WESTINGHOUSE ELECTRIC COMPANY CORPORATE HEADQUARTERS

CRANBERRY, PA

Final Report

Spring 2009

B.A.E.

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Structural Option

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Westinghouse Electric Company Corporate Headquarters Building One Cranberry, PA

Building Information

Occupancy:	Office
Size:	434,800 SQ FT
Height:	5 Levels, 87'-6" Tall
Build Time:	Feb 2008-May 2009
Cost:	$80 Million
Delivery:	Design-Bid-Build

Project Team

Owner:	Wells Real Estate Funds
Architect:	IKM, Inc.
Structural/mech:	LLI Engineering
Civil:	Civil & Environmental Consultants, Inc.
Construction:	Turner Construction Company

Structural

Structural Steel Framing With 2" Composite Steel Deck And 2-1/2" Concrete Slab

Typical Bay Size Is 45'-0"

Moment Connections Resist Wind Forces

Slab On Grade, Grade Beams, And Cassion Foundation System

Mechanical/Electrical/Lighting

Two Gas Fired Boilers Serve To Heat The Building With Capacities Of 1265 CFM And 960 CFM.

Six AHU's With Varying Capacity Serve The Building And VAV Units.

480/277V, 3 Phase, 3 Wire Primary, 208/120V 3 Phase 4 Wire Secondary Delta-Wye Dry Type Transformers.

Mainly Flourescent Lighting To Contribute To LEED Certification.

Generator Protection On Campus For All Controls.

Architecture

Site Is On 83 Acres In Cranberry, PA

LEED Certification Goal

Bio-retention Parking Lot

Brick Façade Is Textured To Create Vertical Element While Polished Concrete Block Emphasizes Imporantance.

Powerful Entrance Makes Use Of Two-Story Atrium

Floor-To-Floor Height Of 14'-0", Entrance Level Is 18'-0".

Roof System Consists Of An EDPM System With A Membrane Over 1/2" Protection Board Over Tapered Insulation Over 5/8" GWB On The Roof Decking.

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Sponsored By Turner Construction Company
www.engr.psu.edu/ae/thesis/portfolios/2009/jll5004/

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EXECUTIVE SUMMARY

The Westinghouse Electric Company Corporate Headquarters will be a three-building campus with site features which include asphalt walking paths and volleyball courts on eighty-three acres in Cranberry, PA. For the purpose of the project, only Building One will be analyzed as the other two are considered a separate project by all parties involved. The truncated V-shape building has been given a look of importance with polished concrete block merging into brick stepped-out columns to accentuate the verticality of the five-story 74'-6" tall structure.

The purpose of this report is to redesign the structural system of the Westinghouse Electric Company Corporate Headquarters Building One using reinforced cast-in-place concrete and a one-way slab with beams floor system. The building was analyzed in concrete by hand and in the RAM Structural System program. The success of this part of the report relies on the implementation of the code effectively and correctly to determine if the proposed modifications could be implemented.

For this report, a detailed analysis of the alternative structural system was performed. In order for this method to be correct, all structural members were designed according to ACI 318-08 and ASCE 7-05 for gravity loads, lateral loads, and torsion. Hand calculations were done for spot checked members in addition to a RAM Structural System model and analysis. The new structural system consists of typical square columns 24"x24" and beams typically 24"x34" with a 10" thick one-way slab. The spread footings and caissons were also spot checked and updated as necessary for the new structural dead load. Uplift and overturning moment were considered and checked for this report, but due to the weight of the building, neither was determined to be an issue.

Since the building material was changed, it is necessary to compare the new building cost estimate and schedule to the as-built structure's cost budget and schedule. The new building was determined to be $30.60/SF without a green roof and $33.28/SF with a green roof, while the original design cost is $30.90/SF. Also, it takes two months longer for the new concrete structure to be erected compared to the original steel structure. Despite the fact that the lead time for steel is much longer than concrete, most of the steel will be on site by the time the foundations are complete, so the lead time did not affect the schedule. While the goal of the project was to obtain a cost and schedule for the new building so a comparison could be made, it can clearly be see that the concrete structure is not the best alternative for this building.

The sustainable architecture study was an attempt to make the corporate headquarters stand out among headquarters buildings by being incorporated into the environment. A green roof was added, and the extra load of the soils and supporting structure was determined and evaluated with the entire building. The green roof was designed for the third floor area above what will be the employee cafeteria. This part of the building also conveniently faces the south, which is the optimum direction for a successful green roof. The area will be extremely beneficial to the company by its multiple purposes, whether it is as a lunch area, a break room, or an informal meeting location. The waterproofing, drainage system including pipe sizes, detail of the materials, specification of materials and plants acceptable for the green roof were all determined. A LEED analysis was performed for the new building also, since one of the goals of the owners was to have a LEED certified building. It was determined that it is possible for the building to be LEED silver rated, but would require further information and investigation to be rated higher.

Overall, the project was a success, even though it was not erected cheaper or faster than the original steel building. It is feasible to build the building in concrete, but it is not an effective alternative. It is recommended to add a green roof to the structure to emphasize the corporate headquarters aspect of the building and to incorporate it into the environment.

ACKNOWLEDGEMENTS

I would like to thank:

- Turner Construction Company for their assistance and support in completing my project and providing supplies for this project. Special thanks to Bob Hennessey for his time and efforts to help me with my questions throughout the year, providing the schedule and estimate, and also taking the time to show me the site.

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- Wells Real Estate Funds, particularly Frank Mitzel for permission to study the Westinghouse Electric Company Corporate Headquarters Building One.

- The Pennsylvania State University Architectural Engineering Department and Staff for teaching us the skills necessary to become the best engineers possible and their advice and help throughout the past five years. In particular, Dr. Linda Hanagan, my thesis advisor, for her assistance and feedback throughout the year, and Prof. Kevin Parfitt and Prof. Robert Holland for teaching the class.

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INTRODUCTION

WESTINGHOUSE ELECTRIC COMPANY CORPORATE HEADQUARTERS BUILDING ONE

The Corporate Headquarters building for the Westinghouse Electric Company is located in Cranberry, Pennsylvania. Just north of the city in Butler County, the site is on 83 acres in an office park easily accessible by I-79 and PA-228. With five above grade floors and a full 17' high basement, Building One will be the main building on this campus. Complete with cafeteria, gym, locker rooms, offices, and executive conference rooms, the flagship building comes well equipped and diverse. At 434,800 square feet, the building makes quite an architectural statement.

The main building utilizes a powerful entrance with a two-story atrium to express its importance. The first floor also has a height of 18'-0" to emphasize a larger space while floors two through four have floor-to-floor heights of 14'-0". The fifth floor has a height of 14'-6". Building One has a 74'-6" above grade with an 18' penthouse, making the final height 92'-6". Aluminum and glass curtain walls add light and make the building feel more open while polished concrete at the base of the brick façade accentuate the height. The foundation system consists of caissons in addition to some spread footings and grade beams. A typical bay is 45'-0" by 24'-0", and uses a steel system with composite beams and deck. In most of the building, the girders are not composite, but the beams framing into the girders have some composite action. The floor system is a 2" 22 gage steel deck with 2-1/2" of lightweight concrete topping. The Westinghouse Electric Company Corporate Headquarters Building One has two expansion joints present, thus creating essentially three structural buildings inside of one. The expansion joints create the East, Center, and West parts of the building. These joints can be seen along column lines 7.9 and 8 between the east and center portions, and column lines 21 and 21.1 between the center and west parts of the building.

A successful redesign of this building will be completed and checked using a computer program, such as RAM Structural System, following the design procedure laid out by ACI 318-08 and ASCE 7-05, and will be constructible. The design will consist of gravity design of member, wind load calculations, seismic load calculation, torsion member checks, resizing of foundations, and uplift and overturning moment. Any changes will be evaluated in terms of cost and schedule implications and be compared to original values for both obtained from the Turner Construction Company. The construction management portion will compare these values. Ideally, the building will be built faster or less expensively than the original, but this is not a main point in the success or failure of this portion. Finally, the redesign will be a success if the building can be further integrated into the environment while providing details and specifications.

SITE PLAN

Figure 1: Site boxed in red and the road leading up to the site highlighted in red

Third floor plan- East with portal analysis Frame 2 and spot checked columns highlighted.

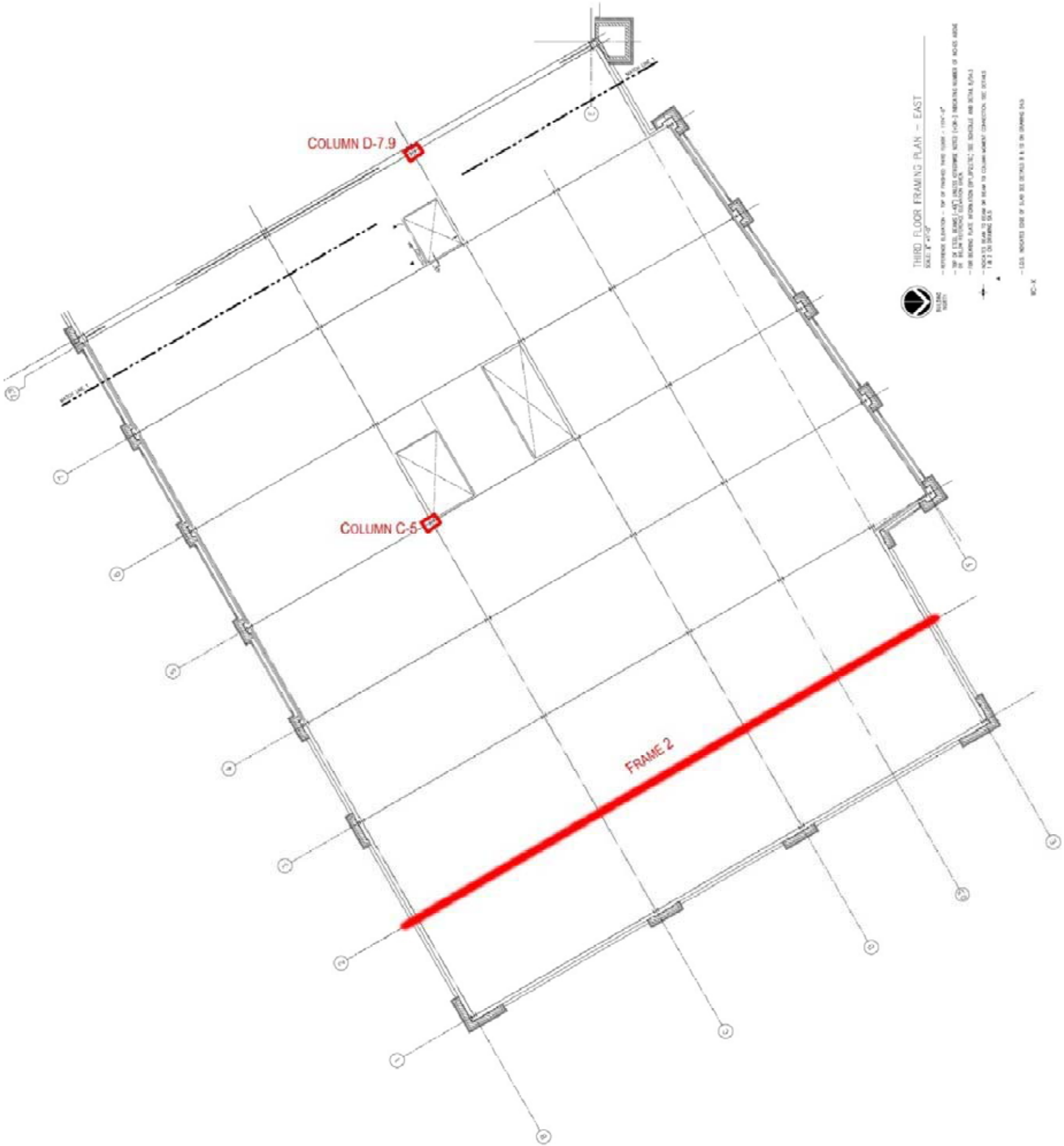

Figure 2: Third Floor Plan East

Third Floor Plan Center with portal analysis Frame 13, interior beam designed, lateral member C.2-D.2- 13 checked, and spot checked columns B-15 and A-15 highlighted

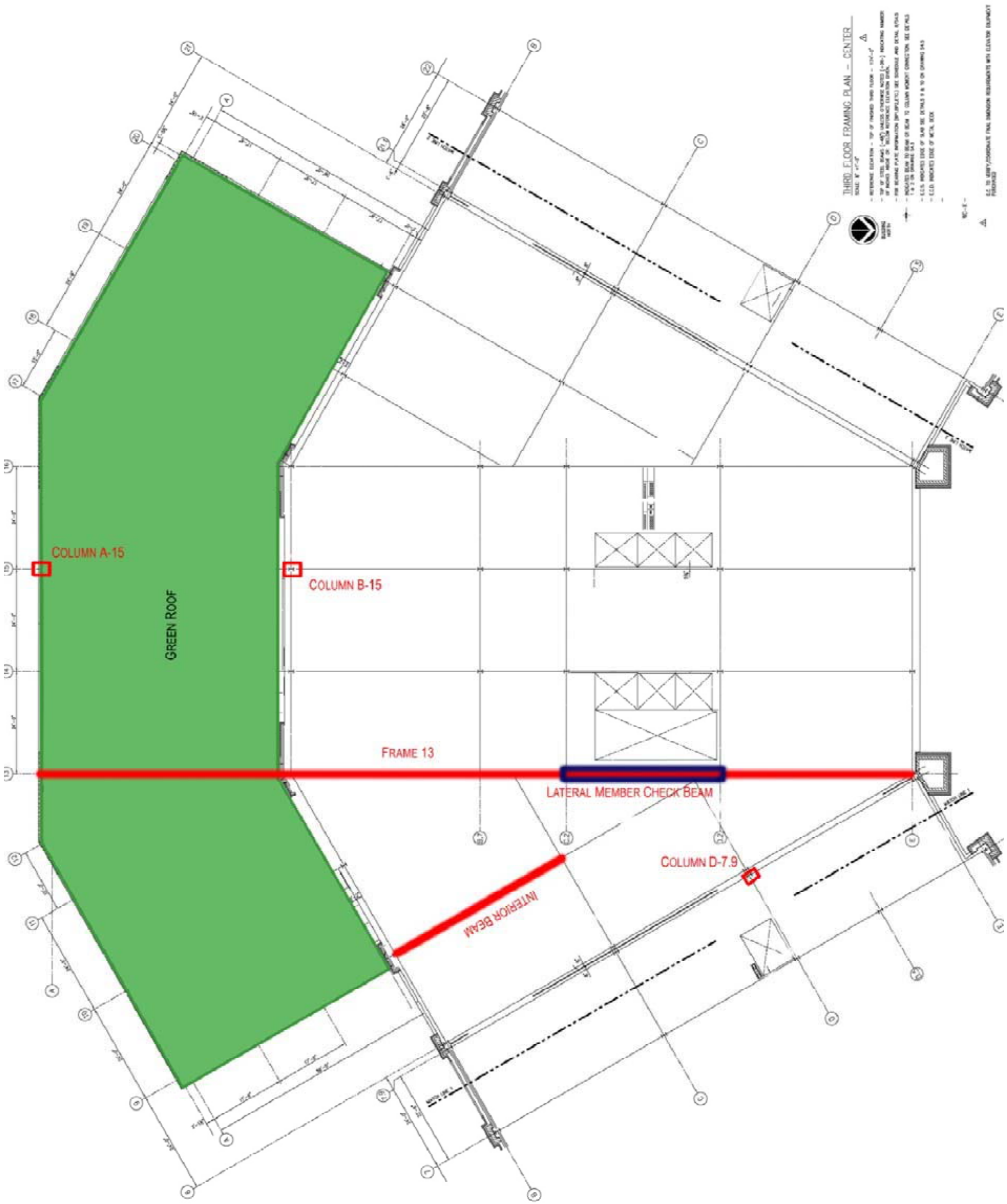

Figure 3: Third Floor Plan Center

Second Floor Plan Center of as-built design with frames indicated

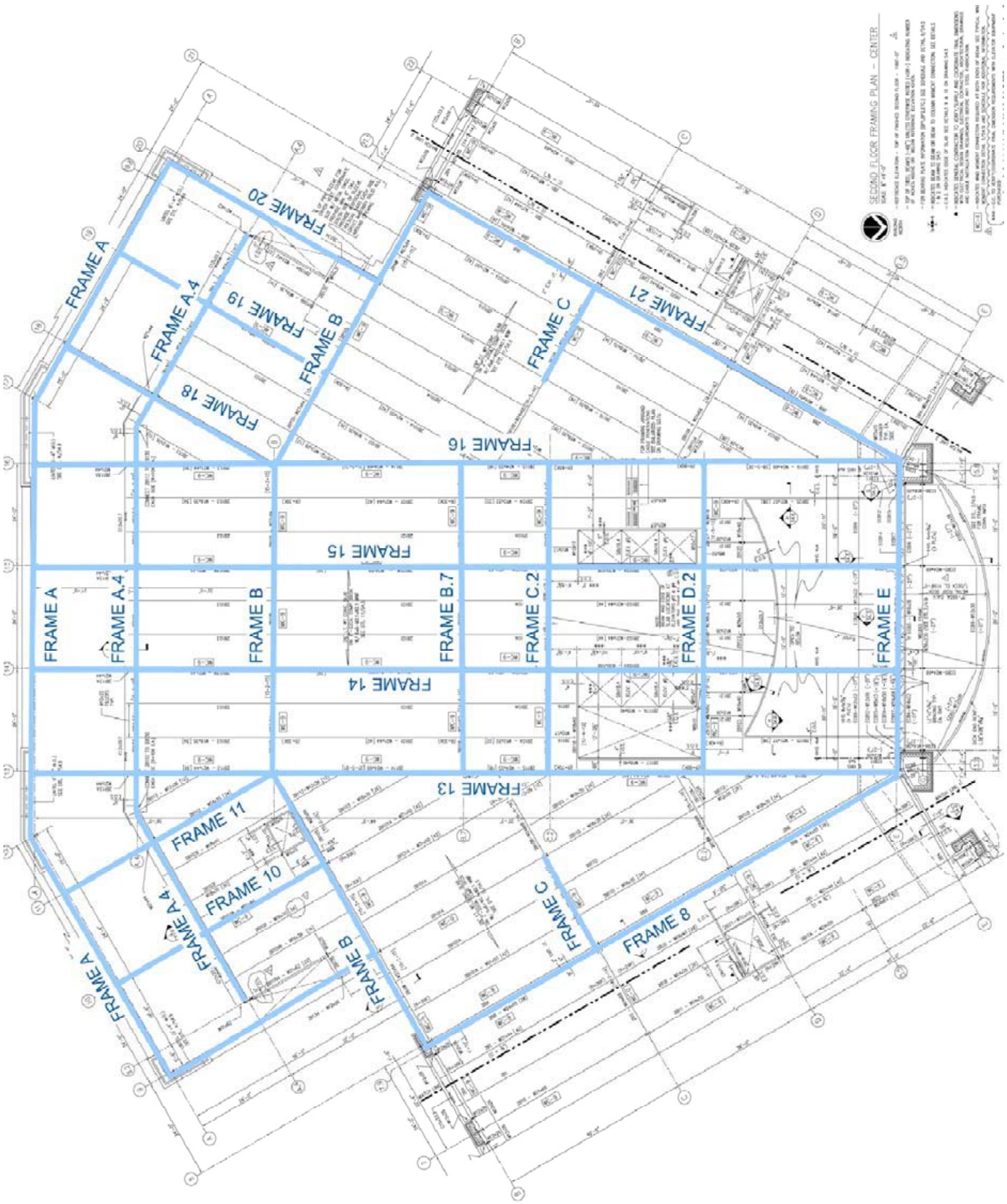

Figure 4: Second Floor Plan Center As-Built

EXISTING STRUCTURAL SYSTEMS

FOUNDATIONS

Sixty-one caissons are the main elements in the foundation system. Each was designed to carry 8,000 psf. The caissons range from 36" to 84" in diameter and from 8'-0" to 30'-8" in height. On top of each caisson, there is a 2'-6" cap with #6 @8" each way on the top and the bottom as well as base plates for the columns. The 5" slab on grade in the basement bears directly on the soil and the thickened slabs under the non-load bearing walls. On the south side and the east portion of the building, where caissons are not present, there are spread footings or grade beams. The sub-grade walls in the basement (referred to as grade beams in the drawings) range from 1'-4" to 1'-8" wide and are 14'-4" deep. The bottom reinforcement in the grade beams is mainly (3) #6, but varies from #6 to #9 and in number. Top reinforcement also varies from #6 to #9 and from two bars to four bars. All end reinforcing bars are #6, but vary from two bars to four bars.

FLOOR AND ROOF SYSTEM

The floor system for the corporate headquarters main building consists of 2" 22 gage metal deck with 2 ½" lightweight concrete topping, for a total slab depth of 4 ½". The typical bay size of this composite steel system is 24'-0" by 45'-0". W21 beams (W21x44 typ.) spaced 24'-0" on center and W18x35 beams spaced 8'-0" on center support the deck and transfer the load to the W24 girders (W24x55 typ.). The girders then continue to transfer the load to the columns. The 5" thick slab-on-grade in the basement of the headquarters is the exception to the typical floors. The roof uses a different system consisting of 1 ½" 20 gage roof deck, steel beams and steel K series joists. However, the penthouse system uses 2" 20 gage metal deck with a 2 ½" lightweight concrete topping. Where the penthouse is absent, roof uses a fully adhered EPDM roofing system including the membrane over ½" protection board over tapered insulation over 5/8" type X GWB over the roof decking.

LATERAL SYSTEM

The Westinghouse Corporate Headquarters Building One uses moment connections at every column to resist lateral loads from wind and seismic forces and torsion forces. Wind moment connections with angles and bolts are provided at all members in the lateral system of the building.

COLUMNS

The columns used in the headquarters are typical for a mid-rise building. The large columns in the basement and first floor of the building are W36x230 at the largest, but typically are W14x90. The W36x230 columns are larger because the entire front façade of the building is bearing on a W36x230 beam and the two columns. On the roof, any columns that do not continue up from the fifth floor are W10x49 or W10x33. The rest of the building is generally the same size, of course with some smaller sizes of columns, such as W10's on the fifth and roof levels. The base plates have four possible layouts and range in thickness from 1 ¾" to 3".

PROBLEM STATEMENT

Based on the analysis performed on the Westinghouse Electric Company Corporate Headquarters Building, it can be concluded that the original composite deck and beam system is well suited for time and space considerations. In depth calculations and comparisons can be seen in Technical Report 2. However, with wind moment connections at every column, the lateral system could be explored further for efficiency. The size of the typical bays is fairly large and leads to larger beam sizes to keep the deflection reasonable. A one-way reinforced cast-in-place concrete slab with beams would be the best way to approach the 2:1 bays.

The building owners have decided to make the new corporate headquarters a LEED certified building. A study on the feasibility of making the building Silver Rated instead would be desirable and beneficial to the project. With a building and campus so large, integrating the site into the building is a must.

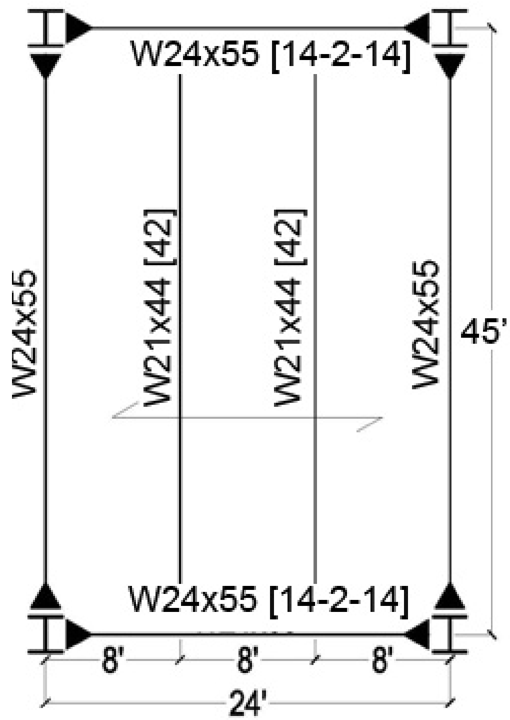

Figure 5: As-Built Typical Bay Framing

With so many changes in regard o the structure of the building, it would be beneficial to the project to perform a cost estimate for the new design and to generate a schedule. These were done in an effort to compare and evaluate the as-built design and the new redesign on a more even level.

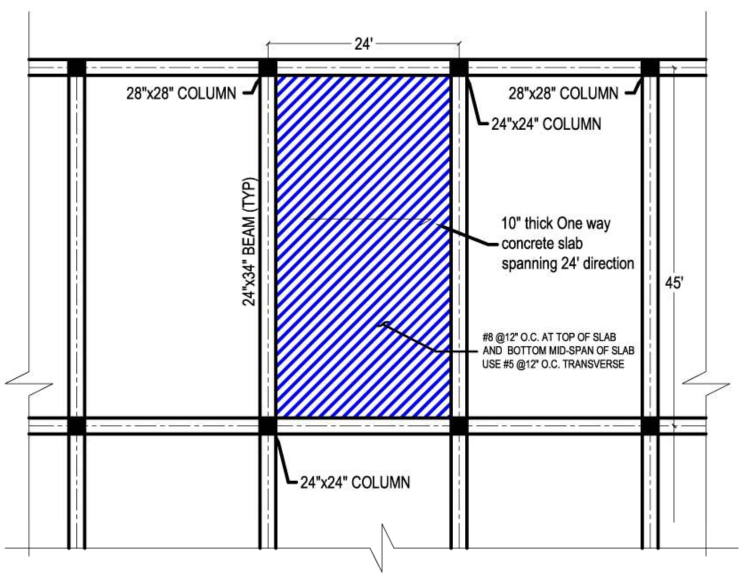

Figure 6: New Design Concrete Typical Framing

SOLUTION METHOD

The building will be redesigned for concrete with one-way reinforced concrete floors with beams. With the current column layout, the one-way slab has been shown to be more efficient than the two-way slab. Concrete moment resisting frames will be considered for the lateral system. Shear walls would have been an option, but without tenant fit-out drawings and a request for an open plan, they could not be the main system. In addition to changing the building to a concrete system, a green roof will be added to bring the building closer to the campus and its surroundings. Since the building is changed to concrete, the foundations will have to be re-examined and resized for the new loads. The building will be designed using a combination of hand calculations with ACI 318-08, IBC 2006, and a RAM model for verification of design. The project will be considered a success if it physically can be built and uses a design following all applicable codes. Also, it will be a success if the number of moment frames can be reduced.

In order to fully gauge which system is more effective overall, the steel and concrete buildings must be compared. Since the material is changing, there will be cost implications that need to be considered. Also, the difference in materials means there is a difference in erection time as well. To be able to make an assessment of the redesigned concrete system, a cost estimate and a schedule will be generated. The estimate will be compared to Turner Construction Company's budget for the building in steel, and the generated schedule will be compared to their actual schedule also. The building is currently under construction, but the structure was finished according to the schedule. Since the building owner wants it to be LEED certified, a LEED analysis of the new structure is required. A green roof was added to the building to integrate it into the surrounding land and to make the building unique as a corporate headquarters in Pittsburgh. The green roof also has structural implications which need to be addressed as well as cost and schedule impact. The potential plant inhabitants, waterproofing, and drainage system including pipes required to drain the water from the roof need to be evaluated. Achieving a LEED Silver Rating would be ideal, but ensuring the building still is capable of being rated would be acceptable. This portion of the project will be considered a success if a green roof can and is properly integrated into the building with proper drainage and detailing, and if a cost estimate can be calculated and a projected schedule can be generated. Ideally, the ultimate goal would be if the project could be completed faster or less expensively than the original steel building. However, the success of this project does not hinge entirely on obtaining the ideal goal.

Code and design requirements

These are the design standards, codes, and design criteria used by the design professional and in the calculations for this report.

Applicable design standards

The 2006 International Building Code

ACI 318-05 (Reinforced Concrete Design)

AISC Steel Construction Manual, 13th Edition

ACI 530 (Masonry structures)

ASCE 7-05 (Minimum design loads for buildings and other structures)

Deflection Criteria

Floor Deflection Criteria

L/240 Total Load

L/360 Live Load

L/600 Curtain Wall Load

Lateral Deflection Criteria

H/400 Total Allowable Wind Drift

H/400 Story Wind Drift

H/50 Total Allowable Seismic Drift ($\Delta=0.02H_{sx}$ From Table 12.12-1 ASCE 7-05)

Materials

The materials used in the Westinghouse Electric Company Corporate Headquarters as listed on the general notes page of the structural drawing set are as follows and were used in design and analysis as appropriate.

Concrete

Freezing Temperature Exposure	Air entrained (6% ±1%)
Slab-on-grade	4,000 PSI
Slab-on-deck	4,000 PSI
Caissons	3,000 PSI
Footings and Caisson Caps	3,000 PSI
Walls and Piers	4,000 PSI
Over excavation fill	2,000 PSI

Reinforcing Steel

Reinforcing Bar	ASTM A-615
Welded Wire Fabric	ASTM A-185

Structural Steel

W-Shapes	ASTM A-992
C-Shapes	ASTM A-36
Steel Pipe	ASTM A-501
Tubes	ASTM A-500 Grade B

Metal Deck

Bolts	ASTM A-325, ¾" diameter
Deck	ASTM A611 Grade C or D
Studs	¾"x 3 ½" headed stud

Masonry

CMU	ASTM C-90
Concrete Brick	ASTM C-55 type N-1
Mortar	ASTM C-270
Grout	ASTM C-476

Gravity and Lateral Loads

The loads on the building are applied as such based on the design professional's specification on the drawings. It is understood the values for the original building are conservative since the live load of 80 PSF was used everywhere on the upper floors and a partition load is also used. The loading on the new redesigned concrete building is a 50 PSF live load and a 20 PSF partition load everywhere on the upper floors. Load combinations from IBC 2006 were taken into consideration and the highlighted combinations were used for the lateral analysis of the frames in the building.

- Loads for the Original Steel Building
  - Dead Loads

Concrete	115 PCF
Steel	490 PCF
Partitions	10 PSF
M.E.P.	5 PSF
Finishes	3 PSF

  - Live Loads

Public Areas	100 PSF
Lobbies	100 PSF
Corridors above 1st	80 PSF
Office	50 PSF
Mechanical	150 PSF
Stairs	100 PSF

- Differences in Loads for New Concrete Building
  - Dead Loads

Concrete	145 PCF

  - Live Loads

Partitions	20 PSF

From IBC 2006:
1605.2.1 Basic Load Combinations
(As applied to this Report)

1.4 D	Eq 16-1
1.2D + 1.6L	Eq 16-2
1.2D+1.0L	Eq 16-3
1.2D+0.8W	Eq 16-3
1.2D+1.0L+1.6W	Eq 16-4
1.2D+1.0E+1.0L	Eq 16-5
0.9D+1.6W	Eq 16-6
0.9D+1.0E	Eq 16-7

Wind Design

Wind loads were determined using Section 6.5 of ASCE 7-05. The building was analyzed using a Main Wind Force Resisting System. Typically, wind would be the controlling design factor for a building in Pennsylvania, and wind was for the original building. However, once the building was redesigned in concrete, the increase in weight was enough to cause the seismic load to control the lateral system. All the coefficients were determined, and the windward and leeward pressures were determined according to ASCE 7-05. A RAM Structural System analysis was performed to confirm the validity of the hand calculations. The RAM values are comparable to the hand checks, but are slightly different. This may be due to a computer program's ability to quickly perform a finite element analysis. More in depth calculations can be seen in Appendix C of the report.

Table 1: Wind Design Properties

Basic Wind Speed (V) mph	90
Exposure Category	B
Importance Factor (I)	1
Wind Directionality Factor (Kd)	0.85
Topographic Factor (Kzt)	1

Table 2: Wind Pressure with Respect to Height

Floor Heights	Level	Total Height	K_Z	q_Z	Wind Pressures (psf)					
					N-S Windward	N-S Leeward	N-S Side Wall	E-W Windward	E-W Leeward	E-W Sidewall
18	Penthouse	92.5	0.9675	14.354	11.54	-8.21	-10.43	12.20	-4.91	-10.49
14.5	Roof	74.5	0.908	13.471	10.99	-8.21	-10.43	11.61	-4.91	-10.49
14	5	60	0.85	12.611	10.46	-8.21	-10.43	11.43	-4.91	-10.49
14	4	46	0.79	11.720	9.91	-8.21	-10.43	11.04	-4.91	-10.49
14	3	32	0.712	10.563	9.20	-8.21	-10.43	10.65	-4.91	-10.49
18	2	18	0.59	8.902	7.90	-8.21	-10.43	10.45	-4.91	-10.49

Table 3: Wind Story Forces, Shears, and Moments

Level	Wind Design					
	Load (kips)		Shear (kips)		Moment (ft-k)	
	N-S	E-W	N-S	E-W	N-S	E-W
Pent	193.4	38.8	0	0	3481.3	698.2
Roof	151.5	30.2	193.4	38.8	2196.7	437.6
5	144.8	29.3	344.9	69.0	2026.7	410.7
4	138.0	28.1	489.7	98.3	1932.5	393.8
3	132.6	27.4	627.7	126.4	1856.3	384.1
2	140.2	31.0	760.3	153.9	2523.7	557.2
Total	900.5	184.8	900.5	184.8	10535.9	2183.4

Note: Total Base Shear includes load from Windward and Leeward pressures

The wind story forces are summarized in these pictures of each side of the building. The story forces are on the left and the story shears are on the right side of the pictures.

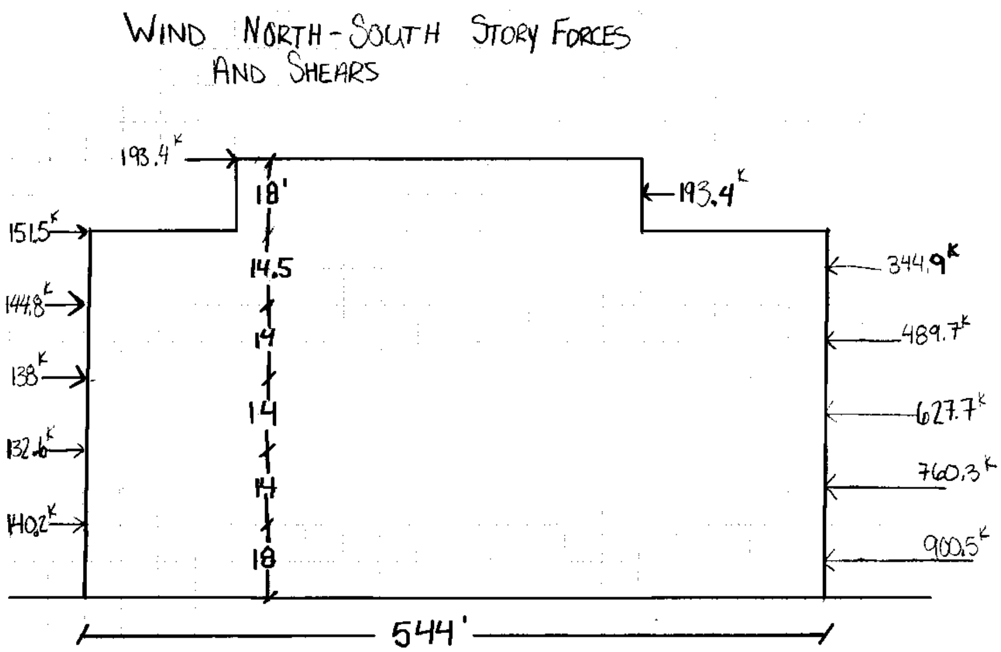

Figure 7: Wind North-South Story Force and Shear Diagram

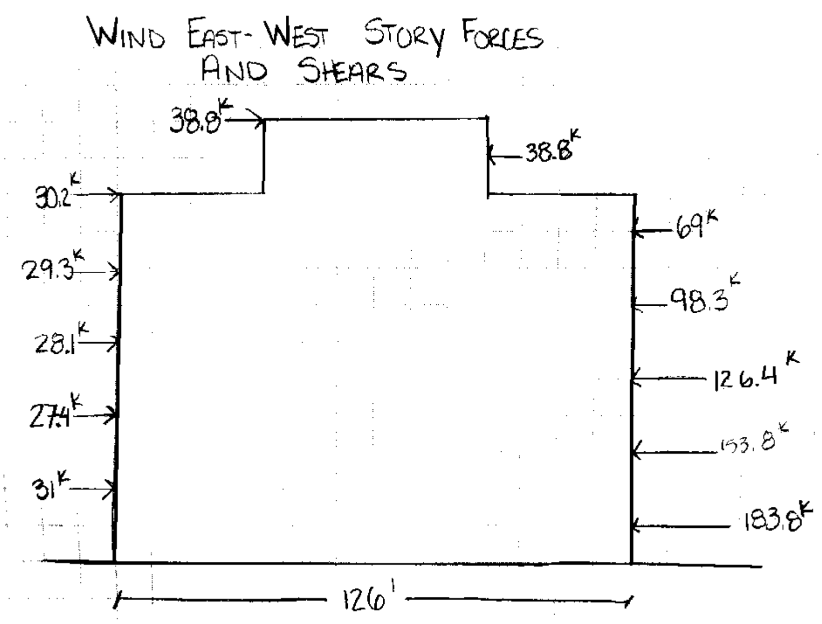

Figure 8: Wind East-West Story Force and Shear Diagram

These values are not extraordinary. The RAM checked values are different from the calculated ones from the point where q_z values come into the picture. They may be different because RAM actually calculated the values using finite element analysis instead of using Table 6-3 in ASCE 7-05.

Table 4: Hand calculation and RAM values Comparison by Height

From Table 6-3

H (ft)	K_z	q_z
92.5	0.9675	14.354
74.5	0.908	13.471
60	0.85	12.611
46	0.79	11.720
32	0.712	10.563
18	0.59	8.902
0	0.57	8.456

From RAM

H (ft)	K_z	q_z
92.5	0.966	14.331
74.5	0.909	13.486
60	0.854	12.670
46	0.792	11.750
32	0.714	10.593
18	0.605	8.976
0	0.575	8.531

Since the wind pressures do not start with the same value, they cannot be expected to be equal at any point. However the values are similar to each other, confirming the accuracy of the hand calculated values.

Seismic Design and Analysis

Typically in Pennsylvania, wind controls the design of the building's lateral system. As previously stated, this is not the case for this particular redesigned concrete building. The weight of the concrete makes the building heavy enough to cause the seismic loads to increase dramatically. The seismic loads were calculated according to ASCE 7-05, Chapters 11 and 12. The loads were determined based on a response modification factor of 3. The structure fits into the "Concrete Moment-Resisting Frame" category of ASCE 7-05's Table 12.8-2 and the C_T and X values for the period calculations were found according to those values. Further calculations can be seen in Appendix D.

Table 5: Seismic Design Values and ASCE 7-05 References

Seismic Design Values, ASCE 7-05		
Occupancy	II	Table 1-1
Importance Factor	I= 1	Table 11.5-1
Site Class	D	Table 20.3-1
Spectral Response Acceleration, short	S_S= 0.12	Figure 22-1
Spectral Response Acceleration, 1 sec	S_1= 0.046	Figure 22-2
Site Coefficient F_a	F_a= 1.6	Table 11.4-1
Site Coefficient F_V	F_V= 2.4	Table 11.4-2
MCE Spectral Response Acceleration, short	S_{MS}= 0.192	Eq. 11.4-1
MCE Spectral Response Acceleration, 1 sec	S_{M1}= 0.1104	Eq. 11.4-2
Design Spectral Acceleration, short	S_{DS}= 0.128	Eq. 11.4-3
Design Spectral Acceleration, 1 sec	S_{D1}= 0.0736	Eq. 11.4-4
Seismic Design Category	B	Table 11.6-1

Table 6: Seismic Design Values and ASCE 7-05 References

Seismic Design Values, ASCE 7-05		
Response Modification Coefficient	R= 3	Table 12.2-1
Coefficient	C_U= 1.7	Table 12.8-1
Fundamental Period	T= 1.600	Sec. 12.8.2
Seismic Response Coefficient	C_S= 0.015	Eq. 12.8-3
Building Height (above grade)	h= 92.5	

The weight of the building in concrete is over three and a half times as much as the weight of the original building in steel. The values in concrete are not even comparable to steel. The concrete loads are significantly larger, in every category. The values were checked in RAM and found to be similar. The different response modification coefficients yield different story forces, story shears, and moments as seen on the next page.

Table 7: Story Shears, Forces, and Moments for R=3.0 in concrete new design

Floor	w_x (k)	h_x (ft)	h_x^k (ft)	$w_x h_x^k$	C_{vx}	Story Force F_x (k)	Story Shear V_x (k)	Moment at Floor (ft-k)
Penthouse	6481.1	92.5	1115.41	7229044	0.179	293.33	0	27133.348
Roof	18245.1	74.5	797.56	14551503	0.361	590.46	293.33	43989.083
5	14162.0	60	570.24	8075727	0.200	327.69	883.79	19661.364
4	13922.9	46	377.75	5259370	0.130	213.41	1211.48	9816.8534
3	16960.3	32	215.24	3650482	0.091	148.13	1424.89	4740.0283
2	17785.3	18	88.23	1569200	0.039	63.67	1573.02	1146.1239
1	19178.2						1636.69	
Sum	106734.9	92.5	3164.42	40335326	1.000	1636.69	1636.69	106486.8

Table 8: Story shears, Forces, and Moments for R=3.0 in steel as-built design

Floor	w_x (k)	h_x (ft)	h_x^k (ft)	$w_x h_x^k$	C_{vx}	Story Force F_x (k)	Story Shear V_x (k)	Moment at Floor (ft-k)
Penthouse	4213	92.5	884.38	3725874	0.330	154.13	0	14256.981
Roof	4240.5	74.5	639.41	2711449	0.240	112.17	154.13	8356.3249
5	4713.6	60	462.27	2178985	0.193	90.14	266.29	5408.3285
4	4726.5	46	310.43	1467216	0.130	60.69	356.43	2791.9616
3	4724.0	32	180.20	851252	0.075	35.21	417.13	1126.8496
2	4653.4	18	76.08	354028	0.031	14.65	452.34	263.61354
1	5444.4						466.99	
Sum	28502.4	74.5	1668.39	11288804	1.000	312.86	466.99	17947.078

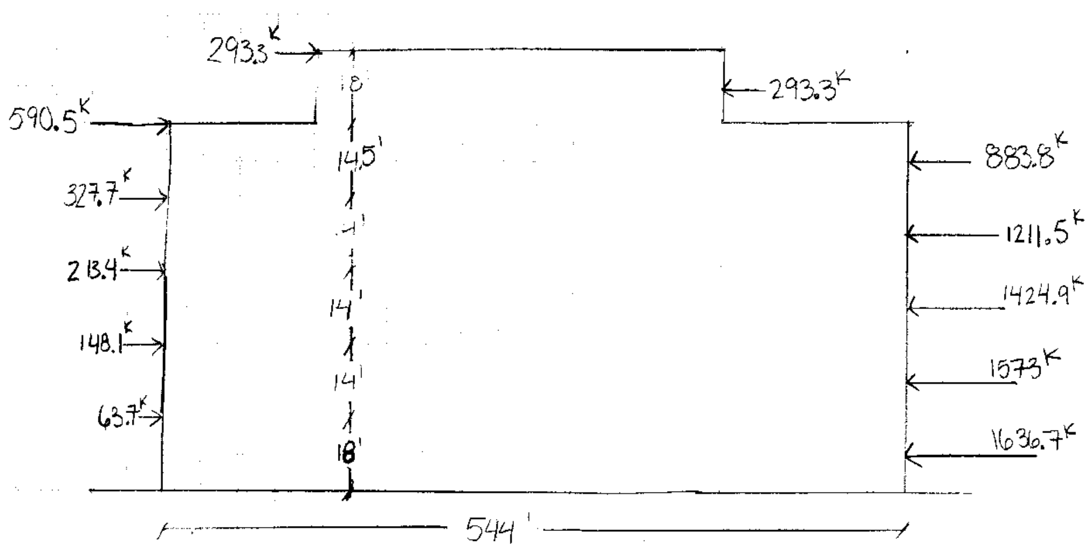

Figure 9: Seismic Forces and Story Shears

MEMBER DESIGN

To determine the members to be used in the RAM Structural System model, hand calculations were performed. The loading used for the building was 70 PSF live load (50 PSF office and 20 PSF partition load) everywhere. An 80 PSF corridor live load could also have been used, but would have been excessive since corridors do not exist everywhere on the floor.

The one-way concrete slab was designed for the 45'x24' bay. Since after the beams are removed from the length, it is a 45'x22' bay, the $L_1>2L_2$ requirement is met for a one-way slab. The slab was determined to be 10" thick with #8 @ 12" O.C. in the top of the slab and also in the bottom at mid-span of the slab. The 10" thickness was determined based on the ACI 318 deflection criteria table and was designed by hand and checked in RAM. The minimum transverse reinforcement for shrinkage and temperature is #5@12" O.C. This design is also appropriate for both green roof areas. The calculation can be viewed in Appendix E. Even though the deflection table was used, the deflections were also checked by hand and found to be within the allowable limits of L/360 for live load and L/240 for total loading. Since the new system used is a one-way slab with beams, there is no punching shear requirement for the slab.

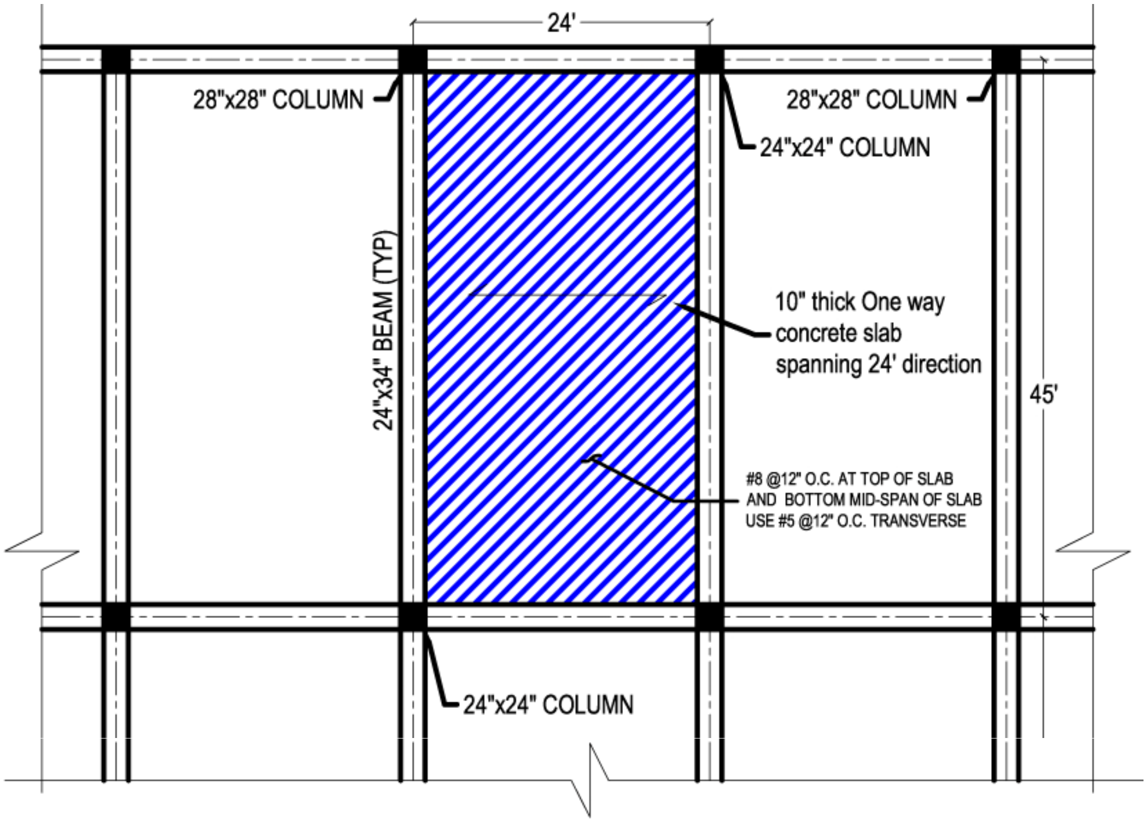

Figure 10: Redesigned Concrete Layout

After the preliminary design was done, the RAM model was built using RAM structural System and the Concrete module of the program. The lateral system was determined to be concrete moment resisting frames and spaced according to the picture below. The blue members are the gravity members and take no lateral forces. They are spaced every other frame on the plan.

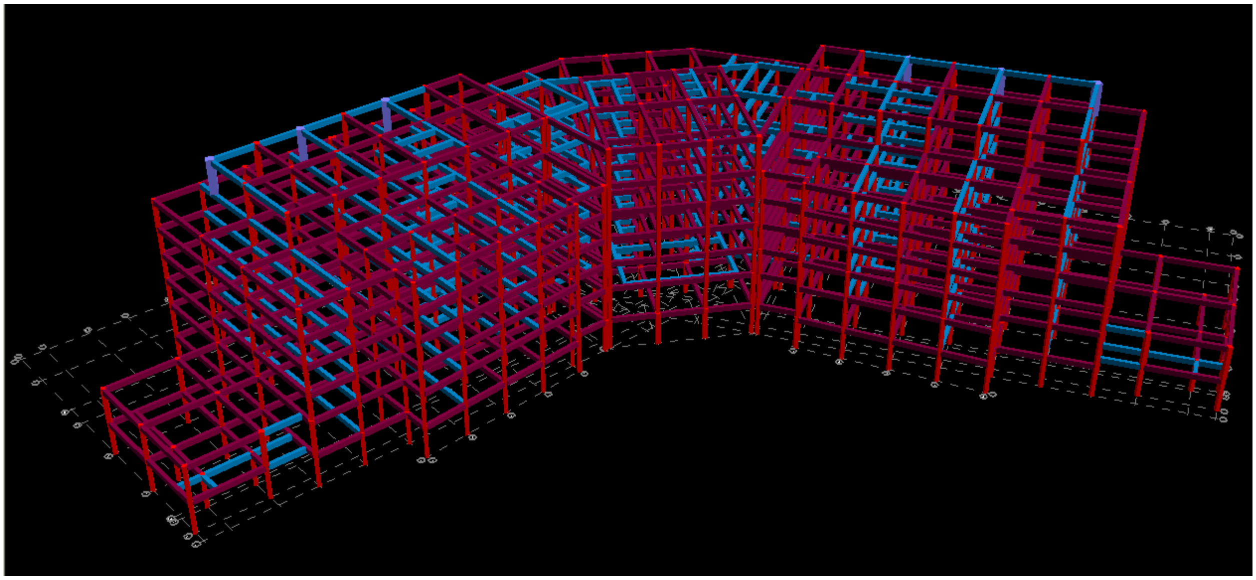

Figure 11: Whole building framing

The beams sizes were calculated also for a 45' length and a 24' distribution along the entire length of the beam. The slab weight was taken into account as well as the weight of the beam. The beams are 24"x34" and for the particular one designed two rows of (6) #8 bars were sufficient. Shear reinforcement for the interior beam was also designed and found to need (3) #3 stirrups @5" at the ends of the beam and another section of the beam was found to require (3) #3 stirrups @12". The calculation can be seen in Appendix E of this report.

The beam design was checked in RAM and the beams were updated as necessary. The green indicates the members were ok as originally designed and needed by RAM no updating to make the members meet code. The blue members needed updating of beam size, rebar size and or placement, or stirrup placement in order to meet all the code requirements. Any red members would indicate a failure to meet one of the code requirements. As seen in the picture below, all beams meet the code requirements.

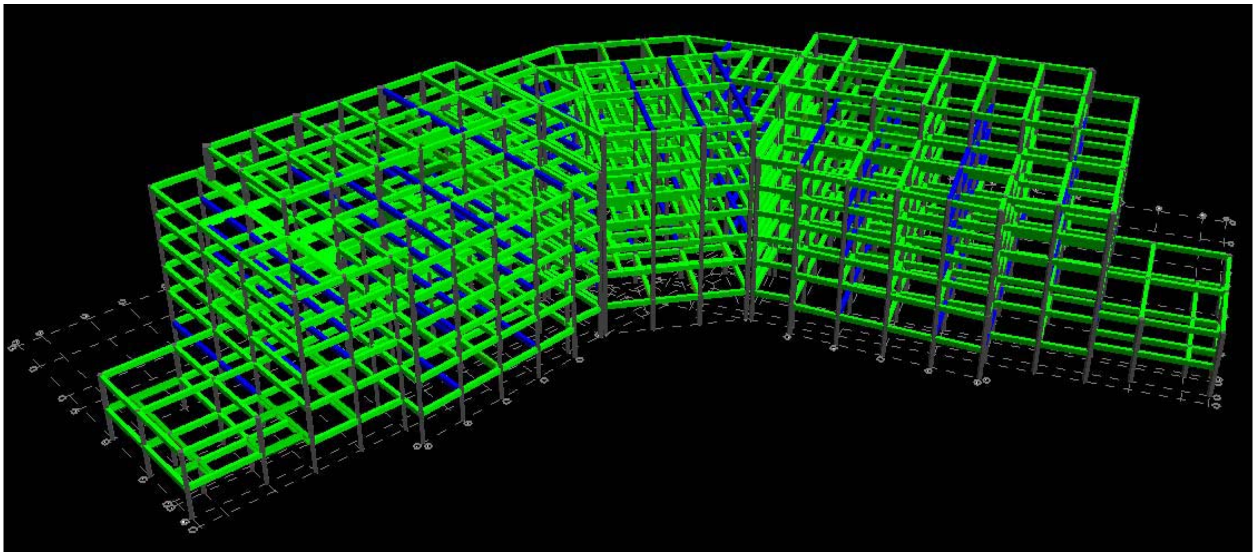

Figure 12: Beam Framing for Whole Building

The columns were originally designed in PCA column for a few members. Once placed in RAM, they were evaluated for strength, slenderness, and torsion. The columns were also updated as required. The different colors represent the percent strength required vs. the available strength of the member. The closer to the color red the column is, the higher the ratio. Blue is the lowest ratio color. As is visible, all the columns also meet the code requirements after updating. Some needed to be resized, the rebar changed, and or the transverse reinforcing altered. The columns were also spot checked after design with PCA column and the loads taken from RAM. The typical column size is 24"x24" but there is also a significant number of 28"x28" columns, mainly in the lateral system. Most of the rebar layouts have 12 bars in them, but a few have 16 bars. The typical rebar size is #10's. The PCA spot checks for select columns can be seen in Appendix E.

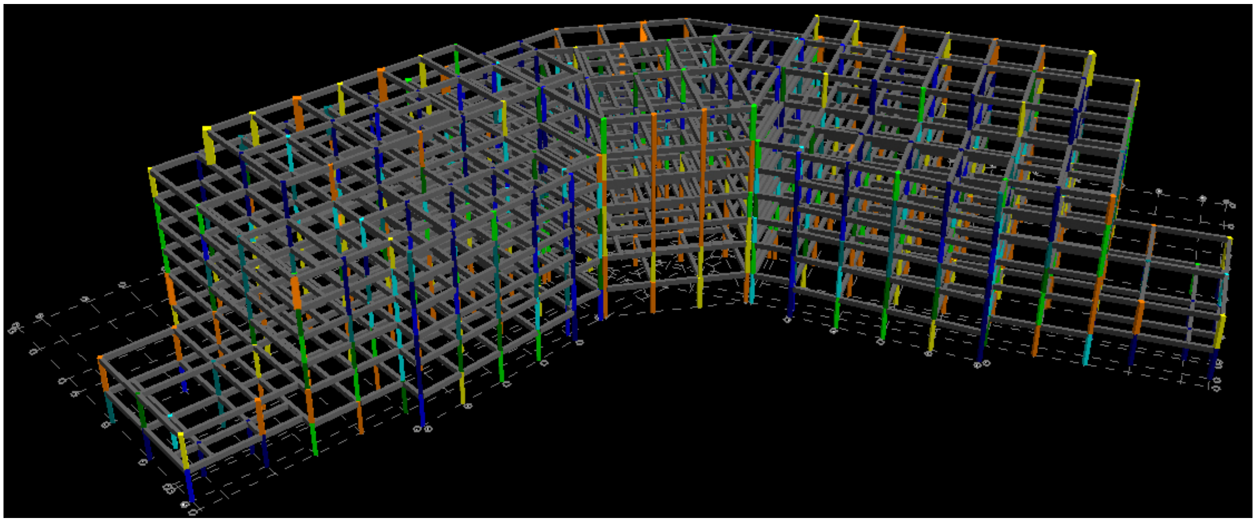

Figure 13: Column Framing for Whole Building

The lateral system is more clearly shown here with the third floor plan. The blue frames are gravity only and the red are lateral members.

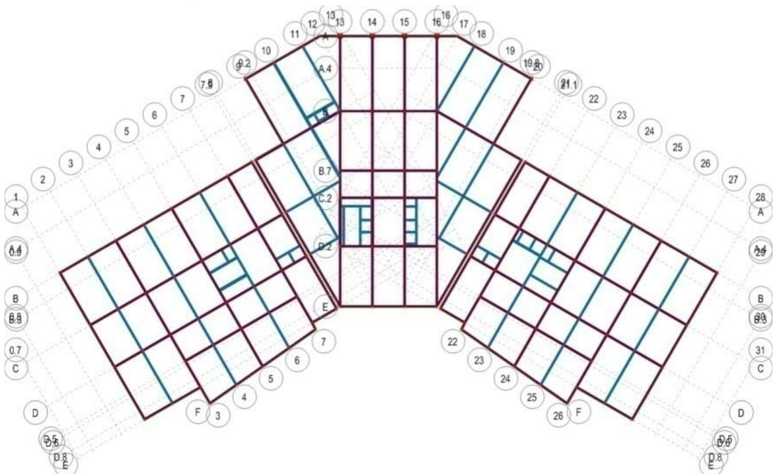

Figure 14: Whole building plan

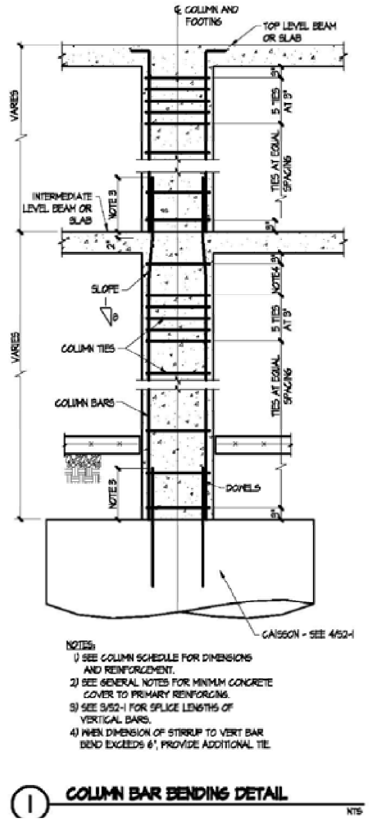

Figure 15: CMRF detail

Concrete moment resisting frames are the main lateral system. A typical detail can be seen on the right. CMRF's are more in the concept phase now. They work by assuming a concrete frame is forced to work a certain way. The rebar proceeds through the slab at an angle and continues up into the above column. The rest of the reinforcing remains the same as is designed for gravity and lateral loading. Because the rebar extends through the slab, it is possible to transfer the moment through the frames easier than in typical concrete columns.

FOUNDATION IMPACT OF NEW STRUCTURE

The redesigned building is much heavier than the as-built steel structure. Since the original spread footings and deep foundation caissons were designed for a lighter dead load, they must be resized and updated. The difference was taken into account in the schedule and cost. A sample of the spread footings and caissons were taken and the original capacity was determined. For the spread footings, a simple Capacity= Area/ Soil Bearing Pressure calculation was performed. The bearing pressure is 8000 PSF for the site in Pittsburgh, according to the current drawings and foundation notes. The required force was determined by comparing all possible combinations and taking the most critical. The area of the spread footing foundation was calculated by using the same equation, with a slight alteration, Area= Capacity/Bearing Pressure. The required height of the new spread footing involved checking punching shear and overturning moment.

The three equations used to check punching shear are:

$$V_C \leq \begin{cases} \phi(2+4\beta_C)\sqrt{(f'c)}b_O d \\ \phi 4\sqrt{(f'c)}b_O d \\ \phi(\alpha_s d/b_o+2)\sqrt{(f'c)}b_o d \end{cases}$$

After punching shear was determined, the required height on the footing could be calculated using

$$d^2\ (4V_C+q) + (2V_C+q)\ w = q\ (BL-w^2)$$

The caisson calculation was more difficult. The depth was kept the same for both the old and new caissons. This calculation consisted of finding the axial capacity of the caisson (uplift was considered, but is resisted based on 0.9*Building Weight). The calculation performed was taking the area of the caisson and multiplying it by the allowable rock bearing pressure (which in this case is 30 KSF) and then subtracting the weight of the caisson. The size for the caissons listed in the next table is the diameter.

$$\pi D/4 * \text{Bearing} - \pi D/4 * H * 145 = \text{Capacity}$$

The equation was entered into Excel to allow for ease of comparison of sizes and to allow for easier evaluation.

The new foundation sizes for the selected columns can be seen in the following table. The table was later used to determine the difference in the amount of concrete required for the foundations and to estimate the cost and labor required for the larger foundation system.

Table 9: Foundation size comparison

Size	Column	Type of Foundation	Size (ft)	Height (in)	Capacity (k)	Required (k)	Required Size	Required Height (in)	New Size (ft)	New Capacity (k)	Final Height (in)	RAM Size (ft)	RAM Height (in)
28	0.7-C	spread footing	5	18	200	384.844	6.936	18.273	7	392	22	8	24
24	1-B	spread footing	9.5	28	722	978.696	11.061	39.397	11.5	1058	44	11	36
24	1-C	spread footing	12	36	1152	1471.816	13.564	49.499	14	1568	54	13	42
24	1-D	spread footing	11	34	968	1606.032	14.169	51.518	14.5	1682	56	14	42
28	2-D	spread footing	12	36	1152	2179.108	16.504	56.044	17	2312	60	16	48
24	4-B	spread footing	10	32	800	1417.268	13.310	47.480	13.5	1458	52	13	42
30	1-E	caisson #48	5.5	146	712.749	957.832		146	7.00	1084.30	150		
28	6-B	spread footing	10	32	800	1454.464	13.484	42.858	13.5	1458	48	13	36
24	7.9-C	spread footing	13	40	1352	1342.364	12.954	45.460	13	1352	50	12	36
28	8-B	spread footing	11	34	968	922.328	10.737	33.426	11	968	38	11	30
24	8-C	spread footing	13	40	1352	1330.536	12.896	45.460	13	1352	50	12	36
48	13-A	spread footing	8	32	512	570.728	8.446	14.111	9	648	18	9	24
24	14-A.4	spread footing	8	32	512	418.416	7.232	25.211	8	512	30	7	18
24	15-B.7	spread footing	12	36	1152	1782.08	14.925	53.536	15	1800	58	14	48
28	16-E	caisson #53	4	306	376.991	1316.164		306	8.25	1399.22	310		

TORSION

Torsion must be accounted for in lateral systems due to the possibility for twisting and portions of the building being loaded in a non-uniform manner. The expansions joints allow the building to be treated structurally as three separate buildings based on the locations of the joints. To find the torsion, relative stiffness needs to be taken into account. Relative stiffness is a measure of stiffness as compared to other members in the frame. These relative stiffness values for each frame are distributed throughout the building by frames using distribution factors. The distribution factors are calculated by finding the total value of the stiffness for all the frames in a particular direction, and then finding what percent of the total each frame makes up. The stiffness is used as a basis to distribute the lateral loading through the building frames. Once both are found, the lateral loads can be distributed throughout the building. The center of mass of each section is shown in red on the following picture.

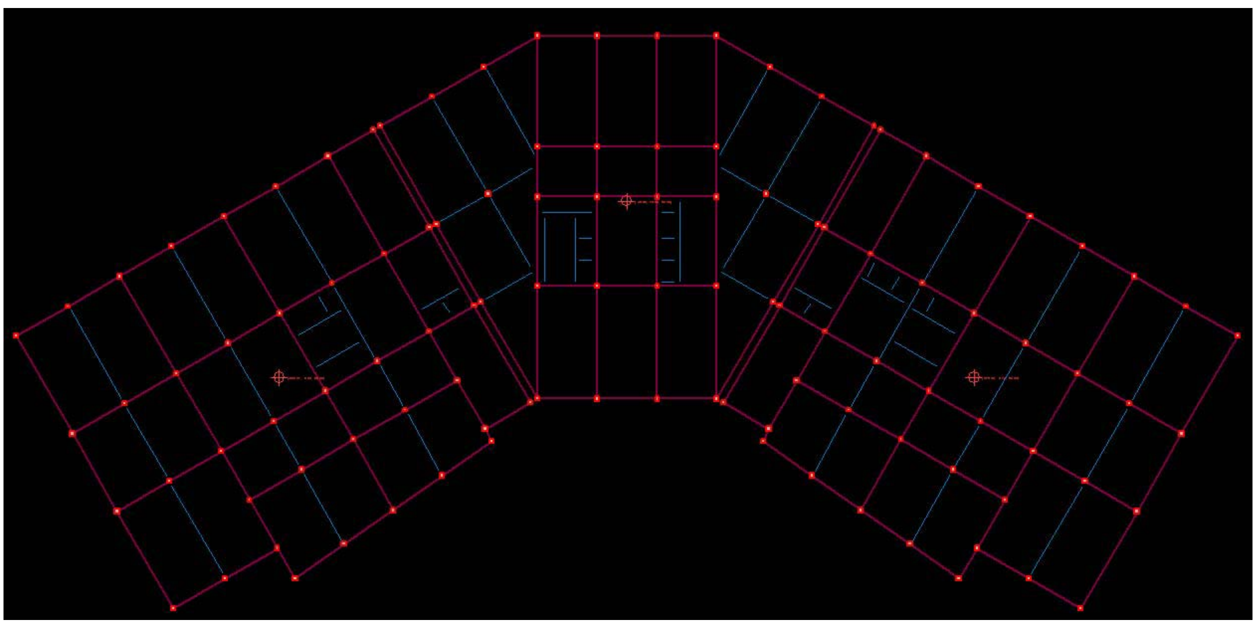

Figure 16: Center of Mass of concrete new design

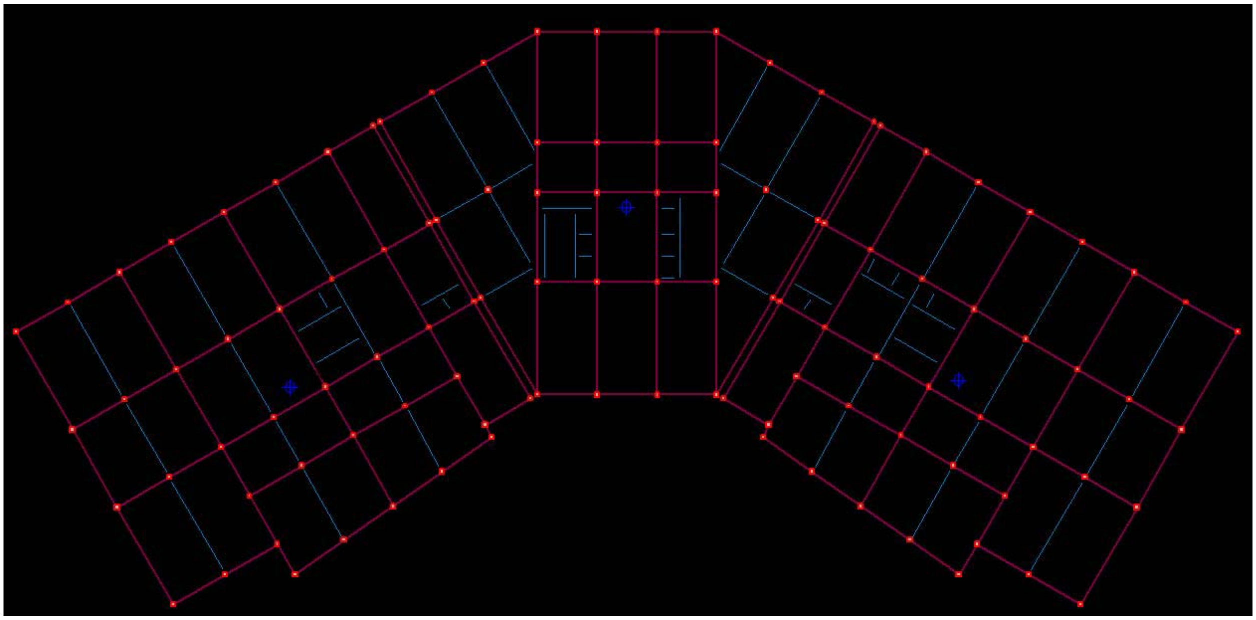

Figure 17: Centers of Rigidity of new concrete design

Torsion was not an issue with the redesigned building in concrete. However, the concepts were taken into consideration accordingly. The RAM model checked for extra torsional requirements of the lateral members and found the concrete and the stirrups were enough to resist the accidental torsion= 5% and inherent torsional loading. The lateral members clearly incurred more torsion that the gravity members in the same direction. The story shears of the lateral system frames are higher than those of the gravity system frames, as they should be. Also, frames further away from the centers of rigidity and mass are more susceptible to torsion, and as such, have higher story shears, which are reflected in the RAM output in Appendix B.

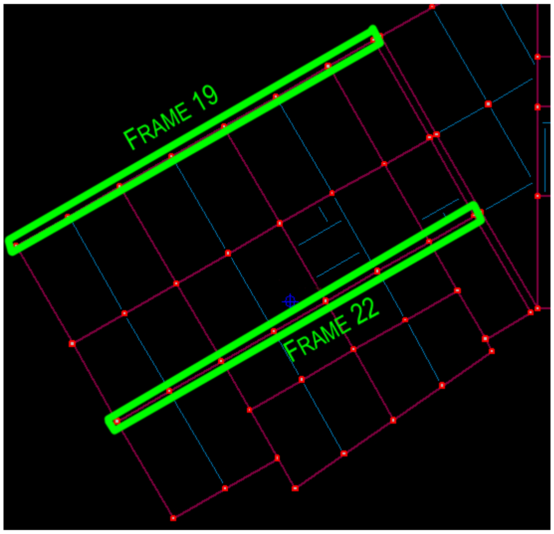

Figure 18: Center of Rigidity and Frames for Comparison

PORTAL METHOD

A portal method analysis was performed to find the moments and shear forces in the members of two frames (one from each the East-Frame 2 and Center- Frame 13 portions). This analysis was performed using the controlling seismic force on the individual frame as determined through the analysis and a RAM confirmation.

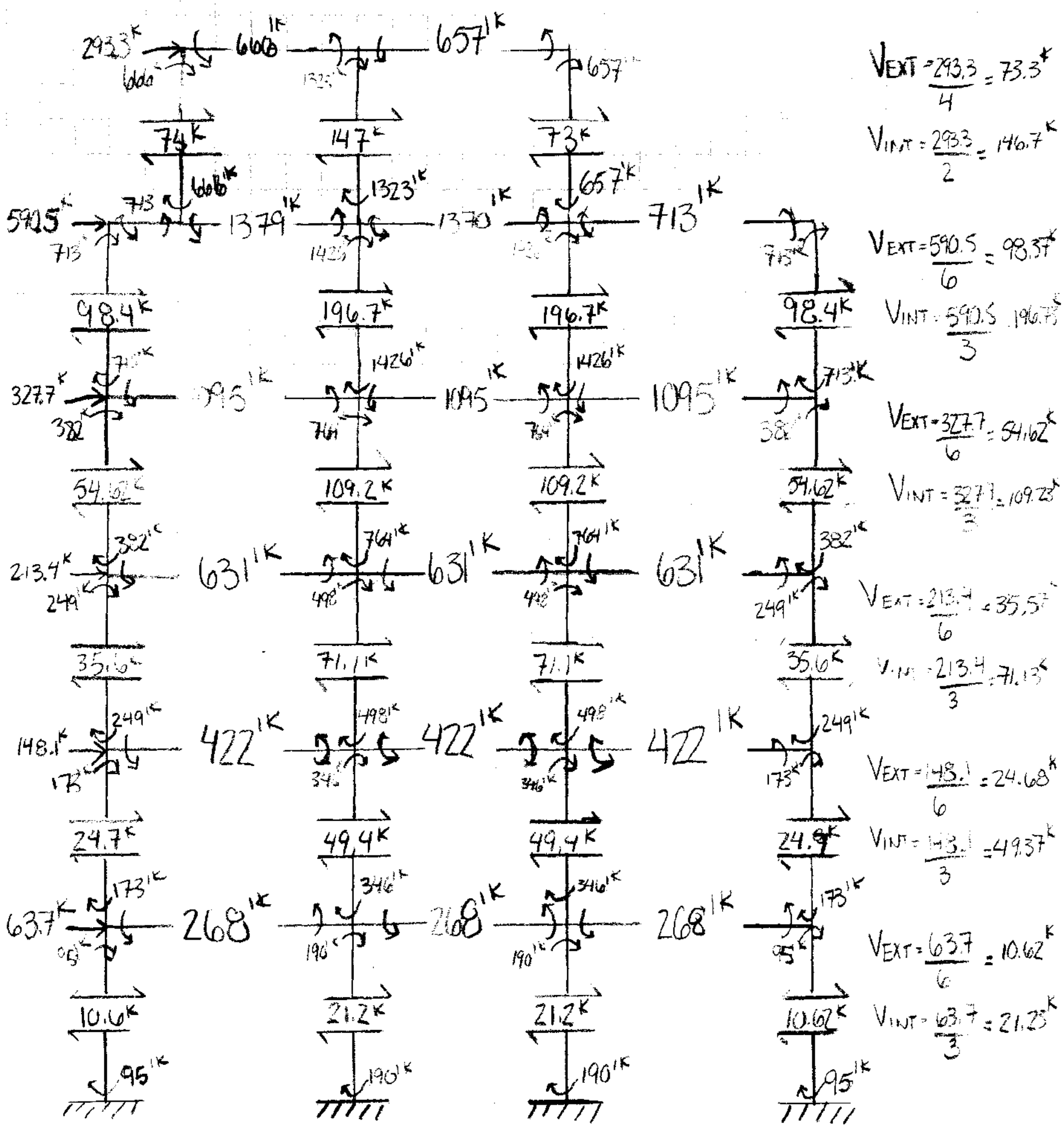

Figure 19: Portal Analysis of Frame 2 East Building with Seismic Loads Applied

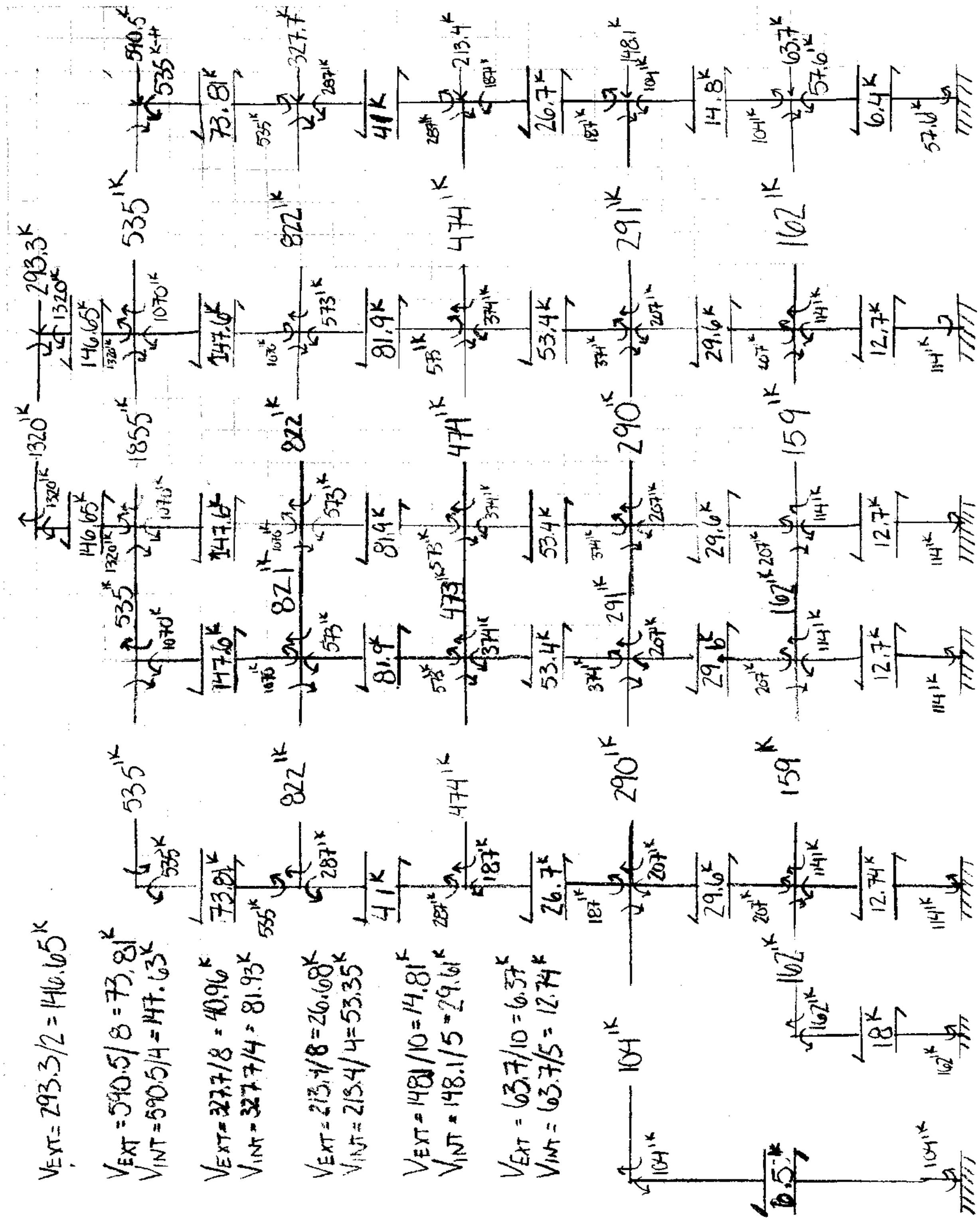

Figure 20: Portal Analysis of Frame 13 in the Center Building with Seismic Loads Applied

MEMBER CHECKS

For the new design of the building, before sizes could be checked in RAM Structural system, preliminary sizes needed to be determined. The preliminary beam checks can be seen in Appendix E. A live load of 70 PSF (50 PSF office loading and 20 PSF partition load) was used to determine the sizes. A live load of 80 could have been used, but the load of 80 PSF is for a corridor, and although there are no tenant fit-out drawings, there will not be corridors everywhere in an office building and it is acceptable to use the current loading. The preliminary design indicated the initial sizes of beams and columns for the building model. Once the sizes were assigned in the RAM Concrete design module, a full analysis was performed to check for the feasibility of all the members in the building and the lateral system's integrity. After the building gravity and lateral loads were determined by hand, they were then checked by RAM and their validity was confirmed. The columns were checked by using PCA column and can also be seen in Appendix E. An example column check is shown on the next page. A lateral beam check was performed after the moments determined through the portal analysis were applied to a specific member. The lateral beam check confirmed the beam is adequate for all load combinations in ASCE 7-05. The lateral beam check can also be seen in the Member Design Appendix.

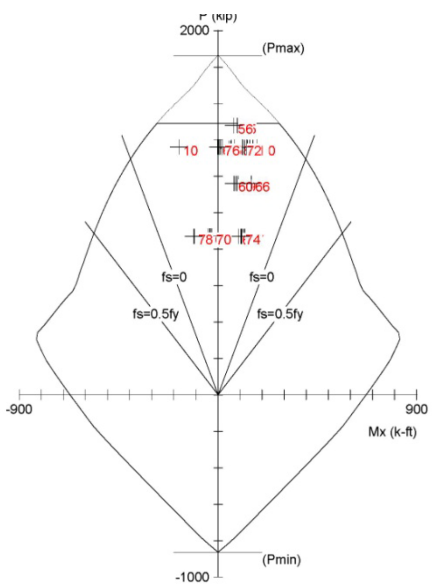

Figure 21: PCA Column Output of Column 5-C 3rd Floor

For the column check, column 5-C was chosen. This column is also on the third floor of the building. The seismic building response controls the design of the building in concrete.

DRIFT

ASCE 7-05 was used to determine the appropriate allowable drifts for wind and seismic effects. Due to drift involving serviceability rather than strength, ASCE 7-05 requires in section CC.1.2 the drift be less than H/400 on a wall or frame. The story drift was determined through RAM Analysis and checked against the ASCE 7-05 requirements. The wind was checked for all three portions of the building for each controlling load combination. The maximum story drift for each of the three portions was checked against the allowable drift according to ASCE 7-05 and the worst case scenario drift is seen below and is compared to the original steel building drift.

Table 10: Wind Drift of Concrete Redesign

Controlling Wind										
Story	Story height (ft)	Story Drift (in)	Allowable Story Drift (in) Δ_{Wind}= H/400			Total Drift (in)	Allowable Total Drift (in) Δ_{Wind}=H/400		
Penthouse	92.5	0.045	<	0.54	Acceptable	0.38814	<	2.775	Acceptable
Roof	74.5	0.033	<	0.435	Acceptable	0.34359	<	2.235	Acceptable
5	60.0	0.050	<	0.42	Acceptable	0.31057	<	1.8	Acceptable
4	46.0	0.068	<	0.42	Acceptable	0.2606	<	1.38	Acceptable
3	32.0	0.088	<	0.42	Acceptable	0.19286	<	0.96	Acceptable
2	18.0	0.105	<	0.54	Acceptable	0.10536	<	0.54	Acceptable

Table 11: Wind Drift of Original Steel Design

Controlling Wind									
Story	Story height (ft)	Story Drift (in)	Allowable Story Drift (in) Δ_{Wind}= H/400			Total Drift (in)	Allowable Total Drift (in) Δ_{Wind}=H/400		
Roof	74.5	0.127	<	0.435	Acceptable	1.02425	<	2.235	Acceptable
5	60.0	0.187	<	0.42	Acceptable	0.89767	<	1.8	Acceptable
4	46.0	0.247	<	0.42	Acceptable	0.71044	<	1.38	Acceptable
3	32.0	0.257	<	0.42	Acceptable	0.46336	<	0.96	Acceptable
2	18.0	0.207	<	0.54	Acceptable	0.20662	<	0.54	Acceptable

For seismic drift, table 12.12-1 was used to find the maximum drift of $0.02h_{sx}$, since the structure falls into the "All other structures" category of the table. This was then converted into an elastic drift ratio using equation 12.8-15 as follows so the values could be compared to RAM output, which is available upon request.

$\delta_x = C_d * \delta_{xe}/I$

$0.02h_{sx} = (3*\delta_{xe})/1.0 =$ 0.06

drift ratio= $\delta_{xe}/h_{sx} = 0.02*1.0/3 =$ 0.0066667

Table 12: Seismic Drift of Concrete Redesign

Controlling Seismic				
Story	Story height (ft)	Acutal Drift Ratio	Allowable Total Drift Ratio δ_{xe}/h_{sx}=0.02*1.0/3	
Pent	92.5	0.0004	<	0.006667
Roof	74.5	0.0005	<	0.006667
5	60.0	0.0008	<	0.006667
4	46.0	0.0009	<	0.006667
3	32.0	0.001	<	0.006667
2	18.0	0.0009	<	0.006667

Table 13: Seismic Drift of Steel Design

Controlling Seismic				
Story	Story height (ft)	Acutal Drift Ratio	Allowable Total Drift Ratio δ_{xe}/h_{sx}=0.02*1.0/3	
Roof	74.5	0.0011	<	0.006667
5	60.0	0.0013	<	0.006667
4	46.0	0.0014	<	0.006667
3	32.0	0.0012	<	0.006667
2	18.0	0.0006	<	0.006667

Comparing the drift ratios for wind and seismic forces to the allowable drift, it can be concluded that drift is not an issue for either load. It can also reasonably be confirmed that the concrete structure is less susceptible to drift than the steel building.

Overturning Moment

Overturning moment is a design issue that needs to be taken into consideration for steel building, but in general concrete buildings resist the overturning moment purely by the weight of the building. The calculation below is just to show how little the overturning moment impacts the design for this concrete building.

M_{WN-S}= 1.6*Moment from Wind Design=	81566	k-ft
M_E = ΣH(ft)*Earthquake Design Load(k)=	106487	k-ft
P_{Uplift}=M/L=	647.3497	k
P_{DBldg}=	87557	kips
Load on Opposite Columns= $0.9P_D$=	78800.99	kips
$M_{Resisting}$= P*Trib Area=	4964462	k-ft

$M_{Resisting} > M_E > M_W$

Clearly the overturning moment is insignificant when compared to the weight of the building.

Depth Study Overview

The intent of this study was to practice concrete design and design a building capable of being built according to all applicable codes. Lateral analysis determined both the wind and seismic loads, and also determined the building to be seismically controlled as opposed to the original steel building being controlled by wind load. Lateral frames were eliminated from every frame, to every other frame, which is definitely a success in the design. Torsion was checked for this building, and found to be as expected. As also would be expected, the lateral members are a bit larger than the gravity only members, though not significantly. The addition of the green roof had some structural implications with the sizing of columns and beams supporting such a massive load, but did not cause any serious issues in the design.

Breadth Study 1- Construction Management Analysis

Cost Analysis

Before a final response to the proposed changes can be evaluated, cost and schedule implications must be considered. The building in concrete is completely structurally different from a steel building. A budget for the as-built building was obtained from Turner Construction Company and was compared to the redesigned concrete building cost. An estimate for the redesign was calculated using RSMeans and the new structure volume of concrete and weight of steel rebar. Cost estimates for the building structure with and without the green roofs were developed. It was necessary to develop a schedule and estimate without a green roof so it could be more accurately compared to the existing steel building, which does not have a green roof.

Table 14: Cost Estimate for Redesigned Building without Green Roofs

Detailed Cost Analysis of the Structure-No Green Roof									
Level	Description	Amount	Material Price	Material Cost	Labor Price	Labor Cost	Equipment Price	Equipment Cost	Total Cost
Reinforcement	Foundation	58 Ton	$935.00	$54,230	$430.00	$24,940	$30.35	$1,760	$80,930
	Columns	156Ton	$935.00	$147,263	$430.00	$430.00	$30.35	$4,780	$152,473
	Beam/Slabs	504 Ton	$935.00	$470,642	$430.00	$216,445	$30.35	$15,277	$702,363
	SUB-TOTAL	719	$935.00	$672,134	$430.00	$241,815	$30.35	$21,817	$935,766
Cast in Place Concrete	Foundations	6100 CY	$109.00	$664,900	$14.90	$90,890	$5.55	$33,855	$789,645
	Columns	1443 CY	$109.00	$157,189	$34.00	$49,031	$16.95	$24,444	$230,664
	Slabs	14192 CY	$109.00	$1,546,928	$18.20	$258,294	$9.15	$129,857	$1,935,079
	Beams	6477 CY	$109.00	$706,026	$26.50	$171,648	$1,320.00	$8,550,036	$9,427,710
	SUB-TOTAL	28211	$109.00	$3,075,043	$20.20	$569,864	$1,352	$8,738,191	$12,383,098
Location Factor: 98.9%	Total Structure Estimate:		$13,173,000			Total Labor Cost:		$812,000	
	Total Material Cost:		$3,748,000			Total Equipment Cost:		$8,761,000	

Table 15: Turner's Budgets

Turner Construction Company Budgets	
Deep foundations (caissons)	$215,000
Concrete (Spread ftgs, slabs)	$5,199,000
Structural Steel	$7,892,000
Total Structure	$13,306,000
Whole Building	$55,878,000

Turner's whole building budget was $55,878,000. Their entire structure budget was $13,306,000, which is approximately equal to the concrete redesign estimate. The cost per square foot for the as-built building is $30.90/SF while the new building in concrete is $30.60/SF. Some reasons for the cost of the building being so much greater for concrete than for the steel could be Turner's budget came directly from subcontractors and there was competition for the work or also that their estimates were real numbers and are therefore more accurate than an RSMeans estimate.

Schedule Analysis

A schedule analysis was also necessary to evaluate the two structures. Microsoft Project was used to generate a schedule for the redesigned concrete building. To develop the duration times of each slab, the building was split into columns. The beams were formed first, then the slab right after. The two were reinforced together, but the beams were placed before the slab. After the slab had accrued three-day strength, the columns on that floor were formed. The columns went through the same process with the beams above being formed after the columns had accrued the same three-day strength. Both structures were started on the same day, March 3, 2008. Turner's schedule has the concrete on the penthouse done being placed on October 10, 2008. The schedule produced for the redesigned concrete building estimates the penthouse slab finished on December 9, 2008. The entire schedule calendar can be seen in Appendix F. A Gantt chart is available upon request but was not included due to its length.

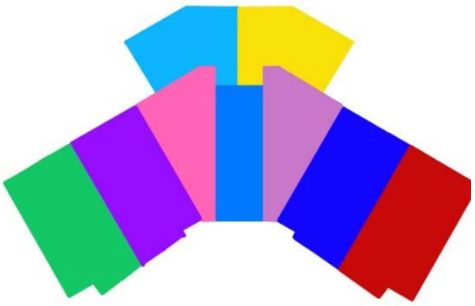

Figure 22: Sample Floor Plan Divided (3rd floor)

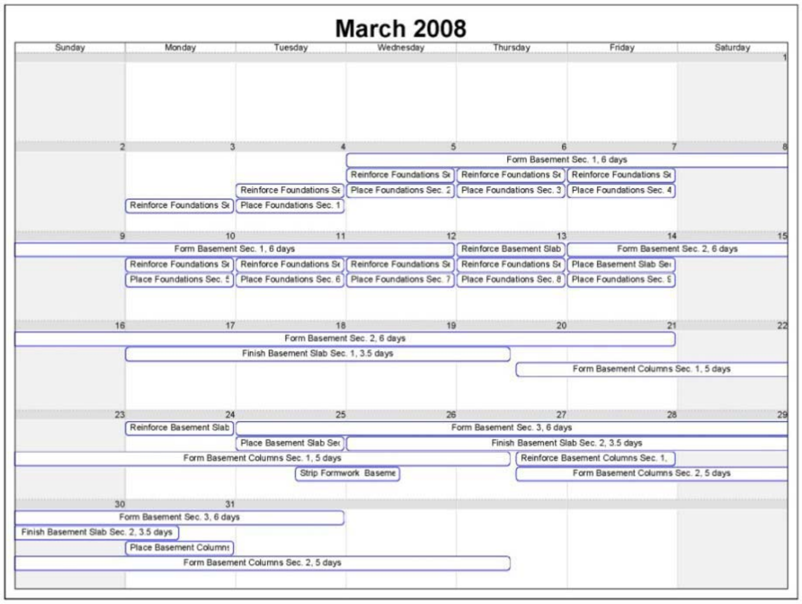

Figure 23: Sample Schedule Calendar for March 2008

BREADTH STUDY 1 OVERVIEW

The goals for the construction management breadth study of calculating a cost estimate and generating a schedule were certainly met. Also, a cost estimate and schedule were generated and compared with the addition of the green roof and to Turner Construction Company's original estimate and schedule. It was determined that Turner Construction Company managed to erect the building much faster and cheaper in steel than the design would have been in concrete. Concrete generally does take longer to erect than steel due to the curing time and placing and stripping of the formwork. This breadth portion of the project was a success even though it was not the most efficient - in time or money- way to build this building.

BREADTH STUDY 2- SUSTAINABLE ARCHITECTURE

Since one of the owner's goals was to have a LEED certified building, adding another green feature seemed to be a realistic option. The way the Westinghouse Electric Company Corporate Headquarters Building One is situated, with a large portion of the building facing south, a green roof would be just one more way to make the building "green". Green roofs help to integrate their buildings into the natural surroundings and can be used for various activities for the office. These areas can be used as patios, lunch areas, or meeting areas.

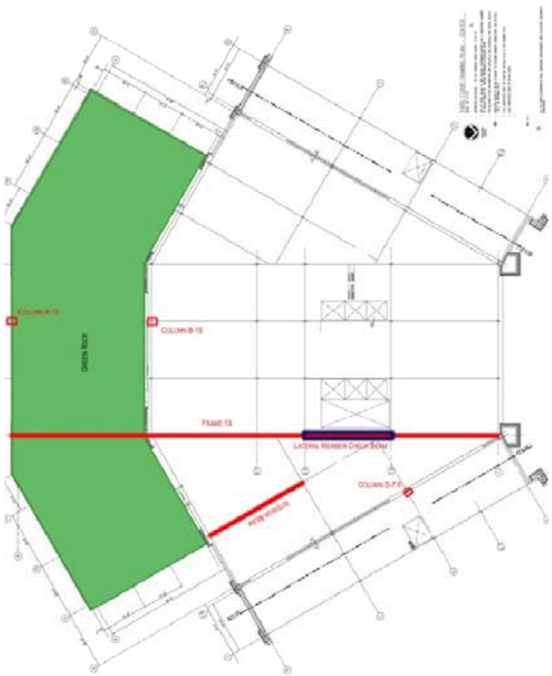

Figure 24: 3rd Floor Plan Center with Green Roof

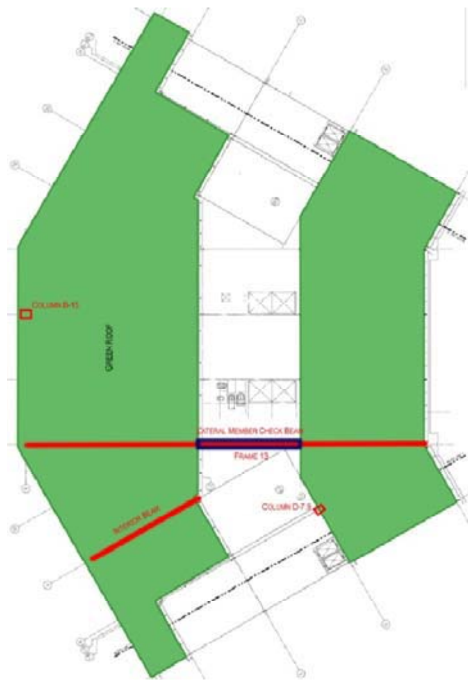

Figure 25: Roof Plan Center with Green Roof

A LEED credit analysis was also performed to check to see the viability of making the redesigned concrete building a LEED Certified building as well as the as-built one. The redesigned building was able to achieve 28 credits, with the requirement for certification at 26. The actual checklist can be seen in Appendix G. The benefits of a green roof are extensive. They make the recycled water content quality better, and provide a clean way to collect it as well as limiting the heat island effects.

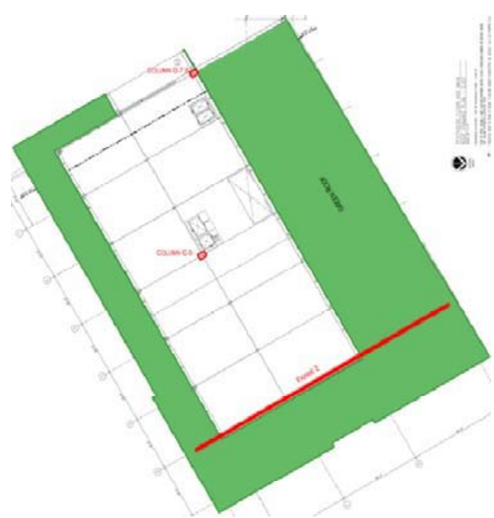

Figure 26: Roof Plan East with Green Roof Areas in Green

The green roof materials selected are modular, meaning the sod and plants come in rectangular sections capable of being moved around if the owner decides to change the layout of the walkways and the soil. Native Pennsylvania plants will be used on the green roof patio. The green roof on the third floor will be available for use as an outdoor patio, and was treated as such with loading.

However, the green roof on the top of the building will not be accessible except for service conditions.

Green roofs typically consist of soil and vegetation, a filter of some sort of fabric, a drainage system, a moisture barrier, insulation, a root barrier, a protection layer, and a waterproofing membrane.

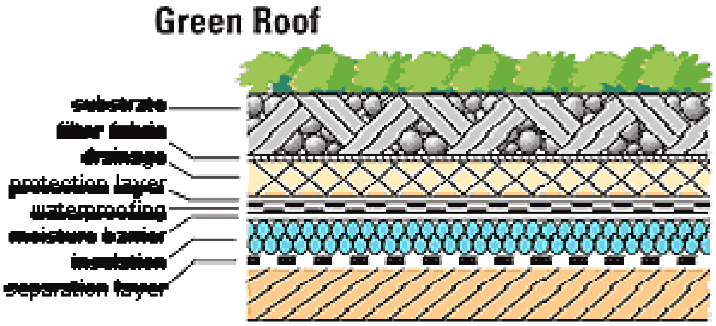

Figure 27: Green Roof Detail courtesy of www.deq.state.mi.us/documents/deq-ess-p2-p2week-greenroofreources.doc

Carlisle Coatings and Waterproofing has a green roof waterproofing system that fits exactly what this building requires. CCW is located in Carlisle, PA, which is approximately 205 miles away from the site. They specify for an extensive green roof a CCS 500R Hot applied waterproofing membrane system, CCW Protection Board HS, a CCW Root Barrier consisting of 40-mil non-reinforced Geomembrane, a CCW MiraDRAIN 9800 Drainage Board, insulation as required, MiraDRAIN GR 9200, and CCW 300HV Water Retention Mat, all underneath the soil. The full specification can be found at www.carlisle-ccw.com/Doco/spec07555613CCW500RGreenRoofWaterproofingSystem.pdf.

Also, for this study, since the green roof will be retaining water, pipes were spaced and sized for water flow. The green roof on the third floor was separated into three parts- two 6,131 SF sections and one 6,419 SF section. The green roof on the roof level of the building was split into six equal 6,434 SF sections. For all the sections used, two 3" pipes were found to meet the code requirements. The Portal Plus Roof Drain Calculator was used to help size the drains and pipes (located at www.portalplus.com/drain_calc.htm). The calculation is performed based on the 100-year storm. This calculation takes each local code and translates it based on the area of the roof area.

Table 16: Cost Estimate with green roof

Detailed Cost Analysis of the Structure									
Level	Description	Amount	Material Price	Material Cost	Labor Price	Labor Cost	Equipment Price	Equipment Cost	Total Cost
Reinforcement	Foundation	58 Ton	$935.00	$54,230	$430.00	$24,940	$30.35	$1,760	$80,930
	Columns	175 Ton	$935.00	$163,625	$430.00	$430.00	$30.35	$5,311	$169,366
	Beam/Slabs	572 Ton	$935.00	$534,820	$430.00	$245,960	$30.35	$17,360	$798,140
	SUB-TOTAL	805	$935.00	$752,675	$430.00	$346,150.00	$30.35	$24,432	$1,123,257
Cast in Place Concrete	Foundations	6100 CY	$109.00	$664,900	$14.90	$90,890	$5.55	$33,855	$789,645
	Columns	1518 CY	$109.00	$165,462	$34.00	$51,612	$16.95	$25,730	$242,804
	Slabs	14192 CY	$109.00	$1,546,928	$18.20	$258,294	$9.15	$129,857	$1,935,079
	Beams	7197 CY	$109.00	$784,473	$26.50	$190,721	$1,320.00	$9,500,040	$10,475,234
	SUB-TOTAL	29007	$109.00	$3,161,763	$23.40	$271,330	$1,352	$9,689,482	$13,122,575
Location Factor: 98.9%	Total Structure Estimate:		$14,332,000			Total Labor Cost:		$863,000	
	Total Material Cost:		$3,915,000			Total Equipment Cost:		$9,714,000	

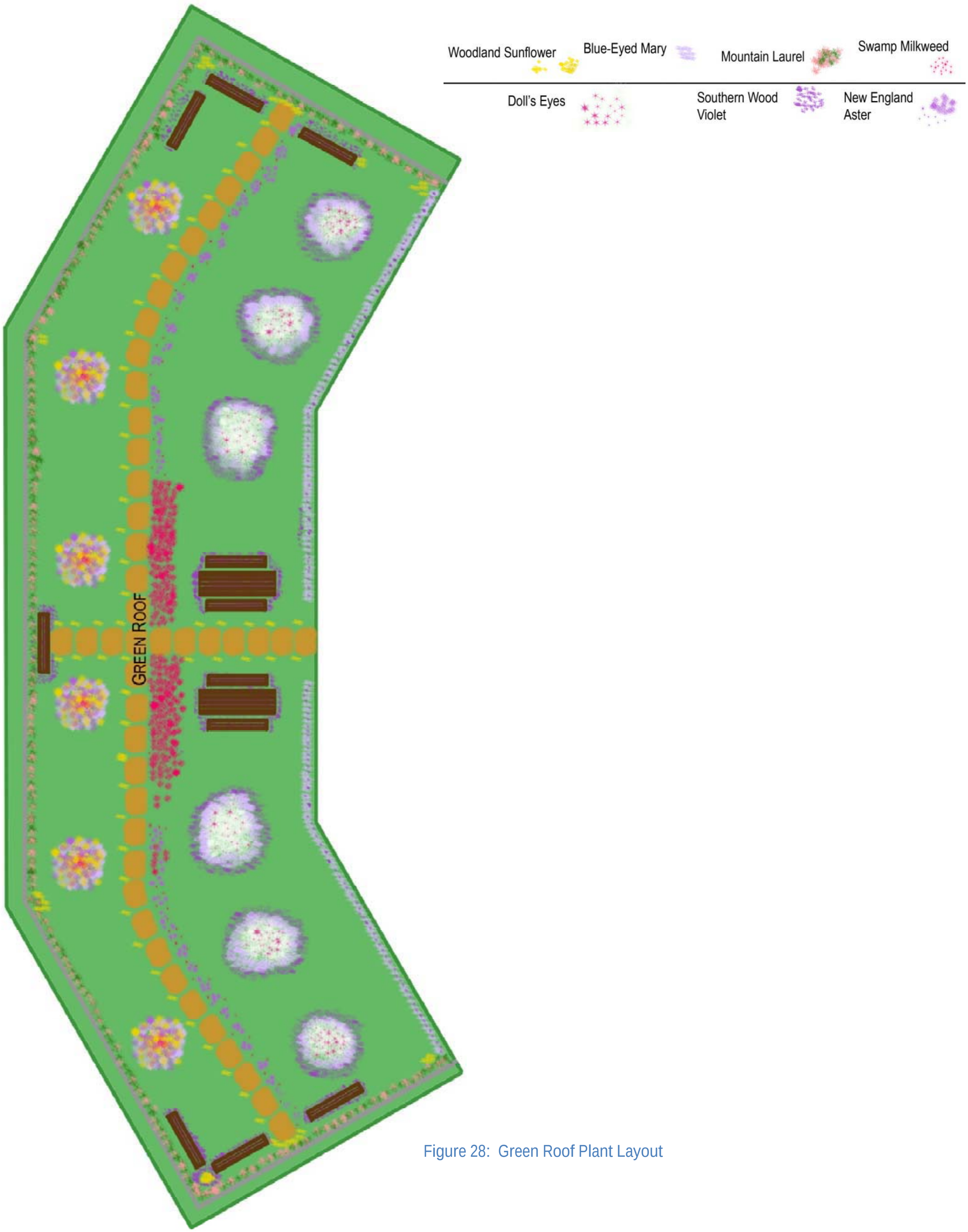

Figure 28: Green Roof Plant Layout

Some of the plants native to Pennsylvania able to be planted on the roof material are:


Figure 29: Doll's Eyes


Figure 30: Swamp Milkweed


Figure 31: Blue-eyed Mary


Figure 32: New England Aster


Figure 33: Woodland Sunflower


Figure 34: Southern Wood Violet

All the plants and figures are from the Audubon Society of Western Pennsylvania, Audubon Center for Native Plants webpage located at: http://www.aswp.org/acnp_culture_and_use_guide.html. Also, all the plants are smaller plants, which will mesh nicely into the green roof environment or rain garden environment and can be moved around and resituated easily. The plants also bloom at different points of the season, so from May until mid-winter there will be plants blooming on the roof. The Blue-eyed Mary emerges in the fall and stays green through the winter. The smallest plants tend to be shrubby, spread easily, and require little maintenance, which will keep costs down. Instead of grass for a base on the roof, the main plant is sedum, which is hardier than grass and is a preferred plant on such surfaces.

Breadth Study 2 Overview

The green roof has a weight much larger than a typical roof and since one of the green roofs is going to be used as a patio, the live load also increases. These differences impact the size of the structural members, which also impact the cost of the structure and the schedule. The columns require an additional 19 tons more reinforcing and the beams and slabs require an additional 68 tons. The volume of concrete required for the structure to be able to support the green roof is: 75 CY for columns and 750 CY for the beams. These differences translate into $1,159,000 which is equivalent to $2.68/SF, more to add a green roof onto this redesigned concrete structure. As far as schedule is concerned, the building could be completed one week earlier without a green roof.

After all these items are considered, it can be concluded that adding a green roof is certainly a viable option for this building. The green roof portion of this report was a success. It met all the goals set for it such as proper integration of a green roof system into the building, detailing, specifying native plants and laying them out, and sizing of a pipe for water flow from the roof for drainage. The system also had a cost estimate and a schedule generated for it so the green roof could be compared to the new concrete design system. The difference in cost from the new system with a green roof and without a green roof was able to be calculated and compared and found to be not considerably higher when compared to the total cost of the building.

Conclusions And Recommendations

The study and redesign of the Westinghouse Electric Company Corporate Headquarters Building One has been an overall success. Instead of using the same 80 PSF live load everywhere used in the previous technical reports, a live load of 70 PSF (50 PSF office load and 20 PSF partition load) was used uniformly throughout the upper floors of the building. The redesigned concrete building was able to be compared to the original steel building in terms of construction cost, schedule impact, and overall effectiveness. The redesigned building also had a green roof added to the building in order to integrate it into the environment and make a statement as a corporate headquarters.

The building was successfully redesigned with a concrete cast-in-place one-way slab with beams system using concrete moment resisting frames as the lateral system. Shear walls were considered for this design, but could not be used effectively due to the necessity of an open plan for tenant fit-out requirements. The slab, beams, columns, and foundations were all designed or resized according to ACI 318-08 and ASCE 7-05 and the applicable sections. Once a preliminary design was established, the building was modeled in RAM Structural System and checked for validity and uniformity of members and reinforcement and torsion.

Since the building material was changed to concrete, the weight of the building significantly increased, causing the seismic loads to change. In the original steel design, wind was the controlling lateral load in one of the directions and seismic controlled the other direction. However, in the concrete redesign seismic load controls the lateral system. The lateral loads were checked in RAM as well. Drift ratios and drift were determined in RAM and checked to the allowable values for serviceability from ASCE 7-05 and found to be acceptable. A hand check was performed on a lateral beam to ensure the validity of the structural design. Uplift is not an issue because the pure weight of the building will hold the building down. The foundations were resized according to the required strength for both the spread footings and the caissons. All the goals for the structural part of this project were met, making it a success.

After all analyses were performed for the design of the concrete building, the building was compared to the original steel building. It can be reasonably concluded steel is a more efficient system than concrete in this particular application. The cost estimate was compared to Turner's budget, and found to be significantly higher. The schedule for the new building was generated and also compared to Turner's and was found to be two months longer. While the project was a success in terms of the goals, it was not ideal since the proposed modifications extended the schedule and increased the cost.

As far as the sustainable architecture breadth is concerned, the project was also a success. The green roof was detailed, materials specified, drainage pipes sized based on local code requirements, and plants specified for the area. Additionally, the green roof impacts the structure and causes the columns and beams to be larger. The green roof increases the structural cost $1,159,000 ($2.68/SF) and increases the schedule by one week.

All parts of this analysis considered, it is not recommended to make the building structure concrete instead of steel. The building can be built at a better value and has a much faster erection time in steel. However, it is recommended to use more sustainable architecture in the form of a green roof. The total cost of the building does not change comparatively when it is added, and it increases the value of the building with respect to LEED certification and incorporation into the environment.

Further calculations can be found in the appendices. Additional calculations for wind and seismic loading and the RAM Model are available upon request.