A Reevaluation of Montgomery County's Judicial Center Annex

THE PENNSYLVANIA STATE UNIVERSITY
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Abstract

The Judicial Center Annex is a 210,000 square foot addition to Montgomery County's Judicial Center located in Rockville, MD. The $67 million dollar project is currently under construction and slated to finish in April of 2013.

The structural system, as designed is a post tensioned slab supported by reinforced concrete columns. The lateral force resisting system is reinforced concrete shear walls and the foundations are core drilled piers.

This report is the result of a semester of research upon the existing structural design. Based upon the findings a proposal was created for a system redesign. Due to the fact that the building lacked a height restriction it was determined to explore a steel alternative to the concrete construction. Also, as seismic design was an interest, the building was "moved" to San Francisco where the greater seismic forces would need to be dealt with.

The redesign in Maryland necessitated a cost and schedule comparison to determine the viability of the change in systems, so this was chosen as one breadth for further exploration. The other breadth was inspired by the sustainable features found upon the roof. The JCA has both green roof and photovoltaic panels. It was determined to investigate if changing the green roof portions to PV panels would be more beneficial for the owner by comparing the life cycle cost, carbon emissions, and LEED impacts of the two systems. The LEED checklist would also be further explored looking for opportunities to improve upon the Gold rated building.

The steel structure was able to be implemented effectively, using braced frames in lieu of the shear walls and maintaining the current grid to avoid impacting the layout. The large floor to floor heights and generous plenum spaces made a height adjustment largely unnecessary, with the total height only increasing by a 1.5'. It was estimated that the system could save in the order of $700k in cost and a month in schedule.

The steel move to California necessitated changing the ordinary concentrically braced frames to special concentrically braced frames in order to deal with the increased forces. This required special detailing and turned out to be slightly uneconomical due to the one chevron configuration. Changing this to an eccentrically braced frame saved in the order of $200k and 70 tons of steel. Adding additional frames also took advantage of certain code provisions and helped mitigate torsion problems.

The sustainability study showed that the green roof was the better option, as it had a lower initial investment which it paid back quicker. It also had other benefits in the form of net negative carbon emissions, storm water runoff control, urban heat island reduction, as well as impacting a possible 7 LEED points.

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Chapter 1 -Building Introduction

The Judicial Center Annex (JCA) is a modern addition to the existing Montgomery County Judicial Center. Located on the corners of Maryland Avenue and East Jefferson Street in downtown Rockville, MD, the JCA is set to provide a bold statement through both its architecture and engineering. Construction on the addition began this past April and is projected to take two years to complete.

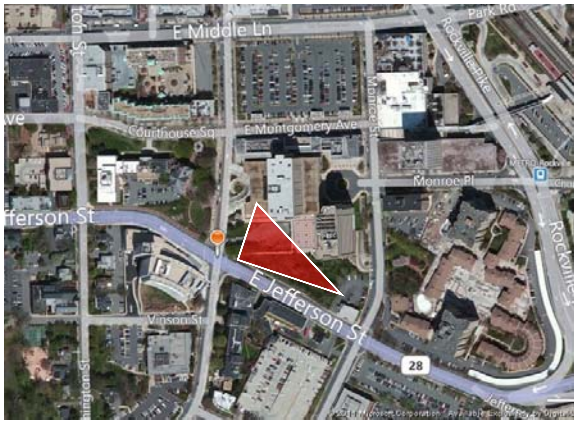

Figure 1-1: Site Location, Source: Bing.com

The JCA will stand 114' tall at the crest of each of the four lanterns located on top of the building, so tall that limitations on local building codes needed to be waived for overall building height. Six stories rise above the ground, with garage and terrace levels located below grade, adding approximately 210,000 sq ft to the Judicial Center which includes 10 more courtrooms and several administrative spaces.

The project team, led by AECOM who provided both architectural and the majority of building engineering services, was able to achieve a unique look through both form and material. The East and West Elevations (Figure 1-2) are dominated by glazing, with the curtain wall that covers the East wrapping around the South corner. This curtain wall system is unique in that it uses glass stabilizing fins instead of traditional aluminum mullions, which enables an all glass look that when combined with the way the slab cantilevers out from the structure gives the illusion of the floors floating without structure. On the North the addition abuts against the original Judicial Center. The elements of the façade not covered in glass are sheathed in either a powder coated aluminum that has a reddish hue or architectural pre-cast panels that are more reminiscent of the exterior of the original building.

Figure 1-2: West Elevation

From the roof projects four lanterns which have a translucent linear glazing system allowing them to light up the night sky in a truly dramatic manner. The roof is also the site of two of the JCA's sustainable features that enabled it to achieve a LEED Gold Rating. The tops of each of the four lanterns are covered in photovoltaic panels, while green roofs cover much of the remaining roof.

1.1 - Structural Overview

The JCA sits atop core-drilled concrete piers due to the rather poor soil conditions, all columns coming to bear atop a pier. The floor systems are post-tensioned slabs, with wide-shallow beams running one-way on the typical levels framing into cast-in-place concrete columns. The lateral system consists of five concrete shear walls, which rise continuously to the penthouse level, with some continuing to support the roof.

This building was designed as Occupancy III according to Sheet 1.S001 due to the detaining cells contained.

Floor Systems

As mentioned previously, the floor systems for the JCA utilize post-tensioning. The economy is achieved by greater span lengths being possible, with thinner slab depths. The typical floor system, which begins on the terrace level and extends to the 5th floor, has both 8” and 9” slab depths, with wide-shallow beams running in the plan NS direction. The beams extend 8” below the slab and are not centered on the column lines, instead offset in plan to allow for the provisions of ACI 318-08 Section 13.2.5 for a drop panel. The bays are essentially uniform in parts of the building, with an alternating long/short/long span pattern. A small portion of the slab on the second level connecting to the existing building is lightweight concrete on metal deck on steel framing.

The penthouse slab is 11” thick due to the larger loads present on this floor. There is an unreducible 150 psf mechanical live load present, as well as a 55 psf green roof dead load in several areas. The mechanical floor also features a ‘floating’ four inch light weight concrete on metal deck isolation slab, to prevent mechanical equipment vibrations from affecting other parts of the building. The roof slab is 10” and features several large voids. This slab has post tensioned beams 36” x 24” typical for additional span stiffness in lieu of the wide-shallow beams.

Foundations

Schnabel Engineering performed the geotechnical services on the JCA project. Reports indicated that for the purposes of shallow continuous wall footings the soil has a bearing capacity of 2 ksf, with any unsuitable conditions requiring excavation and replacement with lean concrete. Core-drilled piers ranging in diameter from 2.5’ to 7’ are located beneath every column and support much of the shallow wall footings. The soil report from Schnabel Engineering indicates that the core drilled piers have an end-bearing capacity of 80 ksf and a skin friction capacity of 800 psf. The slab on grade is 5” thick and reinforced with WWF.

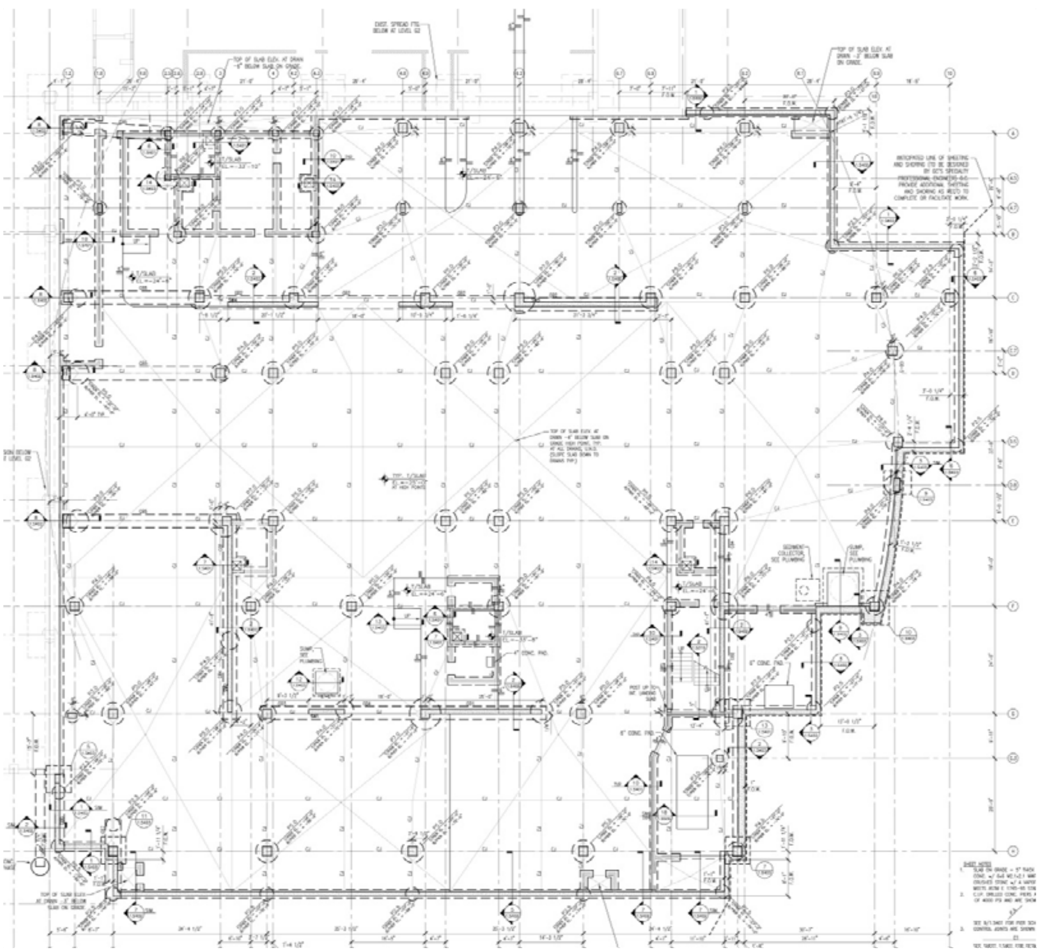

Figure 1.1-1 Foundation Layout

Framing Systems

Cast-in-place columns rise from the garage level to the roof, with the four lanterns extending the extra fourteen feet with steel framing. The column concrete has a compressive strength of 7000 psi at the base, which is reduced to 5000 psi at level 2. Typical column sizes are 24”x24”

Each lantern has a flat roof framed in structural steel with a slight slope on the edges. HSS tubes make up the columns, with the majority of the framing being small steel shapes with spans in the range of 5’ and typical sizes of L3x3x1/4, HSS4x4x1/4, and C6x13. In the center of the roof are several W12x40 girders with a maximum span of 33’ that are framed into by smaller wide flange shapes. These heavier shapes are intended to carry the photovoltaic panels mounted on top of the lanterns. Several HSS braced frames provide lateral stability to the lanterns. The lanterns were given a 30 psf dead load in the shaded region to account for the photovoltaic panels.

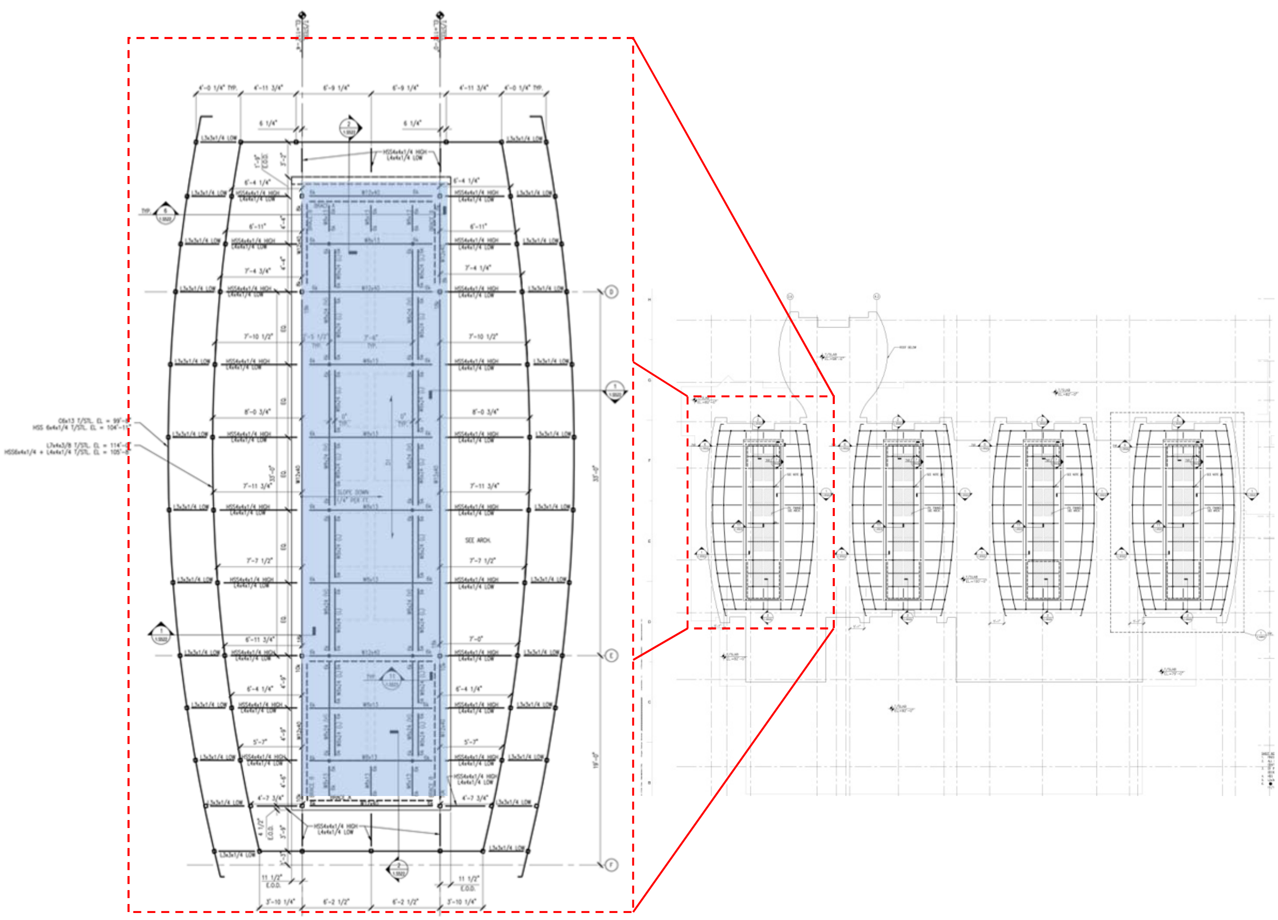

Figure 1.1-2 Lantern Framing Plan

Lateral System

The main lateral resisting elements of the JCA are the five cast-in-place reinforced concrete shear walls that rise continuously through the building. Analysis performed in Technical Report 3 showed that the concrete frames also had a significant contribution to resisting lateral loads on certain levels, particularly the frames running in the North/South direction and formed by the wide/shallow beams.

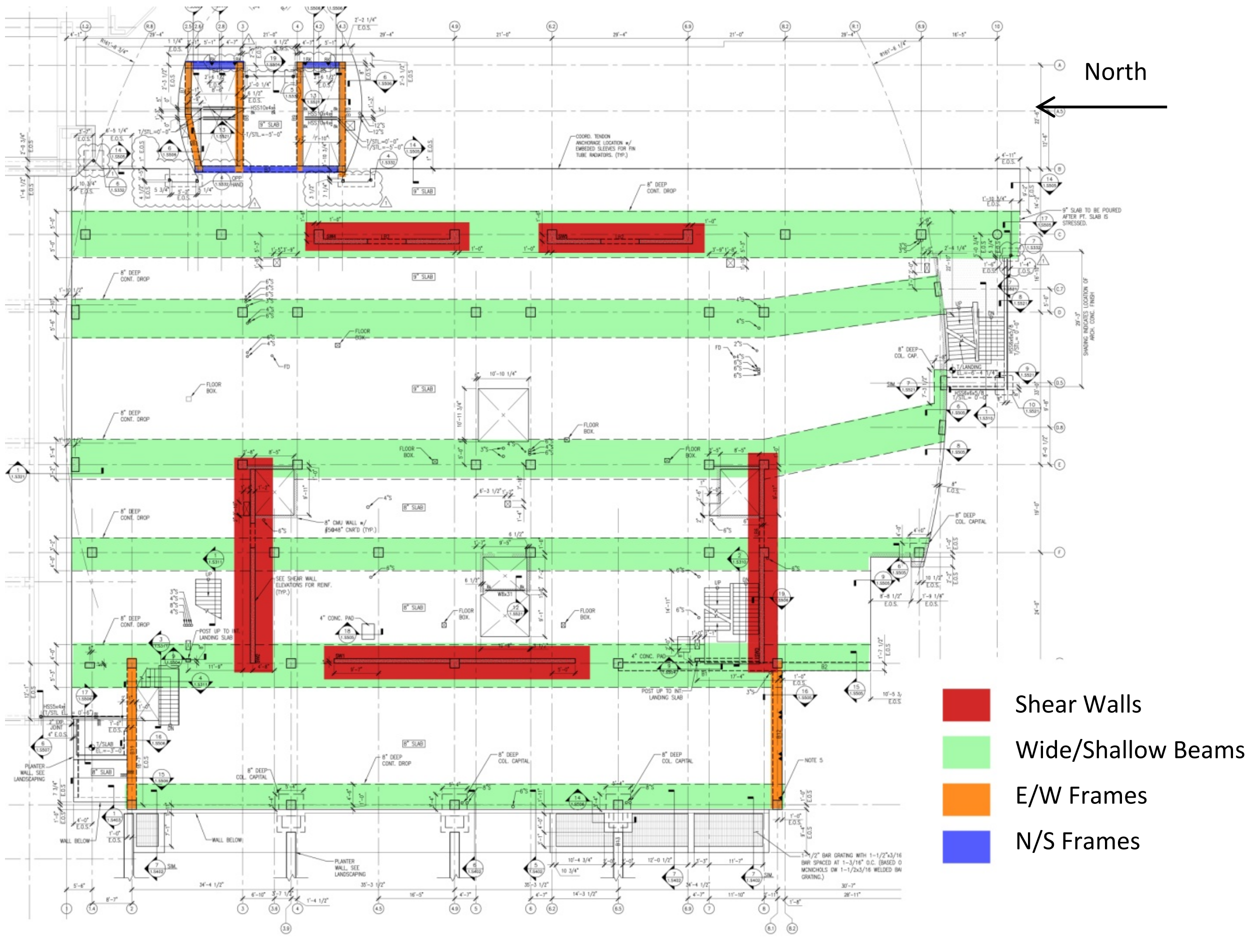

Figure 1.1-3: Lateral Elements

Roof Systems

The roof varies in height in several locations with the floor slabs described earlier in *Floor Systems*. The varying heights made snow drift a concern, and the large loads associated with the penthouse floor, which is the heaviest floor on the building, add a significant contribution to both seismic base shear and overturning. The green roof and pavers on the penthouse and upper roof levels lay overtop a hot applied fluid membrane.

Design Codes

The list of Major Codes and Standards on Sheet 1.S001 is as follows:

- 2009 International Building Code
- ACI 318-08
- AISC LRFD, 13^{th} Edition, 2005
- AWS D1.1, D1.3, D1.4, Current Edition
- ASTM, Current Edition
- Steel Deck Institute Design Manual for Composite Deck, Form Decks and Roof Decks., 2007
- ASCE 7-05 Minimum Design Loads

Materials Used

Sheet 1.S001 was used as the reference for materials used in the construction of this project and summarized in Tables 1.1-1.

Concrete		
Usage	Weight	f'c (psi)
Column (Levels 2-Rf)	Normal	5000
Column (Levels G1-1)	Normal	7000
Floor Slab	Normal	5000
Wall Footings	Normal	3000
Beams	Normal	5000
Slab on Grade	Normal	4500
Walls, Piers, & Pilasters	Normal	5000
Drilled Piers	Normal	4000
LW Concrete Fill on Deck	Light	4000
Isolation Slab @ Penthouse	Light	4000
Steel		
Type	TM Standa	Grade
W Shapes	A992	
Plates, Angles, Channels	A36	
High-Strength Bolts	325 or A490	
Anchor Rods	F1554	36
Tubes	A500	B
Pipes	A53 E or S	B
Reinforcing Steel	A615	60
Reinforcing Steel, Welded	A706	60
Roof Deck	A653	A - F
Floor Deck	A653	C, D, or E
Post-Tensioned Reinforcment	A416-96	
Masonry		
Type	TM Standa	F'm (psi)
CMU	C90	1500
Masonry Mortar	C270	
Grout	C476	
Aggregate	C404	

Table 1.1-1 Materials Used

1.2 - Gravity Loads

This section will describe how dead, live, and snow loads were calculated and compared to loadings given on the structural drawings. Three gravity checks were performed once the loadings were determined for an interior column, the typical long span for the post tensioned slab, and a doubly reinforced beam with full hand calculations available in Appendix A.

Dead and Live Loads

The dead loads listed on 1.S001 shown in Figure 7 were used for the purposes of analyses. The non-load-bearing CMU walls were assumed to be fully grouted for the purposes of worst-case load calculations. The weight of the building was calculated neglecting voids in slabs and with an assumption of 10 psf for the steel lantern framing, which would not have much effect on the building weight were it too small an assumption. The total building weight which was used for the seismic calculations was in the order of 28000 kips.

Dead Loads		
	Design	Student
Vegetated Roof	55	55
MEP/Celing	15	15
CMU Partitions	Actual Weight	91 pcf (Fully Grouted Assumption)

Table 1.2-1 Summary of Dead Loads

Based upon ASCE 7-05 the 100 psf typical live load was found to be correct, possibly for different reasons than the designer decided for, and the 40 psf holding cell load was neglected in favor of using the 100 psf live load in all locations except for the mechanical penthouse and the roof loading.

Live Loads		
	Design	ASCE 7-05
Typical	100	80 (Corrider Above First Floor) + 20 (Partition) = 100
Holding Cells	40	-
Mechanical Penthouse	150	150
Roof	-	20

Table 1.2-2 Summary of Live Loads

Snow Loads

The flat roof snow load was calculated via the method outlined in Chapter 7 of ASCE 7-05. A discrepancy arose as the importance factor, I, listed on the drawings had a value of 1.0, whereas the appropriate importance factor for an Occupancy III building is 1.1. This led to flat roof snow load value of 22 psf which differs from the calculated value of 23.1 psf. Curiously the design load is higher despite the lower importance factor which may be a result of a higher design ground snow load, though this isn't available on the drawings.

The varying roof levels led to eight different drift calculations. Figure 1.2-1 and Table 1.2-4 summarize the snow drift calculations performed.

Flat Roof Snow Load		
pf = .7 CeCtIpg > 20*I		
Ce	1	ASCE 7-05 Tab. 7-2
Ct	1	ASCE 7-05 Tab. 7-3
pg	25	ASCE 7-05 Fig. 7-1
I	1.1	ASCE 7-05 Tab. 7-4
pf =	0	
20*I=	500	
pf =	22	

Table 1.2-3 Snow Load Parameters and Flat Roof Calculation

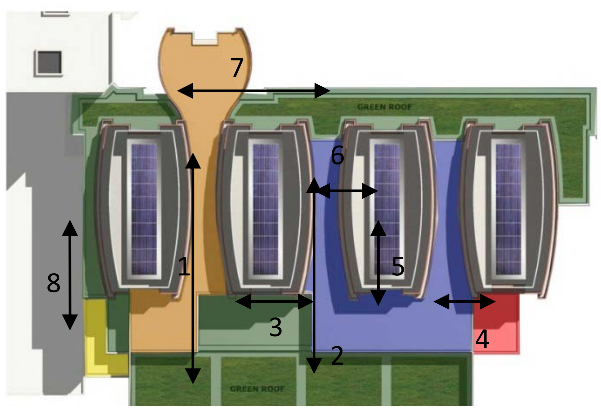

Figure 1.2-1 Roof Snow Drift Diagram

Snow Drift			γ= 17.25						
	Lu	Ll	hc	hd Lee	hd Wind		hd (ft)	w (ft)	Max psf
Drift 1	130	50	16	3.8	1.8	3.8	3.8	15.2	65.5
Drift 2	93	30.33	18	3.2	1.3	3.2	3.2	13.0	55.9
Drift 3	70	50	18	2.8	1.8	2.8	2.8	11.2	48.5
Drift 4	70	20	21	2.8	1.0	2.8	2.8	11.2	48.5
Drift 5	70	20	14	2.8	1.0	2.8	2.8	11.2	48.5
Drift 6	38	12	14	2.0	0.7	2.0	2.0	8.1	34.8
Drift 7	21	147	16	1.4	3.0	3.0	3.0	12.1	52.0
Drift 8	83	24	52	3.1	1.1	3.1	3.1	12.2	52.8

Table 1.2-4 Snow Drift Calculations

1.3 – Lateral Loads

Lateral loads were calculated for the JCA in its existing location Rockville, MD. Wind loads were calculated according to ASCE 7-05 Chapter 6 and seismic forces were calculated according to the provisions in Chapters 11 and 12. The building was modeled in ETABS, a finite element program which provided mode shapes and periods of vibration, which influenced the seismic loading.

Wind Force (NS)								
		Trib Below		Trib Above		Story	Story	Overturning
	Height	Ht	Area	Ht	Area	Force	Shear	Moment
1st	15	7.5	1125	7	1050.00	36.10	319.31	541.46
2nd	14	7	1050	7.75	1162.50	40.44	283.22	1172.76
3rd	15.5	7.75	1162.5	8.25	1237.50	46.88	242.78	2086.05
4th	16.5	8.25	1237.5	8.25	1237.50	50.83	195.90	3126.21
5th	16.5	8.25	1237.5	8.25	1237.50	53.15	145.07	4172.46
Penthouse	16.5	8.25	1237.5	9.5	1425.00	59.31	91.91	5663.87
Roof	19	9.5	1425	0	0.00	32.61	32.61	3733.42
							Base Shear (k)	319.31
							Total Overturning Moment (k-ft)	20496.21

Table 1.3-1 Wind Force NS Direction

Wind Force (EW)								
		Trib Below		Trib Above		Story	Story	Overturning
	Height	Ht	Area	Ht	Area	Force	Shear	Moment
1st	15	7.5	1350	7	1260.00	41.73	371.62	625.92
2nd	14	7	1260	7.75	1395.00	46.91	329.89	1360.46
3rd	15.5	7.75	1395	8.25	1485.00	54.50	282.98	2425.26
4th	16.5	8.25	1485	8.25	1485.00	59.19	228.48	3610.69
5th	16.5	8.25	1485	8.25	1485.00	61.98	169.29	4803.09
Penthouse	16.5	8.25	1485	9.5	1710.00	69.22	107.31	6507.11
Roof	19	9.5	1710	0	0.00	38.09	38.09	4379.98
							Base Shear (k)	371.62
							Total Overturning Moment (k-ft)	24084.13

Table 1.3-2 Wind Force EW Direction

Seismic Forces N/S (X) Direction						
Level	Story Ht (ft)	Story Weight (k)	Cvx	Story Force (k)	Shear Shear (k)	Overturning Moment (k-ft)
1	15	4421.1	0.031	23.4	755.2	350.6
2	29	4868.4	0.067	50.2	731.8	1457.2
3	44.5	4954.1	0.104	78.6	681.6	3497.7
4	61	4977.9	0.143	108.3	603.0	6607.2
5	77.5	4967.1	0.182	137.3	494.7	10639.3
PentHouse	94	6902.0	0.317	239.1	357.4	22476.6
Roof	113	3078.7	0.157	118.3	118.3	13364.6
Base Shear (k)						755.2
Total Overturning Moment (k-ft)						58393.3

Table 1.3-3 Seismic Forces NS Direction

Seismic Forces E/W (Y) Direction						
Level	Story Ht (ft)	Story Weight (k)	Cvx	Story Force (k)	Shear Shear (k)	Overturning Moment (k-ft)
1	15	4421.1	0.031	18.3	591.5	274.6
2	29	4868.4	0.067	39.4	573.2	1141.4
3	44.5	4954.1	0.104	61.6	533.9	2739.7
4	61	4977.9	0.143	84.8	472.3	5175.2
5	77.5	4967.1	0.182	107.5	387.5	8333.5
PentHouse	94	6902.0	0.317	187.3	279.9	17605.3
Roof	113	3078.7	0.157	92.6	92.6	10468.2
Base Shear (k)						591.5
Total Overturning Moment (k-ft)						45738.0

Table 1.3-4 Seismic Forces EW Direction

Modal Information, JCA Concrete		
Mode	Period	
1	1.24	Y Translational
2	1.20	Z Rotational
3	0.92	X Translational

Table 1.3-5 Modal Information JCA As Designed

1.4 –Proposal/Problem Statement

Structural Depth

The current reinforced concrete building, with post-tensioned floor slabs and cast-in-place shear walls was analyzed in three previous technical reports and found to be adequate in all respects. It is hypothesized however, that with no height restrictions, converting the building to steel would be a competitive solution.

The conversion to steel will mean changing the floor system to concrete on metal deck, employing either the composite metal deck construction with light weight concrete that was explored in Technical Report 2 or a more cost effective deck should one be found. The gravity system will be designed based upon the loading outlined in Technical Report 1, with the initial framing based upon existing locations of columns, though this may need to be adjusted as the design is further developed. Composite steel beams and girders will be used to take advantage of the slab strength so that smaller member sizes can be employed.

After the initial framing has been completed lateral loads will be recalculated using ASCE 7-05 prescribed procedures. Braced frames are proposed to replace the existing reinforced concrete shear walls, acting in their stead as the lateral system of the JCA. As with the columns, the initial trials will use the locations of the shear walls to place the braced frames, to minimize architectural impacts and due to the symmetrical layout that did not have torsion issues as reported in Technical Report 3.

Once both gravity loads and lateral loads have been recalculated the existing foundation system will be investigated to see if it can be reduced to a more efficient solution.

As seismic design is of particular interest to part 2 of the proposal will involve moving the building to San Francisco where it will be in Seismic Design Category D. The system will be kept as steel braced frames and the lateral system will be redesigned for the larger seismic loading.

Breadth Study One: Cost and Schedule Analysis

Breadth One will explore a common question in today's industry, "Concrete or Steel?", by evaluating the impacts that changing the system will have on the overall cost and schedule of building. Often designers will push one concrete and one steel solution deep into the design phase before one ends up being chosen, a scenario being emulated by the Structural Depth. The object here is to see if the redesign will lead to a cheaper, faster to construct building that performs on par with the concrete design, and determine if steel was truly a feasible solution for this project. This depth requires that a schedule be established for both the existing construction and the redesign and that both options be priced based upon their materials, associated construction costs, and schedules; the better option will therefore be based upon which structure is completed quicker and for less cost. The critical path of the building will be reevaluated and the cost impact of schedule days included in the evaluation of both systems.

Breadth Study Two: Sustainability

The JCA has achieved a LEED Gold rating which was in part made possible by the sustainable rooftop features. There is approximately 6000 sq ft of green roof as well as photovoltaic panels on the various levels of the roof. It was thought that perhaps utilizing the entire space for PV panels could prove more beneficial. Therefore a life cycle analysis would be performed on both systems, taking into account payback period, carbon output and other factors.

Additionally a comparison of their LEED impacts would be evaluated as well. Areas of possible improvement in LEED rating not related to the green roof and PV panels would also be explored. A summary of the findings will be provided.

Chapter 2 – Structural Depth

The Judicial Center Annex is a reinforced concrete structure located in Rockville, MD. As the building is owned by Montgomery County normal height restrictions have been waived and the building features larger than typical floor to floor heights. A typical advantage of concrete construction is the thinner structural framing which allows either for additional floors in a given height or a height reduction for a given number of floors, allowing for more profitable space or less building envelope for the owner of the building.

For this reason it is thought that, despite being an area typically dominated by concrete construction, a design in steel would be a competitive solution. The proposal is therefore to perform a system redesign of the JCA using steel framing. The floor slab will be converted from post-tensioned to a composite slab on metal deck. Gravity members will be designed as composite in an effort to keep the framing shallow. The lateral system which was made of shear walls will be converted to braced frames.

The new structural system is anticipated to be much lighter than the existing system. This makes it likely that wind will control the design which was previously dominated by seismic. As seismic design is of particular interest to the author a further step was proposed for the structural design. The building will be 'moved' to San Francisco, CA. The west coast is well known for its greater seismicity, so making this move will result in an exploration into seismic design.

2.1 – MD Gravity Design

For this redesign it was attempted to leave the architecture and layout of spaces as unchanged as possible. The structural grid was therefore kept largely unchanged, choosing to keep column locations intact and to work around the existing building. Figure 2.1-1 shows the finalized structural grid with the girders running in the North/South direction. The original thought was that economy could be achieved as the smaller bays on either side of the typical design bay shown in the figure would not require an infill beam and therefore there was the potential for fewer beams. Also of note is that one column was moved and one column was added to the layout as highlighted in Figure 2.1-1. In both cases the architectural plans were checked to ensure that it was possible to do so with little or no impact.

RAM Structural was the primary software used for the design of the gravity system, and with the ability to easily change the framing and determine if the earlier hypothesis was in fact more economical two RAM models were created with girders running in either direction. As hypothesized the N/S girder design was more economical in terms of steel tonnage, though it turns out less pieces were used framing it in the E/W direction. In order to accommodate the 12' bays with no infill beams the deck selected was 2VLI18 to allow for un-shored construction which would have the potential for cost savings. This was increased to a 3VLI16 for the penthouse level. The gravity design was compared to the typical bay designed by hand as well as a gravity check in ETABS. The numbers were all found to be satisfactorily close. Appendix D contains the hand check.

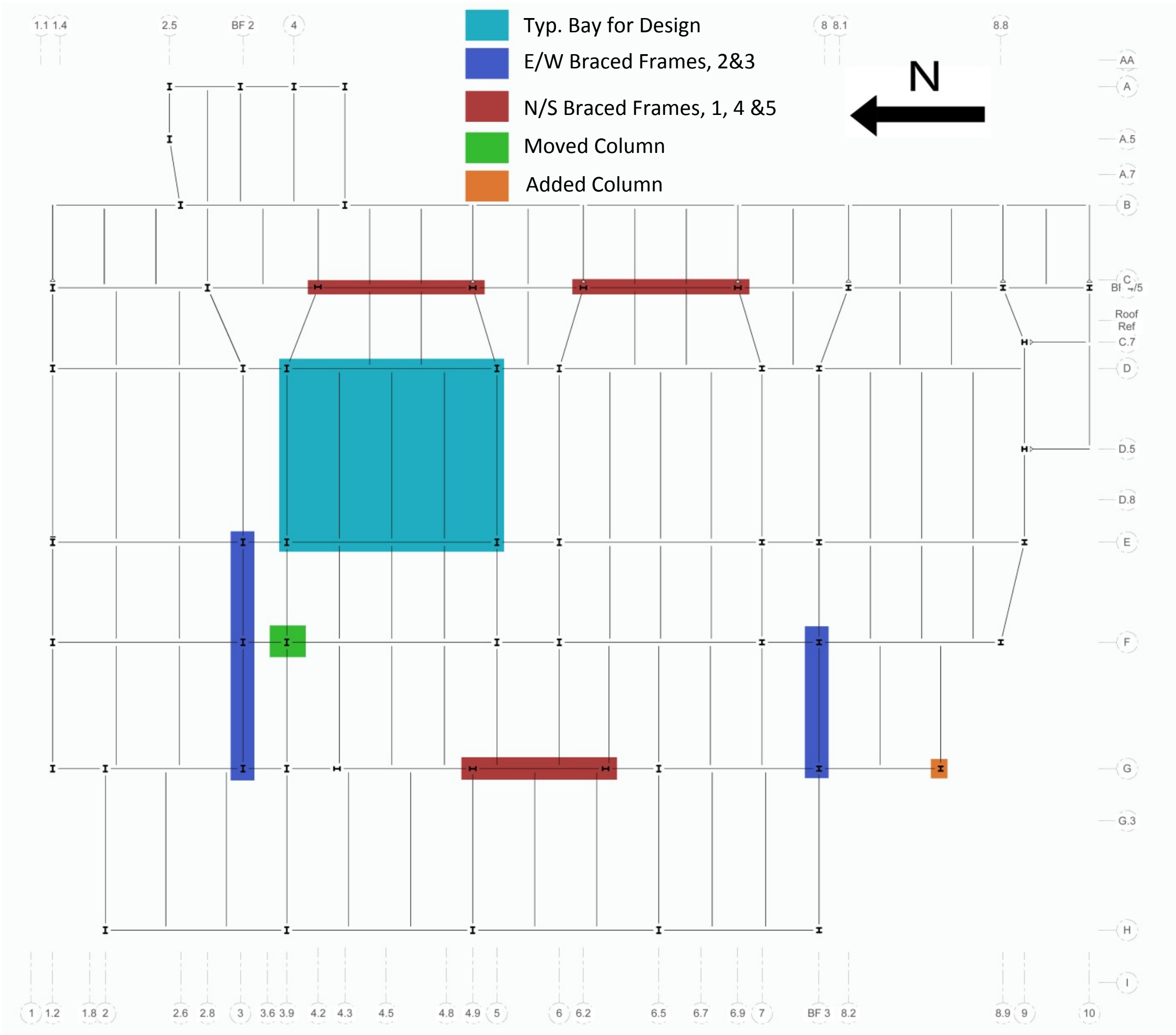

Figure 2.1-1 Steel Gravity System Layout

Typically steel results in larger floor-to-floor heights then concrete due to deeper gravity framing. The steel as designed resulted in some girders and cantilevered beams being as deep as 30” for a typical floor, which combined with the slab led to nearly a 1.5’ increase from the concrete framing. Therefore it was important to investigate to the plenum space to determine if a height increase was required. Additionally this is an addition onto an existing building which required the Terrace, 1st, and 2nd levels to remain at the same level. The mechanical and architectural plans were investigated, looking for typical ceiling heights and the largest ductwork. Sections revealed that the deepest ducts rarely surpassed 20” which could be accommodated by the general 4.5’ existing plenum even with the increase in member depth. Large duct runs also ran parallel to the girders meaning that the worst case ductwork ran under shallower beams. To ensure that this would not be a problem and to provide more clearance at the garage level beams were limited to W24’s. The courtroom spaces on the 3rd through

5th levels, while not featuring large ductwork, were of concern so these levels were increased by 6" each resulting in an overall building height increase of 1.5'.

2.2 – MD Lateral System & Foundation Design

The first step of the lateral system redesign was recalculating the lateral wind and seismic loading. As the building system had changed markedly the weight of the structure needed recalculating to determine the new seismic loads. The wind load was also recalculated, though the 1.5' increase in height did not make a large difference. The terrace level was used as the seismic base.

Seismic

The weight of the building was recalculated to approximately 15500 kips. This meant that the building mass affecting the seismic forces had changed from approximately 160 psf to 80 psf. The Equivalent Lateral Force Procedure as detailed in ASCE7-05 section 12.8 was used to calculate story forces that would represent the inertial response of the building due to seismic loading. The seismic parameters are shown in Table 2.2-1 and the story forces and shears are shown in Table 2.2-2.

Seismic Design Parameters		
I	1.25	
R	3	
SDs	0.1664	
SD1	0.0816	
Ct	0.02	
x	0.75	
hn	114	
Ta	0.707	
Cu	1.7	
T	1.202	
	SDS/(R/I)	SD1/(T(R/I))
Cs	0.069	0.028
W (kips)	15448	
V (kips)	437	

Table 2.2-1 Seismic Design Parameters

Seismic Forces, Both Directions						
Level	Story Ht (ft)	Story Weight (k)	Cvx	Story Force (k)	Shear Shear (k)	Overturning Moment (k-ft)
1	15	2133.8	0.033	14.5	441.3	218.1
2	29	2130.8	0.064	28.1	426.8	813.7
3	44.5	2177.3	0.100	44.1	398.7	1962.1
4	61.5	2207.1	0.140	61.9	354.6	3804.1
5	78.5	2207.1	0.179	79.0	292.8	6197.9
PentHouse	95.5	3207.8	0.328	144.9	213.8	13839.7
Roof	114.5	1383.7	0.156	68.9	68.9	7889.9
					Base Shear (k)	441.3
					Total Overturning Moment (k-ft)	34725.6

Table 2.2-2 Seismic Story Forces and Story Shears

Wind

Method 2 of the Main Wind Force Resisting System (MWRFS) procedure detailed in ASCE7-05 chapter 6 was used to calculate the wind forces the building would see. The building was idealized as a rectangle for simplicity and the lanterns were excluded from the calculation as they represent a relatively small area compared to the rest of the building. Tables 2.2-3 through 2.2-8 summarize the wind calculations.

Wind Load Criteria		
Gcpi	0.18	ASCE 7-05 Fig. 6-5
Exposure	B	ASCE 7-05 6.5.6.3
V	90 mph	ASCE 7-05 Fig. 6-1C
I	1.15	ASCE 7-05 Tab 6-1
Kzt	1	ASCE 7-05 6.5.7.1
Kd	0.85	ASCE 7-05 Fig. 6-4

Table 2.2-3 Wind Load Criteria

Velocity Presssure Coefficients (Kz) and Velocity Pressures (qz)			
	Height	Kz	qz
1st	15	0.570	11.55
2nd	29	0.692	14.03
3rd	44.5	0.783	15.87
4th	61.5	0.856	17.35
5th	78.5	0.924	18.73
Penthouse	95.5	0.982	19.90
Roof	114.5	1.026	20.80

Table 2.2-4 Kz and qz Values

Design Wind Pressure N/S							
		Distance	Wind Pressure	Internal Pressure		Net Pressure	
				(+) Gcpi	(-) Gcpi	(+) Gcpi	(-) Gcpi
Windward	1st	15	7.86	3.74	-3.74	4.11	11.60
	2nd	29	9.54	3.74	-3.74	5.79	13.28
	3rd	44.5	10.79	3.74	-3.74	7.05	14.54
	4th	61.5	11.80	3.74	-3.74	8.06	15.54
	5th	78.5	12.74	3.74	-3.74	8.99	16.48
	Penthouse	95.5	13.54	3.74	-3.74	9.79	17.28
	Roof	114.5	14.14	3.74	-3.74	10.40	17.88
Leeward	All	-	-8.74	3.74	-3.74	-12.48	-5.00
Side Walls	All	-	-12.24	3.74	-3.74	-15.98	-8.50
Roof		0 - 50	-18.19	3.74	-3.74	-21.93	-14.45
		> 50	-14.55	3.74	-3.74	-18.29	-10.81

Table 2.2-5 Design Wind Pressure in the North/South Direction

Design Wind Pressure E/W							
		Distance	Wind Pressure	Internal Pressure		Net Pressure	
				(+) Gcpi	(-) Gcpi	(+) Gcpi	(-) Gcpi
Windward	1st	15	7.86	3.74	-3.74	4.11	11.60
	2nd	29	9.54	3.74	-3.74	5.79	13.28
	3rd	44.5	10.79	3.74	-3.74	7.05	14.54
	4th	61	11.80	3.74	-3.74	8.06	15.54
	5th	77.5	12.74	3.74	-3.74	8.99	16.48
	Penthouse	94	13.54	3.74	-3.74	9.79	17.28
	Roof	115	14.14	3.74	-3.74	10.40	17.88
Leeward	All	-	-8.13	3.74	-3.74	-11.87	-4.39
Side Walls	All	-	-12.37	3.74	-3.74	-16.12	-8.63
Roof		0 - 50	-16.76	3.74	-3.74	-20.50	-13.01
		> 50	-15.49	3.74	-3.74	-19.23	-11.74

Table 2.2-6 Design Wind Pressure in the North/South Direction

Wind Force (NS)								
		Trib Below		Trib Above		Story	Story	Overturning
	Height	Ht	Area	Ht	Area	Force	Shear	Moment
1st	15	7.5	1125	7	1050.00	36.10	324.03	541.46
2nd	14	7	1050	7.75	1162.50	40.44	287.94	1172.76
3rd	15.5	7.75	1162.5	8.5	1275.00	47.61	247.50	2118.64
4th	17	8.5	1275	8.5	1275.00	52.37	199.89	3220.94
5th	17	8.5	1275	8.5	1275.00	54.76	147.51	4298.90
Penthouse	17	8.5	1275	9.5	1425.00	60.14	92.75	5743.64
Roof	19	9.5	1425	0	0.00	32.61	32.61	3733.42
							Base Shear (k)	324.03
							Total Overturning Moment (k-ft)	20829.75

Table 2.2-7 Wind Story Forces and Story Shears in the North/South Direction

Wind Force (EW)								
		Trib Below		Trib Above		Story	Story	Overturning
	Height	Ht	Area	Ht	Area	Force	Shear	Moment
1st	15	7.5	1350	7	1260.00	41.73	377.12	625.92
2nd	14	7	1260	7.75	1395.00	46.91	335.39	1360.46
3rd	15.5	7.75	1395	8.5	1530.00	55.35	288.48	2463.15
4th	17	8.5	1530	8.5	1530.00	60.99	233.13	3720.11
5th	17	8.5	1530	8.5	1530.00	63.85	172.14	4948.63
Penthouse	17	8.5	1530	9.5	1710.00	70.20	108.29	6598.76
Roof	19	9.5	1710	0	0.00	38.09	38.09	4379.98
							Base Shear (k)	377.12
							Total Overturning Moment (k-ft)	24474.13

Table 2.2-8 Wind Story Forces and Story Shears in the East/West Direction

The seismic forces were still found to generate a higher un-factored base shear. It can be seen though that the strength design of the structure will in general be controlled by the wind forces. Seismic will likely control drift as it will be modified by Cd/I and drift due to wind can be reduced by a factor of 0.7 according to ASCE7-05 load combination CC-3.

Braced Frame Design

As shear walls were previously used these same spaces would easily be able to accommodate a concentrically braced frame. A concentrically braced frame is a system in which the members resist lateral loads in the elastic range primarily through axial forces in the members. The members are connected with little or no eccentricity which creates a very stiff and efficient system. As the JCA was located in Seismic Design Category B the frames were designed as Ordinary Concentrically Braced Frames (OCBF) with an R = 3 to avoid special detailing requirements.

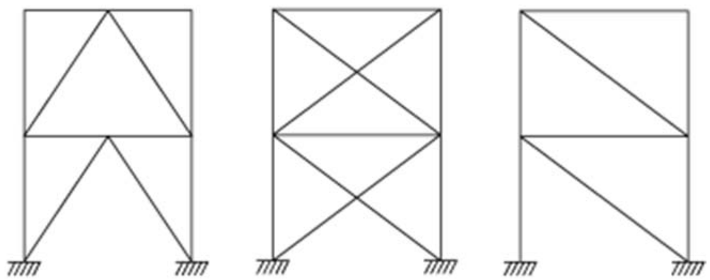

Figure 2.2-1 Concentrically Braced Frame Configurations, Source: structuremag.org

Before modeling the building in ETABS, preliminary layouts and sizes were determined. Sizes were found assuming a percentage of the base shear that frame would see and assuming the brace would take all of the force. Braced frames 4 and 5 had a central doorway on almost every level which necessitated the usage of an inverted V, or Chevron, configuration to accommodate this. The chevron configuration was explored for the other braced frames as well. The initial thought was the shorter unbraced length of the column would prove beneficial in increasing Fcr and the capacity of the bracing members. However, the larger angle caused the axial component of the shear that the brace would experience to be much larger in the chevron configuration which negated this previous advantage. The chevron would also mean an additional piece which would require connections for both ends which would drive the cost up. Table 2.2-9 shows a frame that is representative of previously described comparison. Braced members were chosen to be square HSS tubes whenever possible. Their shape makes them efficient in compression because they have no weak axis and are easy to connect.

Brace Configuration Comparison						
	Member	Wt	L	Connections	Pieces	Equivalent Wt.
Chevron	HSS7x7x3/8	32.6	42.8	4	2	2195
Diagonal	HSS8x8x1/2	48.8	31.1	2	1	1918

Table 2.2-9 BF 1, Story 5 Brace Economy Comparison
***Note: Connection assumed equivalent to 200lbs of steel**

.

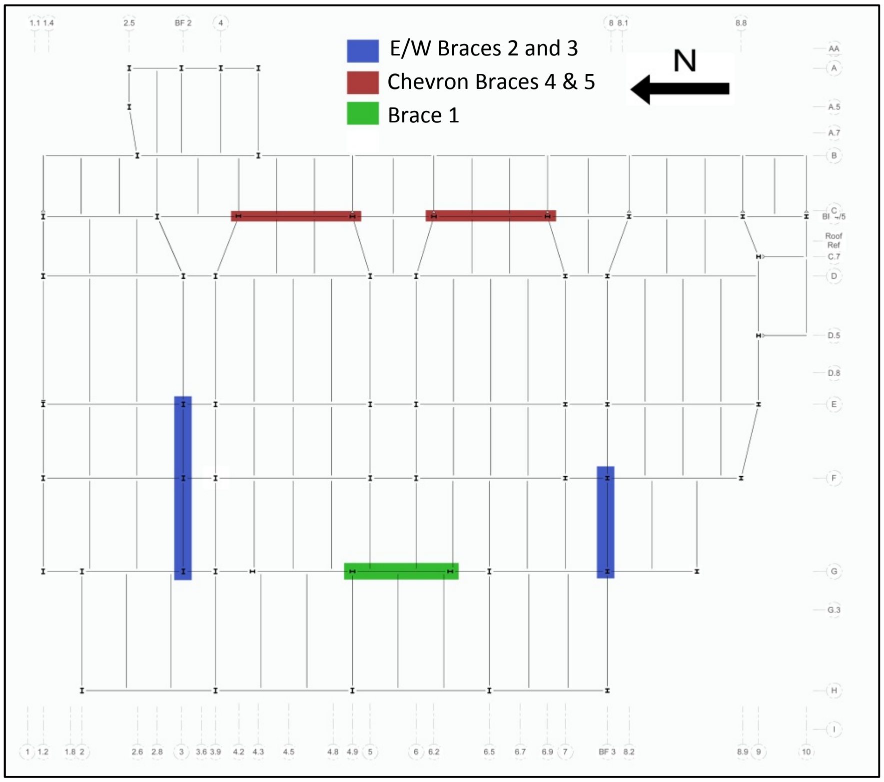

Figure 2.2-2 BF Layout

ETABS, a finite element program, was used to model the structural system. Only the lateral system was modeled, which is an acceptable simplification. Centerline modeling was used and since connections were modeled as pinned (no moment frames) Panel Zones were not explicitly modeled and a rigid end offset factor was kept as 0. The diaphragms were modeled as rigid and the mass of each story was assumed lumped at the respective diaphragm. The X-direction in the model corresponds to the N/S direction in plan.

MD Modes and Participating Mass				
Mode	Period	UX	UY	
1	2.61	3.67	59.94	Y-Translational
2	2.45	35.82	8.69	Z-Rotational
3	2.02	32.10	0.23	X-Translational

Table 2.2-10 Mode Shapes and Participating Mass

Table 2.2-10 shows the building period for the first three modes. The number seems unusually high which may be a result of a very flexible system. As the periods for the motion in both the X and Y directions were found to be greater than Cu*Ta the seismic forces required no adjustment.

The initial sizes were downsized as much as the strength design would allow, but drifts still easily passed the requirements. Tables 2.2-11 through 14 show the worst case seismic and wind drift for both directions.

QCXE								
				Amplified by Cd/I				Δa = .015sx
Story	Height	δxe	δye	δx	δy	Δx	Δy	
Roof	19.50	2.56	-	6.66	-	1.38	-	3.51
Penthouse	18.50	2.03	-	5.28	-	1.07	-	3.33
Level 5	18.00	1.62	-	4.21	-	1.07	-	3.24
Level 4	18.00	1.21	-	3.15	-	1.07	-	3.24
Level 3	17.00	0.80	-	2.08	-	0.88	-	3.06
Level 2	15.50	0.46	-	1.20	-	0.73	-	2.79
Level 1	16.60	0.18	-	0.47	-	0.42	-	2.99
Terrace	11.50	0.02	-	0.05	-	0.05	-	2.07

Table 2.2-11 Deflection and Story Drift Due to Seismic Forces Applied in the N/S + Eccentricity

QCY								
				Amplified by Cd/I				Δa = .015sx
Story	Height	δxe	δye	δx	δy	Δx	Δy	
Roof	19.50	-	2.70	-	7.02	-	1.22	3.51
Penthouse	18.50	-	2.23	-	5.80	-	1.27	3.33
Level 5	18.00	-	1.74	-	4.52	-	1.30	3.24
Level 4	18.00	-	1.24	-	3.22	-	1.17	3.24
Level 3	17.00	-	0.79	-	2.05	-	1.01	3.06
Level 2	15.50	-	0.40	-	1.04	-	0.73	2.79
Level 1	16.60	-	0.12	-	0.31	-	0.26	2.99
Terrace	11.50	-	0.02	-	0.05	-	0.05	2.07

Table 2.2-12 Deflection and Story Drift Due to Seismic Forces Applied Directly in the E/W Direction

WC2XEA						
Story	Height	δxw	δyw	Δx	Δy	Δa = H/400
Roof	19.50	1.88	-	0.38	-	0.59
Penthouse	18.50	1.5	-	0.28	-	0.56
Level 5	18.00	1.22	-	0.29	-	0.54
Level 4	18.00	0.93	-	0.29	-	0.54
Level 3	17.00	0.64	-	0.27	-	0.51
Level 2	15.50	0.37	-	0.21	-	0.47
Level 1	16.60	0.16	-	0.15	-	0.50
Terrace	11.50	0.01	-	0.01	-	0.35

Table 2.2-13 Deflection and Story Drift Due to Wind Forces: Wind Case 2 N/S Direction, Positive Eccentricity

WC2YEB						
Story	Height	δxw	δyw	Δx	Δy	Δa = H/400
Roof	19.50	-	2.49	-	0.43	0.59
Penthouse	18.50	-	2.06	-	0.44	0.56
Level 5	18.00	-	1.62	-	0.42	0.54
Level 4	18.00	-	1.20	-	0.41	0.54
Level 3	17.00	-	0.79	-	0.38	0.51
Level 2	15.50	-	0.41	-	0.28	0.47
Level 1	16.60	-	0.13	-	0.12	0.50
Terrace	11.50	-	0.01	-	0.01	0.35

Table 2.2-14 Deflection and Story Drift Due to Wind Forces: Wind Case 2 E/W Direction, Negative Eccentricity

Foundations

Because the system was significantly lighter a foundation redesign was considered. The Geotech report provided by Schnabel Engineering, Inc. gave an end-bearing value of 80 ksf for the core drilled piers, as well as a skin friction value of 800 psf. These allowances were assumed to already account for a Factor of Safety. The gravity column loading from RAM and the lateral loads on the columns integrated into the braced frames were then used to re-size the core drilled piers, with a minimum diameter of 2.5’ per Schnabel’s recommendation. The pier sizing is shown in Appendix H.

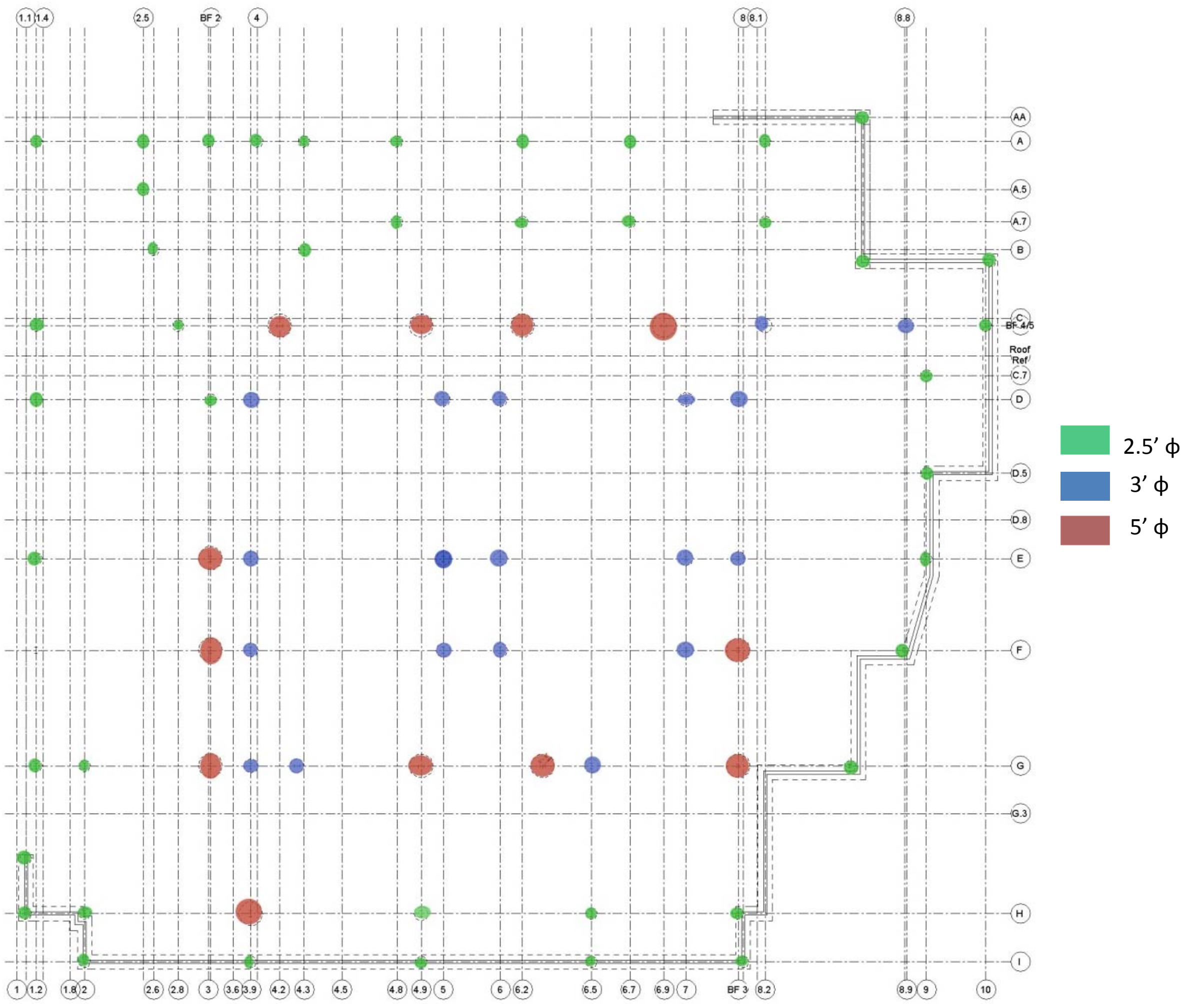

Figure 2.2-3 Core Drilled Pier Sizes and Locations

Summary

The braced frames were able to adequately replace the former shear wall system. Torsional irregularities were not considered in the scope of the first part of this depth as it is not required in SDC B. The weight in steel of the redesign came in at 9.9 lbs/sq ft, with 15% of that accounted for by the lateral system. Figures 2.2-5 through 2.2-8 show the elevations of the braced frames with the sections.

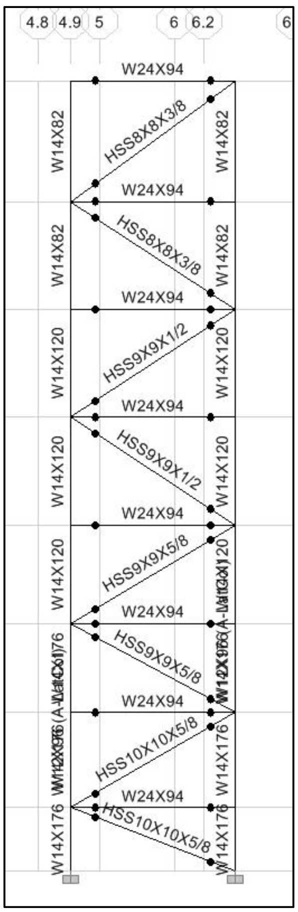

Figure 2.2-4 BF 1

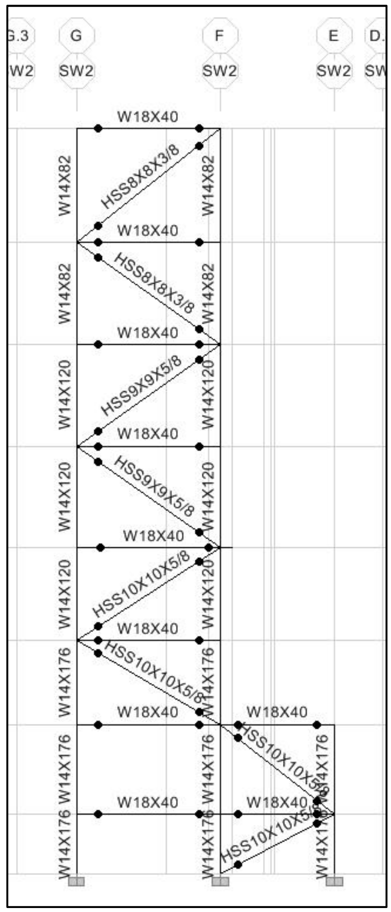

Figure 2.2-5 BF 2

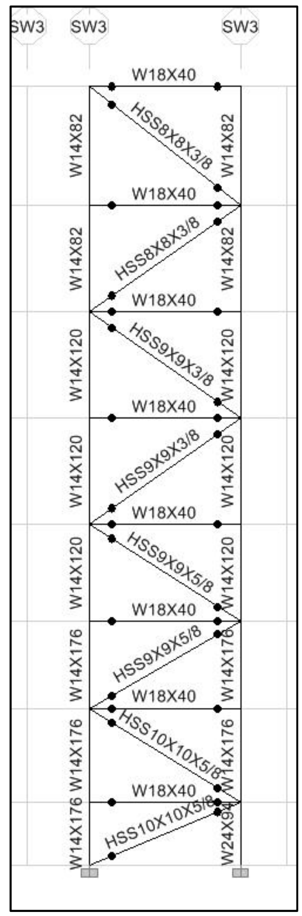

Figure 2.2-6 BF 3

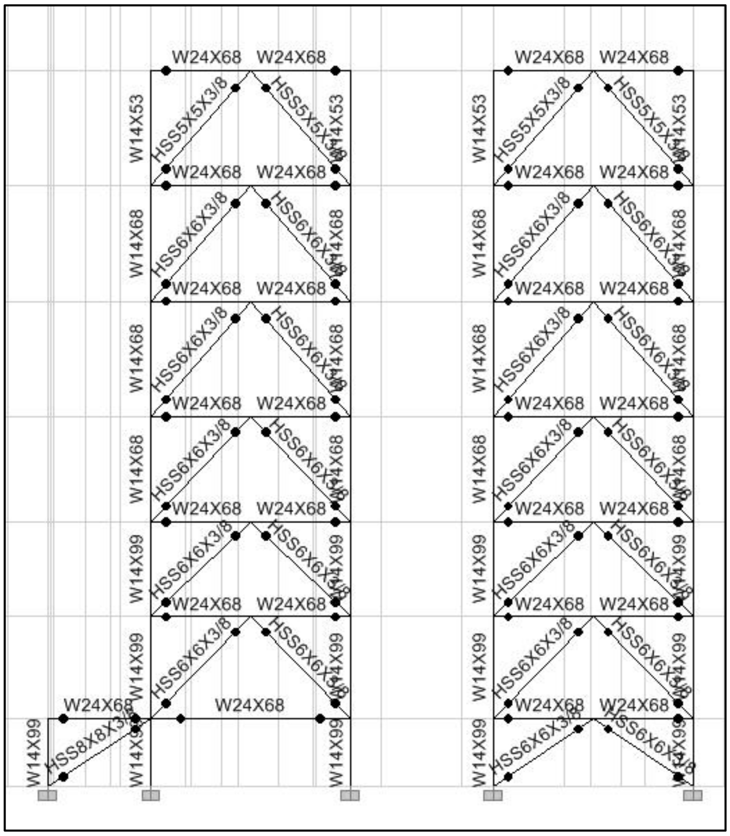

Figure 2.2-8 BF 4 & 5

2.3 – CA Lateral Design, Layout 1

The second phase to be investigated for the structural depth is the move to a region of greater seismicity, which in this case was arbitrarily chosen as San Francisco. Assuming Site Class D the Seismic Design Category increased from B to D. The change in SDC results in a host of provisions from ASCE7-05 needing accounted for that did not previously apply.

One such factor is ρ, the redundancy factor, which drove the seismic exploration. ASCE7-05 section 12.3.4.2 requires that the horizontal and vertical seismic load effects be multiplied by 1.3 unless stories which resist more than 35% of the base shear do not face a 33% strength reduction by the removal of an individual brace or develop an extreme torsion irregularity due to this removal. The current layout would not have a chance of earning this as in the East/West direction there are two frames, so removing one brace would drop the strength by 50%. To remedy this frames would be added in an effort to keep ρ = 1, but for comparison purposes the braces would be sized using the layout from Rockville initially.

Seismic

The move to SDC D resulted in a large markup in seismic forces as seen in Table 2.3-1. This also meant that the OCBF could not be utilized and the system would need to be changed to a Special Concentrically Braced Frame, SCBF, which results in an R=6. The cost of the increased ductility comes in the form of special detailing requirements and seismic provisions that will be discussed more during the frame design section.

Seismic Design Parameters		
I	1.25	
R	6	
SDs	1	
SD1	0.6	
Ct	0.02	
x	0.75	
hn	114	
Ta	0.70	
Cu	1.7	
T	1.19	
	SDS/(R/I)	SD1/(T(R/I))
Cs	0.208	0.105
W (kips)	15448	
V (kips)	1623	

Table 2.3-1 Seismic Parameters

Due to the large loading it was thought prudent to invoke ASCE7-05 section 12.9 and perform a Modal Response Spectrum Analysis. The number of modes used was dictated by having over 90% modal mass participation in both directions. Section 11.4.5 was used to determine the design spectrum as shown in Figure 2.3-1, and the response parameters were combined using square root of the sum squares method, which is included in Appendix J. The resultant base shear in both directions was limited to a reduction of 85% of the base shear calculated using the Equivalent Lateral Force Procedure. Table 2.3-2 shows the revised lateral forces the building experiences do to seismic response.

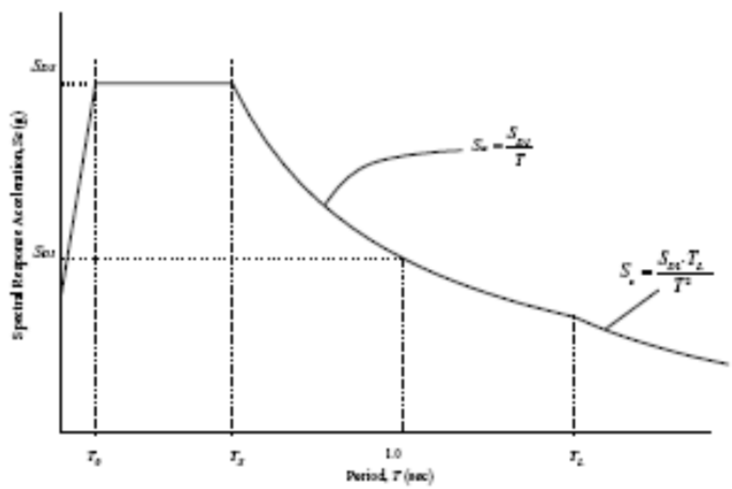

Figure 2.3-1 Design Response Spectrum, ASCE7-05

Seismic Forces, Both Directions						
Level	Story Ht (ft)	Story Weight (k)	Cvx	Story Force (k)	Shear Shear (k)	Overturning Moment (k-ft)
1	15	2133.8	0.033	45.6	1383.7	683.7
2	29	2130.8	0.064	88.0	1338.1	2551.3
3	44.5	2177.3	0.100	138.2	1250.1	6151.7
4	61.5	2207.1	0.140	193.9	1111.9	11927.0
5	78.5	2207.1	0.179	247.5	917.9	19432.1
PentHouse	95.5	3207.8	0.328	454.4	670.4	43391.3
Roof	114.5	1383.7	0.156	216.0	216.0	24737.0
					Base Shear (k)	1383.7
					Total Overturning Moment (k-ft)	108874.1

Table 2.3-2 Seismic Parameters

Wind

The design wind velocity for California is reduced from 90 mph to 85 mph. As the geometry of the building is assumed unaffected by the move this resulted in seismic controlling both strength and drift design by a large margin. The design wind forces can be seen in Tables 2.3-3 and 2.3-4.

Wind Force (EW)								
		Trib Below		Trib Above		Story	Story	Overturning
	Height	Ht	Area	Ht	Area	Force	Shear	Moment
1st	15	7.5	1350	7	1260.00	37.22	336.38	558.31
2nd	14	7	1260	7.75	1395.00	41.84	299.16	1213.49
3rd	15.5	7.75	1395	8.5	1530.00	49.37	257.31	2197.07
4th	17	8.5	1530	8.5	1530.00	54.40	207.94	3318.24
5th	17	8.5	1530	8.5	1530.00	56.96	153.54	4414.06
Penthouse	17	8.5	1530	9.5	1710.00	62.62	96.59	5885.93
Roof	19	9.5	1710	0	0.00	33.97	33.97	3906.84
							Base Shear (k)	336.38
							Total Overturning Moment (k-ft)	21830.32

Table 2.3-3 Wind Forces, EW Direction

Wind Force (NS)								
		Trib Below		Trib Above		Story	Story	Overturning
	Height	Ht	Area	Ht	Area	Force	Shear	Moment
1st	15	7.5	1125	7	1050.00	34.25	304.18	513.77
2nd	14	7	1050	7.75	1162.50	38.16	269.93	1106.65
3rd	15.5	7.75	1162.5	8.5	1275.00	44.77	231.77	1992.18
4th	17	8.5	1275	8.5	1275.00	49.12	187.00	3021.06
5th	17	8.5	1275	8.5	1275.00	51.25	137.88	4023.50
Penthouse	17	8.5	1275	9.5	1425.00	56.20	86.62	5366.63
Roof	19	9.5	1425	0	0.00	30.43	30.43	3484.16
							Base Shear (k)	304.18
							Total Overturning Moment (k-ft)	19507.97

Table 2.3-4 Wind Forces, NS Direction

Braced Frame Design

As before preliminary sizes were chosen on assuming a frame stiffness and sizing the braces for the entire story shear that brace would potentially see. As braced frames are now SCBF other provisions applied per AISC 341-05: Seismic Provisions for Structural Steel Buildings. Several provisions in particular are of note.

- 13.2a Slenderness – Bracing members shall have $Kl/r \leq 4\sqrt{E/F_y}$
  - This meant that the largest HSS shape possible was a 12x12x5/8 and that W Shapes would need employed
- 13.3 Required Strength of Bracing Connections

$$Tensile\ Str = \ R_y F_y A_g \quad Compr\ Str = 1.1 R_y P_n \quad Flexural\ Str = 1.1 R_y M_p$$

  - In a SCBF system the energy dissipation is assumed to occur through tensile yielding and buckling of the bracing members, whilst the rest of the system remains elastic. To achieve this connections must be designed to withstand larger forces than in an OCBF
- 13.4a Inverted V-Type Bracing – For loading acting on the member

$$Brace\ Tensile\ Str = \ R_y F_y A_g \text{ , } Compr\ Str = 0.3 P_n$$

  - Inverted V-Type connections are typically avoided in seismic regions due to the unbalanced compression and tensile forces that are developed in the braces. This causes potentially damage due to large midspan deflections unless properly accounted for. As a result the beam must be oversized to deal with this unbalanced load and can become extremely large, negatively affecting the building in terms of framing depth and cost of steel.

Unlike Maryland, the chevron configuration is extremely undesirable in this higher SDC. Due to the geometry however, the chevrons were kept in braced frames 4 & 5. The beam was sized and the brace to beam connection designed based upon the above factors to satisfy MAE requirements. Figure 2.3-2 shows the connection details.

Strength design drove the member sizes initially as the redundancy factor led the braces to see 30% more force. Once strength design was found adequate the building was checked for torsional irregularities. Table 12.3-2 of ASCE7-05 defines a torsional irregularity as when the maximum story drift of a level exceeds 1.2 times the average. Initial findings denoted that the design was irregular. This invoked section 12.12.1 which stated that the story drift now had to be taken as the largest different between the edges at the top and bottom of the story under consideration rather than the center of the diaphragm. Drift levels failed considerably at this point, and an effort was made to control the torsion such that the center of the diaphragm displacements could be considered for story drift. The end story drifts are summarized in Table 2.3-5 and 2.3-6.

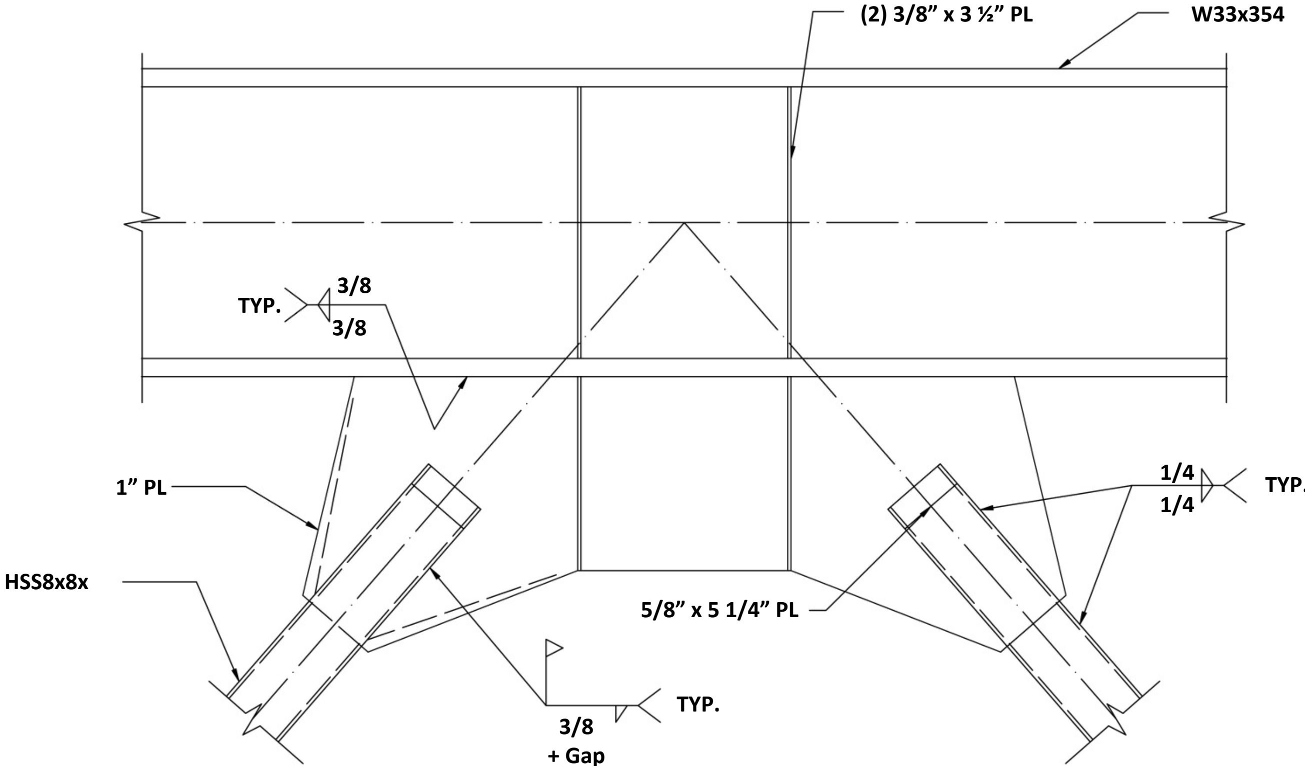

Figure 2.3-2 Designed Chevron SCBF Brace to Beam Connection

Torsional Irregularity X Direction									
Story	δmax	δmin	δavg	δmax/δavg	Δ1	Δ2	Δavg	Δmax/Δavg	Ax
Roof	4.53	3.13	3.83	1.18	0.84	0.69	0.77	1.1	0.97
Penthouse	3.69	2.44	3.065	1.20	0.79	0.50	0.65	1.2	1.01
Level 5	2.9	1.94	2.42	1.20	0.78	0.51	0.65	1.2	1.00
Level 4	2.12	1.43	1.775	1.19	0.74	0.50	0.62	1.2	0.99
Level 3	1.38	0.93	1.155	1.19	0.58	0.39	0.49	1.2	0.99
Level 2	0.8	0.54	0.67	1.19	0.45	0.31	0.38	1.2	0.99
Level 1	0.35	0.23	0.29	1.21	0.35	0.23	0.29	1.2	1.01

Table 2.3-5 Torsional Irregularity Check X Direction

Torsional Irregularity Y Direction									
Story	δmax	δmin	δavg	δmax/δavg	Δ1	Δ2	Δavg	Δmax/Δavg	
Roof	5.31	4.38	4.845	1.10	0.97	0.85	0.91	1.1	0.83
Penthouse	4.34	3.53	3.935	1.10	0.95	0.82	0.89	1.1	0.84
Level 5	3.39	2.71	3.05	1.11	0.95	0.82	0.89	1.1	0.86
Level 4	2.44	1.89	2.165	1.13	0.85	0.75	0.80	1.1	0.88
Level 3	1.59	1.14	1.365	1.16	0.69	0.59	0.64	1.1	0.94
Level 2	0.9	0.55	0.725	1.24	0.51	0.25	0.38	1.1	1.07
Level 1	0.39	0.3	0.345	1.13	0.39	0.30	0.35	1.1	0.89

Table 2.3-6 Torsional Irregularity Check Y Direction

Summary

Difficulties controlling story drift resulted in very large members. The virtual work feature embedded within ETABS indicated that the lower story columns contributed the most and therefore these were typically targeted rather than upsizing the braces adequate for strength conditions. To accommodate the new seismic forces the weight of steel in the lateral system increased by a factor of 2.2, bringing the weight of steel per square foot in the building to 11.7 lbs.

Modal Information, CA Layout 1				
Mode	Period	UX	UY	
1	1.75	0.10	68.14	Y Translational
2	1.55	34.61	0.35	Z Rotational
3	1.39	37.30	0.02	X Tranlational

Table 2.3-7 Modal Information

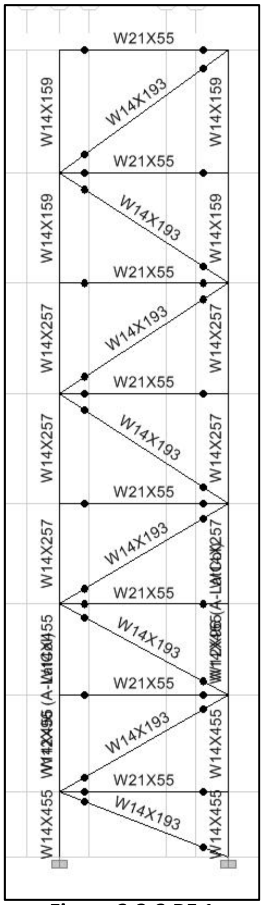

Figure 2.3-3 BF 1

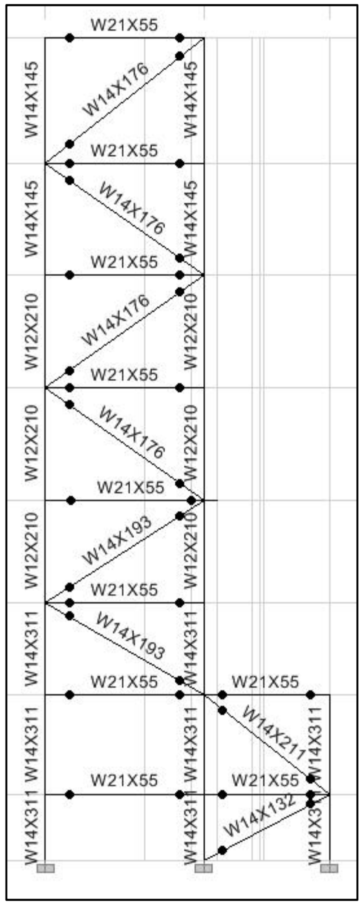

Figure 2.3-4 BF 2

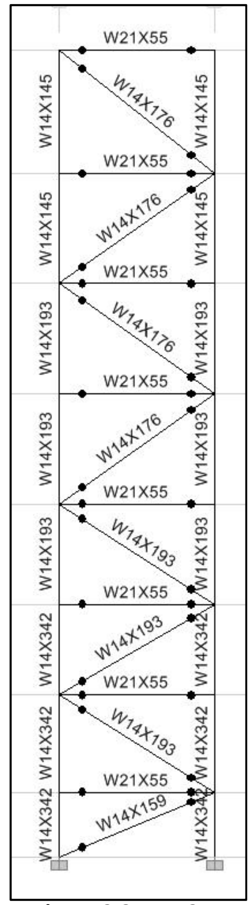

Figure 2.3-5 BF 3

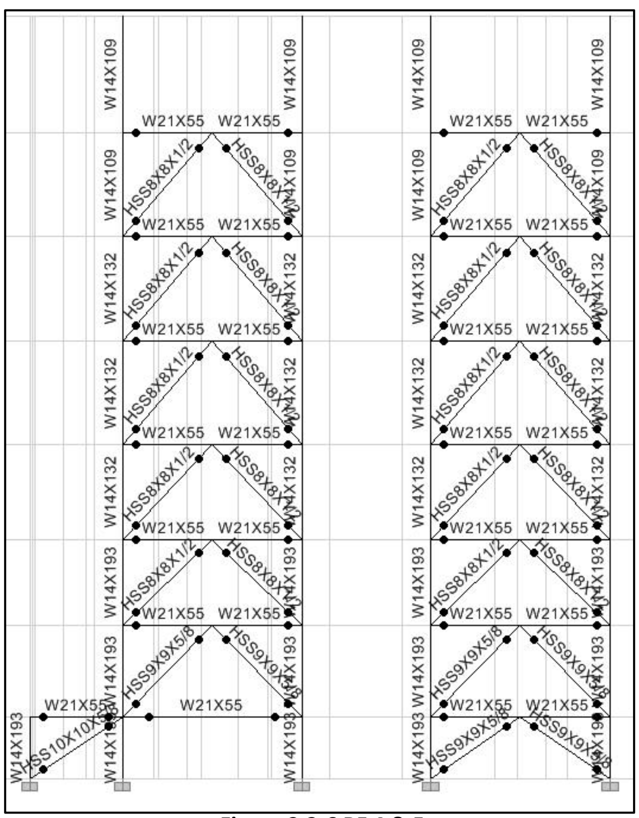

Figure 2.3-6 BF 4 & 5

2.4 – CA Lateral Design, Layout 2

The first lateral design seemed rather inefficient due to the low number of braces. It was hypothesized that a more efficient design could be achieved (less steel tonnage) by adding frames, which would also allow a $\rho = 1$, creating less strength demand on the structure. Three areas were highlighted for addition, two in the EW direction and one in the NS direction. The addition would remove 4 parking spaces of the available 60 which was deemed an acceptable impact. An additional column was added as well, to avoid a beam cantilevering from the weak axis of a column involved in Brace Frame 8.

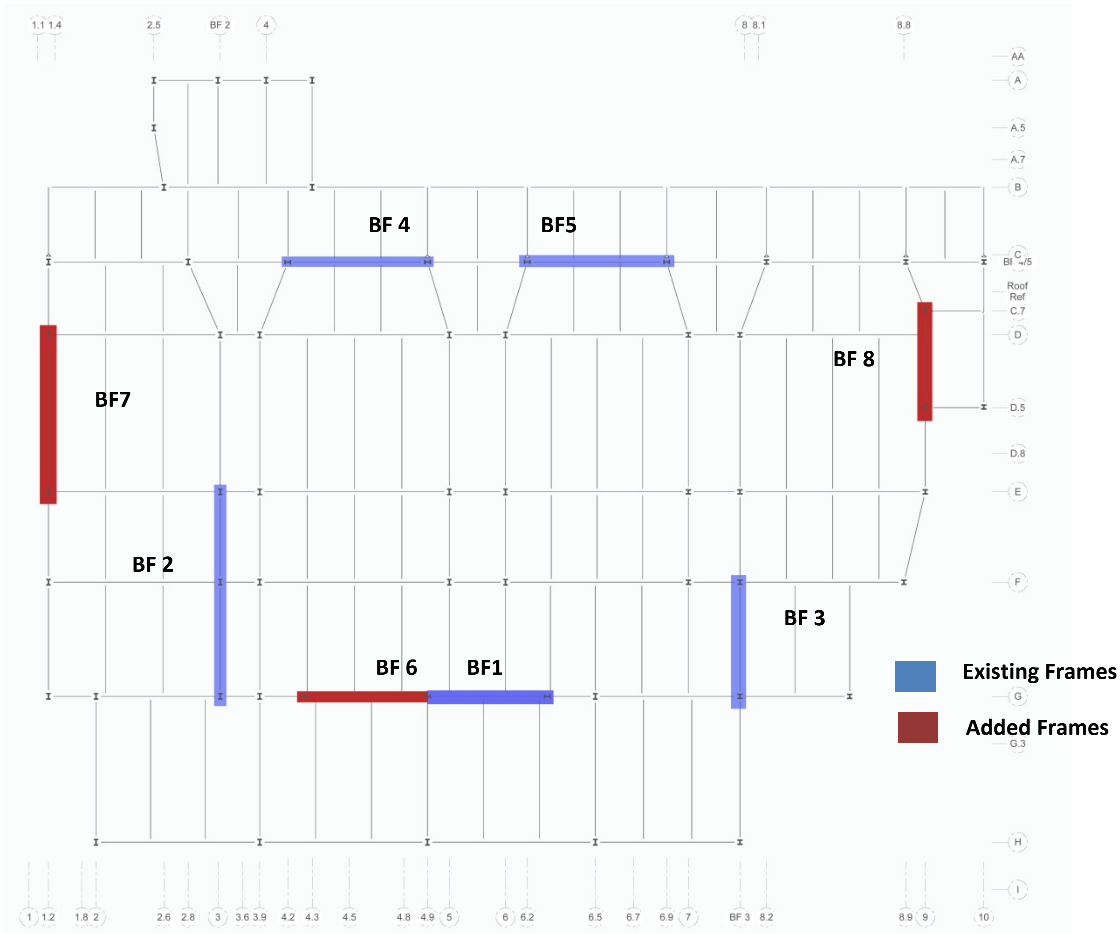

Figure 2.4-1 BF Layout

In order to use $\rho = 1$ two requirements needed to be met. The first was the confirmation that losing a brace would not cause the story to lose more than 33% of its strength. Table 2.4-1 shows the brace strengths and percentages per floor for the two orthogonal directions. Additionally it needed proved that extreme torsional irregularity was not encountered when a brace was removed on a given level. To accomplish this each brace was deleted one by one, the model run, and the results viewed. This data is included in Appendix N. Both conditions were satisfied.

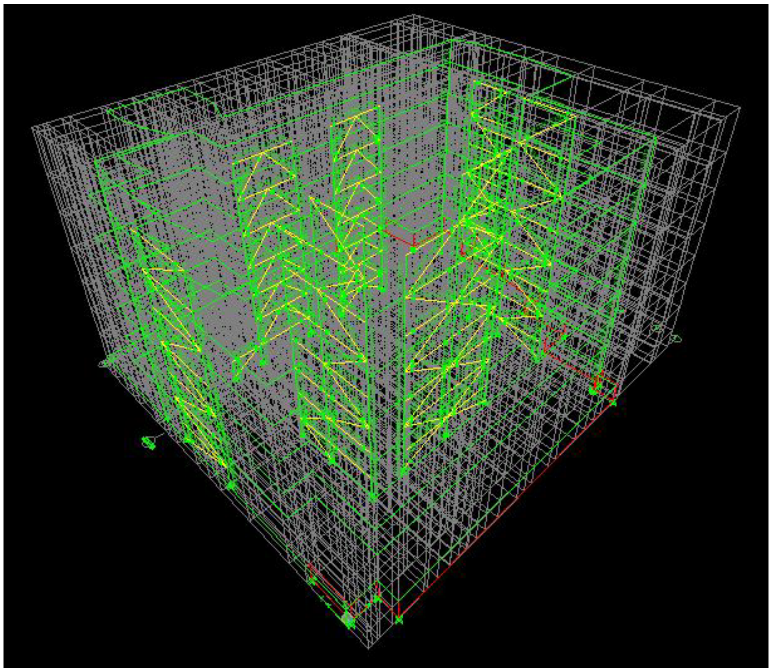

Figure 2.4-2 ETABS Model of CA Layout 2

Strength Summary				
EW Direction				
	BF 2	BF 3	BF 7	BF 8
Pent	28%	28%	22%	21%
5th	28%	28%	22%	21%
4th	28%	28%	22%	21%
3rd	28%	28%	22%	21%
2nd	28%	28%	22%	21%
1st	31%	28%	21%	21%
Terrace	30%	27%	21%	21%
NS Direction				
	BF 4	BF 5	BF 1	BF 6
Pent	21%	21%	28%	30%
5th	21%	21%	28%	30%
4th	21%	21%	28%	30%
3rd	21%	21%	28%	30%
2nd	21%	21%	28%	30%
1st	21%	21%	28%	30%
Terrace	21%	26%	26%	26%

Table 2.4-1 Brace Strength Summary

Summary

Similarly to Layout 1, drift caused members to be upsized from the preliminary sizing. Minimal steel tonnage was saved, which may be offset by the cost of the additional connections for the bracing members and shipping related to the increased number of pieces involved. Layout 2 had a steel weight of 11.6 psf, but dealt markedly better with torsional issues. Appendix P contains the braced frame elevations for this layout.

Modal Information, CA Layout 2				
Mode	Period	UX	UY	
1	1.60	0.00	70.84	Y Translational
2	1.48	73.12	0.00	X Translational
3	1.15	0.04	1.41	Z Rotational

Table 2.4-2 Modal Information CA Layout 2

2.5 CA Lateral Design, Layout 3

Earlier it was noted how chevron frames are typically avoided in seismic applications do to the large beams necessary for the system to perform properly. It was seen that of the 331 tons of steel used for Layout 2's lateral system over 25% of this could be attributed to the beams in the chevrons of frame 4 and 5. Therefore it was decided to investigate an Eccentrically Braced Frame, EBF. Eccentrically braced frames resist lateral forces through shear, flexure and axial forces in members, and are a hybrid of braced frame and moment frames, approaching the stiffness and ductility of each system respectively. In an eccentrically braced frame the brace intersects the beam/column or beam/brace centerlines on one end with the other end intersecting a distance, the eccentricity, away from the centerline. The "link" section of the beam helps the system dissipate energy through shear and is typically the focus of the design. Because EBF's offer greater ductility they have a higher R value, however the SCBF R value would still control. For this study the braced frames were chosen to have non-moment resisting (shear) connections at columns away from links.

The design provisions of particular interest from AISC 341-05 are as follows:

- 15.2a Limitations – Web of a link shall be a single thickness
    - The design of the beam will rely on balancing the shear strength vs. shear demand of the link versus the moment demand on the exterior beam. This provision states that doubler plates are not permitted to increase the shear strength of the link as this is the portion of the system intended to experience inelastic behavior.
- 15.2c Link Rotation Angle – The link rotation shall not exceed 0.08 radians for links of length $1.6M_p/V_p$ or less
    - Links less than this length are dominated by shear yielding, which is an effective means for energy dissipation. The link rotation angle is the angle between the link beam and beam outside the link at the design story drift.

The eccentric braced frame designed resulted in a W18x86 shape being used as opposed to the W33x354 that was used for the SCBF. Figure 2.5-1 shows the beam design including detailing. Connections are purely schematic, but they adhere to the provision which prevents the any part of the connection from entering the link portion of the beam.

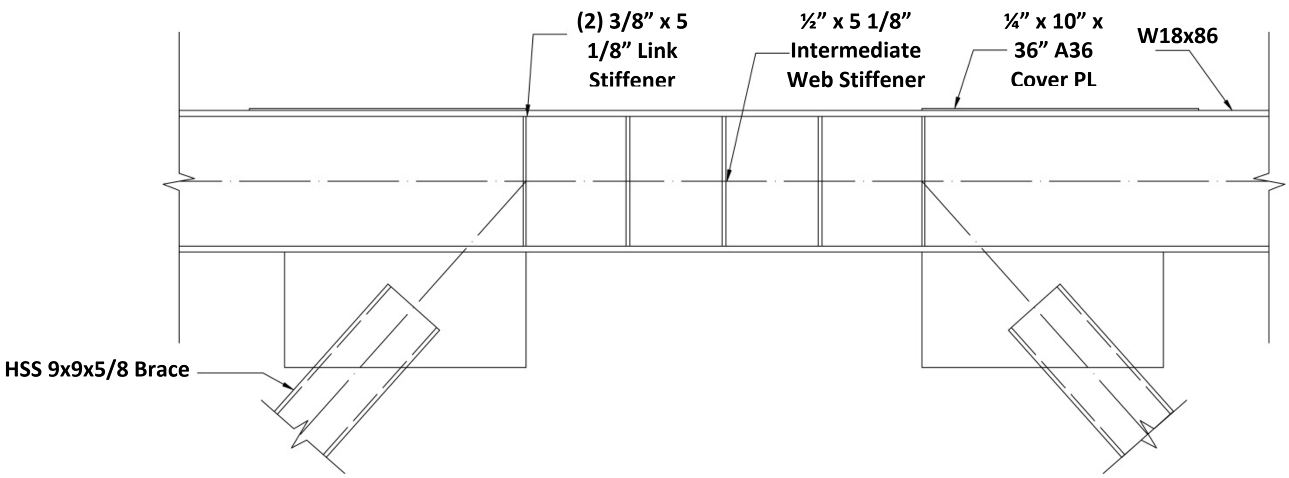

Figure 2.5-1 Eccentric Braced Frame Beam Design

Summary

The replacement of the chevron SCBF's with EBF's proved very beneficial. The frames were more flexible, but despite increasing the bracing to HSS9x9x5/8 from HSS8x8x1/2 the new layout saved approximately 70 tons of steel, resulting in a total steel weight of 10.9psf for the building.

Modal Information, CA Layout 3				
Mode	Period	UX	UY	
1	1.60	12.26	58.04	Y Translational
2	1.57	61.07	12.86	X Translational
3	1.16	1.10	1.81	Z Rotational

Table 2.5-1 Modal Information CA Layout 3

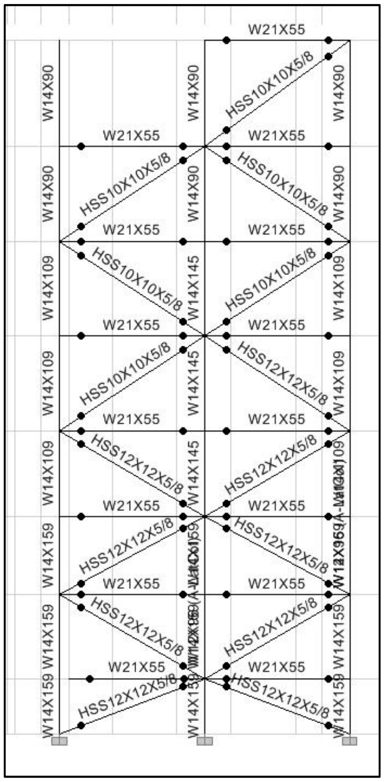

Figure 2.5-2 BF 1 & 6

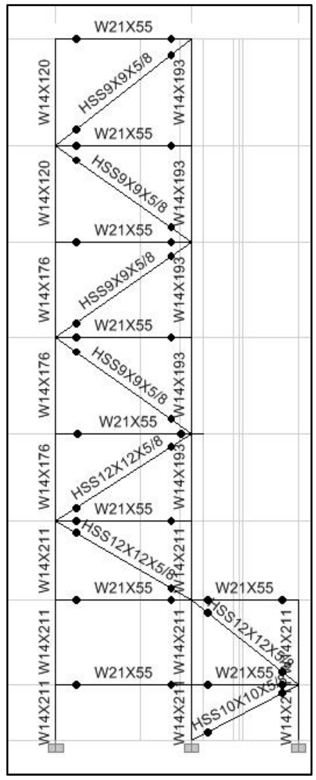

Figure 2.5-3 BF 2

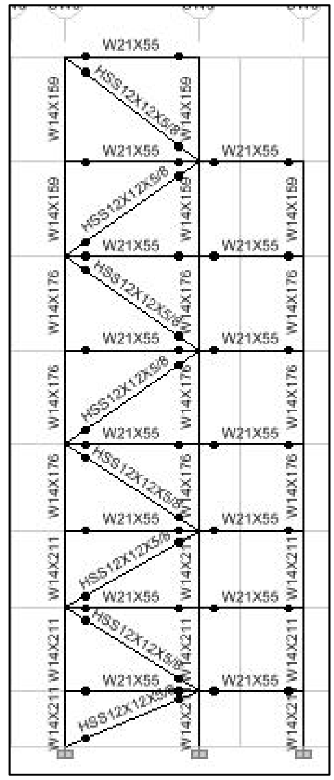

Figure 2.5-4 BF 3

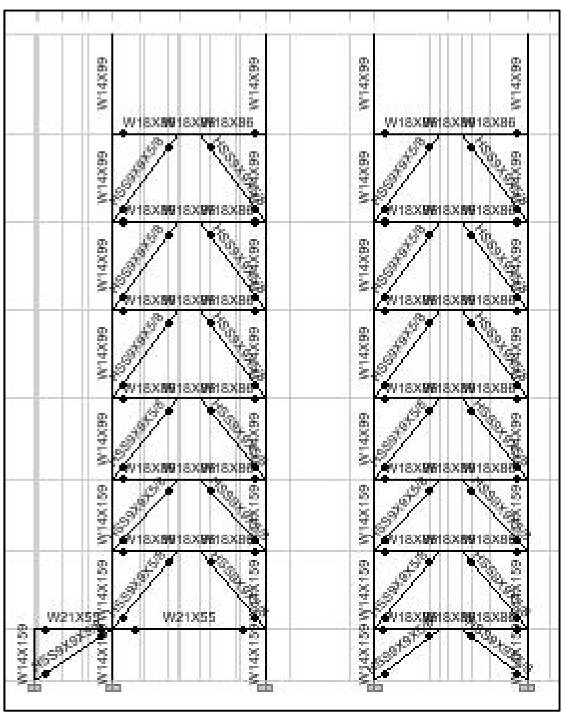

Figure 2.5-5 BF 4 & 5

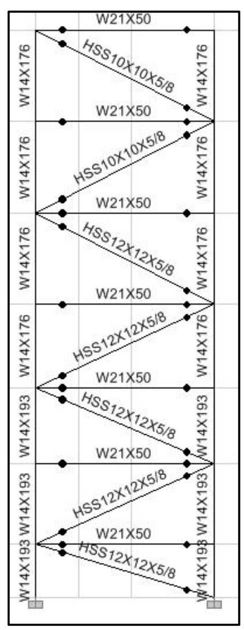

Figure 2.5-6 BF 7

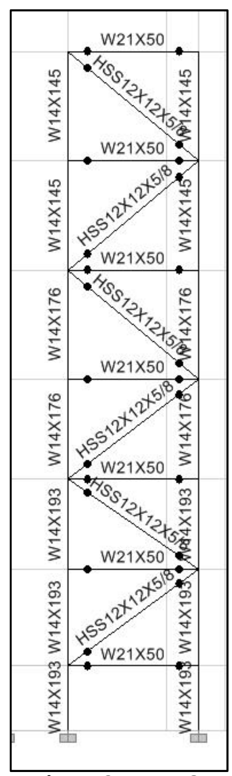

Figure 2.5-7 BF 8

2.6 - MAE Requirements

Throughout the structural redesign graduate level coursework was applied. *AE 597A – Computer Modeling* was relied upon extensively. The complexity of designing a building is drastically reduced by the ability to create finite element models of the structure of concern. This is not to say that the program does all of the work, because to get accurate results one must accurately model and understand the proper assumptions to make. This course provided background knowledge relied upon heavily to create the four different iterations, as well as the original concrete structure.

AE 538-Earthquake Engineering provided many general concepts and design tips for the structure when it was moved to the region of high seismicity. Prior knowledge of code provisions and experience with seismic design were of paramount importance.

The SBCF and EBF details were done by hand using concepts introduced in *AE 534 – Steel Connections.* The brace to beam connection in particular was of difficult geometry that needed careful thought to complete.

2.7 - Summary

In conclusion it was found that the steel redesign in Rockville could be accommodated with minimal impact to the height and layout of the building. The building also remains almost completely architecturally unaffected as well, accept for the addition of beams to the Eastern elevation which features a 14' cantilever.

The move to a higher seismic region proved to be more challenging to make the system work. The addition of braces reduced steel tonnage, though it would not necessarily have reduced the cost due to the additional shipping and connections required. A factor not directly evaluated, the foundations, may make Layout 2 more economically viable over Layout 3 as ρ would be applied to the foundation design. Layout 2 also had less torsional problems, and would be preferred due to the redundancy in the system.

What proved to be the most beneficial was the conversion of the chevron SCBF to EBF which dropped the steel tonnage by 21%. A summary of the system weights and estimated costs are shown in Table 2.6-1. Factors not considered would be the design of diaphragm collector elements to channel the load into the braced frames, which would likely increase the cost of the system as a whole and change certain gravity elements.

Steel Designs				
	MD	CA 1	CA 2	CA 3
Steel Tonnage	154	339	331	261
Est. Cost	$ 511,808.30	$ 1,085,139.94	$ 1,072,457.85	$ 851,582.56

Figure 2.7-1 Steel Design Summary

Chapter 3 – Construction Management Breadth: Cost and Schedule Comparison

Due to the changes made in the substructure and superstructure of the building it was of interest to determine impacts made to both the cost and the schedule of work. The changing height of the building was taken into account by increasing the cost of the building skin proportionally to the height adjustment. While finished floor to ceiling heights were maintained, meaning a possible zero impact to interior finishes and partitions, the CMU and gypsum assemblies were similarly adjusted.

As the original concrete structure would likely require adjustments to meet the demands of a higher seismic design category, and this redesign was not considered in the depth, it was only pertinent to compare cost and schedule of the original structure in Rockville, MD to the redesign that was performed in this location.

Original costs and schedules were provided courtesy of AECOM and Tompkins Builders, Inc.

3.1 – Cost

As the buildings substructure was changed from concrete to steel construction the new prices for the materials had to be tallied and compared. The lighter system also warranted a foundation redesign driven by the smaller column loads present upon the drilled piers. The slab on grade and basement walls was left unchanged. Table 3.1-1 below displays the original estimate as compared to the costs compiled through using RS Means 2011 data for the newly designed system. A detailed cost estimate and the original cost estimate are provided in Appendix R.

Cost Comparison			
	As Designed		**ReDesign**
Super Structure	Value		Value Adj for O&P
Cast-In-Place Concrete	$ 6,281,783.00		$ 1,839,890.40
Structural Steel	$ 1,784,892.00		$ 5,726,574.58
Substructure			
Drilled Piers	$ 953,320.00		$ 510,787.59
Exterior Enclosure			
Arch. Precast	$ 598,000.00		$ 609,960.00
Metal Wall Panels	$ 2,125,533.00		$ 2,168,043.66
Curtain Wall	$ 6,456,000.00		$ 6,585,120.00
Interior Glass (CW)	$ 683,223.00		$ 696,887.46
Louvers & Vents	$ 38,167.00		$ 38,930.34
Interior			
Masonry	$ 1,801,768.00		$ 1,837,803.36
Gypsum Board	$ 3,559,255.00		$ 3,630,440.10
Comparison	$ 24,281,941.00		$ 23,644,437.49
		Savings	$ 637,503.51

Table 3.1-1 Cost of Old System Compared to Redesign

The estimated savings on the structural system were approximately $0.64million, offset to the number shown in the table by the cost increase of the shell and certain interior elements. This number seems high considering Rockville is in an area typically dominated by concrete, but several factors need considered. The large story heights present in the building allowed steel to be implemented with little adjustment. This meant that the cost of extra building skin was not as impactful as is typical in a cost comparison of the two materials, and also meant an increase in the amount of concrete and reinforcing and associated costs due to the higher floor to floor heights. If the building height had to be increased by 10', which may have occurred were the floor heights more conventional, the increase in the building skin alone would be $1 million. A typical cost for the building superstructure is in the order of 10%. As shown in Table 3.1-2 the superstructure is slightly higher than normal, which may be a combination of the large floor heights and the large cantilevered portioned of the slab on the East Elevation.

Percentage Breakdown of Building Costs				
	Original Cost	% Total	Redesign Cost	% Total
Shell				
Super Structure	$ 8,066,675.00	12.9%	$ 7,566,464.98	12.3%
Exterior Enclosure	$ 9,900,923.00	15.9%	$ 10,098,941.46	16.4%
Roofing	$ 965,381.00	1.5%	$ 965,381.00	1.6%
	Subtotal		Subtotal	
	$ 62,332,586.00		$ 61,695,082.49	

Table 3.1-2 Cost Breakdown

3.2 - Schedule

An advantage steel construction has over concrete is typically in the duration the building structure takes to build. A schedule was compiled using RS Means 2011 for the newly designed structure and the original schedule as a reference and then compared to the original schedule. Foundations, while reduced in size, were assumed to take the same duration as the number of drilled piers was increased from 71 to 73. Exterior skin and roofing schedules were similarly assumed to remain consistent with those from the original schedule.

Task Name	Duration	Start	Finish	Predecessors
Foundations	**82 days**	**Thu 11/17/11**	**Fri 3/9/12**	
Area 1 NW	**63 days**	**Mon 11/21/1**	**Wed 2/15/12**	
Area 2 NE	**58 days**	**Wed 11/30/1**	**Fri 2/17/12**	
Area 3 SW	**70 days**	**Thu 11/17/11**	**Wed 2/22/12**	
Area 4 SE	**73 days**	**Wed 11/30/1**	**Fri 3/9/12**	
Building Structure	**99 days?**	**Tue 2/28/12**	**Fri 7/13/12**	
Column Lift One	3 days	Tue 2/28/12	Thu 3/1/12	25FF+1 day
Terrace Level	**28 days**	**Tue 2/28/12**	**Thu 4/5/12**	
Level 1	**33 days**	**Tue 3/6/12**	**Thu 4/19/12**	
Level 2	**38 days**	**Tue 3/13/12**	**Thu 5/3/12**	
Column Lift 2	4 days	Thu 4/12/12	Tue 4/17/12	48FS+2 days
Level 3	**31 days**	**Thu 4/12/12**	**Thu 5/24/12**	
Level 4	**36 days**	**Thu 4/19/12**	**Thu 6/7/12**	
Level 5	**41 days**	**Thu 4/26/12**	**Thu 6/21/12**	
Column Lift 3	3 days	Tue 5/29/12	Thu 5/31/12	70FS+2 days
Penthouse	**31 days?**	**Tue 5/29/12**	**Tue 7/10/12**	
Upper Roof	**15 days**	**Mon 6/25/12**	**Fri 7/13/12**	
Stair 1	**25 days**	**Thu 3/8/12**	**Wed 4/11/12**	
Exterior Skin	**87 days**	**Tue 6/19/12**	**Wed 10/17/1**	

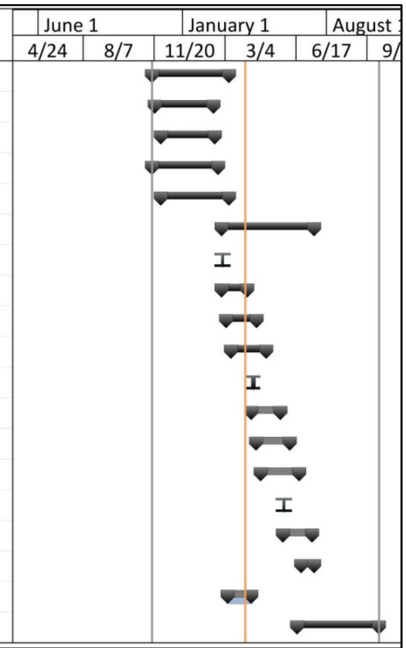

Figure 3.2-1 Redesign Schedule

Figure 3.2-1 above shows the redesign schedule from Microsoft Project. Total project duration was decreased by a month due to the material change. The end date for the exterior skin originally was dated for 11/16/12 but it is estimated that with a steel structure this can be dropped to 10/17/12. The amount of work days required to complete the building structure was reduced from 161 days original estimated to 99 days which may have the potential to cause a larger impact on areas of the schedule that were not considered in the scope of this analysis, such as work done upon the interior.

3.3 - Summary

The results of the findings in the cost and schedule analysis are summarized in Table 3.3-1. The scheduled construction time was reduced by a month, which potentially larger impacts due to the completion of the structure 62 work days ahead of the estimate for the concrete construction.

Cost/Schedule Summary		
	Original	Redesign
Schedule	9 months	8 months
Cost	$24,281,941	$23,644,437

Table 3.3-1 Cost/Schedule Summary

The building cost associated with the changes made was also found to be reduced by $1.2 million. Due to the unusually large floor to floor heights, in part present to accommodate the attachment to an existing structure, may have made steel such a competitive choice with regards to concrete in this application. While location factors were accounted for in the usage of RS Means, the fact that in this area concrete construction is typical may have led to cost increases in steel design not fully accounted for. Table 3.3-2 shows the factors looked at by the AECOM team when choosing a structural system, two of which that were highlighted being the cost and the experienced bidders. As was discussed earlier in Chapter 2 it is believed that vibration would not be as large a factor due to the irregular bay sizes and that the framing depth could be overcome with careful attention to coordination.

Structural System Comparison Chart							
	Composite Steel		PT Concrete		Concrete Skip Joist		Ranking Factor
Cost	2		3		1		
		20		30		10	10
Vibration	1		3		2		
		7		21		14	7
Ease of Future Modification	3		1		2		
		24		8		16	8
Weight of Structure	3		1		2		
(Fdtion Savings)		6		2		4	2
Same Subcontractor for	1		2		3		
Whole Structure		2		4		6	2
Smallest Column Size	3		1		2		
		6		2		4	2
Commonly Constructed	3		2		1		
System with Many Experienced Bidders		18		12		6	6
Structural Framing Depth	1		3		2		
		5		15		10	5
Fire Protection	1		3		2		
		5		15		10	5
Total	18	**93**	19	**109**	17	**80**	

Table 3.3-2 AECOM System Comparison

In summary it would appear that steel could be a very competitive alternative to concrete in this situation in terms of building cost and schedule, though limitations in cost knowledge make it unclear exactly how competitive.

Chapter 4 – Sustainability Study

The rooftop of the Judicial Center Annex has a distinct, multi-tiered shape that gives the building architectural character. It was also an area that the designers took advantage of to provide sustainable energy features that allowed the addition to gain LEED Gold accreditation. Between the penthouse and the lower roof the building features 6270 square feet of extensive green roof as well as photovoltaic panels installed on top of the lantern structures as can be seen in Figure 4-1.

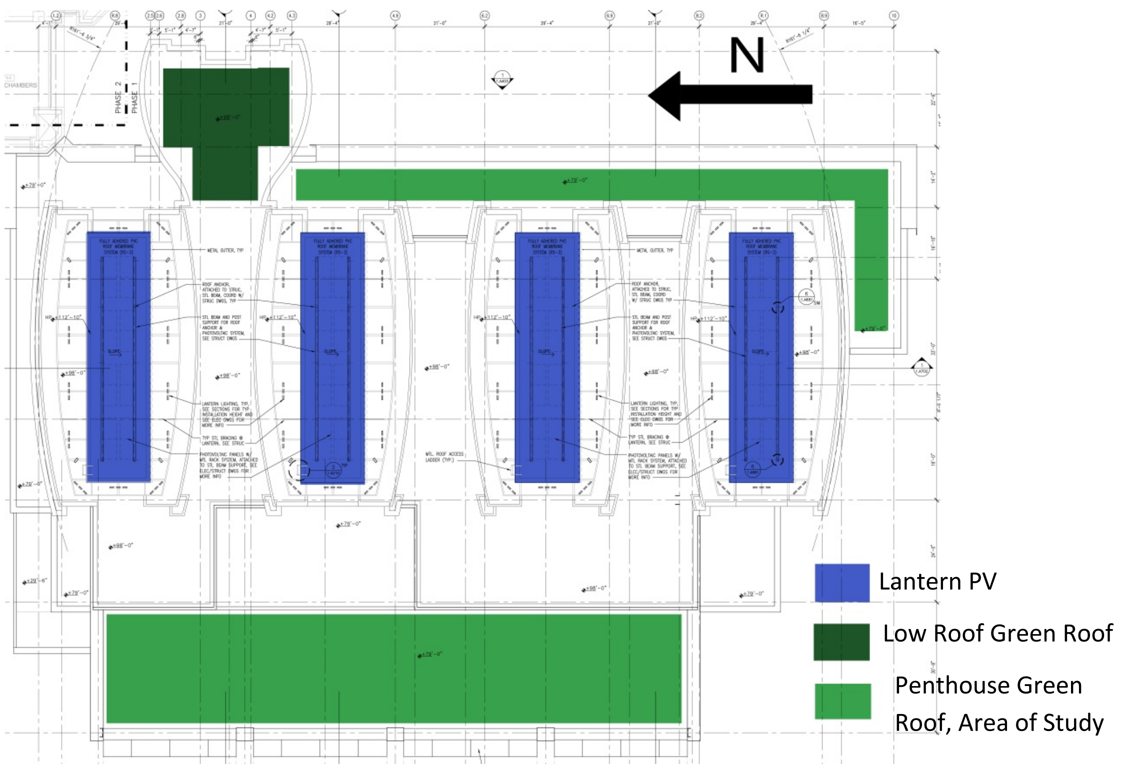

Figure 4-1: Sustainable Roof Features

This study was conducted to see if the area used as a green roof space would be better used through additional PV panels. This required both systems to be analyzed in terms of life cycle cost over the chosen 30 year span as PV Panels typically last 25-30 years, carbon output, LEED impact, and other less tangible factors. Additional areas of the LEED checklist will be explored to highlight achievable points.

4.1 – Green Roof

As stated, the JCA will feature an extensive green roof system in the green areas of Figure 4-1. An extensive system features a soil substrate of 4-6 inches of a lightweight growing medium as opposed to intensive systems which will have a heavier growing medium in depths ranging to 24”. Extensive systems can be utilized on sloped roofs, are low-maintenance and drought-tolerant due to their makeup of grasses, mosses, and flowers. Intensive roofs can feature much more diverse fauna ranging from bushes to trees and require larger degree maintenance. Figure 4.1-1 shows what an extensive green roof might look like when fully installed.

Figure 4.1-1: Extensive Green Roof, nemo.uconn.edu

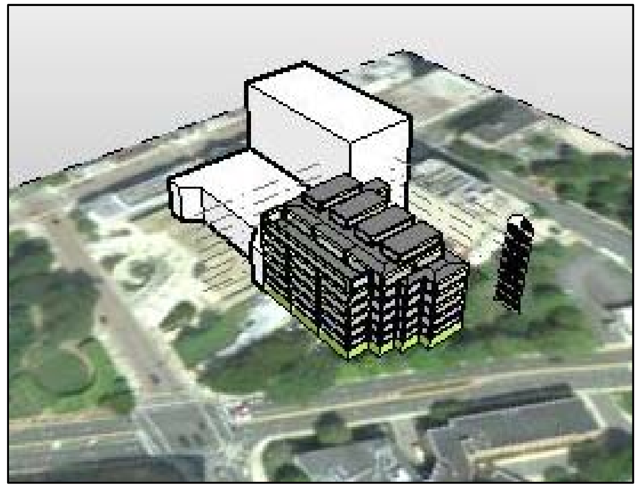

Figure 4.1-2: Autodesk Vasari Energy Model

Green roof systems typically cost from $10-$15 per square foot, twice as much as a normal roof system. However they reduce building energy costs by as much as 25% depending upon roof coverage, help curb the urban heat island effect and mitigate storm water runoff.

Life-Cycle Assessment

For the life cycle analysis the cost of the system needed to be determined over a 30 year period using net present values with an interest rate of 5% to account for future expenses or gains. The system was priced at $15 a square foot, on the high end of system costs. A maintenance rate of $0.50 per square foot for the first year was included with the assumption that it would no longer require heavy maintenance after this point. After ten years it was assumed that 10% of the system would require replacement and after 20 years an additional 20% due to damages and disrepair. However the green roof typically protects the roof membrane which has a life cycle of 15 years which would not need to be replaced, which was priced at $7.50 per square foot covered by the green roof. A summary of the system value and the prices is shown in Table 4.1-1.

Green Roof Cost				
	Cost	Yr	NPV Adj	Final Cost
Initial Cost	-94050		1	-94050.00
Maintenance	-3000	1	0.9524	-2857.20
Replacement	-9405	10	0.613	-5765.27
	-18810	20	0.377	-7091.37
Savings on Roof Repair	30,000	15	0.481	14430.00
Salvage	18810	30	0.231	4345.11
			Total Cost	$(90,988.73)

Table 4.1-1: Green Roof Life-Cycle Cost

Direct Energy

The paper, *Cost-Effectiveness of Green Roofs*, was used to as a reference point to quantify cost savings for the analysis of the system. In this paper it is noted that only the top two floors see a significant energy reduction due to a lower cooling load, so a conservative value of 1% of the total buildings energy usage was determined to be saved due to the green roof as recommended by the paper. To determine the energy savings possible it was necessary to have an estimate for the amount of energy the building used. A model was created using AutoDesk Vasari, a program used for preliminary planning that can help give better insight into a buildings energy usage and green potential. The building mass was modeled and divided into levels; the garage was excluded as minimal energy use was anticipated. The spaces were assigned an open-office occupancy and a percentage of exterior glazing was estimated based off of exterior elevations of the JCA. Based upon the energy usage the green roof saved an estimated $ 4,139 annually. The Vasari output is attached in Appendix T.

Storm Water Treatment

The reduction in storm water is another benefit of a green roof system. Extensive roofs have the capability of reducing storm water by as much as 50%. To determine the annual reduction in run off the area of coverage was multiplied by the half the annual rainfall. Rockville, MD sees 43 inches of rain per year, which results in 80.4 kgal of water reduced annually. Fisher et al.(2008) indicated a market value of $2.27 per kgal of storm water processed. This results in an annual savings of $182.50.

CO_2 Emissions

Electricity use can also be quantified in an equivalent weight in CO_2 emissions. According to Blackhurst et al.(2010) 1.5 lb. of CO_2/kWh is the electricity emissions factor. Due to the reduction in energy use the green roof results in a reduction in 47755 lbs. of CO_2 per year. The amount of CO_2 emissions released during green roof production and installation is 54.3 lbs. After seven years the system will have reduced more emissions than were involved in its creation.

Summarized in Table 4.1-2 are the relevant numbers as discussed in the three sections above. The cost benefit of a reduction in the Urban Heat Island effect is difficult to quantify as it is based upon the surrounding buildings as well and was not accounted for.

Extensive Green Roof			
Annual Energy Use Estimate	Estimated Reduction	Cost of Electricity	Annual Savings
kWh	kWh	$/kWh	
3183686	31836.86	$ 0.13	$4,138.79
Carbon Reductions	Run-off Saved (kgal)	Storm Water Cost	Annual Savings
lbs CO_2	kgal	$/kgal	
47755.29	80.4	2.27	$ 182.51
Total Annual Savings		$ 4,321.30	
Pay Back Period		21.06 Years	

Table 4.1-2: Green Roof Annual Savings and Payback Period

4.2 – PV Panels

Figure 4.2-1: Sunpower T5 Solar Roof Tiles, Source: sunpowercopr.com

PV panels are made of a crystalline silicon material, a semi-conductor that has the ability to convert sunlight into electricity. Solar energy has become increasingly popular, resulting in more efficient systems that are much more cost effective. Additionally federal and state grants for solar products and producing renewable electricity make PV panels an attractive and feasible addition to most buildings.

PV panels vary with efficiency based upon the material used as the semi-conductor as well as by geographic location, tilt, and orientation. Sunpower is one of the leading manufacturers, making extremely efficient panels. Their T5 Solar Roof tiles shown in Figure 4.2-1 were chosen, as their efficiency can offset the poor tilt angle of 5 degrees. The tilt angle is often by default chosen as the latitude of the location of interest. The Solar Roof tiles require no penetration yet are highly resistant to wind forces and lightweight due to their interlocking design meaning little to no impact on the structural system, additional system specifications are included in Appendix U. This also results in a high density of panels with the potential for greater energy gains from a smaller area. To determine the amount of energy the system could produce the number of panels the space permitted had to be determined. A typical system uses panels in increments of eight, forming a string, a schematic wiring diagram for a string and for the system shown in Figure 4.2-2. Based upon the square footage available the larger western portion of the penthouse roof as seen in Figure 4-1 could hold 136 panels and the eastern portion of the roof could hold 40 panels. At 320 watts per panel this resulted in a 56.3 kW system.

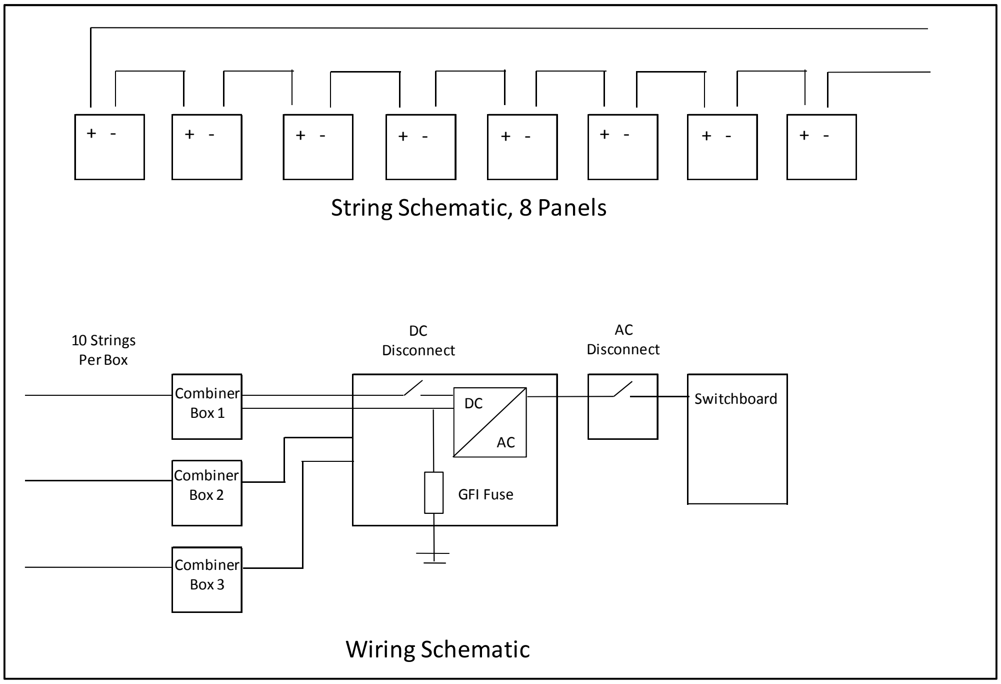

Figure 4.2-2: Schematic wiring diagram for PV Panels

Life-Cycle Assessment

System Advisory Model (SAM), software available from National Renewable Energy Laboratory can provide a life cycle for a given system with a large degree of sophistication. Federal and state tax credit and production incentives, location, tilt, azimuth, and electricity rates among other factors are accounted for. The shading factor was difficult to quantify, so a solar study was done using Vasari and resulted in an estimated value of 0.8 (1 = No shade, 0 = fully shaded). An example screen shot of the solar study is visible in Figure 4.2-3. The SAM life cycle devised a payback period of 27 years. For the full cash flow output from SAM see Appendix V.

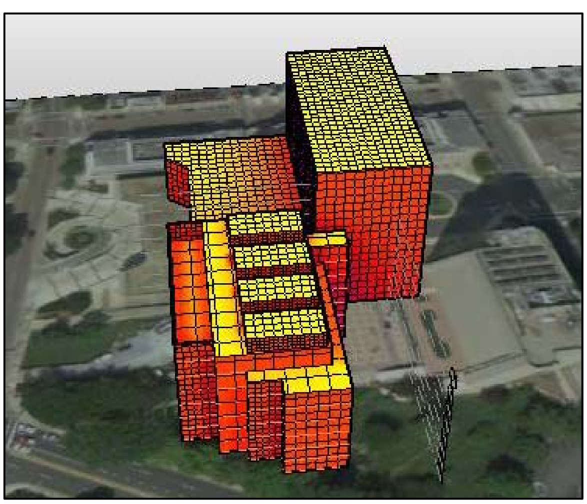

Figure 4.2-3: Solar Study

SAM Study	
Metric	Base
Net Annual Energy	55,522 kWh
LCOE Nominal	16.66 ¢/kWh
LCOE Real	12.78 ¢/kWh
First Year Revenue without System	($250,655.16)
First Year Revenue with System	($240,903.48)
First Year Net Revenue	$9,751.68
After-tax NPV	($10,323.20)
Payback Period (Yrs)	27.0
DC-to-AC Capacity Factor	12.00%
First year kWhac/kWdc	1,048
System Performance Factor	0.81
Total Land Area	0.19 acres

Table 4.2-1: Systems Advisory Model Figures

CO_2 Emissions

PV panels are a source of renewable energy and thus are often considered ‘carbon neutral’. This is a misnomer however, as while they may not produce carbon while in use their manufacturing, deliver, and installation result in carbon emissions. *Carbon Footprint of Electricity Generation* claims the lifecycle carbon production of PV panels results in an equivalent CO_2 emission of 58 g/kWh. This results in the production of over 7000 lbs. of CO_2 annually.

4.3 LEED Investigation

The original LEED checklist was obtained courtesy of AECOM which was completed in 9/9/2008. The building has since received a Gold rating while it was originally striving for Silver during this planning period which is based upon LEED-NC Version 2.2.

Going through the LEED checklist it was determined that the green roof system could impact the earning of 7 credits, of which 5 were confirmed as very likely to be achieved by AECOM’s design team. In the Sustainable Site section, credits 6.1 and 6.2 for Storm water Design were both earned which would be highly impacted by the green roof assembly. Credits 7.1 and 7.2, Heat Island Effect Roof, were also in large part earned due to the usage of a vegetated assembly, though a high albedo roof such as a thermoplastic polyolefin (TPO) membrane could be used in conjunction with the PV panels to possibly earn this credit. Credit 5.2, Site Development, requires that the vegetated open space in the project exceed the local zonings requirement by 25% and vegetated roofs count towards this. Additionally Water Efficiency credits 3.1 and 3.2 could be influenced if a grey water system were incorporated with the green roof.

The PV Panels only influenced one item on the LEED checklist, Energy and Atmosphere credit 2. This can award up to 3 LEED points based upon the percentage of renewable energy generated. The designed PV system earns approximately 1.5% of the annual energy usage, and when combined with the high roof PV panels would likely surpass 2.5% which is enough to earn a point. For 2 points they would need to produce 7.5% of the buildings energy use which is less likely, and finally 12.5% for 3 points which is unfeasible based upon the current study.

There are several additional credits that could be earned with little effort. Energy and Atmosphere credits 5, measurement and verification, is easily earned and potentially at no cost if the mechanical and electrical engineers are involved early on and align their systems so the quantities can be measured in a simple fashion by such means as placing all lighting circuits on one panel so that data can be broken down in a simple manner. Credit 6, green power, could be attained by contacting the local energy provider and exploring the possibilities of certified green power, however this may come at some cost as this energy would likely be cost slightly more.

Interestingly the structure could have a large impact on LEED credits Materials and Reuse 4.2 and 5.2. Credit 4.2 for recycled materials and 5.2 for local materials could be strongly impacted by the structural system. These require 20% of the base cost to be recycled or local respectively, and most of the structure which currently accounts for 13% of the cost as seen in Table 3.1-2 would qualify as both of these.

4.4 - Summary

In summary, due to the multitude of benefits, lower start-up cost and quicker payback period it was determined that the green roof is the more viable option and should be kept as designed. The green roof cost roughly half the initial investment the PV panels did and paid it back in 7 less years. Net carbon output was in the negative and storm water mitigation was improved. Urban heat island effects were reduced. Additionally the green roof heavily impacted 4 LEED credits with the possible influence of 3 more while the PV panels were only seen to account for 2 at most, Table 4.3-1 summarized this. The roof geometry caused shading which reduced the effectiveness of the PV panels in the areas accounted for, but depending on the planned system for the high roof a LEED point might still be earned for renewable energy.

Sustainability Summary		
	Photovoltaics	Green Roof
System Cost	$ 215,769	$90,989
Carbon Footprint (tons CO_2)	106.3	-505.0
Stormwater Mitigation (kgal)	-	80.4
Payback Period (yr)	27.0	21.1
Weight (psf)	3	20
Structural Impact	NA	Moderate
LEED Credits (gained[possible])	2[3]	4[7]

Table 4.4-1: Sustainability Summary

Chapter 5 - Conclusion

This thesis has proven that for the Judicial Center Annex a steel structural system employing braced frames is a viable solution. Though cost data may not be entirely accurate, the system would have the potential to save money or compare favorably due in part to the large story heights. The increased schedule which may have further impact beyond the month that was shown to be saved would be another benefit of this system. The steel system was able to adequately maintain the architecture and floor layout with less perceived impact.

The seismic exploration was of great interest. The steel systems developed, like their predecessor in MD, were able to handle the situation, though a greater effort had to be made to deal with the much larger forces. Layout 3 was seen as the most economical, utilizing eccentric braced frames to reduce the cost of the lateral system by 21%. The impact of chevron frames in a seismic region was proven to be very large. Redundancy was also seen as beneficial; not only in terms of the ρ factor but in terms of better torsional performance which was seen to very adversely affect the building. Additionally the “cost” of detailing a system for an R value greater than 3 was seen.

The sustainability study showed that for this situation the green roofs were the appropriate choice. The tiered roof provided shade and prevented a large enough layout to produce enough electricity to quickly offset the initial cost. The low maintenance green roof was able to pay off its initial cost approximately 6 years earlier and provided other benefits in the form of storm water mitigation and a negative carbon emission. The green roof also had many more potential impacts upon the LEED accreditation process.

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