The Pennsylvania State
University

Architectural Engineering

PARK POTOMAC OFFICE BUILDING "E"

Potomac, MD

Kyle Wagner

Structural Option (IP)

Advisor: Professor Kevin Parfitt

05/07/2010

[SENIOR THESIS FINAL REPORT]

Park Potomac Office Building "E"

Potomac, MD

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BUILDING STATISTICS

Location: 12505 Park Potomac Ave.
Size: 160,000 sq. ft. Office Space
14,000 sq. ft. Retail
213,000 sq. ft. Parking
No. of Stories: 7 Above Grade
2 Levels Below Grade Parking
Delivery Method: Design-Bid-Build
Construction: Oct 2007 - June 2009

PROJECT TEAM

Development Team: Foulger-Pratt
Architect: DAVIS, CARTER, SCOTT
Civil Engineer: VIKA, Inc.
Structural: Cagley & Associates
MEP: Allen & Shariff Corporation
Landscape Architect: Studio 39

STRUCTURAL SYSTEM

- Spread footings on soil with 3000-4000psf bearing capacity
- Post-Tensioned beams span building width
- 7" Thick post-tensioned slab
- Slab cantilevered over 12' at both building ends to provide seamless glass around building corners

MECHANICAL SYSTEM

- Two Rooftop Air Handling Units supply 80,000 cubic feet of air per minute
- Rooftop cooling tower (484 GPM)
- Carbon Monoxide detectors and exhaust fans protect parking garage

LIGHTING/ELECTRICAL

- Power supplied by two utility transformers
- Two 1600A bus duct risers
- Each floor served by 250A Panelboards
- Flourescent lamps used throughout office
- Metal Halide fixtures in garage

Acknowledgements

Cagley & Associates

- Frank Malits
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Executive Summary

The purpose of this report is to examine a possible alternative structural system for Park Potomac Office Building "E." This structure is a seven story, roughly 100 feet tall office building located in Potomac, MD. The seven office levels are each roughly 25,000 square feet and sit on top of two large levels of mostly underground parking. For this report, the seismic base level was taken at the top of the parking levels and the wind load on the parking levels was considered negligible.

The original structure was all cast in place post-tensioned concrete. Concrete columns supported a thin floor and moment frames were utilized to resist the majority of the lateral forces in both directions. This system was adequate and efficient; however, the large self weight left room for improvement and cost savings through a redesign of the system.

The office levels of the project were redesigned using composite beams, lightweight concrete on metal deck, and steel supporting columns. Braced frames were used in both directions to resist the lateral forces on the structure.

The steel beams resulted in a deeper floor depth than the original design, so the overall height of the structure needed to be increased. This increase, as well as the change in seismic weight, required the need for recalculation of lateral design forces. After recalculation of the loads, it was determined that 0.9D + 1.6W was primarily the controlling load case for the structure. Additionally, overall building torsion was found to be negligible, overturning of the building was not critical (although there were several areas of uplift at the base of the office levels at the braced frames), and all drift limitations were satisfied.

After designing the new structure, the five large mat foundations used in the original design were redesigned as a series of 17' x 17' foundations. This resulted in a 79% cost reduction for foundations and schedule improvements as well.

An architectural study was completed, analyzing the location of the braced frames with the existing floor layout. Also, the design of several connections was completed.

The cost and schedule impacts were compared for the two options and it was determined that the steel structure will cost approximately $20.69 /SF versus the post-tensioned structure, which cost $27.83 /SF. This resulted in savings of approximately 25% of the total structure's cost, while the schedule showed duration reduction as well.

Introduction

Park Potomac Office Building "E" is located prominently off I-270 at Seven Locks and Montrose Roads. It is just one of several planned office buildings that are part of an "urban village" which mixes stunning town homes, Class A office space, and a wide range of amenities including dining and shopping.

Office Building "E" is a central part of the Park Potomac Master Plan. Its central location, at the end of Cadbury Avenue, makes it a focal point for this small community (Figure 1). It is located in the main courtyard that will be a retail gathering point as well.

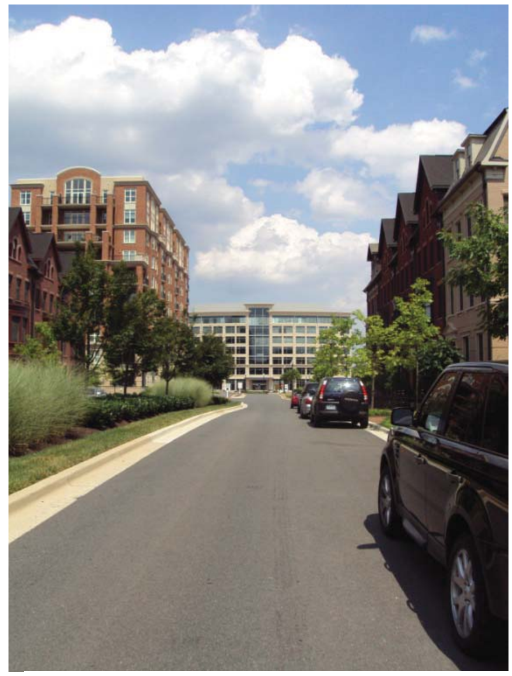

Figure 1: View from Cadbury Ave.

Material Strength Summary

Concrete:

Footings	3000 psi
Foundation Walls	4000 psi
Columns	Varies
Slab-on-Grade	3500 psi
Reinforced Slabs & Beams	5000 psi
Parking Structure	5000 psi
P.T. Concrete	5000 psi

Structural Steel:

Wide Flanges & Tees	ASTM A992, Fy = 50 ksi
Square/Rectangular Hollow Shapes	ASTM A500, Grade B, Fy = 46 ksi

Masonry:

Compressive Strength	1500 psi

Codes & Design Standards

Original Design:

a. "The International Building Code – 2003", International Code Council

b. "Minimum Design Loads for Buildings and Other Structures" (ASCE7-02), American Society of Civil Engineers

c. "Building Code Requirements for Structural Concrete, ACE 318-02", American Concrete Institute

d. " ACI Manual of Concrete Practice- Parts 1 Through 5", American Concrete Institute

e. "Manual of Standard Practice", Concrete Reinforcing Steel Institute

f. "Post Tensioning Manual", Post Tensioning Institute

g. "Manual of Steel Construction- Allowable Stress Design", Ninth Edition, 1989, American Institute of Steel Construction (Including specifications for structural steel buildings, specifications for structural joints using ASTM A325 of A490 bolts and AISC Code of Standard Practice)

Substituted for thesis analysis:

a. "The International Building Code – 2006", International Code Council

b. "Minimum Design Loads for Buildings and Other Structures" (ASCE7-05), American Society of Civil Engineers

c. "Building Code Requirements for Structural Concrete, ACI 318-08", American Concrete Institute

Existing Structural System

Foundations:

Park Potomac Office Building "E" consists of a seven story office building (Approx. 100' high) that sits above two levels of underground parking. The parking structure levels have a footprint of over 103,000 sq. ft. This is much larger than the office structure, which has a footprint of just more than 25,000 sq. ft.

This relationship also has a large impact on the design of the foundations. The net allowable bearing pressures for the site are 4000 psf for undisturbed soil and 3,000 psf for foundations placed on compacted structural fill. Over 150 spread footings are used throughout the project (Figure 2). All footings are 3000 psi concrete, and foundation walls are 4000 psi concrete. Spread footings, mostly ranging from 10' x 10' to 12' x 12', are used beneath the two levels of parking with no office building above. The majority of these footings are between 28" and 34" deep.

Larger mat footings are used in the center of the project, taking load from the two parking levels and also from the office building above. These larger foundations are up to 52' x 64' in size and can be up to 62" deep.

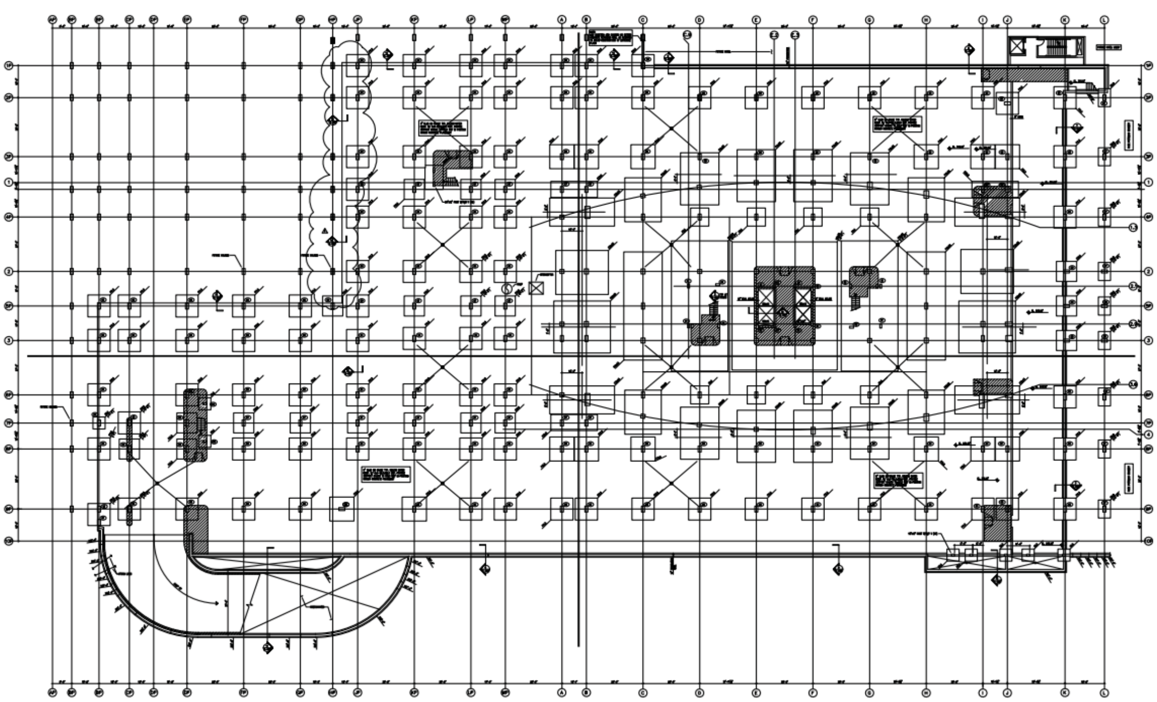

Figure 2: Foundation Plan

Floor System:

The slab on grade at the P2 Parking Level is a 5" thick, 3500 psi concrete slab. It is reinforced with 6x6 – W2.0 x W2.0 welded wire fabric. All other slabs contain 5000 psi concrete. Two-way flat slabs are used at the P1 Parking level and the Plaza/First Floor Level as well. The slab is 8" thick at the P1 Level and 12" thick at the Plaza/First Floor Level. These slabs are reinforced as needed to resist negative moments at the columns and positive moments at midspan. Post-tensioning is not used on the parking levels. Tying a post-tensioned slab into foundation walls or other fixed structure does not allow the post-tensioned slab to shrink when stressed. This would result in cracking of the slab if post-tensioning was used below grade. Using this method for the parking garage would also lead to difficulty in stressing the tendons as well. The designers of Office Building "E" used mild reinforcing below grade, and post-tensioning for the slabs above grade.

Above the Plaza Level, Office Building "E" has seven levels of office floors. These floors are 7" thick post-tensioned slabs. The post-tensioning cables induce forces in the slab ranging from 12.5 k/ft up to 35 k/ft. The post-tensioning system uses grouped tendons in the 20" beams in the E-W direction, and a one way slab with uniform tendon layout in the N-S direction. This design allows for ease of construction when laying out the tendons. The post-tensioned slab also allows for cantilevers that exist at the North and South ends of the structure. The load from a 12' cantilever on each end is taken by the uniformly spaced tendons that run through the slab.

Post-tensioning is crucial to achieving several main goals on this project. The first main goal is that it allows for large spans in the floor layout. The design of this project requires that columns be placed around the exterior walls of the building and the interior core as well. This requires the beams and slab to span long distances over the floor. Post-tensioning achieves these span requirements while maintaining a slab thickness of just seven inches. Deflection over these spans is controlled effectively, while cracking is reduced as well.

Several steel shapes are utilized on the second floor slab to frame out the canopies above the East and West building entrances. This framing consists of TS5x2 shapes that are welded to ¾" plates and hung from the bottom of the slab by L4x4 angles. Steel shapes (W8x10) are also utilized as elevator rail supports throughout all floors.

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Gravity System:

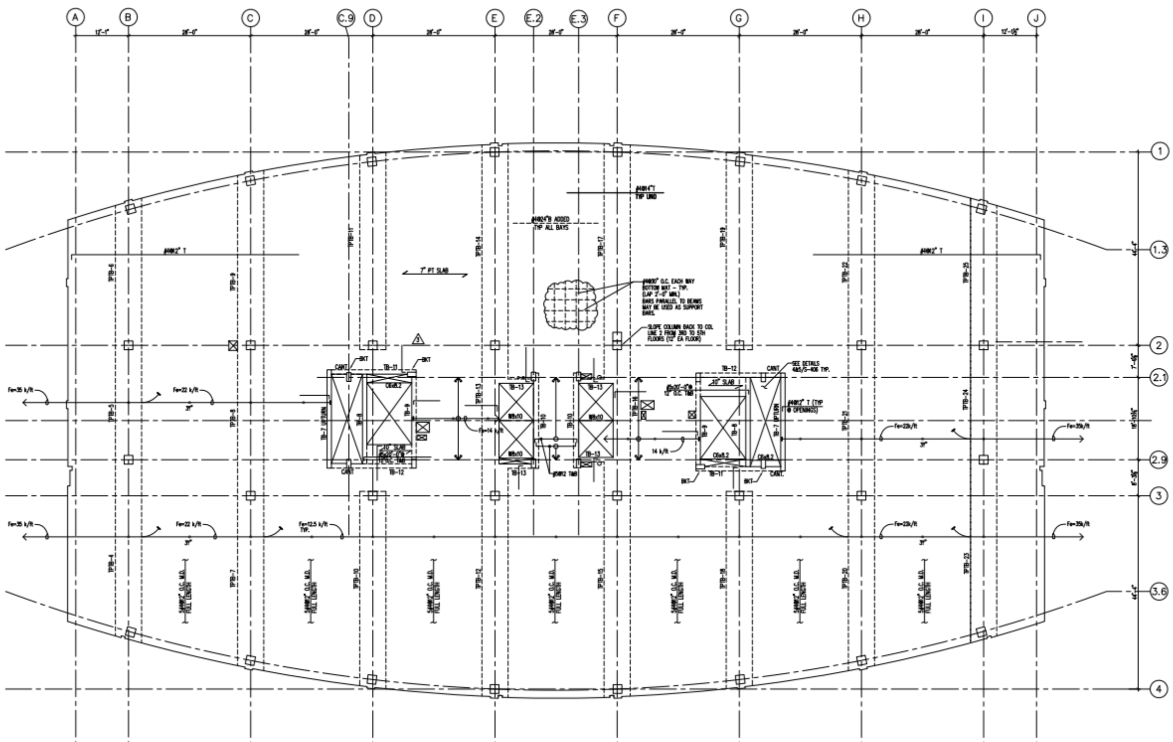

Figure 3: Typical Framing Plan

The majority of the columns in the two levels of parking are 18" x 36" columns reinforced with 10 #9 bars. These columns are typically spaced between 15' and 30' apart. Columns supporting only the two parking levels consist of 4000 psi concrete, while 6000 psi concrete is utilized where load from the office building portion above is carried. Columns in the parking levels utilize drop panels to spread the load and resist punching shear.

In the office portion of the project, a relatively repetitive column layout is achieved. Excluding the central building core, 32 columns are used to transfer the load down through all seven levels. Long span post-tensioned beams are used to transfer load from the floor to the columns. At typically 20" x 72" in size, these shallow, wide beams span in the E-W direction and continue the entire building width. In order to minimize the amount of columns in the tenant spaces and promote flexible space planning, large spans up to nearly 45' exist on each floor.

Columns on the office levels are 24” x 24” at every level, and the concrete strength is varied throughout the levels to support an increased load as required. The plaza level through the fourth floor uses 5000 psi concrete, while 4000 psi concrete is used above the fourth floor.

Lateral System:

Park Potomac Office Building “E” uses concrete moment frames, as well as shear walls to resist lateral forces. In the E-W direction, the wide post-tensioned beams on each floor create a series of parallel frames that run up through all seven floors. These frames resist any lateral forces on the building in the parallel direction.

Similarly, forces in the N-S direction are resisted by concrete moment frames as well as by four shear walls. The concrete columns and the 7” slab, which is post-tensioned in the N-S direction, combine to create a frame that resists lateral forces in this direction as well.

Roof System:

The main roof system consists of a 7” to 8” structural slab. This slab varies in order to create the required roof slopes throughout. The roof contains a Penthouse/Mechanical space, as well as an elevator machine room. The penthouse roof is an 8” two way flat plate system, while the elevator machine room utilizes a 12” thick slab.

TS8x8 posts and TS 6x6 supports are used to frame a 16’ tall screen-wall on the roof level to isolate the mechanical spaces from view.

The penthouse spaces will be largely neglected in the redesign and analysis of the structure.

Problem Statement

The post-tensioned concrete structure used for Office Building "E" has proved sufficient to resist the required lateral and gravity loads for the structure. The shallow post-tensioned slab allows for long spans and minimizes the need for columns in the rentable spaces on all floors. However, the large building self weight creates a need for large mat foundations that have a negative impact on the cost and schedule aspects of the job.

Proposed Solution

In Technical Report #2, several alternative floor systems were explored as possible options for use in the structure. The main emphasis of this study was to maintain the current column layout to maximize the unobstructed rentable area. This study showed that using a composite steel floor system could provide a viable alternative to the current system. Use of lightweight concrete was also considered as a possibility.

Using a steel structure for the office levels rather than a post-tensioned structure had several major impacts resulting from the reduced building self weight. The large mat foundations currently used beneath the office building were reduced in size, which had significant cost and schedule impacts on the project. Additionally, the building's gravity system was not required to carry as much load, which resulted in cost savings due to a reduction in member sizes. The parking levels remained the same in the structural redesign.

The redesign of the structural system also required a redesign of the existing concrete moment frame lateral resisting system. Braced frames were used to resist lateral loads for the new lateral system. Lateral forces were recalculated and reconsidered for wind and seismic forces, taking into account changes in both the height and seismic weight of the structure.

There were a few negative aspects to changing the design as well, which were explored in detail. The first of which is the increased floor depth due to the steel members. This required consideration of mechanical spaces and resulted in increasing the overall building height. Additionally, fireproofing of beams and columns will need to be completed in the new structure, resulting in some additional costs.

- A detailed study was also performed to compare the new structure with the original design. All of this will be discussed in more detail in this report.

Gravity Loads

Floor live loads were determined using ASCE 7-05. These loads were then compared to the design loads used in the original design. The design loads were largely the same as those from ASCE 7-05. A few of the loads used exceeded the required loadings from ASCE 7-05. These loads can be found below.

Table 1: Floor Live Loads

Area	Design Load (psf)	ASCE 7-05 Load (psf)
Assembly Areas	100	100
Corridors	100	100
Corridors Above First Floor	80	80
Lobbies	100	100
Marquees & Canopies	75	75
Mechanical Rooms	150	125
Offices	80 + 20 psf Partitions	50 + 20 psf Partitions
Parking Garages	50	40
Plaza, Top Floor Parking	Fire Truck Load or 250 psf	250
Retail- First Floor	100	100
Stairs and Exitways	100	100
Storage (Light)	125	125

The following superimposed dead loads were also considered in the design of the structure.

Table 2: Superimposed Dead Loads

Area	Design Load (psf)
Floors	5
Roof	10

These gravity loads used in the redesign were the same as the original loads used.

A flat roof snow load was calculated for this report as well. Beginning with a 30 psf ground snow load for Montgomery County, a flat roof snow load of 21 psf was calculated using the variables shown below from ASCE 7-05. This snow load of 21 psf was identical to the design snow load used by the structural engineer. Snow drift loads will occur on the roof level around the screen walls; however, this drift loading was not examined in this report.

Table 3: Flat Roof Snow Load			
Ground Snow Load	P_g=	30	psf
Snow Exposure Factor	C_e=	1.0	
(Terrain Category B)			
Thermal Factor	C_t=	1.0	
Importance Factor	I=	1.0	
Flat Roof Snow Load	**p_f**	**21**	**psf**

RAM Model

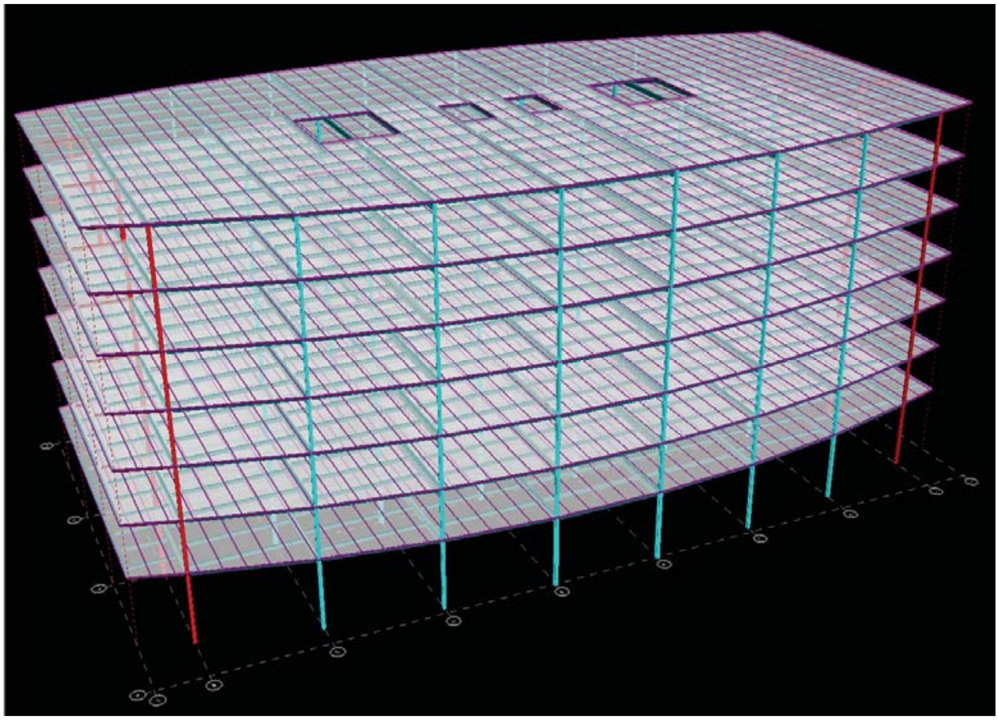

Figure 4: RAM Structural System Model

RAM Structural System was used to perform the gravity load calculations for the beams and columns of the structure. A 5-1/2" thick slab was used with lightweight (115 pcf) concrete and 2" Lok-floor decking. This provided the adequate two hour fire barrier between floors. The beam spacing was chosen to maintain a 10' maximum deck span over the floor, which was adequate for unshored criteria for 18 gage deck spans (United Steel Deck Catalog). This was done in an effort to minimize the number of beams required to carry the load. Composite beams were utilized in the design as well.

The slab cantilever condition, which can be seen in the model above, had to be considered in a unique way using RAM. "Dummy" concrete columns with approximately zero size and stiffness were added at the end of each cantilever beam. The beams at each end were moment connected to the interior column, which was created as a lateral element. The lateral elements can be seen above in red, whereas the gravity only elements are shown in blue.

The cantilevered ends will be considered in more detail later in this report.

Typical Gravity Design

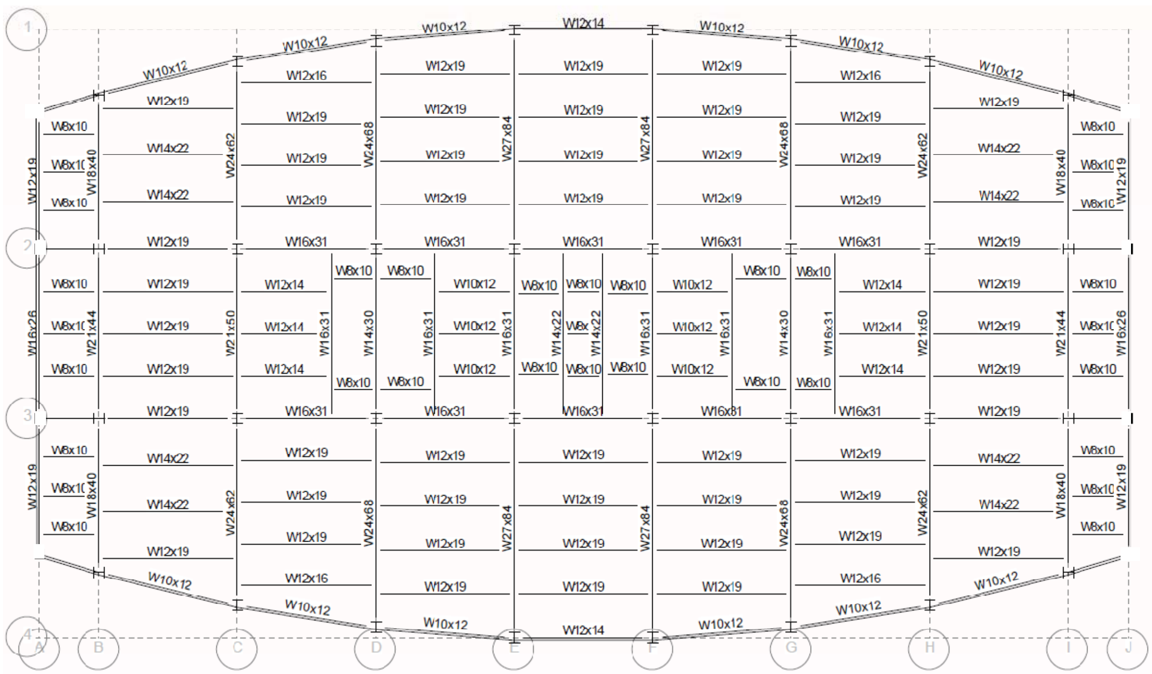

Figure 5: Typical Redesign Framing Plan

Gravity design for the structure was completed using the RAM Model described in the previous section. Beam cambers and the number of studs required were not shown for clarity.

Gravity columns were also designed using RAM. The columns that are a part of the braced frames, as well as the columns at the cantilevered ends (Grids B and I) were designed separately and are available in the later sections of the report. All column designs were determined using a splice at every other story (two story high columns), for ease of construction.

All hand calculations and spot checks for beams and columns are shown in Appendix A.

Cantilevered Ends

The original structure had one feature that was especially desired by the owner of the project. A 12' cantilever was utilized on the North and South ends of the structure. This allowed the column line along that edge to be set back in order to create the sense of unobstructed glass along the outside wall, and especially around the corners of the structure. This situation can be seen in the photo below:

Figure 6: View of South-West Corner

In the original design, the cantilever was achieved by the post-tensioning in the slab, as well as by running #4 reinforcing bars at 12" at the top of the slab.

This cantilever was also considered in the redesign. Using steel beams which cantilever out to transfer the load back to the columns through moment connections, the cantilever was successfully designed for the steel structure. Moment connections were utilized at both beams attaching to both sides of the columns to balance the moment. Column splices would need to be capable of carrying moment down the column line as well.

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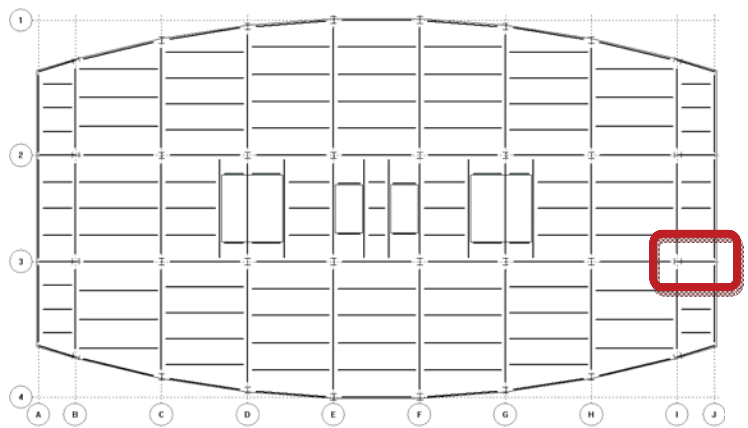
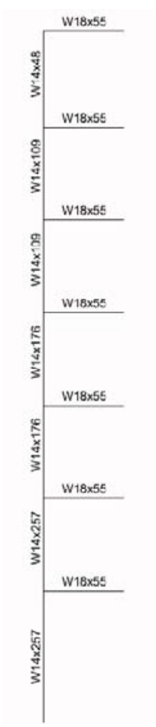
The weight of the cantilever at each end was supported by four beams; one beam along each edge of the structure, one beam on column line two, and one along column line three. After analyzing the tributary areas of each beam, it is clear that the interior beams will be critical. The beam along grid three (shown below) was the beam analyzed below. This design was applied to all of the cantilever beam situations.



After finding the loads on the beam and designing it accordingly, taking into account the composite action in the slab, it was determined that using a W18x55 would be adequate for the cantilever beam situations.



Moment connections were used on both sides of each column. The outside connection carried the load due to the cantilever slab. A moment connection was used on the inside of the columns as well, to help balance the moment at the column induced by the cantilever load. The cantilever side induced a moment of 575 ft-k, while the interior span induced only 376 ft-k. This was the maximum that was possible on the interior span due to the existing column layout.

Taking these moments into account, as well as the gravity load in the column, the columns on grids B and I were designed. The final design is shown at left.

All calculations are available in Appendix A.

Floor Depth Comparison

One main advantage of the original system over the steel redesign is the very shallow floor depth of the original post-tensioned system. The 20" total depth is shown below:

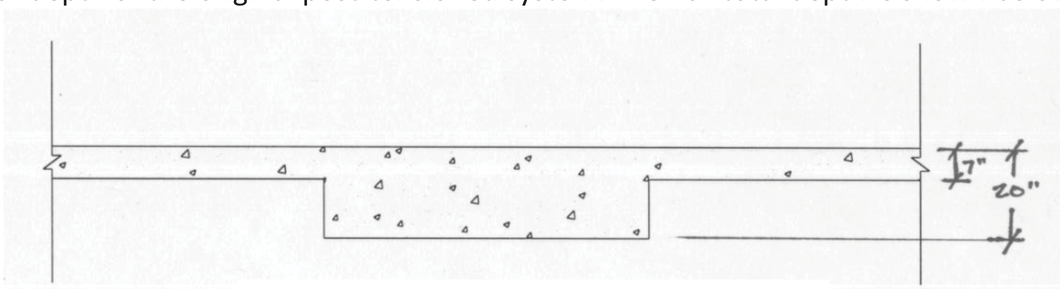


Figure 7: Original Floor Depth

After the redesign of the gravity loads on the floor system, it is evident that the deepest beam (W27x84) occurs at the 45 feet span shown below:

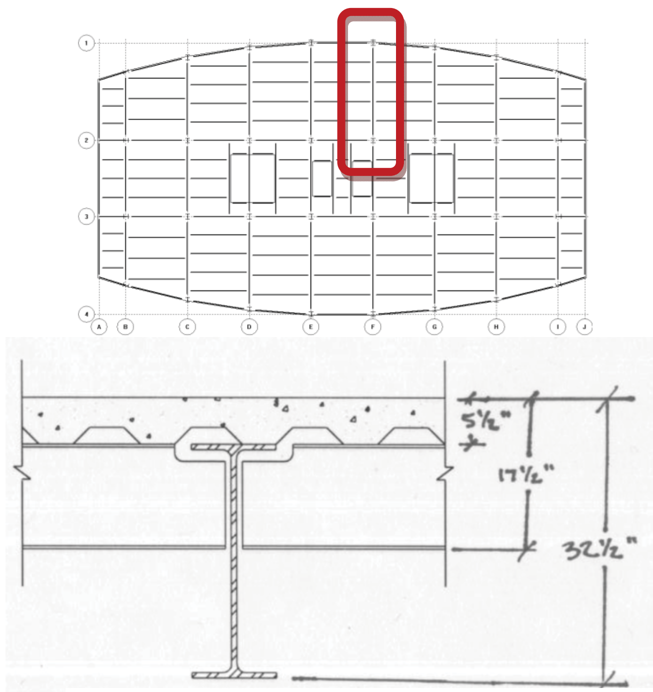


Figure 8: Redesign Floor Depth

It is clear that the new floor system increases the floor depth by approximately 12" per floor. It was important for this analysis to maintain the ceiling height in order to maintain the value of the rental spaces. It was also critical to keep the same amount of space for MEP. This left the option to increase the overall height of the structure by about one foot per floor in order to maintain these spaces. The overall height was increased by seven feet, and the new lateral forces were calculated accordingly below.

Wind Loads

Method two, detailed in Chapter 6 of ASCE 7-05, was used to determine the wind loading for the structure. Wind loadings in the N-S and the E-W directions were both analyzed. Detailed calculations can be found in Appendix B of this report. The analysis revealed the uniform pressures that occurred due to wind, which allowed the base shears and overturning moments to be determined as well.

Wind analysis for the E-W direction can be seen below. Roof uplift forces were not considered for the lateral analysis. Unfactored wind forces and loading diagrams used for the redesigned structure can be found below:

Table 4: E-W Design Pressures

Level	Height (ft above Plaza)	Design Pressure Windward (psf)	Design Pressure Leeward (psf)	Total Pressure (psf)	Force of Total Pressure (k)	Story Shear Total (k)
Plaza Level	0	6.83	-7.41	14.24	28.69	423.35
	9	6.83	-7.41	14.24		
2nd Floor	18	7.37	-7.41	14.78	50.44	394.66
	24.25	8.04	-7.41	15.45		
3rd Floor	30.5	8.59	-7.41	16.00	44.76	344.22
	36.75	9.07	-7.41	16.48		
4th Floor	43	9.49	-7.41	16.90	47.27	299.46
	49.25	9.87	-7.41	17.28		
5th Floor	55.5	10.21	-7.41	17.62	49.29	252.19
	61.75	10.53	-7.41	17.94		
6th Floor	68	10.83	-7.41	18.24	51.01	202.90
	74.25	11.10	-7.41	18.52		
7th Floor	80.5	11.36	-7.41	18.78	53.57	151.88
	87	11.62	-7.41	19.03		
Main Roof	93.5	11.86	-7.41	19.27	28.03	98.32
Penthouse	109.5	12.37	-7.26	19.63	70.28	70.28

Base Shear	423	K

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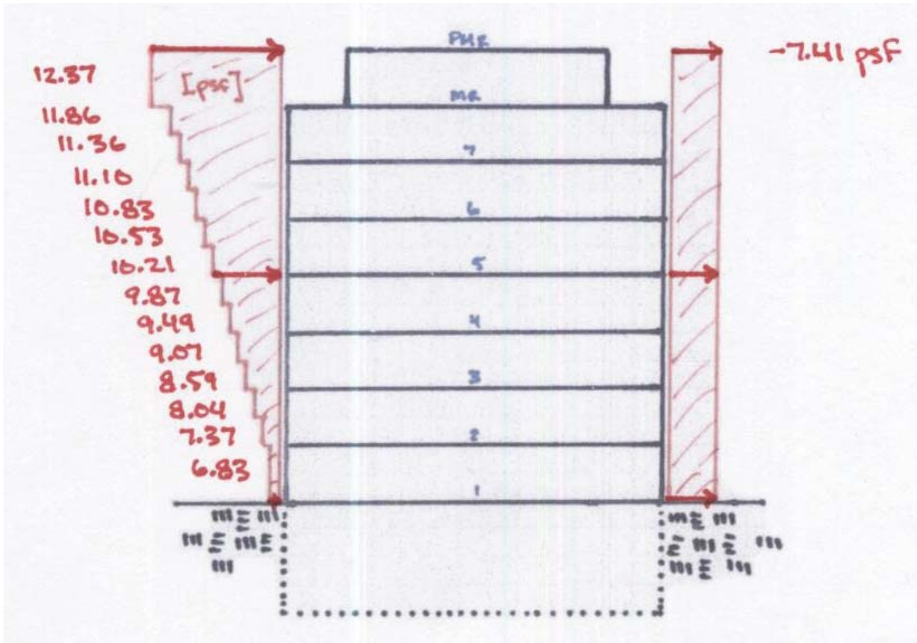

Figure 9: East – West Design Pressures

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Analysis results for the N-S wind direction can be found below. It was assumed that the minimal wind exposure on the below grade parking levels was negligible for this analysis. Unfactored results and loading diagrams can be found below for the N-S wind direction:

Table 5: N-S Design Pressures

Level	Height (ft above Plaza)	Design Pressure Windward (psf)	Design Pressure Leeward (psf)	Total Pressure (psf)	Force of Total Pressure (k)	Story Shear Total (k)
Plaza Level	0	6.83	-5.19	12.02	13.79	210.28
	9	6.83	-5.19	12.02		
2nd Floor	18	7.37	-5.19	12.56	24.42	196.49
	24.25	8.04	-5.19	13.23		
3rd Floor	30.5	8.59	-5.19	13.78	21.96	172.07
	36.75	9.07	-5.19	14.26		
4th Floor	43	9.49	-5.19	14.68	23.39	150.11
	49.25	9.87	-5.19	15.06		
5th Floor	55.5	10.21	-5.19	15.40	24.54	126.72
	61.75	10.53	-5.19	15.72		
6th Floor	68	10.83	-5.19	16.02	25.52	102.17
	74.25	11.10	-5.19	16.29		
7th Floor	80.5	11.36	-5.19	16.55	26.91	76.65
	87	11.62	-5.19	16.81		
Main Roof	93.5	11.86	-5.19	17.05	14.13	49.74
Penthouse	109.5	12.37	-5.08	17.46	35.61	35.61

Base Shear	**210**	**K**

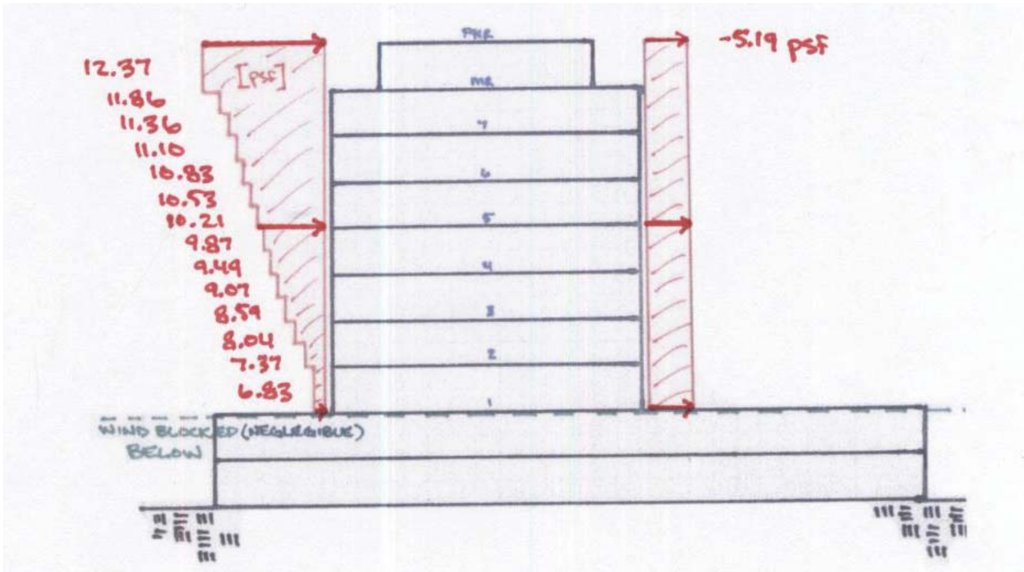

Figure 10: North – South Wind Pressures

The additional height increase for the structure has slightly increased the wind loading of the structure. This was anticipated and is reflected in the analysis. The original structure's base shear due to wind was 207K in the N-S direction and 416K in the E-W direction. This is slightly less than the redesign forces of 210K in the N-S direction and 423K in the E-W direction.

It is also logical that the base shear in the N-S direction would be approximately two times the base shear in the E-W direction due to the fact that the surface area is approximately twice as large.

Seismic Loads

The layout of the parking levels and the surrounding ground created unique seismic considerations for Office Building "E." The two levels of underground parking were mostly below grade, except on the North side of the structure. This scenario can be seen below.

Figure 11: View from North

Although it is evident that the parking levels are partially exposed on the North side, it was assumed for this analysis that the seismic base level will be at the plaza level (above the below grade parking levels) for the structure. This is due to the fact that the parking levels are largely below grade and will act as being mostly fixed. This assumption was confirmed by results obtained in Technical Report #1. For this report, only the office levels will be considered for seismic in both directions.

The seismic analysis in this report was completed using Chapters 11 and 12 from ASCE 7-05. The equivalent lateral force procedure was determined to be valid for this analysis. Detailed calculations, including updated building self weights and other variables, are available in Appendix C. The main variables used in the analysis are shown below.

Table 6: Seismic Design Variables			
			ASCE Reference
Soil Classification		D	Table 20.3-1
Occupancy		II	Table 1-1
Importance Factor		1.0	Table 11.5-1
Structural System		Steel System	Table 12.2-1
Spectral Response Acceleration, Short	S_s	0.156	USGS Website
Spectral Response Acceleration, 1 s	S_1	0.051	USGS Website
Site Coefficient	F_a	1.6	Table 11.4-1
Site Coefficient	F_v	2.4	Table 11.4-2
MCE Spectral Response Acceleration, Short	S_{MS}	0.2496	Eq. 11.4-1
MCE Spectral Response Acceleration, 1 s	S_{M1}	0.1224	Eq. 11.4-2
Design Spectral Acceleration, Short	S_{DS}	0.166	Eq. 11.4-3
Design Spectral Acceleration, 1 s	S_{D1}	0.081	Eq. 11.4-4
Seismic Design Category	S_{DC}	B	Table 11.6-2
Response Modification Coefficient	R	3	Table 12.2-1
Approximate Period Parameter	C_t	0.02	Table 12.8-2
Building Height (E-W)	h_n	100.5'	
Structure Period Exponent	k	1.58	
Approximate Period Parameter	x	0.75	Table 12.8-2
Fundamental Period (E-W)	T	1.6055 s	Eq. 12.8-7
Fundamental Period (N-S)	T	1.6672 s	Eq. 12.8-7
Long Period Transition Period	T_L	8.0 s	Fig. 22-15
Seismic Response Coefficient	C_s	0.025	Eq. 12.8-2

After calculation of the overall building self weight (See Appendix C), base shears were calculated in order to calculate the forces on the structure. The base shears are shown below in Table 7. The base shears obtained were similar in magnitude to the value of 300K calculated by the design engineer. The values calculated in this report will be used for further analysis.

Table 7: Base Shears			
	Effective Seismic Weight	Seismic Response Coefficient	Base Shear (K)
N-S	W = 8895 K	C_s = 0.0250	222
E-W	W = 8895 K	C_s = 0.0250	222

After the calculation of the base shear values for each direction, the forces can be distributed throughout the building to determine forces at each level and story shear values. The values below are all unfactored.

Table 8: Seismic Calculations					
Level	Story Weight (K)	N-S Height (ft)	Forces (K) Fx	Story Shear Vx	Moments (ft-k) Mx
Penthouse	211.8	116.5	6	0	721
Main Roof	423.6	100.5	15	6	1472
7th Floor	1270.7	86.5	66	21	5673
6th Floor	1270.7	73.0	50	86	3661
5th Floor	1270.7	59.5	36	137	2160
4th Floor	1270.7	46.0	24	173	1112
3rd Floor	1270.7	32.5	14	197	454
2nd Floor	1906.1	19.0	11	211	216
Plaza/First Floor	0.0	0.0	0	222	0
Total:	**8895**		**222**		**15469**

$\Sigma\ w_i h_i^k$	312756036

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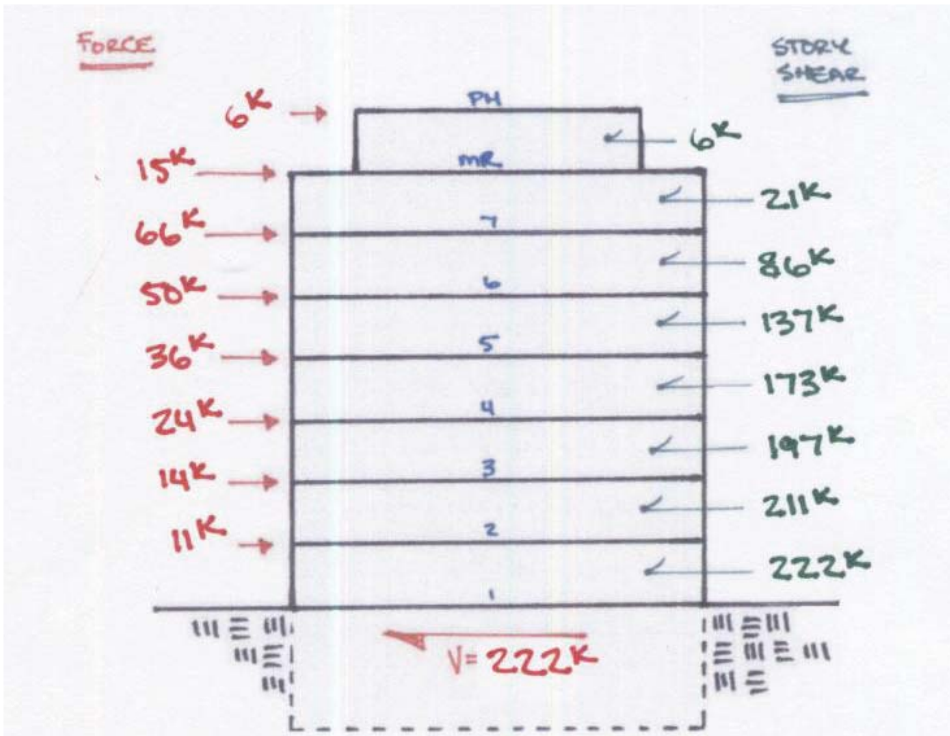


Figure 12: East- West Seismic Forces

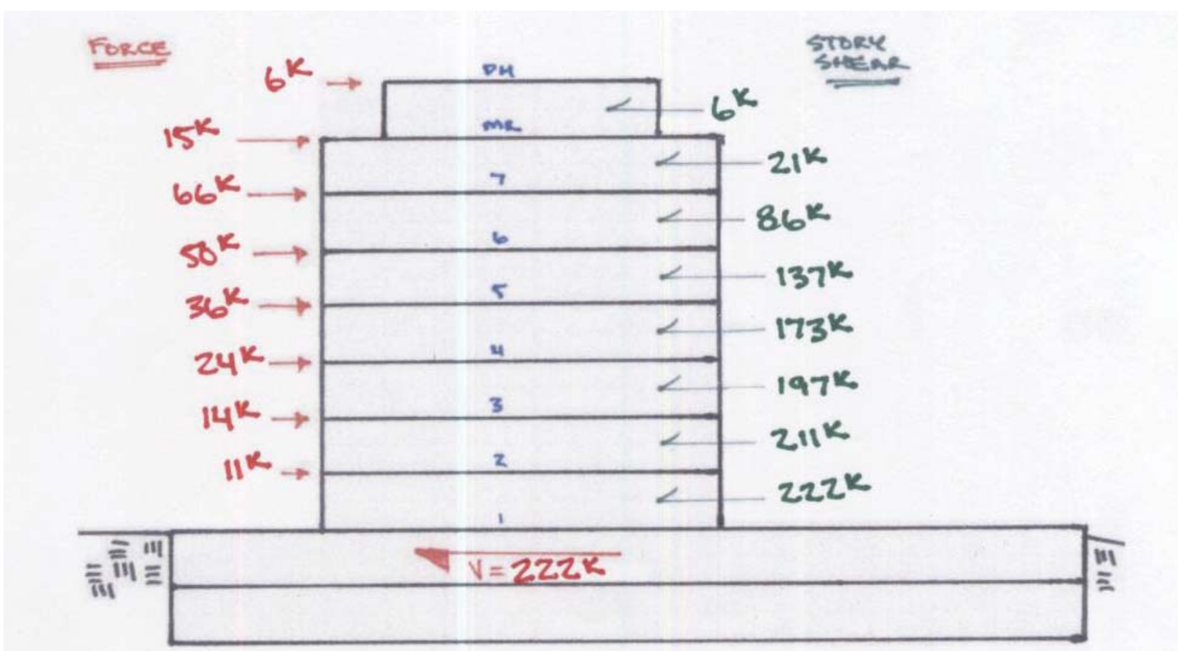


Figure 13: North- South Seismic Forces

Load Path

In the original post-tensioned design, concrete moment frames were used to resist lateral forces in both directions. Essentially, the entire building took part in resisting lateral loads. In the redesign of the structure, braced frames were used to resist lateral forces. In the N-S direction, four braces were used, while only two braced frames were used to resist even larger loads in the E-W direction. This had to be taken into account when designing the braces, which will be detailed later in this report.

In both directions, the floor diaphragm transfers lateral forces to the braced frames at each level. The braced frame columns transfer these loads down the building through shear and axial column forces. This process continues throughout the building and down to the foundations, where the forces are transferred to the soil.

A basic plan of the redesigned lateral system is shown below in the figure. The braced frames for both directions are shown in red and numbered accordingly.

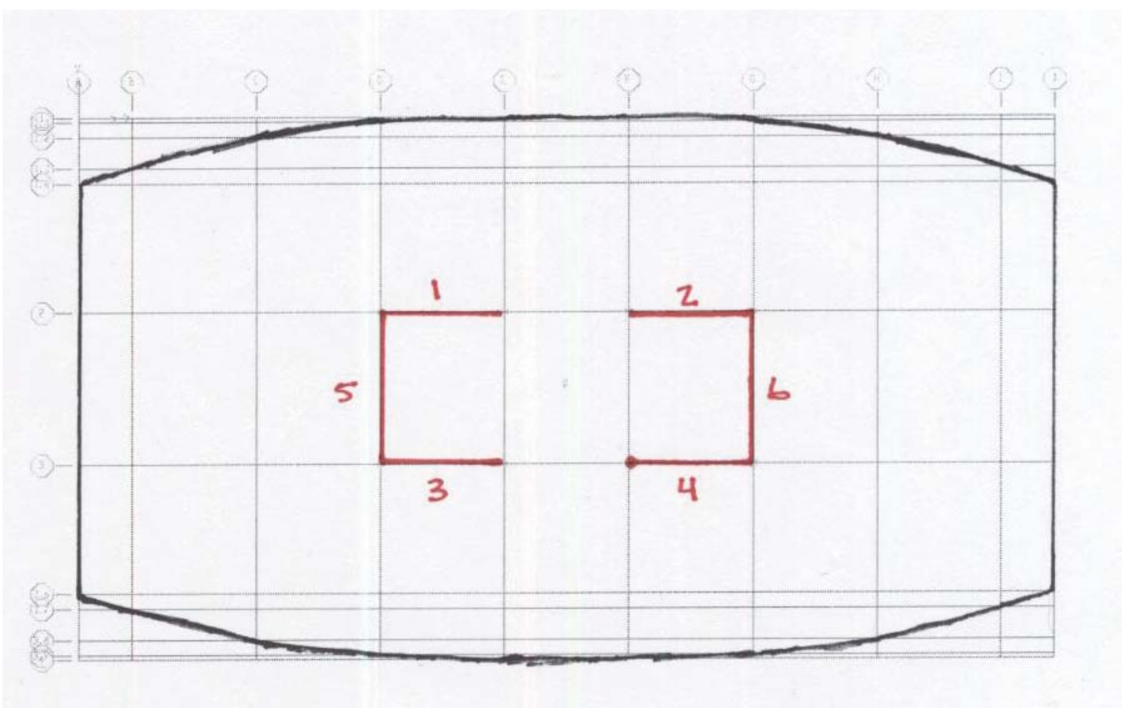


Figure 14: Lateral System Components

Load Combinations

Per ASCE 7-05 Section 2.3.2, seven load combinations must be considered when dealing with strength design. They are outlined below:

1. 1.4(D + F)
2. 1.2(D + F + T) + 1.6(L + H) + 0.5(Lr or S or R)
3. 1.2D + 1.6(Lr or S or R) + (L or 0.8W)
4. 1.2D + 1.6W + L + 0.5(Lr or S or R)
5. 1.2D + 1.0E + L + 0.2S
6. 0.9D + 1.6W + 1.6H
7. 0.9D + 1.0E + 1.6H

The following four wind cases were also considered from ASCE7-05 Figure 6-9 shown below. Case 1 proved to be the most critical case after analyzing all combinations.

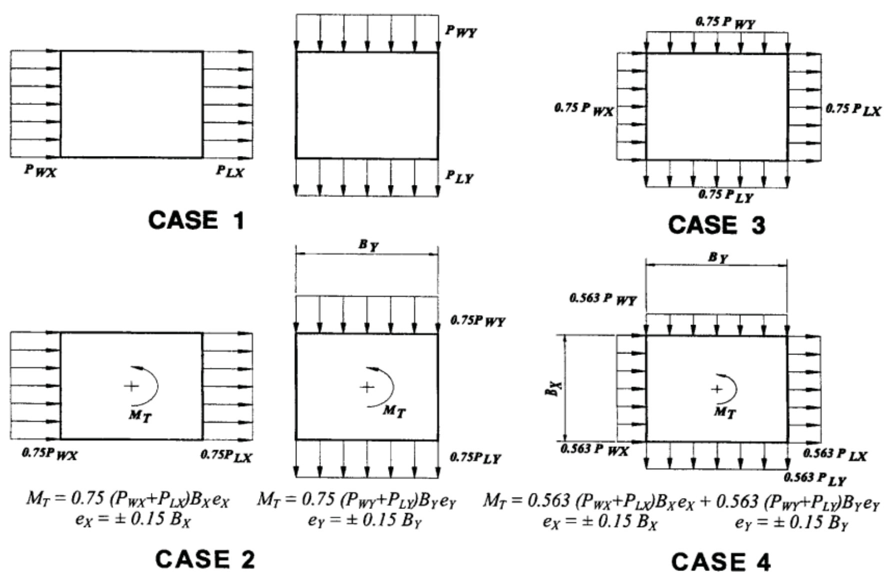
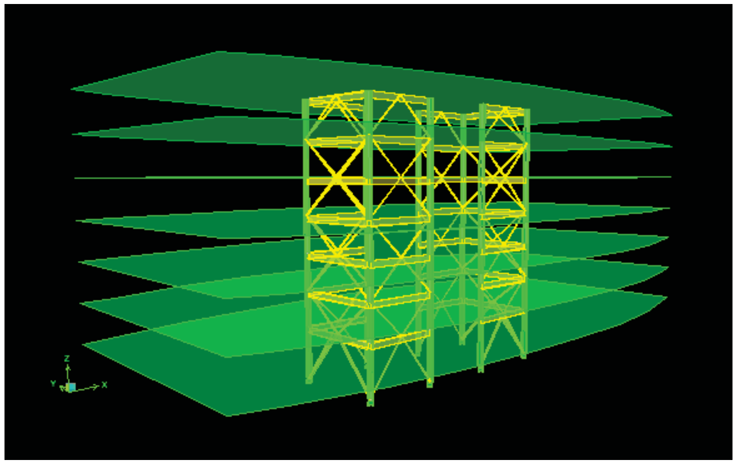


Figure 15: ASCE 7-05 Wind Cases

After analyzing the required load combinations using ETABS and checking the forces and deflections in the different load combinations, it is apparent that for both the N-S and the E-W directions, 0.9D + 1.6 W predominantly controls. This is expected due to the relatively low seismic location. It is also expected that this combination would control over load combination four, due to the fact that a smaller building weight would have less resistance to wind forces, making it more critical.

ETABS Model



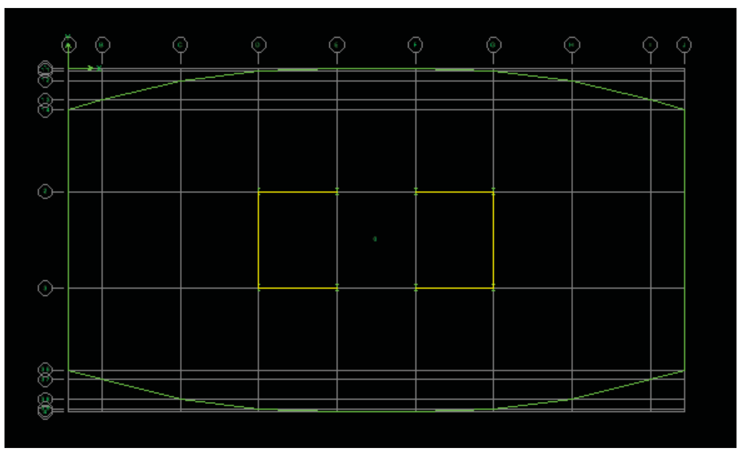


Figure 16: ETABS Lateral Model

A computer model of the structure was used to analyze the lateral system and the forces acting on the structure. ETABS, a computer modeling program from Computers & Structures, Inc. was used for the analysis. In this, only the lateral resisting elements needed to be modeled to gain an accurate representation of a building's performance under lateral loading.

All six braced frames were modeled, along with rigid floor diaphragms. The building's self weight was calculated by hand and applied to the diaphragm as an additional area mass. All load cases and combinations considered were manually added to the model. This model provided useful information with regard to force distributions and building drift that are used in the following section.

Distribution of Lateral Forces

The lateral system design, as well as the overall building shape and floor plans are fairly basic for this structure. The building is symmetrical in shape about its x and y axes. This results in a center of mass located directly in the center of the structure. Similarly, the lateral system is symmetrical as well, both in location and in stiffness of the frames. This creates a center of rigidity located at the building's center, at the same point as the center of mass. These two centrally located points result in negligible eccentricities caused by seismic and concentric wind forces, which eliminates overall building torsion due to these loadings. Building torsion was considered only for the eccentrically loaded wind cases, as well as the accidental moment caused by eccentric seismic forces.

Lateral loads were assumed to be distributed throughout the floor by way of a rigid floor diaphragm, causing the deflections at each point in each level to be the same due to the support of an infinitely rigid floor. This means that determining the relative stiffness of each frame must be done using the stiffness of each frame, rather than by tributary floor widths. The stiffer frames will resist more force than less stiff frames. This basic theory was used to determine the relative stiffness of each frame in the N-S and E-W directions.

In order to determine the relative stiffness of each frame, a 1000K load was applied to the top building level in each direction. Section cuts were used in ETABS to determine the shear forces in the columns at each frame. It was confirmed that the sum of all shears at every level was equal to the story shear, or 1000K. This confirmed that all resistive forces were accounted for on all levels. From these forces, the relative stiffnesses were determined for each frame by examining the percentage of the total 1000K that the frame resisted. This basic method was completed in both directions. The results can be found in the following tables:

Table 9: Resisting Forces (X/ N-S)

Level	Frame 1	Frame 2	Frame 3	Frame 4	Total Force (K)
1	250.0	250.0	250.0	250.0	-1000
2	250.0	250.0	250.0	250.0	-1000
3	250.0	250.0	250.0	250.0	-1000
4	250.0	250.0	250.0	250.0	-1000
5	250.0	250.0	250.0	250.0	-1000
6	250.0	250.0	250.0	250.0	-1000
7	250.0	250.0	250.0	250.0	-1000

Table 10: Relative Stiffness (X/ N-S)					
Level	Frame 1	Frame 2	Frame 3	Frame 4	Total Percent
1	25.0	25.0	25.0	25.0	100
2	25.0	25.0	25.0	25.0	100
3	25.0	25.0	25.0	25.0	100
4	25.0	25.0	25.0	25.0	100
5	25.0	25.0	25.0	25.0	100
6	25.0	25.0	25.0	25.0	100
7	25.0	25.0	25.0	25.0	100

Table 11: Resisting Forces (Y/ E-W)				
Level	Frame 5	Frame 6	Misc Columns	Total Force (K)
1	484.50	484.50	31.04	-1000
2	493.37	493.37	13.28	-1000
3	511.76	511.76	-23.48	-1000
4	485.25	485.25	29.50	-1000
5	513.46	513.46	-26.88	-1000
6	492.11	492.11	15.78	-1000
7	496.08	496.08	7.88	-1000

Table 12: Relative Stiffness (Y/ E-W)				
Level	Frame 5	Frame 6	Misc Columns	Total Percent
1	48.5	48.5	3.1	100
2	49.3	49.3	1.3	100
3	51.2	51.2	-2.3	100
4	48.5	48.5	3.0	100
5	51.3	51.3	-2.7	100
6	49.2	49.2	1.6	100
7	49.6	49.6	0.8	100

It is clear that for the N-S direction, each of the braced frames takes 25% of the total load. In the E-W direction the two frames make up approximately 50% of the total load at each floor. These results were quite predictable. These load distributions were then used to determine the sizes of the braced frame braces in the next section.

Braced Frame Design

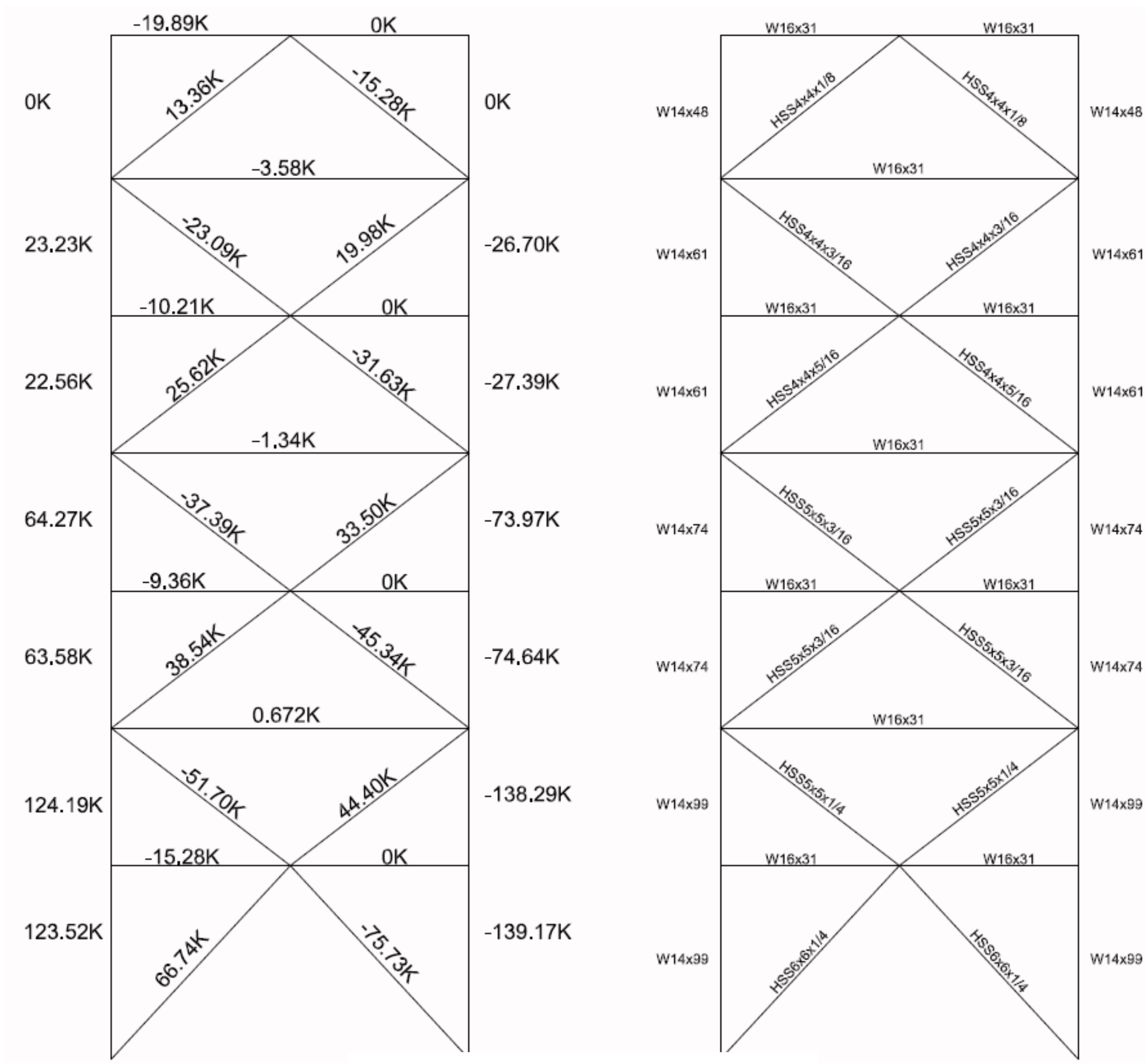


Figure 17: Typical N-S Braced Frame

After finding the relative stiffnesses of the frames, the critical load combination of 1.6W was applied directly to the frames to find the critical axial forces in the braces. SAP was used to perform this analysis. The axial forces due to this load are shown above, at left. After the braces were sized, the axial forces in the columns were considered along with the dead and live loads, to determine column sizes at the braces. These calculations are shown in more detail in the appendix. The final design of the braces in the N-S direction can be seen above, at right.

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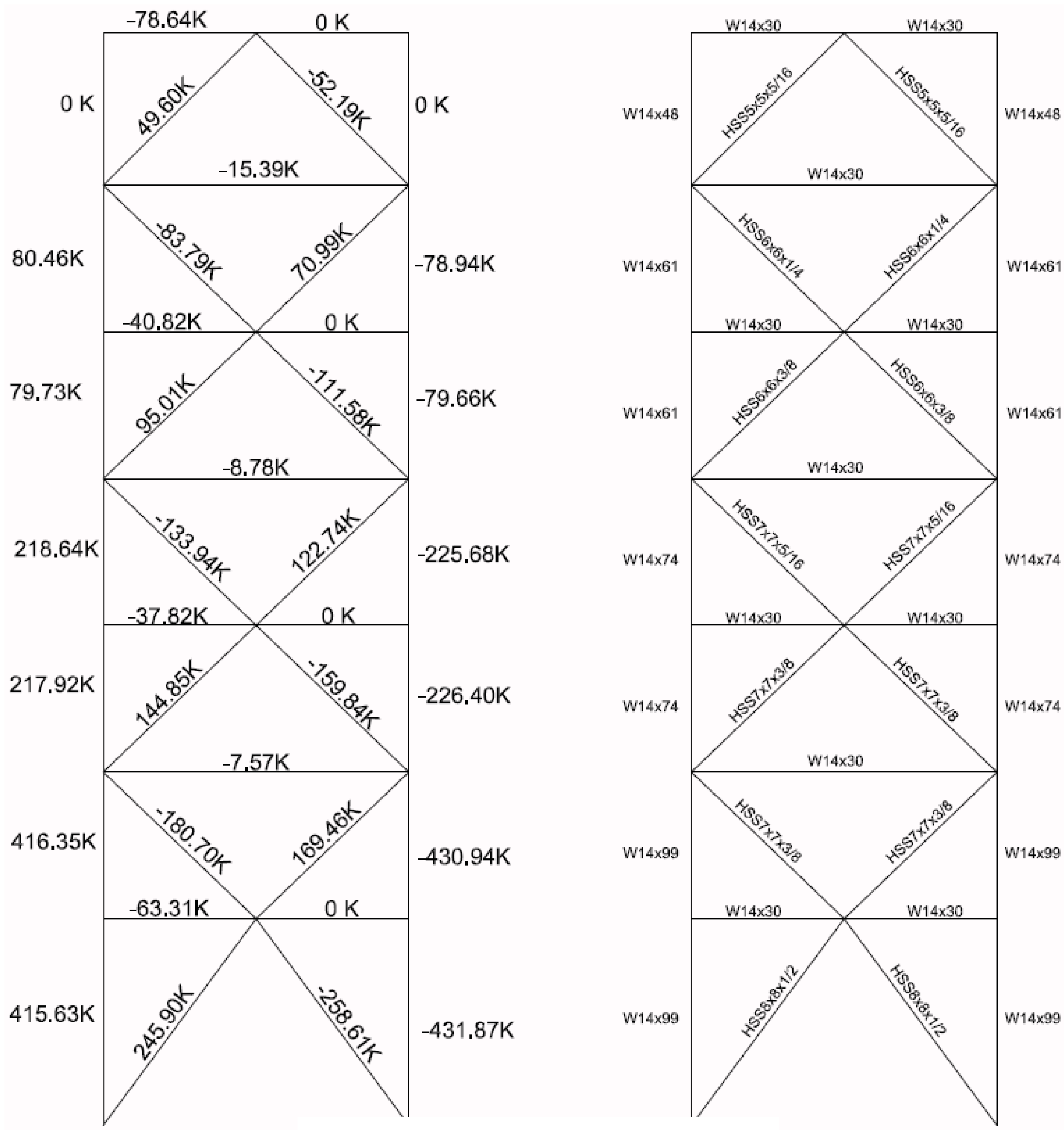

Figure 18: Typical E-W Braced Frame

The same method was used to determine the forces in the E-W direction braces as well. It is clear that the E-W direction braces will need to be much larger due to larger axial forces in the members. This makes sense because there are only two braces in the E-W direction, while there are four in the N-S direction.

When sizing the columns in the E-W direction, the actual sizes required ended up being slightly smaller than the final design shows. This is due to the fact that the E-W braces share columns with the N-S braces. Because of this, the larger N-S braced frame columns became the final size for the E-W direction as well. This can be seen in Appendix D. The final E-W direction design is show above.

Wind Drift

Wind forces were examined to determine if the overall building drift and the individual story drifts were acceptable. In general, drift should be limited as much as possible; however, a limit of 1/400th of the overall building height was used in this case. For this overall structure, the drift is limited to:

$$\Delta_{MAX} = (100.5' \text{ X } 12)/400 = 3.02''$$

After running the ETABS model for unfactored (serviceability consideration) wind forces in both directions, the following results were obtained:

Table 13: Wind Drift (X/ N-S)		
Level	Story Drift (in)	Total Drift (in)
1	0.3718	0.3718
2	0.1853	0.5571
3	0.1935	0.7505
4	0.2033	0.9539
5	0.1667	1.1205
6	0.1905	1.3110
7	0.1936	1.5046

Table 14: Wind Drift (Y/ E-W)		
Level	Story Drift (in)	Total Drift (in)
1	0.2918	0.2918
2	0.3390	0.6308
3	0.3127	0.9435
4	0.3715	1.3150
5	0.3356	1.6506
6	0.3721	2.0227
7	0.3260	2.3486

It is clear that the E-W direction drift is larger than the N-S drift, which seems logical due to the larger wind force in that direction, as well as a smaller building width. From the data, it is clear that the maximum building drift in both directions is acceptable as it is less than the allowable value of 3.02".

The individual story drift was also considered and compared against the allowable values shown in the table below:

Table 15: Allowable Story Drift	
Level	**Story Drift (in)**
1	0.570
2	0.405
3	0.405
4	0.405
5	0.405
6	0.405
7	0.420

These values were calculated using L/400, where L is the individual story height. It is clear when comparing with the actual drift values, that the overall building drift, as well as the individual story drifts, are acceptable for wind.

Seismic Drift

Seismic forces were examined to determine if the overall building drift was acceptable. For this overall structure, based on ASCE7-05 Chapter 12, Table 12.12-1, the overall drift is limited to:

$$\Delta_{MAX} = 0.020 \text{ X } (100.5' \text{ X } 12) = 24.12''$$

After running the ETABS model for factored (strength consideration) seismic forces in both directions, the following results (including secondary effects) were obtained:

Table 16: Seismic Drift (X/ N-S)		
Level	Story Drift (in)	Total Drift (in)
1	0.2523	0.2523
2	0.1460	0.3983
3	0.1609	0.5591
4	0.1705	0.7296
5	0.1367	0.8663
6	0.1403	1.0066
7	0.1046	1.1113

Table 17: Seismic Drift (Y/ E-W)		
Level	Story Drift (in)	Total Drift (in)
1	0.1195	0.1195
2	0.1546	0.2741
3	0.1499	0.4240
4	0.1776	0.6016
5	0.1565	0.7580
6	0.1558	0.9139
7	0.1050	1.0188

These drift values were adjusted based equation 12.8-15 of ASCE 7-05:

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

This resulted in respective amplified drifts of 3.34" and 3.06" for the N-S and E-W directions. These amplified drifts were found using a C_d factor of 3 for steel systems not specifically designed for seismic resistance and an importance factor of 1.0. It is clear that these values will not exceed the allowable value for the structure.

Torsion

Overall building torsion results from several scenarios. The largest and most common case of building torsion results from a center of mass that differs in location from the building's center of rigidity. This creates a case where the loads are applied at an eccentricity on the building. This eccentricity times the force results in a moment on the overall building. Torsion also can result from the accidental eccentricity caused by seismic forces as described in ASCE 7-05 Section 12.8.4.2. Additionally, Cases 2 and 4 from the previously considered wind cases can also result in an additional eccentricity causing torsion. In both of these wind cases, the eccentricity is equal to 15% of the building width.

As previously mentioned, due to this building's symmetrical geometric shape, as well as the symmetrical frame stiffnesses about the x and y axes, the center of mass and center of rigidity are both at the same location. This creates no torsion from eccentricity. In addition to this, it has been shown that seismic does not control and that Wind Case 1 is the controlling wind case. Taking all of this into account, it is clear that the overall torsion on the building due to these forces is negligible, resulting in negligible overall building torsion.

Overturning

Overturning issues can have an impact on a variety of building components, probably the most common of which is the building's foundations. Overturning occurs when the lateral forces on a building are not offset by the moment created by the building's self weight. This creates a scenario where uplift must be considered for the foundations. Foundations must utilize friction from the soil and self weight and are used in tension, rather than in compression.

Overturning moments can also have an effect on the columns in a building as well. Overall building moments are transferred through axial forces in the columns. These moments put some columns in compression, and others in tension. This is something that must be taken into account as well.

The following overturning moments were determined from taking the critical factored story shear from ETABS at each level and assuming that force acted at the floor level of each story. The height and force were used to determine the moments, which were summed to determine the overturning moment in that direction.

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Table 18: Seismic Overturning Moment (X/ N-S)			
Level	**Height**	**Story Shear (K)**	**Overturning Moment (ft - k)**
1	19	222	4218
2	32.5	211	6858
3	46	197	9062
4	59.5	173	10294
5	73	137	10001
6	86.5	86	7439
7	100.5	21	2111
		Total Moment:	49982

Table 19: Seismic Overturning Moment (Y/ E-W)			
Level	**Height**	**Story Shear (K)**	**Overturning Moment (ft - k)**
1	19	222	4218
2	32.5	211	6858
3	46	197	9062
4	59.5	173	10294
5	73	137	10001
6	86.5	86	7439
7	100.5	21	2111
		Total Moment:	49982

Table 20: Wind Overturning Moment (X / N-S)			
Level	**Height**	**Story Shear (K)**	**Overturning Moment (ft - k)**
1	19	336.4	6391
2	32.5	275.3	8946
3	46	240.1	11047
4	59.5	202.7	12061
5	73	163.4	11927
6	86.5	122.6	10607
7	100.5	79.6	7998
		Total Moment:	68977

Table 21: Wind Overturning Moment (Y/ E-W)			
Level	**Height**	**Story Shear (K)**	**Overturning Moment (ft - k)**
1	19	677.2	12866
2	32.5	550.7	17899
3	46	479.0	22034
4	59.5	403.1	23981
5	73	324.3	23677
6	86.5	242.8	21000
7	100.5	157.3	15804
		Total Moment:	137262

After calculation of the moment resulting from the building's self weight, it has been determined that overall building overturning will not occur. This was the case despite significantly reducing the building's weight in the redesign. This expectation was confirmed by the dead load moments of 895615 ft-k for the N-S direction and 510350 ft-k for the E-W directions. These moment calculations can be seen in more detail in Appendix D.

Although overall building overturning does not occur, there are several areas of the structure that may experience uplift. Uplift will likely occur at the base of the columns of the E-W direction braced frames. This tension force must be considered when designing the connection at the base of the brace frame columns. This force is relatively small, and is cancelled out by the gravity load of the parking levels. This prevents the uplift from occurring in any of the structure's foundations. See Appendix D for calculations.

Summary of Lateral Analysis

After analyzing the lateral loads from wind and seismic forces using the computer model which were confirmed by hand calculations, the following conclusions were determined:

- The primary controlling load case from ASCE7-05 was 0.9D + 1.6W.
- The controlling wind case was Wind Case 1.
- The center of mass and center of rigidity were both found to be at the geometric center of the structure.
- Overall building torsion was negligible.
- Overall building drift and story drift were found to be well within limitations.
- Overturning moment was found to not cause building overturning. Uplift will occur at the base of E-W frames, but not at any foundations.

Foundation Redesign

Due to the large self-weight of the original structure, large foundations were required to transfer the loads to the soil. In this analysis, five key mat foundations were redesigned to accommodate the reduced loads from the steel structure. The original design is shown below:

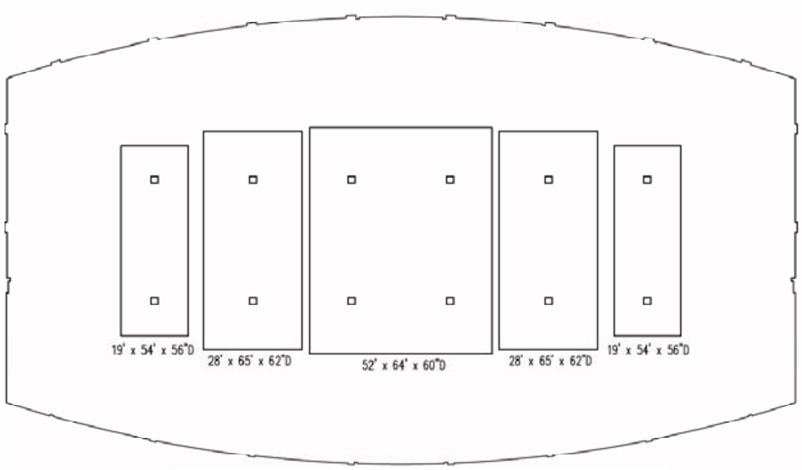

Figure 19: Original Foundations

After recalculating the loads at the base of the structure, it was determined that a series of 17' x 17' (and 34" Deep U.N.O.) square footings would be adequate to carry the required loading. Calculations are available in Appendix A. The redesign is available below:

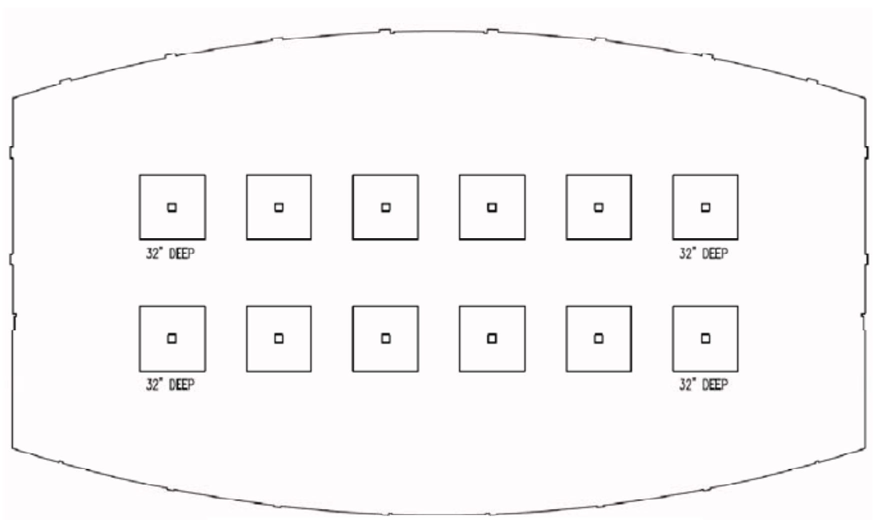

Figure 20: Redesign Foundations

All calculations are available in the appendix. The effects of this significant reduction in foundation size are explained in the Cost/Schedule section of this report.

Architecture Study

Using concrete moment frames in the original design of the structure was quite advantageous with regards to space planning. Because the columns and beams were heavily relied on to resist lateral forces, planning around shear walls and braced frames was not required. This allowed for uninterrupted open space throughout the floor plan.

The modification of the gravity system to a steel system required consideration of the new lateral system as well. It was determined after analyzing the bay size and floor plan layout, that the use of braced frames in both directions would be the most beneficial solution. This, however, created problems with maintaining the integrity of the tenant spaces that were to be leased. Any intrusion into the open space would create a less desirable and less profitable situation for the owner.

Because of this, the placement of the braced frames was a careful consideration when designing the lateral system. Ordinarily, placement of braced frames at a large eccentricity compared to the center of mass is desirable, as the frame would be more beneficial in resisting overall torsion of the structure. In this case though, this was not a factor due to the lack of torsion on the structure. Therefore, the placement of the frames near the core was valid. Placing the frames near the core was preferable, and allowed them to be placed inside of walls, where they would not intrude upon the tenant spaces.

The locations of the braced frames are shown in red on the following diagram, detailing how the frames interact with the usable spaces on the floor plan.

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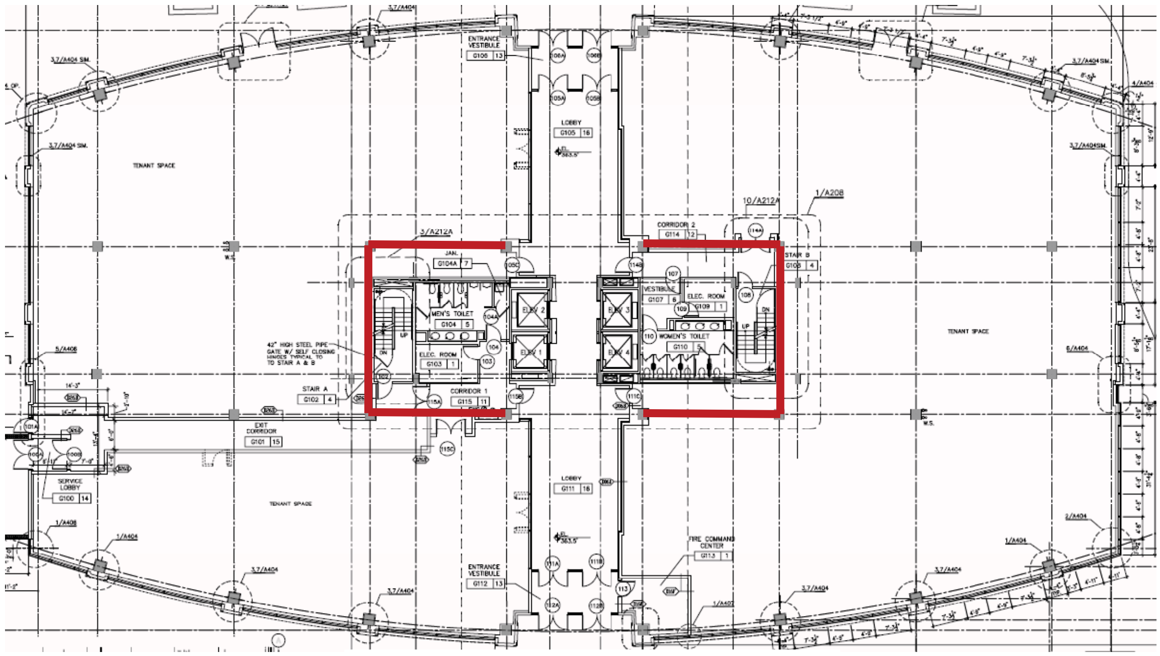


Figure 21: Braced Frame Locations

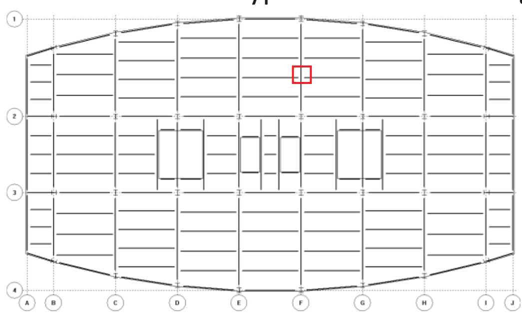
It is clear when examining the existing floor layout, that the location of the braces will have almost a negligible impact on the existing plan. The two braces running in the E-W direction fall directly along the existing exterior wall of the egress stair. The four frames in the N-S direction will have a slight impact on the placement of the existing doors. While these frames do lie along a planned wall, the placement of the door in this wall may have to be shifted to accommodate for the brace locations at that floor. This will have to be considered; however, it is anticipated that these changes result in negligible changes to the architectural floor plan for the structure.

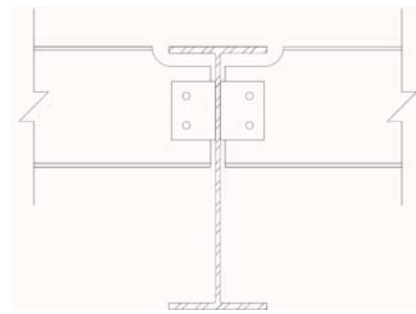
As was mentioned in a previous section of this analysis, the floor depth for the steel redesign was larger than the original floor depth. In order to maintain the same ceiling height and MEP spaces, this change in floor depth will result in a change in the overall height of the structure. As was shown previously, this change in building height will be roughly seven feet. It will be shown later in this report that this height increase could possibly be reduced beyond what has been shown previously. Regardless, it is clear that some increase in overall height would likely occur. This would result in a need for more square footage of building envelope, requiring slight architectural considerations as well as cost considerations.

It is anticipated that, despite the increase in height, the same architectural goals could be achieved with the structural envelope. The issue of cost would require consideration, which is investigated in the cost and schedule analysis of this report.

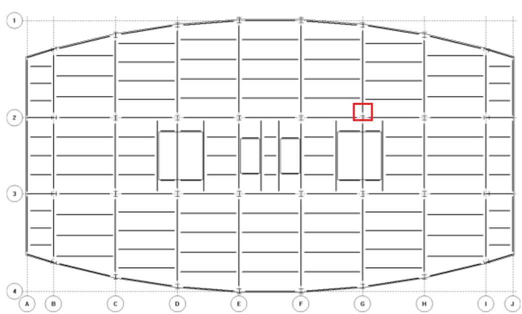
MAE Topic: Connection Design

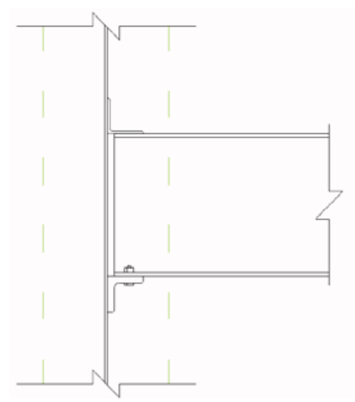
A typical beam-to-girder shear tab connection was designed for the location shown below. This was a typical scenario throughout the structure.



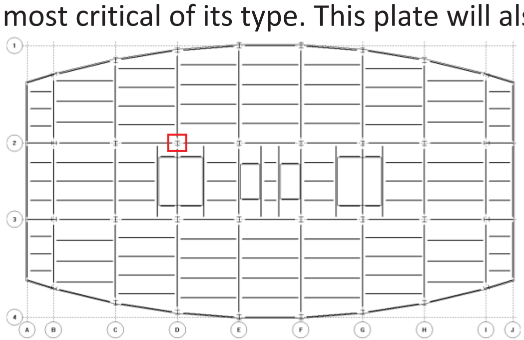


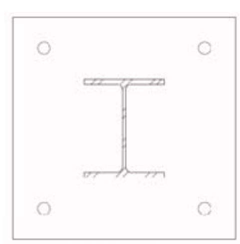
An unstiffened seat connection was designed for the connection of a girder to a column web. The location can be seen on the plan below. This would likely be a common connection in this scenario throughout the structure.





A base plate was designed for the location shown below. This base plate is located under the corner of the braced frames (connecting to the parking levels), and is the most critical of its type. This plate will also require anchorage design for uplift.





All calculations and details for connections are shown in detail in Appendix E.

Summary of Building Design

The redesign of the structure began with the idea of reducing the self-weight of the structure, which effectively requires reduced sizes in the members required to carry the gravity load. That includes mainly columns and foundations.

The gravity design began with choosing a slab thickness that was able to provide an adequate two hour fire barrier without using additional fireproofing sprayed on the decking, as that would not be cost efficient. Metal decking and slab thickness were considered to resist the required gravity loads from ASCE 7-05. Beams were laid out to maintain a ten-foot minimum span (in order to avoid the need for shoring), and minimize the number of beams required.

After laying out the slab and beams, the gravity columns were considered. In an effort to maintain a comparable structure to the original design, the same column grid was used in the redesign. All gravity columns were designed.

The cantilevers at the North and South sides of the structure presented a unique design challenge to the structure. Four beams were moment connected to columns and cantilevered out 12 feet to carry the cantilever loads back to the column. Moment connections were used on the interior of the columns as well, in order to balance most of the moment from the cantilever. This effectively reduced the amount of moment that the columns alone were required to carry.

The redesign of the structure also required the consideration of the lateral force resisting system. In this case, a series of braced frames were used to resist the lateral forces in both directions. The maximum design forces were considered in each column and brace, along with the gravity loads in the columns. These forces were combined using a variety of different load combinations. The members were then sized using the critical case. After the sizing of all the members, all of the load combinations and wind scenarios were analyzed on the structure, and overall building drifts were found to be within the allowable limits.

After completing the redesign of the superstructure, the foundations were analyzed with the reduced gravity loads. Because of the bracing due to the parking levels, it is clear that the moment transferred down to the foundations is negligible. The loads from the superstructure, in addition to the loads from the concrete parking levels were used to design new foundations. Smaller, 17' x 17' square foundations were found to carry the loads to the soil. These foundations took the place of the much larger mat foundations that were required in the original design.

Cost/ Schedule Analysis Study

A detailed cost estimate was completed for the original post-tensioned structure. All rebar, formwork, tendons, shoring/reshoring, and concrete were considered in the analysis.

The cost estimate for the redesign consisted of all beams, columns, fireproofing (applied to all beams and columns), studs, metal decking, welded wire fabric, lightweight concrete, and the cost of foundations. A 3% adjustment factor was used to account for base plates, and a 10% adjustment factor was used to account for connections and column splices, as specified by R.S. Means Unit Price Estimating Methods, 4^{th} Edition.

The cost savings values considered in this report were solely a factor of the material, labor and equipment costs. Although the general conditions were calculated for this report, the $82,000 per month general conditions cost was not taken into account in the project savings. Although the structure most likely falls on the critical path, it was assumed for this report that the end date of the project remained unchanged, despite the shortened schedule for the structure alone.

Schedule durations are an important consideration for choosing an effective structural system. The structure will likely require a large portion of the construction time for the project, and needs to be considered accordingly. Summaries of these schedules are shown in Appendix E of this report. The results showed that the steel structure could be completed roughly 13 months faster than the original design. This makes sense due to the quick erection of steel members, the reduced foundation sizes, and the time required for concrete forming, reinforcing, and curing in the original design. While the potential cost savings of this reduction were not considered, it is clear that additional time built into the schedule would occur, at the very least.

Overall results of the cost and schedule analysis are available below:

Table 22: Original Structure					
	Mat'l	**Labor**	**Equipment**	**Total**	**COST/SF**
Foundations	$272,327	$59,403	$250	$331,980	$1.90
Superstructure	$2,532,939	$1,594,087	$48,370	$4,175,396	$23.86
Total Incl. Additional Costs					**$27.83**
Steel Redesign					
	Mat'l	**Labor**	**Equipment**	**Total**	**COST/SF**
Foundations	$54,082	$17,076	$1,874	$73,033	$0.42
Superstructure	$2,669,627	$290,079	$114,563	$3,074,269	$17.57
Total Incl. Additional Costs					**$19.43**

By analyzing the results in Table 22, it is clear that the proposed system will result in significant cost savings over the original design. However, these numbers do not take into account the increased building envelope costs due to the building height increase that will be required. Assuming a building envelope cost of $50 per SF and a seven foot increase in height, the cost increase would be $224,000 due to the extra envelope costs.

This would result in a steel cost increase of $1.26 per SF, resulting in a final steel cost of around $20.69 per SF of floor area. This results in slightly more than a 25% cost reduction for the overall structural system for the project.

Conclusion

The 20" total floor depth of the original post-tensioned structure was one large advantage of the original system. Changing to a steel system resulted in an increase of the floor depth, which resulted in increasing the overall building height in order to maintain the same ceiling heights as well as MEP spaces. In this analysis, a 27" deep beam was taken to be the critical member, resulting in a new floor depth of 32.5". This increase in the height of the structure resulted in increased costs for the structure and also for the building enclosure.

To go back and reanalyze the critical floor depth, it is obvious that further improvements could take place. Only ten beams per level exceed 21" deep. It would be useful to constrain these beam depths in order to decrease the overall floor depth. The deepest of these beams (W27x84) could be changed to a W21x93. If this constraint was applied to all ten beams, the floor depth would become only six inches deeper than the original post-tensioned design, not 12" as used in the analysis. This would result in a building height increase of only 3.5' rather that seven feet; further reducing lateral loading and the square footage of building enclosure required. These benefits would likely outweigh the slightly higher cost of the shallower, heavier beams on each floor.

One last notion that could improve the overall design would be a closer look at the cantilever situation at both ends of the structure. For this analysis, the cantilever distance and the column grid were kept the same. In an ideal redesign, more thought would have been put into planning the balancing moments at this location. Due to these constraints, the cantilever side had a moment of 575 ft-k, while the interior connection of each column only had a moment of 376 ft-k. This left an unbalanced moment of 199 ft-k at each floor. This left a significant moment for the columns to maintain, especially at the lower levels of the structure, resulting in very large column sizes.

In an ideal redesign, the cantilever distance would be smaller to reduce the exterior moment, or the interior span would be larger to increase the moment at the interior of the column. More effective planning at balancing these moments could have resulted in significantly lesser moments taken by the columns. This would have resulted in much smaller column sizes at these locations and even further cost savings for the project.

Based on the analysis performed on the structure, it appears that the proposed redesign will have the benefits that were initially expected. By changing to a steel composite system, the self-weight of the structure is greatly reduced, and the member sizes can be reduced due to the lesser loads. Large cost savings result, due to the material, labor, and equipment costs associated with the structure. For this report, general conditions savings were not considered; however, further savings could potentially occur depending upon the exact ramifications these changes would have on the schedule duration. Overall, it appears that the proposed redesign could have been a viable and beneficial alternative for this project.

Resources

ACI 318-08: Building Code Requirements for Structural Concrete and Commentary.

AISC Steel Construction Manual, Thirteenth Edition.

ASCE 7-05: Minimum Design Loads for Buildings and Other Structures.

R.S. Means Building Construction Data, 2009.

Unified Design of Steel Structures. Geschwinder, L.