Kaleida Health – Global Heart and Vascular Institute
University at Buffalo – CTRC/Incubator

Buffalo, New York

Final Report

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Structural Option
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Kaleida Health Global Vascular Institute
University at Buffalo CTRC/Incubator
Buffalo, NY

Building Information

Size: 476,500 sf
Number of Stories: 10
Dates of Construction: February 2008 - April 2011
Project Delivery Method: Guaranteed Maximum Price
Overall Project Cost: $291 million

Primary Project Team

Owner(s): Kaleida Health &
Buffalo 2020 Development Corporation
Architect and Engineers: Cannon Design
Construction Manager/General Contractor:
Turner Construction

Structural System

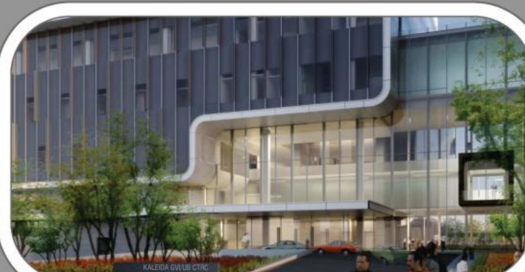
- Foundation consists of grade beams and steel helical piles driven to refusal on limestone bedrock
- Sub basement is a 5" slab on grade
- Remaining floors are composite metal deck with slab thickness ranging from 4" to 7.5"
- Lateral forces are resisted by braced frames
- Typical beam size is W18x40 but sizes vary throughout the structure based on the function of the spaces

Architecture

- State-of-the-art medical facility
- Includes exam rooms, classrooms, offices, a cafe, a wellness center and library, and a research facility
- "Collaborative Core" enables interaction from all levels
- Universal grid design and 18' floor-to-floor height
- Curtain wall facade composed of aluminum and metal panels as well as various types of glazing

MEP Systems

- Steam and chilled water served by campus power plant
- Heating, cooling, and ventilation primarily achieved using a variable air volume system
- Lab exhaust and atrium smoke exhaust systems provided for specialized areas
- 4160V primary service supplied by campus power plant and stepped down to 480Y/277V and 120/240V
- 3 diesel powered 1825 kW emergency generators



William C. McDevitt **Structural Option**

http://www.engr.psu.edu/ae/thesis/portfolios/2011/wcm5016/index.html

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Executive Summary

The following document is the final report for senior thesis and includes information regarding the Kaleida Health and University at Buffalo, Global Heart and Vascular Institute. This project will be referred to throughout this report simply as GHVI. This report includes information regarding the building's existing structural system, a gravity and lateral system concrete redesign, a vibration analysis, a construction management breadth, and a mechanical breadth.

GHVI is a ten story medical facility in the city of Buffalo, NY. The building is square in shape with a length and width of 221 feet, and a height of 185 feet. The foundation is made of grade beams and steel helical piles that are driven 82 to 87 feet deep. Floor construction entails composite metal deck resting on steel superstructure. A standard bay size of 31'-6" by 31'-6" is used throughout the building, utilizing W14 columns of varying weight to make up the gravity system. The lateral system is comprised of braced frames which are located near the perimeter of the building.

As part of the gravity system redesign, the three alternative floor systems explored in Technical Report 2 were reevaluated, and the flat slab system with drop panels was chosen as the best option. This system was designed to meet ACI minimum thickness requirements and resist all instances of punching shear. The second part of the gravity redesign was conducted, using RAM Structural System, spColumn, and hand calculations to determine column sizes and reinforcing. The lateral system was redesigned using reinforced concrete shear walls, and drift and relative stiffness checks were performed with the help of an ETABS model.

Due to the large amount of laboratory and procedural space in the building, GHVI is currently designed to meet minimum vibrational velocities. As a part of this thesis, the redesigned concrete floor slab was analyzed using SAP2000 to determine if it did in fact meet those velocity requirements.

A construction management breadth was undertaken for the purpose of comparing the existing steel structure with the redesigned concrete structure. A detailed cost estimate and a schedule analysis were performed for both materials to determine if the concrete building would in fact be more cost effective than the steel building.

In order to reduce the cooling loads of the building and create a more sustainable facility, a mechanical breadth study was performed. Various glazing configurations were investigated and modeled using Trace 700.

Acknowledgements

Thank you to the following people for their help and support throughout the adventure of senior thesis:

Thank you to all of the AE Faculty and Staff, who have shared their wisdom and knowledge with me over these last five years, especially,

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Mickey Mariacher

And a special thanks to my family and friends. Without your love and support it would have been impossible to make it through these last five years.

Finally, I dedicate this report to my mother. Among other things, you taught me to never give up, even when things become difficult. I love you. I miss you.

Introduction

GHVI is a state-of-the-art medical facility and a fundamental component in a joint undertaking between Kaleida Health Systems and the University at Buffalo School of Medicine. The building spans ten levels and includes exam rooms, classrooms, offices, a café, a wellness center and library, and a research facility. It is intended to bring patients, surgeons, and researchers together to collaborate in an unprecedented way.

Key themes considered throughout the design were collaboration, flexibility, and comfort. Kaleida Health Systems sought a structure that would link clinical and research work and combine all vascular disciplines. A spirit of collaboration was the driving force behind bringing both Kaleida and the University at Buffalo together in a single structure. Keeping this in mind, the design team developed the facility with a "collaborative core" which enables interaction among those working within the facility. This collaborative learning environment brings together research, ideas, and solutions and results in better patient care.

A universal grid design increases the flexibility of space and achieves measurable advantage in initial capital cost, speed to market, operating economy, and future adaptability. The universal grid is comprised of three 10'-6" building modules and forms a 31'-6" x 31'-6" structural grid capable of integrating the building's diverse functions. When combined with an 18' floor-to-floor height, the flexible grid creates an open plan capable of adapting to present and future healthcare needs. The building will be able to incorporate unknown, but rapidly changing technological developments within the industry, also giving it longevity through its adaptability.

With comfort in mind, a separate "hotel" level was designed on the second floor and separated from the procedural floors. Functionally, the "hotel" is comprised of private patient rooms and a small lounge area. Other family lounges are also provided and the perimeter of the building is shaped to bring in as much natural daylight as possible. The vision of GHVI is to create an atmosphere that is more than a simple hospital, but instead a facility for world-class treatment and state-of-the-art technology.

Existing Structural System Overview

Foundation

Based on the recommendations of the October 2008 Geotechnical Report by Empire Geo-Services, Inc., the foundation of GHVI consists of grade beams and pile caps placed on top of steel helical piles.

The helical piles are HP12x74 sections with an allowable axial capacity of 342 kips (171 tons) which are driven to absolute refusal on limestone bedrock 82 to 87 feet below the sub-basement finish level. Grade beams and pile caps have a concrete strength of 4000 psi, and it should be noted that the width of the grade beams equals that of the pile caps at the foundations of the braced frames. The grade beams provide resistance to lateral column base movement, and the pile caps link the steel helical piles and the structural steel columns of the superstructure.

Spanning the grade beams is the sub-basement floor, a 5” slab-on-grade. Due to the slope of the site, part of this sub-basement is below grade, and therefore a one foot thick foundation wall slopes along the west elevation of the sub-basement.

Floor System

The floors of GHVI consist of 3” composite metal deck with a total slab thickness ranging from 4” to 7½”. The metal deck is 18-gage galvanized steel sheets resting on various different beam and girder sizes. These sizes change throughout the structure because of the various functions of the spaces. The bay sizes through the building are mostly 31’-6” by 31’-6”, with beams spaced at 10’-6”. As was discussed in the introduction, this universal grid design increases the future flexibility of the space. A slight variation in the floor can be seen on Levels 6-8. On these levels, part of the floor structure is left open to provide for the collaborative atrium that was designed to bring the various disciplines together.

Gravity System

Steel columns are used throughout the building to transmit the gravity load to the foundation. All of the columns in the building are W14s, but they range in weight from 68 lb/ft to 370 lb/ft, and they are typically spliced every 36 feet. These columns provide an 18’ floor-to-floor height, which also contributes to the universal grid and future flexibility of the space.

Lateral System

The lateral system of GHVI utilizes braced frames located near the perimeter of the building, all of which are HSS sections. A braced frame system is ideal in steel buildings because of its low cost compared to moment connection frames. There are moment connections in some parts of this structure, but they are used to support the small amount of slab overhang that is cantilevered. These moment connections may actually add some stiffness to the lateral system, but they cannot be included in the lateral system design. Figure A depicts the location of the braced frames on the outer part of the structure.

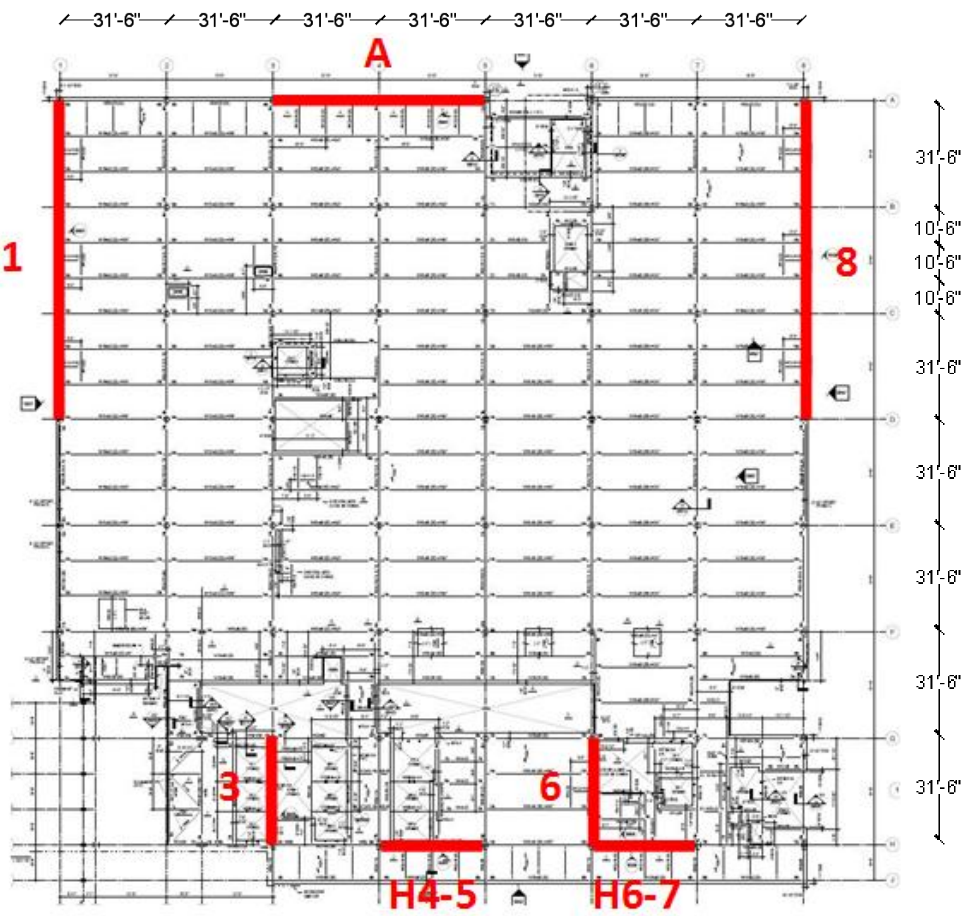

Figure A – Level Two Framing Plan with Braced Frames Highlighted (Cannon Design)

Codes and References

Original Design Codes

- Model Building Code:
  Building Code of New York State 2007

- Design Codes:
  "Load and Resistance Factor Design Specification for Structural Steel Buildings," AISC

  "Code of Standard Practice for Steel Buildings and Bridges", AISC

  "Manual of Steel Construction - Load and Resistance Factor Design," AISC

  ACI 318-05, Building Code Requirements for Structural Concrete

  American Society of Civil Engineers, ASCE/SEI 7-02,
  Minimum Design Loads for Buildings and Other Structures

Thesis Design Codes

- National Model Building Code:
  2009 International Building Code

- Design Codes:
  Steel Construction Manual 13th edition, AISC

  ACI 318-05, Building Code Requirements for Structural Concrete

  PCI Design Handbook, 6th Edition

  RSMeans Building Construction Cost Data

  American Society of Civil Engineers, ASCE/SEI 7-10,
  Minimum Design Loads for Buildings and Other Structures

  AISC/CISC, Design Guide 11, Floor Vibrations due to Human Activity

- References:
  Vibration Design of Concrete Floors for Serviceability, ADAPT,
  Bijan O Aalami, 2008

  Reinforced Concrete: Mechanics and Design, Macgregor, 2009

- Deflection Criteria:
  Allowable Building Drift (Wind) = H/400

  Allowable Story Drift (Seismic) = $0.010h_{sx}$

Materials

Original Design

Structural Steel:

Type	Standard	Grade
Wide Flange Shapes, WT's	ASTM A-992	
Channels & Angles	ASTM A-36	
Pipe	ASTM A-53	Grade B
Hollow Structural Sections (Rectangular & Round)	ASTM A-500	Grade B
Base Plates	ASTM A-572	Grade 42
All Other Steel Members	ASTM A-36	

Concrete:

Type	f'c (psi)	Unit Weight (pcf)
Pile Caps	4000	150
Grade Beams	4000	150
All Other Concrete	4000	150
Slabs-On-Grade	3000	150
Foundation Walls	4000	150

Reinforcing:

Type	Standard	Grade
Typical Bars	ASTM A-615	60
Welded Bars	ASTM A-706	60
Welded Wire Fabric	ASTM A-185	
Steel Fibers	ASTM A-820	Type 1
Bars Noted To Be Field Bent	ASTM A-615	40

Connectors:

Type	Standard
High Strength Bolts, Nuts, & Washers	ASTM A-325 or A-490 (min. 3/4 Diameter)
Anchor Rods	ASTM F1554
Welding Electrode	E70XX
Steel Deck Welding Electrode	E60XX min.

Redesign

Concrete:

Type	f'c (psi)	Unit Weight (pcf)
Columns	6000	150
Slabs	6000	150
Drop Panels	6000	150

Reinforcing:

Type	Standard	Grade
Typical Bars	ASTM A-615	60

Problem Statement

The current design of GHVI utilizes a steel superstructure which rests on steel helical piles driven to bedrock. The superstructure is made up of W14 shapes for columns, a composite metal deck flooring system, and braced frames on the perimeter of the building. A universal grid was designed with 31'-6" by 31'-6" bays and beams spaced at 10'-6". The structure is built using a 3" composite metal deck with a total slab thickness ranging from 4" to 7½", depending on the level and its live load requirements. The metal deck is 18-gage galvanized steel and rests on beams and girders of various sizes. Shear studs are used on the beams and girders in order to create composite action with the slab. A section of a typical composite deck can be seen in Figure B.

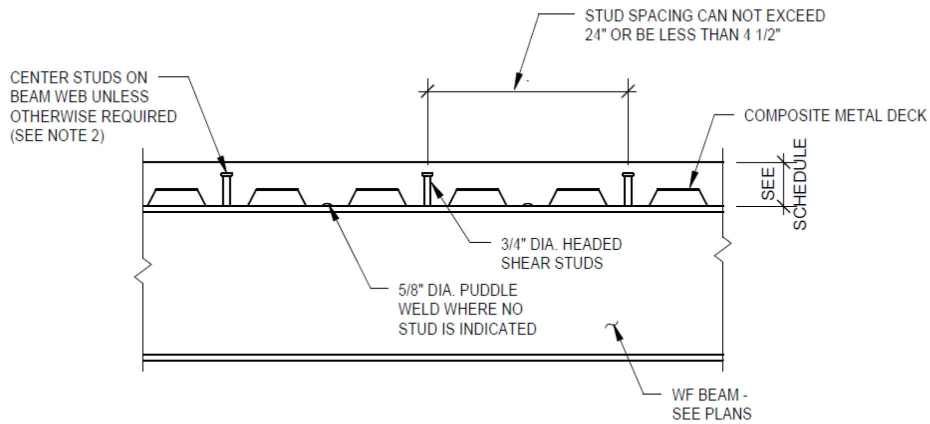

Figure B – Typical Composite Steel Construction (Cannon Design)

A composite deck system is good for long spans and heavy loads. It results in a structure with a reduced weight that is easy to design and quick to construct. However, as was reported in Technical Report 2, steel construction may be the more costly method. Using RSMeans Building Construction Cost Data, the steel system was found to be more expensive than a flat slab system with drop panels. The assembly of a steel structure requires high labor costs, especially when it comes to assembling connections. Steel also requires spray on fireproofing, which is not required by concrete systems. A redesign to a concrete structure may result in a project with a lower cost.

Proposed Solution

Structural Depth

In an attempt to reduce the current cost of GHVI, the building will be redesigned using one of the three reinforced concrete systems explored in Technical Report 2. In that report, it was concluded that a flat slab system with drop panels was most likely the most efficient and cost effective option to replace the current system. This design would require a slab thickness of 11" to meet ACI 9.5.3.2, and would employ 3½" drop panels to resist the large live loads of GHVI. A flat slab system would not result in a change to the current bay size, and would allow for a relatively flat ceiling.

The second alternative system is a one-way pan joist and beam system. This will be considered because it is normally adequate for long spans and heavy live loads. A 4½" slab will be used to meet a two hour fire rating. A 72" pan joist module will be implemented, consisting of 66" pans and 6" ribs as prescribed by ACI requirements. The ribs will be 16" deep, making the total structure thickness 20½". Although this system can carry heavy live loads, it would require an adjustment to the current bay size and column grid, and would entail the use of complex formwork.

The final system to be considered is a pre-cast hollow core plank design. Because the pre-cast planks come in 4' sections, the standard bay size of the building would be altered from 31'-6" by 31'-6" to 32' by 32'. This change is minimal, would be easy to implement, and would have a lower cost than ordering specially designed pre-cast planks. Using the PCI Design Handbook, it was found that 4'-0" by 10" planks with 2" of topping would be sufficient. It may also be necessary to design this system using post-tensioned strands.

After these systems are compared and the best is chosen, it will be necessary to redesign the columns and the lateral force system. First, the column grid layout will be revised if necessary, and then columns throughout the structure will be redesigned. It must also be investigated whether or not the inherent moment connections of the reinforced concrete structures will be enough to resist the lateral load. If this is not true, shear walls will need to be placed throughout the structure. This must be carefully planned so as to not disrupt the flow of the current structure. A vibration study will be conducted to assure that the new design meets the standards of the current building.

Breadth Topic 1 – Cost and Schedule Analysis

An in-depth cost and schedule analysis will be conducted on the redesigned reinforced concrete system. The first step in this analysis will be to determine the cost of the current steel structure using RSMeans Building Construction Cost Data, and to find or develop a schedule for the current construction process. The purpose of developing both of these items will be to create a baseline for equivalent comparison after the concrete design is completed. As soon as the concrete structure has been completed, a detailed cost breakdown and schedule will be developed. This breakdown will then be compared to the original cost and schedule information to determine if the proposed structural redesign is in fact more efficient.

Breadth Topic 2 – Building Envelope and Façade Study

A mechanical breadth into the building envelope and façade will also be performed as a part of this thesis. The current curtain wall designs will be obtained from Cannon Design, and research will be conducted to determine a more efficient type of glazing, with the intent of creating a more sustainable facility. Thermal calculations will be performed for a room on each of the four façades, and the effects on the lighting of the building will also be considered.

MAE Requirements

MAE Requirements for this thesis will be met using methods from both AE 597A, Computer Modeling of Building Structures, and AE 542, Building Enclosure Science and Design. By building a detailed computer model in ETABS, material taught in AE 597A will be applied to this thesis. The building envelope and façade breadth will also implement material that is covered in AE 542, including glass type and thickness, as well as thermal, lighting, and acoustic considerations. Finally, the vibration research and analysis will constitute a MAE level of work.

Building Loads

Floor Dead Loads

The dead load placed on the building during the redesign consisted of self-weight and the superimposed dead load used by Cannon Design in the original design.

Superimposed Dead Load

MEP	15.0 psf
Ceiling	5.0 psf
Leveling Concrete for Deflection	5.0 psf
Total	25.0 psf

Floor Live Loads

The live loads shown below are a combination of information obtained from Cannon Design and values determined from ASCE 7-10.

Occupancy or Use	**Design (psf)**	**ASCE 7-10 (psf)**
Vivarium	80	60
Hotel (Patient) Floor	125	40
Procedure and Lab Floors	125	60
Mechanical Floors	150	--
Mechanical Floors with Catwalks below	175	--
Electrical Floors	200	--
Mechanical Mezzanine (Low)	40	40
Storage	--	20
Lobby	--	100
Stairs	--	100
Corrridors	--	100
Roof	--	20

It should be noted that there is a large difference between the live loads used by Cannon Design and the live loads referenced from ASCE 7-10. This difference can most likely be attributed to the fact that the building was designed to adapt to the ever changing needs of the healthcare industry. By over-designing the floors, it can be assured that they can be used for a variety of functions in the future without the need for redesign and renovation. For simplification purposes during the redesign, a live load of 125 psf was conservatively assumed for all levels.

Wind Loads

The wind loads for GHVI were analyzed using Chapters 26 and 27 of ASCE 7-10. Wind loads for the Main Wind-Force Resisting System were determined using the directional procedure for buildings of all heights. Based on an occupancy category of IV, a basic wind speed of 120 mph was used to find the windward and leeward pressures. By code, flexible buildings can be affected by wind gusts and have the potential for resonance response. Because this building is considered flexible, a gust-effect factor also had to be determined. Detailed calculations including the initial parameters, an effective length check, gust-effect factor calculations, and wind pressure coefficients can be found in Appendix B.

The location and direction of the impact of wind on a building is difficult to predict, and so ASCE 7-10 requires that four cases be considered when applying wind. These cases can be seen in Figure C below. For each of the four cases the wind pressures, eccentricities, and torsional moments were tabulated in Microsoft Excel. The wind pressures and torsional moments for the four different wind cases can be found in Appendix B.

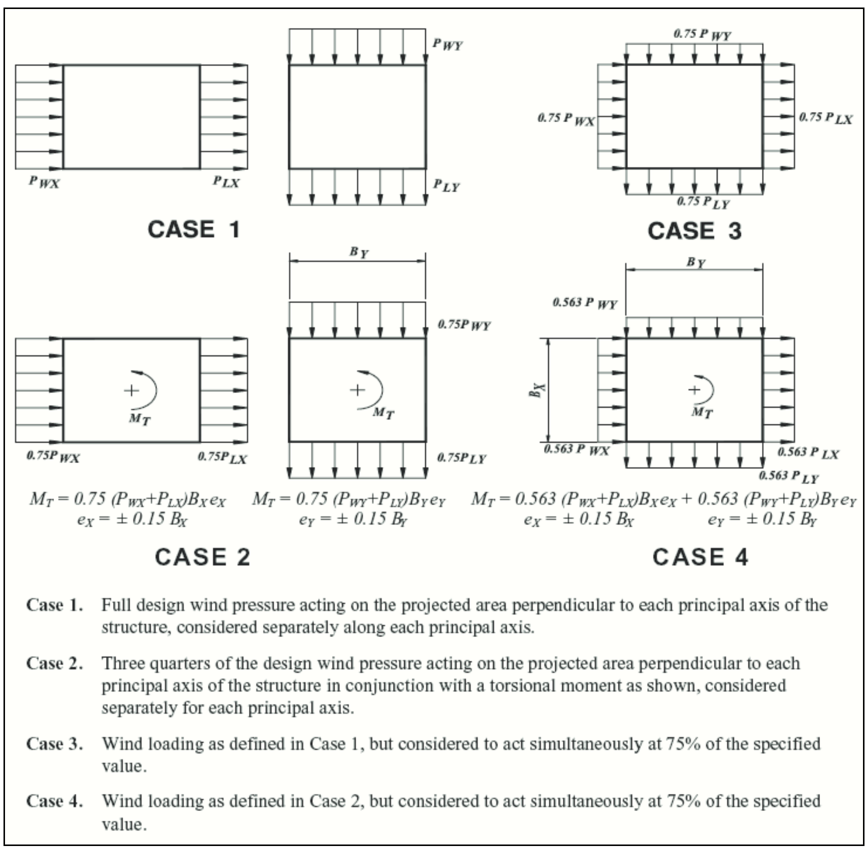

Figure C – Design Wind Load Cases (ASCE 7-10 Figure 27.4-8)

To determine which of the cases controlled, the wind story forces and torsional moments were applied to the ETABS model in four separate analyses. With the wind loading in question applied, the deformed shapes of the shear walls were viewed, and section cuts were drawn through the base of each of the walls. The shear in the walls was recorded in Table 1 below, and the load case with the greatest shear, Case 1, was determined to be the controlling wind load case.

Load Case	Direction	Shear in Wall (k)			
		A	H	1	8
1	W_x	730.0	784.4	11.0	4.0
	W_y	104.1	106.2	788.0	612.8
2	W_x	541.1	606.7	13.2	7.8
	W_y	84.4	86.3	597.3	464.3
3	W_{xy}	475.2	668.0	593.9	668.0
4	W_{xy}	342.9	511.3	434.5	354.2

Table 1 – Determination of Controlling Wind Load Case

Wind Story Forces							
		Load (kips)		Shear (kips)		Moment (ft-kips)	
Level	Height (ft)	N-S	E-W	N-S	E-W	N-S	E-W
Roof	185	57.3	80.4	0.0	0.0	10599.02	14872.28
9	169	146.3	169.4	57.3	80.4	24730.99	28634.67
8	151	176.4	176.4	203.6	249.8	26634.72	26634.72
7	133	172.8	172.8	380.0	426.2	22985.1	22985.1
6	115	168.8	168.8	552.8	599.0	19415.03	19415.03
5	97	164.5	164.5	721.7	767.9	15956.65	15956.65
4	79	159.7	159.7	886.2	932.4	12616.63	12616.63
3	61	153.7	153.7	1045.9	1092.1	9375.787	9375.787
2	43	126.2	126.2	1199.6	1245.8	5427.234	5427.234
1	30	85.0	85.0	1325.8	1372.0	2550.022	2550.022
Mechanical	21	51.9	51.9	1410.8	1457.0	1089.248	1089.248
Basement	16	72.8	72.8	1462.7	1508.9	1164.818	1164.818
	Total	1535.5	1581.7	1535.5	1581.7	152545.3	160722.2

Table 2 – Wind loads, shears, and moments calculated for each story

With Case 1 determined to be the controlling design wind load case, story forces, story shears, and the total base shear were computed. From Table 2 it can be seen that there is a base shear of 1535.5 kips in the North-South direction and 1581.7 kips in the East-West direction. This is expected, due to the fact that the area of wind projection decreases slightly at the roof level in the North-South direction.

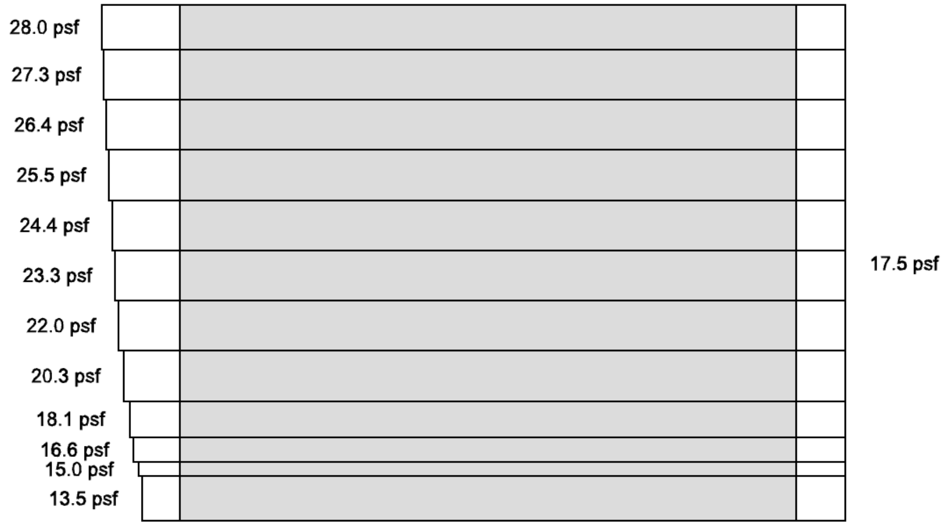

Figure D – Wind pressure diagram for East-West direction

Figure D shows the wind pressure diagram for the East-West direction. The windward loads are on the left, and the leeward loads are on the right. Figure E shows the wind force diagram and the base shear the building experiences.

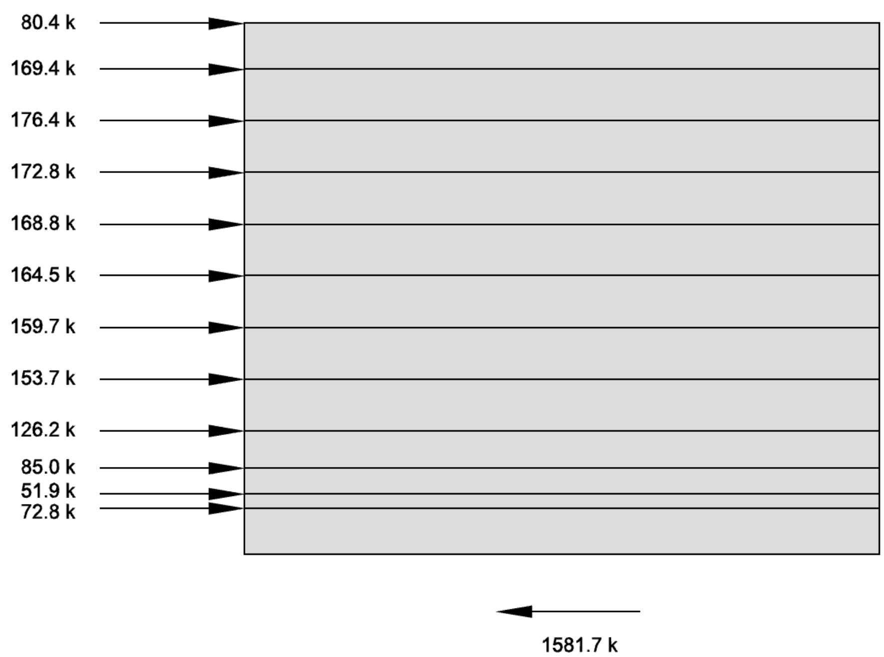

Figure E – Wind force diagram for East-West direction

Seismic Loads

Seismic analysis for GHVI was done with reference to Chapters 11 and 12 in ASCE 7-10. Because the building is relatively square, both the north-south and east-west directions were considered the same. The first step in this analysis was the estimated summation of the entire building weight above grade, which included the columns, slabs, drop panels, exterior walls, superimposed dead load, and partitions of each level. An Excel spreadsheet was set up to go through the building floor-by-floor and estimate as precisely as possible the building weight. The estimated building weight was found to be 86240 kips, which is 33604 kips more than the estimated weight of the steel structure. The Equivalent Lateral Force Procedure was then used to determine the base shear, and this base shear was then distributed to the diaphragm of each level as seen in Table 3. A more detailed set of calculations can be found in Appendix C.

Level	h_i (ft)	h (ft)	w (k)	$w*h^k$	C_{VX}	f_i (k)	V_i (k)	M_i (ft-k)
Roof	16	185	4030	5038548	0.105	145	145	26907
9	18	169	7441	8220803	0.172	237	383	40104
8	18	151	8787	8323951	0.174	240	623	36282
7	18	133	8787	6998877	0.146	202	825	26870
6	18	115	8787	5737996	0.120	166	991	19048
5	18	97	9203	4762621	0.100	137	1128	13335
4	18	79	9630	3765258	0.079	109	1237	8586
3	18	61	9711	2667069	0.056	77	1314	4696
2	13	43	9303	1584711	0.033	46	1360	1967
1	9	30	2167	225691	0.005	7	1366	195
Mechanical	5	21	5617	359437	0.008	10	1376	218
Basement	16	16	2777	122592	0.003	4	1380	57
		Σ =	86240.43	47807553	1.000	1380		178264

Table 3 – Seismic Design Loads

Table 3 shows a total base shear of 1380 kips, and a moment of 178264 foot-kips. The total base shear of the steel structure was calculated to by 1316 kips, which means the base shear has increased slightly by 64 kips. This increase was expected due to the increased weight of the building, but was not drastic due to the decrease in the building's fundamental period and the increase in R value.

Snow Loads

Snow loading for GHVI was calculated based on Chapter 7 in ASCE 7-10. A ground snow load of 50 psf was determined from a site-specific case study provided by Cannon Design. The exposure factor, thermal factor, and importance factor were then obtained from the code and used to calculate the flat roof snow load of 42 psf, which matched the value obtained by the design engineers. Because part of the roof is lower than the rest of the building, drift calculations were performed to find the maximum snow loading in these areas. The detailed calculations for snow loading can be found in Appendix D.

Load Combinations

There are 13 basic load combinations prescribed by ASCE 7-10 section 2.3.2 that were considered for this building:

1) $1.4D$
2) $1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$
3) $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + L$
4) $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + 0.5W_x$
5) $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + 0.5W_y$
6) $1.2D + 1.0W_x + L + 0.5(L_r \text{ or } S \text{ or } R)$
7) $1.2D + 1.0W_y + L + 0.5(L_r \text{ or } S \text{ or } R)$ ⟵ Controlling Load Combination
8) $1.2D + 1.0E_x + L + 0.2S$
9) $1.2D + 1.0E_y + L + 0.2S$
10) $0.9D + 1.0W_x$
11) $0.9D + 1.0W_y$
12) $0.9D + 1.0E_x$
13) $0.9D + 1.0E_y$

Considering both X and Y directions, the 13 different load cases were input into ETABS for analysis. Snow load was previously calculated and is larger than the roof live load and the rain load. Therefore, the snow load controlled in any combination that included these three load types. Also, in the combinations with wind or earthquake, both an East-West (X) direction and a North-South (Y) direction were considered.

After checking the shear in the base of each wall under each combination, it can be concluded that combination seven controls the design of this building. This case includes dead load, live load, snow load, and wind load in the Y direction. This is reasonable because the wind load in the Y direction has a larger base shear than the wind load in the X direction and the earthquake loads in both the X and Y direction. Refer to Table 4 on the following page for a summary of the shear at the base of each wall under the different load combinations.

Load Combo	Shear in Wall (k)			
	A	H	1	8
1	3.3	4.3	1.5	0.9
2	2.9	2.7	5.0	3.8
3	2.1	1.7	3.7	2.6
4	371.7	399.5	7.8	1.3
5	52.8	56.8	395.4	305.6
6	727.9	786.1	7.5	1.6
7	106.1	107.8	804.9	623.8
8	647.7	675.6	7.6	0.1
9	93.2	94.4	711.4	575.6
10	739.9	784.2	10.1	0.5
11	104.5	108.8	799.1	622.6
12	647.1	673.7	2.2	1.8
13	92.2	92.5	708.8	583.3

Table 4 – Determination of Controlling Load Combination

Gravity System Redesign

Flat Slab with Drop Panel Design

After exploring three alternative floor systems in Technical Report 2 and reinvestigating those systems for this thesis, a flat slab system with drop panels was chosen for the redesign of this structure. Not only did the flat slab system have the lowest cost, but it will not disrupt the building architecture because it will utilize the current bay size and will allow for a relatively flat ceiling.

The first step in the design of this system was a set of hand calculations to determine approximate sizes and the amount of reinforcing that would be necessary. An interior bay of the building with 32" by 32" columns was designed. Referencing ACI 9.5.3.2, for a slab without interior beams and with drop panels, an interior span requires a minimum thickness of 9.6" and an exterior span a thickness of 10½". An 11" slab was chosen to meet both of these requirements. Next it was determined that the direct design method could be used for computing the slab design moments. These moments were calculated and distributed to the column strip and middle strip. It was then possible to design the top and bottom reinforcing for both the column and middle strips, and because the bay is square, this reinforcing was applied in both directions. The slab was checked for shear, and it was determined that punching shear would occur at the columns, so drop panels were necessary. Drop panels were sized at 10½' by 10½' with a trial depth of 3½". Checking this depth, it was determined that a 5½" drop would be required, or the strength of the concrete needed to be increased to 6000 psi. This would play an important part in the later decision to design the building using 6000 psi concrete. A copy of these hand calculations can be found in Appendix E.

In order to check the hand calculations of the required reinforcing the computer program spSlab was utilized and two models were constructed. The first modeled a series of interior bays and the second a series of exterior bays. Plan views of the interior bay and exterior bay models can be seen in Figures F and G, respectively.

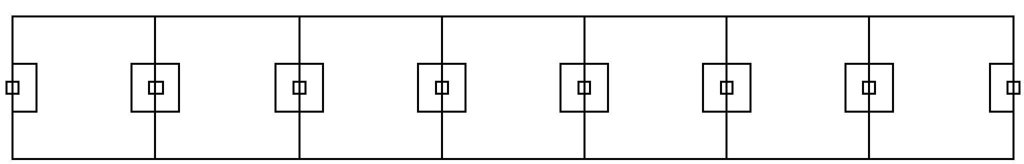

Figure F – Interior Bay Model Plan View

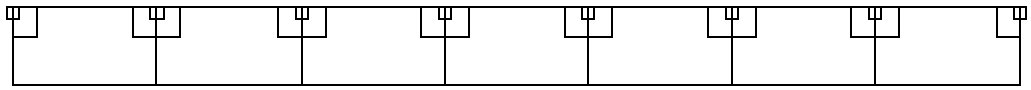

Figure G – Exterior Bay Model Plan View

Each model consisted of seven 31'-5" long spans with a slab thickness of 11". The interior model included a left width of 15'-9" and right width of 15'-9", and the exterior model a left width of 1'-4" and right width of 15'-9". Columns were assumed to be 32" by 32", with a height of 18' above and below. Initially, the drop panels were modeled with a thickness of 3½" and a concrete strength of 4000 psi. With a superimposed dead load of 25 psf and a live load of 125 psf, this design encountered a problem with punching shear at the first interior columns on either side. In order to correct this problem several alternatives were explored and multiple iterations were performed. The first consisted of changing the depth of all of the drop panels to 5½". While this was effective, it seemed to be an inefficient use of concrete for 64 drop panels on 10 levels. The second alternative involved increasing the depth of the first interior drops to 7½", the next adequate size. This seemed more efficient, but unlikely due to the changes in formwork that would be required during construction. It was also questioned whether each of the first two options would actually be more cost effective than using 3½" drops and changing the concrete strength to 6000 psi. A quick cost analysis was conducted, and it was determined that using 3½" drops with 6000 psi concrete would in fact have a lower cost than either of the other two options. Table 5 shows a summary of this analysis.

Option	Drop Depth (in)	f'_c	Area (ft^2)	Number of Drops	Volume (CY)	Cost/CY	Cost ($)
1	5.5	4000	110.25	8	14.97	106	1587.06
2	3.5	4000	110.25	6	7.15	106	1298.50
	7.5	4000	110.25	2	5.10		
3	3.5	6000	110.25	8	9.53	124	1181.44

Table 5 – Drop Panel Depth Cost Analysis

In the end the decision was made to use 3½" drop panels and change the strength of the concrete to 6000 psi, which resulted in a successful solution to all instances of punching shear. This decision would also play a role in the selection of 6000 psi concrete for the construction of the entire building. With the drop panel depth and concrete strength selected, spSlab was used to redesign the slab reinforcement. A summary of the selected reinforcement for the interior bay and exterior bay can be found in Figures H, I, J, and K.

Final Report

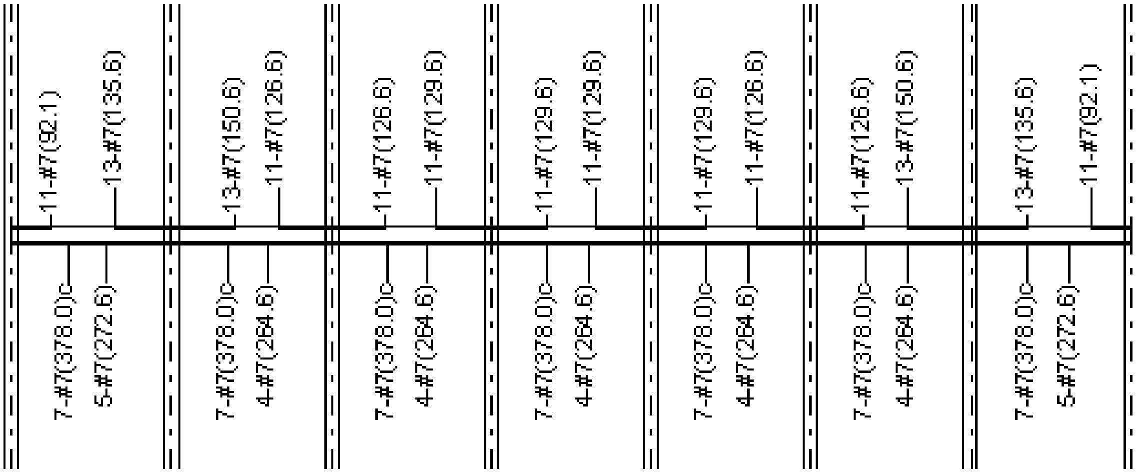

Figure H – Interior Bay Middle Strip Flexural Reinforcement

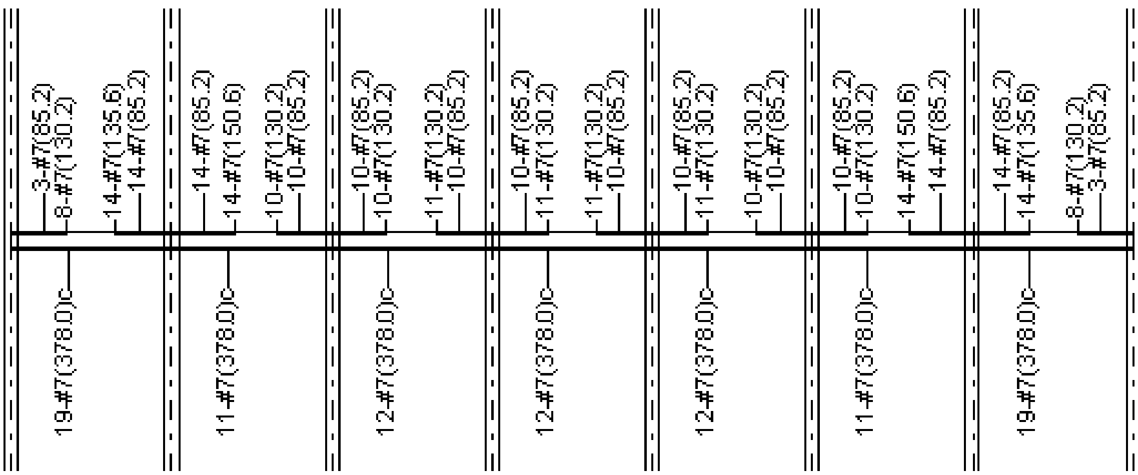

Figure I – Interior Bay Column Strip Flexural Reinforcement

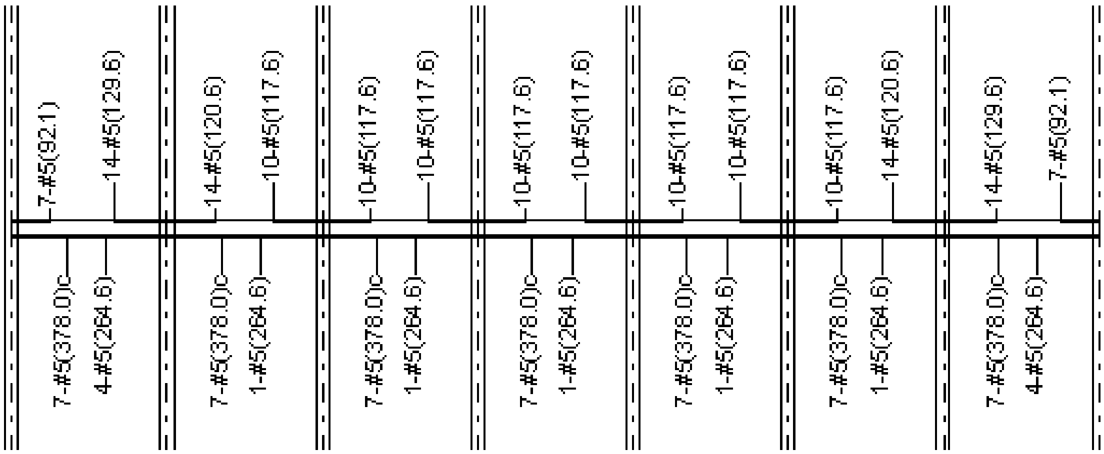

Figure J – Exterior Bay Middle Strip Flexural Reinforcing

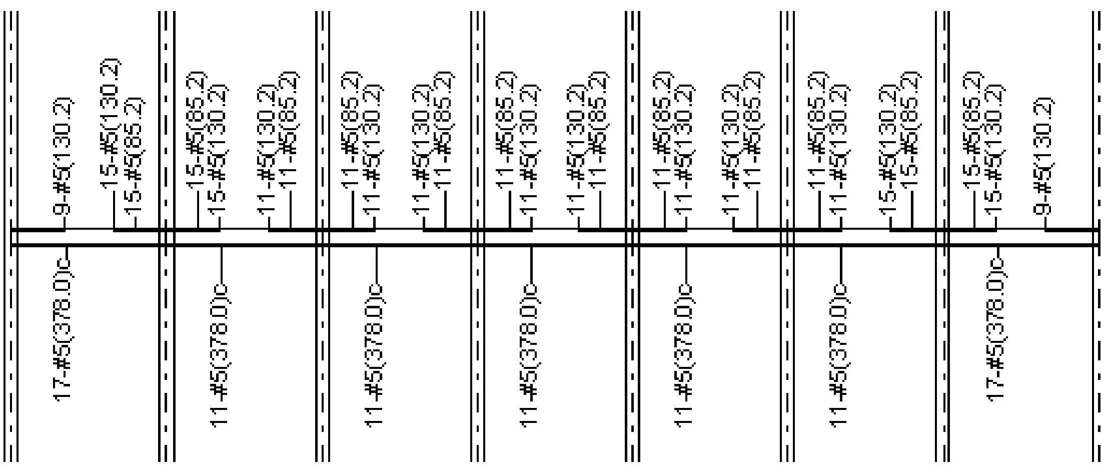

Figure K – Exterior Bay Column Strip Flexural Reinforcing

The final design consideration for the flat slab with drop panels system was the deflection of the slab. For the interior bay, the maximum total load deflection was 0.623 in, and the maximum live load deflection was 0.419 in. Both of these values were well within their respective limits of L/240 and L/360. The deflections of the exterior bay also met this limits, with a maximum total load deflection of 0.435 in and maximum live load deflection of 0.266 in.

Column Design

The second part of the gravity system redesign involved the design of the concrete columns. This was done using a combination of hand calculations, RAM Structural System, and spColumn. The first step was to approximate a reasonable column size in RAM Structural System. Using the slab thickness and drop panel depth determined in the slab design as well as a concrete strength of 6000 psi, a model of the building was built and loaded with the superimposed dead load of 25 psf and live load of 125 psf. A gravity analysis was then performed to find the gravity loads that would be placed on the columns. Next, iterations of column design were completed with various sizes of square columns ranging from 18” by 18” to 40” by 40”. From this model it was concluded that unbraced length would control the design of columns on levels with a story height of 18’, and would require a minimum column size of 24” by 24”. It was also determined that column sizes throughout the building would range from 20” by 20” on the roof level to 36” by 36” at the base. Due to the large gravity loads of the build, and the large size of the columns on levels with a story height of 18’, the columns at the base are forced to carry a high axial load and are therefore quite large. A three dimensional view of the RAM Structural System model can be seen in Figure L.

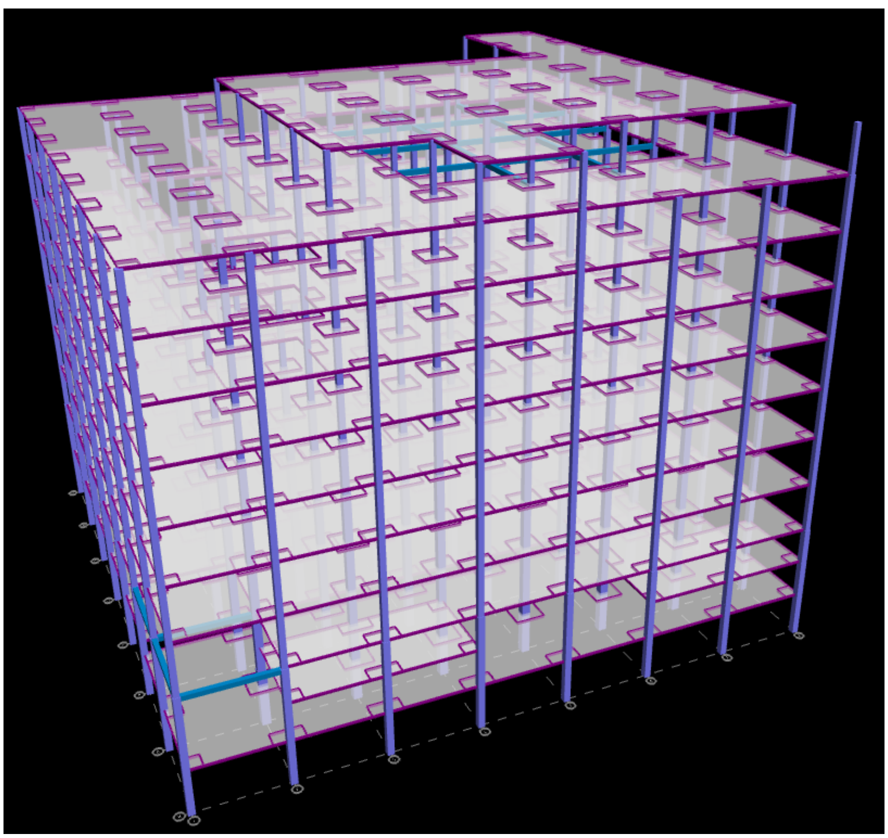

Figure L – Three Dimensional View of RAM Structural System Model

After using RAM Structural System to determine a base size for the columns, the design proceeded with calculations by hand and in spColumn. The first step was to create a Microsoft Excel sheet summing the total axial load by tributary area on a column at any given level. To simplify the design procedure, three different column locations were considered: a column at the corner of the building, a column along the exterior edge of the building, and a column at the interior of the building. Moments were also taken from spSlab for each of these column locations and included in the tables. These values are shown below in Tables 6, 7, and 8.

Corner Column												
Level	Height (ft)	Size (in)	A_T (ft^2)	Slab (in)	w_c (pcf)	SDL (psf)	Live (psf)	Factored (lb)	Slab Weight (lb)	SW (lb)	P_u (k)	M_u
Roof	16	32	248.1	11	150	25	20	15380	34109	17067	66.6	131
9	18	32	248.1	11	150	25	125	57054	34109	19200	176.9	131
8	18	32	248.1	11	150	25	125	57054	34109	19200	287.3	131
7	18	32	248.1	11	150	25	125	57054	34109	19200	397.6	131
6	18	32	248.1	11	150	25	125	57054	34109	19200	508.0	131
5	18	32	248.1	11	150	25	125	57054	34109	19200	618.4	131
4	18	32	248.1	11	150	25	125	57054	34109	19200	728.7	131
3	18	32	248.1	11	150	25	125	57054	34109	19200	839.1	131
2	13	32	248.1	11	150	25	125	57054	34109	13867	944.1	131
1	9	32	0.0	11	150	25	125	0	0	9600	953.7	131
Mech	5	32	248.1	11	150	25	125	57054	34109	5333	1050.2	131
Base	13	32	0.0	11	150	25	125	0	0	13867	1064.1	131
SB	3	32	248.1	11	150	25	125	57054	34109	3200	1158.5	131

Table 6 - Summation of Axial Load on Corner Column

Exterior Column												
Level	Height (ft)	Size (in)	A_T (ft^2)	Slab (in)	w_c (pcf)	SDL (psf)	Live (psf)	Factored (lb)	Slab Weight (lb)	SW (lb)	P_u (k)	M_u
Roof	16	32	496.1	11	150	25	20	30760	68217	17067	116.0	174.5
9	18	32	496.1	11	150	25	125	114109	68217	19200	317.6	174.5
8	18	32	496.1	11	150	25	125	114109	68217	19200	519.1	174.5
7	18	32	496.1	11	150	25	125	114109	68217	19200	720.6	174.5
6	18	32	496.1	11	150	25	125	114109	68217	19200	922.1	174.5
5	18	32	496.1	11	150	25	125	114109	68217	19200	1123.7	174.5
4	18	32	496.1	11	150	25	125	114109	68217	19200	1325.2	174.5
3	18	32	496.1	11	150	25	125	114109	68217	19200	1526.7	174.5
2	13	32	496.1	11	150	25	125	114109	68217	13867	1722.9	174.5
1	9	32	0.0	11	150	25	125	0	0	9600	1732.5	174.5
Mech	5	32	496.1	11	150	25	125	114109	68217	5333	1920.2	174.5
Base	13	32	0.0	11	150	25	125	0	0	13867	1934.0	174.5
SB	3	32	496.1	11	150	25	125	114109	68217	3200	2119.6	174.5

Table 7 - Summation of Axial Load on Exterior Column

Interior Column												
Level	Height (ft)	Size (in)	A_T (ft^2)	Slab (in)	w_c (pcf)	SDL (psf)	Live (psf)	Factored (lb)	Slab Weight (lb)	SW (lb)	P_u (k)	M_u
Roof	16	32	992.3	11	150	25	20	61520	136434	17067	215.0	117.5
9	18	32	992.3	11	150	25	125	228218	136434	19200	598.9	117.5
8	18	32	992.3	11	150	25	125	228218	136434	19200	982.7	117.5
7	18	32	992.3	11	150	25	125	228218	136434	19200	1366.6	117.5
6	18	32	992.3	11	150	25	125	228218	136434	19200	1750.4	117.5
5	18	32	992.3	11	150	25	125	228218	136434	19200	2134.3	117.5
4	18	32	992.3	11	150	25	125	228218	136434	19200	2518.1	117.5
3	18	32	992.3	11	150	25	125	228218	136434	19200	2902.0	117.5
2	13	32	992.3	11	150	25	125	228218	136434	13867	3280.5	117.5
1	9	32	0.0	11	150	25	125	0	0	9600	3290.1	117.5
Mech	5	32	992.3	11	150	25	125	228218	136434	5333	3660.1	117.5
Base	13	32	0.0	11	150	25	125	0	0	13867	3674.0	117.5
SB	3	32	992.3	11	150	25	125	228218	136434	3200	4041.8	117.5

Table 8 - Summation of Axial Load on Interior Column

When the summation of axial forces was complete, a sample column was designed by hand. The first step of this design was to check if the 36” by 36” column was part of a sway or non-sway frame. With reference to ACI 10.10.5.2, Q was calculated to be less than 0.05, meaning that the column is part of a non-sway frame. Next, a slenderness check showed the column to be slightly above the slenderness limit, requiring that the moment magnification factor be calculated. This was done, and the moment magnification factor was determined to be less than one, meaning that moment magnification does not influence the column behavior, and the predetermined moment could be used for the column design. The design of the 36” by 36” subbasement column continued using an axial load of 4042 kips and a moment of 118 foot-kips. Considering a concrete strength of 6000 psi and a steel strength of 60000 psi, and using the column design tables in the back of Macgregor’s ‘Reinforced Concrete: Mechanics and Design’ textbook, the required area of steel was calculated to be 28.5 in^2. Utilizing 24 number 10 bars in a square pattern would satisfy this requirement with an area of steel equal to 29.2 in^2. To view these hand calculations refer to Appendix E.

In order to check the hand design of the subbasement level 36” by 36” column, it was modeled in spColumn. Again a concrete strength of 6000 psi and steel strength of 60000 psi were used, and reinforcement was assumed to be equal on all sides. The axial load of 4042 kips and moment of 118 foot-kips were applied to the model, and the solution was executed. Slightly differing from the hand design, spColumn suggested that 20 number 10 bars be used with an area of steel of 25.4 in^2. This is adequately close, and the small difference could be attributed to the accuracy of reading the rho value off the Macgregor chart. For the column design summary, as well as the design interaction diagram, refer to Appendix E.

After designing one of the columns by hand and then checking it using the computer, it was concluded that the column design should continue using spColumn. A column on each level was designed for all three locations using the corresponding axial load and moment that were previously calculated and are shown in Tables 9, 10, and 11. The area of steel, number of bars, and bar size was recorded for each column and placed in the following tables. It should be noted that the corner and exterior column designs have a much smaller size and require a smaller area of steel compared to interior columns on the same level. This makes sense, and is intuitive, because the interior columns obviously have a larger tributary area and therefore carry much greater axial loads.

Corner Column Design			
Level	**Size (in x in)**	**A_s (in^2)**	**Long. Reinforcing**
Roof	20 x 20	6.32	8 #8
9	24 x 24	6.32	8 #8
8	24 x 24	6.32	8 #8
7	24 x 24	6.32	8 #8
6	24 x 24	6.32	8 #8
5	24 x 24	6.32	8 #8
4	24 x 24	6.32	8 #8
3	24 x 24	6.32	8 #8
2	28 x 28	8.00	8 #9
1	28 x 28	8.00	8 #9
Mech	28 x 28	8.00	8 #9
Base	28 x 28	8.00	8 #9
SB	28 x 28	8.00	8 #9

Table 9 - Corner Column Size and Reinforcing Design

Exterior Column Design			
Level	**Size (in x in)**	**A_s (in^2)**	**Long. Reinforcing**
Roof	20 x 20	6.32	8 #8
9	24 x 24	6.32	8 #8
8	24 x 24	6.32	8 #8
7	24 x 24	6.32	8 #8
6	24 x 24	6.32	8 #8
5	24 x 24	6.32	8 #8
4	24 x 24	6.32	8 #8
3	24 x 24	6.32	8 #8
2	28 x 28	8.00	8 #9
1	28 x 28	8.00	8 #9
Mech	28 x 28	8.00	8 #9
Base	28 x 28	8.00	8 #9
SB	28 x 28	8.00	8 #9

Table 10 - Exterior Column Size and Reinforcing Design

Interior Column Design			
Level	**Size (in x in)**	**A_s (in^2)**	**Long. Reinforcing**
Roof	20 x 20	6.32	8 #8
9	24 x 24	6.32	8 #8
8	24 x 24	6.32	8 #8
7	24 x 24	6.32	8 #8
6	24 x 24	8.00	8 #9
5	28 x 28	8.00	8 #9
4	32 x 32	12.00	12 #9
3	32 x 32	12.00	12 #9
2	32 x 32	20.32	16 #10
1	32 x 32	20.32	16 #10
Mech	36 x 36	15.24	12 #10
Base	36 x 36	15.24	12 #10
SB	36 x 36	25.40	20 #10

Table 11 - Interior Column Size and Reinforcing Design

Lateral System Redesign

Shear Wall Design

After the completion of the gravity system design the lateral system was redesigned in order to replace the existing steel braced frames with reinforced concrete shear walls. The first step in this process was the construction of a model in the computer program ETABS. A three-dimensional view of this model is shown in Figure M.

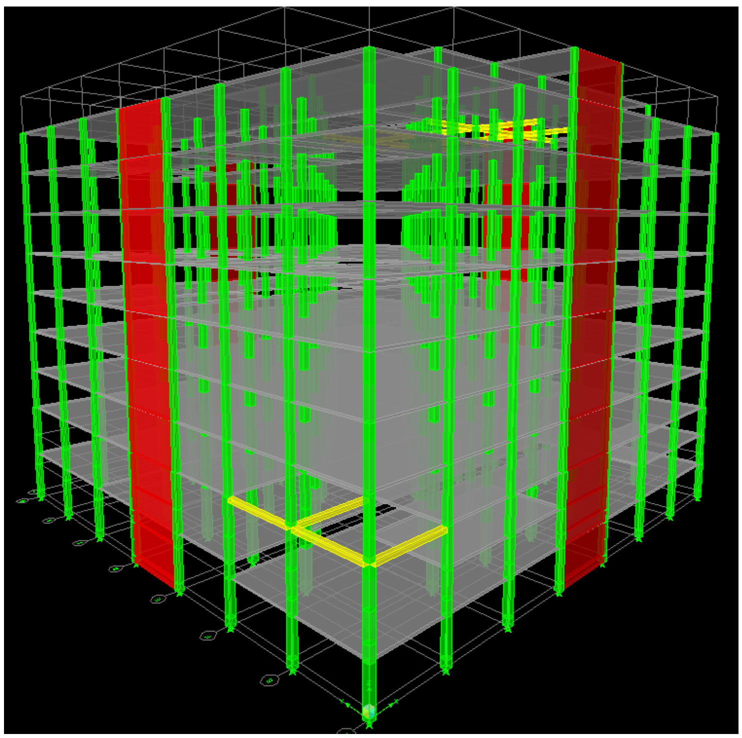

Figure M – Three Dimensional ETABS Model

Columns were assigned based on the sizes determined during the gravity system redesign, with 6000 psi strength concrete. They were assumed to be braced by the slabs, had a 0.7 modification factor on the moment of inertia in both directions, and were pinned at the bases. The slabs were originally modeled as rigid diaphragms, but an area load transfer error resulted in changing them to 11” plate elements. Finally, four shear walls were placed in the model at an assumed thickness of 16”. The location of the existing braced frames was considered when determining the placement of the shear walls. They were modeled on the perimeter of the building and are shown in red in Figure N.

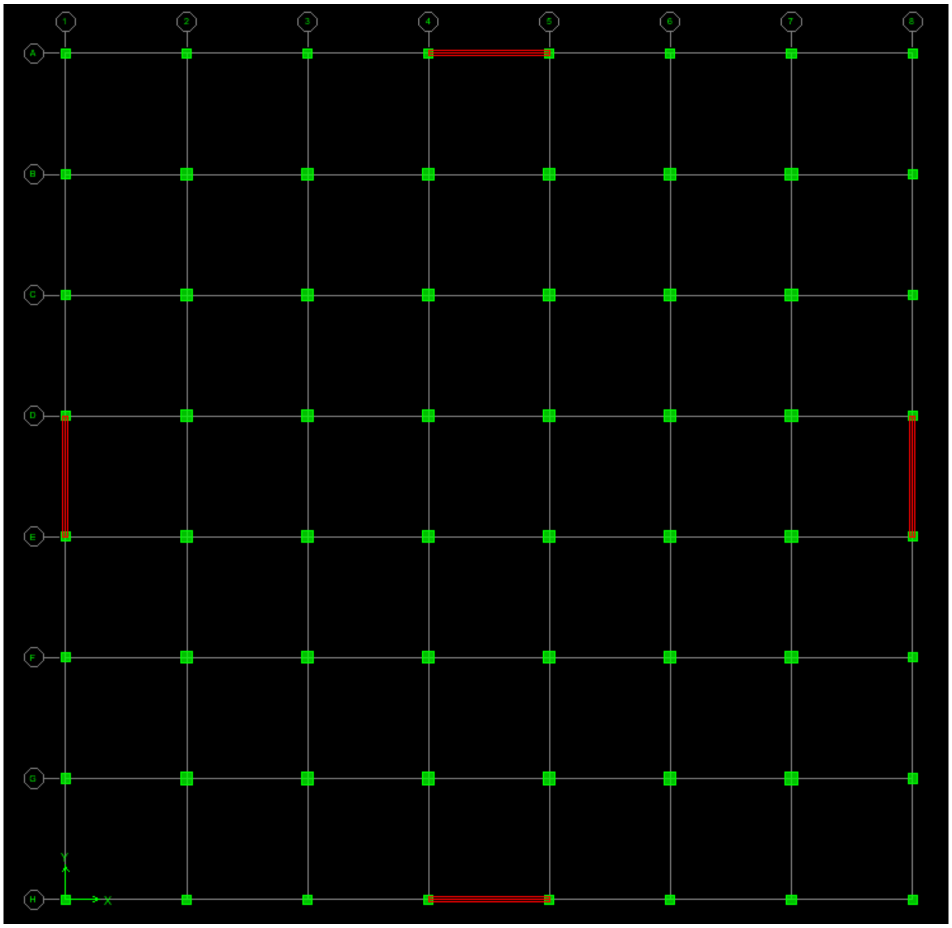

Figure N – Plan View Showing Shear Wall Location

The assumption made when designing the shear walls was that the square layout of the building would cause each wall to take fifty percent of the shear in that direction. While this would be ideal, it may not be entirely true. Due to the geometry of some of the floors, there may be a small amount of inherent torsion. Also, one wall in each direction is slightly taller than the other. Therefore the shorter wall will be inherently stiffer and may take more of the load.

To design the shear walls it had to be determined what the greatest shear was that any of the walls could experience. In order to do this the load combinations that were discussed in the load combination section were again considered, as well as an extra combination as per ASCE 7-10 section 12.5.3a. This section of ASCE 7-10 requires that the building design consider 100 percent of the earthquake loading in one direction, and 30 percent of the earthquake loading in the other direction. Therefore 14 combinations were modeled in ETABS. Section cuts were drawn through the base of each shear wall to determine the shear that wall experienced under each load case. These shears are summarized in Table 12.

Load Combo	Shear in Wall (k)			
	A	H	1	8
1	3.3	4.3	1.5	0.9
2	2.9	2.7	5.0	3.8
3	2.1	1.7	3.7	2.6
4	371.7	399.5	7.8	1.3
5	52.8	56.8	395.4	305.6
6	727.9	786.1	7.5	1.6
7	106.1	107.8	804.9	623.8
8	647.7	675.6	7.6	0.1
9	93.2	94.4	711.4	575.6
10	739.9	784.2	10.1	0.5
11	104.5	108.8	799.1	622.6
12	647.1	673.7	2.2	1.8
13	92.2	92.5	708.8	583.3
QUAKE	628.5	718.6	206.6	182.2

Table 12 – Determination of Controlling Shear

As it can be seen from Table 12, even when considering the special seismic load combination, the controlling combination is still number seven, with a shear at the base of Wall 1 of 804.9 kips. It was assumed that this was the controlling shear that any of the walls would see, and therefore all four walls were designed for this value. Using a 16" thickness, a length of 31'-6", a height of 16', and a self-weight of the walls above of 1090 kips, a shear wall was designed by hand to resist 800 kips. The resulting design required two number four bars at 10" for horizontal reinforcement, two number four bars at 10" for vertical reinforcement, and ten number nine bars at 2" for flexural reinforcement. For the full hand design refer to Appendix F.

Drift Analysis

Story drift and total drift were determined for the controlling seismic loading and wind loading. Checking seismic drift is necessary from a strength standpoint, in order to prevent building damage or failure. Wind drift is a serviceability issue, and addressing it is necessary to prevent sway that would cause discomfort to building occupants, as well as damage to curtain walls and other façade components.

For seismic loading, drift values were obtained from the ETABS model and were then compared to the allowable story drift of $0.010h_{sx}$ as per Table 12.12-1 in ASCE 7-10. The wind load drifts were also acquired from ETABS, and were evaluated against the limit of H/400.

As it can be seen from the following tables, all story drift and total drift values were within the allowable limits.

Controlling Seismic Drift: East-West				
Level	**Height (ft)**	**Story Drift (in)**	**Allowable Story Drift (in)**	
Roof	185	0.155770	0.16	Acceptable
9	169	0.143312	0.18	Acceptable
8	151	0.132276	0.18	Acceptable
7	133	0.117439	0.18	Acceptable
6	115	0.100395	0.18	Acceptable
5	97	0.079443	0.18	Acceptable
4	79	0.058065	0.18	Acceptable
3	61	0.038308	0.18	Acceptable
2	43	0.020726	0.13	Acceptable
1	30	0.012000	0.09	Acceptable
Mechanical	21	0.007287	0.05	Acceptable
Basement	16	0.003456	0.16	Acceptable
SB	3	0.000597	0.03	Acceptable

Table 13 – East-West Direction Controlling Seismic Drift

Controlling Seismic Drift: North-South				
Level	Height (ft)	Story Drift (in)	Allowable Story Drift (in)	
Roof	185	0.159470	0.16	Acceptable
9	169	0.146185	0.18	Acceptable
8	151	0.134994	0.18	Acceptable
7	133	0.119833	0.18	Acceptable
6	115	0.102465	0.18	Acceptable
5	97	0.081092	0.18	Acceptable
4	79	0.059408	0.18	Acceptable
3	61	0.038979	0.18	Acceptable
2	43	0.018619	0.13	Acceptable
1	30	0.010830	0.09	Acceptable
Mechanical	21	0.005943	0.05	Acceptable
Basement	16	0.005184	0.16	Acceptable
SB	3	0.001026	0.03	Acceptable

Table 14 – North-South Direction Controlling Seismic Drift

Wind Deflection: East-West				
Level	Height (ft)	Total Deflection(in)	Allowable Total Deflection (in)	
Roof	185	1.280716	5.55	Acceptable
9	169	1.157314	5.07	Acceptable
8	151	1.015849	4.53	Acceptable
7	133	0.868758	3.99	Acceptable
6	115	0.718702	3.45	Acceptable
5	97	0.567508	2.91	Acceptable
4	79	0.422255	2.37	Acceptable
3	61	0.288135	1.83	Acceptable
2	43	0.169761	1.29	Acceptable
1	30	0.101421	0.90	Acceptable
Mechanical	21	0.061281	0.63	Acceptable
Basement	16	0.041924	0.48	Acceptable
SB	3	0.007381	0.09	Acceptable

Table 15 – East-West Direction Wind Drift

Wind Deflection: North-South				
Level	**Height (ft)**	**Total Deflection (in)**	**Allowable Total Deflection (in)**	
Roof	185	1.215410	5.55	Acceptable
9	169	1.099820	5.07	Acceptable
8	151	0.968110	4.53	Acceptable
7	133	0.830436	3.99	Acceptable
6	115	0.689243	3.45	Acceptable
5	97	0.546207	2.91	Acceptable
4	79	0.408127	2.37	Acceptable
3	61	0.279683	1.83	Acceptable
2	43	0.166710	1.29	Acceptable
1	30	0.114844	0.90	Acceptable
Mechanical	21	0.083211	0.63	Acceptable
Basement	16	0.067298	0.48	Acceptable
SB	3	0.013594	0.09	Acceptable

Table 16 – North-South Direction Wind Drift

Relative Stiffness Check

The relative stiffness of each shear wall was calculated for both the North-South and East-West directions, and is shown in the tables below. Finding the relative stiffness of each shear wall provided a reasonable method of checking the distribution of the lateral load throughout the building. It was done by placing a 100 kip load at the top of each individual frame, and then measuring the lateral displacement in inches. The formula for stiffness is:

$$k_i = \frac{P}{\delta}$$

where k_i is the stiffness, P is the force, or 100 kips, and δ is the lateral displacement. After the stiffness for each frame was found, they were summed, and used to find the relative stiffness with the equation:

$$Relative\ Stiffness = \frac{k_i}{\sum k_i}$$

Refer to Tables 17 and 18 for the relative stiffness of each shear wall.

East-West Direction Relative Stiffness					
Frame	Height (ft)	Load (k)	Displacement (in)	Stiffness (k/in)	Relative Stiffness
A	169	100	0.6659	150.1695	0.5666
H	185	100	0.8707	114.8510	0.4334
			Σ =	265.0206	1.0000

Table 17 – East-West Relative Stiffness

North-South Direction Relative Stiffness					
Frame	Height (ft)	Load (k)	Displacement (in)	Stiffness (k/in)	Relative Stiffness
1	169	100	0.6659	150.1695	0.5666
8	185	100	0.8707	114.8510	0.4334
			Σ =	265.0206	1.0000

Table 18 – North-South Relative Stiffness

The assumption made when designing the shear walls was that the square layout of the building would cause each wall to take fifty percent of the shear in that direction. While this would be ideal, it is not entirely true. As it can be seen from the Tables above, the shorter walls in each direction are slightly stiffer than the larger walls, and therefore take more shear. The difference between the two is very small, and can be considered negligible. Also, the ETABS model would have accounted for this slight difference, and the controlling wall shear that was used to design the shear walls would still be valid.

Overturning and Impact on Foundation

Overturning moments are a result of wind and seismic loading, and cause the building to try and 'topple over'. This 'toppling' produces uplift in the foundation, and the foundation must be able to resist this uplift. The foundation of GHVI consists of steel helical piles with an allowable axial capacity of 342 kips. These piles are driven to refusal at about a depth of 82 to 87 feet.

In order to check the foundation of this building against uplift the controlling load combination was placed on the ETABS model. From the model the reactions at the base of the structure were found, and negative reactions were deemed significant. A negative reaction on the base means that there is a positive uplift force on the foundation. The location of each uplift occurrence was determined, and the foundation plan was referenced to determine the type of pile cap and the number of piles at this region. The axial load was calculated for this part of the foundation, and it was then compared to the uplift force. Only two points were found to have negative support reactions, and the foundation was deemed to be adequate at each location for uplift. Refer to Table 19 for the uplift locations, forces, and corresponding axial capacities.

Level	Point	FZ	Pile Cap	Axial Capacity (k)
Base	590	-480	PC4	1368
Base	593	-412	PC4	1368

Table 19 – Uplift Reactions and Corresponding Axial Capacity

Although uplift on the current foundation would not be considered an issue, the large increase in building weight would have an impact on the design. Changing the building from steel to concrete increased the weight of the building by 33604 kips, a 64 percent increase. This added load would be placed on the steel piles, and a redesign of the foundation may be necessary. Although not a part of the proposal for this thesis, this redesign of the foundation would result in an increase in overall building cost that must be considered as part of the construction management breadth.

Vibration Analysis

After the completion of the gravity system and lateral system designs, the structure was analyzed for vibration to determine if the vibrational velocities of the existing design were achieved with the redesigned structure. Currently, the building is designed to meet four different criteria based on a moderate walking pace of 75 steps/minute. Typical lab and surgery areas throughout the building, utilizing bench microscopes up to 100x magnification, are required to meet a velocity of 4,000 µ in/sec. Laboratory areas near corridors must meet a velocity of 2,000 µ in/sec, for bench microscopes up to 400x magnification. Central lab areas are designed to house 3-micron photography equipment and sensitive systems with a maximum vibrational velocity of 1,000 µ in/sec. Finally, extremely sensitive areas call for a vibrational velocity that will not exceed 500 µ in/sec.

In order to complete this analysis, research was conducted to determine how to analyze a concrete structure for vibration. A majority of information for this study was found from a technical note published by Bijan O Aalami from ADAPT, titled 'Vibration Design of Concrete Floors for Serviceability', and from AISC and CISC's Design Guide 11, 'Floor Vibrations due to Human Activity'.

The next step was to build a three dimensional SAP2000 model, consisting of a three-bay-by-three-bay area of the building. This was done so that both an interior bay and an exterior bay could be studied. The concrete slab area was modeled as a shell element, with a thickness of 11", and the drop panels were each modeled as shell elements with thicknesses of 14½". The drop panels were then offset downward 1¾" so that they were even with the top of the slab. Columns were modeled halfway to the next level above and below the slab, and were pinned at the ends, assuming zero moment at this point. In order to assure that the slab and drop panels would mesh properly, they were separately discretized into 9" by 9" squares. This guaranteed that the edges of the columns, slab, and drop panels all lined up properly and would mesh together when the model was run. A view of the model used in the vibration analysis can be seen below in Figure O.

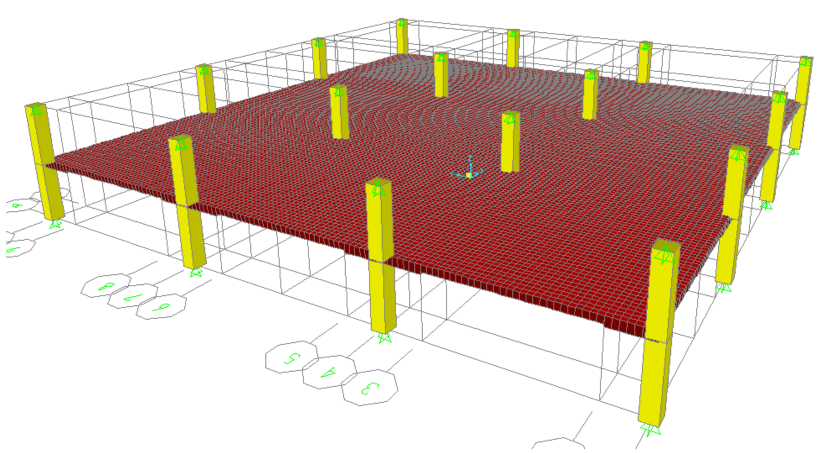

Figure O – SAP2000 Model used for Vibration Analysis

Several assumptions were made with regards to the strength and cracked section properties of the concrete members. The value of elastic modulus, E_c, for the columns, slab, and drops was multiplied by 1.2 to account for dynamic loading. Also, the cracked section properties from ACI 10.10.4.1 were taken into account, including $0.7I_g$ for the columns, and $0.25I_g$ for the slab and drops.

The next step in determining the vibrational velocities of the slab was to set up two static load cases. The first load case placed a 1 kip load at the center of the interior bay, and the second placed this 1 kip load at the center of the exterior bay. The model was run, and from each of the load cases the respective point load deflection and excited mode was found. Table 20 shows a summary of each bay and its point load deflection, fundamental period, and natural frequency. Figures P and Q show the excited modal shapes for the exterior and interior bays, respectively.

Bay	Mode	Δ_p (in)	T (s)	f_n (Hz)
Exterior	7	0.00472	0.15133	6.60793
Interior	11	0.00420	0.12631	7.91720

Table 20 – Results of Vibration Analysis Model

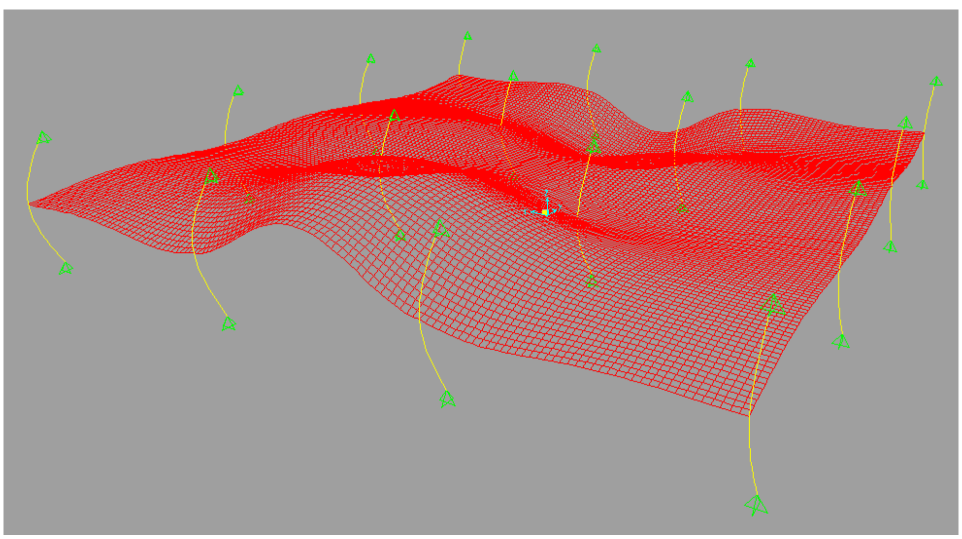

Figure P – Mode 7 Shape for Exterior Bay

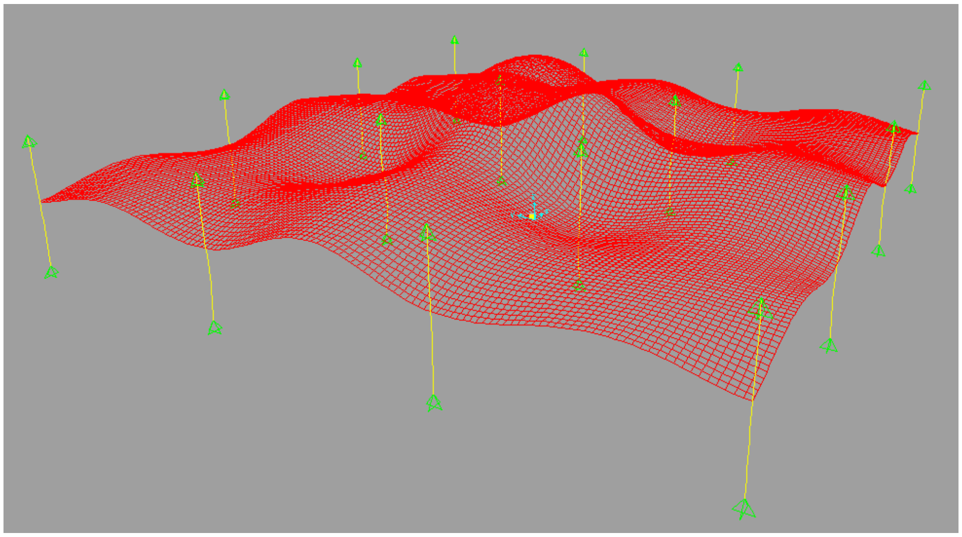

Figure Q – Mode 11 Shape for Interior Bay

Using the results in Table 20, the vibrational velocity of the floor was calculated for both and exterior and interior bay. For a floor with a natural frequency greater than 5 Hz, equation 6.4b from Design Guide 11 can be used:

$$V = \frac{U_v \Delta_p}{f_n}$$

The vibrational velocity was calculated to be 3929 μ in/sec for the exterior bay and 2918 μ in/sec for the interior bay. The hand calculations for the determination of these numbers can be found in Appendix G. It can be seen that these velocities do in fact meet the minimum required vibrational velocity of the current design, but do meet any of the other three requirements. Improvements would need to be made to the redesigned concrete structure to ensure that it met the same criteria as its counterpart. Potential improvements include increasing the concrete strength, increasing the slab thickness, or decreasing the span. Each of these changes could have serious consequences, such as an increase in cost or a change in the architecture and layout of the building.

Construction Management Breadth

Detailed Cost Analysis

In Technical Report 2 it was concluded that a change from a steel superstructure to a concrete structure utilizing a flat slab with drop panels could lower the cost of the building. To determine if this was in fact true, a detailed cost analysis was conducted. In order to have a relevant baseline cost with which to compare the redesigned concrete structure, a cost for the existing steel structure had to be determined. A steel takeoff was obtained from Cannon Design, and using RSMeans Building Construction Cost Data a cost of about $11.9 million was calculated. Table 21 shows a simple breakdown of costs, including the gravity beams and columns, the lateral system framing members, and the composite metal deck. Note that the cost of fireproofing the steel members is also included. Refer to Appendix H for more detailed examples of the steel system takeoff.

Category	Description	Cost ($)
Gravity Beam	Grade: 50	5771092.02
	Grade: Other	29273.21
	Fireproofing	499254.65
Gravity Column	I Section	1313676.83
	Fireproofing	69863.42
Structure Frame	Columns	823162.57
	Beams	349404.53
	Braces	129188.26
	Fireproofing	84842.30
Composite Deck	Metal Decking	1231183.80
	Shear Studs	89844.99
	Concrete Fill	993242.25
	Placing Concrete	197149.05
	Finishing Concrete	318710.70
	Total Cost ($)	11899888.58

Table 21 –Steel Structure Construction Cost

After the completion of the gravity and lateral system designs, the cost of the redesigned structure was tabulated, again using RSMeans Building Construction Cost Data. This guaranteed a level comparison between the steel and concrete designs. The simplified breakdown of costs for the concrete system is shown in Table 22. This breakdown includes the cost of the 6000 psi concrete that was used in the columns, slabs, and shear walls. The total tonnage of reinforcing for the columns was determined from RAM Structural System, and was a conservative estimate. For the slabs a more accurate weight of reinforcing was taken from spSlab, and the amount of reinforcing in the shear walls was calculated by hand. Formwork was conservatively assumed to be used only once, and it was expected that placing the concrete would be done by pump. Refer to Appendix I for more detailed examples of the concrete system takeoff.

Category	Description	Cost ($)
Columns	6000 psi Concrete	292821.23
	Reinforcing Steel	736412.60
	Formwork	1133271.98
	Placing Concrete	60460.03
Slabs and Drops	6000 psi Concrete	1994706.97
	Reinforcing Steel	1816459.43
	Formwork	4076464.63
	Placing Concrete	261323.82
	Finishing Concrete	319072.69
Shear Walls	6000 psi Concrete	146862.80
	Reinforcing Steel	115049.84
	Formwork	575849.91
	Placing Concrete	26266.80
	Total Cost ($)	11555022.74

Table 22 – Concrete Structure Construction Cost

As Table 22 shows, the estimated cost for the concrete structure is $11.6 million, which represents a 2.9% cost savings from the steel system. While this looks good, it is probably unrealistic for a few reasons. The steel estimate included a more detailed takeoff of the steel in the building due to the fact that it was taken from Cannon Design's structural model. The concrete structure that was modeled for this thesis was much simpler, and added cost must be considered for a more realistic building. Another concern to take into account would be the increase of the building weight associated with changing to a concrete structure. This increase would result in a redesign of the building's foundation, and an increase in cost. Another possible increase in cost would be due to the fact that concrete buildings are not common in Buffalo, NY. Buffalo experiences very long and cold winters, which makes designing and constructing a concrete building extremely undesirable. After discussion with a Cannon Design structural engineer, it was discovered that a concrete alternative was not even examined in the schematic design of GHVI. The increased cost due to admixtures and winter protection has historically been so high that for most projects in Buffalo a steel superstructure is more cost effective and efficient. Finally, the vibration analysis proved that the newly proposed structure did not meet the existing requirements for vibrational velocities. The increase in floor stiffness that would be required to meet these stringent velocities would require much added cost in the form of concrete volume or strength, or even architectural redesign. It can be argued that while concrete could be a viable alternative on this project, too many factors would increase the cost and make steel the more ideal solution.

Schedule Analysis

The second part of the construction management breadth was a schedule analysis between the existing building and the redesigned concrete structure. The goal of this analysis was to determine which system was more efficient and would take less time to complete. It was assumed for both schedules that no other construction trades would be working during this time period. This is unrealistic, but gives a pure comparison between the two options. To begin this analysis, Cannon Design was contacted and a schedule of the current steel construction was process was requested. Although the actual project schedule could not be obtained, an early schematic design schedule was provided. This schedule was input into Microsoft Project, and is shown in Figure R below. As it can be seen, the construction of the steel structure was schedule to span from Monday, January 1, 2010 to Tuesday, October 5, 2010, for a total of 192 days, or about nine months.

ID	Task Name	Duration	Start	Finish
1	**Steel Superstructure**	**192 days**	**Mon 1/11/10**	**Tue 10/5/10**
2	Mobilize Crane for Steel Erection	2 days	Mon 1/11/10	Tue 1/12/10
3	Structural Steel start	105 days	Wed 1/13/10	Tue 6/8/10
4	Steel Decking Basement/Trim out-tack & studs	20 days	Wed 1/27/10	Tue 2/23/10
5	Steel Decking	110 days	Wed 1/27/10	Tue 6/29/10
6	Layout/MEP sleeves, Pour stops Basement	12 days	Wed 3/10/10	Thu 3/25/10
7	Layout/Set MEP sleeves, Pour stops	108 days	Wed 3/10/10	Fri 8/6/10
8	Steel Decking 1 Level/Trim out-tack & stud	20 days	Wed 2/10/10	Tue 3/9/10
9	Layout/MEP sleeves, Pour stops, 1st Levl	12 days	Mon 3/22/10	Tue 4/6/10
10	Steel Decking 2nd Level/Trim out-tack & studs	20 days	Wed 2/24/10	Tue 3/23/10
11	Layout/MEP sleeves, Pour stops 2nd Level	12 days	Thu 4/1/10	Fri 4/16/10
12	Steel Decking 3rd Level/Trim out-tack & studs	20 days	Wed 3/10/10	Tue 4/6/10
13	Layout/MEP sleeves, Pour stops 3rd Level	12 days	Tue 4/13/10	Wed 4/28/10
14	Steel Decking 4th Level/Trim out-tack & studs	20 days	Wed 3/24/10	Tue 4/20/10
15	Layout/MEP sleeves, Pour stops 4th Level	12 days	Fri 4/23/10	Mon 5/10/10
16	Steel Decking 5th Level/Trim out-tack & studs	20 days	Wed 3/24/10	Tue 4/20/10
17	Layout/MEP sleeves, Pour stops 5th Level	12 days	Wed 5/5/10	Thu 5/20/10
18	Steel Decking 6th Level/Trim out-tack & studs	20 days	Wed 4/7/10	Tue 5/4/10
19	Layout/MEP sleeves, Pour stops 6th Level	12 days	Mon 5/17/10	Tue 6/1/10
20	Steel Decking 7th Level/Trim out-tack & studs	20 days	Wed 4/21/10	Tue 5/18/10
21	Layout/MEP sleeves, Pour stops 7th Level	12 days	Wed 6/2/10	Thu 6/17/10
22	Steel Decking 8th Level/Trim out-tack & studs	20 days	Wed 5/5/10	Tue 6/1/10
23	Layout/MEP sleeves, Pour stops 8th Level	12 days	Fri 6/18/10	Mon 7/5/10
24	Steel Decking 9th Level/Trim out-tack & studs	20 days	Wed 5/19/10	Tue 6/15/10
25	Layout/MEP sleeves, Pour stops 9th Level	12 days	Tue 7/6/10	Wed 7/21/10
26	Steel Decking Roof	10 days	Wed 6/2/10	Tue 6/15/10
27	Roof Deck	10 days	Wed 6/30/10	Tue 7/13/10
28	Roofing - Main	30 days	Wed 7/14/10	Tue 8/24/10
29	Layout/MEP sleeves, Pour stops Roof	12 days	Thu 7/22/10	Fri 8/6/10
30	Screenwall	15 days	Wed 8/25/10	Tue 9/14/10
31	Skylights	30 days	Wed 8/25/10	Tue 10/5/10

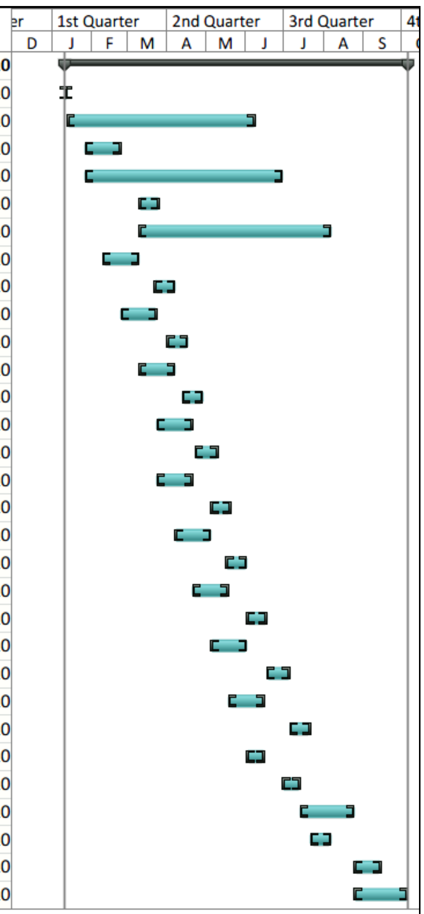

Figure R – Original Steel Construction Schedule (Obtained from Cannon Design)

The next step was to assemble the concrete structure schedule. Microsoft Project was once again used, and several assumptions were made using the daily output numbers from RSMeans Building Construction Cost Data and other concrete projects of comparable size. First, it was assumed that framing the columns and shear walls on each level would take eight days, setting the rebar five days, and pouring the concrete another six days. The placing of the concrete for the columns and shear walls could not begin until the formwork and rebar was set. The framing of the slabs and drop panels would commence three days after the framing of the columns and shear walls, allowing time for these to be properly framed. The framing for the slab and drop panels would take eight days, setting the rebar five days, and pouring the concrete six days. Seven days would then be allocated for the concrete to cure before the next level was started. Overall, it would take about a month for each level to be framed, set, and placed. Assuming this process for each level, it would take 242 days to complete the concrete structure, spanning from Monday, January 11, 2010 to Tuesday, December 14, 2010. A view of the Microsoft Project concrete construction schedule can be found in Figure S on the next page.

As per the schedule analysis, the redesigned concrete structure takes 50 days longer to complete than the steel structure. This is reasonable, as it generally takes longer to construct a concrete building than one made from steel. It is unknown if there were specific time contraints surrounding this project, but assuming there were, this could cause a substantial increase in the project cost. Another factor that was not considered in the concrete schedule analysis was the cold Buffalo, NY winters. As was previously discussed, placing concrete in the middle of winter in Buffalo is not an ideal situation. This process would assuredly suffer difficulties and setbacks that would push the schedule back even further and create an even more significant difference between the steel and concrete schedules.

Final Report

ID	Task Name	Duration	Start	Finish
	Concrete Superstructure	**242 days**	**Mon 1/11/10**	**Tue 12/14/10**
1	Frame Columns and Walls - Basement	8 days	Mon 1/11/10	Wed 1/20/10
2	Set Rebar Columns and Walls - Basement	5 days	Mon 1/18/10	Fri 1/22/10
3	Pour Columns and Walls - Basement	6 days	Sat 1/23/10	Fri 1/29/10
4	Frame Slab and Drops - Basement	8 days	Thu 1/14/10	Mon 1/25/10
5	Set Rebar Slabs and Drops - Basement	5 days	Thu 1/21/10	Wed 1/27/10
6	Pour Slabs and Drops - Basement	6 days	Thu 1/28/10	Thu 2/4/10
7	Frame Columns and Walls - Mechanical	8 days	Fri 2/12/10	Tue 2/23/10
8	Set Rebar Columns and Walls - Mechanical	5 days	Fri 2/19/10	Thu 2/25/10
9	Pour Columns and Walls - Mechanical	6 days	Fri 2/26/10	Fri 3/5/10
10	Frame Slab and Drops - Mechancial	8 days	Mon 2/15/10	Wed 2/24/10
11	Set Rebar Slabs and Drops - Mechanical	5 days	Mon 2/22/10	Fri 2/26/10
12	Pour Slabs and Drops - Mechanical	6 days	Sat 2/27/10	Fri 3/5/10
13	Frame Columns and Walls - Level 1	8 days	Sat 3/13/10	Tue 3/23/10
14	Set Rebar Columns and Walls - Level 1	5 days	Sat 3/20/10	Thu 3/25/10
15	Pour Columns and Walls - Level 1	6 days	Fri 3/26/10	Fri 4/2/10
16	Frame Slab and Drops - Level 1	8 days	Tue 3/16/10	Thu 3/25/10
17	Set Rebar Slabs and Drops - Level 1	5 days	Tue 3/23/10	Mon 3/29/10
18	Pour Slabs and Drops - Level 1	6 days	Tue 3/30/10	Tue 4/6/10
19	Frame Columns and Walls - Level 2	8 days	Wed 4/14/10	Fri 4/23/10
20	Set Rebar Columns and Walls - Level 2	5 days	Wed 4/21/10	Tue 4/27/10
21	Pour Columns and Walls - Level 2	6 days	Wed 4/28/10	Wed 5/5/10
22	Frame Slab and Drops - Level 2	8 days	Sat 4/17/10	Tue 4/27/10
23	Set Rebar Slabs and Drops - Level 2	5 days	Sat 4/24/10	Thu 4/29/10
24	Pour Slabs and Drops - Level 2	6 days	Fri 4/30/10	Fri 5/7/10
25	Frame Columns and Walls - Level 3	8 days	Sat 5/15/10	Tue 5/25/10
26	Set Rebar Columns and Walls - Level 3	5 days	Sat 5/22/10	Thu 5/27/10
27	Pour Columns and Walls - Level 3	6 days	Fri 5/28/10	Fri 6/4/10
28	Frame Slab and Drops - Level 3	8 days	Tue 5/18/10	Thu 5/27/10
29	Set Rebar Slabs and Drops - Level 3	5 days	Tue 5/25/10	Mon 5/31/10
30	Pour Slabs and Drops - Level 3	6 days	Tue 6/1/10	Tue 6/8/10
31	Frame Columns and Walls - Level 4	8 days	Wed 6/16/10	Fri 6/25/10
32	Set Rebar Columns and Walls - Level 4	5 days	Wed 6/23/10	Tue 6/29/10
33	Pour Columns and Walls - Level 4	6 days	Wed 6/30/10	Wed 7/7/10
34	Frame Slab and Drops - Level 4	8 days	Sat 6/19/10	Tue 6/29/10
35	Set Rebar Slabs and Drops - Level 4	5 days	Sun 6/27/10	Thu 7/1/10
36	Pour Slabs and Drops - Level 4	6 days	Fri 7/2/10	Fri 7/9/10
37	Frame Columns and Walls - Level 5	8 days	Sat 7/17/10	Tue 7/27/10
38	Set Rebar Columns and Walls - Level 5	5 days	Sat 7/24/10	Thu 7/29/10
39	Pour Columns and Walls - Level 5	6 days	Fri 7/30/10	Fri 8/6/10
40	Frame Slab and Drops - Level 6	8 days	Tue 7/20/10	Thu 7/29/10
41	Set Rebar Slabs and Drops - Level 6	5 days	Tue 7/27/10	Mon 8/2/10
42	Pour Slabs and Drops - Level 6	6 days	Tue 8/3/10	Tue 8/10/10
43	Frame Columns and Walls - Level 7	8 days	Tue 8/17/10	Thu 8/26/10
44	Set Rebar Columns and Walls - Level 7	5 days	Tue 8/24/10	Mon 8/30/10
45	Pour Columns and Walls - Level 7	6 days	Tue 8/31/10	Tue 9/7/10
46	Frame Slab and Drops - Level 7	8 days	Fri 8/20/10	Tue 8/31/10
47	Set Rebar Slabs and Drops - Level 7	5 days	Sun 8/29/10	Thu 9/2/10
48	Pour Slabs and Drops - Level 7	6 days	Fri 9/3/10	Fri 9/10/10
49	Frame Columns and Walls - Level 8	8 days	Sat 9/18/10	Tue 9/28/10
50	Set Rebar Columns and Walls - Level 8	5 days	Sat 9/25/10	Thu 9/30/10
51	Pour Columns and Walls - Level 8	6 days	Fri 10/1/10	Fri 10/8/10
52	Frame Slab and Drops - Level 8	8 days	Tue 9/21/10	Thu 9/30/10
53	Set Rebar Slabs and Drops - Level 8	5 days	Tue 9/28/10	Mon 10/4/10
54	Pour Slabs and Drops - Level 8	6 days	Tue 10/5/10	Tue 10/12/10
55	Frame Columns and Walls - Level 9	8 days	Wed 10/20/10	Fri 10/29/10
56	Set Rebar Columns and Walls - Level 9	5 days	Wed 10/27/10	Tue 11/2/10
57	Pour Columns and Walls - Level 9	6 days	Wed 11/3/10	Wed 11/10/10
58	Frame Slab and Drops - Level 9	8 days	Sat 10/23/10	Tue 11/2/10
59	Set Rebar Slabs and Drops - Level 9	5 days	Sun 10/31/10	Thu 11/4/10
60	Pour Slabs and Drops - Level 9	6 days	Fri 11/5/10	Fri 11/12/10
61	Frame Columns and Walls - Roof	8 days	Sat 11/20/10	Tue 11/30/10
62	Set Rebar Columns and Walls - Roof	5 days	Sun 11/28/10	Thu 12/2/10
63	Pour Columns and Walls - Roof	6 days	Fri 12/3/10	Fri 12/10/10
64	Frame Slab and Drops - Roof	8 days	Tue 11/23/10	Thu 12/2/10
65	Set Rebar Slabs and Drops - Roof	5 days	Tue 11/30/10	Mon 12/6/10
66	Pour Slabs and Drops - Roof	6 days	Tue 12/7/10	Tue 12/14/10

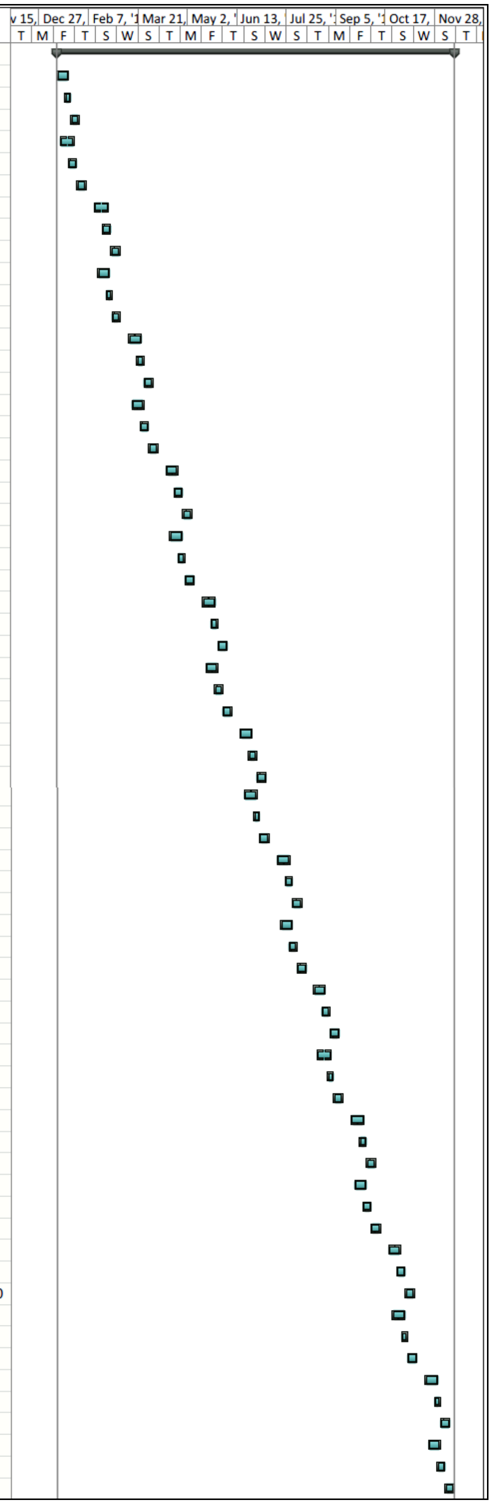

Figure S – Redesigned Concrete Construction Schedule

Mechanical Breadth

Scope

The main purpose of this breadth was to examine the building envelope and façade of GHVI and produce a more efficient glazing configuration with the idea of minimizing solar heat gain and creating a more sustainable facility. The existing glazing types and thermal characteristics were obtained from Cannon Design and the design drawings and specifications. The following glazing types are currently in use on the building:

GL-01 – 1" VNE 13-63 insulated HS/HS Silkscreen Unit:
¼" Starfire HS with V175
½" mill air spacer
¼" Clear HS VE-85 #3

GL-02 – 1" Insulated clear vision VNE-63 #2 (Viracon)
¼" Clear HS VNE1-63 #2
½" black metal
¼" Clear HS

GL-03 – 1" Insulated clear vision VE-52 #2 (Viracon)
¼" Clear HS VE1-52 #2
¼" black metal
¼" Clear HS

GL-03A – 1" Insulated translucent vision VE1-52 low-e on #2 (Viracon)
¼" Clear HS VE1-52 #2
½" black metal
¼" Clear HS 100% flood coat V1086 simulated Sandblast

GL-04 – Same as GL-03A with shadowbox

Table 23 shows the solar heat gain coefficient (SHGC) for each type of glazing. To determine the shading coefficient (SC), the solar heat gain coefficent was divided by a factor of 0.87.

Glazing Type	SHGC	SC
GL-01	0.44	0.51
GL-02	0.29	0.33
GL-03	0.40	0.46
GL-03A	0.36	0.41
GL-04	0.36	0.41

Table 23 – Glazing Values

Procedure

The first step to determining a more efficient glazing type was to model a 21' by 15'-9" typical procedural room in the academic version of Trace 700. This room was located on the second level of the building, with a story height of 18'. All four facades were considered to obtain a representative view of the entire facility. The north and south walls are similar, with a main section of glazing that is 3 panels wide and 9' high with a summer U-value of 0.29. Above and below this section of glazing is an aluminum panel wall on 6" metal studs with an R-value of 19, and a U-value of 0.05. A view of this wall with designated glazing types can be seen in Figure T. Note that the dashed lines represent the floor levels, and that each panel is 5'-3" wide.

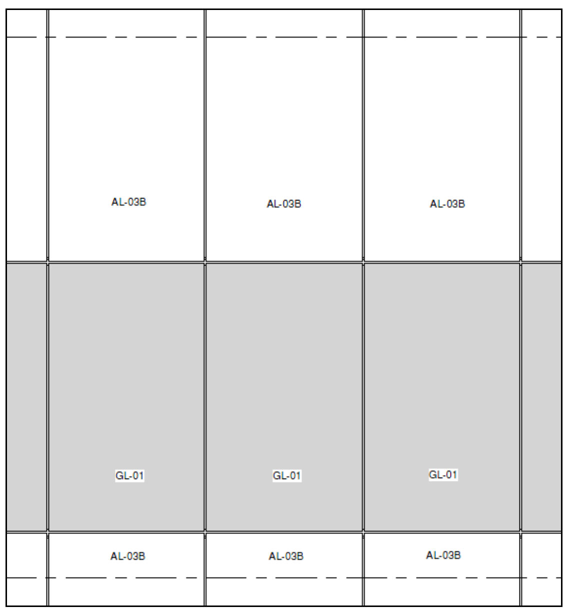

Figure T – North/South Curtain Wall

The east and west walls are also similar to each other and are fully glazed. They have a main section of glazing that is 3 panels wide and 9' high with a summer U-value of 0.29, and above and below this section there is a second type of glazing with a shadowbox. A view of this wall with designated glazing types can be seen in Figure U. Note that the dashed lines represent the floor levels, and that each panel is 5'-3" wide.

After running the first simulation with the current glazing types and shading coefficients, it was determined that four separate alternatives would be considered. The first alternative involved replacing the entire east and west facades with the makeup of the north and south facades, so that there were 9' high glazing panels with aluminum panel walls above and below. Next, the glazing on the east and west facades was entirely replaced with a triple insulating laminated glass, Viracon type VE 6-42. This glazing type had a solar heat gain coefficient of 0.24 and a shading coefficient of 0.28. The third alternative consisted of changing the glazing on all four facades to this triple insulating laminated glass. Finally, the fourth alternative involved changing all four walls so that they consisted of triple insulating laminated glazing that was 9' high with aluminum wall panels on 6" metal studs above and below.

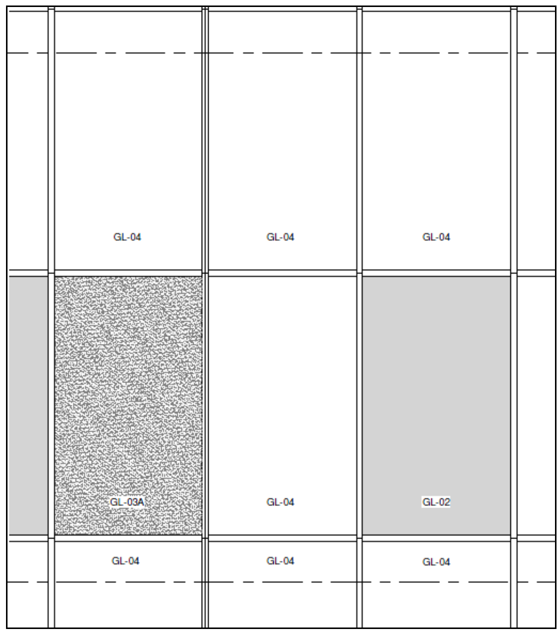

Figure U – East/West Curtain Wall

Results

The analysis results in terms of total sensible and latent heat entering the room (BTU/hr) are summarized in Table 24. In the current building design the east and west walls have the highest thermal loads because they are completely glass. By simply reducing the surface area of glass to that of the north and south walls, as in alternative 1, the thermal load is reduced by about 30 percent. Changing the glazing in the east and west walls to triple insulating laminated panels, shown in alternative 2, also has a significant effect. Obviously, the most total heat gain is prevented by building four similar facades with aluminum panel walls and 9' high triple insulating laminated panels. However, this may not be the most desirable option in terms of building architecture and lighting. Adding aluminum paneling to the east and west facades will create an obvious change in architecture. The reduced amount of glazing may also result in an insufficient level of daylight, and the need for lighting redesign. Therefore, it can be concluded that the best option would be to replace all of the current glazing with Viracon type VE 6-42 triple insulating laminated glass.

Glazing Thermal Analysis Results (BTU/hr)					
Alternative	**North**	**South**	**East**	**West**	**Total**
Current	2615	15563	23421	21443	63042
1	2615	15563	14052	14160	46391
2	2615	15563	15430	15136	48744
3	1436	8203	15430	15136	40205
4	1436	8203	7715	7568	24922

Table 24 – Glazing Thermal Analysis Results

MAE Requirements

MAE Requirements for this thesis were met in three separate ways. First, the construction of the RAM Structural System and ETABS models utilized subjects learned in AE 597A, Computer Modeling of Building Structures. These models were complex in construction, and required knowledge that would have been unknown without the assistance of that class. The second MAE level class that was used in the completion of this thesis was AE 542, Building Enclosure Science and Design. The mechanical breadth considered material that was covered in this class, including glass types, thermal analysis, and solar heat transfer. Finally, the vibration analysis also constituted a MAE level of knowledge and understanding, due to the fact that it was complex material that was never covered in class and it required extensive research.

Conclusion

The main goal of this thesis was to complete the design process of a concrete building in the hope that it would prove to be a more cost effective alternative to its steel counterpart. Although the concrete structure was found to be about $300,000 less expensive, several factors must be considered before making a conclusion.

First, the redesigned structure was found to be inadequate when it came to the vibrational velocity requirements. Fixing this problem could impose quite a large cost.

Second, the schedule for the concrete building was almost two months longer than the schedule for the steel building. Any deadlines that would not be met could result in substantial fines.

And last of all, concrete construction is extremely difficult during the long, cold winters in Buffalo, NY. Added cost would almost certainly be accrued due to the demands of cold weather concreting.

In the end, it seems as though the current steel building is the more economical design, and the correct solution for this Buffalo hospital. Although the main thesis goal was not accomplished, the entire process in itself was insightful and enjoyable, and should therefore be labeled a success.

Appendix

Appendix A: Typical Floor Plans and Elevations

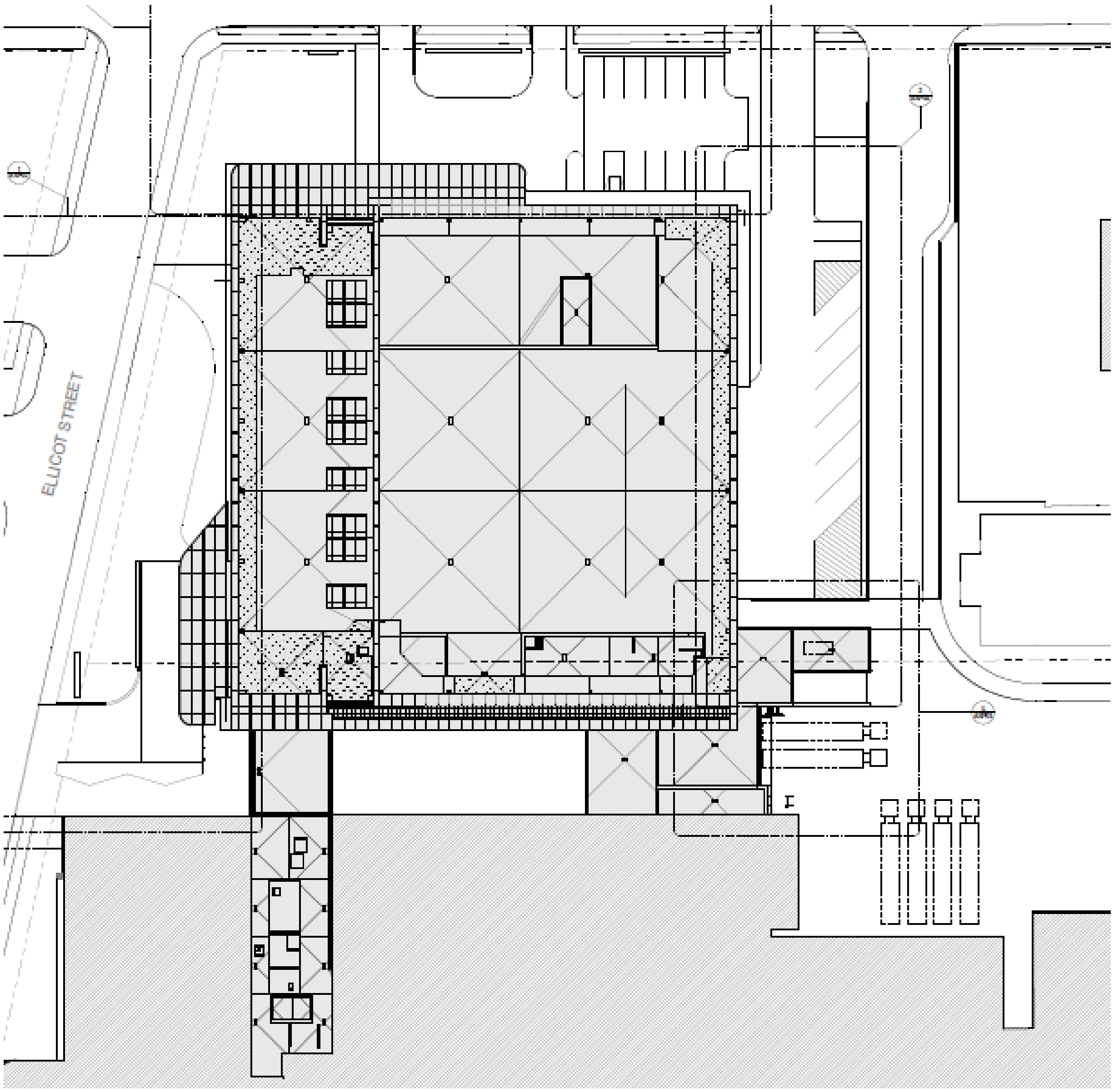

Figure V – Site Plan (Cannon Design)

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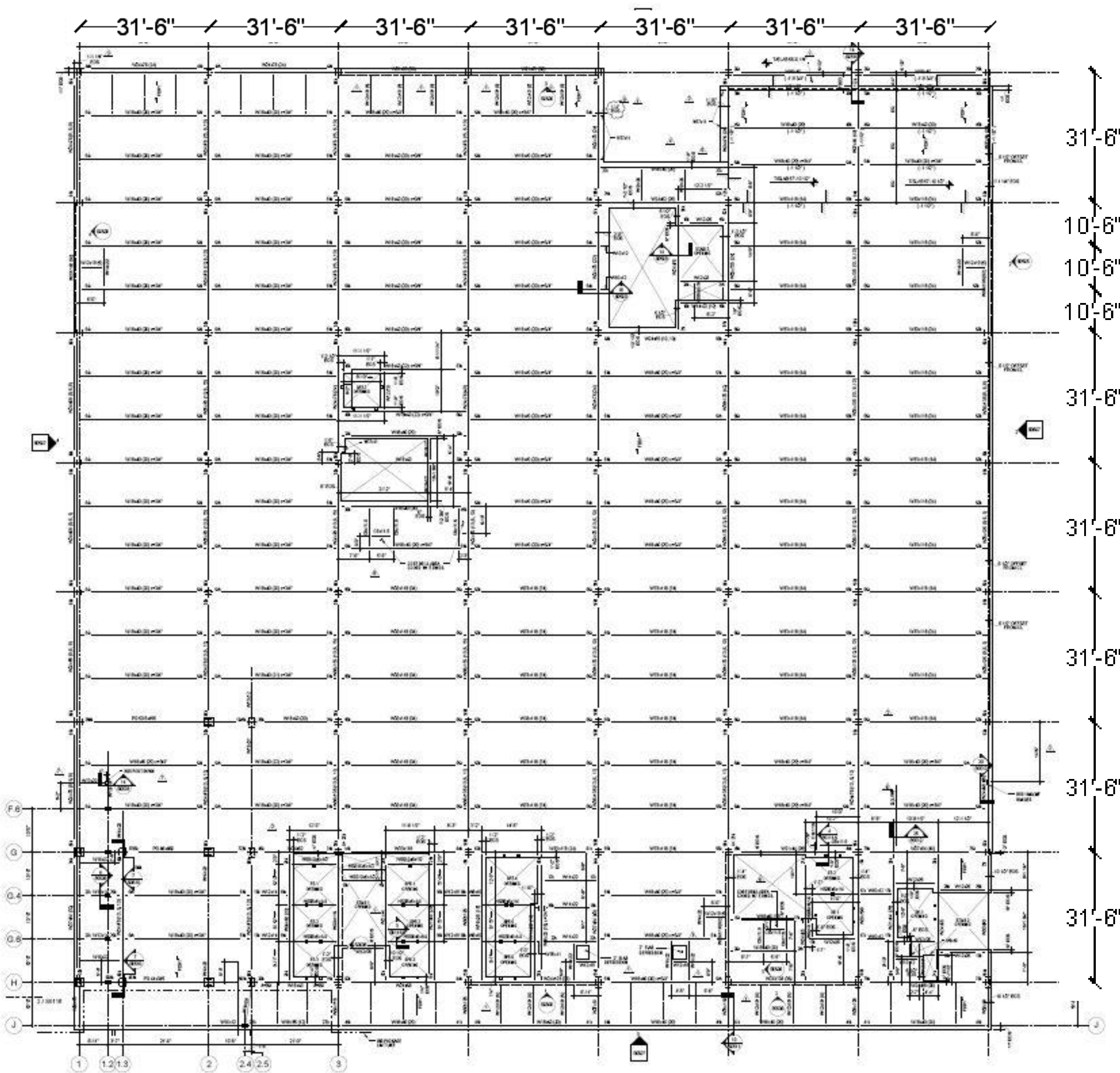

Figure W – Typical floor framing plan (Cannon Design)

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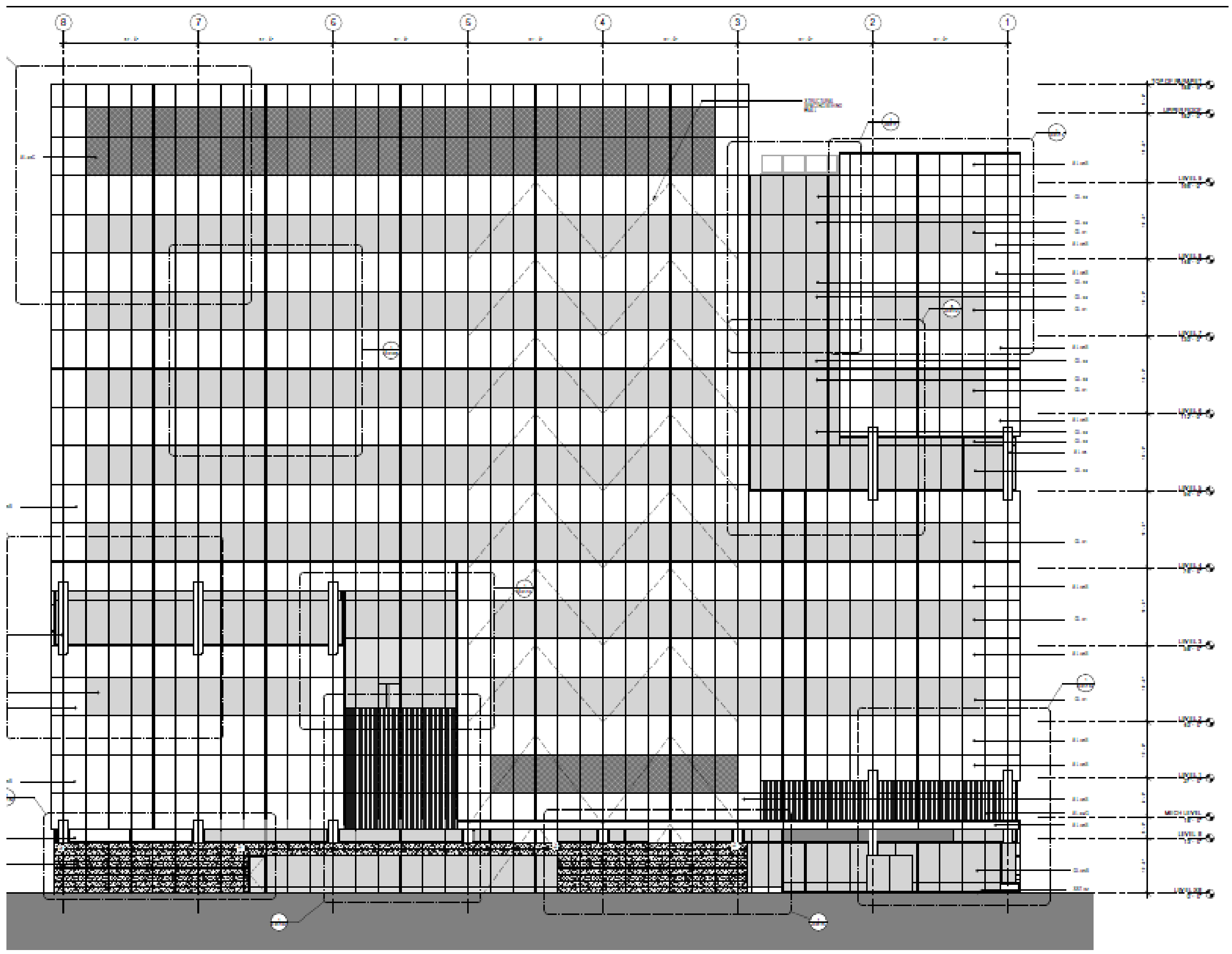

Figure X – North Elevation (Cannon Design)

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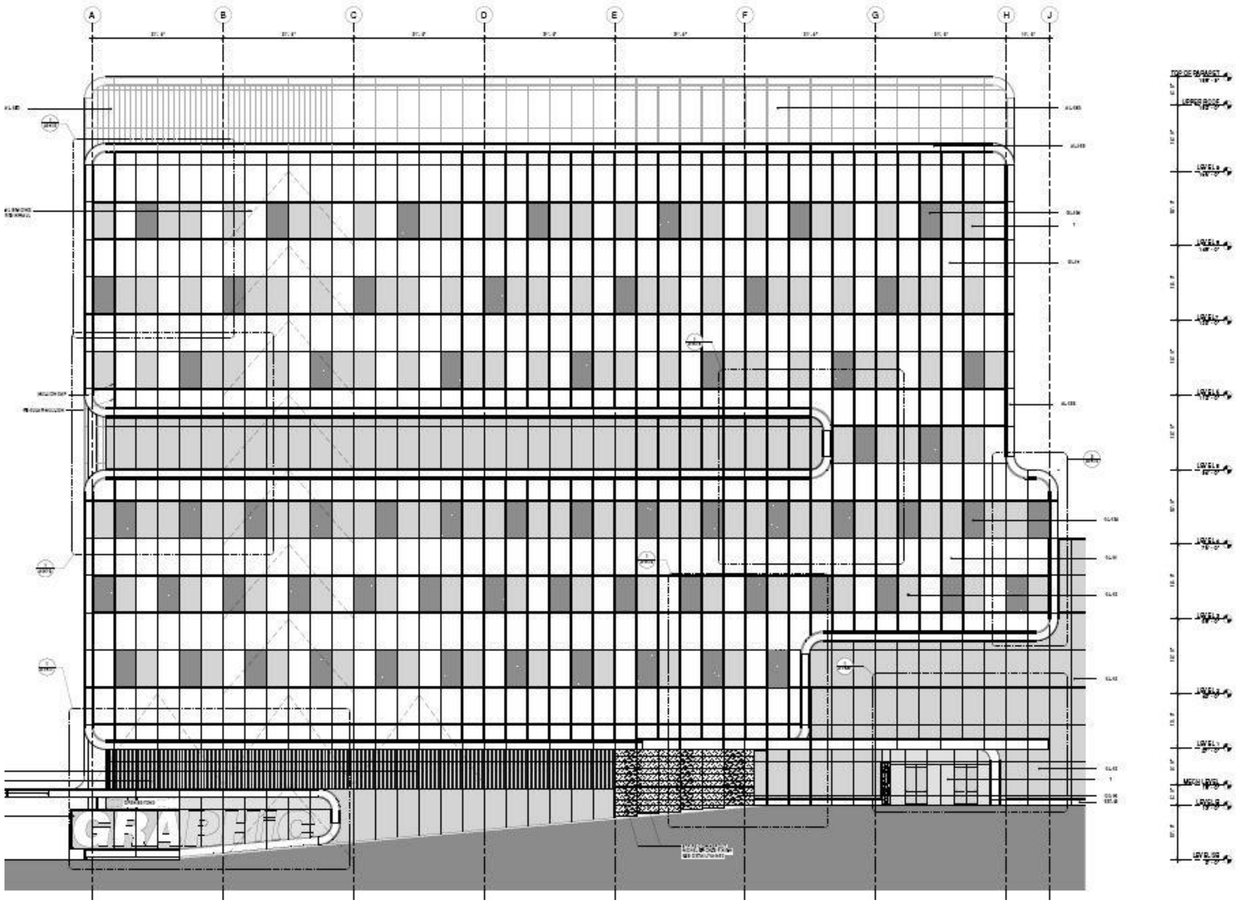

Figure Y – West Elevation (Cannon Design)

Appendix B: Wind Analysis

The following table contains the initial parameters used in the wind analysis as determined from ASCE 7-10:

V	120
K_d	0.85
Exposure	B
K_{zt}	1
GC_{pi}	0.18

Table 25 - Parameters

The following table contains the effective length calculations completed to assure that the natural frequency could be approximated:

N-S Direction				E-S Direction			
Level	**h_i**	**l_i**	**h_il_i**	**Level**	**h_i**	**l_i**	**h_il_i**
Sub basement	13	221	2873	Sub basement	13	174	2262
Basement	18	221	3978	Basement	18	221	3978
Mechanical	27	221	5967	Mechanical	27	221	5967
1	40	221	8840	1	40	221	8840
2	58	221	12818	2	58	221	12818
3	76	221	16796	3	76	221	16796
4	94	221	20774	4	94	221	20774
5	112	221	24752	5	112	221	24752
6	130	221	28730	6	130	221	28730
7	148	221	32708	7	148	221	32708
8	166	221	36686	8	166	221	36686
9	189	158	29862	9	189	221	41769
Σ =	1071		224784	**Σ =**	1071		236080
L_{eff} =	209.9			**L_{eff} =**	220.4		

Table 26 – Effective Length Check Calculations

The following table contains the calculations to determine the gust-effect factor:

Gust Effect Calculation		
	N-S	**E-W**
B	221	221
L	221	221
h	189	189
n_a	0.3888	0.3888
	FLEXIBLE	FLEXIBLE
I_z	0.244	0.244
c	0.30	0.30
z	113.4	113.4
g_Q	3.4	3.4
g_v	3.4	3.4
g_R	3.96	3.96
R	0.589	0.589
R_n	0.0967	0.0967
N_1	1.741	1.741
L_z	482.89	482.89
V_z	107.83	107.83
R_h	0.2682	0.2682
n	3.13	3.13
R_B	0.2356	0.2356
n	3.67	3.67
R_L	0.0782	0.0782
n	12.27	12.27
Q	0.799	0.799
β	0.01	0.01
G_f	0.95	0.95

Table 27 – Gust Effect Calculations

The following table contains the wind pressure coefficients:

Wind Pressure Coefficients			
Surface	**L/B**	**Cp**	**Use With**
Windward	All	0.8	q_z
Leeward	1	-0.5	q_h
Side	All	-0.7	q_h

Table 28 – Wind Pressure Coefficients

The following tables contains the wind pressures and torsional moments (if applicable) for each of the four design wind load cases as prescribed in ASCE 7-10. Wind pressure units are pounds per square feet for both the windward and leeward directions:

	Level	Height (ft)	K_z	q_z	Wind Pressure	
					N-S	E-W
Windward	Roof	185	1.18	36.9	28.0	28.0
	9	169	1.15	36.0	27.3	27.3
	8	151	1.11	34.8	26.4	26.4
	7	133	1.07	33.6	25.5	25.5
	6	115	1.03	32.2	24.4	24.4
	5	97	0.98	30.7	23.3	23.3
	4	79	0.93	29.0	22.0	22.0
	3	61	0.85	26.8	20.3	20.3
	2	43	0.76	23.8	18.1	18.1
	1	30	0.70	21.9	16.6	16.6
	Mechanical	21	0.63	19.7	15.0	15.0
	Basement	16	0.57	17.9	13.5	13.5

Table 29 – Windward Wind Pressures – Case 1

	Level	q_h	Wind Pressure	
			N-S	E-W
Leeward	Remaining	36.9	17.5	17.5

Table 30 – Leeward Wind Pressures – Case 1

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	Level	Height (ft)	K_z	q_z	Wind Pressure		M_T	
					N-S	E-W	N-S	E-W
Windward	Roof	185	1.18	36.9	21.0	21.0	250.2	250.2
	9	169	1.15	36.0	20.5	20.5	246.4	246.4
	8	151	1.11	34.8	19.8	19.8	241.7	241.7
	7	133	1.07	33.6	19.1	19.1	236.5	236.5
	6	115	1.03	32.2	18.3	18.3	230.6	230.6
	5	97	0.98	30.7	17.5	17.5	224.5	224.5
	4	79	0.93	29.0	16.5	16.5	217.3	217.3
	3	61	0.85	26.8	15.2	15.2	207.9	207.9
	2	43	0.76	23.8	13.5	13.5	195.6	195.6
	1	30	0.70	21.9	12.5	12.5	187.8	187.8
	Mechanical	21	0.63	19.7	11.2	11.2	178.6	178.6
	Basement	16	0.57	17.9	10.2	10.2	170.8	170.8

Table 31 – Windward Wind Pressure – Case 2

	Level	q_h	Wind Pressure	
			N-S	E-W
Leeward	Remaining	36.9	13.1	13.1

Table 32 – Leeward Wind Pressures – Case 2

	Level	Height (ft)	K_z	q_z	Wind Pressure	
					N-S	E-W
Windward	Roof	185	1.18	36.9	21.0	21.0
	9	169	1.15	36.0	20.5	20.5
	8	151	1.11	34.8	19.8	19.8
	7	133	1.07	33.6	19.1	19.1
	6	115	1.03	32.2	18.3	18.3
	5	97	0.98	30.7	17.5	17.5
	4	79	0.93	29.0	16.5	16.5
	3	61	0.85	26.8	15.2	15.2
	2	43	0.76	23.8	13.5	13.5
	1	30	0.70	21.9	12.5	12.5
	Mechanical	21	0.63	19.7	11.2	11.2
	Basement	16	0.57	17.9	10.2	10.2

Table 33 – Windward Wind Pressures – Case 3

	Level	q_h	Wind Pressure	
			N-S	E-W
Leeward	Remaining	36.9	13.1	13.1

Table 34 – Leeward Wind Pressures – Case 3

	Level	Height (ft)	K_z	q_z	Wind Pressure		M_T
					N-S	E-W	
Windward	Roof	185	1.18	36.9	15.8	15.8	370.0
	9	169	1.15	36.0	15.4	15.4	364.3
	8	151	1.11	34.8	14.9	14.9	357.3
	7	133	1.07	33.6	14.3	14.3	349.7
	6	115	1.03	32.2	13.7	13.7	341.0
	5	97	0.98	30.7	13.1	13.1	332.0
	4	79	0.93	29.0	12.4	12.4	321.4
	3	61	0.85	26.8	11.4	11.4	307.4
	2	43	0.76	23.8	10.2	10.2	289.3
	1	30	0.70	21.9	9.4	9.4	277.7
	Mechanical	21	0.63	19.7	8.4	8.4	264.1
	Basement	16	0.57	17.9	7.6	7.6	252.5

Table 35 – Windward Wind Pressure – Case 4

	Level	q_h	Wind Pressure	
			N-S	E-W
Leeward	Remaining	36.9	9.8	9.8

Table 36 – Leeward Wind Pressures – Case 4

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AMPAD

USE ASCE 7-10 – MWFRS (DIRECTIONAL PROCEDURE)

27.2.1

- BASIC WIND SPEED (26.5)
  OCCUPANCY CATEGORY IV (TABLE 1.5-1)
  ↳ USE FIGURE 26.5-1B → $V = 120$ mph

- WIND DIRECTIONALITY FACTOR (26.6)
  $K_d = 0.85$ (TABLE 26.6-1)

- EXPOSURE CATEGORY (26.7)
  B

- TOPOGRAPHIC FACTOR (26.8)
  $K_{zt} = 1.0$

- GUST-EFFECT FACTOR (26.9)
  RIGID?
  26.9.2.1 APPROXIMATE NATURAL FREQUENCY LIMITATIONS
  1) BUILDING HEIGHT = 189' < 300' ✓ OK
  2) BUILDING HEIGHT = 189' $\overset{?}{<}$ 4 L_{eff}
  CHECK N-S DIRECTION:

$$L_{eff} = \frac{\sum_{i=1}^{n} h_i L_i}{\sum_{i=1}^{n} h_i} = 209.9 \qquad 4(209.9) = 839.6 > 189 \ \checkmark \text{ OK}$$

CHECK E-W DIRECTION

$$L_{eff} = 220.4 \qquad 4(220.4) = 881.6 > 189 \ \checkmark \text{ OK}$$

∴ CAN APPROXIMATE

26.9.3
CONCRETE MOMENT-RESISTING FRAME BUILDING (26.9-3)

$$n_a = \frac{385\, C_w^{0.5}}{h} = \frac{385(0.006)^{0.5}}{189} = 0.16 < 1.0 \text{ Hz} \rightarrow \text{FLEXIBLE}$$

26.9.5 FLEXIBLE BUILDING

$$G_f = 0.925\left[\frac{1 + 1.7 I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right] = 0.95 \text{ (FOR BOTH N-S/E-W)}$$

B = L

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$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.30\left(\frac{33}{113.4}\right)^{1/6} = 0.244$$

$$c = 0.30$$

$$\bar{z} = \max \begin{cases} 0.6h = 0.6(189) = 113.4 \\ z_{min} = 30 \end{cases}$$

$$g_Q = g_v = 3.4$$

$$g_R = \sqrt{2\ln(3600 n_1)} + \frac{0.577}{\sqrt{2\ln(3600 n_1)}} = 3.96$$

$$n_1 = n_a = 0.3888$$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)} = 0.589$$

$$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} = \frac{7.47(1.741)}{[1 + 10.3(1.741)]^{5/3}} = 0.0967$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{0.3888(482.89)}{107.83} = 1.741$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{113.4}{33}\right)^{1/3} = 482.89$$

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}}\left(\frac{88}{60}\right)V = 0.45\left(\frac{113.4}{33}\right)^{1/4}\left(\frac{88}{60}\right)(120) = 107.83$$

R_h: $$\eta = \frac{4.6 n_1 h}{\bar{V}_{\bar{z}}} = \frac{4.6(0.3888)(189)}{107.83} = 3.13$$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right) = 0.2685$$

R_B: $$\eta = \frac{4.6 n_1 B}{\bar{V}_{\bar{z}}} = \frac{4.6(0.3888)(221)}{107.83} = 3.67 \qquad R_B = 0.2354$$

R_L: $$\eta = \frac{15.4 n_1 L}{\bar{V}_{\bar{z}}} = \frac{15.4(0.3888)(221)}{107.83} = 12.27 \qquad R_L = 0.0782$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = 0.799$$

ASSUME $\beta = 0.01 \rightarrow$ 1% DAMPING FOR CONCRETE STRUCTURE

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- ENCLOSURE CLASSIFICATION (26.10)
  ENCLOSED

- INTERNAL PRESSURE COEFFICIENT
  ENCLOSED BUILDING = ±0.18

- WALL PRESSURE COEFFICIENTS, C_p

SURFACE	L/B	C_p	USE WITH
WINDWARD	ALL	0.8	q_z
LEEWARD	1	-0.5	q_h
SIDE	ALL	-0.7	q_h

- 27.4.2 ENCLOSED FLEXIBLE BUILDING
  $p = q G_f C_p - q_i (GC_{pi})$

- 27.4.5 PARAPETS
  $P_p = q_p (GC_{pn})$
  GC_{pn} = +1.5 FOR WINDWARD PARAPET
  -1.0 FOR LEEWARD PARAPET
  $P_p = 37.1(1.5) = 55.65$ psf
  WINDWARD

  $P_p = 37.1(-1.0) = -37.1$ psf
  LEEWARD

- DESIGN PRESSURES (N-S + E-W WILL BE EQUAL BECAUSE B = L)
  WINDWARD: $p = q_z G_f C_p - q_h (GC_{pi})$
  $p = q_z (0.95)(0.8) - 37.1(\pm 0.18) = 0.76 q_z + 6.14$ psf

  ADD $P_p = 55.65$ psf TO PARAPET

  LEEWARD: $p = q_h G_f C_p - q_h (GC_{pi})$
  $p = 37.1(0.95)(-0.5) - 37.1(\pm 0.18) = -24.30$ psf

  ADD $P_p = -37.1$ psf TO PARAPET

Appendix C: Seismic Analysis

The following table contains the summation of the total building weight above grade:

Level	Weight (k)
Roof	4030
9	7441
8	8787
7	8787
6	8787
5	9203
4	9630
3	9711
2	9303
1	2167
Mech	5617
Base	2777
Total	86240

Table 37 – Total Weight

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ASCE/SEI 7-10
11.4.2 SITE CLASS

D – AS PER GEOTECHNICAL REPORT

11.4.3 SPECTRAL RESPONSE ACCELERATION

$S_S = 0.277$ BUFFALO, NY 14203

$S_1 = 0.058$

F_a: D $S_S \le 0.25$: 1.6; 0.277: $\boxed{1.58}$; $S_S = 0.5$: 1.4 $F_a = 1.58$

F_v: D $S_1 \le 0.1$: 2.4 $F_v = 2.4$

$$S_{MS} = F_a S_S = 1.58(0.277) = 0.438$$

$$S_{M1} = F_v S_1 = 2.4(0.058) = 0.139$$

11.4.4 DESIGN SPECTRAL RESPONSE ACCELERATION

$$S_{DS} = \tfrac{2}{3} S_{MS} = \tfrac{2}{3}(0.438) = 0.292 \Rightarrow \text{SEISMIC DESIGN CATEGORY C} \checkmark$$

OK FOR ELFP

$(0.167 \le S_{DS} < 0.33 + \text{IV})$

$$S_{D1} = \tfrac{2}{3} S_{M1} = \tfrac{2}{3}(0.139) = 0.093$$

ADJACENT STRUCTURE WITHIN 10'

12.8 EQUIVALENT LATERAL FORCE PROCEDURE
12.8.1 SEISMIC BASE SHEAR

$V = C_S W$

$W = 86240\text{K}$ (TABULATED IN EXCEL)

12.8.1.1 SEISMIC RESPONSE COEFFICIENT

$$C_S = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$$

$R = 5.0$ (TABLE 12.2-1, ORDINARY REINFORCED CONCRETE SHEAR WALLS – NL)

$$C_S = \frac{0.292}{\left(\frac{5.0}{1.50}\right)}$$

$I_e = 1.50$ (TABLE 1.5-2, RISK IV)

$C_S = 0.088 \Rightarrow$ SHALL NOT EXCEED: $C_S = \dfrac{S_{D1}}{T\left(\frac{R}{I_e}\right)}$ FOR $T \le T_L$

OR

$$C_S = \frac{S_{D1} T_L}{T^2\left(\frac{R}{I_e}\right)} \text{ FOR } T > T_L$$

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T = FUNDAMENTAL PERIOD – 12.8.2
T_L = LONG-PERIOD TRANSITION PERIOD(S) – 11.4.5 $T_L = 6$ (FIGURE 22-12)

12.8.2 PERIOD DETERMINATION

$$T = \min \begin{cases} C_u T_a \\ T_b \text{ - CAN BE DETERMINED LATER FROM COMPUTER MODEL} \end{cases}$$

$C_u = 1.7$ (TABLE 12.8-1, $S_{D1} = 0.093 \leq 0.1$)

$T_a = C_t h_n^x$ $C_t = 0.02$ $x = 0.75$ (TABLE 12.8-2, ALL OTHER STRUCTURAL SYSTEMS)

$T_a = 0.02(189)^{0.75}$ $h_n = 189$ ft

$T_a = 1.019$s

$T = C_u T_a = 1.7(1.019) = 1.732$s ← SAME FOR BOTH N-S/E-W DIR.

$$C_s = \min \begin{cases} \dfrac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \dfrac{0.292}{\left(\frac{5.0}{1.50}\right)} = 0.088 \\[2ex] \dfrac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \dfrac{0.093}{1.732\left(\frac{5.0}{1.50}\right)} = 0.016 \\[2ex] \dfrac{S_{D1} T_L}{T^2\left(\frac{R}{I_e}\right)} = \dfrac{0.093(6)}{(1.732)^2\left(\frac{5.0}{1.50}\right)} = 0.056 \end{cases}$$

$V = C_s W = 0.016(86240) = 1380$ k

12.8.3 VERTICAL DISTRIBUTION OF SEISMIC FORCES

$F_x = C_{vx} V$

$w_x \neq w_i$ ✓
$h_x \neq h_i$ ✓

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

	x_0	x	x_1
T	0.5	1.732	2.5
K	1	1.366	2

→ $k = 1.366$

SEE EXCEL SPREADSHEET FOR DETERMINATION OF VERTICAL FORCES

Appendix D: Snow Loading

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7.2 GROUND SNOW LOADS

P_g = CS FROM FIGURE 7-1

AS PER CASE STUDY, $P_g = 50$ psf

7.3 FLAT ROOF SNOW LOADS, P_f

$P_f = 0.7\, C_e C_t I_s P_g$

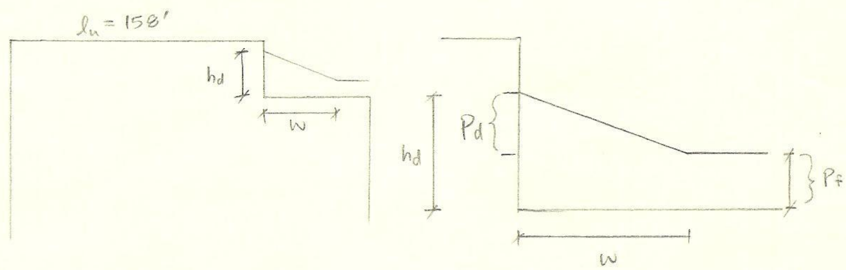
$P_f = 0.7(1.0)(1.0)(1.2)(50) = 42$ psf

7.3.1 $C_e = 1.0$ (EXPOSURE B, PARTIALLY EXPOSED - PARAPET)

7.3.2 $C_t = 1.0$

7.3.3 $I_s = 1.2$ (RISK CATEGORY IV)

7.7 DRIFT



$h_d = 0.43\sqrt[3]{l_u}\,\sqrt[4]{P_g + 10} - 1.5$

$h_d = 0.43(158)^{1/3}(50+10)^{1/4} - 1.5 = 4.97$ ft

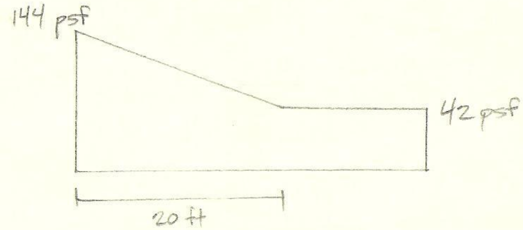
$w = 4h_d = 4(4.97)$

$w = 19.9$ ft

$\gamma = \begin{cases} 0.13P_g + 14 = 0.13(50) + 14 = 20.5 \text{ pcf} \\ \min\ 30 \text{ pcf} \end{cases}$

$P_d = h_d\gamma = 4.97(20.5) = 101.9$ psf

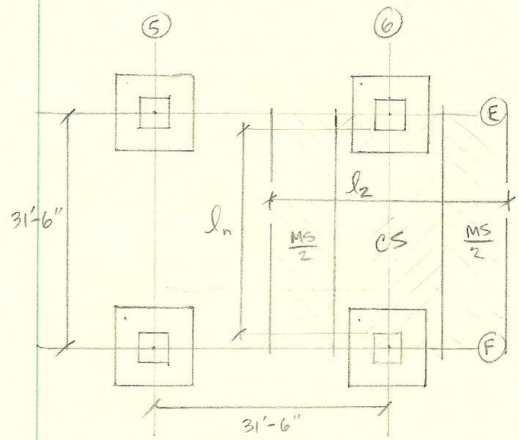
$P_{total} = 42 + 101.9 = 143.9$ psf



Appendix E: Gravity System Redesign

WILLIAM McDEVITT | FINAL REPORT | FLAT SLAB WITH DROP PANELS | (1)

FLAT SLAB WITH DROP PANELS



UTILIZE 31'-6" x 31'-6" BAYS

ASSUME: 32" x 32" COLUMNS

BECAUSE THIS IS A SQUARE BAY, BOTH DIRECTIONS WILL BE THE SAME

$$\ell_n = 31.5' - 2\left(\tfrac{32}{2\cdot 12}\right) = 28.8'$$

DETERMINE SLAB THICKNESS

- SLAB WITHOUT INTERIOR BEAMS, WITH DROP PANELS

ACI 9.5.3.2:

$$t = \max\begin{cases} 4\text{ in} \\ \text{INT.} = \frac{\ell_n}{36} = \frac{28.8'(12\text{in/ft})}{36} = 9.6\text{ in} \\ \text{EXT.} = \frac{\ell_n}{33} = \frac{28.8'(12\text{in/ft})}{33} \approx 10.5\text{ in} \end{cases} \rightarrow \text{USE } 11'' \text{ SLAB}$$

METHOD FOR COMPUTING DESIGN MOMENTS

DIRECT DESIGN METHOD CAN BE USED:

- MORE THAN 3 CONTINUOUS SPANS IN EACH DIRECTION ✓
- $\frac{\text{LONG SPAN}}{\text{SHORT SPAN}} = 1 < 2$ ✓
- SUCCESSIVE SPAN LENGTHS SHALL NOT DIFFER MORE THAN 1/3 LONGER SPAN ✓
- COLUMN OFFSETS ✓
- GRAVITY ONLY LOADS + UNIFORMLY DISTRIBUTED ✓
- UNFACTORED LL ≤ 2 · UNFACTORED DL

LL = 125 psf

SDL = 25 psf

$SLAB = \frac{11''}{12''/ft}(150\text{ pcf}) = 137.5\text{ psf}$ $\quad 125 \le 2(25 + 137.5)$ ✓

COMPUTE POSITIVE + NEGATIVE MOMENTS IN SLAB

$$M_o = \frac{q_u \ell_2 \ell_n^2}{8}$$

$$q_u = 1.2\left[\frac{11''}{12\text{in/ft}}(150\text{pcf}) + 25\text{psf}\right] + 1.6(125\text{psf}) = 0.395\text{ ksf}$$

$$\ell_n = 31.5' - 2\left(\frac{32''}{2 \cdot 12}\right) = 28.8'$$

$$\ell_2 = 2\left(\frac{31.5'}{2}\right) = 31.5' \qquad \ell_1 = 31.5'$$

$$M_o = \frac{0.395(31.5)(28.8)^2}{8} = 1290\ 'k$$

NEGATIVE MOMENT $= -0.65\,M_o = -0.65(1290) = -839\ 'k$

POSITIVE MOMENT $= 0.35\,M_o = 0.35(1290) = 452\ 'k$

DISTRIBUTE TO COLUMN STRIP + MIDDLE STRIP

$\alpha_{f1} = 0$ (NO BEAMS)

$\frac{\ell_2}{\ell_1} = 1$

NEGATIVE MOMENTS:

COLUMN STRIP NEG $= 0.75(-839) = -629.3\ 'k / 15.76' = -39.9\ 'k/FT$

ACI 13.6.4.1

MIDDLE STRIP NEG $= (1-0.75)(-839) = -209.8\ 'k / 15.74' = -13.3\ 'k/FT$

POSITIVE MOMENTS:

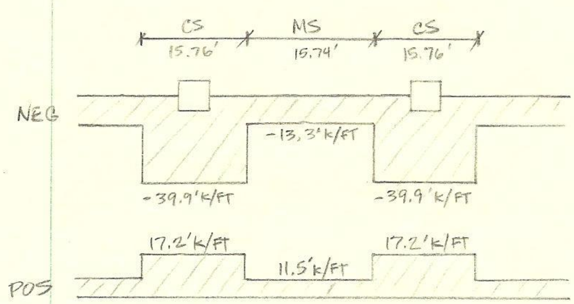
COLUMN STRIP POS $= 0.60(452) = 271.2\ 'k / 15.76' = 17.2\ 'k/FT$

ACI 13.6.4.4

MIDDLE STRIP POS $= (1-0.60)(452) = 180.8\ 'k / 15.74' = 11.5\ 'k/FT$

COLUMN STRIP SIZE:

$$\ell_{min}/4 = \min \begin{cases} \frac{\ell_1}{4} = \frac{31.5}{4} = 7.88' \\ \frac{\ell_2}{4} = \frac{31.5}{4} = 7.88' \end{cases}$$



DESIGN SLAB REINFORCING

BECAUSE OF THE SQUARE BAY, THE REINFORCEMENT IN BOTH DIRECTIONS WILL BE THE SAME

$d_{LONG\ SPAN} = 11'' - 1.15 = 9.85''$

$d_{SHORT\ SPAN} = 11'' - 1.9 = 9.1''$ ← THIS IS CONSERVATIVE FOR BOTH DIRECTIONS

ASSUME $jd = 0.95d = 0.95(9.1) = 8.7$ (USE $jd = 0.85d = 6.87$ WHEN NEEDED)

- COLUMN STRIP POSITIVE MOMENT = 17.2'K/FT

$A_{s\,req} = \frac{M_u}{\phi f_y jd} = \frac{17.2(12^{in}/ft)}{0.9(60)(8.7)} = 0.439\ in^2/FT$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.439(60)}{0.85(4)(12)} = 0.65\ in$

$c = \frac{a}{\beta_1} = \frac{0.65}{0.85} = 0.76\ in \overset{\checkmark}{\leq} 0.375 d_t = 0.375(9.1) = 3.41\ in \checkmark \therefore \phi = 0.9$

$jd = d - \frac{a}{2} = 9.1 - \frac{0.65}{2} = 8.78 > 8.70$ ∴ CONSERVATIVE

$A_{s\,min} = 0.0018bh = 0.0018(12'')(11'') = 0.238\ in^2 < 0.439\ in^2 \checkmark$ ∴ OK

$S_{MAX} = \min \begin{cases} 2h = 2(11) = 22'' \\ 18'' \end{cases}$ → USE 12" SPACING SINCE DESIGNING A 12" STRIP

USE #7 @ 12" WITH $A_s = 0.60\ in^2 > 0.439\ in^2$

- COLUMN STRIP NEGATIVE MOMENT = 39.9'K/FT

$A_{s\,req} = \frac{40.3(12)}{0.9(60)(6.87)} = 1.30\ in^2/FT > \underbrace{0.238\ in^2}_{A_{s\,min}} \checkmark$ ∴ OK

$a = \frac{1.30(60)}{0.85(4)(12)} = 1.91\ in$

$c = \frac{a}{\beta_1} = \frac{1.91}{0.85} = 2.25\ in \overset{\checkmark}{\leq} 0.375 d_t = 0.375(8.1) = 3.04\ in \checkmark \therefore \phi = 0.9$

$jd = 8.1 - \frac{1.91}{2} = 7.15 > 6.87$ ∴ CONSERVATIVE

$A_{req-4''} = 1.30\ in^2/ft \left(\frac{1}{3}\right) = 0.43\ in^2/4\ in$ → USE #6 @ 4"

USE #6 @ 4" WITH $A_s = 0.44\ in^2 > 0.43\ in^2$

Final Report

– MIDDLE STRIP POSITIVE MOMENT = 11.6 'k/FT

$A_{s_{req}} = \frac{11.6(12)}{0.9(60)(7.7)} = 0.33\ in^2 > 0.216\ in^2$ ✓ ∴ OK
↑ $A_{s_{min}}$

$a = \frac{0.33(60)}{0.85(4)(12)} = 0.49\ in$

$c = \frac{a}{\beta_1} = \frac{0.49}{0.85} = 0.58\ in < 3.04\ in$ ✓ $\therefore \phi = 0.9$

$jd = 8.1 - \frac{0.49}{2} = 7.86 > 7.70$ ∴ CONSERVATIVE

USE #6 @ 12" WITH $A_s = 0.44\ in^2 > 0.33\ in^2$

– MIDDLE STRIP NEGATIVE MOMENT = 13.4 'k/FT

$A_{s_{req}} = \frac{13.4(12)}{0.9(60)(7.7)} = 0.39\ in^2 > 0.216\ in^2$ ✓ ∴ OK
↑ $A_{s_{min}}$

$a = \frac{0.39(60)}{0.85(4)(12)} = 0.57\ in$

$c = \frac{a}{\beta_1} = \frac{0.57}{0.85} = 0.67\ in < 3.04\ in$ ✓ $\therefore \phi = 0.9$

$jd = 8.1 - \frac{0.57}{2} = 7.82 > 7.70$ ∴ CONSERVATIVE

USE #6 @ 12" WITH $A_s = 0.44\ in^2 > 0.39\ in^2$

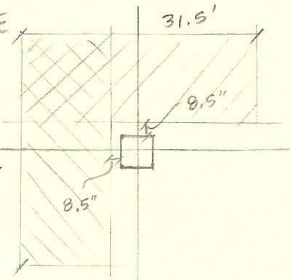
DESIGN FOR SHEAR – BOTH DIRECTIONS THE SAME

ONE-WAY SHEAR:

$d = 10 - \underset{COVER}{3/4} - \underset{d_b}{0.75} = 8.5"$

$V_u = 0.380\ ksf\left(\frac{31.5'}{2} - \frac{24"/2 + 8.5"}{12\ in/ft}\right)(31.5')$

$V_u = 168.1\ k$



$\phi V_n = \phi V_c$

$\phi V_c = 0.75(2\lambda\sqrt{f'_c}\,bd)$

$= 0.75(2)(1.0)\sqrt{4000}\,(31.5')(12\ in/ft)(8.5) = 305\ k > 168.1\ k$ ✓ ∴ OK

WILLIAM McDEVITT FINAL REPORT FLAT SLAB WITH DROP PANELS (5)

TWO-WAY PUNCHING SHEAR:

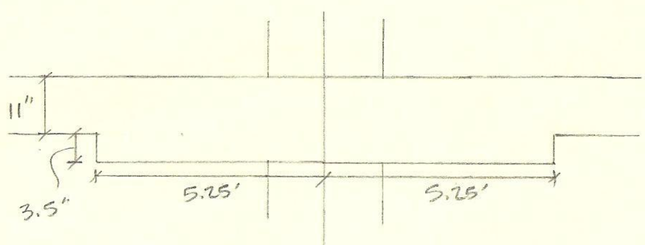
$$V_u = 0.380\,ksf\left[(31.5')(31.5') - \left(\frac{24+2(8.5/2)}{12in/ft}\right)\left(\frac{24+2(8.5/2)}{12in/ft}\right)\right] = 374\ k$$

$$b_o = 4(24 + 2(8.5/2)) = 130\ in$$

$$V_c = \min\begin{cases} 4\lambda\sqrt{f'_c}\,b_o d = 4(1.0)\sqrt{4000}(130)(8.5) = 279.5\ k \\ \left(2+\frac{4}{\beta}\right)\lambda\sqrt{f'_c}\,b_o d = \left(2+\frac{4}{1}\right)(1.0)\sqrt{4000}(130)(8.5) = 419.3\ k \\ \left(\frac{\alpha_s d}{b_o}+2\right)\lambda\sqrt{f'_c}\,b_o d = \left[\frac{40(8.5)}{130}+2\right](1.0)\sqrt{4000}(130)(8.5) = 322.6\ k \end{cases}$$

$\beta = \frac{24}{24} = 1$, $\alpha_s = 40$ FOR INTERIOR COLUMN

$\phi V_c = 0.75(279.5) = 209.6\ k < 374\ k = V_u$ X NO GOOD – ADD DROP PANELS



$h_d = \frac{10}{4} = 2.5 \rightarrow$ USE 3.5" FOR EASE OF CONSTRUCTION

$\frac{l_1}{6} = \frac{l_2}{6} = \frac{31.5'}{6} = 5.25'$

$11 + 3.5'' = 14.5''$

$d = 14.5 - 3/4 - 0.75 = 13.0''$

NEW $V_u = 373\ k$

$b_o = 4\left(24 + 2\left(\frac{13.0}{2}\right)\right) = 148''$

$$V_c = \min\begin{cases} 4(1.0)\sqrt{4000}(148)(13.0) = 487\ k \\ \left(2+\frac{4}{1}\right)(1.0)\sqrt{4000}(148)(13.0) = 730\ k \\ \left[\frac{40(13.0)}{147}+2\right](1.0)\sqrt{4000}(148)(13.0) = 674\ k \end{cases}$$

$\phi V_c = 0.75(487k) = 365.3\ k > 373\ k = V_u$ X $\therefore$ INCREASE DEPTH OR TRY 6000 psi

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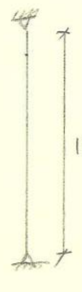
CHECK COLUMN H8 ON SUBBASEMENT LEVEL (CORNER COLUMN) – 28×28



$16' = 192'' = l_c$

USE LOAD CASE $1.2D + 1.0L + 1.6W$

$\Sigma P_u = 75936\text{ k}$ (TABULATED IN EXCEL)

$V_{us} = 1535.5\text{ k}$ (WIND BASE SHEAR)

$\Delta_o = 0.030\text{ in}$ (LATERAL DRIFT)

(1) IS IT A SWAY FRAME? (ACI 10.10.5.2)

$$Q = \frac{\Sigma P_u \Delta_o}{V_{us} l_c} \le 0.05$$

$$Q = \frac{75936(0.030)}{1535.5(192)} = 0.008 < 0.05 \checkmark \therefore \text{NONSWAY FRAME}$$

(2) CHECK SLENDERNESS

$$\frac{k l_u}{r} \le 22$$

$k = 1.0$ (FOR NONSWAY)
$l_u = 192''$
$r = 0.3h = 0.3(28) = 8.4$

$$\frac{1.0(192)}{8.4} = 23 > 22 \longrightarrow \text{COLUMN IS SLENDER}$$

(3) MOMENT MAGNIFICATION FACTOR

$$M_c = \delta_{ns} M_2$$

$P_u = 502\text{ k}$
$M_2 = 135\text{ in-k}$
$M_1 = 0$

$$\delta_{ns} = \frac{C_m}{1 - \frac{P_u}{0.75 P_c}} = 0.6 < 1.0$$ MOMENT MAGNIFICATION DOES NOT INFLUENCE COLUMN BEHAVIOR, $\delta_{ns} = 1.0$

$$C_m = 0.6 + 0.4(M_1/M_2) = 0.6$$

$$P_c = \frac{\pi^2 EI}{(kl_u)^2} = 98737\text{ k}$$

$$EI_{eff} = \frac{0.4 E_c I_g}{1 + \beta_{dns}} = \frac{0.4(3600)(28^4/12)}{1 + 1.0} = 3.69 \times 10^8\text{ k-in}^2$$

$$M_c = \delta_{ns} M_2 = 1.0(135\text{ in-k}) = 135\text{ in-k} = 11.3'\text{k}$$

$\longrightarrow$ CHECK IN MACGREGOR – WELL OVER DESIGNED, UNBRACED LENGTH CONTROLS

Final Report

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DESIGN 36x36 COLUMN

$P_u = 4042$ k

$M_u = 118'$k

ASSUME $d' = 2.5$ in

$e = \frac{M_u}{P_u} = \frac{118(12)}{4042} = 0.35$ in

$h = 36''$, $\gamma = \frac{h - 2d}{h} = \frac{36 - 2(2.5)}{36} = 0.86$, $\frac{e}{h} = 0.010$

$\frac{\phi P_n}{bh} = \frac{4042}{36(36)} = 3.12$

$\frac{\phi M_n}{bh^2} = \frac{118(12)}{36(36)^2} = 0.03$

FROM MACGREGOR 2009 TEXT, APPENDIX A

$f'_c = 6$, $f_y = 60$, $\gamma = 0.75$ $\qquad \gamma = 0.90$

$\rho = 0.023 \qquad \rho = 0.022$

FROM INTERPOLATION $\rightarrow$ $\rho = 0.022 < 0.05$ ✓ OK

$A_{s\,req} = \rho bh = 0.022(36)(36) = 28.5$ in²

$\frac{28.5}{1.27} = 22.4 \rightarrow 24 - \#10 \quad A_s = 29.2 > 28.5$ ✓ OK

THIS IS RELATIVELY CLOSE TO THE spCOLUMN OUTPUT

TIE SPACING

$$s \le \min \begin{cases} 16(1.27) = 20.32'' \\ 48(0.375) = 18'' \\ 36'' \end{cases}$$

USE TIES SPACED AT 12" O.C.

CHECK b_{min}

$b_{min} = 2(1.5) + 2(0.375) + 7(1.27) + 6(1.5)(1.27) = 24.1 < 36$ ✓ OK

Final Report

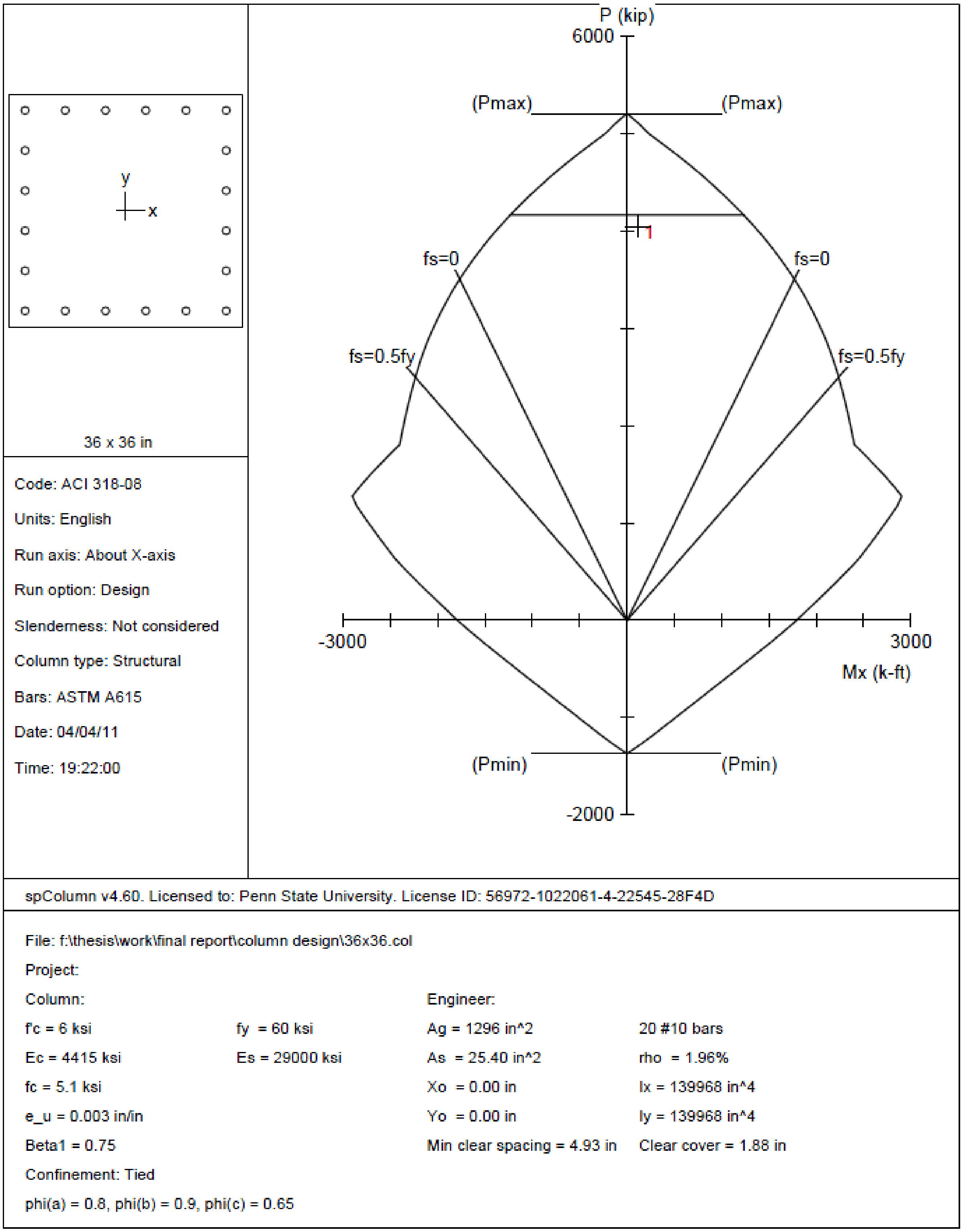


Figure Z – 36" x 36" Column Design (spColumn)

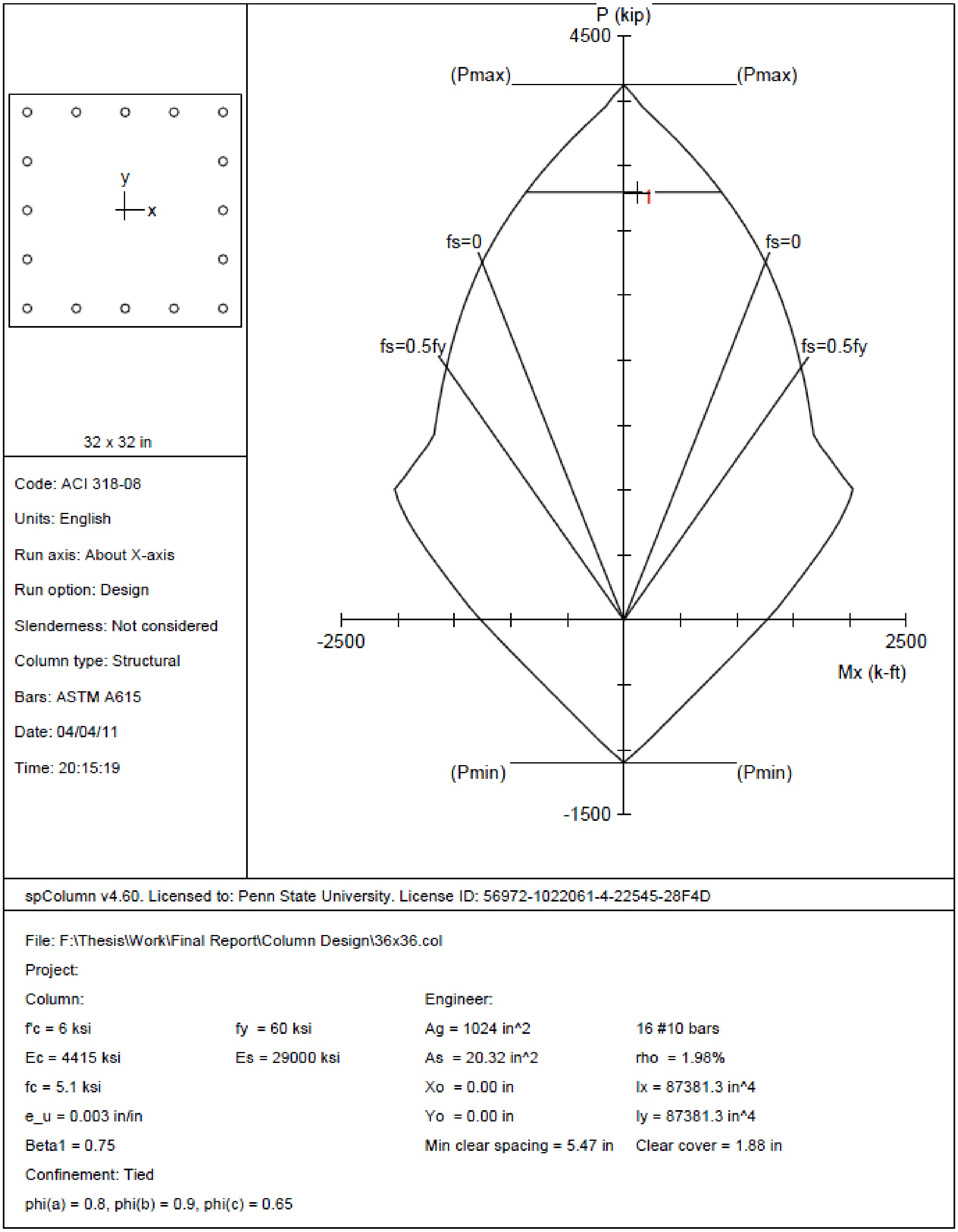
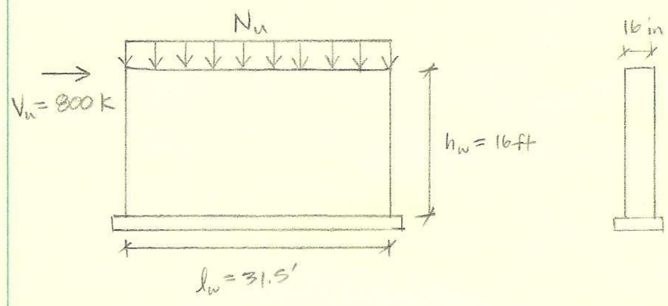


Figure AA – 32" x 32" Column Design (spColumn)

Appendix F: Lateral System Redesign

WILLIAM McDEVITT | FINAL REPORT | SHEAR WALL DESIGN | 1



N_u = SELF WEIGHT OF SHEAR WALLS ABOVE (173')

$N_u = (173')(16\text{in}/12\text{in/ft})(31.5')(150\text{ pcf}) = 1090\text{ k}$

$\phi V_n \geq V_u = V_c + V_s$

V_c

SIMPLIFIED METHOD:

$V_c \leq 2\sqrt{f'_c}\, hd$

$d = 0.8\, l_w$
$= 0.8(31.5' \times 12\text{in/ft}) = 302.4\text{ in}$

$\leq 2\sqrt{6000}(16)(302.4)/1000 = 750\text{ K}$ ← USE THIS

OTHER METHOD: (MIN OF 1 OR 2)

① $V_c \leq 3.3\sqrt{f'_c}\, hd + \dfrac{N_u d}{4 l_w}$

$\leq 3.3\sqrt{6000}(16)(302.4)/1000 + \dfrac{1090(302.4)}{4(31.5\times12)} = 1455\text{ K}$

② $V_c \leq \left[0.6\sqrt{f'_c} + \dfrac{l_w(1.25\sqrt{f'_c} + 0.2 N_u/l_w h)}{M_u/V_u - l_w/2}\right] hd$

$M_u = (800)(16) = 12800'\text{k} = 153600\text{ in-k}$

$V_c \leq \left[0.6\sqrt{6000} + \dfrac{(31.5\times12)(1.25\sqrt{6000} + 0.2(1090)/(31.5\times12\times16))}{\frac{153600}{800}}\right](16)(302.4)$

$V_c \leq 1148\text{ K}$

HORIZONTAL REINFORCEMENT

$\frac{1}{2}\phi V_c = \frac{1}{2}(0.75)(750) = 281.3\text{ k} < 800\text{ k}$ ∴ NEED REINF.

$V_u \leq \phi V_n = \phi(V_c + V_s)$

$800 = 0.75(750 + V_s)$

$V_s = 316.7\text{ k}$

TRY 2-#4 @ 10"

$V_s = \frac{A_v f_y d}{s} = \frac{2(0.2)(60)(302.4)}{10} = 725.8 > 316.7\text{k}$ ✓ OK

$\rho_t = \frac{A_v}{sh} = \frac{2(0.2)}{10(16)} = 0.0025 \geq 0.0025$ ✓ OK

$$s \leq \begin{cases} l_w/5 = 31.5(12)/5 = 75.6\text{ in} \\ 3h = 3(16) = 48\text{ in} \\ 18\text{ in} > 10\text{ in} \quad ✓ \therefore \text{OK} \end{cases}$$

∴ USE 2-#4 @ 10" O.C. FOR HORIZONTAL REINFORCEMENT

VERTICAL REINFORCEMENT

$\rho_l = \frac{A_v}{sh} \geq 0.0025 + 0.5\cancel{\left(2.5 - \frac{h_w}{l_w}\right)}^{0}(\rho_t - 0.0025)$ $\rho_t = 0.0025$

$\rho_l = \frac{A_v}{sh} \geq 0.0025$

∴ USE 2-#4 @ 10" O.C. FOR VERTICAL REINFORCEMENT

FLEXURAL DESIGN

$C = T$

$0.85 f'_c ba = A_s f_y$

$M_u = A_s f_y (d - \frac{a}{2})$ OR $M_u = A_s f_y (jd)$

$jd = 0.9d$

$d = 302.4$

$jd = 0.9(302.4) = 272.2\text{ in}$

$M_u = 800(16) = 12800\text{'k}$

$M_u = \phi M_n = \phi A_s f_y jd$

$12800(12\text{in/ft}) = 0.9 A_s (60)(272.2) \rightarrow A_s = 10.4\text{ in}^2$

Final Report

$C = T$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{10.4\,(60000)}{0.85(6000)(16)} = 7.65 \text{ in}$$

$$jd = d - \frac{a}{2} = 302.4 - \frac{7.65}{2} = 298.6$$

$$A_s = \frac{12800\,(12)}{0.9(60)(298.6)} = 9.5 \text{ in}^2$$

TRY $10 - \#9s$, $A_s = 10.0 \text{ in}^2$

TENSION CONTROLLED SECTION

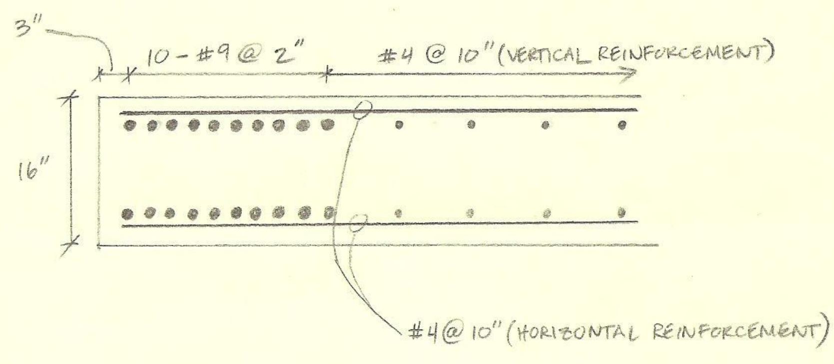
$$d_t = 31.5'(12 \text{ in/ft}) - 3'' = 375 \text{ in}$$

$C = T$
$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{10\,(60000)}{0.85(6000)(16)} = 7.35 \text{ in}$$

$$c = \frac{a}{\beta_1} = \frac{7.35}{0.85} = 8.65 \text{ in}$$

$$\varepsilon_t = \varepsilon_u\left(\frac{d_t - c}{c}\right) = 0.003\left(\frac{375 - 8.65}{8.65}\right) = 0.127 > 0.005 \ \checkmark \text{ OK}$$

∴ TENSION CONTROLS



Appendix G: Vibration Analysis

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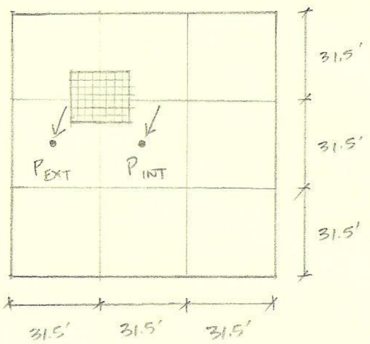


P = 1 K

FIND DEFLECTION

COLUMNS HALFWAY ABOVE + BELOW

ASSUMPTIONS

$1.2 E_c$ – FOR ALL MEMBERS

$0.7 I_g$ – COLUMNS

$0.25 I_g$ – SLAB + DROPS

CRACKED SECTION PROPERTIES FROM ACI 10.10.4.1

FROM DESIGN GUIDE 11

FOR $f_n > 5.0$

$$V = \frac{U_v \Delta_p}{f_n} \qquad (6.4b)$$

FOR MODERATE WALKING (75)

$U_v = 5500\ \text{lb} \cdot \text{Hz}^2$

FOR EXTERIOR BAY

– ASSUME MODE 7

$T = 0.15133\ s,\quad f_n = 6.60793\ \text{Hz},\quad \Delta_p = 0.00472\ \text{in}$

$$V = \frac{(5500\ \text{lb}\cdot\text{Hz}^2)(4.72 \times 10^{-6}\ \text{in/lb})}{6.60793\ \text{Hz}} = \boxed{3929\ \mu\text{in/s}}$$

FOR INTERIOR BAY

– ASSUME MODE 11

$T = 0.12631\ s,\quad f_n = 7.91720\ \text{Hz},\quad \Delta_p = 0.00420\ \text{in}$

$$V = \frac{(5500\ \text{lb}\cdot\text{Hz}^2)(4.20 \times 10^{-6}\ \text{in/lb})}{7.91720\ \text{Hz}} = \boxed{2918\ \mu\text{in/s}}$$

Appendix H: Construction Management Breadth – Steel Takeoff Examples

Total Gravity Column Takeoff									
Steel Grade: 50 - I Section									
Size	**#**	**Length (ft)**	**Weight (lbs)**	**Unit**	**Crew**	**Daily Output**	**Labor Hours**	**Bare Total**	**Cost**
W14X68	43	1221	83062	L.F.	E2	765	0.073	87.36	106625.00
W14X74	3	107	7900	L.F.	E2	760	0.074	94.64	10079.16
W14X82	11	355	28992	L.F.	E2	750	0.075	104.47	37085.08
W14X90	25	864	77910	L.F.	E2	740	0.076	114.29	98746.56
W14X99	7	234	23171	L.F.	E2	734	0.077	125.14	29281.59
W14X109	13	468	50960	L.F.	E2	727	0.077	137.19	64202.58
W14X120	18	655	78662	L.F.	E2	720	0.078	150.44	98523.16
W14X132	5	164	21702	L.F.	E2	710	0.079	165.00	27125.43
W14X145	26	905	131459	L.F.	E2	698	0.080	180.77	163557.16
W14X159	15	531	84421	L.F.	E2	686	0.081	197.75	105063.88
W14X176	5	194	34173	L.F.	E2	671	0.083	218.37	42342.03
W14X193	12	439	84825	L.F.	E2	657	0.084	238.99	104893.67
W14X211	12	444	93645	L.F.	E2	641	0.086	260.83	115781.09
W14X233	6	210	48920	L.F.	E2	622	0.088	287.51	60349.17
W14X257	9	345	88848	L.F.	E2	601	0.090	316.63	109362.95
W14X283	7	269	76142	L.F.	E2	578	0.092	348.17	93517.41
W14X311	2	56	17533	L.F.	E2	554	0.095	382.13	21552.21
W14X370	2	56	20910	L.F.	E2	503	0.100	453.70	25588.73
								Total Cost ($)	1313676.83

Table 38 – Steel Column Takeoff

Metal Decking								
Level	**Area (ft2)**	**Deck Depth**	**Unit**	**Crew**	**Daily Output**	**Labor Hours**	**Bare Total ($)**	**Cost ($)**
Roof	22822	3	S.F.	E4	3600	0.009	2.82	64357.34
9	37706	3	S.F.	E4	3600	0.009	2.82	106329.51
8	43990	3	S.F.	E4	3600	0.009	2.82	124051.10
7	43990	3	S.F.	E4	3600	0.009	2.82	124051.10
6	43990	3	S.F.	E4	3600	0.009	2.82	124051.10
5	46636	3	S.F.	E4	3600	0.009	2.82	131512.82
4	48620	3	S.F.	E4	3600	0.009	2.82	137109.11
3	48620	3	S.F.	E4	3600	0.009	2.82	137109.11
2	46636	3	S.F.	E4	3600	0.009	2.82	131512.82
1	7938	2	S.F.	E4	3600	0.009	2.82	22385.16
Mech	31752	3	S.F.	E4	3600	0.009	2.82	89540.64
Base	13892	3	S.F.	E4	3600	0.009	2.82	39174.03
							Total Cost ($)	1231183.80

Table 39 – Metal Decking Takeoff

Note: For brevity, not all tables were included. Contact author to view remaining tables.

Appendix I: Construction Management Breadth – Concrete Takeoff Examples

Columns - 6000 psi Concrete						
Level	**Number**	**Size (in)**	**Height (ft)**	**Unit**	**Bare Total ($)**	**Cost ($)**
Roof	37	20	16	C.Y.	127.00	7734.98
9	64	24	18	C.Y.	127.00	21674.67
8	64	24	18	C.Y.	127.00	21674.67
7	64	24	18	C.Y.	127.00	21674.67
6	64	24	18	C.Y.	127.00	21674.67
5	36	28	18	C.Y.	127.00	16594.67
	28	24	18	C.Y.	127.00	9482.67
4	36	32	18	C.Y.	127.00	21674.67
	28	24	18	C.Y.	127.00	9482.67
3	36	32	18	C.Y.	127.00	21674.67
	28	24	18	C.Y.	127.00	9482.67
2	36	32	13	C.Y.	127.00	15653.93
	28	28	13	C.Y.	127.00	9321.70
1	36	32	9	C.Y.	127.00	10837.33
	28	28	9	C.Y.	127.00	6453.48
Mech	36	36	5	C.Y.	127.00	7620.00
	28	28	5	C.Y.	127.00	3585.27
Base	36	36	16	C.Y.	127.00	24384.00
	28	28	16	C.Y.	127.00	11472.86
Sub	36	36	3	C.Y.	127.00	4572.00
	28	28	3	C.Y.	127.00	2151.16
					Total ($)	278877.37
					Total Cost with Waste ($)	292821.23

Table 40 – Concrete Column Takeoff by Level

Columns - Reinforcing Steel						
Weight (lb)	**Unit**	**Crew**	**Daily Output**	**Labor Hours**	**Bare Total ($)**	**Cost ($)**
913805	Ton	4 Rodm	2.3	13.913	1535	701345.34
					Total Cost with Waste ($)	736412.60

Table 41 – Column Reinforcing Steel Takeoff

Slabs and Drops - 6000 psi Concrete							
Level	Slab Area (ft2)	Slab Thickness (in)	Drop Area (ft2)	Drop Thickness (in)	Unit	Bare Total ($)	Cost ($)
Roof	22822	11	575	3.5	C.Y.	144.00	112467.44
9	37706	11	963	3.5	C.Y.	144.00	185835.22
8	44155	11	1113	3.5	C.Y.	144.00	217600.06
7	44155	11	1113	3.5	C.Y.	144.00	217600.06
6	44155	11	1113	3.5	C.Y.	144.00	217600.06
5	46636	11	1175	3.5	C.Y.	144.00	229824.78
4	48620	11	1225	3.5	C.Y.	144.00	239604.56
3	48620	11	1225	3.5	C.Y.	144.00	239603.33
2	46636	11	1175	3.5	C.Y.	144.00	229824.78
1	7938	11	200	3.5	C.Y.	144.00	39119.11
Mech	31752	11	800	3.5	C.Y.	144.00	156476.44
Base	13892	11	350	3.5	C.Y.	144.00	68458.44
						Total Cost ($)	2154014.28

Table 42 – Concrete Slab and Drop Panel Takeoff by Level

Note: For brevity, not all tables were included. Contact author to view remaining tables.

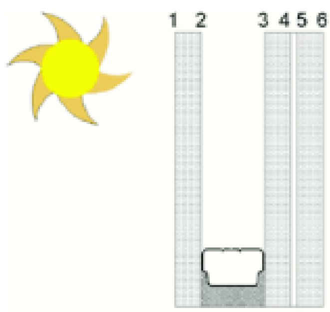
Appendix J: Mechanical Breadth

VIRACON

Low-E (VE) Insulating Laminated Glass

VE 6-42

Make-Up



Notes:
1/4" (6mm) Blue-Green VE-42 #2
1/2" (13.2mm) airspace
1/4" (6mm) clear
.030" (.76mm) PVB
1/4" (6mm) clear

Performance Data

Product Code	VE 6-42
Transmittance	
Visible Light	31%
Solar Energy	15%
Ultra-Violet*	<1%
Reflectance	
Visible Light-Exterior	15%
Visible Light-Interior	13%
Solar Energy	11%
NFRC U-Value	
Winter	0.31 Btu/(hr x sqft x °F)
Summer	0.28 Btu/(hr x sqft x °F)
European U-Value	1.6
Shading Coefficient	0.28
Relative Heat Gain	60 Btu/(hr x sqft)
Solar Factor (SHGC)	0.24
LSG	1.28

*Ultra-violet defined as 300 to 380 nanometers (nm)