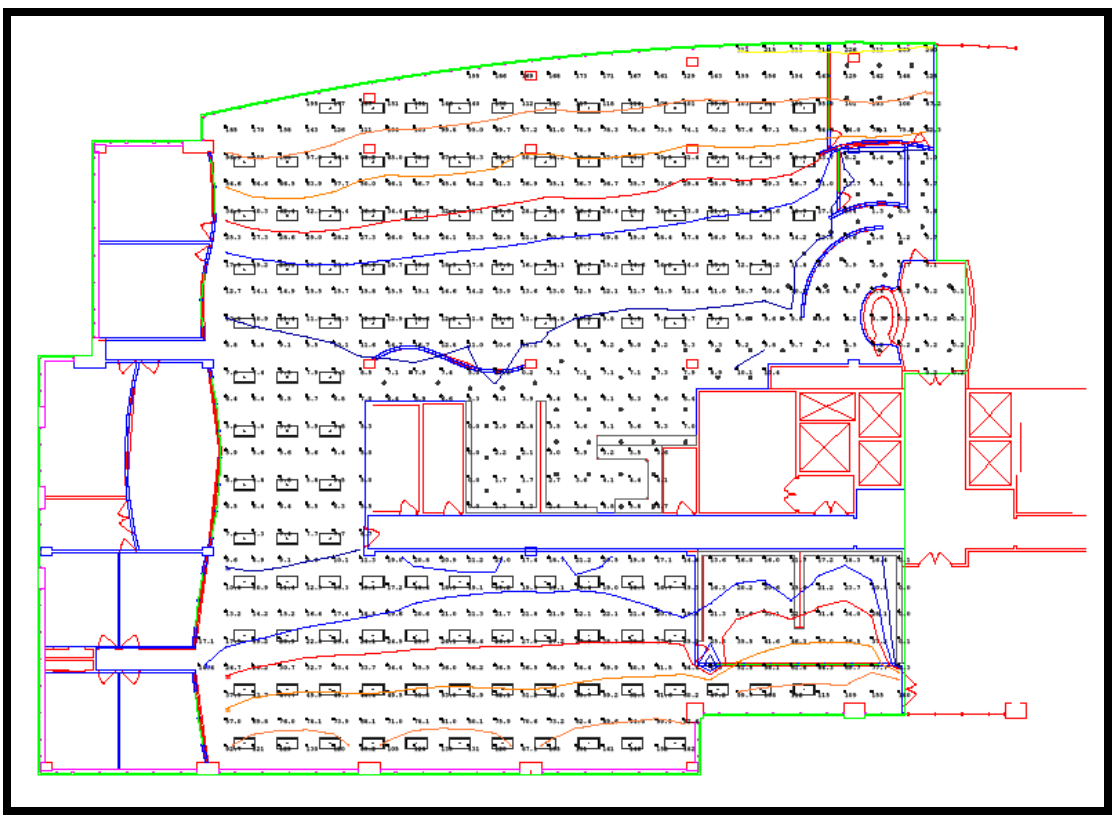
Since the layout of the floor space is the responsibility of the tenant if follows that there are not fixtures in the rental spaces before they are leased. As a result the first step in the lighting calculations is to layout a general lighting plan. This was laid out to match the architectural floor plan from the first depth study. Two different general lighting fixtures were picked to achieve different goals. The lobby space, kitchen area, and walkways are light by recessed downlights made by Cooper Lighting. These were picked because they will create a more interesting lighting pattern as the fall on the curved walls. Additionally, the smaller fixtures are able to follow the curves in these areas better than the larger 2’ x 4’ fixtures.

The other type of general light fixture is a 2x4 recessed troffer designed by Lightolier. These will provide an even light over desks in the work space. The specific luminary which was picked has wavy shields over the halogen tubes which serve to diffuse light and prevent glair on computer screens. However, these shields should also echo the curving walls which surround the cubicle area.

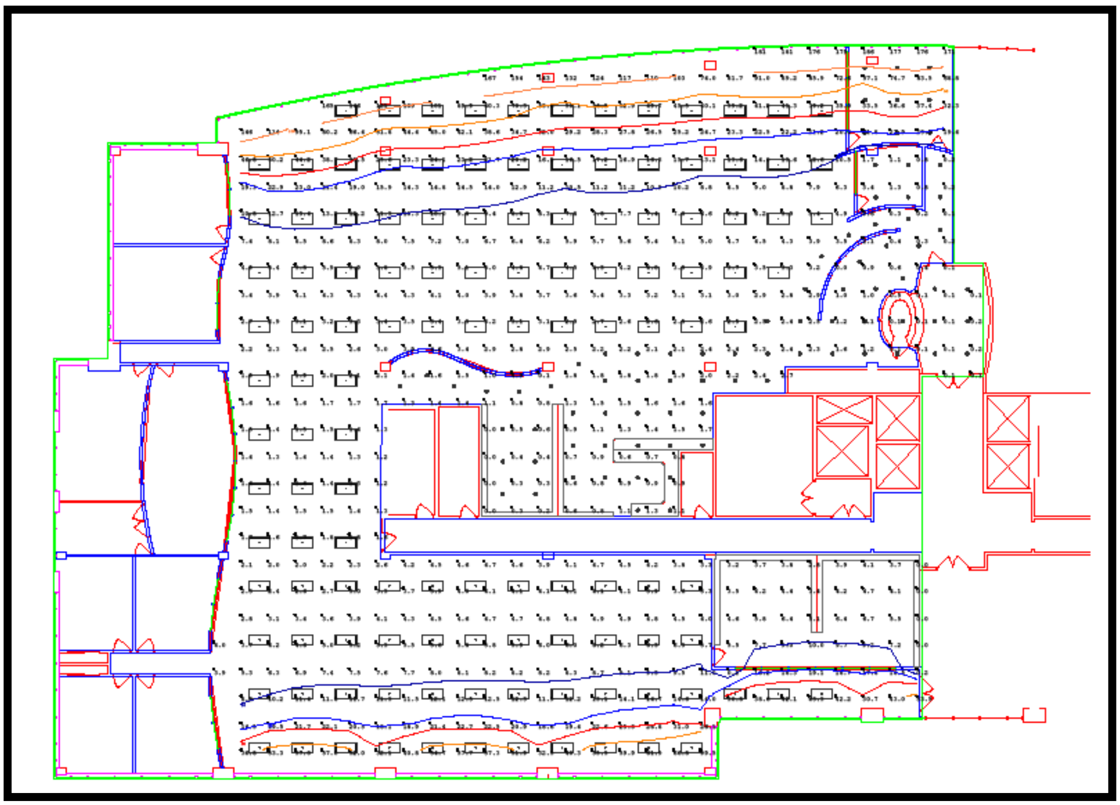
Preliminary spacing was determined for each luminare by multiplying the spacing criteria by the 7.5’ distance between the ten foot ceiling and the desk tops. This resulted in an approximate spacing for the downlights to be six feet, and eight and ten feet for the long and short directions of the 2x4 troffer respectively. These guidelines should ensure even consistent lighting over the work plane. It was also determined that since the space is an office with high VDT use, this area should fall under luminance category “D” which results in a required luminance of 30 foot candles.

The first diagram shows the potential of daylighting in what is effectively the best case scenario, the winter solstice around 1 o’clock, where you can see the red line which marks where the luminance drops below 30 footcandles. Light clearly penetrates the entire depth of the southern side of the building and since most of that space is not used by engineers it is ok if it is light by direct harsh sunlight. The ambient northern light which is much better to work by still penetrates about 20’ into the space which would allow most of the first two rows to be shut off.





This next diagram is of the summer solstice when the least direct sunlight will enter the building on the south side. This is obvious from the fact that the 30 footcandle line only at the third row of lights about 25' in. However, this will still save three rows of lights from being turned on. More interesting is that the ambient northern light actually penetrates deeper about 25' as well, allowing three rows on that side of the building to be shut off as well. The final diagram is the worst case scenario which is when it is cloudy or overcast. Even under this situation ambient light still reaches past the first row of light approximately 10' into the building which would allow one row of lights on both sides of the building to be shut off.



To determine the total power savings average the luminaries which are not used during the winter and summer. Then figure the total unused fixtures per year based on the statistic that 53% of days in Philadelphia are sunny. This totals 15,659 fixtures per year, which when multiplied by the average work day and the wattage per fixture results in 13,529 kilowatt-hours saved per year. At the current price of energy in Philadelphia, $0.151 per kWh, that totals $2042.87 per year. This calculation includes only the general area of the office and does not include the offices or conference rooms which also have the same potential for savings. This is also only half of one floor. The best way to make use of these savings would be to have the first four rows of lights nearest he windows be on four individual zones and turn a whole row on or off as needed. The savings could also be even greater if dimmers with light sensors were attached to the different zones; the luminaries and ballast are already compatible with such systems. Then as the light fluctuated throughout the day from sun movement or cloud cover the light could gradually adjust their output to match.

VIII. CONCLUSIONS & RECOMMENDATIONS

In order to reduce inefficiency it the design of the steel structural system in the existing building at 1000 Continental Square, this thesis proposed that an all concrete structural system would more efficiently handle the design loads. Additionally the concrete system would reduce lead time, be fire resistant, and be better able to handle serviceability issues. Although, the final design did manage to control these issues it did not end up being a more efficient system. Despite the reduction in lead time the overall construction time could be up to two months longer. The prices of the two systems are comparable however the concrete design still costs approximately $2.50 more per square foot. A filigree slab and beam system might be able to better compete with steel on these two aspects; however, it was not possible to get a design from the proprietors of the system in time for this paper. Had the conditions of the design been different such a stricter height limitations, desire for more floors, more stringent vibration or deflection limits, or more room for MEP systems in the ceiling plenum, the concrete system would have been the better choice because of its more massive structure and thinner overall slab depth.

The results of the breadth studies were such that architectural layout would be a feasible and adaptable layout for an office in a typical floor of this structure. An assortment of architectural aspects makes it an appealing place to work. Additionally the modular cubicle units make the space versatile enough to fit any number of tenants not just an AE firm.

The lighting design, which makes use of the incredible amounts of daylight the curtain walls let in, is equally suited any number of purposes because of the generic uniformity of the lighting layout. Additionally, if exploited, the zone system would have the ability to save a tenant several thousand dollars a year. If they were willing to spend a little more upfront to fit the system with light sensors, the system could actively maintain itself at the most efficient, ideal lighting levels saving even more energy and money.

All of the aspects of this thesis are equally feasible and suitable for use in 1000 Continental Square, and although I doubt any will ever come to realization, under different circumstances and with different design constraints all have proven to be viable alternatives.

IX. APPENDICES

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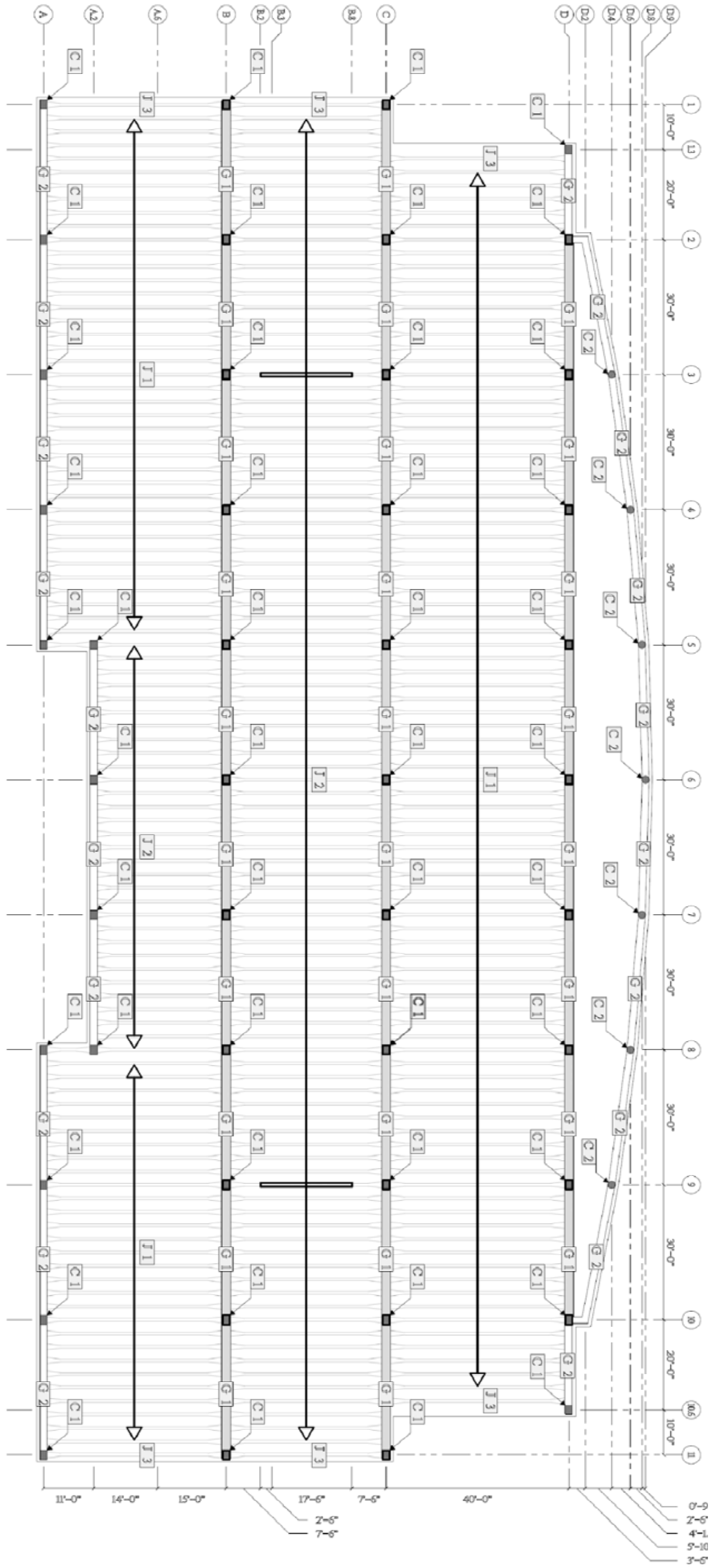
A.O ACKNOWLEDGEMENTS

- The Harmon Group - For providing my building and support through the entire process
    - Kirk Harman
    - Chris Shaffer
    - Chris Godshall
- BPG Property Group, Ltd. - For providing building statistics and architectural drawings
    - Margret Michel
- Thornton Tomasetti - For providing floor plans for my architectural breadth
    - Ken Murphy
- Kristin Maruszewski - For showing me the ropes on the AGi32 software
- Eastern State Filligree – For providing technical support on filigree slab design
- Everyone else who help along the way there are too many to list.

A.1 FLOOR PLAN AND SCHEDULE

BEAM SCHEDULE						
MARK	BASE	HEIGHT	LONGITUDINAL REINFORCEMENT		STIRRUPS	
			TOP BARS	BOTTOM BARS	SIZE	SPACING
J 1	6"-7 ¾"	20"	# 6 @ 9"	(1) # 7, (1) # 8	-	-
J 2	6"-7 ¾"	20"	# 5 @ 9"	(1) # 5, (1) # 6	-	-
J 3	6"-7 ¾"	20"	# 6 @ 9	(4) #8 TWO LAYERS	-	-
G 1	24"	24 ½"	(7) # 11	(4) # 9	4 LEGS # 4	1@2", 15@8", 2@11"
G 2	18"	24 ½"	(6) # 10	(3) # 6, (2) # 8	4 LEGS # 3	1@2", 15@7", 3@11"

COLUMN SCHEDULE			
	MARK	C 1	
ROOF	DETAILS		
LEVEL 5	F'c	4000 PSI	
	SIZE	24" X 18"	
	VERT. REINF.	(12) # 6 (4 X 2)	
	TIES	#3 @ 12"	
LEVEL 4	F'c		
	SIZE		
	VERT. REINF.		
	TIES		
LEVEL 3	F'c		
	SIZE		
	VERT. REINF.		
	TIES		
LEVEL 2	F'c		
	SIZE		
	VERT. REINF.		
	TIES		
LEVEL 1	F'c	5500 PSI	
	SIZE	24" X 18"	
	VERT. REINF.	(12) # 6 (4 X 2)	
	TIES	#3 @ 12"	
FOUNDATION	F'c		
	SIZE		
	VERT. REINF.		
	TIES		



A.2 WIND DESIGN CALCULATIONS

WIND DESIGN - ASCE 7-05 METHOD 2

1) BASIC WIND SPEED, $V = 90$ MPH (FIGURE 6-1)

2) IMPORTANCE FACTOR, $I = 1.0$ (TABLE 6-1)
BUILDING CAT. II (TABLE 1-1)

3) EXPOSURE CAT. = B (SECTION 6.5.6.3)
SURFACE ROUGHNESS = B (SECTION 6.5.6.2)

VELOCITY PRESSURE EXPOSURE COEFFICIENT, K_z
FROM (TABLE 6-3)

FLOOR	TRUE HEIGHT	EST. HEIGHT	K_z
1	13′	15′	0.57
2	26′	30′	0.70
3	39′	40′	0.76
4	52′	60′	0.85
5	65′	70′	0.89
ROOF	77′ 6½″	80′	0.93

4) TOPOGRAPHIC FACTOR, $K_{ZT} = 1.0$
SITE IS FLAT, THEREFORE $K_{ZT} = 1.0$

5) GUST EFFECT FACTOR, $G = .828$ E-W OR $.798$ N-S

$$G = 0.925\left(\frac{(1+1.7 g_Q I_z Q)}{1+1.7 g_v I_z}\right) = 0.925\left(\frac{(1+1.7(3.4)(.28)(Q)}{1+1.7(3.4)(.28)}\right)$$

$h = 78'$

$c = .30$

$z_{MIN} = 30'$

$\bar{z} = 0.6h = 46.8'$

$$I_z = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.3\left(\frac{33}{46.8}\right)^{1/6} = .283$$

$Q = .831$ or $.778$
E-W N-S

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_z}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{B+78'}{359.5}\right)^{0.63}}} = \begin{array}{c} .83 \\ \text{or} \\ .78 \end{array}$$

$B = 132'$ or $300'$

$h = 78'$

$\ell = 320'$

$\bar{\epsilon} = 1/3.0$

$$L_z = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320'\left(\frac{46.8'}{33}\right)^{1/3.0} = 359.523 \text{ ft}$$

6) ENCLOSURE CLASSIFICATION = ENCLOSED (SECTION 6-2)

7) INTERNAL PRESSURE COEFF., $GC_{pi} = \pm 0.18$ (FIGURE 6-5)

8) EXTERNAL PRESSURE COEFF. (FIGURE 6-6)

WINDWARD WALL, $C_p = 0.8$

LEEWARD WALL, $C_p = -0.3$ E-W OR -0.5 N-S

9) VELOCITY PRESSURE (EQ. 6-15)

SEE SPREAD SHEET

10) DESIGN WIND PRESSURES, P (EQ. 6-17)

SEE SPREAD SHEET

LEEWARD WALL $p = q_h GC_p - q_h(GC_{pi}) = 16.4(GC_p) - 16.4(\pm.18)$

E-W : -7.03 lb/ft²

N-S : -9.50 lb/ft²

A.3 SEISMIC DESIGN CALCULATIONS

SEISMIC

SITE LOCATION : LAT : 40.0°N LONG : 75.4°W

OCCUPANCY CATAGORY : II

IMPORTANCE FACTOR $(I) = 1.0$

SPECTRAL ACCELERATION

SHORT PERIODS $(S_S) = 0.278$

ONE SEC. PERIODS $(S_1) = 0.060$

SOIL SITE CLASS C

SITE COEFFICIENTS

$F_A = 1.2$

$F_V = 1.7$

MCE SPECTRAL RESPONSE PARAMETERS

$S_{MS} = F_A S_S = 1.2\,(0.278) = 0.3336$

$S_{M1} = F_V S_1 = 1.7\,(0.060) = 0.102$

DESIGN SPECTRAL ACCELERATION PARAMETERS

$S_{DS} = 2/3\, S_{MS} = 2/3\,(0.3336) = 0.2224$

$S_{D1} = 2/3\, S_{M1} = 2/3\,(0.102) = 0.068$

SEISMIC DESIGN CATAGORY

$S_{DS} = 0.2224 \Rightarrow B$

$S_{D1} = 0.068 \Rightarrow B$

DESIGN COEFFICIENTS AND FACTORS

(E-W) ORDINARY STEEL MOMENT FRAME

$R = 3.5 \quad \Omega_0 = 3.0 \quad C_d = 3.0$

(N-S) COMP. STEEL + CONCRETE ECC. BRACED FRAME

$R = 8.0 \quad \Omega_0 = 2.0 \quad C_d = 4.0$

BUILDING FUNDAMENTAL PERIOD

$T_a = C_t h_n^x = 0.028\,(78')^{0.8} = (E\text{-}W)\ 0.914\,s$

$= 0.02\,(78')^{0.75} = (N\text{-}S)\ 0.525\,s$

SEISMIC RESPONSE COEFFICIENT

(E-W) $C_s = \dfrac{S_{DS}}{R/I} = \dfrac{.2224}{3.5/1.0} = 0.0635$

(N-S) $C_s = \dfrac{S_{DS}}{R/I} = \dfrac{.2224}{8.0/1.0} = 0.0278$

BECAUSE $T_a \le T_L = 6$

$C_s \le \dfrac{S_{D1}}{T_a\left(\frac{R}{I}\right)} = \dfrac{.0635}{.914(3.5/1.0)} = (E\text{-}W)\ \boxed{.0199}$

$= \dfrac{.0278}{.525(8.0/1.0)} = (N\text{-}S)\ .0066$

SEISMIC BASE SHEAR

$V = C_s W = .0199\,(17910 + 3217.5) = 419.6$ kips

VERTICAL DISTRIBUTION : SEE SPREAD SHEET

SEISMIC WEIGHT

ROOF DL = 5 PSF
FLOOR DL = 50 PSF
SUPERIMPOSED DL = 30 PSF
PARTITION LL = 10 PSF
SNOW LOAD = NA P_f < 30 PSF
STORAGE LOAD = 25 PSF (.25 × 100 PSF)

APPROXIMATE FLOOR AREA
120' × 300' = 36000 SF / FLOOR

FLOOR	AREA (SF)	UNIFORM LOAD (PSF)*	WEIGHT (K)
1	36000	90 × A + 25 × .10A	3330
2	36000	90 × A + 25 × .10A	3330
3	36000	90 × A + 25 × .10A	3330
4	36000	90 × A + 25 × .10A	3330
5	36000	90 × A + 25 × .10A	3330
R	36000	35	1260
TOTAL	216000		17910

* ACTUAL STORAGE AREA UNKNOWN ASSUMED 10% FLOOR AREA

APPROXIMATE BUILDING PERIMETER
2 × 150' + 2 × 300' = 900' / FLOOR

ARCH. PANEL SELF WEIGHT = 50 PSF

FLOOR	HEIGHT	PERIMETER	PANEL S.W. (PSF)	WEIGHT (K)
1	13'	900'	50	585
2	13'	900'	50	585
3	13'	900'	50	585
4	13'	900'	50	585
5	13'	900'	50	585
R	6'6"	900'	50	292.5
TOTAL				3217.5

DESIGN COEFFICIENTS AND FACTORS

(E-W) SPECIAL REINFORCED CONCRETE MOM. FR.

$R = 6.0 \quad \Omega_o = 2.5 \quad C_d = 5$

(N-S) SPECIAL REINFORCED CONCRETE SHEAR WL.

$R = 8.0 \quad \Omega_o = 3.0 \quad C_d = 5.5$

BUILDING'S FUNDIMENTAL PERIOD

$T_a = C_t h_n^x =$

$C_t = 0.016$ (E-W) $x = 0.9$

$= 0.02$ (N-S) $= 0.75$

$T_{a_{E-W}} = 0.016 (78')^{0.9} = 0.807s$

$T_{a_{N-S}} = 0.02 (78')^{0.75} = 0.525s$

SEISMIC RESPONSE COEFFICIENT

(E-W) $C_s = \dfrac{S_{DS}}{(R/I)} = \dfrac{0.2224}{(6.0/1.0)} = 0.037$

(N-S) $C_s = \dfrac{0.2224}{(8.0/1.0)} = 0.028$

BECAUSE $T_{MAX} = T_a \times C_u \leq T_L = 6$

(E-W) $C_s \leq \dfrac{S_{D1}}{T_a \times C_u (R/I)} = \dfrac{0.068}{.8(1.7)(6/1)} = .008 < \boxed{.01}$

(N-S) $C_s \leq \dfrac{0.068}{.5(1.7)(8/1)} = .0095 < \boxed{.01}$

SEISMIC WEIGHT

- SUPERIMPOSED LOADS

ROOF FINISHES : 5 PSF
FLOOR FINISHES: 12 PSF
CEILING DL : 10 PSF
(MEP + FINISHES)

- SELF WEIGHTS (CONCRETE) (REBAR)

SLABS : 0.74 CF/SF × 150 PCF + 2 PSF = 113 PSF
COLUMNS : 18" × 24" × 150 PCF = 450 PLF
ROOF SLAB : 0.74 CF/SF × 150 PCF + 2 PSF = 113 PSF
CURTIAN WALLS : 50 PSF
BEAMS (INT) : 24.5" × 48" × 150 PCF = 1225 PLF
(EXT) : 24.5" × 36" × 150 PCF = 920 PLF

- SEE SPREADSHEET FOR CALCS

FLOOR	SEISMIC WEIGHT
ROOF	5986 KIPS
5	6804 KIPS
4	6804 KIPS
3	6804 KIPS
2	6875 KIPS
1	6216 KIPS
GROUND	287 KIPS
TOTAL	39775 KIPS

SEISMIC BASE SHEAR

$$V = C_s W = 0.010 (39775) = 397.8 \text{ KIPS}$$

LIVE LOADS

ALL FLOORS: 100 PSF
ROOF (SNOW): 21 PSF

LIVE LOAD REDUCTION NOT APPLICABLE OTHER THAN MEMBERS SUPPORTING 2 MORE FLOORS.

DEAD LOADS (SUPERIMPOSED)

CEILING DEAD LOADS (MEP + FINISHES) : 10 PSF
ROOF FINISHES : 5 PSF
FLOOR FINISHES : 12 PSF ?? PER FLOOR SLAB DEPRESSION

APPROXIMATE FLOOR AREAS

FLOOR	AREA (SF)	PERIMETER (FT)
R	35697.5	864
5	35697.5	864
4	35697.5	864
3	35697.5	864
2	36027.5	886
1	33370.0	830
TOTAL	212187.5	5172

DISTRIBUTION OF LATERAL LOADS FROM SEISMIC ACROSS THE MOMENT FRAMES AT EACH STORY BASED ON RELATIVE STIFFNESS

VERTICAL DISTRIBUTION OF SEISMIC LOADS

ROOF	179.6 kips
5	141.8 kips
4	34.4 kips
3	28.2 kips
2	8.7 kips
1	5.2 kips

RELATIVE STIFFNESSES

	MF #1	MF #2	MF #3
ROOF	29.8%	34.2	36.0
5	29.8	35.9	34.3
4	29.8	35.9	34.3
3	29.7	35.9	34.4
2	26.6	37.8	35.6
1	33.5	52.2	14.3
	53.5	61.4	64.7
	42.2	50.8	48.7
	10.2	12.3	11.8
	8.4	10.1	9.7
	2.3	3.3	3.1
	1.7	2.7	0.7
TOTAL	118.5	140.7	138.6

A.4 PAN JOIST DESIGN

PAN JOIST DESIGN (FROM CRSI HANDBOOK)

- SUPERIMPOSED LOADING

FLOOR LL = 100 PSF
ROOF LL = 21 PSF (SNOW LOAD)
FLOOR FINISHES = 12 PSF
ROOF FINISHES = 5 PSF
CEILING DEAD LOAD = 10 PSF (MEP + FINISHES)
BRIDGING = 2 PSF

- TOTAL FACTORED SUPERIMPOSED LOAD

$W_F = 1.2(12+2+10) + 1.6(100) = 188.8$ PSF

OR

$W_R = 1.2(5+2+10) + 1.6(21) = 54.0$ PSF

FLOORS

TRY 30" FORMS + 6" RIBS @ 36" C.C.

END SPANS = 40' - 1' = 39' = ℓ_n
INT SPAN = 35 - 1' = 34' = ℓ_n

- END SPAN = 20" + 4.5"

TOP BARS : #6 @ 9 IN AT 1ST INTERIOR SUPPORT
BOTTOM BARS : (1) #7 + (1) #8 (PER RIB)
WEIGHT : 1.96 PSF

TOP BARS @ EXTERIOR : $A_S \approx \frac{1}{3}(0.60 + 0.79) \times \frac{12}{36} = 0.15 \text{ IN}^2/\text{ft}$

USE #4 @ 12" w/ STANDARD 90° END HOOKS

- INT SPAN

TOP BARS: #5 @ 9.5 IN
BOTTOM BARS: (1) #5 + (1) #6 (PER RIB)
WEIGHT: 1.38 PSF

- CONCRETE QUANTITY = 109 PSF

INCLUDING BRIDGING = $\left(\frac{109+2}{150}\right) = 0.74$ CF/SF

- DEFLECTION$_u$ = $\left(\frac{100}{(188.8)(1.6)}\right)\left(\frac{39}{480}\right) = .027 \leq \left(\frac{39}{750}\right) = .052$

OK

ROOF

USE SAME 30" FORMS + 6" RIBS @ 36" C.C

- END SPAN

TOP BARS: #5 @ 10.5 IN
BOTTOM BARS: (2) #6 (PER RIB)
WEIGHT: 1.24 PSF

TOP @ EXTERIOR: $A_s \approx 1/3\,(2 \times .44)\,12/36 = .097$
USE #3 @ 12" W/ STANDARD 90° END HOOKS

- INT. SPAN

TOP BARS: #5 @ 11 IN
BOTTOM BARS: (2) #5 (PER RIB)
WEIGHT: 1.18 PSF

A.5 BEAM DESIGN

BEAM DESIGN (TYPICAL FLOOR)

SERVICE LIVE LOAD = 100 PSF

SUPERIMPOSED DEAD LOAD = 22 PSF

SLAB WEIGHT = 0.74 CF/SF × 150 PSF = 111 PSF

TOTAL SERVICE DEAD LOAD = 133 PSF

FACTORED LIVE LOAD = 1.6 (100) = 160 PSF

FACTORED DEAD LOAD = 1.2 (133) = 159.6 PSF

TOTAL LOAD = 160 + 159.6 = 320 PSF

$M_U = w_u l_N^2/12 = 320 \times \left(\frac{35+40}{2}\right) \times (30-2)^2/12 = 784$ kip-ft

BEAM TRIAL SIZE

$\rho = .85(.85)(4/60)(.003/.007) = 0.0206$

$\phi M_N \geq M_U \Rightarrow .9(0.0206)(60)\,bd^2\left(1 - 0.59(.0206 \times 60)/4\right)$

$\geq 784 \times 12 \Longrightarrow bd^2 \geq 10343 \text{ IN}^3$

$d = 24.5'' - 2.5'' = 22''$

$b \geq 21.3$ TRY $24''$

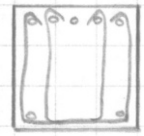
BEAM SELF WEIGHT

$1.2\left[24.5'' \times 24'' \times 150\text{PCF}/144\ \text{IN}^2/\text{SF}^2\right] = .735$ KLF

$W_u = \left(\frac{35+40}{2}\right) 320 \text{ PSF} - 111 \text{ PSF} \times 2 = 11.8$ KLF

$l_N = 30' - 2' = 28'$

INTERIOR BEAM

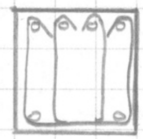


USE DESIGN 4 FROM CRSI

W = 11.9 > 11.8 KLF
STIRRUP = 155H ⇒ (15) #5 ; 1@2", 14@10"
PROVIDE 4 LEGS
TOP BARS = (5) #14
BOTTOM BARS = (2) #14

EXTERIOR BEAM

$W_u = (40/2)320$ PSF = 6.4 KLF



USE DESIGN 3 FROM CRSI

W = 8.2 > 6.4 KLF
STIRRUP = 174H ⇒ (17) #4 ; 1@2", 16@10"
PROVIDE 4 LEGS
TOP BARS = (4) #14
BOTTOM BARS = (2) #14

BEAM DESIGN (ROOF)

ROOF SNOW LOAD = 21 PSF

SUPERIMPOSED DEAD LOAD = 15 PSF
SLAB WEIGHT = 0.74 CF/SF × 150 = 111 PSF

TOTAL SERVICE DEAD LOAD = 126 PSF

FACTORED LIVE LOAD = 1.6 (21) = 33.6 PSF
FACTORED DEAD LOAD = 1.2 (126) = 151.2 PSF

TOTAL LOAD = 34 + 151 = 185 PSF

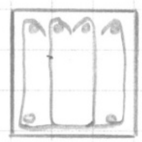
BEAM SELF WEIGHT

$1.2\left[24.5'' \times 24'' \times 150\ \text{PCF}/144\ \text{IN}^2/\text{SF}\right] = 0.735$ KLF

$W_U = \left(\frac{35+40}{2}\right) 185 - 111\ \text{PSF} \times 2 = 67.2$

$\ell_N = 30' - 2' = 28'$

INTERIOR BEAM

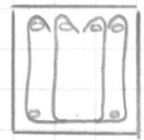


USE DESIGN 2 FROM CRSI

W = 7.2 > 6.4
STIRRUP = 133H ⇒ (13) #3 ; 1 @ 2", 12 @ 10"
PROVIDE 4 LEGS
TOP BARS = (4) #11
BOTTOM BARS = (2) #10

EXTERIOR BEAM

$W_U = (40/2)\,185 = 3.7$ KLF



USE DESIGN 1 FROM CRSI

$W = 4.7 > 3.7$ KLF

STIRRUP = 133 H ⇒ (13) #3 ; 1@2" , 12@10"

PROVIDE 4 LEGS

TOP BARS = (4) #9

BOTTOM BARS = (2) #10

BEAM DESIGN 2

INTERIOR

$$W_U = [1.2(22) + 1.2(113) + 1.6(100)] = 322 \text{ PSF}$$

BEAM TRIAL SIZE

$$M_U = w_u \ell_n^2/12 = 322(37.5)(28')^2/12 = 789 \text{ kip}\cdot\text{ft}$$

ASSUME $\phi = 0.9$ AND $\therefore \epsilon_t = 0.005$

$$\rho = 0.85 \beta_1 \frac{f'_c}{f_y} \frac{\epsilon_u}{\epsilon_u + 0.005} = 0.85 \times 0.85 \left(\frac{4}{60}\right)\left(\frac{0.003}{0.003+0.005}\right)$$

$$= 0.0181$$

$$M_U = \phi M_N = 789 \times 12 = 0.9 \times 0.0181 \times 60\, bd^2 \left(1 - 0.59 \frac{0.0181 \times 60}{4}\right)$$

$$bd^2 = 11534.6 \text{ IN}^2 \Rightarrow d = 22'' \Rightarrow b = 23.83 \approx 24''$$

BEAM SELF WEIGHT

$$1.2[24.5 \times 24 \times 145 / 144 \text{ IN}^2/\text{FT}^2] = 0.711 \text{ KLF}$$

$$W_U = 322 \times 37.5' - 113 \times 3' + 711 = 12447 \text{ KLF}$$

$$M_U = \frac{12447(28)^2}{12} = 813 \text{ kip}\cdot\text{ft} \times 12 = 9758 \text{ IN}\cdot\text{kip}$$

MINIMUM REINFORCEMENT AT ENDS

$$A_s = \rho bd = 0.0181(24)(24.5) = 10.64 \text{ IN}^2$$

$$a = \frac{10.64 \times 60}{.85(4)(24)} = 7.83$$

STEEL DESIGN

$As_{REQ} = 10.64''^2 \Rightarrow (7)\#11 = 10.92''^2$

$b_{MIN} = 2\times2.5 + 13\times1.41 = 23.33'' < 24''$ OK

MID-SPAN MOMENT

$M_u = w_u \ell_n^2/24 = \frac{12.447\,(28)^2}{24} = 406 \text{ kip}\cdot\text{ft} \times 12 =$

$= 4879 \text{ in-kips}$

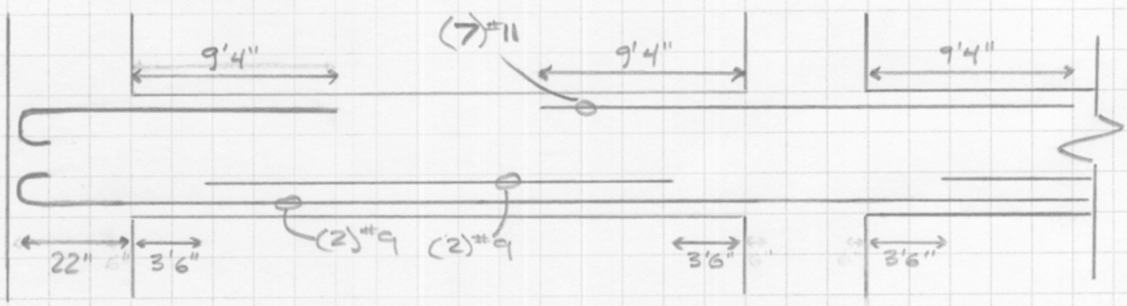
ASSUME $\phi = 0.9 \Rightarrow M_n = M_u/\phi = 4879/0.9 = 5421$

$R = \frac{M_n}{bd^2} = \frac{4879\times1000}{24''(22)^2} = 420.0 \text{ psi}$

FROM TBL A.5a $\Rightarrow \rho = 0.0075$

$As = \rho bd = 0.0075(24)(22) = 3.96 \text{ in}^2$

USE (4)#9



$\ell_{dh} = \left(\frac{0.02\, f_y}{\sqrt{f'_c}}\right) d_b = \left(\frac{0.02(60000)}{\sqrt{4000}}\right) 1.128 = 21.4'' \approx 22''$

EXTERIOR E-W

$W_u = (322 \text{ PSF} \times 20 \text{ ft}) + 1.2\,(50 \times 13 \text{ ft}) = 7220 \text{ PLF}$

BEAM TRIAL SIZE

$M_u = w_u \ell_n^2/12 = 7220 \times (30-2)^2/12 = 472 \text{ ft-kip}$

ASSUME $\phi = 0.9$ AND $\therefore \varepsilon_t = 0.005$

$\rho = 0.85\, \beta_1 \frac{f'_c}{f_y} \frac{\varepsilon_u}{\varepsilon_u + 0.005} = 0.0181$

$M_u = \phi M_N = 472 \times 12 = 0.9 \times 0.0181 \times 60\, bd^2 \left(1 - 0.59 \frac{0.0181 \times 60}{4}\right)$

$bd^2 = 6895 \text{ in}^2 \Rightarrow d = 22'' \Rightarrow b \geq 14.2$ USE 18''

BEAM SELF WEIGHT

$1.2\left[24.5 \times 18 \times 145 / 144 \text{ in}^2/\text{ft}^2\right] = 533 \text{ PLF}$

$W_u = 7220 - 113 \times 1.5 + 533 = 7584 \text{ PLF}$

$M_u = 7584\,(28)^2/12 = 495 \text{ FT-KIPS} \times 12 = 5945 \text{ IN-KIPS}$

MINIMUM REINFORCEMENT AT BEAM ENDS

$A_s = \rho bd = 0.0181\,(18)(22) = 7.17 \text{ in}^2$ TRY (6) #10

$b_{MIN} = 2 \times 1.5 + 2 \times .375 + 11 \times 1.27 = 17.72 < 18$ OK

$a = \frac{7.62 \times 60}{.85\,(4)(18)} = 7.47$

MIDSPAN MOMENT

$M_U = w_u l_n^2/24 = 7584(28)^2/24 = 248 \text{ FT-KIPS} \times 12$

$= 2973 \text{ IN-KIPS}$

ASSUME $\phi = 0.9 \Rightarrow M_N = M_U/\phi = 2973/0.9 = 3303 \text{ IN-KIPS}$

$R = \frac{M_N}{bd^2} = \frac{3303}{(18)(22)^2} = 379.2$

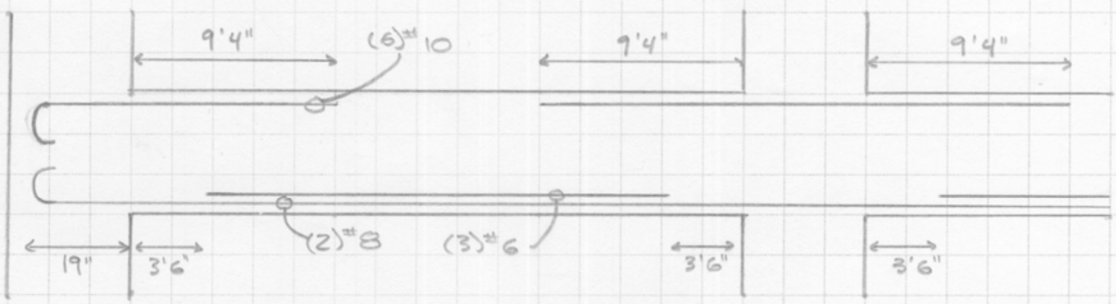
FROM TBL A.5a $\Rightarrow \rho = (0.007 - 0.0065)\left(\frac{379.2 - 368}{394 - 368}\right) + 0.0065$

$= 0.0067$

$A_s = \rho b d = 0.0067(18)(22) = 2.66 \text{ IN}^2$

TRY $(2)\#8 + (3)\#6 = 2(.79) + 3(.44) = 2.9 \text{ IN}^2$

$b_{MIN} = 2 \times 1.5 + 2 \times .375 + 2 \times 1.0 + 3 \times .75 + 4 \times 1.0 = 12$ OK



$l_{dh} = \left(\frac{0.02 f_y}{\sqrt{f'_c}}\right) d_b = \left(\frac{0.02(60000)}{\sqrt{4000}}\right) 1.0 = 18.97 \text{ IN} \approx 19"$

EXTERIOR N-S

$W_u = [1.2(50 \times 13) + 1.2(1.5 \times 4.5/12 \times 145) + 1.2(22 \times 1.5)$
$+ 1.6(100 \times 1.5)] = 1157$ KLF

BEAM TRIAL SIZE - ASSUME $\phi = 0.9$ AND $\therefore \varepsilon_t = 0.005$

$M_u = w_u \ell_n^2/8 = 1157(40-1.5)^2/8 = 214$ FT-KIPS $\times 12 = 2572$ IN-KIP

$\rho = 0.85 \times 0.85 \left(\frac{4}{60}\right)\left(\frac{0.003}{0.003+0.005}\right) = 0.0181$

$M_u = \phi M_N = 2572 = 0.9 \times 0.0181 \times 60\, bd^2 \left(1 - 0.59 \frac{0.0181 \times 60}{4}\right)$

$bd^2 = 3134$ IN^2 $\Rightarrow d = 22 \Rightarrow b = 6.48 \Rightarrow 7''$ SAME AS OTHER RIBS.

BEAM SELF WEIGHT

$1.2[7'' \times 20'' \times 145/144\ \text{IN}^2/\text{FT}^2] = 169$ KLF

$M_u = w_u \ell_n^2/8 = (1157+169)(38.5)^2/8 = 246 \times 12 = 2948$

$M_N = M_u/\phi = 2948/0.9 = 3276$ IN-KIPS

$R = M_N/bd^2 = 3276 \times 1000/7 \times (22)^2 = 966$ PSI

FROM TBL A.5a $\Rightarrow \rho = 0.0195$

$A_s = \rho bd = 0.0195(7 \times 22) = 3.00$ IN^2

USE 4 #8 IN TWO LAYERS $= A_s = 3.16$

$b_{MIN} = 2 \times 1.5 + 1.0 + 2 \times 1.0 = 6.0$ OK

A. 6 SHEAR CHECKS

SHEAR CHECK

PAN JOIST

$V_u \le \phi V_n$

$V_u = [1.2(22) + 1.2(113) + 1.6(100)]\left(\frac{40' - 2}{2}\right)(3')$

$= 174283$ lbs

$V_u \le \frac{1}{2}\phi(2\sqrt{f'_c}\ b_w d) = \frac{1}{2}(0.75)\left[2\sqrt{4000}\,(7''(22))\right]$

$V_u = 17$ kips $\not\le \frac{1}{2}\phi V_c = 7.3$ kips $\therefore$ TAPER ENDS

$b_w d = \frac{V_u}{\phi\sqrt{f'_c}} = \frac{174283}{.75\sqrt{4000}} = 367.4\ \text{in}^2$

$367.4/d = 367.4/23 = 15.9'' \approx 16''$ TAPERED WIDTH OF RIBS.

BEAMS

INTERIOR

$V_u = [1.2(22) + 1.2(113) + 1.6(100)]\left[\left(\frac{30-2}{2}\right) - \frac{22}{12}\right]\left(\frac{40+35}{2}\right)$

$= 147$ kips

$V_u \le \frac{1}{2}\phi(2\sqrt{f'_c}\ b_w d) = \frac{1}{2}(0.75)\left[2\sqrt{4000}\,(24'')(22'')\right]$

$V_u = 147$ kips $\not\le \frac{1}{2}\phi V_c = 25$ kips

SHEAR REINFORCEMENT IS NEEDED, TRY (2) #4 VERT "U"s

$12.17\left(\frac{147-25}{147}\right) = 12'$ FROM FACE OF SUPPORT

NO REINFORCEMENT REQ.

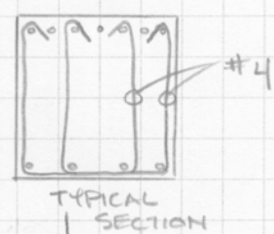
$$S_{MAX} = \min \begin{cases} A_v f_y / 0.75\sqrt{f'_c}\, b_w = \dfrac{4(.2)60000}{0.75(\sqrt{4000})(24)} = 42'' \le \dfrac{4(.2)(60000)}{50(24)} = 4 \\ \dfrac{d}{2} = 22/2 = 11 \leftarrow \\ 24 \text{ in} \end{cases}$$

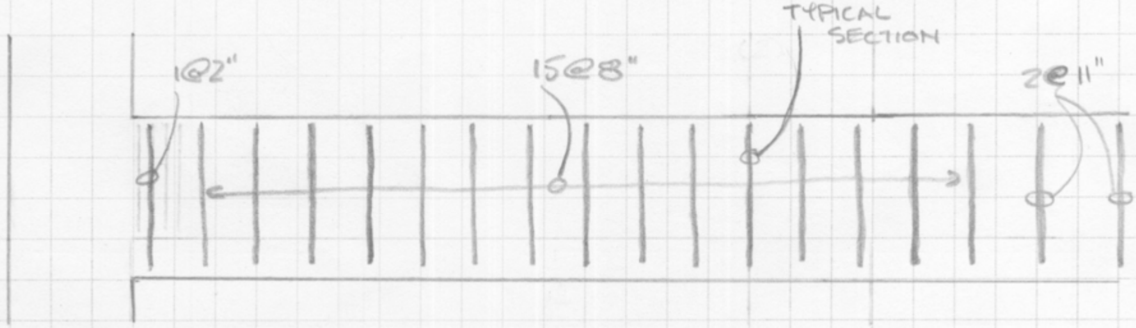
$$S = \frac{\phi A_v f_y d}{V_u - \phi V_c} = \frac{0.75(.8)(60000)(22)}{(147-50)\times 1000} = 8.2'' \Rightarrow 8''$$

$$\frac{\phi A_v f_y d}{s} = \frac{0.75(.8)(60000)(22)}{11''} = 72000 \text{ lb}$$

$$12.17\left(\frac{147-50}{147}\right) + 22/12 = 10'$$

1 SPACE @ 2"
15 SPACES @ 8"
2 SPACES @ 11"





BEAM
EXTERIOR E-W

$$V_u = W_u \times \frac{l_N}{2} - d = 7584 \times \left(\frac{28'}{2} - 22/12\right) = 92.2 \text{ kips}$$

$$V_u \le 1/2\, \phi\left(2\sqrt{f'_c}\, b_w d\right) = 1/2(0.75)\left[2\sqrt{4000}(18'')(22'')\right] = 18.8 \text{ kip}$$

$V_u \nless \frac{1}{2} \phi V_c = 18.8$ kips

∴ REINFORCEMENT IS NEEDED TRY (2) #3 VERT. 'U'S

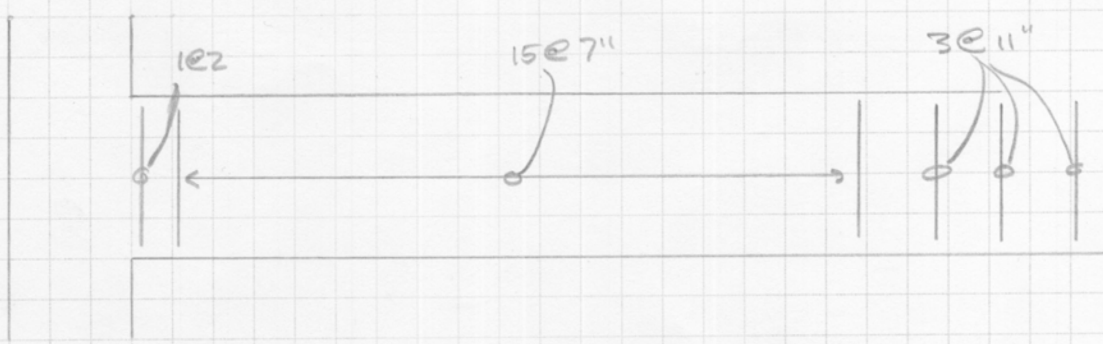
$12.17\left(\frac{92.2-18.8}{92.2}\right) = 9.7'$ FROM THE CRITICAL SECTION
NO REINF. REQ.

$12.17\left(\frac{92.2-37.6}{92.2}\right) = 7.2'$ FROM THE CRITICAL SECTION
MIN. REINF. REQ.

$$S_{MAX} = \min \begin{cases} A_v f_y / 0.75\sqrt{f'_c}\, b_w = \frac{4(.11)60000}{0.75(\sqrt{4000})(18)} = 30.9 \nless \frac{4(.11)(60000)}{50(18)} = 29.3 \\ d/2 = 22/2 = 11 \leftarrow \text{CONTROLS} \\ 24_{IN} \end{cases}$$

$$S = \frac{\phi A_v f_y d}{V_u - \phi V_c} = \frac{0.75(.44)(60000)(22)}{(92.2-37.6)(1000)} = 7.97 \Rightarrow 7''$$

1 SPACE @ 2"
15 SPACES @ 7"
3 SPACES @ 11"



A.7 COLUMN SPOT CHECK

EXTERIOR COLUMNS

TRIBUTARY AREA : $(20+1.5)(30) = 645\ ft^2$

LIVE LOAD (LIVE LOAD REDUCTION $.40\ L_o$)

$645 \times 21 + (5)(40)(645) = 143$ kips

DEAD LOAD

$645 \times 15 + (5)(17)(645) = 65$ kips

SELF WEIGHT

$$\left[\underbrace{113 \times (645 - 30 \times 3)}_{\text{SLAB}} + \underbrace{940\ (30')}_{\text{BEAM}} + \underbrace{11'(450)}_{\text{COLUMN}}\right] 5$$

$= 480$ kips

TOTAL DEAD LOAD

$480 + 65 = 545$ kips

CONTROLLING COMBINATION

1.2 DEAD + 1.6 LIVE

$1.2(545) + 1.6(143) = 883$ kips $= P_u$

$P_{MAX} = 1175$ kips $> P_u = 883$ kips <u>OK</u>

INTERIOR COLUMNS

TRIBUTARY AREA: $(20 + 17.5)(30) = 1125 \text{ ft}^2$

LIVE LOAD

$1125 \times 21 + (5)(100)(1125) = 586 \text{ kips}$

DEAD LOAD

$1125 \times 15 + (5)(17)(1125) = 113 \text{ kips}$

SELF WEIGHT

$$\Big[\underbrace{113 \times (1125 - 30 \times 4)}_{\text{SLAB}} + \underbrace{(1225 \text{ PLF})(30')}_{\text{BEAM}} + \underbrace{(11')(450)}_{\text{COLUMN}}\Big](5)$$

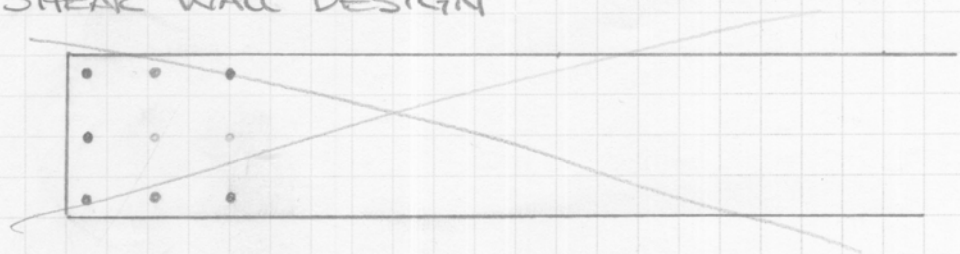
$= 776 \text{ kips}$

TOTAL DEAD

$113 + 776 = 889 \text{ kips}$

A.8 SHEAR WALL DESIGN

SHEAR WALL DESIGN



NOMINAL SHEAR CAPACITY

$$V_N \le 8 A_{cv}\sqrt{f'_c} = 8(12'')(240'')\sqrt{5500}\left(\frac{1\ \text{kip}}{1000\ \text{lbs}}\right)$$

$$\le 1708.69\ \text{kips}$$

$$\phi V_N = 0.6(1708.69) = 1025.22 > 187.16\ \text{kips}$$

LONG + TRANS REINFORCE

$$V_N \ge 2A_{cv}\sqrt{f'_c} = 1/4(1708.69) = 427.17\ \text{kips}$$

$$V_U = 187.16 < V_N = 592.5 \quad \text{OK}$$

REQUIRED $\rho_l = 0.0015$ $\rho_h = 0.0025$

$$A_{sv} = 0.0015(12)(12) = 0.216\ \text{in}^2/\text{ft}$$

#5 @ 16"

$$A_{sh} = 0.0025(12)(12) = 0.36\ \text{in}^2/\text{ft}$$

#5 @ 10"

CHECK NEED FOR BOUNDARY ELEMENT

$$P_{u_{BE}} = \tfrac{1}{2}P + M_u\left(\frac{1}{\ell_w - b/2}\right)$$

$$= \tfrac{1}{2}(234) + 7200\left(\frac{1}{20' - 24/2}\right) = 495.9$$

$$A_g = 20\ ft^2$$

$$I_g = \tfrac{1}{2}(1')(20')^3 = 666.67$$

$$f_c = P_u/A_g + M_u(h_w/2)/I_g =$$

$$= 495.9/20 + \left[7200(20/2)/666.67\right]\left(\frac{1}{144}\right) =$$

$$= 25.5 > 0.2 f'_c = 1.1\ ksi$$

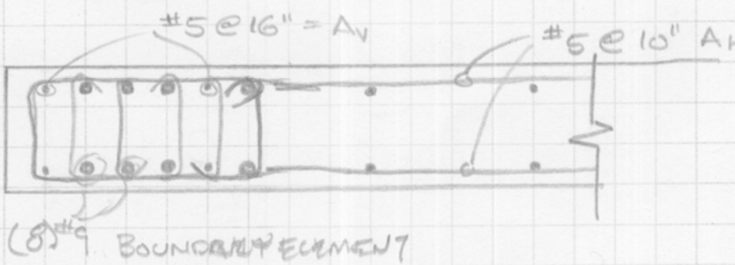
- BOUNDRY ELEMENT CAPACITY

$$\phi P_n > P_{u_{BE}} = 495.9$$

$$0.9(60)(A_{st}) > 495.5$$

$$54 A_{st} > 495.5$$

$$A_{st} \geq 9.1 \qquad (4)\#5 + (8)\#9$$



A.9 BUILDING COST ESTIMATE

RS MEANS COST ESTIMATE

ORIGINAL BUILDING

- 6 STORY, 13 FOOT STORY
- PRECAST CONCRETE FACADE
- STEEL STRUCTURAL SYSTEM
- NORRISTOWN, PA
- FLOOR AREA = 212,188 SQ.FT.
- PERIMETER = 5172 FT.

COST PER SQ.FT. = $247.22

REDESIGNED BUILDING

- 6 STORY, 13 FOOT STORY
- PRECAST CONCRETE FACADE
- REINFORCED CONCRETE STRUCTURAL SYSTEM
- NORRISTOWN, PA
- FLOOR AREA = 212,188 SQ.FT.
- PERIMETER = 5172 FT.

COST PER SQ.FT = $249.54

TOTAL COSTS

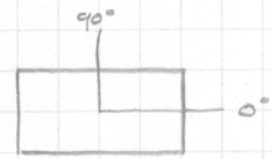
ORIGINAL	REDESIGNE
$52,457,500	$52,949,000

A.10 LIGHTING CALCULATIONS

LIGHTING DESIGN

2 GENERAL FIXTURES

RECESSED DOWNLIGHT
- COOPER LIGHTING - C7042-740061-42WPLT
- USE IN KITCHEN, LOBBY, WALKWAYS
- SPACING CRITERIA 0.96



RECESSED TROFFER
- LIGHTOLIER - QVL2GPFS5432
- USE IN ALL OTHER COMMON SPACES
- SPACING CRITERIA ⇒ 0° : 1.24 90° : 1.32

APPROXIMATE SPACING

WORK SURFACE 2.5' OFF FLOOR
DISTANCE TO WORK SURFACE 10' − 2.5' = 7.5'

COOPER
0.96 × 7.5' = 7.2'
USE 6'

LIGHTOLIER
1.24 × 7.5' = 9.3'
USE 8'

1.32 × 7.5' = 9.9'
USE 10'

REQUIRED ILLUMINANCE

OFFICE - HEAVY VDT USE ⇒ ILLUMINANCE CATAGORY = D
REQUIRED ILLUMINANCE = 30 Fc

REDUCTION IN POWER USE FROM DAYLIGHTING

NUMBER OF UNNECESSARY LUMINARIES

WINTER SOLSTICE

SOUTHERN SIDE = 41
NORTHERN SIDE = 18

TOTAL = 59

SUMMER SOLSTICE

SOUTHERN SIDE = 25
NORTHERN SIDE = 30

TOTAL = 55

FULL YEAR AVERAGE = 57 FIXTURES

OVERCAST DAYS

SOUTHERN SIDE = 13
NORTHERN SIDE = 14

TOTAL = 27

ACCORDING TO THE NATIONAL CLIMATE DATA CENTER
PHILADELPHIA HAS 53% SUNNY WEATHER

TOTAL UNUSED FIXTURES = $[365 \times .53 \times 57 + 365 \times .47 \times 27]$
= 15,659 FIXTURES

TOTAL POWER SAVINGS = $15{,}659 \times 8\text{ hr} \times 108$ WATTS
= 13,529 KILOWATT-HOURS

A.11 STORY DISPLACEMENTS

Story Displacements

RAM Frame v11.2
DataBase: Second Try
Building Code: IBC

04/08/08 15:58:23

CRITERIA:

Rigid End Zones: Ignore Effects
Member Force Output: At Face of Joint
P-Delta: Yes Scale Factor: 1.00
Ground Level: Base
Wall Mesh Criteria :
Wall Element Type : Shell Element with No Out-of-Plane Stiffness
Max. Allowed Distance between Nodes (ft) : 8.00

LOAD CASE DEFINITIONS:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
E1	Seismic	EQ_IBC06_X_+E_F
E2	Seismic	EQ_IBC06_X_-E_F
E3	Seismic	EQ_IBC06_Y_+E_F
E4	Seismic	EQ_IBC06_Y_-E_F
W1	Wind	Wind_IBC06_1_X
W2	Wind	Wind_IBC06_1_Y
W3	Wind	Wind_IBC06_2_X+E
W4	Wind	Wind_IBC06_2_X-E
W5	Wind	Wind_IBC06_2_Y+E
W6	Wind	Wind_IBC06_2_Y-E
W7	Wind	Wind_IBC06_3_X+Y
W8	Wind	Wind_IBC06_3_X-Y
W9	Wind	Wind_IBC06_4_X+Y_CW
W10	Wind	Wind_IBC06_4_X+Y_CCW
W11	Wind	Wind_IBC06_4_X-Y_CW
W12	Wind	Wind_IBC06_4_X-Y_CCW

Level: Roof, Diaph: 1

Center of Mass (ft): (149.52, 64.84)

LdC	Disp X in	Disp Y in	Theta Z rad
D	-0.00365	0.00021	-0.00000
Lp	-0.00537	0.00051	-0.00000
E1	1.17084	-0.00211	0.00006
E2	1.19742	-0.00743	0.00029
E3	0.02585	2.15831	0.00022
E4	-0.03393	2.17029	-0.00028
W1	0.37651	-0.00149	0.00004
W2	-0.00312	1.70393	-0.00002
W3	0.27435	0.00072	-0.00003
W4	0.29042	-0.00297	0.00010
W5	0.04230	1.26758	0.00036
W6	-0.04698	1.28832	-0.00039
W7	0.28005	1.27683	0.00002