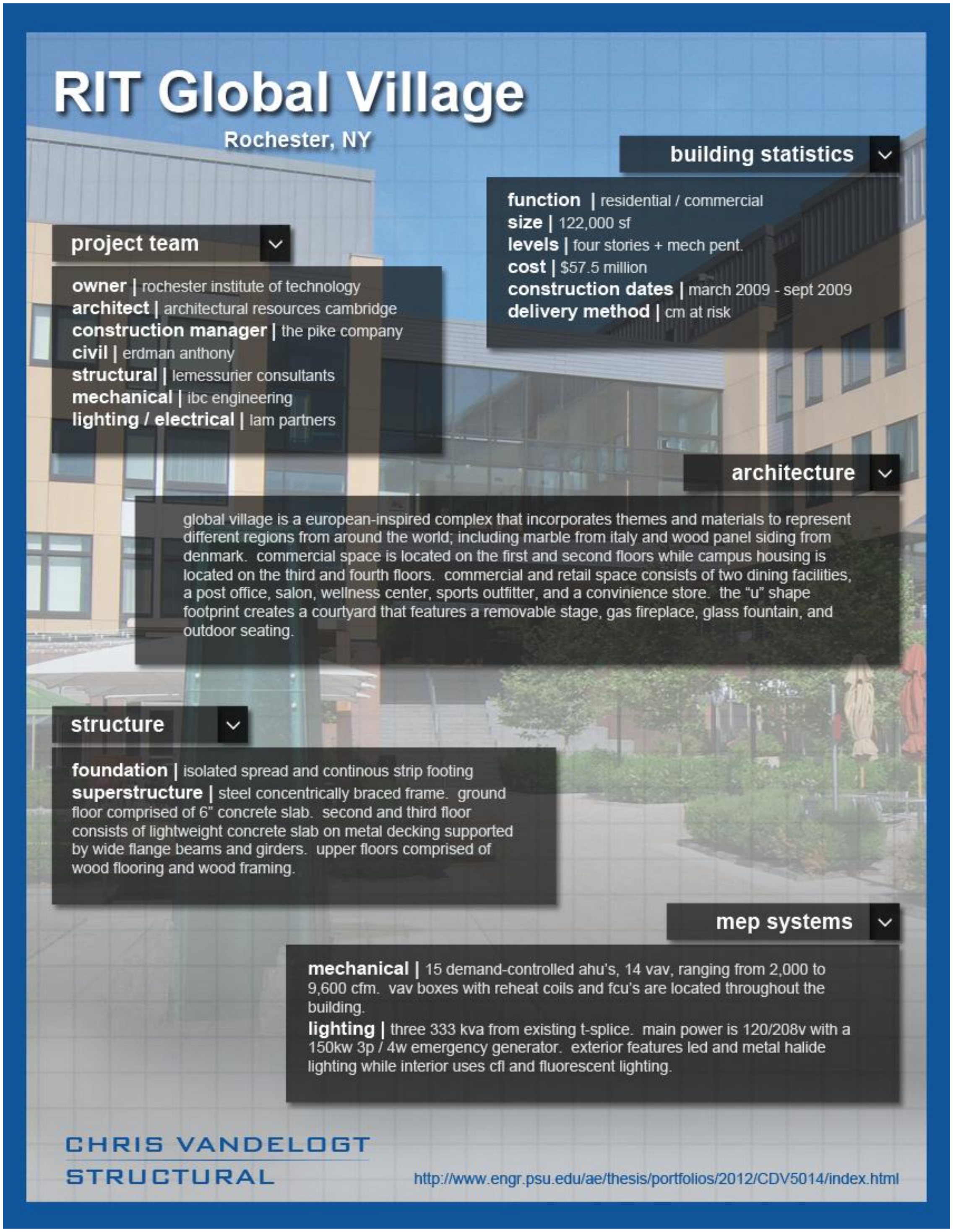
Final Report

CHRIS VANDELOGT | Structural Option

Global Village
Rochester Institute of Technology
The Pennsylvania State University
Faculty Advisor: Dr. Hanagan

4/4/2012


RIT Global Village
Rochester, NY
building statistics
function | residential / commercial
size | 122,000 sf
levels | four stories + mech pent.
cost | $57.5 million
construction dates | march 2009 - sept 2009
delivery method | cm at risk
project team
owner | rochester institute of technology
architect | architectural resources cambridge
construction manager | the pike company
civil | erdman anthony
structural | lemessurier consultants
mechanical | ibc engineering
lighting / electrical | lam partners
architecture
global village is a european-inspired complex that incorporates themes and materials to represent different regions from around the world; including marble from italy and wood panel siding from denmark. commercial space is located on the first and second floors while campus housing is located on the third and fourth floors. commercial and retail space consists of two dining facilities, a post office, salon, wellness center, sports outfitter, and a convinience store. the "u" shape footprint creates a courtyard that features a removable stage, gas fireplace, glass fountain, and outdoor seating.
structure
foundation | isolated spread and continous strip footing
superstructure | steel concentrically braced frame. ground floor comprised of 6" concrete slab. second and third floor consists of lightweight concrete slab on metal decking supported by wide flange beams and girders. upper floors comprised of wood flooring and wood framing.
mep systems
mechanical | 15 demand-controlled ahu's, 14 vav, ranging from 2,000 to 9,600 cfm. vav boxes with reheat coils and fcu's are located throughout the building.
lighting | three 333 kva from existing t-splice. main power is 120/208v with a 150kw 3p / 4w emergency generator. exterior features led and metal halide lighting while interior uses cfl and fluorescent lighting.
CHRIS VANDELOGT
STRUCTURAL
http://www.engr.psu.edu/ae/thesis/portfolios/2012/CDV5014/index.html

Executive Summary

Global Village is a European-inspired complex that provides commercial and residential space for the campus at the Rochester Institute of Technology in Rochester, NY. Each location has been designed to incorporate themes and materials that represent different regions from around the world, including marble from Italy and wood siding from Denmark. Global Village is a four-story building that also supports a fifth story dedicated to mechanical equipment; making it rise to an overall height of 62.5 feet. The building is constructed of steel with metal deck and lightweight concrete at the first, second, and third floors while the other floors have wood framing. The building's main lateral-resisting system consists of concentrically braced frames in both directions.

This report focusses on altering the existing dual structural system to a more uniform system. Concrete was chosen as the main material since most on-campus residential buildings are constructed of either concrete or masonry. A reinforced concrete flat plate was then selected for the gravity system due to its flexibility to work around the floor plan. Columns were placed as best as possible to avoid altering the floor plan. However, some interior columns interfered with the fan coil unit areas located on the third floor and thus the fan coil units had to be relocated. A new floor plan for the second floor was also designed as a result of the new column layout.

After the column layout was finalized, column sizes were found using hand calculations and verified using spColumn. The size of the column was mainly dependent on the unbalanced moment transferred by eccentricity of shear. Multiple slab thicknesses and column sizes were tried and a 20" by 20" column with (8) #10 bars was determined to be adequate. A slab thickness was then found using Table 9.5c of ACI 318-08. The table gave a minimum slab thickness of 8.25" but since deflection checks were inadequate, the slab thickness was increased to 8.5". In order to calculate the required reinforcement due to gravity loads, a spreadsheet following the direct design procedure was created. The spreadsheet was also used to design the reinforcement for the moment connections.

To analyze the proposed buildings lateral system, a model was built in ETABS and was used to check story drift and to find column moments in order to design the moment connections. These moments were input into the unbalanced moment section of the spreadsheet and the reinforcement was designed. The maximum drifts in both the N-S and E-W Directions were controlled by loads due to seismic. The total drift from ETABS in the N-S Direction is 1.751" and 1.488" in the E-W Direction; which are well below the allowed 10.441". As a note, a maximum total drift of 1.696" caused by wind in the N-S Direction is below the allowable 1.740". As a result, the lateral system is adequate for drift.

As a result of using concrete as the main structural material, many areas in construction and building serviceability are improved. The use of concrete provides a more durable building and improves the fire rating. A drawback of using concrete is that the cost of the proposed building is more than triple the cost of the existing building. RSMeans was used to calculate the cost of each system and it was found that the proposed structure costs $1,826,436 where the existing design was calculated to cost $571,588.

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Acknowledgments

I would specifically like to thank the following people for their support and guidance throughout the completion of this report. I would also like to thank all the professionals and the entire AE faculty at The Pennsylvania State University that have given me the knowledge to become a structural engineer over the past five years. Aside from my education, I would like to thank my family and friends for their support and good times.

Professionals

Rochester Institute of Technology:

- James Yarrington: *Director, Campus Planning and Design & Construction Services*
- Ted Weymouth: *Sr. Project Manager*

The Pennsylvania State University:

- David Manoz: *Assistant Director of Housing*

AE Faculty

- Dr. Linda Hanagan
- Dr. Ali Memari
- Dr. Andrés Lepage
- Professor Robert Holland
- Professor M. Kevin Parfitt

Purpose

The purpose of this report is to alter the existing dual steel-wood structural system to a uniform structural system. This report will detail the design of the gravity and lateral systems of the proposed structure and provide checks for adequacy. A comparison of the proposed structure to the existing structure will be accomplished through an architecture breadth and also through a construction management breadth.

Introduction




Global Village is a mixed-use building that provides commercial and residential space for the campus at RIT. Global Village has achieved LEED Gold certification and has been designed to be community friendly. In total, the Global Village project provides 414 beds for on campus living and 24,000 square feet of commercial and retail space.

The $57.5 million dollar project consists of three independent structures on the campus at RIT. The main four-story Global Village building (Building 400) is 122,000 square feet and the two additional three-story Global Way buildings (Buildings 403 and 404) are 32,000 square feet each. The main project team includes RIT as the owner, Architectural Resources Cambridge as the architect, and The Pike Company as the CM-at-Risk. Eleven other firms were also employed to handle MEP, lighting, acoustics, and so forth.

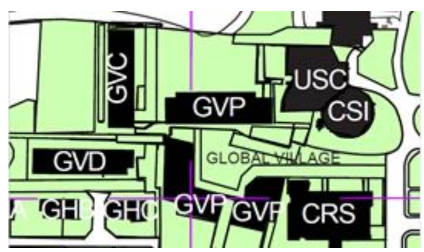


Figure 1: GVP is Building 400 (Global Village Building). GVC and GVD are Buildings 403 and 404 (Global Way Buildings). Courtesy of RIT.

Commercial space is located on the first and second floors, which consist of two dining facilities, a post office, salon, wellness center, sports outfitter, and a convenience store. Campus housing is located on the third and fourth floor which provides room for 210 beds. There is also a fifth floor; however, it is used primarily as a mechanical penthouse. Building 400's unique "U" shape creates a courtyard that features a removable stage, gas fireplace, and a glass fountain. See Figure 1 for a campus map of the Global Village complex. The area also includes outdoor seating with tables equipped with umbrellas.

The 28,000 square foot courtyard is also heated to extend its use during the winter and to minimize winter maintenance.

The façade of Building 400 is made up of a cement fiber board rain screen, brick masonry veneer, and flat seamed sheet metal with aluminum clad wood windows, and a coated extruded aluminum storefront.

Global Village Building 400 is a LEED Gold Certified Building. Green aspects include a green roof above the restaurant, daylight sensor lighting, and sensors to shut off mechanical equipment when windows are opened. Global Village is located on a sustainable site that is walk-able and transit oriented, encourages low-emitting vehicles, and reflects solar heat. The building reduces water consumption through water efficient landscaping and technologies such as high-efficiency toilets, faucets, and shower heads. Through the implementation of several energy efficient systems, the building is predicted to use 29.4% less energy. To encourage sustainable energy, seventy percent of the building's electricity consumption is provided from renewable sources (wind) through the engagement in a two-year renewable energy contract. Construction of Global Village included waste management recycling, air quality control, and low emitting materials. Along with regional materials, recycled content were also installed that constitute 20% of the total value of the materials in the project.

Global Village is a part of RIT's campus outreach program. The buildings not only provide student housing and retail space, but were also designed to be community friendly and to provide students with a global living experience. Global Village is LEED Gold certified and the courtyard created promotes outdoor activity.

Existing Structural Overview

The structure of Global Village Building 400 consists of steel and wood framing on a concrete foundation wall. The first, second, and third floor slabs use a lightweight concrete on metal decking system while the fourth floor, mechanical penthouse, and roof use wood framing. The lateral system consists of concentrically braced frames in both directions.

Foundation

In January 2009, Tierney Geotechnical Engineering, PC (TGE) provided a subsurface exploration and geotechnical investigation for Global Village. TGE performed 14 test borings and 2 test pits on the site of Building 400 and recommended foundation types and allowable bearing pressures along with seismic, floor slab, and lateral earth pressure design parameters.

In general, the borings and test pits encountered up to 8 inches of topsoil at the ground surface, or fill. The fill, generally consists of varying amounts of silt, sand, and gravel. At several locations, the fill also contained varying amounts of construction-type debris and deleterious material such as asphalt, topsoil, and wood. The fill was generally encountered to depths of approximately 4 to 8 feet. Below the fill, native soils with a very high compactness were encountered. Overall, most of the structure's foundation is on very compact glacial fill.

From these results, it was determined that the structure may then be supported on a foundation system consisting of isolated spread and continuous strip footings. TGE recommends an allowable bearing pressure of 7,500 psf to be used in the foundation design. It was also recommended by TGE that, due to lateral earth pressure, retaining walls are to be backfilled to a minimum distance of 2 feet behind the walls with an imported structural fill. To prevent storm run-off, permanent drains should also be installed behind all retaining walls.

Floor System

The first floor consists of a 6" concrete on grade slab. For the second and third floors, the floor system is comprised of 3¼" lightweight concrete slab on 3" composite metal (18-gage) decking. Individual steel deck panels are to be continuous over two or more spans except where limited by the structural steel layout. The rest of the floors are made up of wood framing with ¾" plywood sheathing. Shear stud connectors are welded to beams and girders where appropriate. See Figure 2 for details.

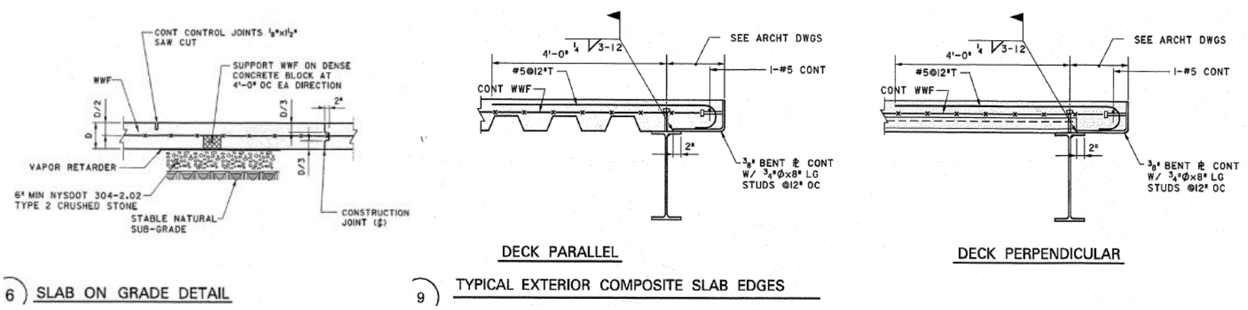


Figure 2: Typical composite slab details. Courtesy of RIT. Drawings not to scale.

Framing System

The framing grid that Global Village possesses is very unique and very complicated. The bay sizes on each floor vary dramatically and the beams don't line up on each side of the transfer girders. The framing is also not consistent between floors. There is no simple consistent grid except for a couple areas highlighted in Figure 3. In these highlighted areas, the beams vary from W18x35 to W16x31 while the transfer girders vary from W14x22 to W21x44. Column sizes also vary significantly throughout the structure where the majority is in between W10x54 to W12x106.

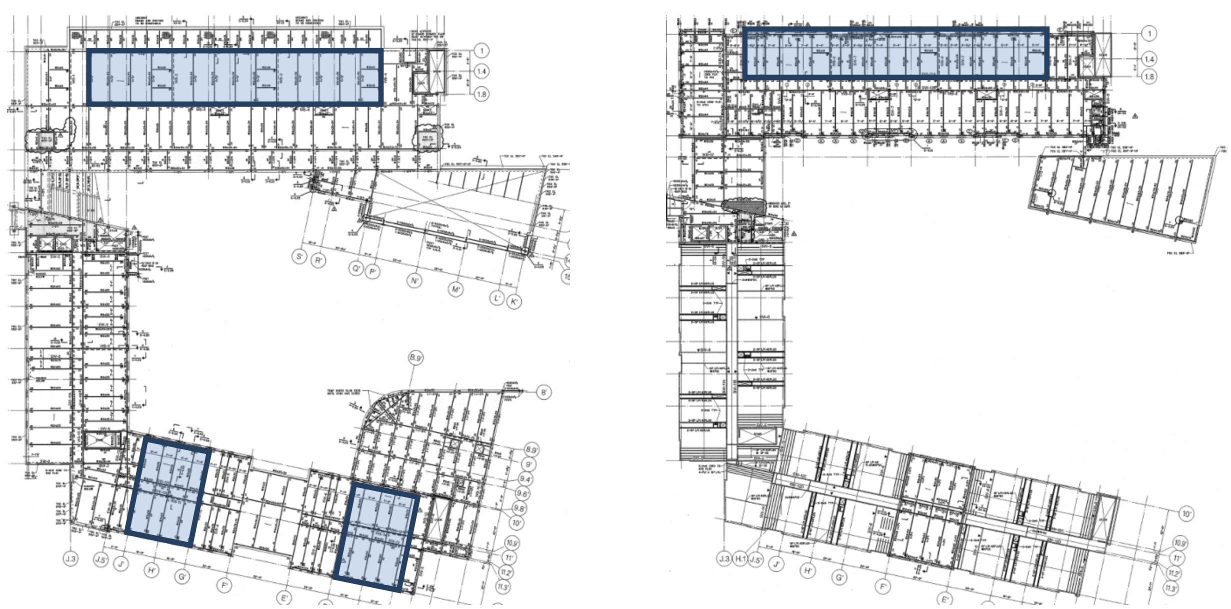


Figure 3: 2nd Floor (left) and 3rd Floor (right) framing plans. Typical bays on each level highlighted. Courtesy of RIT. Drawings not to scale.

Lateral System

The lateral load resisting system consists of concentrically braced frames and wood shear walls, each acting on separate floors. Braced frames are used between the ground and the third floor while shear walls are placed on the third, fourth, and fifth (penthouse) floors.

The lateral HSS bracing ranges in size where the majority is HSS7x7x½. See **Figure 4** for details and placements of the braced framing used on the second floor. The shear walls are made of wood blocking, consisting of 2x4's, and sheathing. These wood shear walls are used due to the use of wood structuring above the third floor. For placements and details, see **Figure 5**.

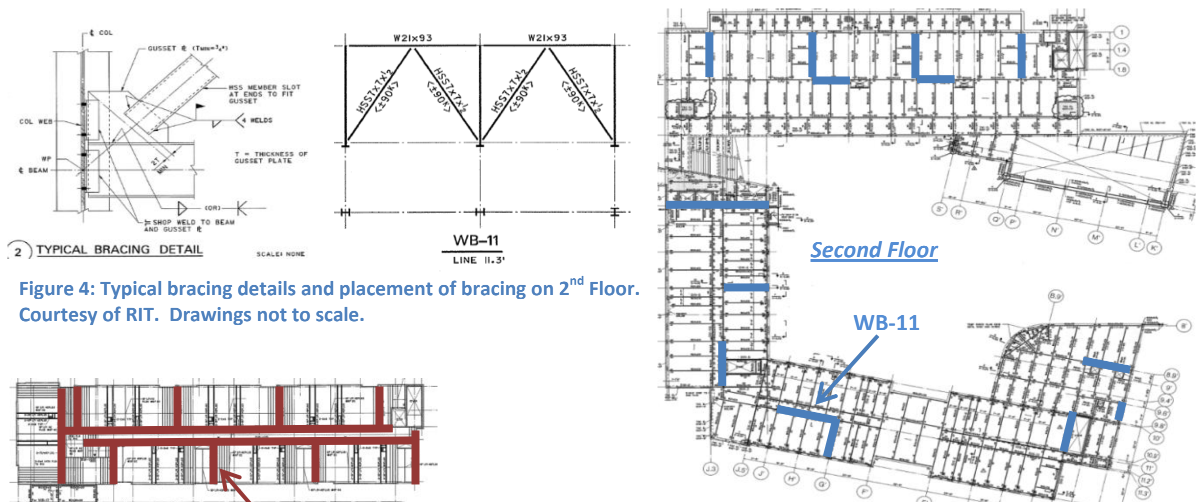


Figure 4: Typical bracing details and placement of bracing on 2nd Floor. Courtesy of RIT. Drawings not to scale.

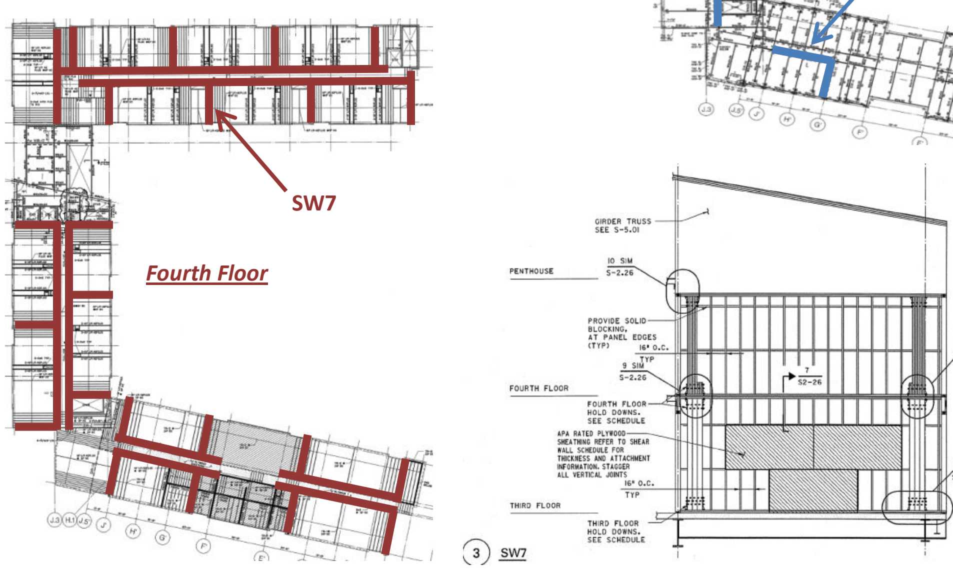


Figure 5: Typical shear wall details and placements used on the 4th Floor. Courtesy of RIT. Drawings not to scale.

Load Path and Distribution

As the façade collects the forces due to wind, they are transferred to the slabs of the building. The slab forces are then transferred to the braced frames that run parallel to the load. As shown in **Figure 6**, this load is then resisted by the beam and HSS cross bracing. The blue arrow represents the lateral load acting on the braced frame while the red arrows show the load within the members.

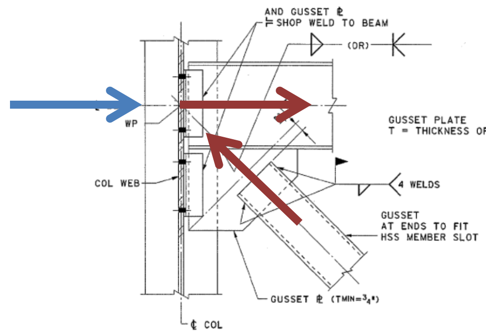


Figure 6: Lateral load path through a HSS cross braced connection. Courtesy of RIT.

Seismic loads originate from the mass of the structure itself. These loads are created predominantly from the slabs of the structure. When seismic loads are created by ground motion, the braced frames incur the forces from the slabs and transfer them to the foundation and thus to grade.

Design Codes

Below is a list of codes and standards that the design team used on Global Village. As a comparison, codes and standards used for this report are given.

Design Codes

Design Codes:

- American Concrete Institute (ACI) 318-99, Building Code Requirements for Reinforced Concrete
- American Concrete Institute (ACI) 301-99, Specifications for Structural Concrete for Buildings
- ACI Detailing Manual-1994 (SP-66)
- CRSI Manual of Standard Practice (MSP 1-97)
- Structural Welding Code – Reinforced Steel (AWS DI.4-92)
- Code of Standard Practice for Steel Buildings & Bridges (AISC 1992)
- Part II published in the Timber Construction Manual (AITC 4^{th} Edition)
- National Design Specification for Wood Construction (NF.PA, 1991 Edition)

Model Codes:

- 2007 Building Code of New York State / 2003 International Building Code
- 2007 Fire Code of New York State / 2003 International Fire Code
- Accessibility: BCNY Chapter 11, 2003 ICC/ANSI 117.1
- Electrical Code of New York, NFPA 70 2005
- 2007 Mechanical Code of New York State / 2003 International Mechanical Code
- 2007 Plumbing Code of New York State / 2003 International Plumbing Code

Standards:

- American Society of Civil Engineers (ASCE) 7-02, Minimum Design Loads for buildings and Other Structures

Thesis Codes

Design Codes:

- AISC Steel Construction Manual, 14^{th} Edition
- American Concrete Institute (ACI) 318-08, Building Code Requirements for Structural Concrete

Standards:

- American Society of Civil Engineers (ASCE) 7-10, Minimum Design Loads for buildings and Other Structures

Material Properties

Listed below are materials and their strengths used in Global Village. These material strengths are followed best as possible in this report.

Steel

Unless Noted Otherwise	F_y = 50 ksi (A992 or A588 Grade 50)
Where Noted by (*) on Drawings	F_y = 36 ksi (A36)
Square and Rectangular HSS (Tubes)	F_y = 46 ksi (A500 Grade B)
Round HSS (Pipes)	F_y = 46 ksi (A500 Grade C)
Anchor Bolts (Unless Noted Otherwise)	F_y = 36 ksi (F1554)
High Strength Bolts (Unless Noted Otherwise)	F_u = 105 ksi (A325)
Metal Deck	F_y = 33 ksi (A653)
Weld Strength	F_y = 70 ksi (E70XX)

Concrete

Slabs-on-Grade	4000 psi (Normal Weight)
Walls, Piers	4000 psi (Normal Weight)
Concrete on Steel Deck	3000 psi (Light Weight)
Topping Slabs & Housekeeping Pads	3000 psi (Normal Weight)

Other

Bars, Ties, and Stirrups	60 ksi
Masonry	F'_m = 3000 psi
Wood	F_b = 1000 psi (Bending Stress)
	F_v = 70 psi (Shear Stress)

* Material strengths are based on American Society for Testing and Materials (ASTM) standard rating

* Other wood strengths are given in the structural drawings

Simplifications

For the purposes of this report, only the north leg of Global Village will be analyzed, see Figure 7. Reasoning behind this decision was due to greater wind and seismic loads acting on this section of the building as found in previous technical reports.

Due to the unique shape of the first floor, the building in this report will be dimensioned as the second and upper floor dimensions used in the existing building. The full story grade level change on either side of the building is also neglected and both sides are assumed to be exposed to lateral loads.

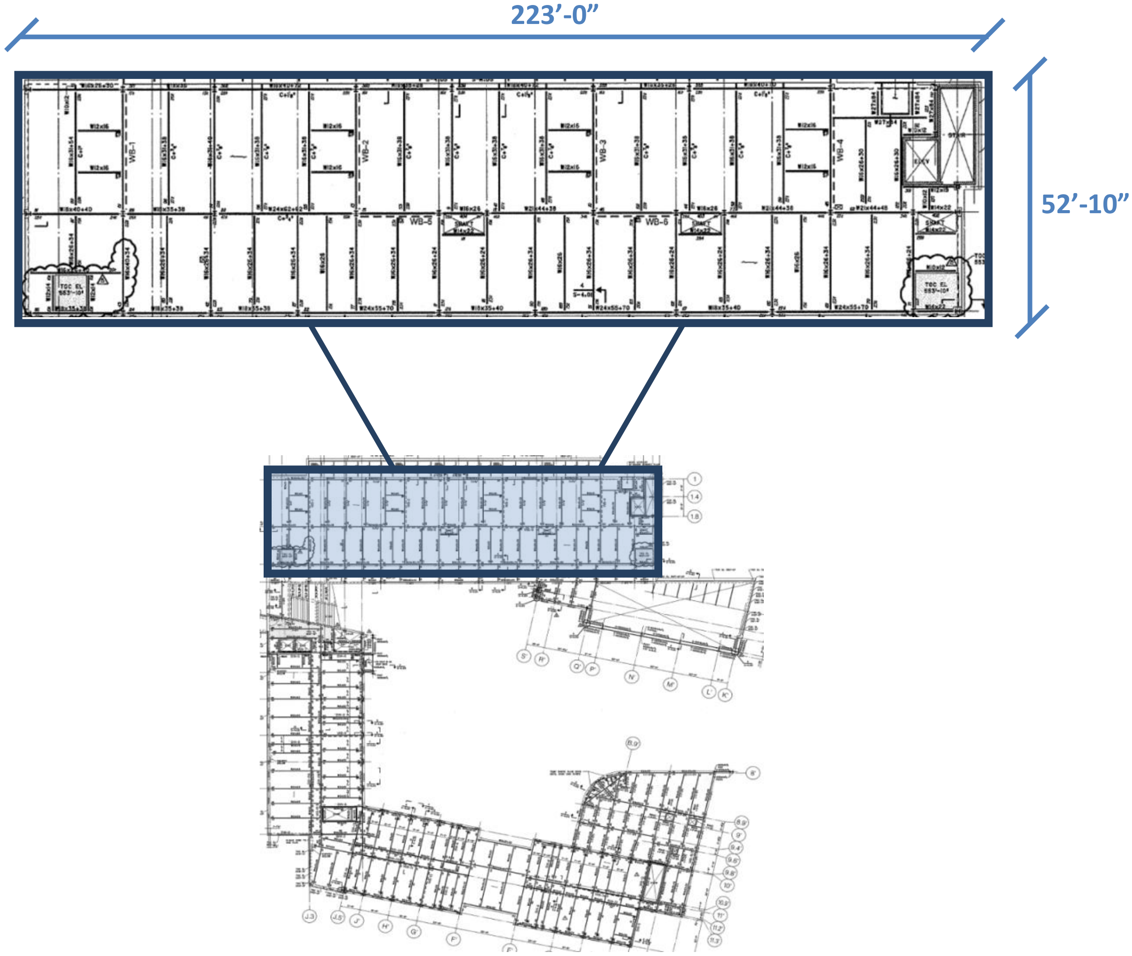


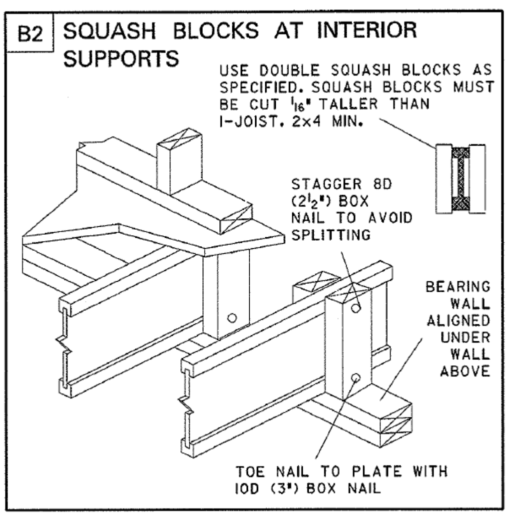
Figure 7: Expanded view of the 2nd Floor of the north leg. Courtesy of RIT. Drawings not to scale.

Problem Statement

As mentioned above, Global Village consists of two different structural systems. A steel frame is used between the ground and third floor while wood framing is used on the third and fourth floor, mechanical penthouse, and roof. The use of different structural materials within the building is very complex and is very complicated to design. Not only does the designer have to have an extensive knowledge of both wood and steel design, the designer must also consider the connection between the steel and wood. An outside firm may have to be contacted to design or analyze the connections, which in turn requires more communication, time, and money.

Using different structural materials also has an impact on how the lateral system is designed. In order to accommodate the lateral loads, this building has two types of lateral systems. Concentrically braced frames are used on the bottom floors where steel is used. These braced frames rise to the third floor where wood shear walls are then used on the floors above. The wood shear walls are made up of 2x4's similar to shear walls used in residential structures.

In terms of construction, different materials require more coordination from the construction manager. Additional contractors may also have to be hired for their knowledge of structural wood construction. Figure 8 shows the complexity of typical wood sections. This impacts the schedule and cost of the project which are significant for university buildings.



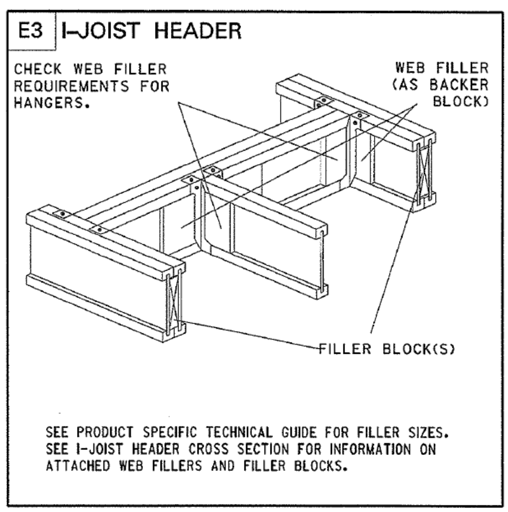


Figure 8: Typical wood sections and details. Courtesy of RIT.

Proposed Solution

To speed up the design and construction process, it is proposed to use reinforced concrete throughout the entire structure. Replacing the steel-wood framing system with an entire concrete system will minimize extra considerations that the existing dual structure creates. By using a uniform structural system, the additional firms and contractors that are needed in the design and construction of the dual system can be eliminated. This saves time and improves communication throughout the entire project.

To structure the proposed building, a flat plate system will be used. To accommodate lateral loads, moment connections will be assessed in ETABS. Breadth topics will then be completed to compare the existing to the proposed building.

Breadth Topics

Breadth topics are used to compare the existing building to the proposed building. A construction management breath will examine the constructability of re-design and address the predominant use of concrete and masonry in university buildings. An architecture breadth will also be completed to analyze any changes that the proposed building creates.

Construction Management

The purpose of the construction management breadth is to assess the constructability of re-design. A study will be completed as to why most university buildings are constructed of concrete and/or masonry. This will involve contacting professionals at the Office of Physical Plant at Penn State. Professionals at RIT will also be contacted in order to determine the use of steel and wood in Global Village.

The information found will be used to compare the proposed building to the existing building in terms of constructability. This entails general reality checks and examining any improvements in construction methods, safety, or use of recycled materials. A reduction of field labor will also be checked.

Architecture

Designing the proposed building could have several impacts on the architecture of the building. The use of wood creates a more flexible floor plan than concrete. This is due to wood frames using load bearing walls instead of columns used in concrete structures. In a concrete system, the column placement affects the bay size which in turn affects the floor plan. Columns may also create an aesthetically unpleasing effect if not appropriately incorporated into the theme of the space. Therefore, the column layout will need to consider the current floor plan and appearance of the space.

Adjustments to the floor plan and appearance of the existing building will be analyzed using Revit. Renders of newly designed spaces considering column placements will be completed for the proposed building.

Gravity Loads

Dead, live, and snow loads were found primarily through the use of the AISC Steel Construction Manual and ASCE Standard 7-10. These loads were then compared to the loads used by the design team for consistency.

Dead and Live Loads

Although the structural drawings only gave a typical floor partition allowance of 20 psf as a dead load, dead loads were found or assumed by using the AISC Steel Construction Manual and textbooks on structural design. For a summary of assumed superimposed dead loads used, see Table 1.

Superimposed Dead Loads	
Description	**Load (psf)**
Superimposed DL	5
MEP Allowance	10
Partitions	15
Acoustical Ceiling	5
Slab (8½") Self Weight	106
Roofing	18

Table 1: Summary of superimposed dead loads

Most live loads, however, were provided in the structural drawings. These loads were compared to live loads found using Table 4-1 in ASCE 7-10 based on the usage of the spaces. The results are given in Table 2. Most live loads found match designer loads except for fan and mechanical equipment room loadings. Since these were not able to be found in ASCE 07-10, the loads were taken from the design team to be consistent.

Live Loads			
Space	**Design Live Load (psf)**	**Live Load Used (psf)**	**Reference**
Lobbies and Common Areas	100	100	ASCE 7-10: Residential
1st Floor Corridors	100	100	ASCE 7-10: Schools
Typical Floors	40	40	ASCE 7-10: Residential
Stairways	100	100	ASCE 7-10: Stairways
Fan Room	80	80	Assumed
Mechanical Equipment Rooms	150	150	Assumed
Mechanical Floor Walkways	---	30	ASCE 7-10: Residential - Attics
Roof Live Load	---	20	ASCE 7-10: Roofs

Table 2: Comparison of design live loads and live loads used

Snow Loads

The roof snow load was calculated in accordance to Chapter 7 of ASCE 7-10. The factors used to find the roof snow load can be found in **Table 3**. Using the flat roof procedure, the roof snow load was determined to be 30.8 psf where the snow load used by the design team was 39 psf. Since the factors used here match the factors listed on the structural drawings, the difference must be the equation used to calculate the flat roof snow load. On the structural sheet, the flat roof snow load procedure was used but in accordance with the "2007 Building Code of New York State." Therefore, it may be valid that the equations used to calculate roof snow load differ between ASCE 7-10 and the 2007 Building Code of New York State.

Flat Roof Snow Calculations	
Variable	**Value**
Ground Snow Load, p_g (psf)	40
Exposure Factor, C_e	1.0
Thermal Factor, C_t	1.0
Importance Factor, I_s	1.1
Flat Roof Snow Load, p_f (psf)	30.8

Table 3: Snow load factors

Lateral Loads

In order to analyze the lateral system of Global Village, wind and seismic loads were calculated for this report. Wind loads were calculated using the MFRS (Directional) Procedure and seismic loads were calculated using the Equivalent Lateral Force Procedure outlined in ASCE 7-10. A summary of the story forces for both wind and seismic can be found at the end of this section.

Wind Loads

Winds loads were calculated using the Main Wind-Force Resisting System (Directional Procedure) outlined in Chapter 26 and 27 of ASCE 7-10. Global Village was found to be categorized as a Type III Occupancy and Exposure Category C. General building dimensions, constants used, and calculation of gust factors for the direction normal to the long dimension (length) are given in **Table 4**. General building dimensions, constants used, and calculation of gust factors for the direction normal to the short dimension (width) are given in **Table 6**.

Calculations were done on Microsoft Excel to reduce calculation errors and save time. The wind pressure calculations in the long dimension are given in **Table 5**. The wind pressure calculations in the short dimension are given in **Table 7**. A summary of the wind pressures calculated in both directions can be found in **Figure 9**. As a note, internal pressure was not included in the calculations because internal pressure can be considered self-cancelling unless there are large openings in the structure.

The structural sheets provide values to which the designer used but no overall base shear or wind pressures. The calculated values are similar to the values used in design except the designer's Basic Wind Speed is 90 mph where the value that was calculated was 120 mph. This is due to the different versions of ASCE 07. The designers used ASCE 7-02 where the values calculated for this report were from ASCE 7-10.

Normal to Long Dimension (Length)

Building Dimensions		
Length (ft)	Width (ft)	Height (ft)
223.0	52.8	62.5

Constants			
V (mph) =	120	$C_{p,leeward}$ =	-0.5
k_d =	0.9	$C_{p,sides}$ =	-0.7
k_{zt} =	1.0	$C_{p,roof:<h/2}$ =	-1.3
$C_{p,windward}$ =	0.8	$C_{p,roof:>h/2}$ =	-0.7

Gust Factor Calculations				
z_{bar}	I_{zbar}	L_{zbar}	Q	G
37.50	0.196	512.95	0.84	0.84

Table 4: Building dimensions, constants, and gust factors

Floor	Height	k_z	q_z (lb/ft^2)	p_{wind} (lb/ft^2)	p_{lee} (lb/ft^2)	p_{side} (lb/ft^2)	$p_{roof<h/2}$ (lb/ft^2)	$p_{roof>h/2}$ (lb/ft^2)
2nd	14.0	0.850	26.634	17.98	-15.07	-21.10		
3rd	26.6	0.953	29.862	20.16	-15.07	-21.10		
4th	37.3	1.024	32.086	21.66	-15.07	-21.10		
Pent	48.0	1.080	33.841	22.84	-15.07	-21.10		
Roof	62.5	1.140	35.721	24.11	-15.07	-21.10	-39.18	-21.10

Table 5: Wind pressure loads normal to long dimension

Normal to Short Dimension (Width)

Building Dimensions		
Length (ft)	**Width (ft)**	**Height (ft)**
223.0	52.8	62.5

Constants			
V (mph) =	120	$C_{p,leeward}$ =	-0.5
k_d =	0.9	$C_{p,sides}$ =	-0.7
k_{zt} =	1.0	$C_{p,roof:<h/2}$ =	-1.3
$C_{p,windward}$ =	0.8	$C_{p,roof:>h/2}$ =	-0.7

Gust Factor Calculations				
z_{bar}	I_{zbar}	L_{zbar}	Q	G
37.50	0.196	512.95	0.90	0.87

Table 6: Building dimensions, constants, and gust factors

Floor	Height	k_z	q_z (lb/ft^2)	p_{wind} (lb/ft^2)	p_{lee} (lb/ft^2)	p_{side} (lb/ft^2)
2nd	14.0	0.850	26.634	18.62	-15.61	-21.85
3rd	26.6	0.953	29.862	20.88	-15.61	-21.85
4th	37.3	1.024	32.086	22.43	-15.61	-21.85
Pent	48.0	1.080	33.841	23.66	-15.61	-21.85
Roof	62.5	1.140	35.721	24.97	-15.61	-21.85

Table 7: Wind pressure loads normal to short dimension

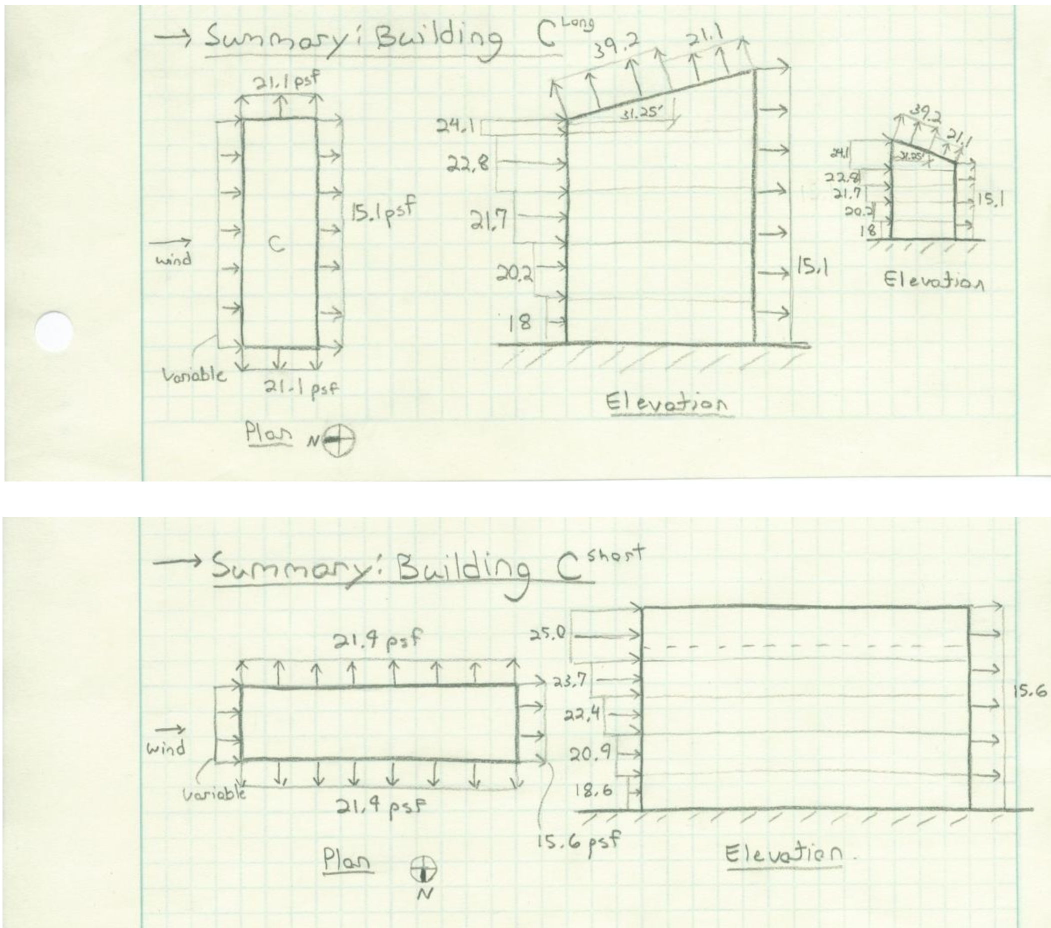


Figure 9: Summary of wind pressures

Seismic Loads

Seismic Loads were calculated using the Equivalent Lateral Force Procedure outlined in Chapters 11 and 12 of ASCE 7-10. While performing the procedure, many seismic values were found which are noted in Table 8. Concrete moment connections in both directions were chosen as the proposed building's lateral system. This corresponds to a Response Modification Coefficient value of 3. Spectral Response Acceleration values were taken directly from the USGS website instead of using the ASCE maps to provide a more accurate result.

The structural drawings give a list of values that the design team used. Comparing these with the values calculated; it was found that all values were exact except for the Response Modification Coefficient.

The weight of each floor was then computed using the dead loads listed in the gravity loads section of this report. As a note, 20 percent of the flat roof snow load and the full mechanical room live load were added per section 12.7.2 of ASCE 7-10. See Table 9 for calculations and Figure 10 for a summary of forces acting on the building.

Seismic Variable	Value	Reference	Drawings
I_e	1.25	Table 1.5-2	-
S_S	.21	USGS Website	.21
S_1	.06	USGS Website	.06
Site Class	C	Geotechnical Report	C
Occupancy Category	III	Table 1.5-1	-
S_{DS}	.168	Table 11.6-1	.17
S_{D1}	.068	Table 11.6-2	.06
Seismic Category	B	Table 11.6-1	B
R	3.0	Table 12.2-1	5.0
T_L	6 sec	Figure 22-12	-
C_t	.02	Table 12.8-2	-
x	.75	Table 12.8-2	-
T_a	.445 sec		-
T	.7565 sec		-
C_s	.038	Equation 12.8-2	.038

Table 8: Seismic values

Floor	Floor Weight, w_x (k)	Story Height, h_x (ft)	$w_x h_x^k$	C_{vx}	Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)
Ground	2345	0.0	0.00	0.00	0.0	361.1	0.0
2nd	1760	14.0	89799.12	0.09	33.8	361.1	473.2
3rd	1760	26.7	185685.30	0.19	69.9	327.3	1863.3
4th	1698	37.3	260630.10	0.27	98.1	257.4	3662.1
Pent	1735	48.0	354584.69	0.37	133.5	159.3	6406.4
Roof	335	58.0	68682.22	0.07	25.9	25.9	1499.4
Sum:	9632		959381.4	1.00	361.1		
				√ ok	√ ok		
	Base Shear ($V=C_sW$) =		361	**Total Overturning Moment =**			13904

Table 9: Seismic calculations

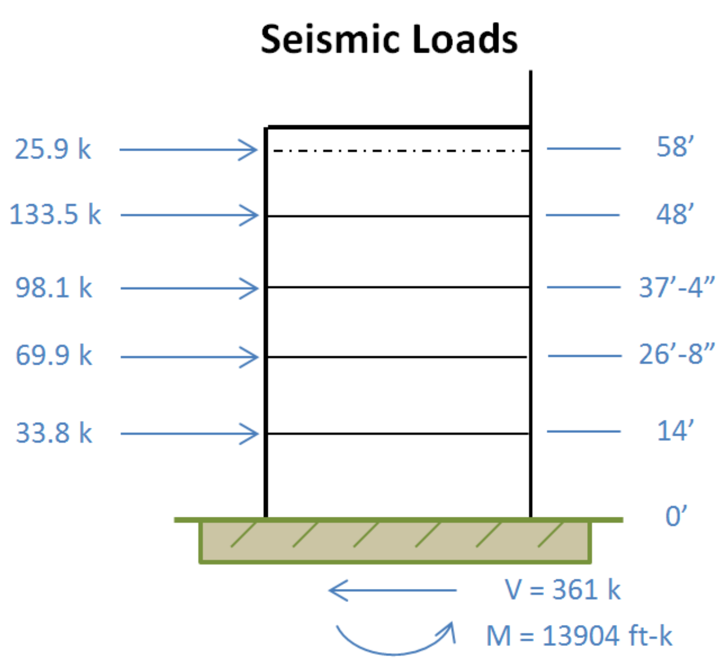


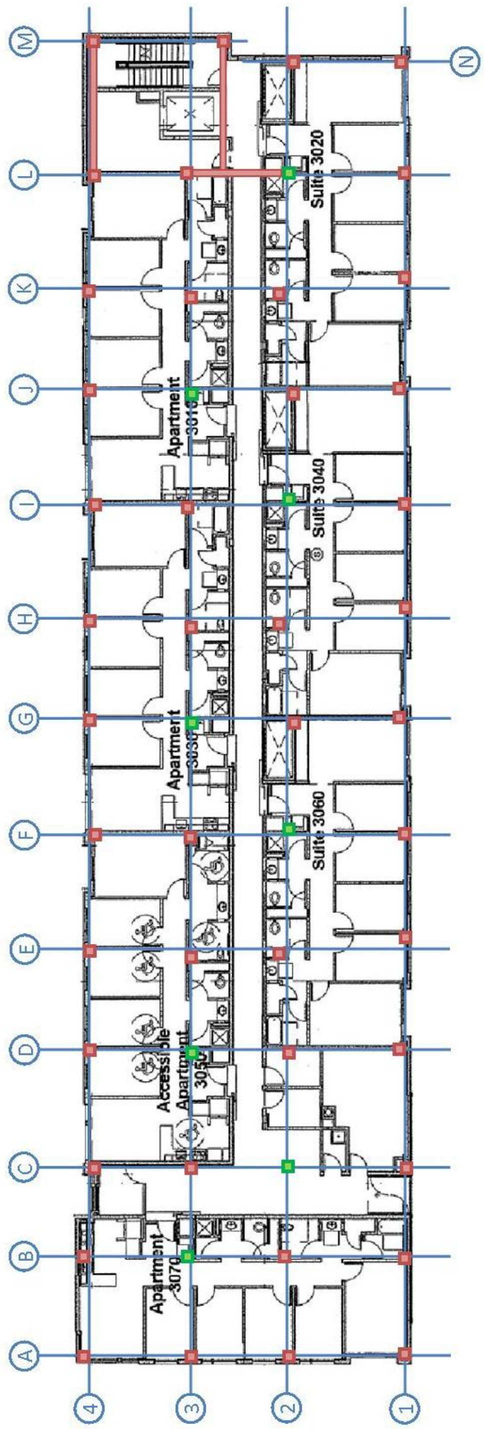
Figure 10: Summary of seismic loading

Column Layout

As stated in the structural overview, the existing building uses a steel frame between the ground and third floor while wood framing is used on the third and fourth floor, mechanical penthouse, and roof. The use of wood framing on the residential third and fourth floors creates a more flexible floor plan which needs to be considered when creating the column layout for the proposed structure. The steel framing in the existing building primarily affects the second floor plan which is just academic fit-out space and can easily be adjusted. Therefore, the column layout needs to work around the third and fourth floor plans.

In order to use a flat plate structural system, the efficient bay width needed to be in between 15 and 25 feet which split the width of the building into thirds. The building length was then split up depending on where columns could be placed in the existing third and fourth floor plans. As a note, the third and fourth floor plans are identical and the third floor plan was used to position the columns. Column lines where then placed and column locations where edited so that the center of each column would not exceed 10 percent of the bay width from the column line. This was done in order to use the direct design method to design the gravity system. For dimensions of the column line spacing, see Figure 11.

Figure 11 also shows which columns will affect the architecture or floor plan, highlighted in green. More information on how each column affects the floor plan or architecture along with a solution will be given later on in this report in the architecture breadth.



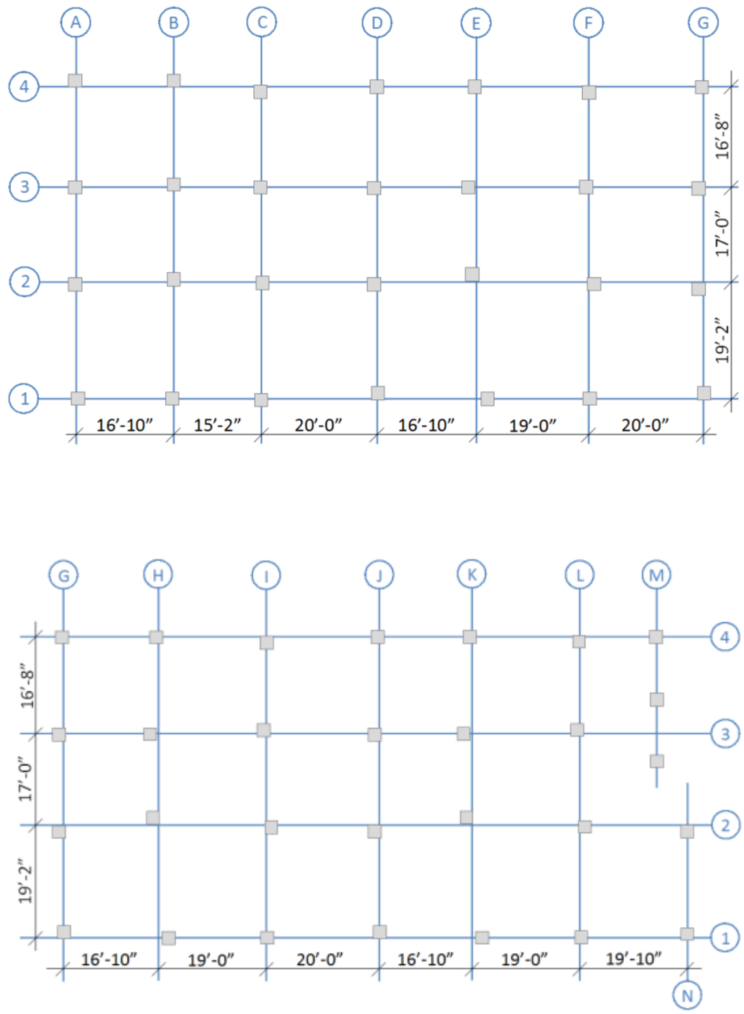


Figure 11: Column grid layout over 3rd Floor Plan of Global Village. Columns in green signify that the architecture or floor plan will be affected

Gravity System

A concrete flat plate structural system was chosen primarily for its flexibility to work around the floor plan. A flat plate also provides a thinner and lower costing floor than the other floor types. A slab with beams was not an option since the columns would have to line up and the ceiling heights would be severely affected. The other option would be a flat slab but since spans were relatively small, a flat plate was sufficient. A Flat Plate differs from a Flat Slab by not having drop panels, see **Figure 12**.

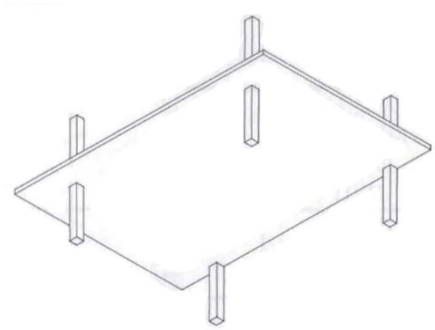


Figure 12: Two-Way Flat Plate floor construction. Courtesy of RSMeans

Since the spans differ considerably throughout the proposed structure, an Excel spreadsheet was used to design the gravity system. The spreadsheet was mainly used to design the reinforcement due to gravity and lateral loads but the spreadsheet was also used to design the columns, calculate the slab thickness, check for deflection, and more. For complete gravity system calculations of the sections discussed below, see **Appendices E-J**.

Materials used in designing the gravity system were the same materials used on the existing system. As shown above in the materials section, a concrete compressive strength, f'_c, of 4000 psi and a rebar yield strength, f_y, of 60 ksi were used in order to be consistent. Although not specified in the structural drawings, #5 bars were used for the slab reinforcement and #10 bars were used to reinforce the columns.

A summary of the gravity loads used for each floor can be found in **Table 10**. The live loads displayed were then reduced through the live load reduction equations given in Section 4.7.2 and Section 4.8.2 of ASCE 7-10. A live load of 150 psf for the penthouse floor is used where mechanical rooms are located and 30 psf elsewhere. Live loads were not reduced when 100 psf was exceeded which corresponds to the mechanical rooms and the entire second floor.

Floor	Total Loads	
	Dead (psf)	Live (psf)
2^{nd}	141	100
3^{rd}	141	40
4^{th}	136	40
Pent	141	150 / 30
Roof	23	$20(L_r)$ / 30.8(S)

Table 10: Gravity loads by floor

Column Size Calculations

The ground column on grid lines F - 2, see Figure 11, was used to design the columns for the entire structure. This column was used since it incurs the greatest load due to having the largest panel size of 19'-6" by 18'-1". Only one column was designed to reduce the construction costs of producing multiple sized columns throughout the floor plan. The size of the column is also uniform between stories and the weight of the columns above the ground floor is considered into the design.

The size of the column was mainly dependent on the unbalanced moments transferred by eccentricity of shear. Multiple slab thicknesses and column sizes were tried and a 20" by 20" column was the most economical. The shear capacity was found to be 190 psi where the shear due to the applied loads was 184 psi.

To reinforce the columns, (8) #10 bars are used with an edge spacing of 2.5". The total compressive strength and moment capacity was then checked using the strength interaction curve. The total compressive force due to the applied loads was found to be 437 kips with a maximum moment of 121 ft-kips. This is well in the interaction curve given that the pure compression capacity is 1245 kips and the column has a balanced-strain strength of 394 kips by 361 ft-kips. A check was also done using the spColumn software and determined to be adequate, see Figure 13.

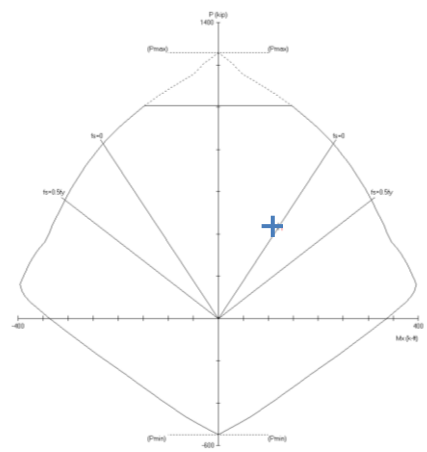


Figure 13: Balanced-strain interaction curve analyzed in spColumn

Calculation of Slab Thickness

A maximum clear span of 18'-4" and equations from Section 9.5.3 of ACI 318-08 were used to calculate the required slab thickness. A minimum slab thickness for an interior panel was calculated to be 6.67" where the minimum slab thickness for an exterior panel was 8.07". Since there are no edge beams, the thickness required for an exterior panel was increased by 10 percent. For construction purposes, the slab thickness would be rounded to 8.25".

Deflection checks were then performed on the maximum panel size and the slab was determined to be inadequate. The thickness was then rounded to 8.5" and deflection was no longer an issue. For the deflection calculations, it was assumed that 25 percent of the live load is sustained and 90 percent of the immediate deflection due to dead load occurs before partitions are installed. It was also considered that nonstructural attached elements would be damaged by excessive deflection. The deflection limit from Table 9.5b of ACI 318-08 gives a value of .5" where the maximum calculated deflection was .448".

Wide beam action and punching shear were also checked on the maximum bay sizes and proved to be adequate. Punching shear controlled over wide beam action with an ultimate shear of 114.6 kips. The shear capacity was calculated to be 146.7 kips.

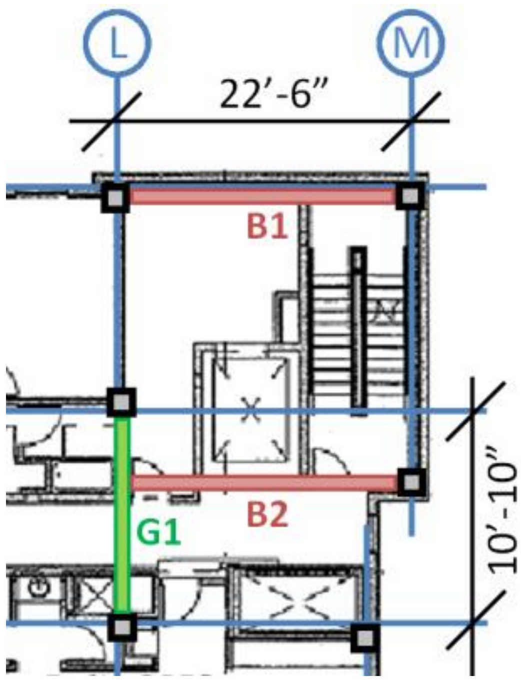
Gravity Reinforcement Design

To calculate the reinforcement required for gravity loads, the direct design method was used. Reinforcement required for the moment connections will be discussed later on in the lateral analysis section of this report. The direct design method was allowed to be used to design the reinforcement since the structure met all the conditions needed in order to follow this method.

Due to the extensive process of finding the reinforcement for each bay; calculations for a corner, exterior, and an interior bay are given in **Appendix G**. A summary of the required reinforcement for the second floor can be found in **Figures 15, 16, 17, and 18**. The numbers listed refer to the amount of #5 bars that are equally spanned over the distance given. For the required reinforcement of each floor see **Appendix H**. As a note, the bars spanning in the long direction would be placed lower in the slab and the bars spanning in the short direction would be placed on top of the long direction bars.

Stairwell Corner Design

A separate analysis needed to be completed for the stairwell corner because of the complexity of the area, see **Figure 14**. Due to the elevator shaft, the analysis done for the rest of the building was not able to be performed. Therefore, it was decided to use beams and girders to transfer the load to the columns. Two beams, highlighted in red, and one girder, highlighted in green, were designed. A summary of the sizes along with reinforcement required are given below.



B1: b=14", h=20" with (3) #9 bars

B2: b=18", h=25" with (5) #9 bars

G2: b=12", h=25" with (3) #9 bars

Figure 14: Stairwell corner support design

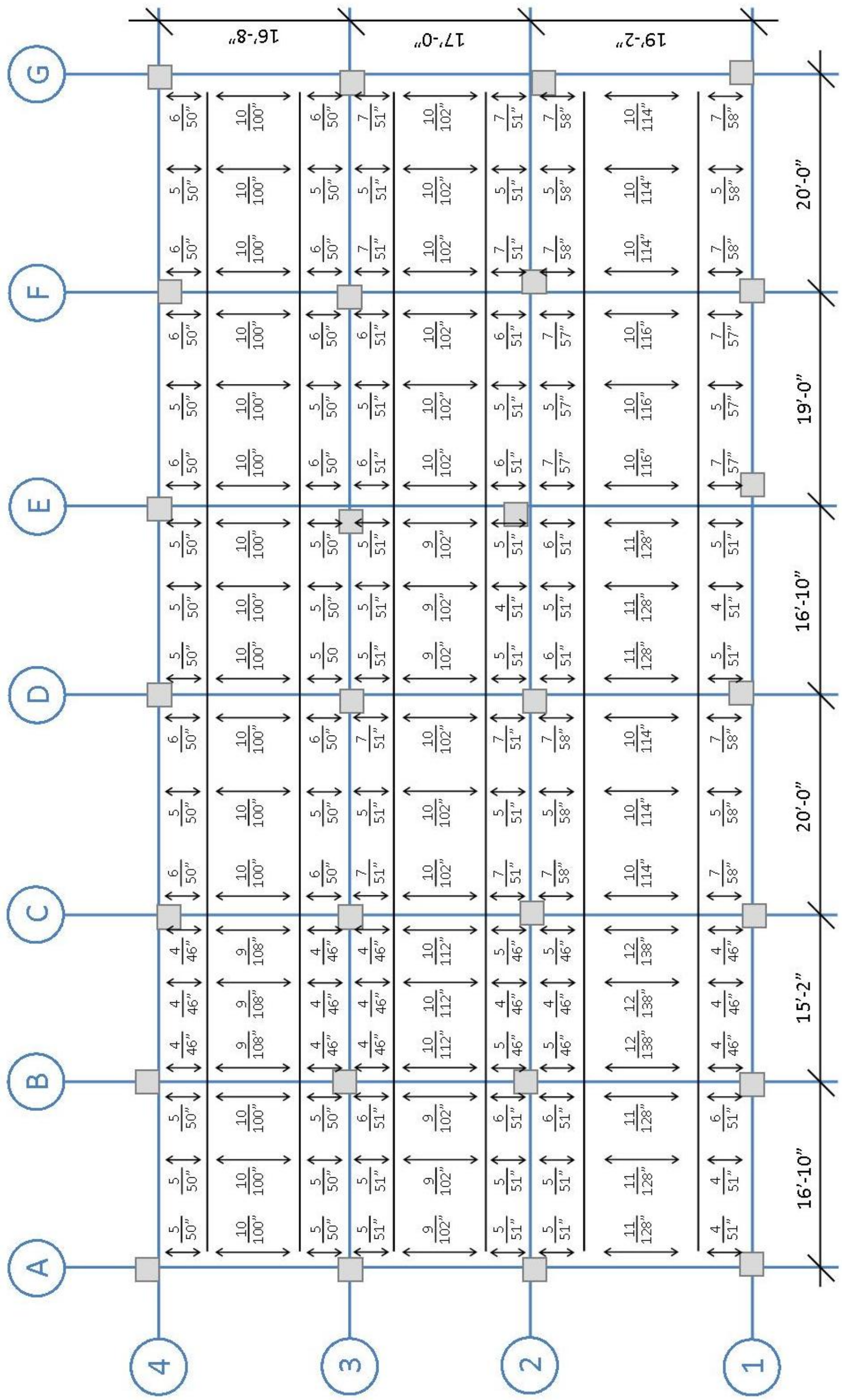


Figure 15: Gravity reinforcement required for the 2nd Floor (Part A)

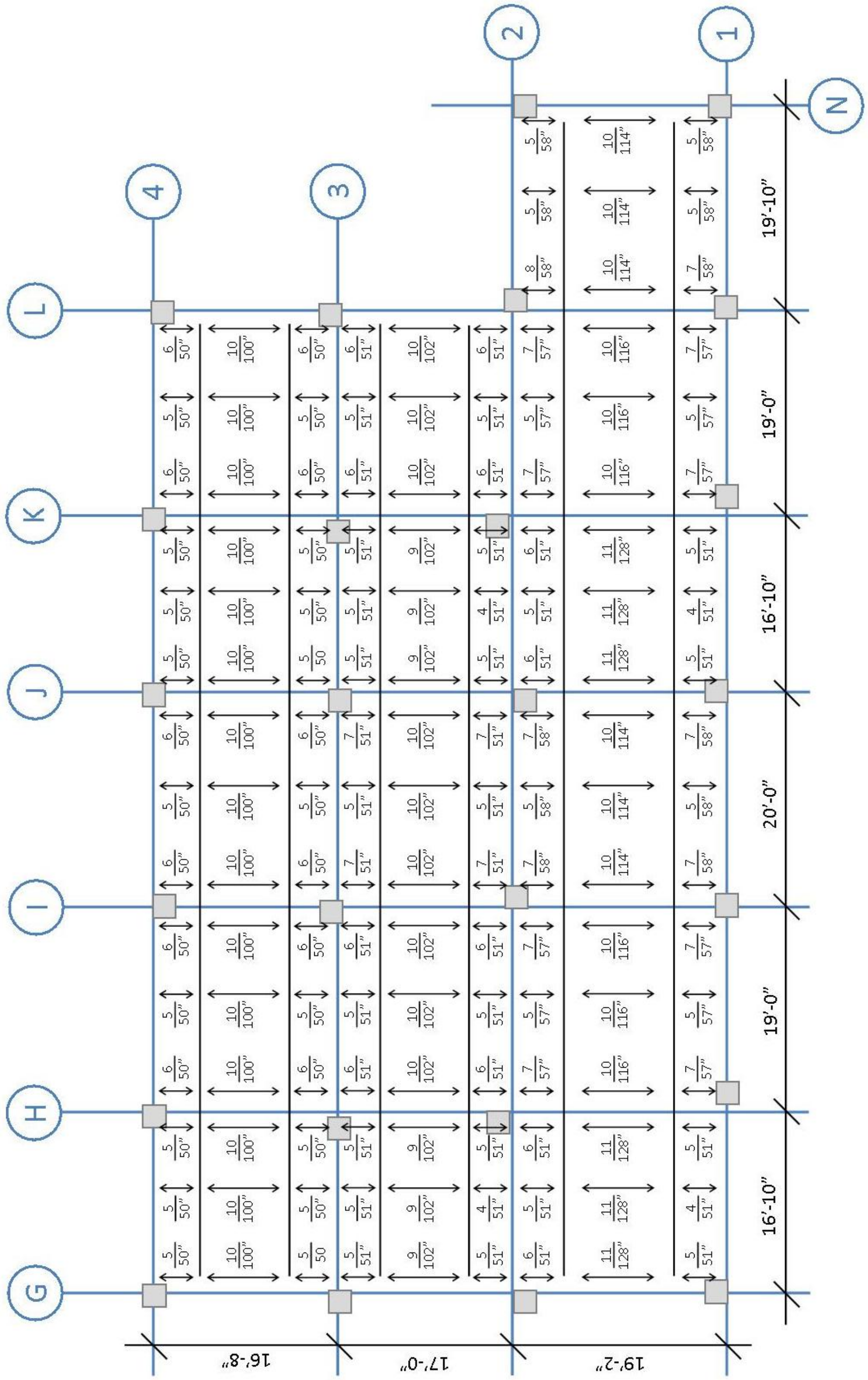


Figure 16: Gravity reinforcement required for the 2nd Floor (Part B)

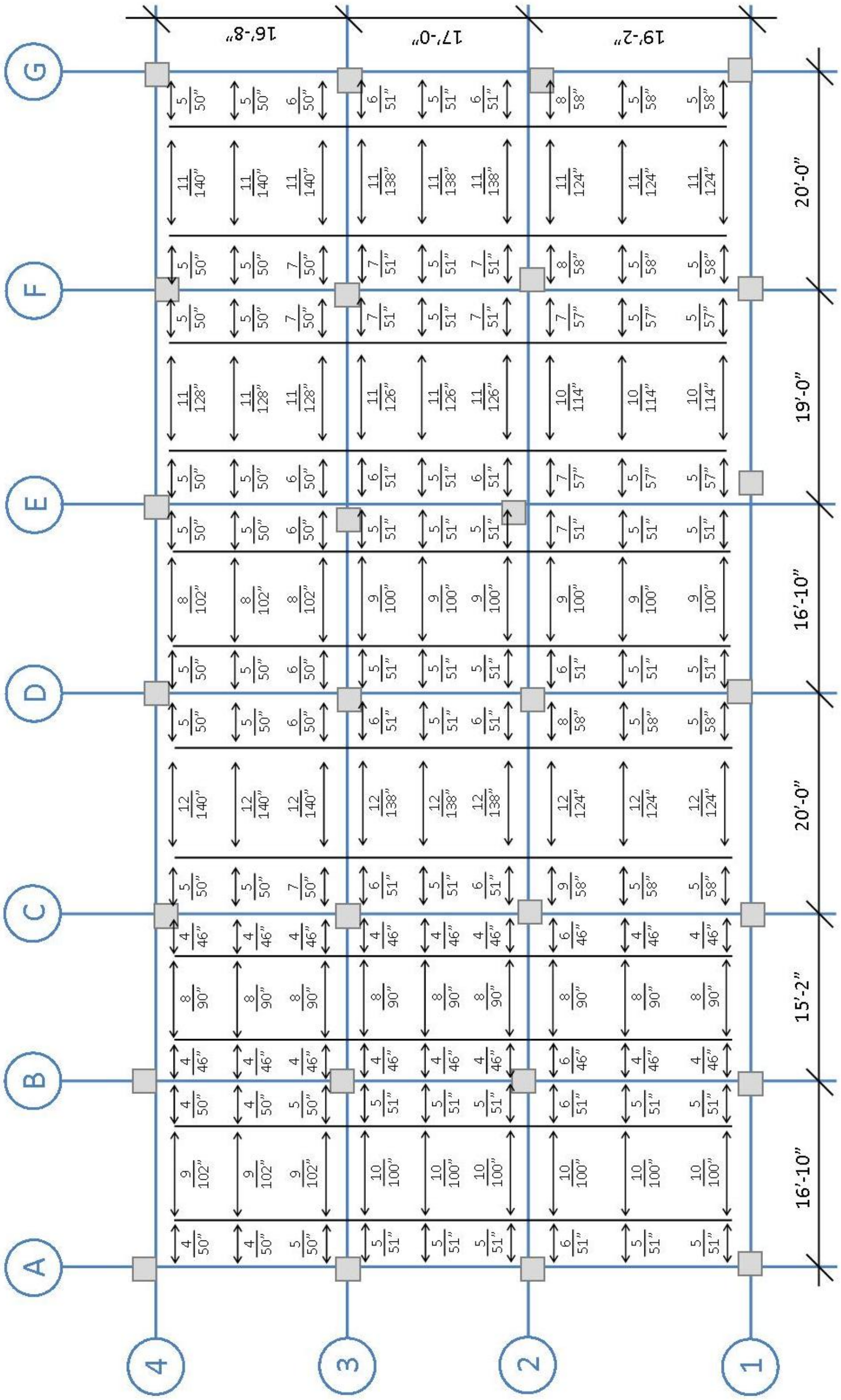


Figure 17: Gravity reinforcement required for the 2nd Floor (Part A)

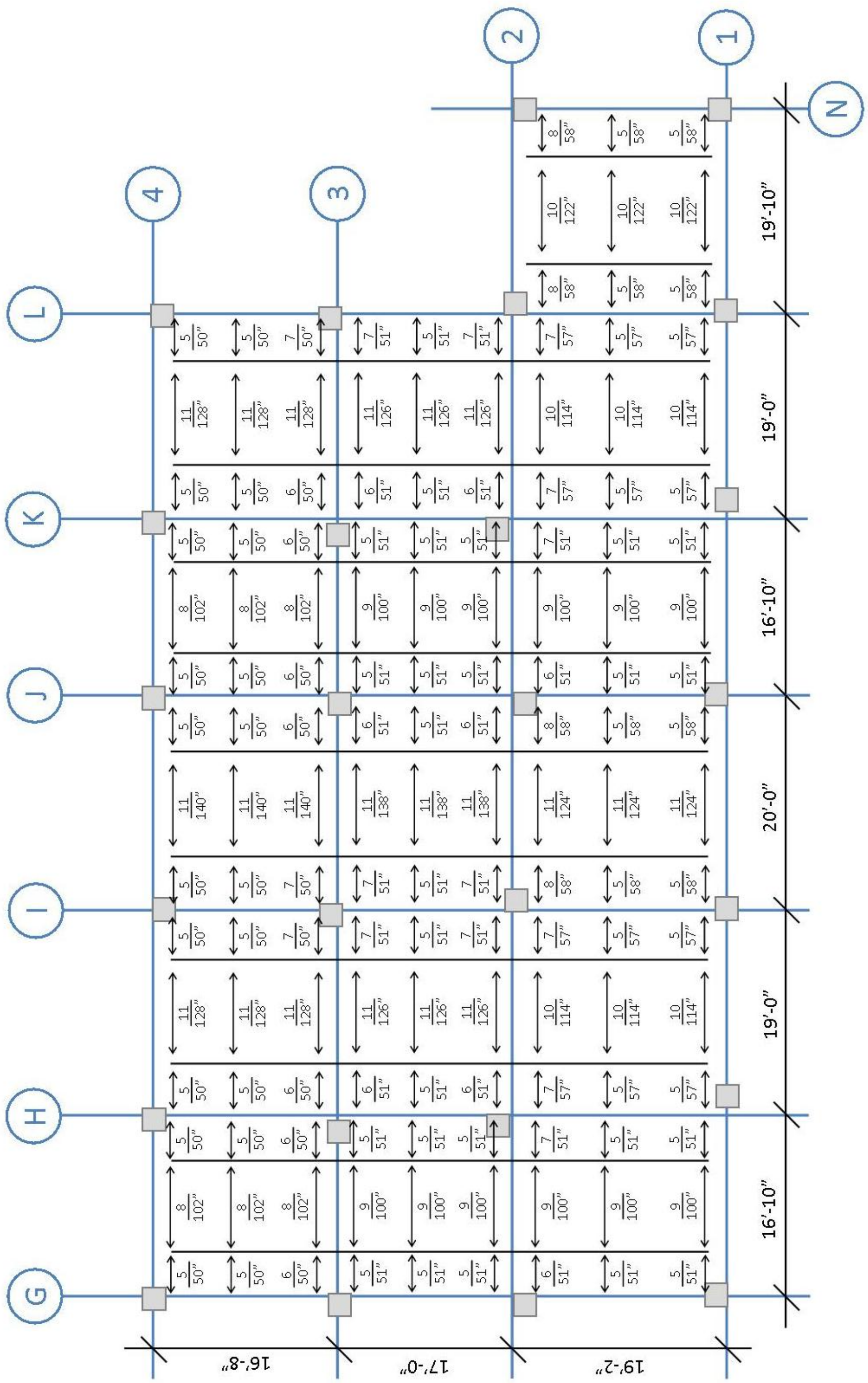


Figure 18: Gravity reinforcement required for the 2nd Floor (Part B)

Lateral Analysis

To analyze the lateral system of Global Village, a model was built using ETABS as shown in Figure 19. The geometry of the building was assumed to be a rectangular prism with dimensions: 223'-0" long by 52'-10" wide by 58'-0" high. The height of the building was changed to a flat roof mainly because of a lacking knowledge of ETABS to make a sloped roof. A height of 58'-0" was chosen since this is the midpoint of the roof and where the centroid of the roof weight would be located. Columns, shown in green, are 20" by 20" as found above and the slabs were modeled as rigid diaphragms with the weight of each floor used in the seismic analysis. Concrete beams, shown in yellow, with a width equal to that of the columns and a depth equal to that of the slab were spanned between each column to represent the moment connections.

As stated in the simplifications section, the building model did not take into account the 14'-0" grade level change from one side of the building to the other. Instead, the model was designed to have the same ground to roof height on each side.

Using this program, relative story drifts were obtained and then compared to accepted values. Moments due to lateral loads were also obtained in order to design the moment connections which will be explained later in this report.

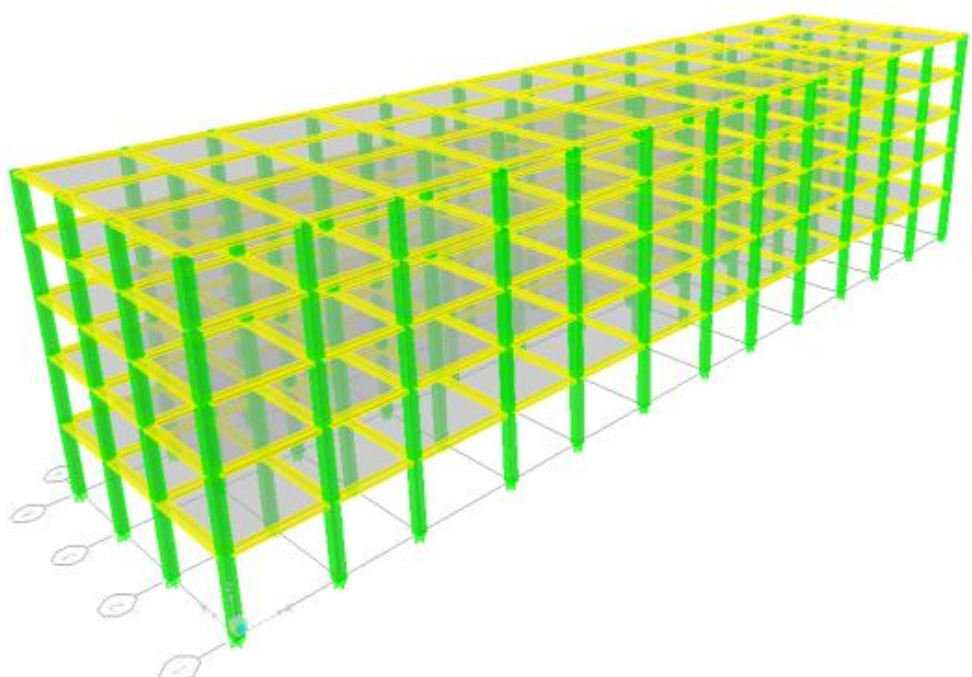


Figure 19: North leg of Global Village modeled in ETABS

Lateral Load Summary

A summary of the lateral loads acting on the building found from the lateral loads section is shown below in Figure 20. These loads were input into the ETABS model using load cases discussed below in order to analyze and design the proposed building's lateral system.

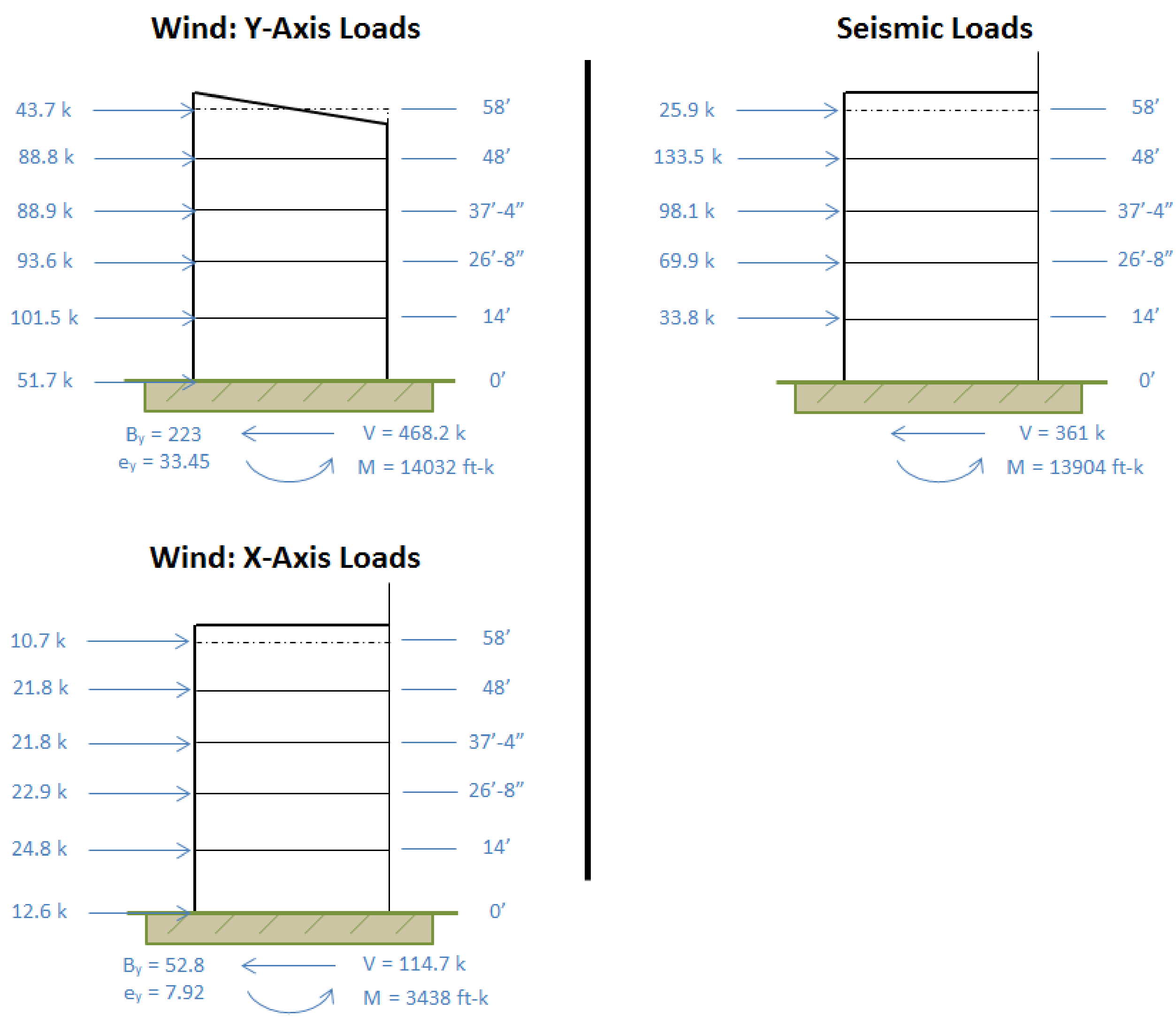


Figure 20: Summary of lateral loads acting upon north leg of Global Village

Applied Loads

Eight different load cases were input into ETABS, two of which are for seismic forces acting in the X and Y-Directions. The other six are for the various wind load cases described in Figure 27.4-8 of ASCE 7-10 or in Figure 21 below.

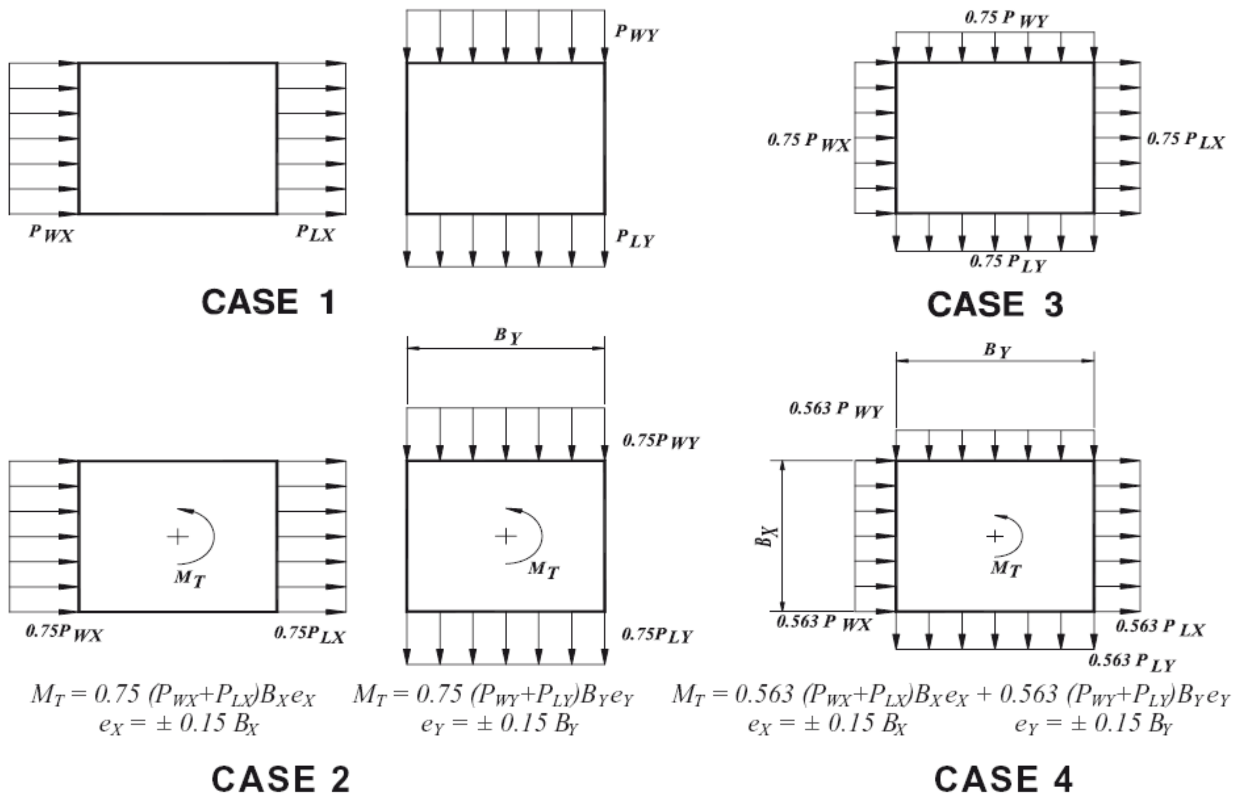


Figure 21: Wind load cases used in ETABS. Courtesy of ASCE 7-10 Figure 27.4-8.

Lateral Movement

Story Drift is a serviceability consideration and is defined as the displacement of one level with respect to the level below it. ETABS was used to find the maximum story drift caused by both wind and seismic forces in the X and Y-Directions. These values were then compared to allowable values outlined in ASCE 7-10. For seismic, Table 12.12-1 in ASCE 7-10 was used to find an allowable story drift of $0.015h_{sx}$. For wind, an allowance of $h_{sx}/400$ was used. As shown in **Table 11**, the maximum story drifts for both seismic and wind in the X and Y-Directions are below the allowable values proving that this lateral system is acceptable for drift.

Story Drifts (in)						
Level	Seismic			Wind		
	$\Delta_{X\text{-}Frame}$	$\Delta_{Y\text{-}Frame}$	$\Delta_{Allowable}$	$\Delta_{X\text{-}Frame}$	$\Delta_{Y\text{-}Frame}$	$\Delta_{Allowable}$
Roof	0.079	0.091	1.800	0.019	0.114	0.300
Pent	0.141	0.160	1.921	0.031	0.189	0.320
4th	0.201	0.223	1.921	0.046	0.267	0.320
3rd	0.272	0.300	2.279	0.066	0.378	0.380
2nd	0.178	0.193	2.520	0.046	0.261	0.420
Total Drift	**0.871**	**0.967**	**10.441**	**0.208**	**1.209**	**1.740**
	√ ok	√ ok		√ ok	√ ok	

Table 11: Maximum story drifts found using ETABS

Overturning Moment

From **Figure 20** in the lateral load summary section, wind loads control the overturning moment of the building. The wind forces in the Y-Direction result in an overturning moment, M_o, of $14{,}032^{ft\text{-}k}$. The critical moment occurs in the direction with the least depth, corresponding to the Y-Direction of the model or the width of the building.

To resist this moment, the building weight is multiplied by the moment arm. The moment arm in this case is half the building width. The resisting moment, M_R, calculates out to $254{,}445^{ft\text{-}k}$ which is much greater than M_o. Therefore, the building has the capacity to withstand the overturning moment due to both wind and seismic loads.

Lateral Load Moments

To design the moment connections used for the buildings lateral system, the maximum moments in the columns for each story in both the X and Y-Direction were found using the ETABS model. The controlling load case in the X-Direction was due to earthquake loads. Wind load Case 2 was the controlling load case for the Y-Direction. See **Table 12** for a list of the moments found for each story and direction.

Floor	Lateral Load Moments (ft-k)	
	X-Direction	Y-Direction
2nd	96	68
3rd	118	64
4th	106	43
Pent	48	19

Table 12: Lateral load moments

Lateral Reinforcement Design

These moments were then input into the unbalanced moment section of the spreadsheet to calculate the required reinforcement for the moment connections. Due to the amount of calculations; a corner, exterior, and an interior bay are given in **Appendix G**. A summary of the required reinforcement for the second floor can be found in **Figures 22, 23, 24, and 25**. As in the gravity system reinforcement plans, the numbers listed refer to the amount of #5 bars that are equally spanned over the distance given. For the required lateral reinforcement of each floor see **Appendix J**.

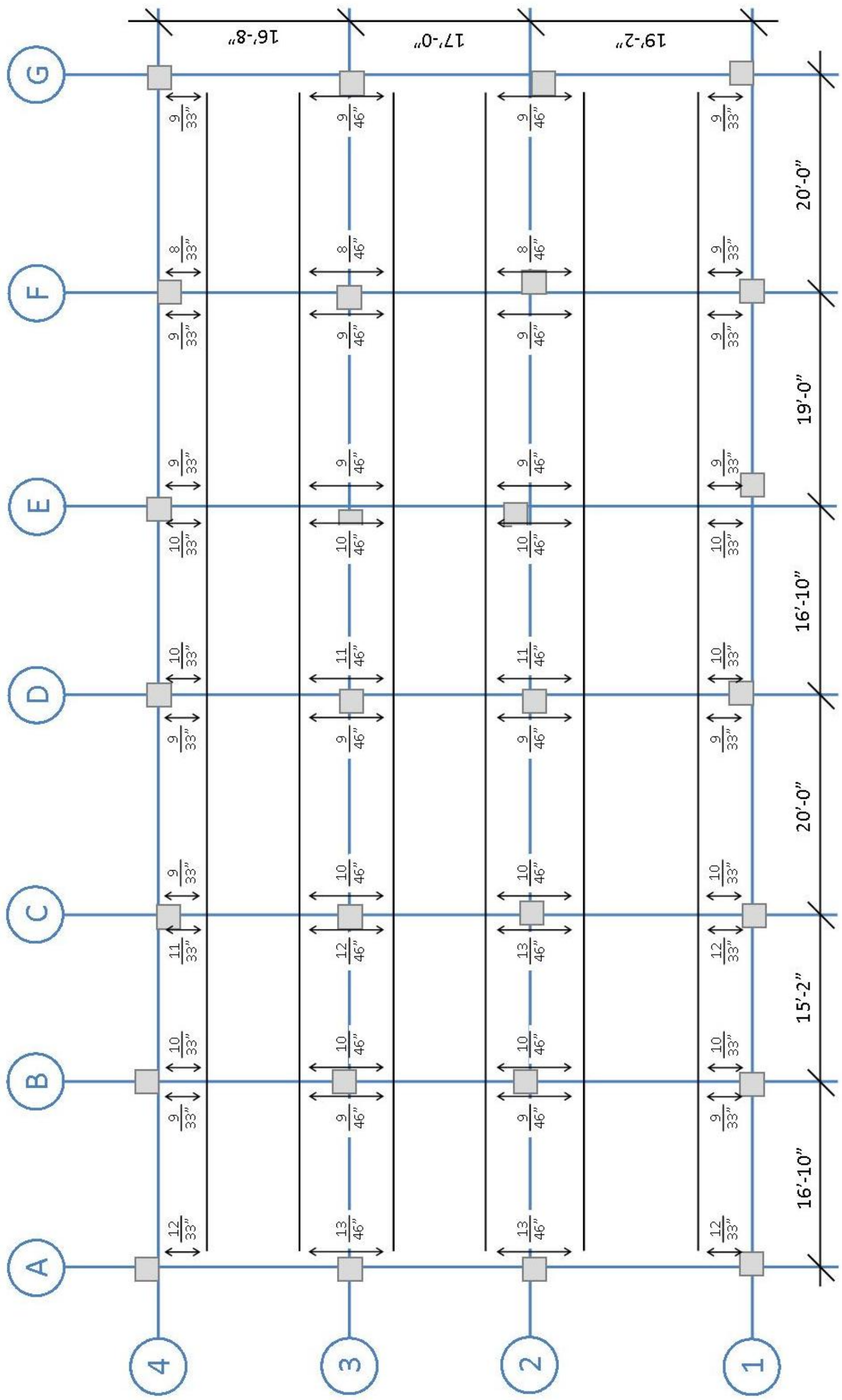


Figure 22: Lateral reinforcement required for the 2nd Floor (Part A)

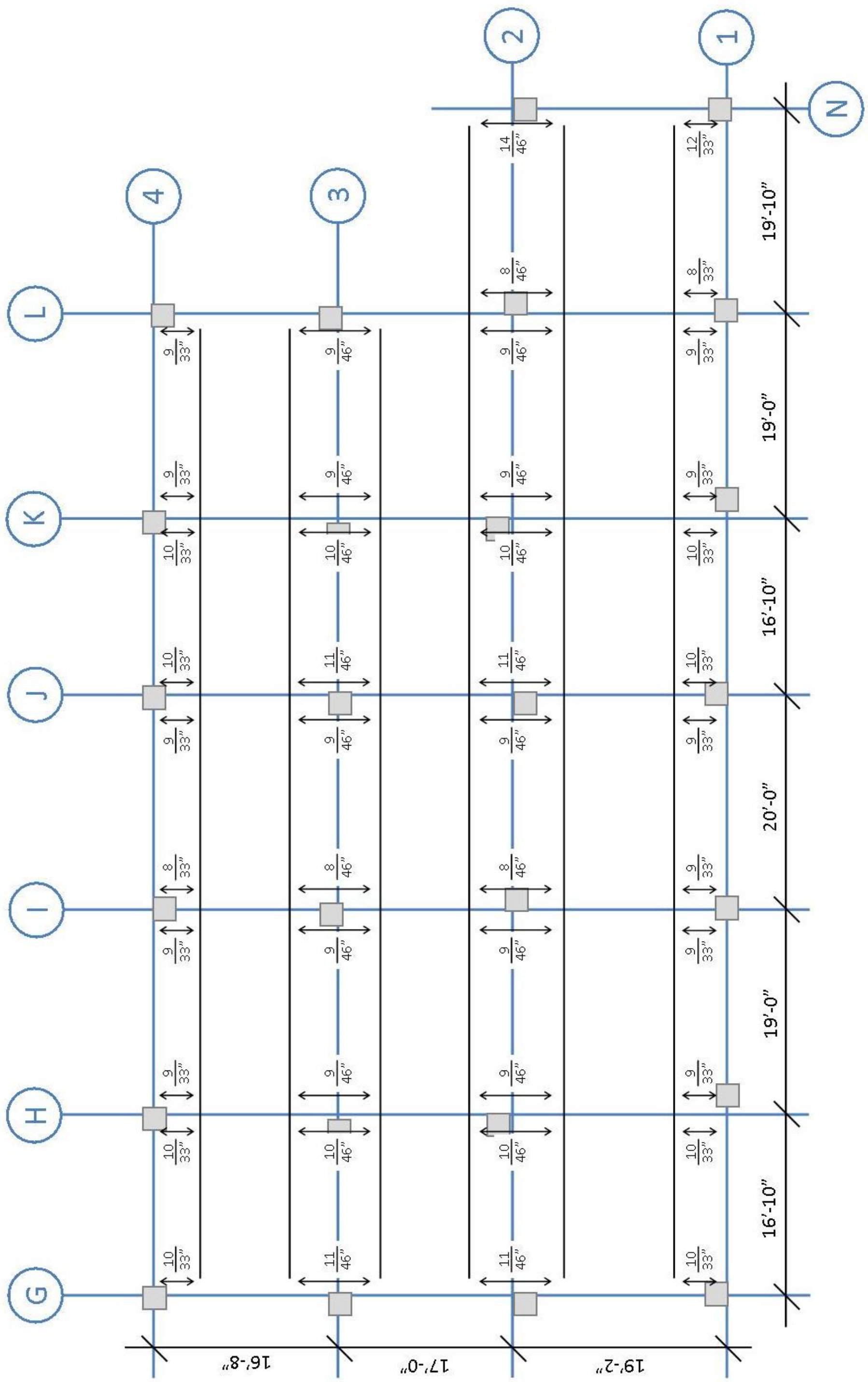


Figure 23: Lateral reinforcement required for the 2nd Floor (Part B)

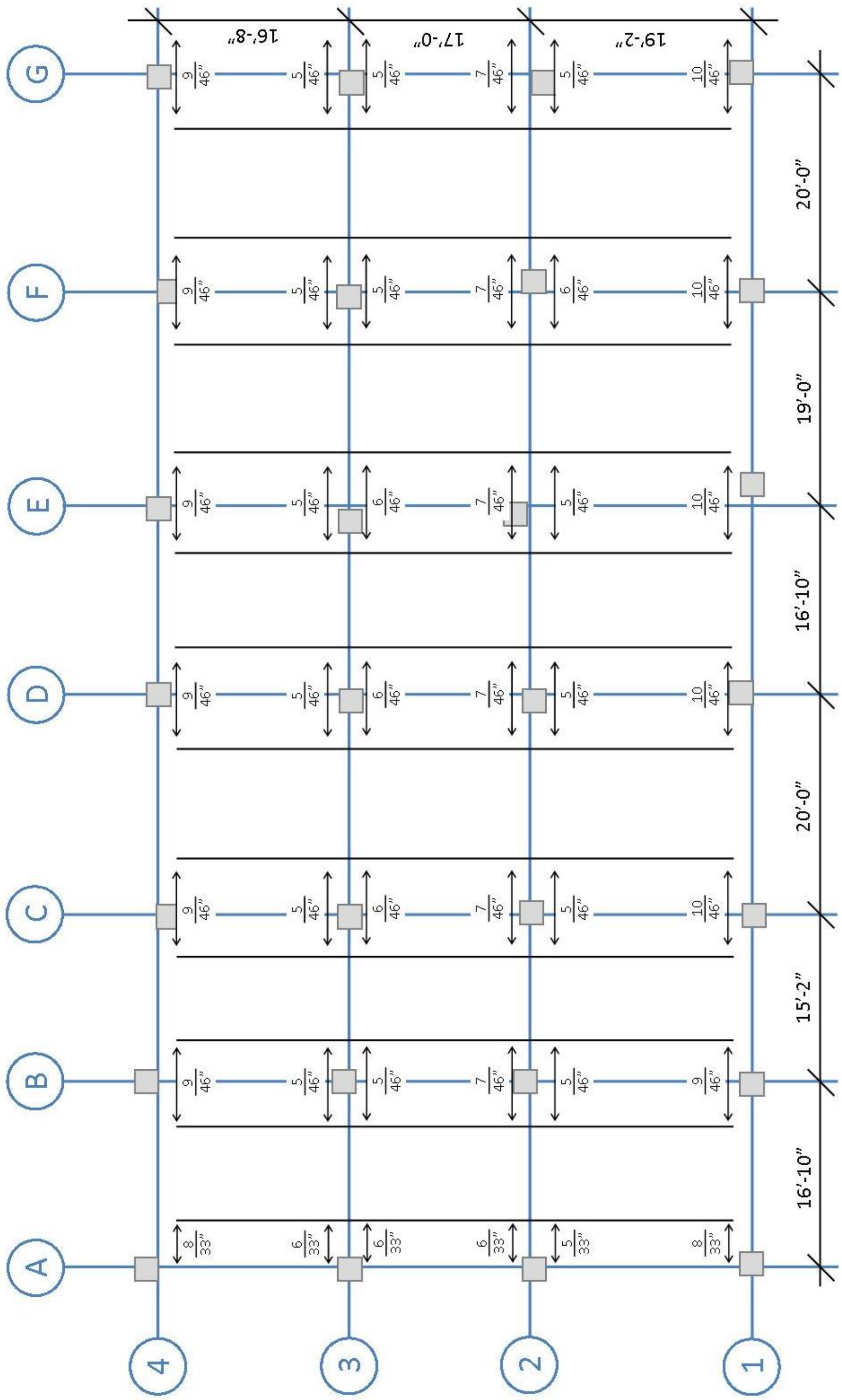


Figure 24: Lateral reinforcement required for the 2nd Floor (Part A)

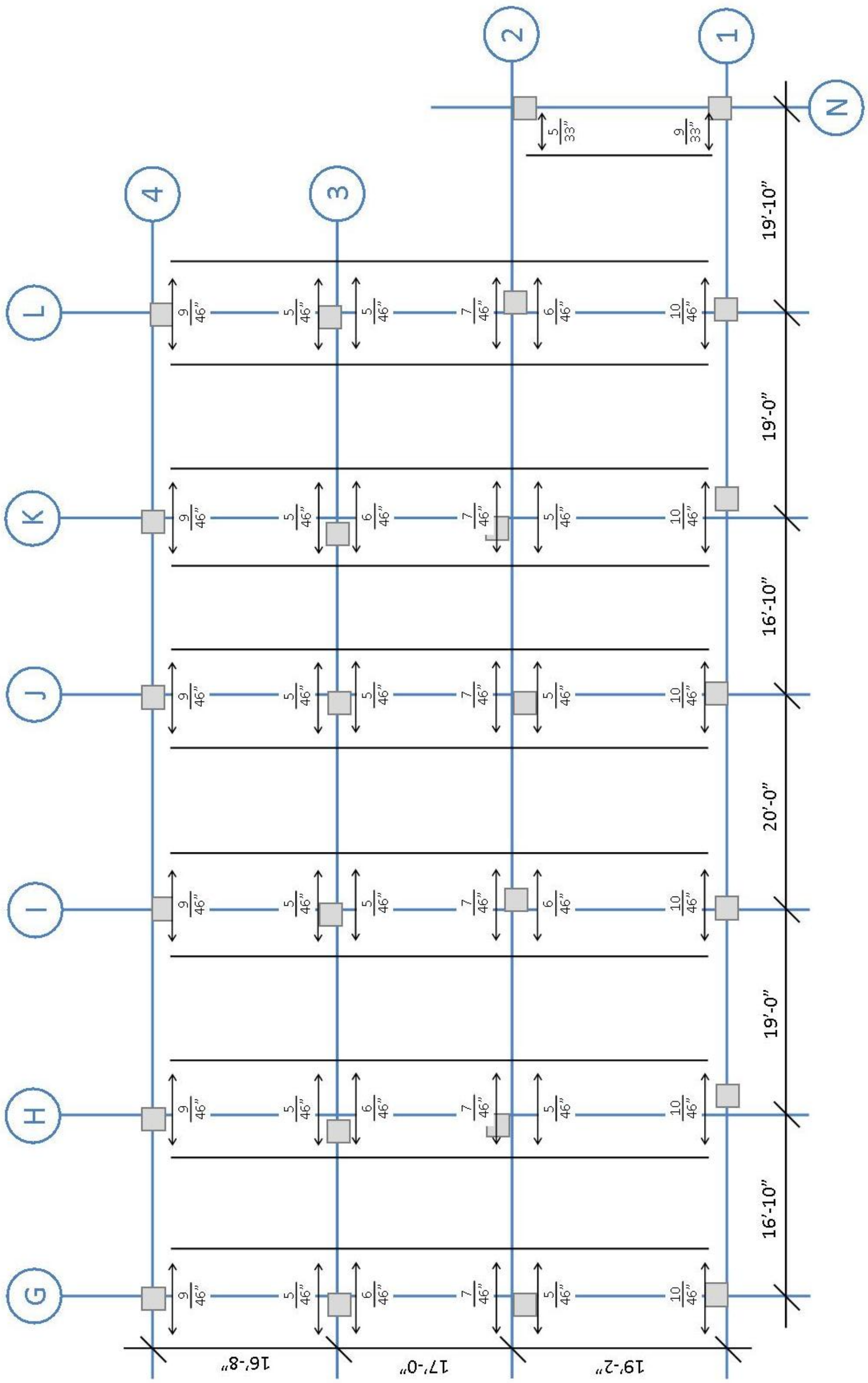


Figure 25: Lateral reinforcement required for the 2nd Floor (Part B)

Architecture Breadth

An architecture breadth is completed in order to address the impacts that the new structural system produces and to provide a possible solution. Since the original design uses wood on the residential floors, the floor plan will be affected due to the placement of the concrete columns. A column layout was made to work around the architecture as best as possible but some areas could not be avoided.

In total, there are 15 areas on the third floor that are affected by the column placements. All of these areas occur due to interior placed columns and most columns affect bathrooms and fan coil unit spaces. Out of the 15 areas, eight column locations directly affect fan coil unit spaces and six locations affect bathroom spaces. Even though the columns placed in the bathrooms take up space, the bathroom areas are still manageable and thus no alteration needs to be done. Therefore, only the changes to the fan coil unit spaces and the column in the corridor area will be addressed. The columns affecting these areas are highlighted in green in Figure 26.

Even though the academic fit-out space on the second floor can easily be adjusted, many changes were made to the floor plan. Almost every wall was moved since the walls followed the steel columns in the existing floor plan. This produced long narrow classrooms or rooms with columns in the middle of the space.

In order to specifically show the solution and changes made to the floor plan, expanded areas are shown both before and after modifications have been made. Revit was used to display the changes in the floor plan and provide 3D images. Since these areas are similar to other locations in the building, any alterations made can be considered to be replicated.

Due to the third and fourth floor being identical, any changes applied to the third floor are also considered to be changed on the fourth floor. The ground floor was not analyzed in this report since the area is mainly open and designed by the retail owners. The mechanical penthouse is also not examined here due to minimal adjustments needed.

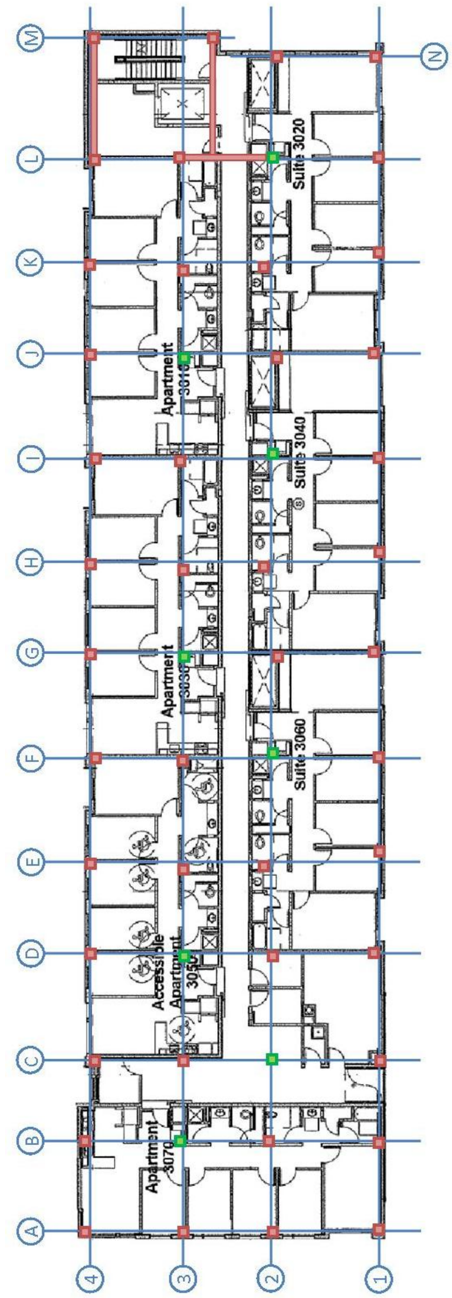


Figure 26: Column grid layout over 3rd Floor Plan of Global Village. Columns in green signify that the architecture or floor plan will be affected

Fan Coil Unit Area Re-Design

Figure 27 shows a portion of the third floor plan from the existing building while Figure 28 shows the modifications that have been made due to the placement of the columns, shown in blue. The main alteration in the floor plan is that the fan coil unit space was moved to the other side of the door way. This creates a narrower entrance and also a narrower kitchen. Figure 28 also shows the column taking up space in the bathroom but not necessarily disrupting the space to where it needs to be modified.

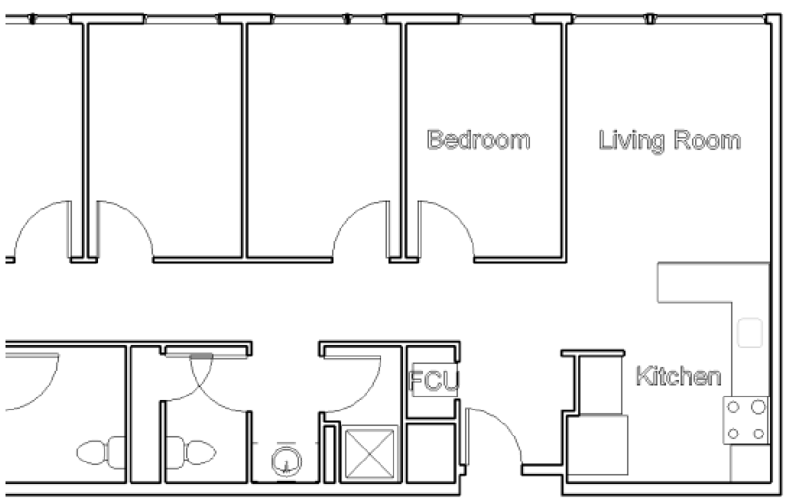


Figure 27: Original fan coil unit area modeled in Revit

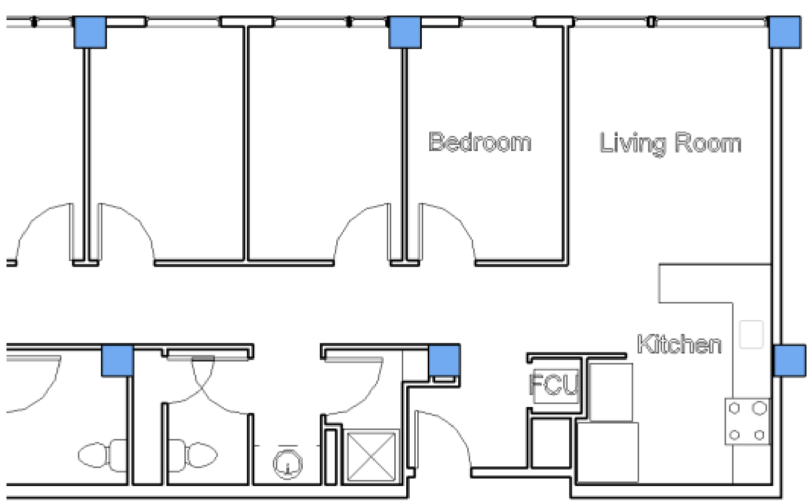


Figure 28: Modified fan coil unit area modeled in Revit

Corridor Area Re-Design

Figure 29 shows the original floor plan and view of the corridor space while Figure 30 shows the modifications that have been made due to the placement of the columns. To make the column more aesthetically pleasing, the space has been transformed into a lounge area with a small table around the column to put books or drinks.

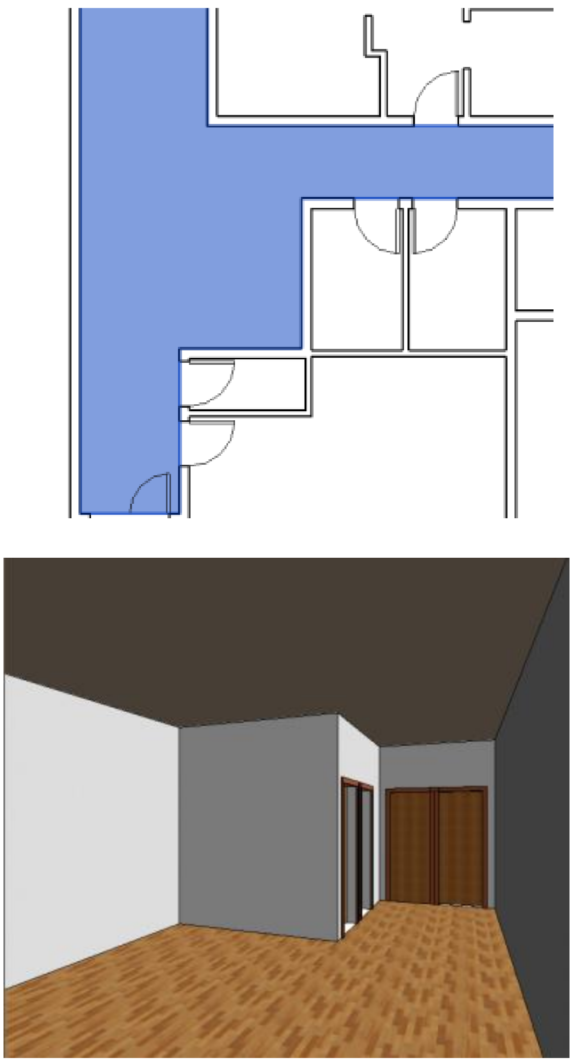


Figure 29: Original corridor area modeled in Revit

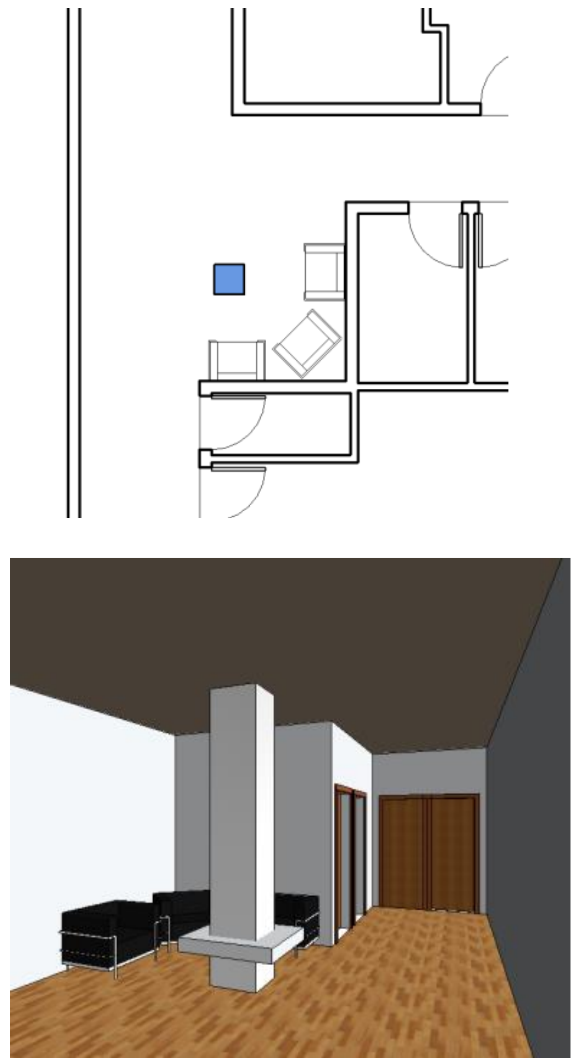


Figure 30: Modified corridor area modeled in Revit

2nd Floor Re-Design

Figure 31 shows the existing second floor plan while Figure 32 shows a possible solution to the new column layout. As a result of smaller bay sizes, the width of the rooms decreased producing long narrow classrooms. Classrooms were put where the maximum spans occur in order to obtain the greatest width possible. Storage areas were place where the smallest spans occur since the width would not be acceptable for a classroom. Rooms that have columns in the middle of the space were chosen as computer labs since visibility or aesthetics are not considered a necessity.

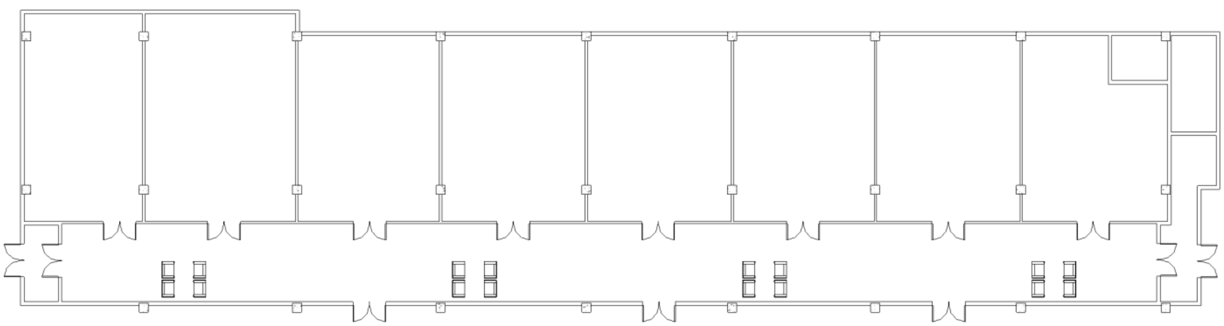


Figure 31: Original 2nd Floor Plan modeled in Revit

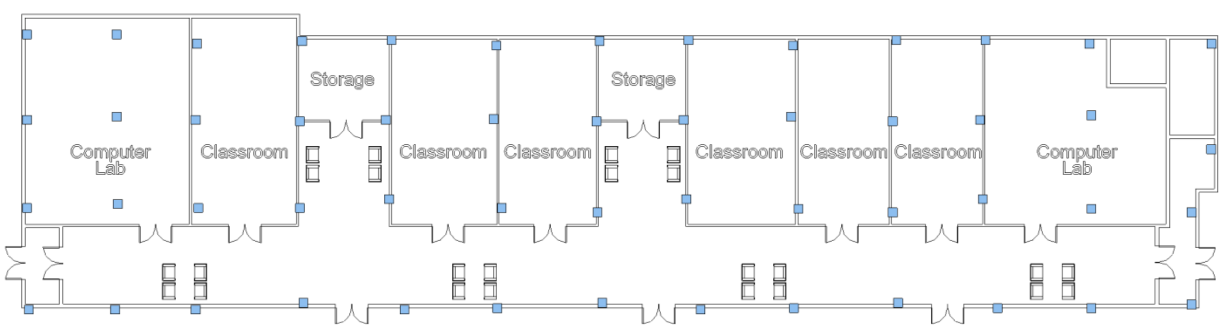


Figure 32: Modified 2nd Floor Plan modeled in Revit

Const. Management Breadth

A construction management breadth is completed in order to assess the constructability of re-design for the proposed structure. This includes a study on why most university buildings are constructed out of concrete or masonry. This information would then be used to compare the proposed building to the existing building based on improvements in construction methods, safety, and more.

Study of Residential Buildings

To find out why most university buildings are made of masonry or concrete, David Manoz who is the assistant director of housing at Penn State, was contacted. He stated that most on-campus housing uses concrete as the main material for its durability and fire rating. Concrete buildings can last for a long time without any upkeep or maintenance. Even if the room isn't equipped with sprinklers, the structure will still be fine after a fire. A couple of fires actually occurred in the dorms and nothing was damaged aside from the student's belongings. Other benefits of using concrete are that you get a stiffer structural system and it provides good proofing between rooms.

There are some advantages of using wood for a structural system. The main reasons are that it is a lot cheaper and initial construction is easier. It also increases the flexibility of the floor plan since it's not restrained by bay sizes like in concrete or steel. However, even though the initial cost may be lower, maintenance costs become higher and the durability of the structure is a lot less.

After talking to David Manoz, Jim Yarrington who is the director of construction services at the Rochester Institute of Technology was contacted to find the main reason behind the use of wood in Global Village. The main reason was in fact driven by cost. The building was intended to be all steel and concrete in the initial design but was cost-prohibitive. By switching to this steel-wood hybrid system, the third building of the complex was affordable and thus a larger volume of rooms was constructed.

Cost Analysis

Since the building cost needed to be considered, a cost analysis between the proposed and existing building was completed. It was found, as from the study, that the proposed system is more than triple the existing building's cost. Through the use of RSMeans, it was determined that the total cost of the existing steel-wood system is $571,588.23 where the proposed buildings structure was calculated to cost $1,826,436.50.

Constructability

In terms of constructability, the use of concrete would improve many areas in construction. Even though wood can be considered to be more recyclable, less construction waste is produced if concrete is used. This is primarily due to wood having to be cut if not adequately sized where concrete is just poured. Since the buildings materials are uniform throughout the structure, construction should be faster since fewer firms are involved. The use of one main material also improves safety since the firm providing the work is mainly specialized in this material. A drawback of using concrete is that more field labor is required since the structural system is basically made on-site instead of structural members being shipped to the site as done in steel construction.

Conclusion

The overall goal of this project was to alter the dual structural system to a more uniform system. Global Village consists of two different structural systems. Steel framing is used on the bottom half of the building while wood framing is used on the top half. The use of different structural materials within the building complicates the design and requires more people to be involved than a uniform structural system.

Concrete was chosen due to its predominant use in most on-campus residential buildings. A reinforced flat plate system was then selected in order to avoid altering the existing floor plan. Through calculations along with deflection and unbalanced moments checks, it was determined that an 8.5" slab supported by 20" by 20" columns with (8) #10 bars was adequate. Slab reinforcement was then found using a spreadsheet following the direct design method.

Since the building is relatively short, it was determined that moment connections in both directions would be sufficient to accommodate lateral loads. Eight different lateral load cases were analyzed on a model of the proposed building using ETABS. Moments found in the columns were then input into the unbalanced moment section of the spreadsheet to calculate the required reinforcement for the moment connections. Story drift values were taken directly from ETABS and compared to allowable values outlined in ASCE 7-10. The maximum story drift that the lateral frame induced was 1.751" in the N-S Direction as a result of seismic loads. This is much less than the allowable 10.441". As a note, the maximum wind drift of 1.696" is also below the allowable 1.740" for wind loads.

Although columns were placed as best as possible to avoid altering the existing floor plan, some areas could not be avoided. An architecture breadth was completed in order to analyze these changes and to provide a solution. In total, 15 areas were affected by the column placements but only eight locations needed to be modified. Most of these areas were due to columns being placed where fan coil units were located. As a result, the fan coil units were relocated which in turn made the entrance and kitchen spaces narrower. A new floor plan for the second floor was also required due to the new column layout.

The use of concrete provides many benefits for on-campus residential buildings. Buildings made of concrete are more durable and offer sound proofing benefits which may be desired in dormitory buildings. The fire rating of the building is also improved and maintenance costs tend to be lower than other materials. The drawback of using concrete is that it is a more expensive structural system. RSMeans was used to calculate the cost of each system and it was found that the proposed structure costs $1,826,436 where the existing design was calculated to cost $571,588.

Although the proposed building would be more durable and have lower maintenance costs, the upfront cost of the structure is too great and would not be permitted due to budget constraints. The preliminary design of the existing building was a steel and concrete frame but in order to construct a third building in the project, the hybrid structural system needed to be used.

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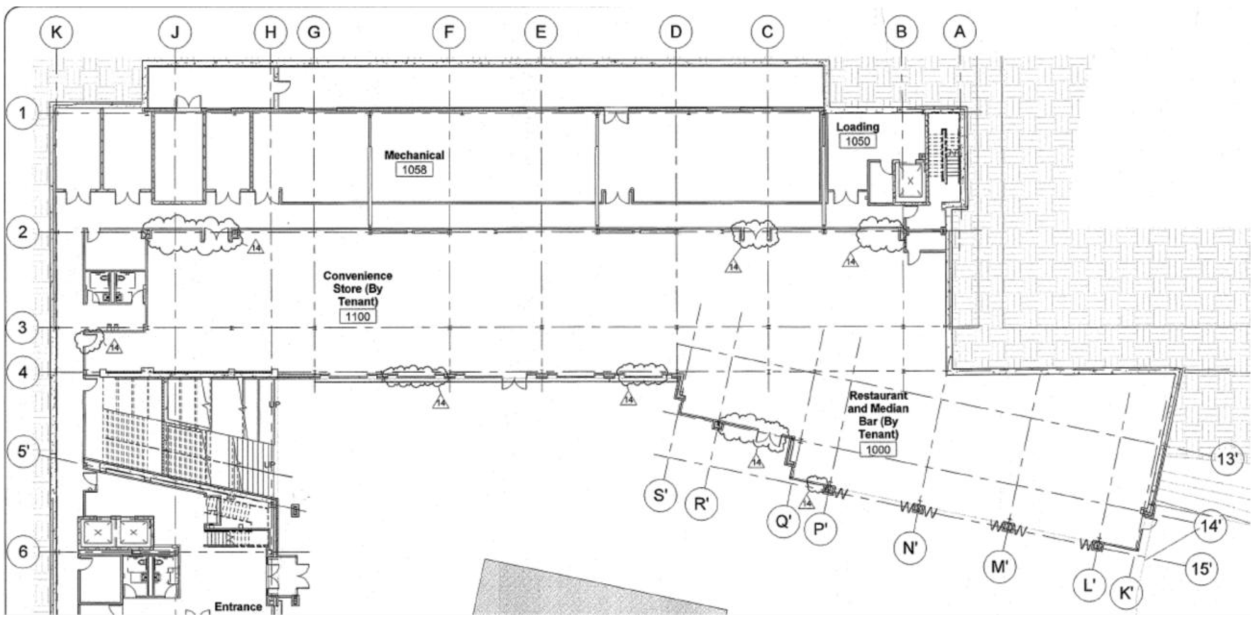
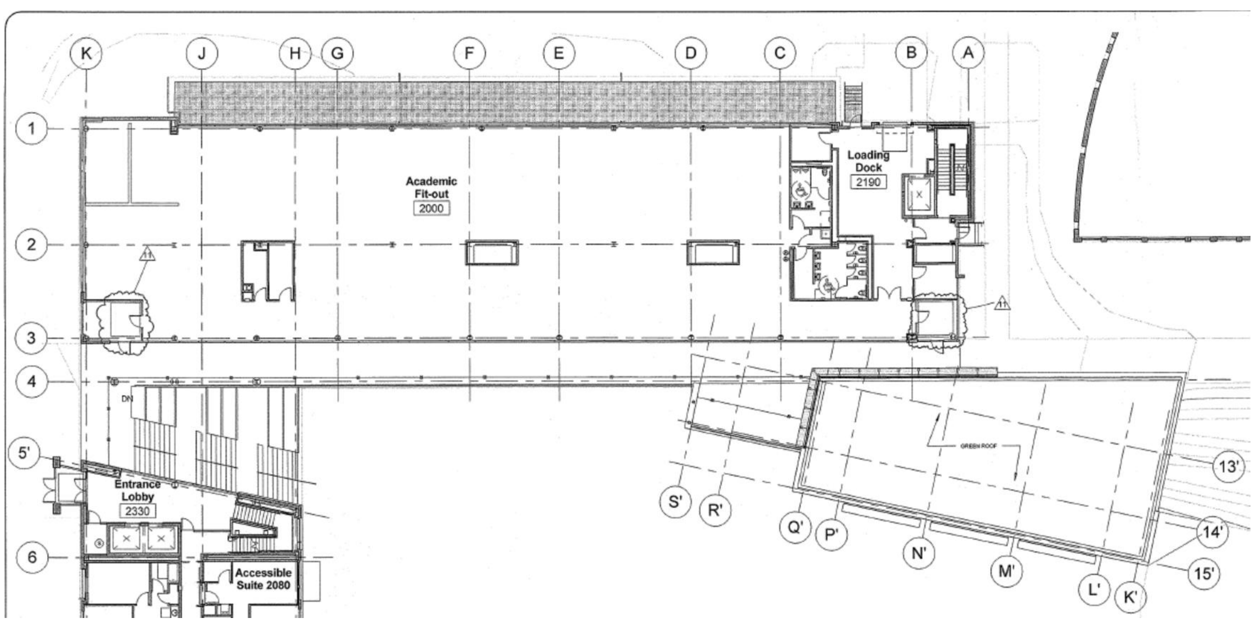
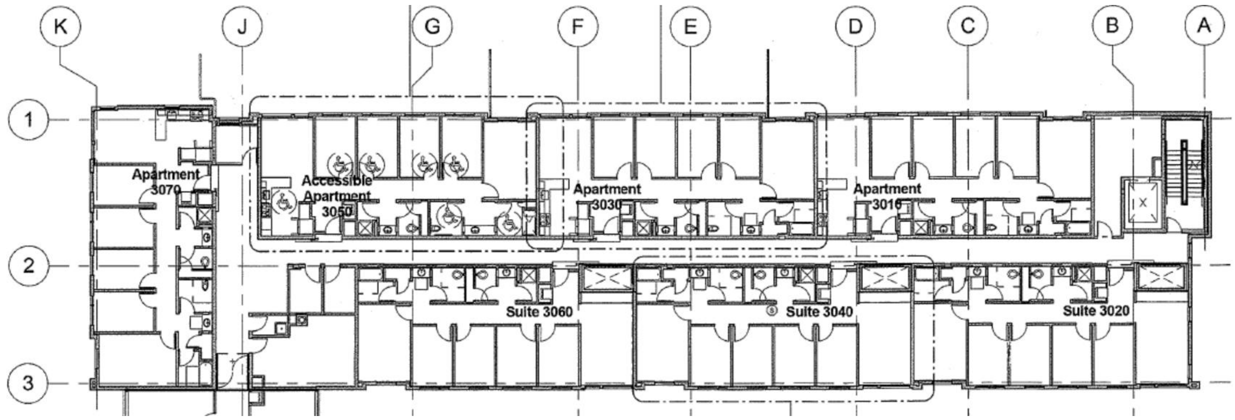
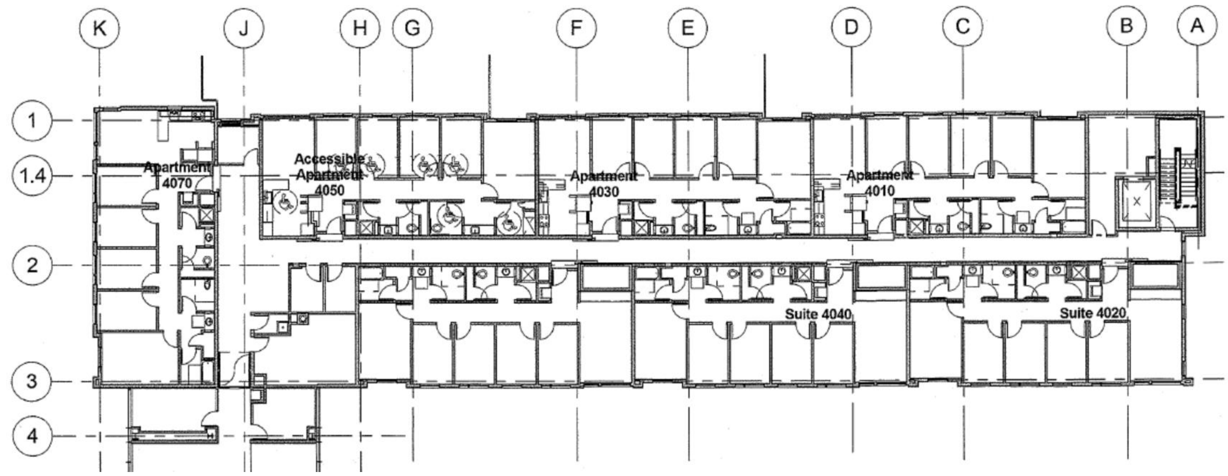
Appendix A: Typical Plans and Details







K
J
G
F
E
D
C
B
A
1
2
3
Apartment 3070
Accessible Apartment 3050
Apartment 3030
Apartment 3010
Suite 3060
Suite 3040
Suite 3020



K
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1.4
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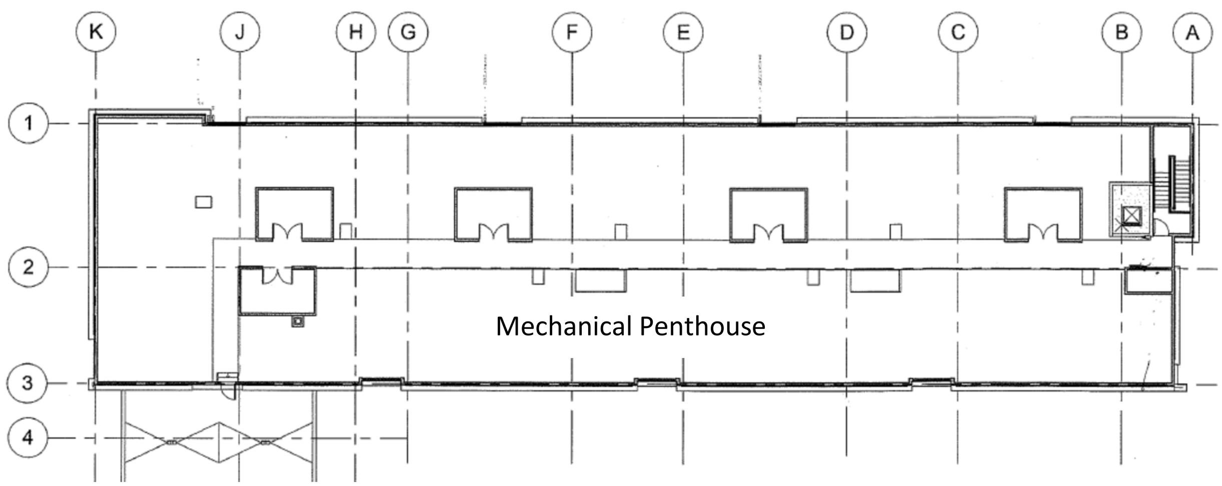
4
Apartment 4070
Accessible Apartment 4050
Apartment 4030
Apartment 4010
Suite 4040
Suite 4020



K
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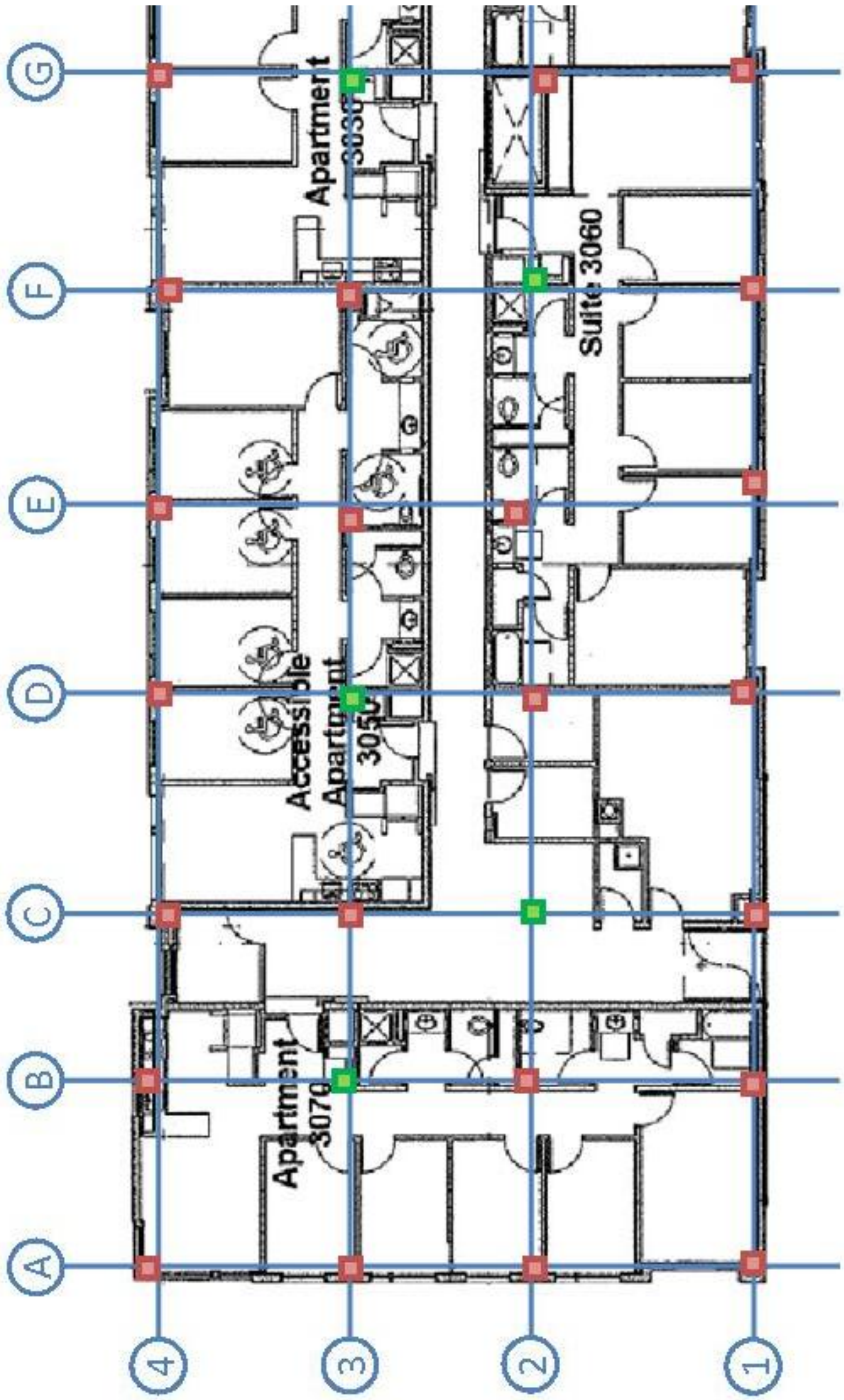
Mechanical Penthouse


G
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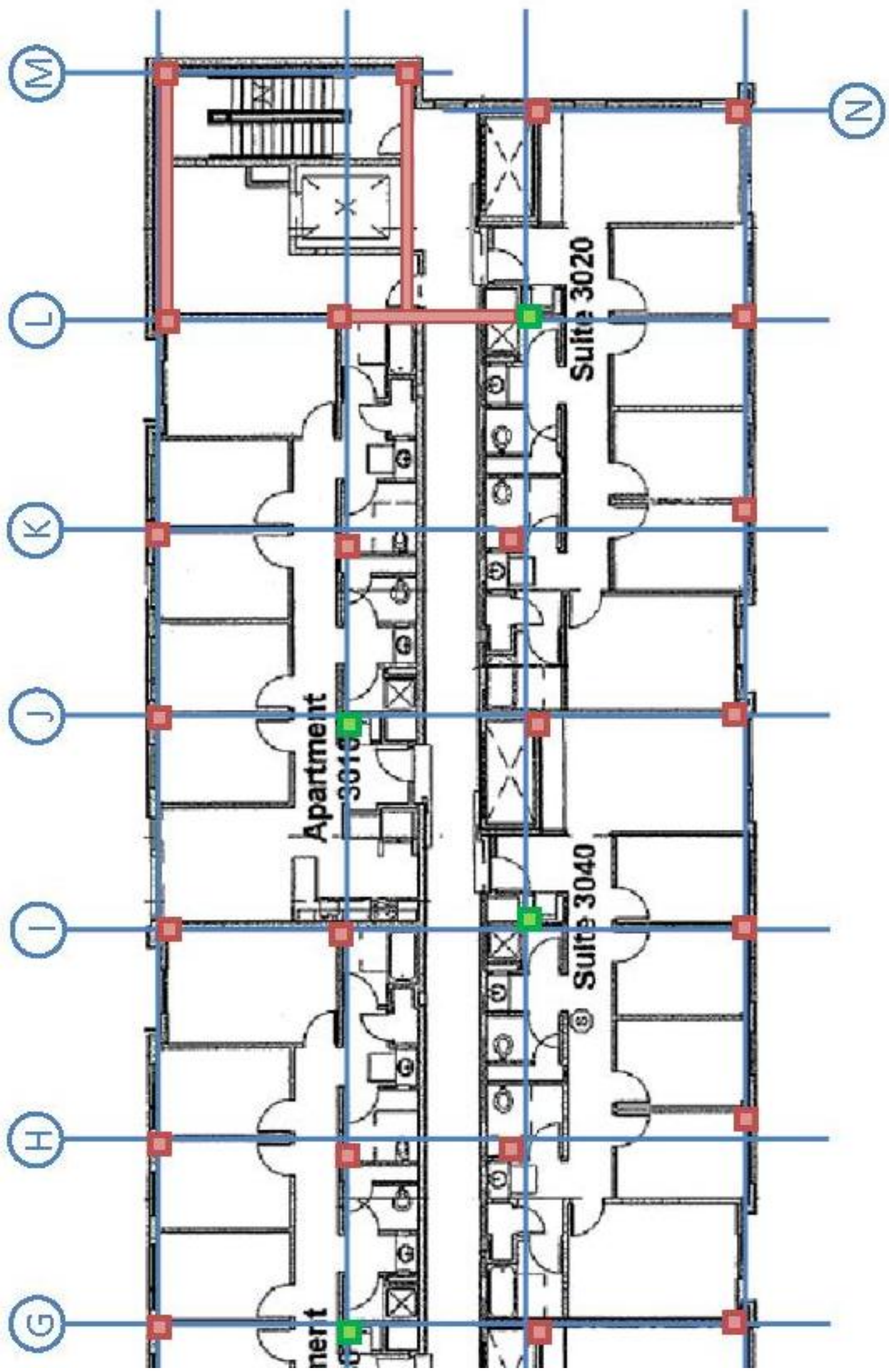
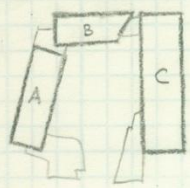
E
D
C
B
A
4
3
2
1
Apartment 3030
Suite 3060
Accessible Apartment 3050
Apartment 3070


M
N
L
K
J
I
H
G
Suite 3020
Apartment
3010
Suite 3040

Appendix B: Wind Load Calculations

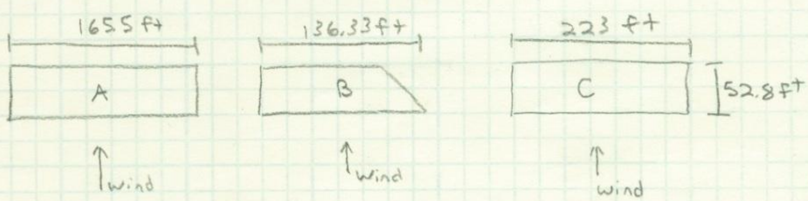
Total: 23 | Chris Vandelogt | Tech 1 | Wind Analysis | 1



- Thick outline represents the tallest height of each section of the structure
- To simplify, analyze the structure as 3 different buildings (outlined and labeled a, b, and c)

→ Dimensions

- Building A
  - Length: 165.5 ft
  - Width: 52.8 ft
  - Height: 51.83 ft
- Building B
  - Length: 136.33 ft
  - Width: 52.8 ft
  - Height: 62.5 ft
- Building C
  - Length: 223 ft
  - Width: 52.8 ft
  - Height: 62.5 ft



Total: 24 | Chris Vandelogt | Tech 1 | Wind Analysis | 2

→ Basic Wind Speed: From Figure 26.5-1B ASCE 7-10

$V = 120$ mph

→ Wind Directionality Factor: From Table 26.6-1

$k_d = .85$

→ Occupancy Category III

→ Exposure Category: C From Section 26.7.3

→ Topography Factor: From Section 26.8.2

$K_{zt} = 1.0$

→ Frequency: From Sect 26.9.2.1

$$L_{eff} = \frac{\Sigma h_i L_i}{\Sigma h_i} = 52.8$$

$h = 62.5 < 4(52.8)$ ← Allowed to use approx Freq

$n_a = 75/h$ (Equation 26.9-4)

$= 75/62.5$ or $75/50$

$= 1.2$ or $1.5 > 1.0$ ∴ Rigid

→ Gust Factor: From Sect 26.9

$$G = .925\left(\frac{1 + 1.7 g_Q I_{\bar{z}} Q}{1 + 1.7 g_v I_{\bar{z}}}\right)$$

where: $I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6}$

- $\bar{z} = .6h > z_{min} = 15'$ (Table 26.9-1) ✓ ok
- $c = .2$ (Table 26.9-1)

g_Q and $g_v = 3.4$

* See spreadsheet for calculations

Total: 25	Chris Vandelogt	Tech 1	Wind Analysis	3

$$Q = \sqrt{\frac{1}{1+.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{.63}}}$$

- $L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}}$
  - $\ell = 500$
  - $\bar{\epsilon} = 1/5$

→ *Note: Ignore internal pressure since net addition is zero and no large openings are located in the building

→ Velocity Pressure Exposure: From Table 27.3-1

k_z @ 14' = .85 k_z @ 37.33' = 1.024

k_z @ 26.66' = .953 k_z @ 51.83' = 1.097

k_z @ 48' = 1.08 k_z @ 62.5 = 1.14

→ Velocity Pressure: From Sect 27.3.2

$q_z = .00256\, K_z K_{zt} K_d V^2$

* See spreadsheet for Calculations

→ Wind Loads: From Section 27.4.1

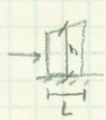
$p = qGC_p$

q_z for windward, q_h for sides and leeward

where $C_p = \begin{cases} .8 & \text{windward} \\ -.5 & \text{leeward} \\ -.7 & \text{sides} \end{cases}$ From Fig 27.4-1

$L/B < 1.0$

since roofs are monoslope:



use $h/L \geq 1.0$

$\theta < 10°$

$C_p = \begin{cases} 0 \text{ to } h/2: & -1.3,\ -.18 \\ > h/2: & -.7,\ -.18 \end{cases}$ ← worst cases

↑ From Fig 27.4-1

* See spreadsheet for Calculations

Total:26 Chris Vandelogt Tech 1 Wind Analysis 4

Wind Analysis -Wind Normal to Long Dimension (Length)

Building Dimensions				Gust Factor Calculations				
Building	Length (ft)	Width (ft)	Height (ft)	z_{bar}	I_{zbar}	L_{zbar}	Q	G
A	165.500	52.800	51.830	31.098	0.202	494.099	0.853	0.852
B	136.330	52.800	62.500	37.500	0.196	512.948	0.862	0.857
C	223.000	52.800	62.500	37.500	0.196	512.948	0.835	0.844

Constants

V (mph) =	120.000	$C_{p,windward}$ =	0.800	$C_{p,roof<h/2}$ =	-1.300
k_d =	0.850	$C_{p,leeward}$ =	-0.500	$C_{p,roof>h/2}$ =	-0.700
k_{zt} =	1.000	$C_{p,sides}$ =	-0.700		

Building A

Floor	Height	k_z	q_z (lb/ft^2)	p_{wind} (lb/ft^2)	p_{lee} (lb/ft^2)	p_{side} (lb/ft^2)	$p_{roof<h/2}$ (lb/ft^2)	$p_{roof>h/2}$ (lb/ft^2)
2nd	14.000	0.850	26.634	18.145	-14.636	-20.490		
3rd	26.660	0.953	29.862	20.344	-14.636	-20.490		
Penthouse	37.330	1.024	32.086	21.859	-14.636	-20.490		
Roof	51.830	1.097	34.374	23.418	-14.636	-20.490	-38.054	-20.490

Building B

Floor	Height	k_z	q_z (lb/ft^2)	p_{wind} (lb/ft^2)	p_{lee} (lb/ft^2)	p_{side} (lb/ft^2)	$p_{roof<h/2}$ (lb/ft^2)	$p_{roof>h/2}$ (lb/ft^2)
2nd	14.000	0.850	26.634	18.262	-15.308	-21.431		
3rd	26.660	0.953	29.862	20.475	-15.308	-21.431		
4th	37.330	1.024	32.086	22.001	-15.308	-21.431		
Penthouse	48.000	1.080	33.841	23.204	-15.308	-21.431		
Roof	62.500	1.140	35.721	24.493	-15.308	-21.431	-39.801	-21.431

Building C

Floor	Height	k_z	q_z (lb/ft^2)	p_{wind} (lb/ft^2)	p_{lee} (lb/ft^2)	p_{side} (lb/ft^2)	$p_{roof<h/2}$ (lb/ft^2)	$p_{roof>h/2}$ (lb/ft^2)
2nd	14.000	0.850	26.634	17.979	-15.071	-21.099		
3rd	26.660	0.953	29.862	20.158	-15.071	-21.099		
4th	37.330	1.024	32.086	21.659	-15.071	-21.099		
Penthouse	48.000	1.080	33.841	22.844	-15.071	-21.099		
Roof	62.500	1.140	35.721	24.113	-15.071	-21.099	-39.184	-21.099

Total: 27 Chris Vandelogt Tech 1 Wind Analysis 5

Wind Analysis -Wind Normal to Short Dimension (Width)

Building Dimensions				Gust Factor Calculations				
Building	Width (ft)	Length (ft)	Height (ft)	z_{bar}	I_{zbar}	L_{zbar}	Q	G
A	52.800	165.500	51.830	31.098	0.202	494.099	0.899	0.875
B	52.800	136.330	62.500	37.500	0.196	512.948	0.896	0.874
C	52.800	223.000	62.500	37.500	0.196	512.948	0.896	0.874

Constants					
V (mph) =	120.000	$C_{p,windward}$ =	0.800	$C_{p,roof<h/2}$ =	-1.300
k_d =	0.850	$C_{p,leeward}$ =	-0.500	$C_{p,roof>h/2}$ =	-0.700
k_{zt} =	1.000	$C_{p,sides}$ =	-0.700		

Building A

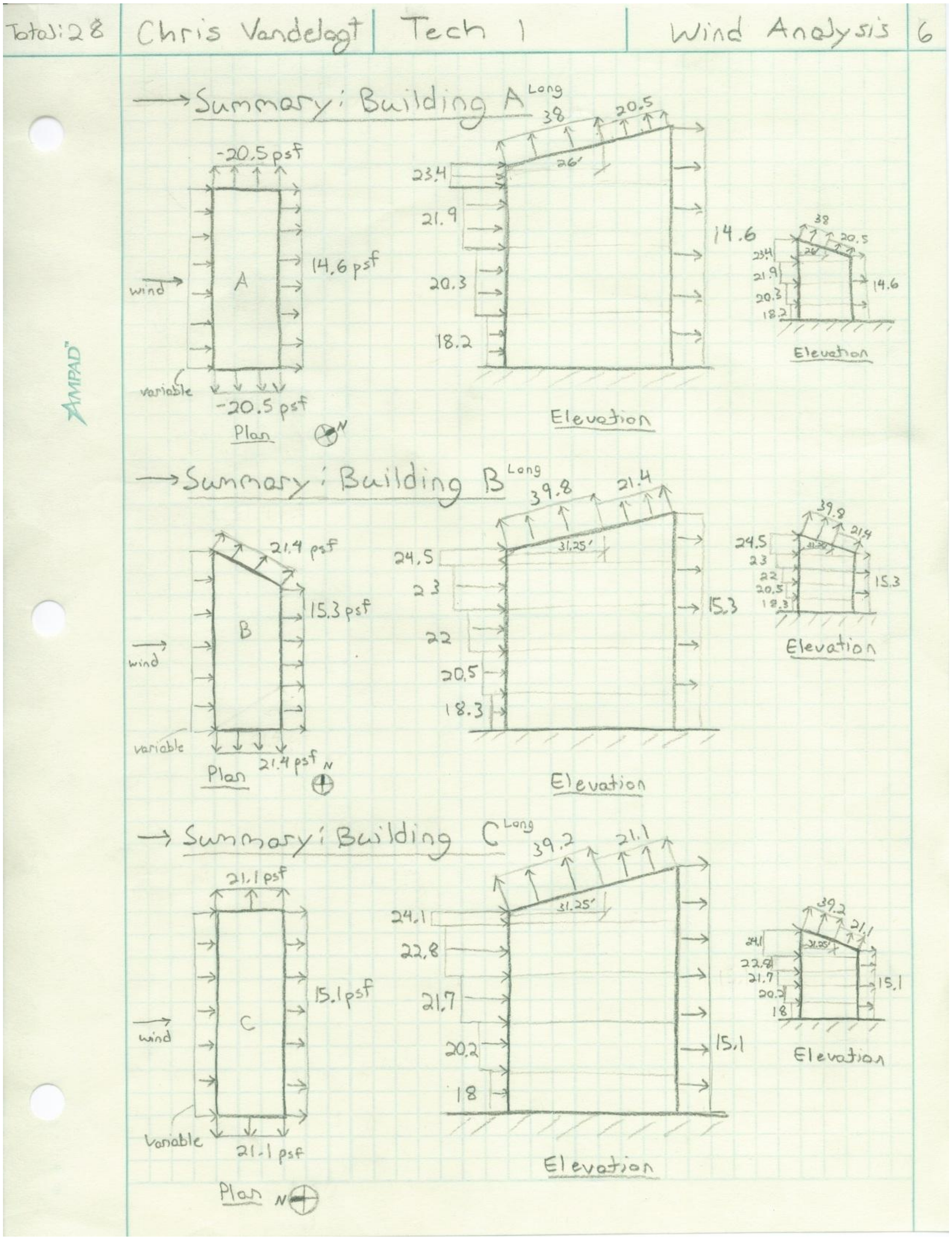
Floor	Height	k_z	q_z (lb/ft^2)	p_{wind} (lb/ft^2)	p_{lee} (lb/ft^2)	p_{side} (lb/ft^2)
2nd	14.000	0.850	26.634	18.639	-15.034	-21.048
3rd	26.660	0.953	29.862	20.897	-15.034	-21.048
Penthouse	37.330	1.024	32.086	22.454	-15.034	-21.048
Roof	51.830	1.097	34.374	24.055	-15.034	-21.048

Building B

Floor	Height	k_z	q_z (lb/ft^2)	p_{wind} (lb/ft^2)	p_{lee} (lb/ft^2)	p_{side} (lb/ft^2)
2nd	14.000	0.850	26.634	18.620	-15.608	-21.851
3rd	26.660	0.953	29.862	20.876	-15.608	-21.851
4th	37.330	1.024	32.086	22.431	-15.608	-21.851
Penthouse	48.000	1.080	33.841	23.658	-15.608	-21.851
Roof	62.500	1.140	35.721	24.972	-15.608	-21.851

Building C

Floor	Height	k_z	q_z (lb/ft^2)	p_{wind} (lb/ft^2)	p_{lee} (lb/ft^2)	p_{side} (lb/ft^2)
2nd	14.000	0.850	26.634	18.620	-15.608	-21.851
3rd	26.660	0.953	29.862	20.876	-15.608	-21.851
4th	37.330	1.024	32.086	22.431	-15.608	-21.851
Penthouse	48.000	1.080	33.841	23.658	-15.608	-21.851
Roof	62.500	1.140	35.721	24.972	-15.608	-21.851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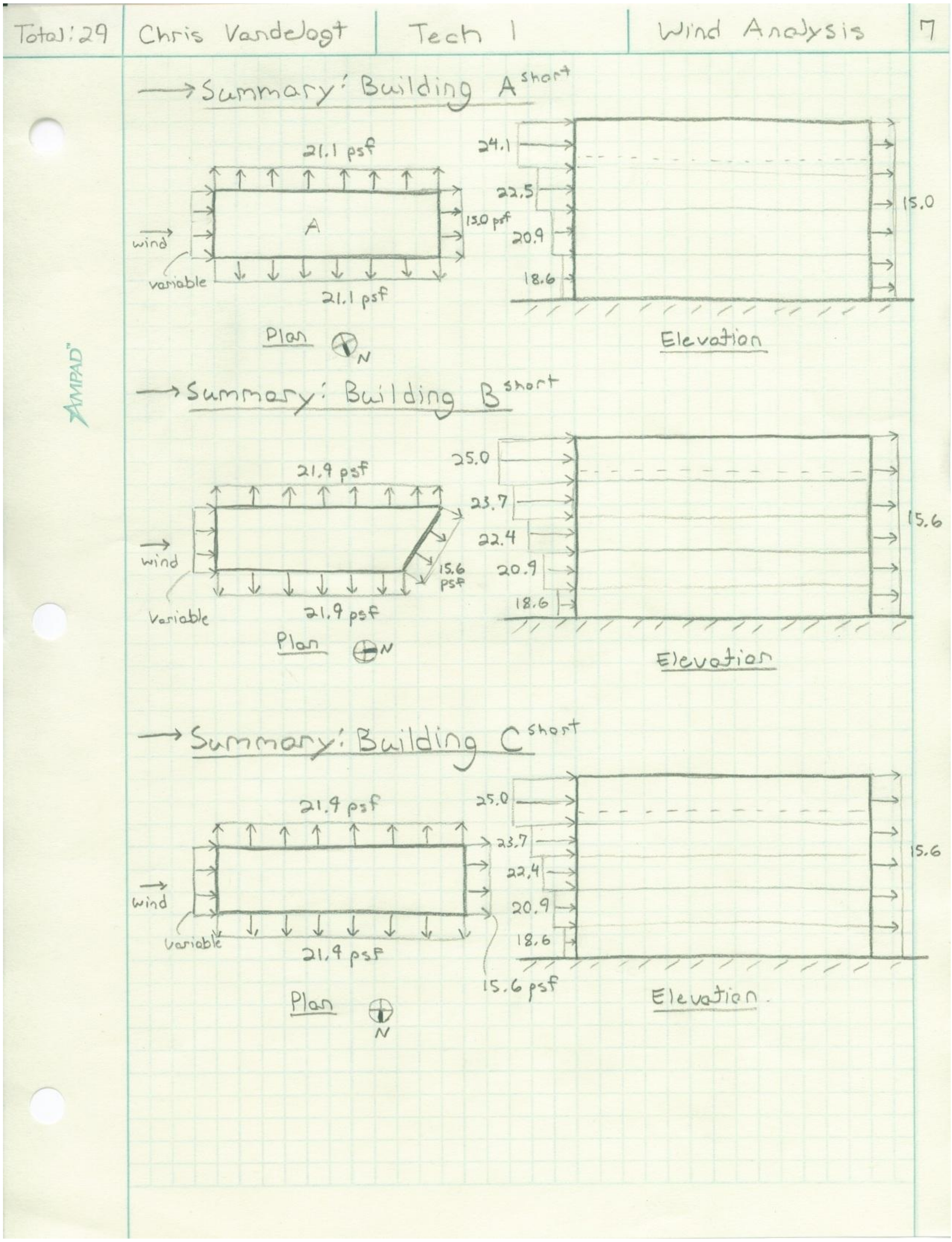
Chris Vandelogt
Tech 1
Wind Analysis
6
→ Summary: Building A Long
-20.5 psf
14.6 psf
wind
A
variable
-20.5 psf
Plan
38
20.5
26'
23.4
21.9
20.3
18.2
14.6
Elevation
→ Summary: Building B Long
21.4 psf
15.3 psf
B
wind
variable
21.4 psf
Plan
39.8
21.4
31.25'
24.5
23
22
20.5
18.3
15.3
Elevation
→ Summary: Building C Long
21.1 psf
15.1 psf
C
wind
Variable
21.1 psf
Plan
39.2
21.1
31.25'
24.1
22.8
21.7
20.2
18
15.1
Elevation


Total: 29
Chris Vandelogt
Tech 1
Wind Analysis
7
Summary: Building A short
21.1 psf
A
wind
variable
13.0 psf
21.1 psf
Plan
N
24.1
22.5
20.9
18.6
15.0
Elevation
Summary: Building B short
21.9 psf
wind
Variable
15.6 psf
21.9 psf
Plan
N
25.0
23.7
22.4
20.9
18.6
15.6
Elevation
Summary: Building C short
21.9 psf
wind
Variable
21.9 psf
15.6 psf
Plan
N
25.0
23.7
22.4
20.9
18.6
15.6
Elevation

Appendix C: Seismic Load Calculations

Variable	Value	Reference
I_e=	1.25	Table 1.5-2
S_s=	0.21	USGS
S_1=	0.06	USGS
Site Class:	C	Geotech Report
F_a=	1.2	Table 11.4-1
F_v=	1.7	Table 11.4-2
S_{ms}=	0.252	
S_{m1}=	0.102	
S_{DS}=	0.168	
S_{D1}=	0.068	
Category:	B	Table 11.6-1,2
R=	3	Table 12.2-1: Ordinary RC Moment Frame
T_L=	6 sec	Fig 22-12

Equivalent Lateral Force Procedure			
C_t=	0.02		Table 12.8-2: Other Structures
x=	0.75		
h_n=	62.5	ft	
T_a=	0.445	sec	
C_u=	1.7		Table 12.8-1
T=	0.756	sec	
k=	1.128		
C_S=	0.070		
$C_{S,max}$=	0.037		
$C_{S,min}$=	0.010		
Use C_S=	0.037		

Weight of Floors

1st Floor:			2nd Floor:			3rd Floor:		
SDL=	5	psf	SDL=	5	psf	SDL=	5	psf
MEP=	10	psf	MEP=	10	psf	MEP=	10	psf
Partitions=	15	psf	Partitions=	15	psf	Partitions=	15	psf
Slab=	106.3	psf	Ceiling=	5	psf	Ceiling=	5	psf
MEP Equip=	150	psf	Slab=	106.3	psf	Slab=	106.3	psf
A_{Mech}=	4314	ft^2	A_{Total}=	12456	ft^2	A_{Total}=	12456	ft^2
A_{Other}=	12456	ft^2						
Weight:	2345	kips	Weight:	1760	kips	Weight:	1760	kips

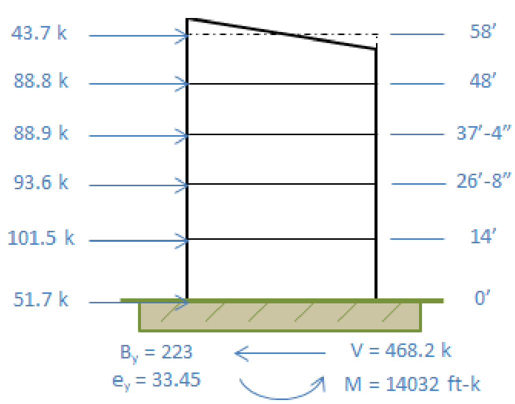
4th Floor:			Penthouse:			Roof:		
SDL=	5	psf	SDL=	5	psf	SDL=	5	psf
MEP=	10	psf	MEP=	10	psf	Framing=	15	psf
Partitions=	15	psf	Partitions=	20	psf	Insulation=	3	psf
Ceiling=	0	psf	Ceiling=	0	psf	20% Snow=	6.16	psf
Slab=	106.3	psf	Slab=	106.3	psf			
			MEP Equip=	150	psf			
A_{Total}=	12456	ft^2	A_{Mech}=	744	ft^2	A_{Total}=	11487	ft^2
			A_{Other}=	11487	ft^2			
Weight:	1698	kips	Weight:	1735	kips	Weight:	335	kips

Seismic Forces

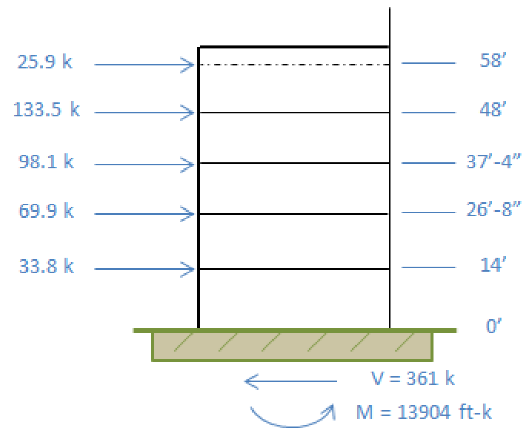
Building C							
Floor	Floor Weight, w_x (k)	Story Height, h_x (ft)	$w_x h_x^k$	C_{vx}	Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)
Ground	2345	0.0	0.00	0.00	0.0	361.1	0.0
2nd	1760	14.0	89799.12	0.09	33.8	361.1	473.2
3rd	1760	26.7	185685.30	0.19	69.9	327.3	1863.3
4th	1698	37.3	260630.10	0.27	98.1	257.4	3662.1
Pent	1735	48.0	354584.69	0.37	133.5	159.3	6406.4
Roof	335	58.0	68682.22	0.07	25.9	25.9	1499.4
Sum:	9632		959381.4	1.00	361.1		
				√ ok	√ ok		
	Base Shear ($V=C_sW$) =		361	**Total Overturning Moment =**			13904

Appendix D: Story Loads

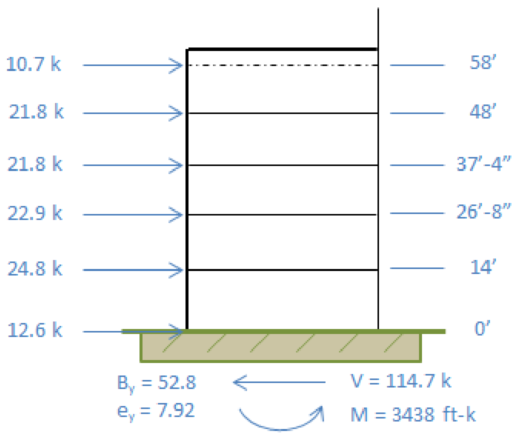
Wind: Y-Axis Loads



Seismic Loads



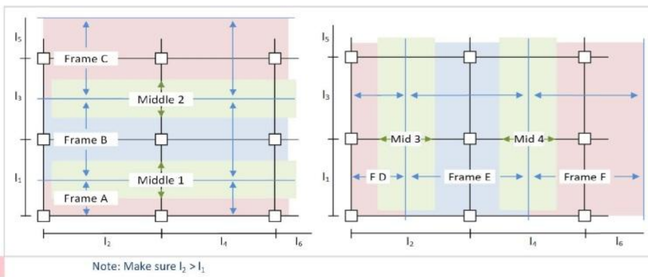
Wind: X-Axis Loads



Appendix E: Column Calculations

Flat Plate With No Edge Beams (By Direct Design Method)

$l_{max,int}$=	20	ft	l_1=	19.163	ft
$l_{max,ext}$=	20	ft	l_3=	17	ft
f'_c=	4000	psi	l_5=	16.67	ft
f_y=	60000	psi	l_2=	16.833	ft
w_{SD}=	35	psf	l_4=	19	ft
w_L=	100	psf	l_6=	20	ft
$t_{col,1dir}$=	20	in	$Width_{FA}$=	9.5815	ft
$t_{col,2dir}$=	20	in	$Width_{FB}$=	18.0815	ft
$l_{n,1}$=	17.50	ft	$Width_{FC}$=	16.835	ft
$l_{n,2}$=	15.17	ft	$Width_{FD}$=	8.4165	ft
$l_{n,3}$=	15.33	ft	$Width_{FE}$=	17.9165	ft
$l_{n,4}$=	17.33	ft	$Width_{FF}$=	19.5	ft
$l_{n,5}$=	15.00	ft	Need to change orientation so l2 > l1		
$l_{n,6}$=	18.33	ft			

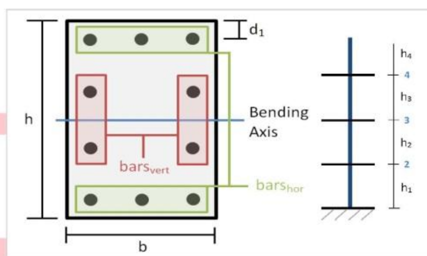


Note: Make sure $l_2 > l_1$

Column Design of Ground Floor Columns

Trial Column			
b=	20	in	
h=	20	in	
Use #	10	bars	
d_1=	2.5	in	
$bars_{vert}$=	2		
$bars_{hor}$=	6		
Floors=	5		
Note: Includes roof but not ground			
h_5=	10	ft	
h_4=	10.67	ft	
h_3=	10.67	ft	
h_2=	12.66	ft	
h_1=	14	ft	

Roof Slope=	2	/ 12	
$w_{SD,roof}$=	23	psf	
$w_{L,roof}$=	20	psf	
w_{snow}=	30.8	psf	
$w_{SD,5}$=	35	psf	
$w_{L,5}$=	150	psf	Non-Reducible
$w_{SD,4}$=	30	psf	
$w_{L,4}$=	40	psf	
$w_{SD,3}$=	35	psf	
$w_{L,3}$=	40	psf	
$w_{SD,2}$=	35	psf	
$w_{L,2}$=	100	psf	Non-Reducible
$w_{SD,ground}$=	N/A	psf	
$w_{L,ground}$=	N/A	psf	



Column Strength / Strength Interaction Curve

Pure Compression		
P_o=	1915.7	kips
ϕP_o=	1245.2	kips

Pure Tension		
T_o=	-589.0	kips
ϕT_o=	-530.1	kips

Pure Bending (Solve by Hand)

Balanced-Strain Strength

ε_y=	0.00207			β_1=	0.85	
c=	10.36	in < h	OK	A_s=	1.227	in^2
d_1=	2.50	in		f_{s1}=	60.00	ksi
d_2=	10.00	in		f_{s2}=	3.00	ksi
d_3=	17.50	in		f_{s3}=	-60.00	ksi
d_4=		in		f_{s4}=		ksi
d_5=		in		f_{s5}=		ksi
d_6=		in		f_{s6}=		ksi
d_7=		in		f_{s7}=		ksi
d_8=		in		f_{s8}=		ksi
P_b=	606.0	kips		M_b=	555.4	ft-k
ϕP_b=	393.9	kips		ϕM_b=	361.0	ft-k

Column BD			
$t_{col,1dir}$=	20	in	
$t_{col,2dir}$=	20	in	
A_T=	608.731779	ft^2	
$A_{T,roof}$=	152.182945	ft^2	
$K_{LL}A_T$=	2434.92712	ft^2	
$K_{LL}A_T$ >	$400ft^2$		OK
α=	0.55		
α_{roof}=	1.00		

Column BE			
$t_{col,1dir}$=	20	in	
$t_{col,2dir}$=	20	in	
A_T=	1295.82878	ft^2	
$A_{T,roof}$=	323.957195	ft^2	
$K_{LL}A_T$=	5183.31512	ft^2	
$K_{LL}A_T$ >	$400ft^2$		OK
α=	0.46		
α_{roof}=	0.88		

Column BF			
$t_{col,1dir}$=	20	in	
$t_{col,2dir}$=	20	in	
A_T=	1410.357	ft^2	
$A_{T,roof}$=	352.58925	ft^2	
$K_{LL}A_T$=	5641.428	ft^2	
$K_{LL}A_T$ >	$400ft^2$		OK
α=	0.45		
α_{roof}=	0.85		

Column BD		
$M_{ETABS,long}$=	96	ft-k
$M_{ETABS,short}$=	68	ft-k
$M_{unb,long}$=	31.7	ft-k
$M_{unb,short}$=	13.3	ft-k
P_L=	44.8	kips
P_D=	105.9	kips
$P_{S,Lr}$=	7.7	kips
$M_{u,long}$=	127.7	ft-k
$M_{u,short}$=	81.3	ft-k
P_T=	202.6	kips

Column BE		
$M_{ETABS,long}$=	96	ft-k
$M_{ETABS,short}$=	68	ft-k
$M_{unb,long}$=	27.3	ft-k
$M_{unb,short}$=	27.5	ft-k
P_L=	92.9	kips
P_D=	206.0	kips
$P_{S,Lr}$=	15.7	kips
$M_{u,long}$=	123.3	ft-k
$M_{u,short}$=	95.5	ft-k
P_T=	403.6	kips

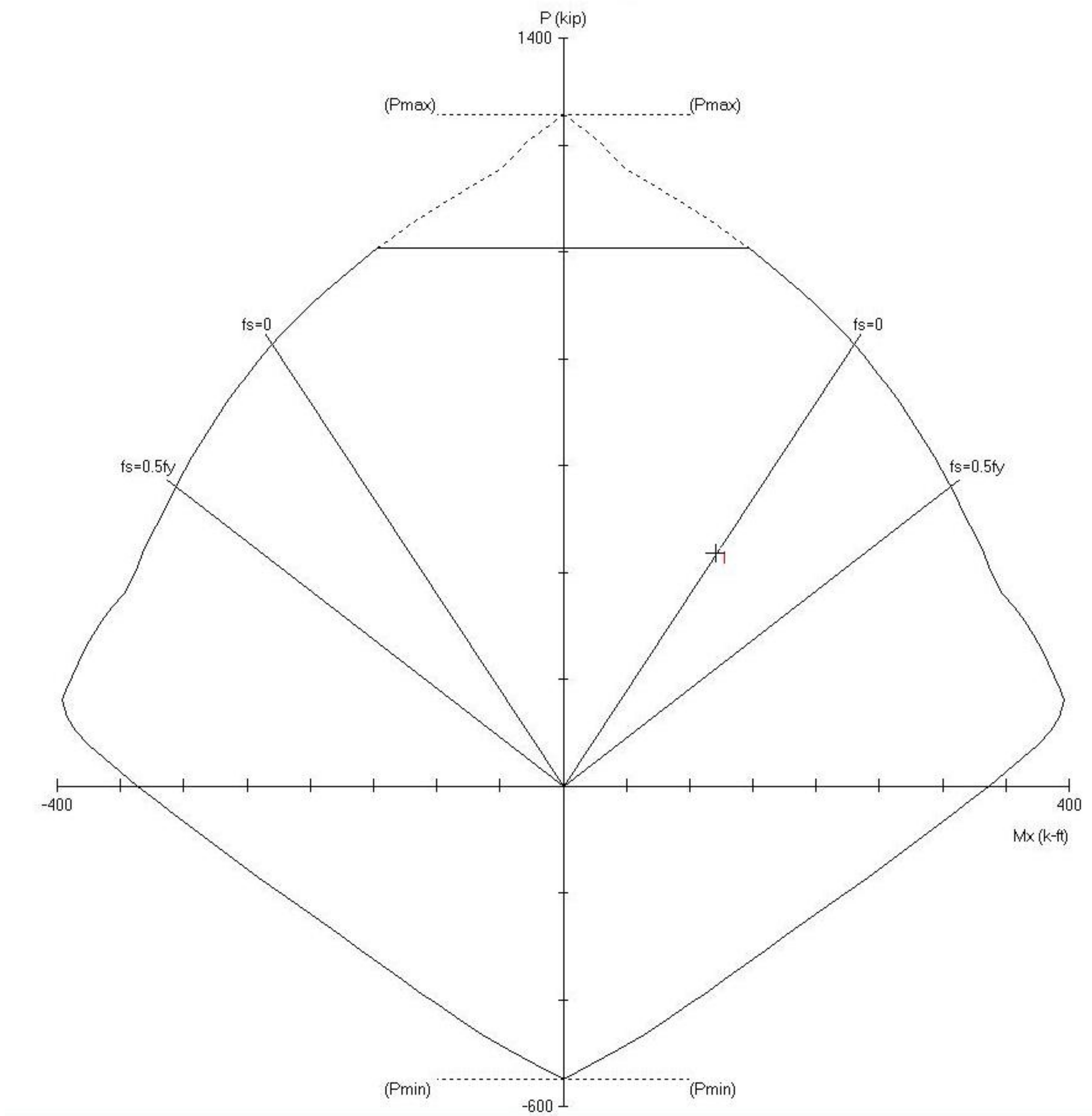
Column BF		
$M_{ETABS,long}$=	96	ft-k
$M_{ETABS,short}$=	68	ft-k
$M_{unb,long}$=	25.0	ft-k
$M_{unb,short}$=	29.9	ft-k
P_L=	100.8	kips
P_D=	222.7	kips
$P_{S,Lr}$=	16.8	kips
$M_{u,long}$=	121.0	ft-k
$M_{u,short}$=	97.9	ft-k
P_T=	437.0	kips

Interior Column BF (Reinforcement Needed)

$t_{col,1dir}$=	20	in	b_o=	108.50	in
$t_{col,2dir}$=	20	in	b_1=	27.13	in
$M_{u,long}$=	41.7	ft-k	b_2=	27.13	in
$M_{u,short}$=	49.9	ft-k	$V_{c,1}$=	195.6	kips
			$V_{c,2}$=	293.4	kips
			$V_{c,3}$=	226.2	kips
V_u=	116.2	kips	ϕV_c=	146.7	kips

Transferred by Eccentricity of Shear

V_u=	116.2	kips					V_u=	116.2	kips				
$M_{uv,long}$=	16.7	ft-k					$M_{uv,short}$=	19.9	ft-k				
Centroid=	13.56	in					Centroid=	13.56	in				
J_c=	96434	in^4					J_c=	96434	in^4				
A_c=	773	in^2					A_c=	773	in^2				
ν_l=	122	psi					ν_l=	117	psi				
ν_r=	178	psi					ν_r=	184	psi				
ν_u=	178	psi					ν_u=	184	psi				
$\phi\nu_n$=	190	psi	>	ν_u	OK		$\phi\nu_n$=	190	psi	>	ν_u	OK	

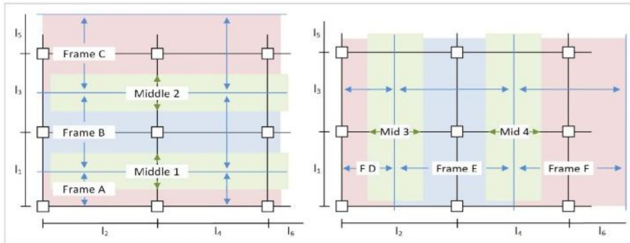

P (kip)
1400
(Pmax)
(Pmax)
fs=0
fs=0
fs=0.5fy
fs=0.5fy
-400
400
Mx (k-ft)
(Pmin)
(Pmin)
-600

Appendix F: Slab Thickness Calculations

Flat Plate With No Edge Beams (By Direct Design Method)

$l_{max,int}$=	20	ft	l_1=	19.163	ft
$l_{max,ext}$=	20	ft	l_3=	17	ft
f'_c=	4000	psi	l_5=	16.67	ft
f_y=	60000	psi	l_2=	19	ft
w_{SD}=	35	psf	l_4=	20	ft
w_L=	100	psf	l_6=	16.833	ft
$t_{col,1dir}$=	20	in	$Width_{FA}$=	9.5815	ft
$t_{col,2dir}$=	20	in	$Width_{FB}$=	18.0815	ft
$l_{n,1}$=	17.50	ft	$Width_{FC}$=	16.835	ft
$l_{n,2}$=	17.33	ft	$Width_{FD}$=	9.5	ft
$l_{n,3}$=	15.33	ft	$Width_{FE}$=	19.5	ft
$l_{n,4}$=	18.33	ft	$Width_{FF}$=	18.4165	ft
$l_{n,5}$=	15.00	ft			
$l_{n,6}$=	15.17	ft			

Need to change orientation so l2 > l1



Note: Make sure $l_2 > l_1$

Slab Thickness

t_{input}=	0.00	in			
$t_{min,int}$=	6.67	in			
$t_{min,ext}$=	8.07	in			
Use t_{slab}=	8.50	in	>	5"	OK

Wide Beam Action

$l_{max,FB}$=	18.1	ft	$l_{max,1dir}$=	19.2	ft
$l_{max,2dir}$=	20	ft	$l_{max,FE}$=	19.5	ft
d_{avg}=	7.13	in			
w_u=	329.5	psf			

Note: Dimensions from Same Bay

Long Direction

V_u=	51.1	kips			
ϕV_c=	146.8	kips	>	V_u	OK

Short Direction

V_u=	52.5	kips			
ϕV_c=	158.2	kips	>	V_u	OK

Punching Shear

	$l_{max,FB}$=	18.1	ft			
	$l_{max,FE}$=	19.5	ft			
	V_u=	114.6	kips			
	b_o=	108.5	in			
Take Minimum Value	$V_{c,1}$=	195.6	kips			
	$V_{c,2}$=	293.4	kips			
	$V_{c,3}$=	226.2	kips			
	ϕV_c=	146.7	kips	>	V_u	OK

Deflection Check

Assume:	25	% of w_L is sustained
	90	% of immediate deflection due to dead load occurs before partitions are installed
	x	Check if: Nonstructural attached elements will be damaged by excessive deflection

Interior Panel l_3 - l_4

Column Strip			Middle Strip		
$I_{g,col}$=	5552	in^4	$I_{g,mid}$=	7062	in^4
w_D=	1.724	k/ft	w_D=	0.918	k/ft
w_L=	1.221	k/ft	w_L=	0.650	k/ft
$\Delta_{D,max}$=	0.062	in	$\Delta_{D,max}$=	0.014	in
$\Delta_{L,max}$=	0.081	in	$\Delta_{L,max}$=	0.018	in
$\Delta_{long-term}$=	0.246	in	$\Delta_{long-term}$=	0.054	in

Check Live Load Deflection

Δ_L=	0.099	in	
ACI Limit=	0.667	in	OK

Check Total Load Deflection

Δ_T=	0.406	in	
ACI Limit=	0.500	in	OK

Exterior Panel l_1 - l_2

Column Strip			Middle Strip		
$I_{g,col}$=	5552	in^4	$I_{g,mid}$=	5834	in^4
w_D=	1.724	k/ft	w_D=	0.872	k/ft
w_L=	1.221	k/ft	w_L=	0.618	k/ft
$\Delta_{D,max}$=	0.050	in	$\Delta_{D,max}$=	0.025	in
$\Delta_{L,max}$=	0.066	in	$\Delta_{L,max}$=	0.033	in
$\Delta_{long-term}$=	0.201	in	$\Delta_{long-term}$=	0.100	in

Check Live Load Deflection

Δ_L=	0.099	in	
ACI Limit=	0.633	in	OK

Check Total Load Deflection

Δ_T=	0.407	in	
ACI Limit=	0.475	in	OK

Exterior Panel l_3 - l_2

Column Strip			Middle Strip		
$I_{g,col}$=	5552	in^4	$I_{g,mid}$=	6448	in^4
w_D=	1.724	k/ft	w_D=	0.872	k/ft
w_L=	1.221	k/ft	w_L=	0.618	k/ft
$\Delta_{D,max}$=	0.050	in	$\Delta_{D,max}$=	0.014	in
$\Delta_{L,max}$=	0.066	in	$\Delta_{L,max}$=	0.026	in
$\Delta_{long\text{-}term}$=	0.201	in	$\Delta_{long\text{-}term}$=	0.062	in

Check Live Load Deflection

Δ_L=	0.092	in	
ACI Limit=	0.633	in	OK

Check Total Load Deflection

Δ_T=	0.361	in	
ACI Limit=	0.475	in	OK

Exterior Panel l_1 - l_4

Column Strip			Middle Strip		
$I_{g,col}$=	5859	in^4	$I_{g,mid}$=	5884	in^4
w_D=	1.859	k/ft	w_D=	0.880	k/ft
w_L=	1.316	k/ft	w_L=	0.623	k/ft
$\Delta_{D,max}$=	0.053	in	$\Delta_{D,max}$=	0.030	in
$\Delta_{L,max}$=	0.070	in	$\Delta_{L,max}$=	0.039	in
$\Delta_{long\text{-}term}$=	0.212	in	$\Delta_{long\text{-}term}$=	0.119	in

Check Live Load Deflection

Δ_L=	0.109	in	
ACI Limit=	0.667	in	OK

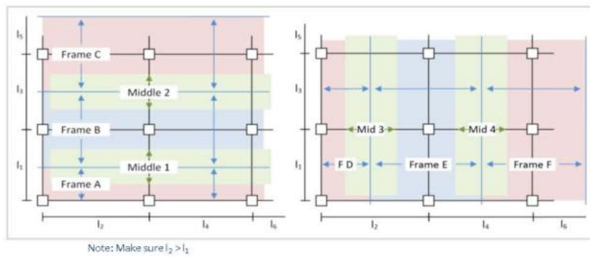
Check Total Load Deflection

Δ_T=	0.448	in	
ACI Limit=	0.500	in	OK

Appendix G: 2nd Floor Reinf Calcs

Flat Plate With No Edge Beams (By Direct Design Method)

$l_{max,int}$=	20	ft	l_1=	16.833	ft
$l_{max,ext}$=	20	ft	l_3=	15.166	ft
f'_c=	4000	psi	l_5=	20	ft
f_y=	60000	psi	l_2=	19.163	ft
w_{SD}=	35	psf	l_4=	17	ft
w_L=	100	psf	l_6=	16.67	ft
$t_{col,1dir}$=	20	in	$Width_{FA}$=	8.4165	ft
$t_{col,2dir}$=	20	in	$Width_{FB}$=	15.9995	ft
l_{n1}=	15.17	ft	$Width_{FC}$=	17.583	ft
l_{n2}=	17.50	ft	$Width_{FD}$=	9.5815	ft
l_{n3}=	13.50	ft	$Width_{FE}$=	18.0815	ft
l_{n4}=	15.33	ft	$Width_{FF}$=	16.835	ft
l_{n5}=	18.33	ft			
l_{n6}=	15.00	ft			



Slab Thickness

t_{input}=	0.00	in			
$t_{min,int}$=	6.67	in			
$t_{min,ext}$=	8.07	in			
Use t_{slab}=	8.50	in	>	5"	OK

Wide Beam Action

$l_{max,FB}$=	18.1	ft	$l_{max,1dir}$=	19.2	ft
$l_{max,2dir}$=	20	ft	$l_{max,FE}$=	19.5	ft
d_{avg}=	7.13	in			
w_u=	329.5	psf			

Note: Dimensions from Same Bay

Long Direction

V_u=	51.1	kips			
ϕV_c=	146.8	kips	>	V_u	OK

Short Direction

V_u=	52.5	kips			
ϕV_c=	158.2	kips	>	V_u	OK

Punching Shear

$l_{max,FB}$=	18.1	ft			
$l_{max,FE}$=	19.5	ft			
V_u=	114.6	kips			
b_o=	108.5	in			
V_{c1}=	195.6	kips			
V_{c2}=	293.4	kips			
V_{c3}=	226.2	kips			
ϕV_c=	146.7	kips	>	V_u	OK

Take Minimum Value

Deflection Check

Assume:	25	% of w_L is sustained
	90	% of immediate deflection due to dead load occurs before partitions are installed
	x	Check if: Nonstructural attached elements will be damaged by excessive deflection

Interior Panel l_3 - l_4

Column Strip			Middle Strip		
$I_{g,col}$=	5399	in^4	$I_{g,mid}$=	5783	in^4
w_D=	1.676	k/ft	w_D=	0.780	k/ft
w_L=	1.187	k/ft	w_L=	0.553	k/ft
$\Delta_{D,max}$=	0.032	in	$\Delta_{D,max}$=	0.009	in
$\Delta_{L,max}$=	0.042	in	$\Delta_{L,max}$=	0.012	in
$\Delta_{long\text{-}term}$=	0.129	in	$\Delta_{long\text{-}term}$=	0.035	in

Check Live Load Deflection

Δ_L=	0.054	in	
ACI Limit=	0.567	in	OK

Check Total Load Deflection

Δ_T=	0.222	in	
ACI Limit=	0.425	in	OK

Exterior Panel l_1 - l_2

Column Strip			Middle Strip		
$I_{g,col}$=	4913	in^4	$I_{g,mid}$=	6600	in^4
w_D=	1.525	k/ft	w_D=	0.880	k/ft
w_L=	1.080	k/ft	w_L=	0.623	k/ft
$\Delta_{D,max}$=	0.052	in	$\Delta_{D,max}$=	0.013	in
$\Delta_{L,max}$=	0.068	in	$\Delta_{L,max}$=	0.017	in
$\Delta_{long\text{-}term}$=	0.208	in	$\Delta_{long\text{-}term}$=	0.053	in

Check Live Load Deflection

Δ_L=	0.086	in	
ACI Limit=	0.639	in	OK

Check Total Load Deflection

Δ_T=	0.353	in	
ACI Limit=	0.479	in	OK

Exterior Panel $l_3 \cdot l_2$

Column Strip			Middle Strip		
$I_{g,col}$=	5399	in^4	$I_{g,mid}$=	7112	in^4
w_D=	1.676	k/ft	w_D=	0.880	k/ft
w_L=	1.187	k/ft	w_L=	0.623	k/ft
$\Delta_{D,max}$=	0.052	in	$\Delta_{D,max}$=	0.008	in
$\Delta_{L,max}$=	0.068	in	$\Delta_{L,max}$=	0.015	in
$\Delta_{long-term}$=	0.208	in	$\Delta_{long-term}$=	0.036	in

Check Live Load Deflection

Δ_L=	0.083	in	
ACI Limit=	0.639	in	OK

Check Total Load Deflection

Δ_T=	0.333	in	
ACI Limit=	0.479	in	OK

Exterior Panel $l_1 \cdot l_4$

Column Strip			Middle Strip		
$I_{g,col}$=	5169	in^4	$I_{g,mid}$=	5169	in^4
w_D=	1.724	k/ft	w_D=	0.773	k/ft
w_L=	1.221	k/ft	w_L=	0.547	k/ft
$\Delta_{D,max}$=	0.033	in	$\Delta_{D,max}$=	0.016	in
$\Delta_{L,max}$=	0.044	in	$\Delta_{L,max}$=	0.020	in
$\Delta_{long-term}$=	0.133	in	$\Delta_{long-term}$=	0.062	in

Check Live Load Deflection

Δ_L=	0.064	in	
ACI Limit=	0.567	in	OK

Check Total Load Deflection

Δ_T=	0.264	in	
ACI Limit=	0.425	in	OK

Total Static Moment

w_u=	329.5	psf		
	l_1 / l_2		l_3 / l_4	
$M_{0,A}$=	106.1	ft-k	81.5	ft-k
$M_{0,B}$=	201.7	ft-k	154.9	ft-k
$M_{0,C}$=	221.7	ft-k	170.3	ft-k
$M_{0,D}$=	90.8	ft-k	71.9	ft-k
$M_{0,E}$=	171.3	ft-k	135.7	ft-k
$M_{0,F}$=	159.5	ft-k	126.4	ft-k

Longitudinal Moments (ft-k)

Frame A:	-26.5	55.2	-74.3	-53.0	28.5	-53.0
Frame B:	-50.4	104.9	-141.2	-100.7	54.2	-100.7
Frame C:	-55.4	115.3	-155.2	-110.7	59.6	-110.7
Frame D:	-22.7	47.2	-63.5	-46.7	25.2	-46.7
Frame E:	-42.8	89.1	-119.9	-88.2	47.5	-88.2
Frame F:	-39.9	82.9	-111.6	-82.1	44.2	-82.1

α=	0				Since flat plate (no beams)			
$C_{A,B,C}$=	2998.0	in^4			$C_{D,E,F}$=	2998.0	in^4	
$I_{s,A}$=	5169	in^4			$I_{s,D}$=	5884	in^4	
$I_{s,B}$=	9826	in^4			$I_{s,E}$=	11104	in^4	
$I_{s,C}$=	10798	in^4			$I_{s,F}$=	10339	in^4	
$\beta_{t,A}$=	0.2900	<	2.5	so	Use % col strip value below			
$\beta_{t,B}$=	0.1526	<	2.5	so	Use % col strip value below			
$\beta_{t,C}$=	0.1388	<	2.5	so	Use % col strip value below			
$\beta_{t,D}$=	0.2547	<	2.5	so	Use % col strip value below			
$\beta_{t,E}$=	0.1350	<	2.5	so	Use % col strip value below			
$\beta_{t,F}$=	0.1450	<	2.5	so	Use % col strip value below			

Frame A	Ext	Col Strip=	97.1	%
		Mid Strip=	2.9	%
	Pos	Col Strip=	60.0	%
		Mid Strip=	40.0	%
	Int	Col Strip=	75.0	%
		Mid Strip=	25.0	%
Frame B	Ext	Col Strip=	98.5	%
		Mid Strip=	1.5	%
	Pos	Col Strip=	60.0	%
		Mid Strip=	40.0	%
	Int	Col Strip=	75.0	%
		Mid Strip=	25.0	%

Summary of Moments (ft-k)

Frame A:	Col Strip:	4.2	ft	Col Strip:	4.2	ft
	Mid Strip:	4.2	ft	Mid Strip:	4.2	ft
M_{tot}=	-26.5	55.2	-74.3	-53.0	28.5	-53.0
M_{col}=	-25.8	33.1	-55.7	-39.7	17.1	-39.7
M_{mid}=	-0.8	22.1	-18.6	-13.2	11.4	-13.2
Frame B:	Col Strip:	8.0	ft	Col Strip:	8.0	ft
	Mid Strip:	8.0	ft	Mid Strip:	8.0	ft
M_{tot}=	-50.4	104.9	-141.2	-100.7	54.2	-100.7
M_{col}=	-49.7	62.9	-105.9	-75.5	32.5	-75.5
M_{mid}=	-0.8	42.0	-35.3	-25.2	21.7	-25.2
Frame C:	Col Strip:	8.8	ft	Col Strip:	8.8	ft
	Mid Strip:	8.8	ft	Mid Strip:	8.8	ft
M_{tot}=	-55.4	115.3	-155.2	-110.7	59.6	-110.7
M_{col}=	-54.7	69.2	-116.4	-83.0	35.8	-83.0
M_{mid}=	-0.8	46.1	-38.8	-27.7	23.8	-27.7
Frame D:	Col Strip:	4.2	ft	Col Strip:	3.8	ft
	Mid Strip:	5.4	ft	Mid Strip:	5.8	ft
M_{tot}=	-22.7	47.2	-63.5	-46.7	25.2	-46.7
M_{col}=	-22.1	28.3	-47.7	-35.1	15.1	-35.1
M_{mid}=	-0.6	18.9	-15.9	-11.7	10.1	-11.7

Frame C	Ext	Col Strip=	98.6	%
		Mid Strip=	1.4	%
	Pos	Col Strip=	60.0	%
		Mid Strip=	40.0	%
	Int	Col Strip=	75.0	%
		Mid Strip=	25.0	%
Frame D	Ext	Col Strip=	97.5	%
		Mid Strip=	2.5	%
	Pos	Col Strip=	60.0	%
		Mid Strip=	40.0	%
	Int	Col Strip=	75.0	%
		Mid Strip=	25.0	%
Frame E	Ext	Col Strip=	98.7	%
		Mid Strip=	1.3	%
	Pos	Col Strip=	60.0	%
		Mid Strip=	40.0	%
	Int	Col Strip=	75.0	%
		Mid Strip=	25.0	%
Frame F	Ext	Col Strip=	98.6	%
		Mid Strip=	1.4	%
	Pos	Col Strip=	60.0	%
		Mid Strip=	40.0	%
	Int	Col Strip=	75.0	%
		Mid Strip=	25.0	%

Frame E:

	Col Strip: 8.4 ft	Mid Strip: 9.7 ft		Col Strip: 7.6 ft	Mid Strip: 10.5 ft	
M_{tot}=	-42.8	89.1	-119.9	-88.2	47.5	-88.2
M_{col}=	-42.2	53.4	-89.9	-66.2	28.5	-66.2
M_{mid}=	-0.6	35.6	-30.0	-22.1	19.0	-22.1

Frame F:

	Col Strip: 8.4 ft	Mid Strip: 8.5 ft		Col Strip: 7.6 ft	Mid Strip: 9.3 ft	
M_{tot}=	-39.9	82.9	-111.6	-82.1	44.2	-82.1
M_{col}=	-39.3	49.8	-83.7	-61.6	26.5	-61.6
M_{mid}=	-0.6	33.2	-27.9	-20.5	17.7	-20.5

Assume:

#	5	bars

Interpolate Machine:

	ρ =	R =
Low	0.003	175
High	0.0035	204
Result	0.00329	192

Design of Slab Reinforcement for Frame A

Column Strip

Description	Exterior Span			Interior Span	
	M^-_{ext}	M^+	M^-_{int}	M^-	M^+
Moment: $M_{u,col}$	-25.8	33.1	-55.7	-39.7	17.1
Col. Strip Width: b	50.5	50.5	50.5	50.5	50.5
Effective Depth: d	7.44	7.44	7.44	7.44	7.44
$M_u \times 12/b$	-6.1	7.9	-13.2	-9.4	4.1
$M_n = M_u/\phi$	-28.6	36.8	-61.9	-44.1	19.0
$R = M_n \times 12000/bd^2$	123.0	158.0	265.9	189.6	81.7
ρ = See Table A.5a	0	0	0.00463	0	0
ρ_{min} = See Table A.4	←		0.0033	→	
ρ_{max} = See Table A.4	←		0.0206	→	
Check ρ_{min}	N.G.	N.G.	OK	N.G.	N.G.
Check ρ_{max}	OK	OK	OK	OK	OK
Use ρ	0.0033	0.0033	0.00463	0.0033	0.0033
$A_s = \rho bd$	1.24	1.24	1.74	1.24	1.24
$A_{s,min}$ = .0018bt	0.77	0.77	0.77	0.77	0.77
Check $A_s > A_{s,min}$	OK	OK	OK	OK	OK
Use A_s	1.24	1.24	1.74	1.24	1.24
No. of Bars	5	5	6	5	5
Min No. of Bars	3	3	3	3	3
Use No. of Bars	5	5	6	5	5

Middle Strip

Description	Exterior Span			Interior Span	
	M^-_{ext}	M^+	M^-_{int}	M^-	M^+
Moment: $M_{u,col}$	-0.8	22.1	-18.6	-13.2	11.4
Col. Strip Width: b	50.5	50.5	50.5	50.5	50.5
Effective Depth: d	7.44	7.44	7.44	7.44	7.44
$M_u \times 12/b$	-0.2	5.2	-4.4	-3.1	2.7
$M_n = M_u/\phi$	-0.9	24.5	-20.6	-14.7	12.7
$R = M_n \times 12000/bd^2$	3.7	105.4	88.6	63.2	54.5
ρ = See Table A.5a	0	0	0	0	0
ρ_{min} = See Table A.4	←		0.0033	→	
ρ_{max} = See Table A.4	←		0.0206	→	
Check ρ_{min}	N.G.	N.G.	N.G.	N.G.	N.G.
Check ρ_{max}	OK	OK	OK	OK	OK
Use ρ	0.0033	0.0033	0.0033	0.0033	0.0033
$A_s = \rho bd$	1.24	1.24	1.24	1.24	1.24
$A_{s,min}$ = .0018bt	0.77	0.77	0.77	0.77	0.77
Check $A_s > A_{s,min}$	OK	OK	OK	OK	OK
Use A_s	1.24	1.24	1.24	1.24	1.24
No. of Bars	5	5	5	5	5
Min No. of Bars	3	3	3	3	3
Use No. of Bars	5	5	5	5	5

Design of Slab Reinforcement for Frame B

Column Strip

Description	Exterior Span			Interior Span	
	M^-_{ext}	M^+	M^-_{int}	M^-	M^+
Moment: $M_{u,col}$	-49.7	62.9	-105.9	-75.5	32.5
Col. Strip Width: b	96.0	96.0	96.0	96.0	96.0
Effective Depth: d	7.44	7.44	7.44	7.44	7.44
$M_u \times 12/b$	-6.2	7.9	-13.2	-9.4	4.1
$M_n = M_u/\phi$	-55.2	69.9	-117.7	-83.9	36.2
$R = M_n \times 12000/bd^2$	124.7	158.0	265.9	189.6	81.7
ρ = See Table A.5a	0	0	0.00463	0	0
ρ_{min} = See Table A.4	←		0.0033	→	
ρ_{max} = See Table A.4	←		0.0206	→	
Check ρ_{min}	N.G.	N.G.	OK	N.G.	N.G.
Check ρ_{max}	OK	OK	OK	OK	OK
Use ρ	0.0033	0.0033	0.00463	0.0033	0.0033
$A_s = \rho bd$	2.36	2.36	3.31	2.36	2.36
$A_{s,min}$ = .0018bt	1.47	1.47	1.47	1.47	1.47
Check $A_s > A_{s,min}$	OK	OK	OK	OK	OK
Use A_s	2.36	2.36	3.31	2.36	2.36
No. of Bars	8	8	11	8	8
Min No. of Bars	6	6	6	6	6
Use No. of Bars	8	8	11	8	8

Middle Strip

Description	Exterior Span			Interior Span	
	M^-_{ext}	M^+	M^-_{int}	M^-	M^+
Moment: $M_{u,col}$	-0.8	42.0	-35.3	-25.2	21.7
Col. Strip Width: b	96.0	96.0	96.0	96.0	96.0
Effective Depth: d	7.44	7.44	7.44	7.44	7.44
$M_u \times 12/b$	-0.1	5.2	-4.4	-3.1	2.7
$M_n = M_u/\phi$	-0.9	46.6	-39.2	-28.0	24.1
$R = M_n \times 12000/bd^2$	1.9	105.4	88.6	63.2	54.5
ρ = See Table A.5a	0	0	0	0	0
ρ_{min} = See Table A.4	←		0.0033	→	
ρ_{max} = See Table A.4	←		0.0206	→	
Check ρ_{min}	N.G.	N.G.	N.G.	N.G.	N.G.
Check ρ_{max}	OK	OK	OK	OK	OK
Use ρ	0.0033	0.0033	0.0033	0.0033	0.0033
$A_s = \rho bd$	2.36	2.36	2.36	2.36	2.36
$A_{s,min}$ = .0018bt	1.47	1.47	1.47	1.47	1.47
Check $A_s > A_{s,min}$	OK	OK	OK	OK	OK
Use A_s	2.36	2.36	2.36	2.36	2.36
No. of Bars	8	8	8	8	8
Min No. of Bars	6	6	6	6	6
Use No. of Bars	8	8	8	8	8

Design of Slab Reinforcement for Frame C

Column Strip

Description	Exterior Span			Interior Span	
	M^-_{ext}	M^+	M^-_{int}	M^-	M^+
Moment: $M_{u,col}$	-54.7	69.2	-116.4	-83.0	35.8
Col. Strip Width: b	105.5	105.5	105.5	105.5	105.5
Effective Depth: d	7.44	7.44	7.44	7.44	7.44
$M_u \times 12/b$	-6.2	7.9	-13.2	-9.4	4.1
$M_n = M_u/\phi$	-60.7	76.9	-129.3	-92.2	39.7
$R = M_n \times 12000/bd^2$	124.9	158.0	265.9	189.6	81.7
ρ = See Table A.5a	0	0	0.00463	0	0
ρ_{min} = See Table A.4	←		0.0033	→	
ρ_{max} = See Table A.4	←		0.0206	→	
Check ρ_{min}	N.G.	N.G.	OK	N.G.	N.G.
Check ρ_{max}	OK	OK	OK	OK	OK
Use ρ	0.0033	0.0033	0.00463	0.0033	0.0033
$A_s = \rho bd$	2.59	2.59	3.63	2.59	2.59
$A_{s,min}$ = .0018bt	1.61	1.61	1.61	1.61	1.61
Check $A_s > A_{s,min}$	OK	OK	OK	OK	OK
Use A_s	2.59	2.59	3.63	2.59	2.59
No. of Bars	9	9	12	9	9
Min No. of Bars	7	7	7	7	7
Use No. of Bars	9	9	12	9	9

Middle Strip

Description	Exterior Span			Interior Span	
	M^-_{ext}	M^+	M^-_{int}	M^-	M^+
Moment: $M_{u,col}$	-0.8	46.1	-38.8	-27.7	23.8
Col. Strip Width: b	105.5	105.5	105.5	105.5	105.5
Effective Depth: d	7.44	7.44	7.44	7.44	7.44
$M_u \times 12/b$	-0.1	5.2	-4.4	-3.1	2.7
$M_n = M_u/\phi$	-0.9	51.2	-43.1	-30.7	26.5
$R = M_n \times 12000/bd^2$	1.8	105.4	88.6	63.2	54.5
ρ = See Table A.5a	0	0	0	0	0
ρ_{min} = See Table A.4	←		0.0033	→	
ρ_{max} = See Table A.4	←		0.0206	→	
Check ρ_{min}	N.G.	N.G.	N.G.	N.G.	N.G.
Check ρ_{max}	OK	OK	OK	OK	OK
Use ρ	0.0033	0.0033	0.0033	0.0033	0.0033
$A_s = \rho bd$	2.59	2.59	2.59	2.59	2.59
$A_{s,min}$ = .0018bt	1.61	1.61	1.61	1.61	1.61
Check $A_s > A_{s,min}$	OK	OK	OK	OK	OK
Use A_s	2.59	2.59	2.59	2.59	2.59
No. of Bars	9	9	9	9	9
Min No. of Bars	7	7	7	7	7
Use No. of Bars	9	9	9	9	9

Design of Slab Reinforcement for Frame D

Column Strip

Description	Exterior Span			Interior Span	
	M^-_{ext}	M^+	M^-_{int}	M^-	M^+
Moment: $M_{u,col}$	-22.1	28.3	-47.7	-35.1	15.1
Col. Strip Width: b	50.5	50.5	50.5	45.5	45.5
Effective Depth: d	6.81	6.81	6.81	6.81	6.81
$M_u \times 12/b$	-5.3	6.7	-11.3	-9.2	4.0
$M_n = M_u/\phi$	-24.6	31.5	-53.0	-39.0	16.8
$R = M_n \times 12000/bd^2$	125.8	161.1	271.1	221.4	95.4
ρ = See Table A.5a	0	0	0.00472	0.00381	0
ρ_{min} = See Table A.4	←		0.0033	→	

Middle Strip

Description	Exterior Span			Interior Span	
	M^-_{ext}	M^+	M^-_{int}	M^-	M^+
Moment: $M_{u,col}$	-0.6	18.9	-15.9	-11.7	10.1
Col. Strip Width: b	64.5	64.5	64.5	69.5	69.5
Effective Depth: d	6.81	6.81	6.81	6.81	6.81
$M_u \times 12/b$	-0.1	3.5	-3.0	-2.0	1.7
$M_n = M_u/\phi$	-0.6	21.0	-17.7	-13.0	11.2
$R = M_n \times 12000/bd^2$	2.6	84.1	70.8	48.3	41.6
ρ = See Table A.5a	0	0	0	0	0
ρ_{min} = See Table A.4	←		0.0033	→	

ρ_{max} = See Table A.4	←		0.0206	→	
Check ρ_{min}	N.G.	N.G.	OK	OK	N.G.
Check ρ_{max}	OK	OK	OK	OK	OK
Use ρ	0.0033	0.0033	0.00472	0.00381	0.0033
$A_s = \rho bd$	1.14	1.14	1.62	1.18	1.02
$A_{s,min}$ = .0018bt	0.77	0.77	0.77	0.70	0.70
Check $A_s > A_{s,min}$	OK	OK	OK	OK	OK
Use A_s	1.14	1.14	1.62	1.18	1.02
No. of Bars	4	4	6	4	4
Min No. of Bars	3	3	3	3	3
Use No. of Bars	4	4	6	4	4

ρ_{max} = See Table A.4	←		0.0206	→	
Check ρ_{min}	N.G.	N.G.	N.G.	N.G.	N.G.
Check ρ_{max}	OK	OK	OK	OK	OK
Use ρ	0.0033	0.0033	0.0033	0.0033	0.0033
$A_s = \rho bd$	1.45	1.45	1.45	1.56	1.56
$A_{s,min}$ = .0018bt	0.99	0.99	0.99	1.06	1.06
Check $A_s > A_{s,min}$	OK	OK	OK	OK	OK
Use A_s	1.45	1.45	1.45	1.56	1.56
No. of Bars	5	5	5	6	6
Min No. of Bars	4	4	4	5	5
Use No. of Bars	5	5	5	6	6

Design of Slab Reinforcement for Frame E

Column Strip

Description	Exterior Span			Interior Span	
	M^-_{ext}	M^+	M^-_{int}	M^-	M^+
Moment: $M_{u,col}$	-42.2	53.4	-89.9	-66.2	28.5
Col. Strip Width: b	101.0	101.0	101.0	91.0	91.0
Effective Depth: d	6.81	6.81	6.81	6.81	6.81
M_u x 12/b	-5.0	6.4	-10.7	-8.7	3.8
$M_n = M_u/\phi$	-46.9	59.4	-99.9	-73.5	31.7
R = M_n x 12000/bd^2	120.2	152.0	255.8	208.9	90.0
ρ = See Table A.5a	0	0	0.00444	0.00359	0
ρ_{min} = See Table A.4	←		0.0033	→	
ρ_{max} = See Table A.4	←		0.0206	→	
Check ρ_{min}	N.G.	N.G.	OK	OK	N.G.
Check ρ_{max}	OK	OK	OK	OK	OK
Use ρ	0.0033	0.0033	0.00444	0.00359	0.0033
$A_s = \rho bd$	2.27	2.27	3.05	2.23	2.05
$A_{s,min}$ = .0018bt	1.55	1.55	1.55	1.39	1.39
Check $A_s > A_{s,min}$	OK	OK	OK	OK	OK
Use A_s	2.27	2.27	3.05	2.23	2.05
No. of Bars	8	8	10	8	7
Min No. of Bars	6	6	6	6	6
Use No. of Bars	8	8	10	8	7

Middle Strip

Description	Exterior Span			Interior Span	
	M^-_{ext}	M^+	M^-_{int}	M^-	M^+
Moment: $M_{u,col}$	-0.6	35.6	-30.0	-22.1	19.0
Col. Strip Width: b	116.0	116.0	116.0	126.0	126.0
Effective Depth: d	6.81	6.81	6.81	6.81	6.81
M_u x 12/b	-0.1	3.7	-3.1	-2.1	1.8
$M_n = M_u/\phi$	-0.6	39.6	-33.3	-24.5	21.1
R = M_n x 12000/bd^2	1.4	88.3	74.3	50.3	43.3
ρ = See Table A.5a	0	0	0	0	0
ρ_{min} = See Table A.4	←		0.0033	→	
ρ_{max} = See Table A.4	←		0.0206	→	
Check ρ_{min}	N.G.	N.G.	N.G.	N.G.	N.G.
Check ρ_{max}	OK	OK	OK	OK	OK
Use ρ	0.0033	0.0033	0.0033	0.0033	0.0033
$A_s = \rho bd$	2.61	2.61	2.61	2.83	2.83
$A_{s,min}$ = .0018bt	1.77	1.77	1.77	1.93	1.93
Check $A_s > A_{s,min}$	OK	OK	OK	OK	OK
Use A_s	2.61	2.61	2.61	2.83	2.83
No. of Bars	9	9	9	10	10
Min No. of Bars	7	7	7	8	8
Use No. of Bars	9	9	9	10	10

Design of Slab Reinforcement for Frame F

Column Strip

Description	Exterior Span			Interior Span	
	M^-_{ext}	M^+	M^-_{int}	M^-	M^+
Moment: $M_{u,col}$	-39.3	49.8	-83.7	-61.6	26.5
Col. Strip Width: b	100.5	100.5	100.5	91.0	91.0
Effective Depth: d	6.81	6.81	6.81	6.81	6.81
M_u x 12/b	-4.7	5.9	-10.0	-8.1	3.5
$M_n = M_u/\phi$	-43.7	55.3	-93.0	-68.4	29.5
R = M_n x 12000/bd^2	112.3	142.2	239.3	194.5	83.8
ρ = See Table A.5a	0	0	0.00414	0.00334	0
ρ_{min} = See Table A.4	←		0.0033	→	
ρ_{max} = See Table A.4	←		0.0206	→	
Check ρ_{min}	N.G.	N.G.	OK	OK	N.G.
Check ρ_{max}	OK	OK	OK	OK	OK
Use ρ	0.0033	0.0033	0.00414	0.00334	0.0033
$A_s = \rho bd$	2.26	2.26	2.83	2.07	2.05
$A_{s,min}$ = .0018bt	1.54	1.54	1.54	1.39	1.39
Check $A_s > A_{s,min}$	OK	OK	OK	OK	OK
Use A_s	2.26	2.26	2.83	2.07	2.05
No. of Bars	8	8	10	7	7
Min No. of Bars	6	6	6	6	6
Use No. of Bars	8	8	10	7	7

Middle Strip

Description	Exterior Span			Interior Span	
	M^-_{ext}	M^+	M^-_{int}	M^-	M^+
Moment: $M_{u,col}$	-0.6	33.2	-27.9	-20.5	17.7
Col. Strip Width: b	101.5	101.5	101.5	111.0	111.0
Effective Depth: d	6.81	6.81	6.81	6.81	6.81
M_u x 12/b	-0.1	3.9	-3.3	-2.2	1.9
$M_n = M_u/\phi$	-0.6	36.9	-31.0	-22.8	19.7
R = M_n x 12000/bd^2	1.6	93.9	79.0	53.1	45.8
ρ = See Table A.5a	0	0	0	0	0
ρ_{min} = See Table A.4	←		0.0033	→	
ρ_{max} = See Table A.4	←		0.0206	→	
Check ρ_{min}	N.G.	N.G.	N.G.	N.G.	N.G.
Check ρ_{max}	OK	OK	OK	OK	OK
Use ρ	0.0033	0.0033	0.0033	0.0033	0.0033
$A_s = \rho bd$	2.28	2.28	2.28	2.50	2.50
$A_{s,min}$ = .0018bt	1.55	1.55	1.55	1.70	1.70
Check $A_s > A_{s,min}$	OK	OK	OK	OK	OK
Use A_s	2.28	2.28	2.28	2.50	2.50
No. of Bars	8	8	8	9	9
Min No. of Bars	6	6	6	7	7
Use No. of Bars	8	8	8	9	9

Slab Reinforcement for Middle Strip 1

Description	Exterior Span			Interior Span	
	M^-_{ext}	M^+	M^-_{int}	M^-	M^+
Frame A Width (ft)	4.2	4.2	4.2	4.2	4.2
Frame B Width (ft)	4.2	4.2	4.2	4.2	4.2
No. of Bars from Frame A	5	5	5	5	5
No. of Bars from Frame B	4	4	4	4	4
Use No. of Bars	10	10	10	10	10

Slab Reinforcement for Middle Strip 2

Description	Exterior Span			Interior Span	
	M^-_{ext}	M^+	M^-_{int}	M^-	M^+
Frame B Width (ft)	3.8	3.8	3.8	3.8	3.8
Frame C Width (ft)	3.8	3.8	3.8	3.8	3.8
No. of Bars from Frame B	4	4	4	4	4
No. of Bars from Frame C	4	4	4	4	4
Use No. of Bars	8	8	8	8	8

Slab Reinforcement for Middle Strip 3

Description	Exterior Span			Interior Span	
	M^-_{ext}	M^+	M^-_{int}	M^-	M^+
Frame D Width (ft)	5.4	5.4	5.4	5.8	5.8
Frame E Width (ft)	5.4	5.4	5.4	5.8	5.8
No. of Bars from Frame D	5	5	5	6	6
No. of Bars from Frame E	5	5	5	6	6
Use No. of Bars	11	11	11	12	12

Slab Reinforcement for Middle Strip 4

Description	Exterior Span			Interior Span	
	M^-_{ext}	M^+	M^-_{int}	M^-	M^+
Frame E Width (ft)	4.3	4.3	4.3	4.7	4.7
Frame F Width (ft)	4.3	4.3	4.3	4.7	4.7
No. of Bars from Frame E	4	4	4	4	4
No. of Bars from Frame F	4	4	4	5	5
Use No. of Bars	9	9	9	10	10

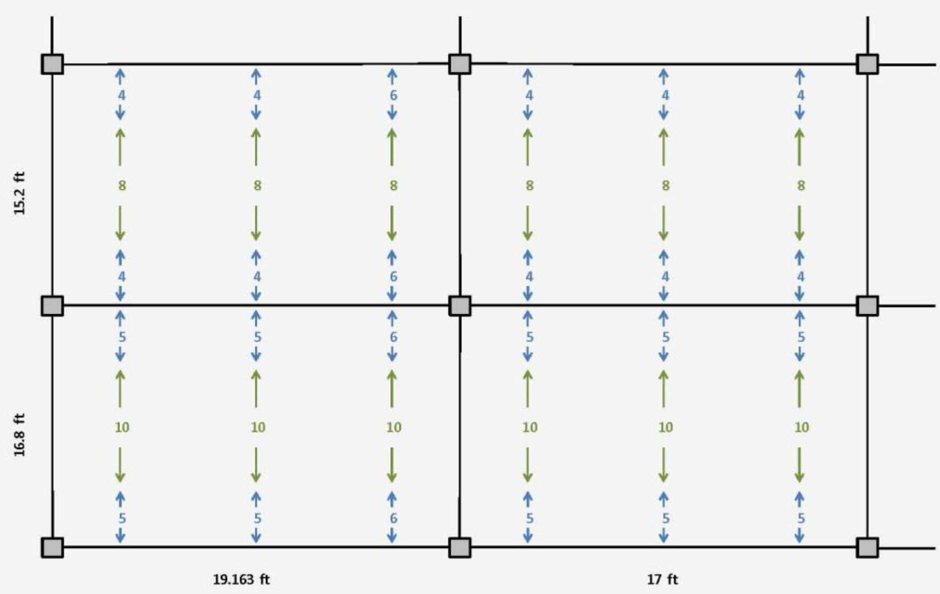
Summary of Required Reinforcement

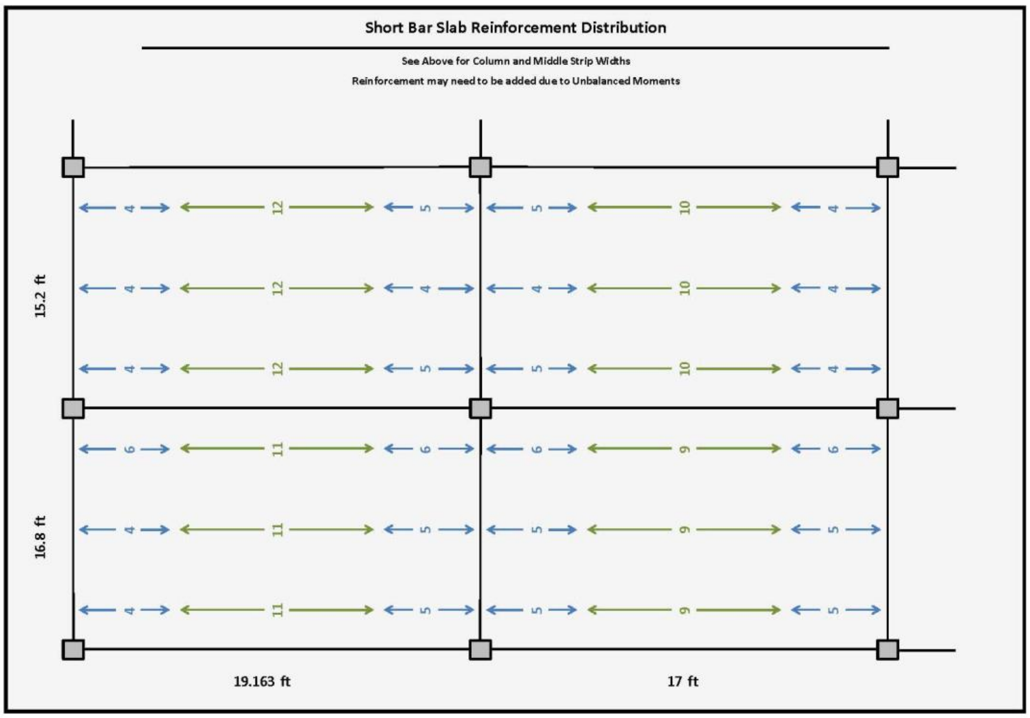
l_1:	16.833	ft	t_{slab}:	8.50	in
l_2:	19.163	ft	f'_c:	4000	psi
l_3:	15.166	ft	$t_{col,1\,dir}$:	20	in
l_4:	17	ft	$t_{col,2\,dir}$:	20	in

Long Bar Slab Reinforcement Distribution

See Above for Column and Middle Strip Widths

Note: Reinforcement may need to be added due to Unbalanced Moments

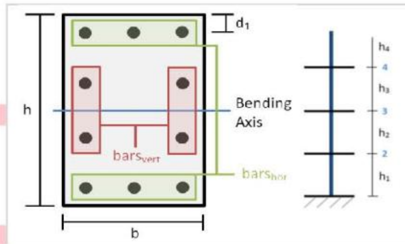



Short Bar Slab Reinforcement Distribution
See Above for Column and Middle Strip Widths
Reinforcement may need to be added due to Unbalanced Moments
15.2 ft
16.8 ft
19.163 ft
17 ft

Column Design of Ground Floor Columns

Trial Column			
b=		20	in
h=		20	in
Use	#	9	bars
d_1=		2.5	in
$bars_{vert}$=		4	
$bars_{hor}$=		8	
Floors=		5	
Note: Includes roof but not ground			
h_5=		10	ft
h_4=		10.67	ft
h_3=		10.67	ft
h_2=		12.66	ft
h_1=		14	ft

Roof Slope=	2	/ 12	
$w_{SD,roof}$=	23	psf	
$w_{L,roof}$=	20	psf	
w_{snow}=	30.8	psf	
$w_{SD,5}$=	35	psf	
$w_{L,5}$=	150	psf	Non-Reducible
$w_{SD,4}$=	30	psf	
$w_{L,4}$=	40	psf	
$w_{SD,3}$=	35	psf	
$w_{L,3}$=	40	psf	
$w_{SD,2}$=	35	psf	
$w_{L,2}$=	100	psf	Non-Reducible
$w_{SD,ground}$=	N/A	psf	
$w_{L,ground}$=	N/A	psf	



Column Strength / Strength Interaction Curve

Pure Compression		
P_o=	2035.1	kips

Pure Tension		
T_o=	-715.7	kips

Pure Bending (Solve by Hand)

Balanced-Strain Strength								
ε_y=	0.00207				β_1=	0.85	
c=	10.36	in	< h	OK	A_s=	0.994	in^2
d_1=	2.50	in			f_{s1}=	60.00	ksi
d_2=	7.50	in			f_{s2}=	24.00	ksi
d_3=	12.50	in			f_{s3}=	-18.00	ksi
d_4=	17.50	in			f_{s4}=	-60.00	ksi
d_5=		in			f_{s5}=		ksi
d_6=		in			f_{s6}=		ksi
d_7=		in			f_{s7}=		ksi
d_8=		in			f_{s8}=		ksi
P_b=	610.6	kips			M_b=	594.9	ft-k

Live Load Reduction ($L = L_o \times \alpha$)

Column CD			
$t_{col,1dir}$=	20	in	
$t_{col,2dir}$=	20	in	
A_T=	673.886058	ft^2	
$A_{T,roof}$=	168.471515	ft^2	
$K_{LL}A_T$=	2695.54423	ft^2	
$K_{LL}A_T$ >	400ft^2		OK
α=	0.54		
α_{roof}=	1.00		

Column CE			
$t_{col,1dir}$=	20	in	
$t_{col,2dir}$=	20	in	
A_T=	1271.70806	ft^2	
$A_{T,roof}$=	317.927015	ft^2	
$K_{LL}A_T$=	5086.83223	ft^2	
$K_{LL}A_T$ >	400ft^2		OK
α=	0.46		
α_{roof}=	0.88		

Column CF			
$t_{col,1dir}$=	20	in	
$t_{col,2dir}$=	20	in	
A_T=	1184.03922	ft^2	
$A_{T,roof}$=	296.009805	ft^2	
$K_{LL}A_T$=	4736.15688	ft^2	
$K_{LL}A_T$ >	400ft^2		OK
α=	0.47		
α_{roof}=	0.90		

Column BD			
$t_{col,1dir}$=	20	in	
$t_{col,2dir}$=	20	in	
A_T=	613.196837	ft^2	
$A_{T,roof}$=	153.299209	ft^2	
$K_{LL}A_T$=	2452.78735	ft^2	
$K_{LL}A_T$ >	400ft^2		OK
α=	0.55		
α_{roof}=	1.00		

Column BE			
$t_{col,1dir}$=	20	in	
$t_{col,2dir}$=	20	in	
A_T=	1157.17984	ft^2	
$A_{T,roof}$=	289.294959	ft^2	
$K_{LL}A_T$=	4628.71935	ft^2	
$K_{LL}A_T$ >	400ft^2		OK
α=	0.47		
α_{roof}=	0.91		

Column BF			
$t_{col,1dir}$=	20	in	
$t_{col,2dir}$=	20	in	
A_T=	1077.40633	ft^2	
$A_{T,roof}$=	269.351583	ft^2	
$K_{LL}A_T$=	4309.62532	ft^2	
$K_{LL}A_T$ >	400ft^2		OK
α=	0.48		
α_{roof}=	0.93		

Column AD			
$t_{col,1dir}$=	20	in	
$t_{col,2dir}$=	20	in	
A_T=	322.570779	ft^2	
$A_{T,roof}$=	80.6426948	ft^2	
$K_{LL}A_T$=	1290.28312	ft^2	
$K_{LL}A_T$ >	400ft^2		OK
α=	0.67		
α_{roof}=	1.00		

Column AE			
$t_{col,1dir}$=	20	in	
$t_{col,2dir}$=	20	in	
A_T=	608.731779	ft^2	
$A_{T,roof}$=	152.182945	ft^2	
$K_{LL}A_T$=	2434.92712	ft^2	
$K_{LL}A_T$ >	400ft^2		OK
α=	0.55		
α_{roof}=	1.00		

Column AF			
$t_{col,1dir}$=	20	in	
$t_{col,2dir}$=	20	in	
A_T=	566.76711	ft^2	
$A_{T,roof}$=	141.691778	ft^2	
$K_{LL}A_T$=	2267.06844	ft^2	
$K_{LL}A_T$ >	400ft^2		OK
α=	0.57		
α_{roof}=	1.00		

Total Loads

Column CD		
$M_{ETABS,long}$=	68	ft-k
$M_{ETABS,short}$=	96	ft-k
$M_{unb,long}$=	41.0	ft-k
$M_{unb,short}$=	21.9	ft-k
P_L=	49.4	kips
P_D=	115.4	kips
$P_{S,Lr}$=	8.6	kips
$M_{u,long}$=	109.0	ft-k
$M_{u,short}$=	117.9	ft-k
P_T=	221.7	kips

Column CE		
$M_{ETABS,long}$=	68	ft-k
$M_{ETABS,short}$=	96	ft-k
$M_{unb,long}$=	27.0	ft-k
$M_{unb,short}$=	40.2	ft-k
P_L=	91.2	kips
P_D=	202.5	kips
$P_{S,Lr}$=	15.4	kips
$M_{u,long}$=	95.0	ft-k
$M_{u,short}$=	136.2	ft-k
P_T=	396.6	kips

Column CF		
$M_{ETABS,long}$=	68	ft-k
$M_{ETABS,short}$=	96	ft-k
$M_{unb,long}$=	15.1	ft-k
$M_{unb,short}$=	37.5	ft-k
P_L=	85.1	kips
P_D=	189.7	kips
$P_{S,Lr}$=	14.5	kips
$M_{u,long}$=	83.1	ft-k
$M_{u,short}$=	133.5	ft-k
P_T=	371.0	kips

Column BD		
$M_{ETABS,long}$=	68	ft-k
$M_{ETABS,short}$=	96	ft-k
$M_{unb,long}$=	37.3	ft-k
$M_{unb,short}$=	11.0	ft-k
P_L=	45.1	kips
P_D=	106.5	kips
$P_{S,Lr}$=	7.8	kips
$M_{u,long}$=	105.3	ft-k
$M_{u,short}$=	107.0	ft-k
P_T=	203.9	kips

Column BE		
$M_{ETABS,long}$=	68	ft-k
$M_{ETABS,short}$=	96	ft-k
$M_{unb,long}$=	24.5	ft-k
$M_{unb,short}$=	20.1	ft-k
P_L=	83.2	kips
P_D=	185.8	kips
$P_{S,Lr}$=	14.2	kips
$M_{u,long}$=	92.5	ft-k
$M_{u,short}$=	116.1	ft-k
P_T=	363.2	kips

Column BF		
$M_{ETABS,long}$=	68	ft-k
$M_{ETABS,short}$=	96	ft-k
$M_{unb,long}$=	13.8	ft-k
$M_{unb,short}$=	18.7	ft-k
P_L=	77.6	kips
P_D=	174.2	kips
$P_{S,Lr}$=	13.3	kips
$M_{u,long}$=	81.8	ft-k
$M_{u,short}$=	114.7	ft-k
P_T=	339.9	kips

Column AD		
$M_{ETABS,long}$=	68	ft-k
$M_{ETABS,short}$=	96	ft-k
$M_{unb,long}$=	19.1	ft-k
$M_{unb,short}$=	16.3	ft-k
P_L=	24.5	kips
P_D=	64.2	kips
$P_{S,Lr}$=	4.1	kips
$M_{u,long}$=	87.1	ft-k
$M_{u,short}$=	112.3	ft-k
P_T=	118.2	kips

Column AE		
$M_{ETABS,long}$=	68	ft-k
$M_{ETABS,short}$=	96	ft-k
$M_{unb,long}$=	12.5	ft-k
$M_{unb,short}$=	30.0	ft-k
P_L=	44.8	kips
P_D=	105.9	kips
$P_{S,Lr}$=	7.7	kips
$M_{u,long}$=	80.5	ft-k
$M_{u,short}$=	126.0	ft-k
P_T=	202.6	kips

Column AF		
$M_{ETABS,long}$=	68	ft-k
$M_{ETABS,short}$=	96	ft-k
$M_{unb,long}$=	7.0	ft-k
$M_{unb,short}$=	27.9	ft-k
P_L=	41.8	kips
P_D=	99.8	kips
$P_{S,Lr}$=	7.2	kips
$M_{u,long}$=	75.0	ft-k
$M_{u,short}$=	123.9	ft-k
P_T=	190.2	kips

Unbalanced Moments in Columns

			Check if: In accordance to ACI 13.5.3.3
	#	5	bars
$d=$		7.13	in
$w_u=$		329.5	psf
$w_D=$		141.3	psf

Exterior Column CD (Reinforcement Needed)

$t_{col,1dir}=$	20	in	$b_o=$	74.25	in
$t_{col,2dir}=$	20	in	$b_1=$	23.56	in
$M_{o,long}=$	221.7	ft-k	$b_2=$	27.13	in
$M_{u,long}=$	66.5	ft-k	$V_{c1}=$	133.8	kips
$M_{u,short}=$	35.5	ft-k	$V_{c2}=$	200.8	kips
			$V_{c3}=$	163.2	kips
$V_u=$	55.5	kips	$\phi V_c=$	100.4	kips

Transferred by Flexure

$\gamma_f=$	0.617				
$M_{ub,long}=$	41.0	ft-k	$M_{ub,short}=$	21.9	ft-k
$M_{ub,totlong}=$	109.0	ft-k	$M_{ub,totshort}=$	117.9	ft-k
$M_{col,lright}=$	23.6	ft-k	$M_{col,sleft}=$	25.2	ft-k
$M_{ub} < M_{col}$	Need Reinforcement		$M_{ub} < M_{col}$	Need Reinforcement	

Description	Value	Description	Value
Moment: M_u	85.4	Moment: M_u	92.7
Strip Width: b	45.5	Strip Width: b	32.75
Effective Depth: d	7.13	Effective Depth: d	7.13
$M_u \times 12/b$	22.5	$M_u \times 12/b$	34.0
$M_n = M_u/\phi$	94.9	$M_n = M_u/\phi$	103.0
$R = M_n \times 12000/bd^2$	493.2	$R = M_n \times 12000/bd^2$	743.2
ρ = See Table A.5a	0.00892	ρ = See Table A.5a	0.01416
ρ_{min} = See Table A.4	0.0033	ρ_{min} = See Table A.4	0.0033
ρ_{max} = See Table A.4	0.0206	ρ_{max} = See Table A.4	0.0206
Check ρ_{min}	OK	Check ρ_{min}	OK
Check ρ_{max}	OK	Check ρ_{max}	OK
Use ρ	0.00892	Use ρ	0.01416
$A_s = \rho bd$	2.89	$A_s = \rho bd$	3.30
$A_{s,min}$ = .0018bt	0.70	$A_{s,min}$ = .0018bt	0.50
Check $A_s > A_{s,min}$	OK	Check $A_s > A_{s,min}$	OK
Use A_s	2.89	Use A_s	3.30
No. of Bars	10	No. of Bars	11
Min No. of Bars	3	Min No. of Bars	2
Use No. of Bars	10	Use No. of Bars	11

Transferred by Eccentricity of Shear

$V_u=$	55.5	kips	$V_u=$	55.5	kips
$M_{uv,long}=$	25.5	ft-k	$M_{uv,short}=$	13.6	ft-k
Centroid=	7.48	in	Centroid=	13.56	in
$J_c=$	33980	in^4	$J_c=$	87096	in^4
$A_c=$	529	in^2	$A_c=$	529	in^2
$v_l=$	-40	psi	$v_l=$	79	psi
$v_r=$	172	psi	$v_r=$	130	psi
$v_u=$	172	psi	$v_u=$	130	psi
$\phi v_n=$	190	psi	$\phi v_n=$	190	psi
$\phi v_n > v_u$	OK		$\phi v_n > v_u$	OK	

Exterior Column BD (Reinforcement Needed)

$t_{col,1dir}$=	20	in	b_o=	74.25	in
$t_{col,2dir}$=	20	in	b_1=	23.56	in
$M_{o,long}$=	201.7	ft-k	b_2=	27.13	in
$M_{u,long}$=	60.5	ft-k	V_{c1}=	133.8	kips
$M_{u,short}$=	17.8	ft-k	V_{c2}=	200.8	kips
			V_{c3}=	163.2	kips
V_u=	50.5	kips	ϕV_c=	100.4	kips

Transferred by Flexure

γ_f=	0.617				
$M_{ub,long}$=	37.3	ft-k	$M_{ub,short}$=	11.0	ft-k
$M_{ub,totlong}$=	105.3	ft-k	$M_{ub,totshort}$=	107.0	ft-k
$M_{col,lright}$=	23.5	ft-k	$M_{col,sleft}$=	34.3	ft-k
$M_{ub} < M_{col}$	Need Reinforcement		$M_{col,sright}$=	25.2	ft-k
			$M_{ub} < M_{col}$	Need Reinforcement	

Description	Value
Moment: M_u	81.8
Strip Width: b	45.5
Effective Depth: d	7.13
$M_u \times 12/b$	21.6
$M_n = M_u/\phi$	90.9
$R = M_n \times 12000/bd^2$	472.1
ρ = See Table A.5a	0.0085
ρ_{min} = See Table A.4	0.0033
ρ_{max} = See Table A.4	0.0206
Check ρ_{min}	OK
Check ρ_{max}	OK
Use ρ	0.0085
$A_s = \rho bd$	2.76
$A_{s,min}$ = .0018bt	0.70
Check $A_s > A_{s,min}$	OK
Use A_s	2.76
No. of Bars	9
Min No. of Bars	3
Use No. of Bars	9

Description	Left Side	Right Side
Moment: M_u	72.7	81.7
Strip Width: b	32.75	32.75
Effective Depth: d	7.13	7.13
$M_u \times 12/b$	26.6	29.9
$M_n = M_u/\phi$	80.7	90.8
$R = M_n \times 12000/bd^2$	582.7	655.4
ρ = See Table A.5a	0.01072	0.01225
ρ_{min} = See Table A.4	0.0033	0.0033
ρ_{max} = See Table A.4	0.0206	0.0206
Check ρ_{min}	OK	OK
Check ρ_{max}	OK	OK
Use ρ	0.01072	0.01225
$A_s = \rho bd$	2.50	2.86
$A_{s,min}$ = .0018bt	0.50	0.50
Check $A_s > A_{s,min}$	OK	OK
Use A_s	2.50	2.86
No. of Bars	9	10
Min No. of Bars	2	2
Use No. of Bars	9	10

Transferred by Eccentricity of Shear

V_u=	50.5	kips			V_u=	50.5	kips			
$M_{uv,long}$=	23.2	ft-k			$M_{uv,short}$=	6.8	ft-k			
Centroid=	7.48	in			Centroid=	13.56	in			
J_c=	33980	in^4			J_c=	87096	in^4			
A_c=	529	in^2			A_c=	529	in^2			
ν_l=	-36	psi			ν_l=	83	psi			
ν_r=	157	psi			ν_r=	108	psi			
ν_u=	157	psi			ν_u=	108	psi			
$\phi \nu_n$=	190	psi	>	ν_u OK	$\phi \nu_n$=	190	psi	>	ν_u	OK

Corner Column AD (Reinforcement Needed)

$t_{col,1dir}$=	20	in	b_o=	47.13	in
$t_{col,2dir}$=	20	in	b_1=	23.56	in
$M_{o,long}$=	106.1	ft-k	b_2=	23.56	in
$M_{u,long}$=	31.8	ft-k	V_{c1}=	84.9	kips
$M_{o,short}$=	90.8	ft-k	V_{c2}=	127.4	kips
$M_{u,short}$=	27.2	ft-k	V_{c3}=	106.7	kips
V_u=	26.6	kips	ϕV_c=	63.7	kips

Transferred by Flexure

γ_f=	0.600				
$M_{ub,long}$=	19.1	ft-k	$M_{ub,short}$=	16.3	ft-k
$M_{ub,totlong}$=	87.1	ft-k	$M_{ub,totshort}$=	112.3	ft-k
$M_{col,lright}$=	16.7	ft-k	$M_{col,sright}$=	14.3	ft-k
M_{ub} < M_{col}	Need Reinforcement		M_{ub} < M_{col}	Need Reinforcement	

Description	Value	Description	Value
Moment: M_u	70.4	Moment: M_u	98.0
Strip Width: b	32.75	Strip Width: b	32.75
Effective Depth: d	7.13	Effective Depth: d	7.13
M_u x 12/b	25.8	M_u x 12/b	35.9
$M_n = M_u/\phi$	78.2	$M_n = M_u/\phi$	108.9
$R = M_n \times 12000/bd^2$	564.5	$R = M_n \times 12000/bd^2$	785.9
ρ = See Table A.5a	0.01035	ρ = See Table A.5a	0.01511
ρ_{min} = See Table A.4	0.0033	ρ_{min} = See Table A.4	0.0033
ρ_{max} = See Table A.4	0.0206	ρ_{max} = See Table A.4	0.0206
Check ρ_{min}	OK	Check ρ_{min}	OK
Check ρ_{max}	OK	Check ρ_{max}	OK
Use ρ	0.01035	Use ρ	0.01511
$A_s = \rho bd$	2.42	$A_s = \rho bd$	3.53
$A_{s,min}$ = .0018bt	0.50	$A_{s,min}$ = .0018bt	0.50
Check $A_s > A_{s,min}$	OK	Check $A_s > A_{s,min}$	OK
Use A_s	2.42	Use A_s	3.53
No. of Bars	8	No. of Bars	12
Min No. of Bars	2	Min No. of Bars	2
Use No. of Bars	8	Use No. of Bars	12

Transferred by Eccentricity of Shear

V_u=	26.6	kips	V_u=	26.6	kips
$M_{uv,long}$=	12.7	ft-k	$M_{uv,short}$=	10.9	ft-k
Centroid=	7.85	in	Centroid=	7.85	in
J_c=	32489	in^4	J_c=	32489	in^4
A_c=	336	in^2	A_c=	336	in^2
v_l=	5	psi	v_l=	16	psi
v_r=	116	psi	v_r=	111	psi
v_u=	116	psi	v_u=	111	psi
ϕv_n=	190	psi	ϕv_n=	190	psi
ϕv_n > v_u	OK		ϕv_n > v_u	OK	

Interior Column CE (Reinforcement Needed)

$t_{col,1dir}$=	20	in	b_0=	108.50	in
$t_{col,2dir}$=	20	in	b_1=	27.13	in
$M_{u,long}$=	45.0	ft-k	b_2=	27.13	in
$M_{u,short}$=	67.0	ft-k	V_{c1}=	195.6	kips
			V_{c2}=	293.4	kips
			V_{c3}=	226.2	kips
V_u=	104.8	kips	ϕV_c=	146.7	kips

Transferred by Flexure

γ_f=	0.600				
$M_{ub,long}$=	27.0	ft-k	$M_{ub,short}$=	40.2	ft-k
$M_{ub,totlong}$=	95.0	ft-k	$M_{ub,totshort}$=	136.2	ft-k
$M_{col,lleft}$=	50.2	ft-k	$M_{col,sleft}$=	33.1	ft-k
$M_{col,lright}$=	35.8	ft-k	$M_{ub} < M_{col}$	Need Reinforcement	
$M_{ub} < M_{col}$	Need Reinforcement				

Description	Left Side	Right Side
Moment: M_u	44.8	59.2
Strip Width: b	45.5	45.5
Effective Depth: d	7.13	7.13
$M_u \times 12/b$	11.8	15.6
$M_n = M_u/\phi$	49.8	65.7
$R = M_n \times 12000/bd^2$	258.5	341.6
ρ = See Table A.5a	0.0045	0.006
ρ_{min} = See Table A.4	0.0033	0.0033
ρ_{max} = See Table A.4	0.0206	0.0206
Check ρ_{min}	OK	OK
Check ρ_{max}	OK	OK
Use ρ	0.0045	0.006
$A_s = \rho bd$	1.46	1.95
$A_{s,min}$ = .0018bt	0.70	0.70
Check $A_s > A_{s,min}$	OK	OK
Use A_s	1.46	1.95
No. of Bars	5	7
Min No. of Bars	3	3
Use No. of Bars	5	7

Description	Left Side
Moment: M_u	103.1
Strip Width: b	45.5
Effective Depth: d	7.13
$M_u \times 12/b$	27.2
$M_n = M_u/\phi$	114.6
$R = M_n \times 12000/bd^2$	595.4
ρ = See Table A.5a	0.011
ρ_{min} = See Table A.4	0.0033
ρ_{max} = See Table A.4	0.0206
Check ρ_{min}	OK
Check ρ_{max}	OK
Use ρ	0.011
$A_s = \rho bd$	3.57
$A_{s,min}$ = .0018bt	0.70
Check $A_s > A_{s,min}$	OK
Use A_s	3.57
No. of Bars	12
Min No. of Bars	3
Use No. of Bars	12

Transferred by Eccentricity of Shear

V_u=	104.8	kips			V_u=	104.8	kips		
$M_{uv,long}$=	18.0	ft-k			$M_{uv,short}$=	26.8	ft-k		
Centroid=	13.56	in			Centroid=	13.56	in		
J_c=	96434	in^4			J_c=	96434	in^4		
A_c=	773	in^2			A_c=	773	in^2		
v_l=	105	psi			v_l=	90	psi		
v_r=	166	psi			v_r=	181	psi		
v_u=	166	psi			v_u=	181	psi		
ϕv_n=	190	psi	> v_u	OK	ϕv_n=	190	psi	> v_u	OK

Interior Column BE (Reinforcement Needed)

$t_{col,1dir}$=	20	in	b_o=	108.50	in
$t_{col,2dir}$=	20	in	b_1=	27.13	in
$M_{u,long}$=	40.9	ft-k	b_2=	27.13	in
$M_{u,short}$=	33.5	ft-k	V_{c1}=	195.6	kips
			V_{c2}=	293.4	kips
			V_{c3}=	226.2	kips
V_u=	95.3	kips	ϕV_c=	146.7	kips

Transferred by Flexure

γ_f=	0.600				
$M_{ub,long}$=	24.5	ft-k	$M_{ub,short}$=	20.1	ft-k
$M_{ub,totlong}$=	92.5	ft-k	$M_{ub,totshort}$=	116.1	ft-k
$M_{col,lleft}$=	50.2	ft-k	$M_{col,sleft}$=	40.5	ft-k
$M_{col,lright}$=	35.8	ft-k	$M_{col,sright}$=	33.1	ft-k
M_{ub} < M_{col}	Need Reinforcement		M_{ub} < M_{col}	Need Reinforcement	

Description	Left Side	Right Side
Moment: M_u	42.3	56.7
Strip Width: b	45.5	45.5
Effective Depth: d	7.13	7.13
M_u x 12/b	11.2	15.0
$M_n = M_u/\phi$	47.1	63.1
$R = M_n$ x 12000/bd^2	244.4	327.6
ρ = See Table A.5a	0.00423	0.00575
ρ_{min} = See Table A.4	0.0033	0.0033
ρ_{max} = See Table A.4	0.0206	0.0206
Check ρ_{min}	OK	OK
Check ρ_{max}	OK	OK
Use ρ	0.00423	0.00575
A_s = ρbd	1.37	1.86
$A_{s,min}$ = .0018bt	0.70	0.70
Check $A_s > A_{s,min}$	OK	OK
Use A_s	1.37	1.86
No. of Bars	5	7
Min No. of Bars	3	3
Use No. of Bars	5	7

Description	Left Side	Right Side
Moment: M_u	75.6	83.0
Strip Width: b	45.5	45.5
Effective Depth: d	7.13	7.13
M_u x 12/b	19.9	21.9
$M_n = M_u/\phi$	84.0	92.3
$R = M_n$ x 12000/bd^2	436.5	479.4
ρ = See Table A.5a	0.00782	0.00865
ρ_{min} = See Table A.4	0.0033	0.0033
ρ_{max} = See Table A.4	0.0206	0.0206
Check ρ_{min}	OK	OK
Check ρ_{max}	OK	OK
Use ρ	0.00782	0.00865
A_s = ρbd	2.54	2.80
$A_{s,min}$ = .0018bt	0.70	0.70
Check $A_s > A_{s,min}$	OK	OK
Use A_s	2.54	2.80
No. of Bars	9	10
Min No. of Bars	3	3
Use No. of Bars	9	10

Transferred by Eccentricity of Shear

V_u=	95.3	kips				V_u=	95.3	kips			
$M_{uv,long}$=	16.4	ft-k				$M_{uv,short}$=	13.4	ft-k			
Centroid=	13.56	in				Centroid=	13.56	in			
J_c=	96434	in^4				J_c=	96434	in^4			
A_c=	773	in^2				A_c=	773	in^2			
v_l=	96	psi				v_l=	101	psi			
v_r=	151	psi				v_r=	146	psi			
v_u=	151	psi				v_u=	146	psi			
ϕv_n=	190	psi	>	v_u	OK	ϕv_n=	190	psi	>	v_u	OK

Exterior Column AE (Reinforcement Needed)

$t_{col,1dir}$=	20	in	b_o=	74.25	in
$t_{col,2dir}$=	20	in	b_1=	27.13	in
$M_{u,long}$=	21.5	ft-k	b_2=	23.56	in
$M_{o,short}$=	171.3	ft-k	V_{c1}=	133.8	kips
$M_{u,short}$=	51.4	ft-k	V_{c2}=	200.8	kips
			V_{c3}=	163.2	kips
V_u=	50.1	kips	ϕV_c=	100.4	kips

Transferred by Flexure

γ_f=	0.583				
$M_{ub,long}$=	12.5	ft-k	$M_{ub,short}$=	30.0	ft-k
$M_{ub,totlong}$=	80.5	ft-k	$M_{ub,totshort}$=	126.0	ft-k
$M_{col,lleft}$=	36.1	ft-k	$M_{col,sright}$=	19.0	ft-k
$M_{col,lright}$=	25.8	ft-k	M_{ub} < M_{col}	Need Reinforcement	
M_{ub} < M_{col}	Need Reinforcement				

Description	Left Side	Right Side
Moment: M_u	44.4	54.8
Strip Width: b	32.75	32.75
Effective Depth: d	7.13	7.13
M_u x 12/b	16.3	20.1
$M_n = M_u/\phi$	49.3	60.9
R = M_n x 12000/bd^2	356.2	439.3
ρ = See Table A.5a	0.00628	0.00787
ρ_{min} = See Table A.4	0.0033	0.0033
ρ_{max} = See Table A.4	0.0206	0.0206
Check ρ_{min}	OK	OK
Check ρ_{max}	OK	OK
Use ρ	0.00628	0.00787
A_s = ρbd	1.47	1.84
$A_{s,min}$ = .0018bt	0.50	0.50
Check $A_s > A_{s,min}$	OK	OK
Use A_s	1.47	1.84
No. of Bars	5	6
Min No. of Bars	2	2
Use No. of Bars	5	6

Description	Value
Moment: M_u	106.9
Strip Width: b	45.5
Effective Depth: d	7.13
M_u x 12/b	28.2
$M_n = M_u/\phi$	118.8
R = M_n x 12000/bd^2	617.2
ρ = See Table A.5a	0.01144
ρ_{min} = See Table A.4	0.0033
ρ_{max} = See Table A.4	0.0206
Check ρ_{min}	OK
Check ρ_{max}	OK
Use ρ	0.01144
A_s = ρbd	3.71
$A_{s,min}$ = .0018bt	0.70
Check $A_s > A_{s,min}$	OK
Use A_s	3.71
No. of Bars	13
Min No. of Bars	3
Use No. of Bars	13

Transferred by Eccentricity of Shear

V_u=	50.1	kips				V_u=	50.1	kips			
$M_{uv,long}$=	9.0	ft-k				$M_{uv,short}$=	21.4	ft-k			
Centroid=	13.56	in				Centroid=	7.48	in			
J_c=	87096	in^4				J_c=	33980	in^4			
A_c=	529	in^2				A_c=	529	in^2			
v_l=	78	psi				v_l=	-27	psi			
v_r=	112	psi				v_r=	151	psi			
v_u=	112	psi				v_u=	151	psi			
ϕv_n=	190	psi	>	v_u	OK	ϕv_n=	190	psi	>	v_u	OK

Interior Column CF (Reinforcement Needed)

$t_{col,1dir}$=	20	in	b_o=	108.50	in
$t_{col,2dir}$=	20	in	b_1=	27.13	in
$M_{u,long}$=	25.2	ft-k	b_2=	27.13	in
$M_{u,short}$=	62.4	ft-k	$V_{c,1}$=	195.6	kips
			$V_{c,2}$=	293.4	kips
			$V_{c,3}$=	226.2	kips
V_u=	97.5	kips	ϕV_c=	146.7	kips

Transferred by Flexure

γ_f=	0.600				
$M_{ub,long}$=	15.1	ft-k	$M_{ub,short}$=	37.5	ft-k
$M_{ub,totlong}$=	83.1	ft-k	$M_{ub,totshort}$=	133.5	ft-k
$M_{col,lleft}$=	35.8	ft-k	$M_{col,sleft}$=	30.8	ft-k
M_{ub} < M_{col}	Need Reinforcement		M_{ub} < M_{col}	Need Reinforcement	

Description	Value	Description	Value
Moment: M_u	47.3	Moment: M_u	102.7
Strip Width: b	45.5	Strip Width: b	45.5
Effective Depth: d	7.13	Effective Depth: d	7.13
M_u x 12/b	12.5	M_u x 12/b	27.1
$M_n = M_u/\phi$	52.6	$M_n = M_u/\phi$	114.1
$R = M_n \times 12000/bd^2$	273.3	$R = M_n \times 12000/bd^2$	592.6
ρ = See Table A.5a	0.00476	ρ = See Table A.5a	0.01093
ρ_{min} = See Table A.4	0.0033	ρ_{min} = See Table A.4	0.0033
ρ_{max} = See Table A.4	0.0206	ρ_{max} = See Table A.4	0.0206
Check ρ_{min}	OK	Check ρ_{min}	OK
Check ρ_{max}	OK	Check ρ_{max}	OK
Use ρ	0.00476	Use ρ	0.01093
$A_s = \rho bd$	1.54	$A_s = \rho bd$	3.54
$A_{s,min}$ = .0018bt	0.70	$A_{s,min}$ = .0018bt	0.70
Check $A_s > A_{s,min}$	OK	Check $A_s > A_{s,min}$	OK
Use A_s	1.54	Use A_s	3.54
No. of Bars	6	No. of Bars	12
Min No. of Bars	3	Min No. of Bars	3
Use No. of Bars	6	Use No. of Bars	12

Transferred by Eccentricity of Shear

V_u=	97.5	kips	V_u=	97.5	kips
M_{uv}=	10.1	ft-k	M_{uv}=	25.0	ft-k
Centroid=	13.56	in	Centroid=	13.56	in
J_c=	96434	in^4	J_c=	96434	in^4
A_c=	773	in^2	A_c=	773	in^2
v_l=	109	psi	v_l=	84	psi
v_r=	143	psi	v_r=	168	psi
v_u=	143	psi	v_u=	168	psi
ϕv_n=	190	psi	ϕv_n=	190	psi
ϕv_n > v_u	OK		ϕv_n > v_u	OK	

Interior Column BF (Reinforcement Needed)

$t_{col,1dir}$=	20	in	b_o=	108.50	in
$t_{col,2dir}$=	20	in	b_1=	27.13	in
$M_{u,long}$=	23.0	ft-k	b_2=	27.13	in
$M_{u,short}$=	31.2	ft-k	$V_{c,1}$=	195.6	kips
			$V_{c,2}$=	293.4	kips
			$V_{c,3}$=	226.2	kips
V_u=	88.8	kips	ϕV_c=	146.7	kips

Transferred by Flexure

γ_F=	0.600				
$M_{ub,long}$=	13.8	ft-k	$M_{ub,short}$=	18.7	ft-k
$M_{ub,totlong}$=	81.8	ft-k	$M_{ub,totshort}$=	114.7	ft-k
$M_{col,lleft}$=	35.8	ft-k	$M_{col,sleft}$=	37.9	ft-k
M_{ub} < M_{col}	Need Reinforcement		$M_{col,sright}$=	30.8	ft-k
			M_{ub} < M_{col}	Need Reinforcement	

Description	Left Side
Moment: M_u	46.0
Strip Width: b	45.5
Effective Depth: d	7.13
M_u x 12/b	12.1
$M_n = M_u/\phi$	51.1
$R = M_n$ x $12000/bd^2$	265.4
ρ = See Table A.5a	0.00461
ρ_{min} = See Table A.4	0.0033
ρ_{max} = See Table A.4	0.0206
Check ρ_{min}	OK
Check ρ_{max}	OK
Use ρ	0.00461
$A_s = \rho bd$	1.49
$A_{s,min}$ = .0018bt	0.70
Check $A_s > A_{s,min}$	OK
Use A_s	1.49
No. of Bars	5
Min No. of Bars	3
Use No. of Bars	5

Description	Left Side	Right Side
Moment: M_u	76.8	83.9
Strip Width: b	45.5	45.5
Effective Depth: d	7.13	7.13
M_u x 12/b	20.3	22.1
$M_n = M_u/\phi$	85.4	93.3
$R = M_n$ x $12000/bd^2$	443.5	484.5
ρ = See Table A.5a	0.00795	0.00875
ρ_{min} = See Table A.4	0.0033	0.0033
ρ_{max} = See Table A.4	0.0206	0.0206
Check ρ_{min}	OK	OK
Check ρ_{max}	OK	OK
Use ρ	0.00795	0.00875
$A_s = \rho bd$	2.58	2.84
$A_{s,min}$ = .0018bt	0.70	0.70
Check $A_s > A_{s,min}$	OK	OK
Use A_s	2.58	2.84
No. of Bars	9	10
Min No. of Bars	3	3
Use No. of Bars	9	10

Transferred by Eccentricity of Shear

V_u=	88.8	kips				V_u=	88.8	kips			
$M_{uv,long}$=	9.2	ft-k				$M_{uv,short}$=	12.5	ft-k			
Centroid=	13.56	in				Centroid=	13.56	in			
J_c=	96434	in^4				J_c=	96434	in^4			
A_c=	773	in^2				A_c=	773	in^2			
v_l=	99	psi				v_l=	94	psi			
v_r=	130	psi				v_r=	136	psi			
v_u=	130	psi				v_u=	136	psi			
ϕv_n=	190	psi	>	v_u	OK	ϕv_n=	190	psi	>	v_u	OK

Exterior Column AF (Reinforcement Needed)

$t_{col,1dir}$=	20	in	b_o=	74.25	in
$t_{col,2dir}$=	20	in	b_1=	27.13	in
$M_{u,long}$=	12.1	ft-k	b_2=	23.56	in
$M_{o,short}$=	159.5	ft-k	V_{c1}=	133.8	kips
$M_{u,short}$=	47.8	ft-k	V_{c2}=	200.8	kips
			V_{c3}=	163.2	kips
V_u=	46.7	kips	ϕV_c=	100.4	kips

Transferred by Flexure

γ_f=	0.583				
$M_{ub,long}$=	7.0	ft-k	$M_{ub,short}$=	27.9	ft-k
$M_{ub,totlong}$=	75.0	ft-k	$M_{ub,totshort}$=	123.9	ft-k
$M_{col,lleft}$=	25.8	ft-k	$M_{col,sright}$=	17.8	ft-k
M_{ub} < M_{col}	Need Reinforcement		M_{ub} < M_{col}	Need Reinforcement	

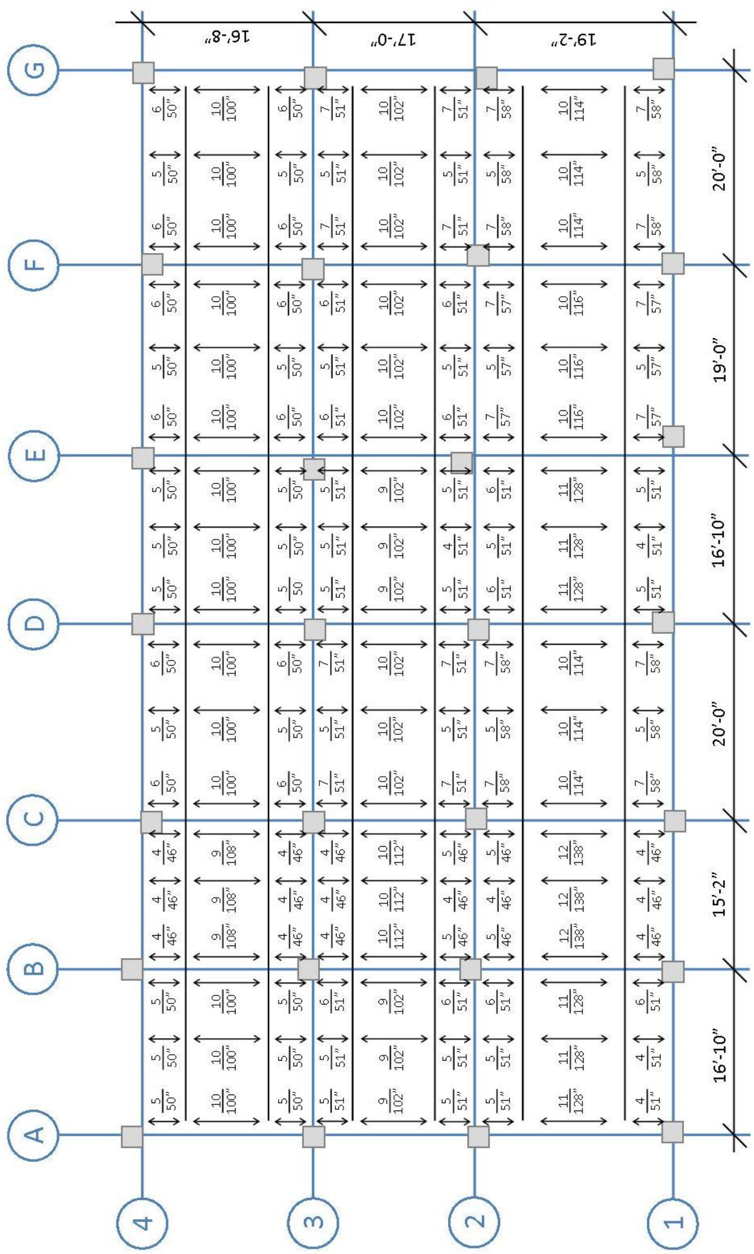
Description	Left Side	Description	Value
Moment: M_u	49.3	Moment: M_u	106.1
Strip Width: b	32.75	Strip Width: b	45.5
Effective Depth: d	7.13	Effective Depth: d	7.13
M_u x 12/b	18.1	M_u x 12/b	28.0
$M_n = M_u/\phi$	54.8	$M_n = M_u/\phi$	117.9
$R = M_n$ x 12000/bd^2	395.2	$R = M_n$ x 12000/bd^2	612.5
ρ = See Table A.5a	0.007	ρ = See Table A.5a	0.01134
ρ_{min} = See Table A.4	0.0033	ρ_{min} = See Table A.4	0.0033
ρ_{max} = See Table A.4	0.0206	ρ_{max} = See Table A.4	0.0206
Check ρ_{min}	OK	Check ρ_{min}	OK
Check ρ_{max}	OK	Check ρ_{max}	OK
Use ρ	0.007	Use ρ	0.01134
$A_s = \rho bd$	1.63	$A_s = \rho bd$	3.68
$A_{s,min}$ = .0018bt	0.50	$A_{s,min}$ = .0018bt	0.70
Check $A_s > A_{s,min}$	OK	Check $A_s > A_{s,min}$	OK
Use A_s	1.63	Use A_s	3.68
No. of Bars	6	No. of Bars	12
Min No. of Bars	2	Min No. of Bars	3
Use No. of Bars	6	Use No. of Bars	12

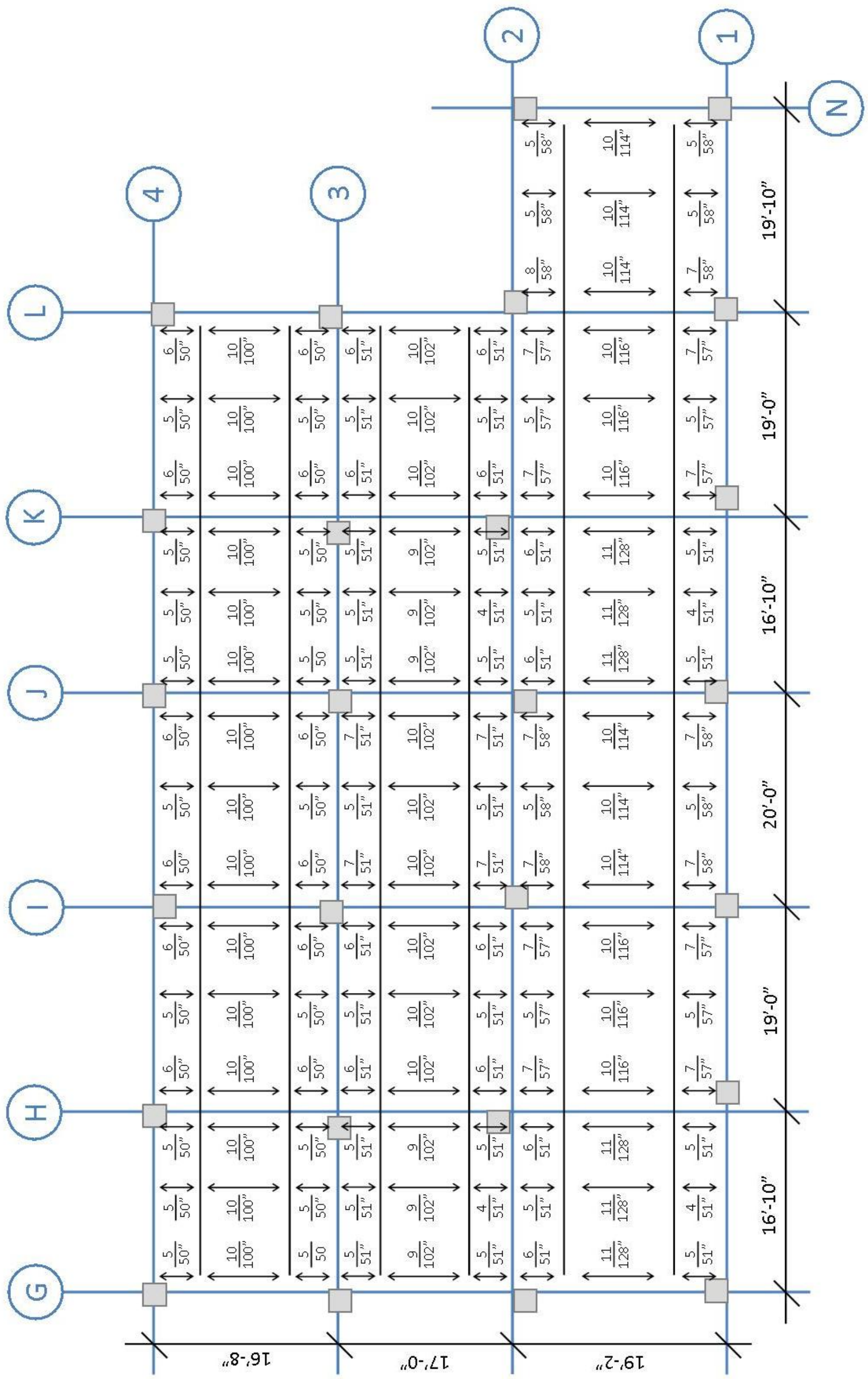
Transferred by Eccentricity of Shear

V_u=	46.7	kips	V_u=	46.7	kips
$M_{uv,long}$=	5.0	ft-k	$M_{uv,short}$=	20.0	ft-k
Centroid=	13.56	in	Centroid=	7.48	in
J_c=	87096	in^4	J_c=	33980	in^4
A_c=	529	in^2	A_c=	529	in^2
v_l=	79	psi	v_l=	-25	psi
v_r=	98	psi	v_r=	141	psi
v_u=	98	psi	v_u=	141	psi
ϕv_n=	190	psi	ϕv_n=	190	psi
ϕv_n > v_u	OK		ϕv_n > v_u	OK	

Appendix H: Gravity System Reinf

Second Floor




1
2
3
4
N
L
K
J
I
H
G
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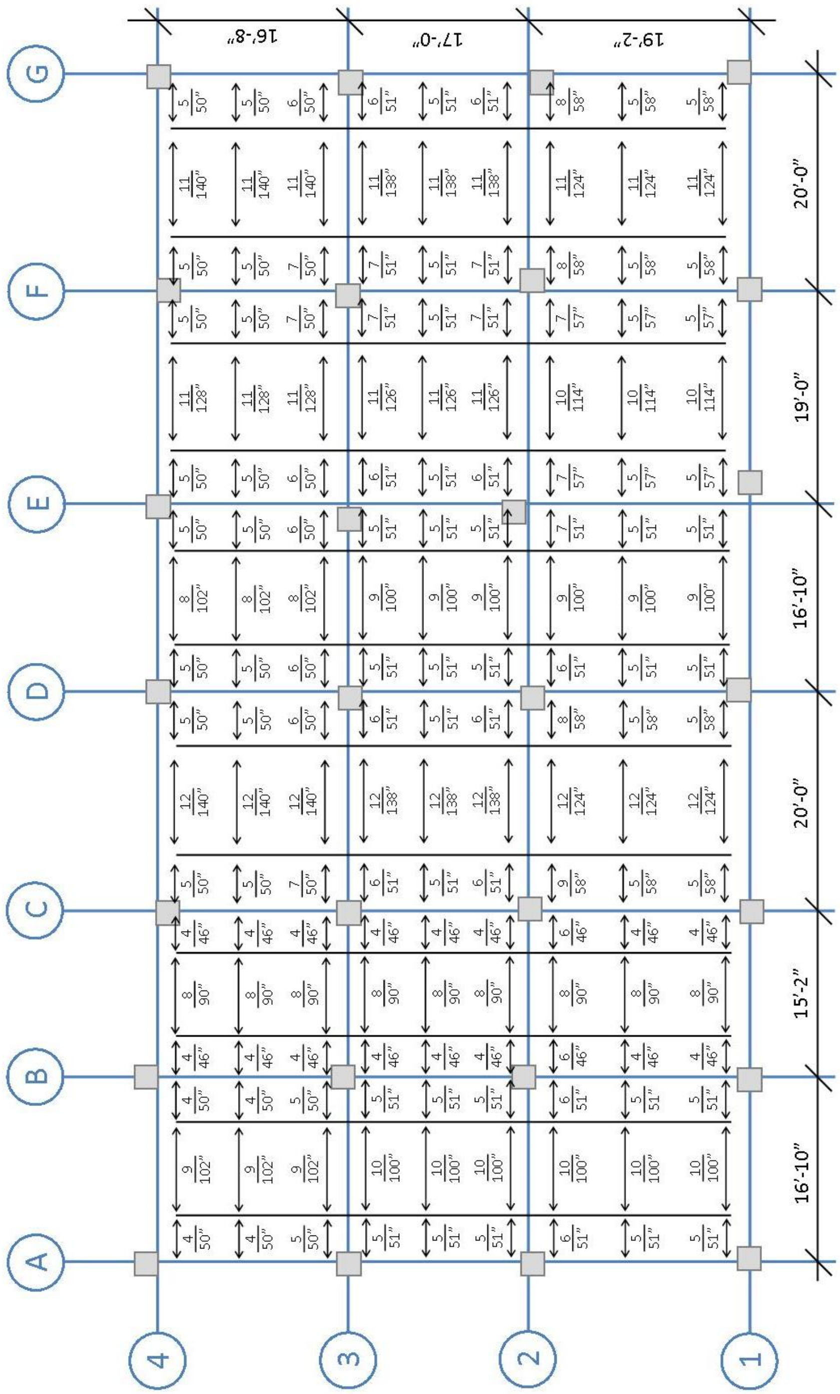
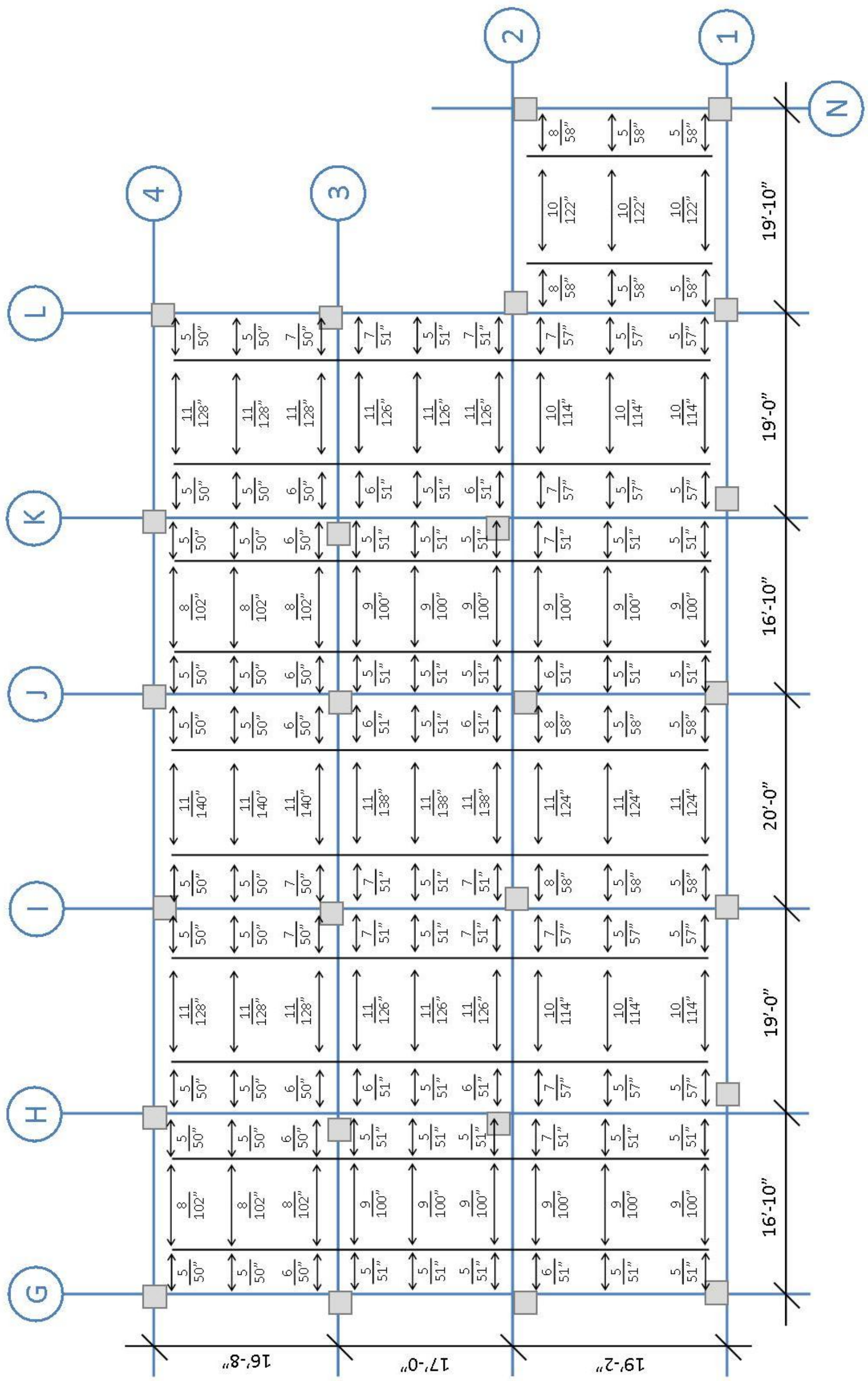
19'-0"
16'-10"
20'-0"
19'-0"
16'-10"
16'-8"
17'-0"
19'-2"

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4
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L
K
J
I
H
G
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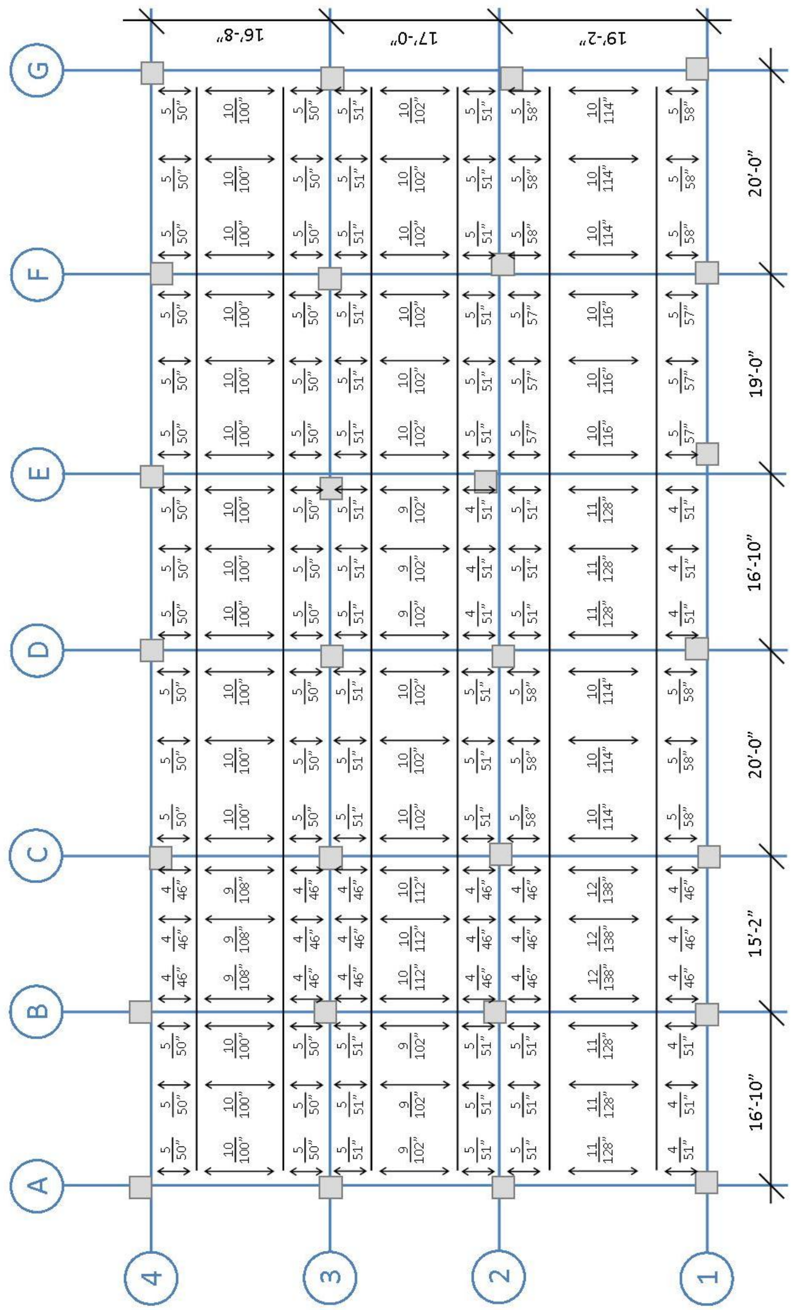
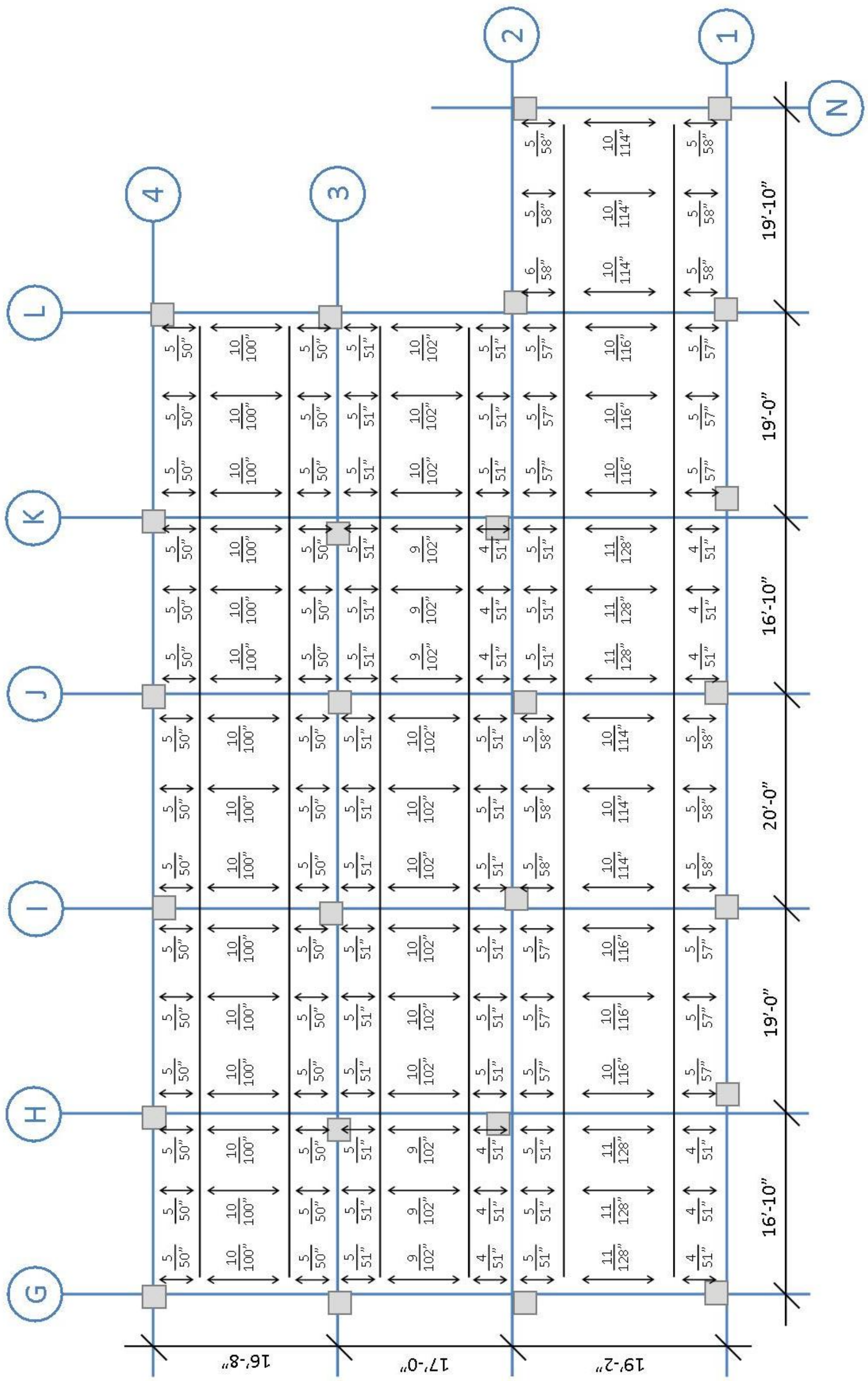
16'-10"
16'-8"
17'-0"
19'-2"

Third Floor

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J
I
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G
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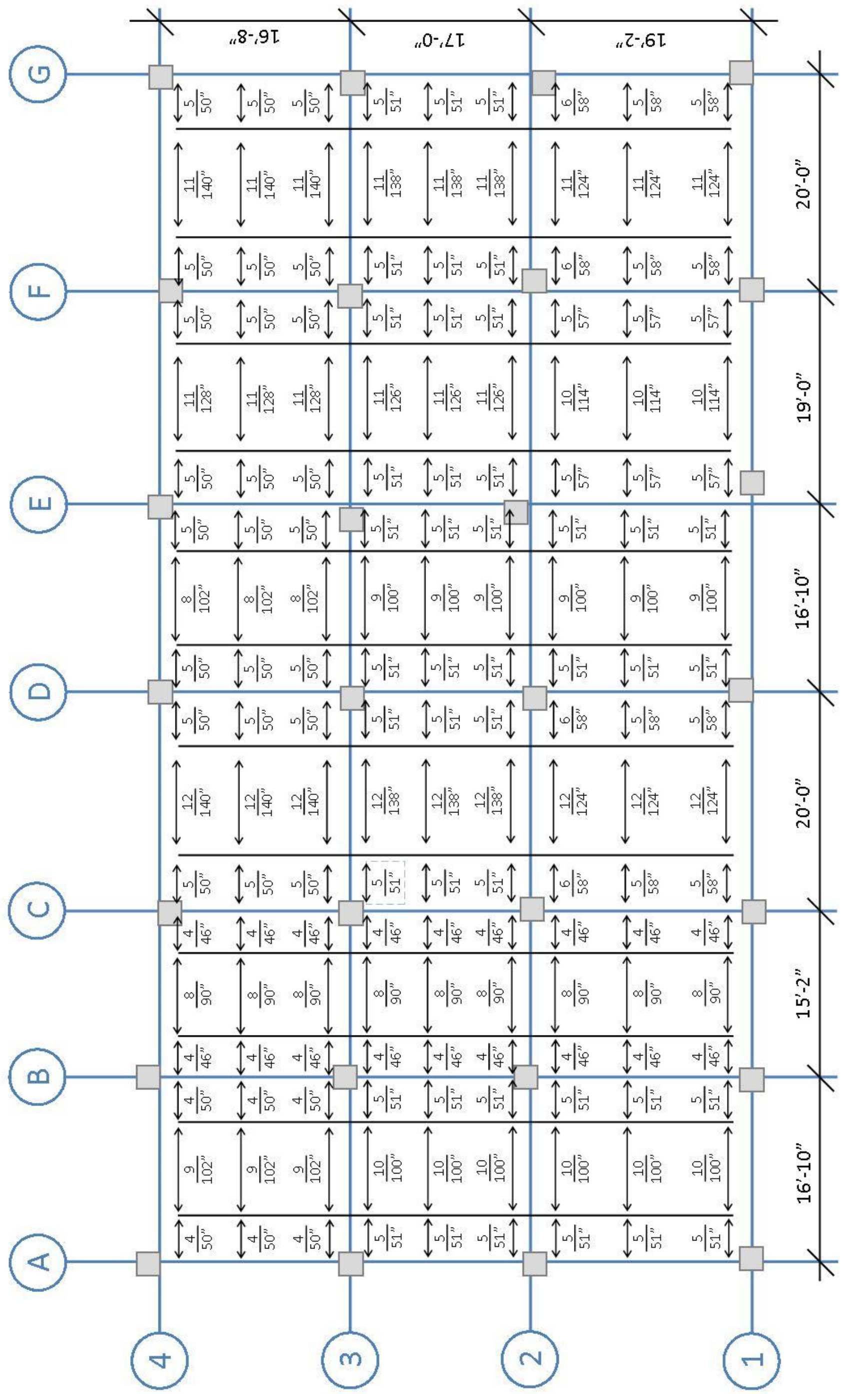
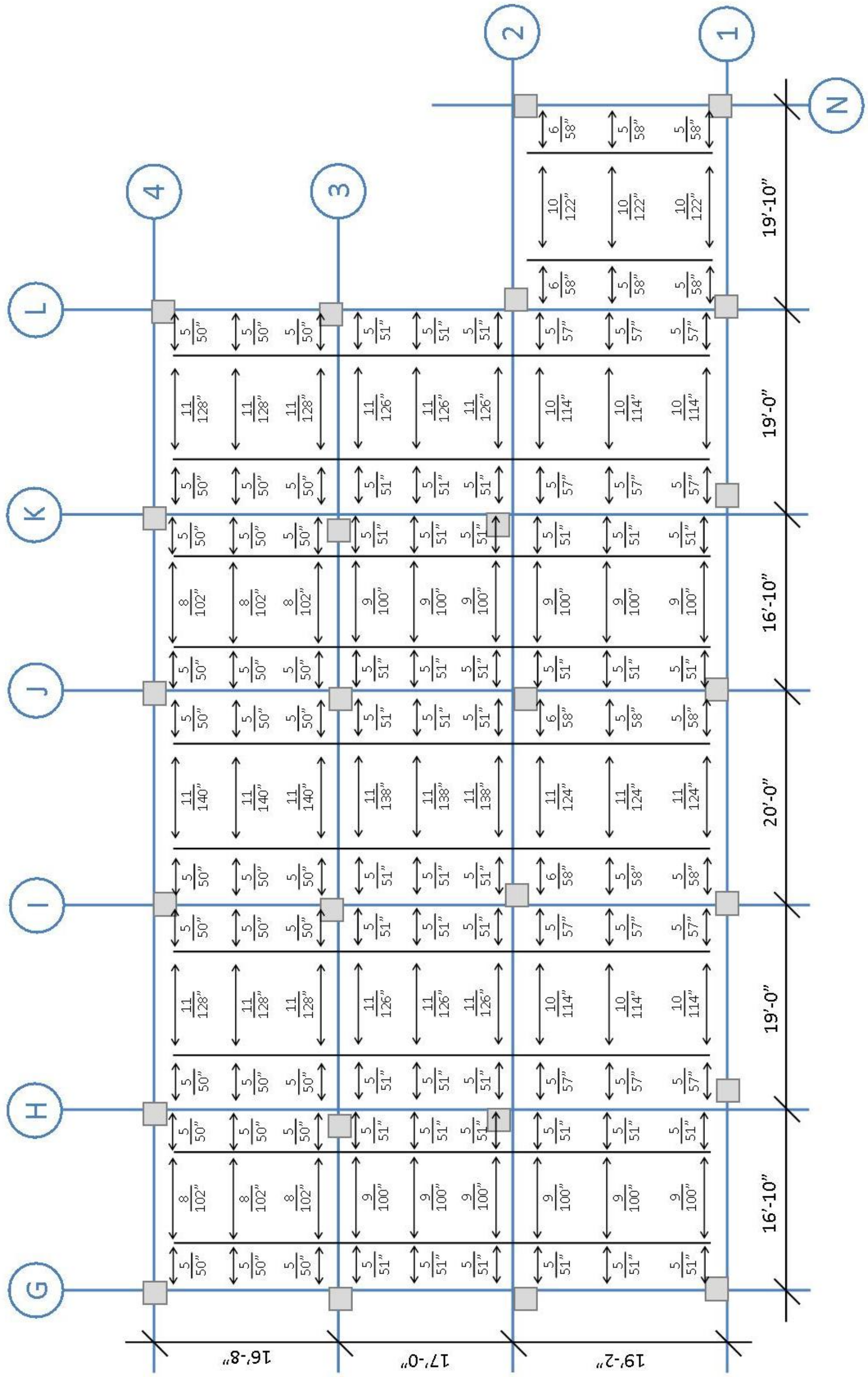
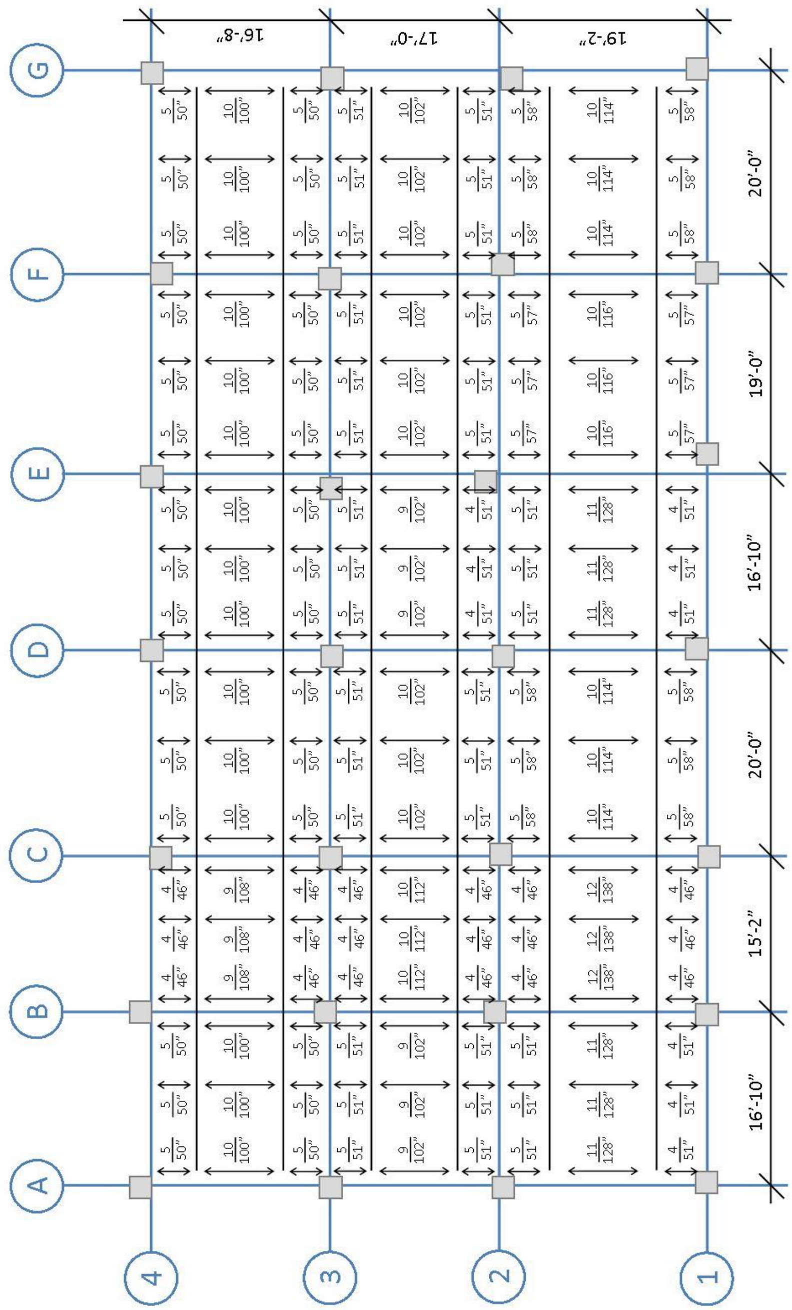
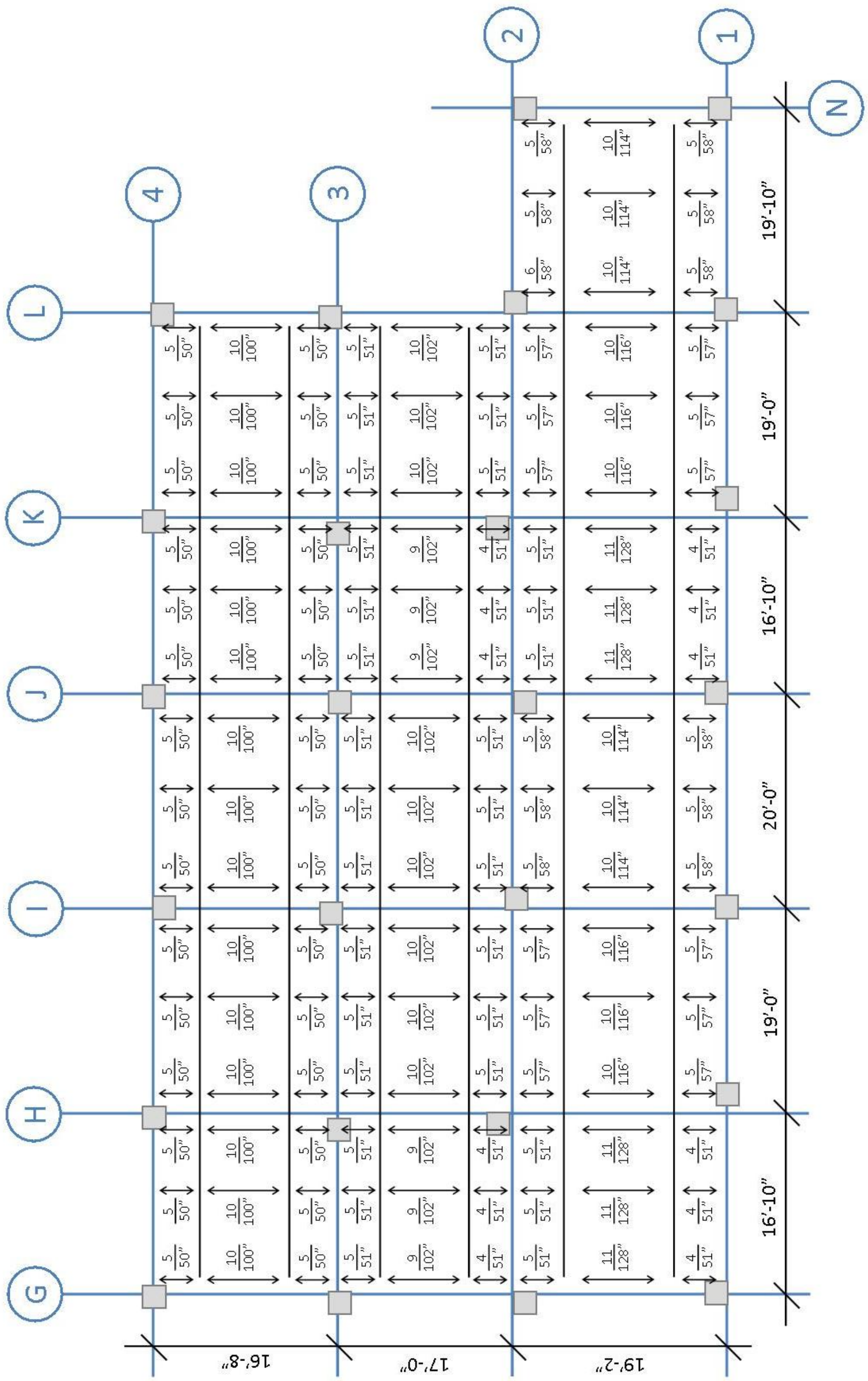
16'-10"
16'-8"
17'-0"
19'-2"

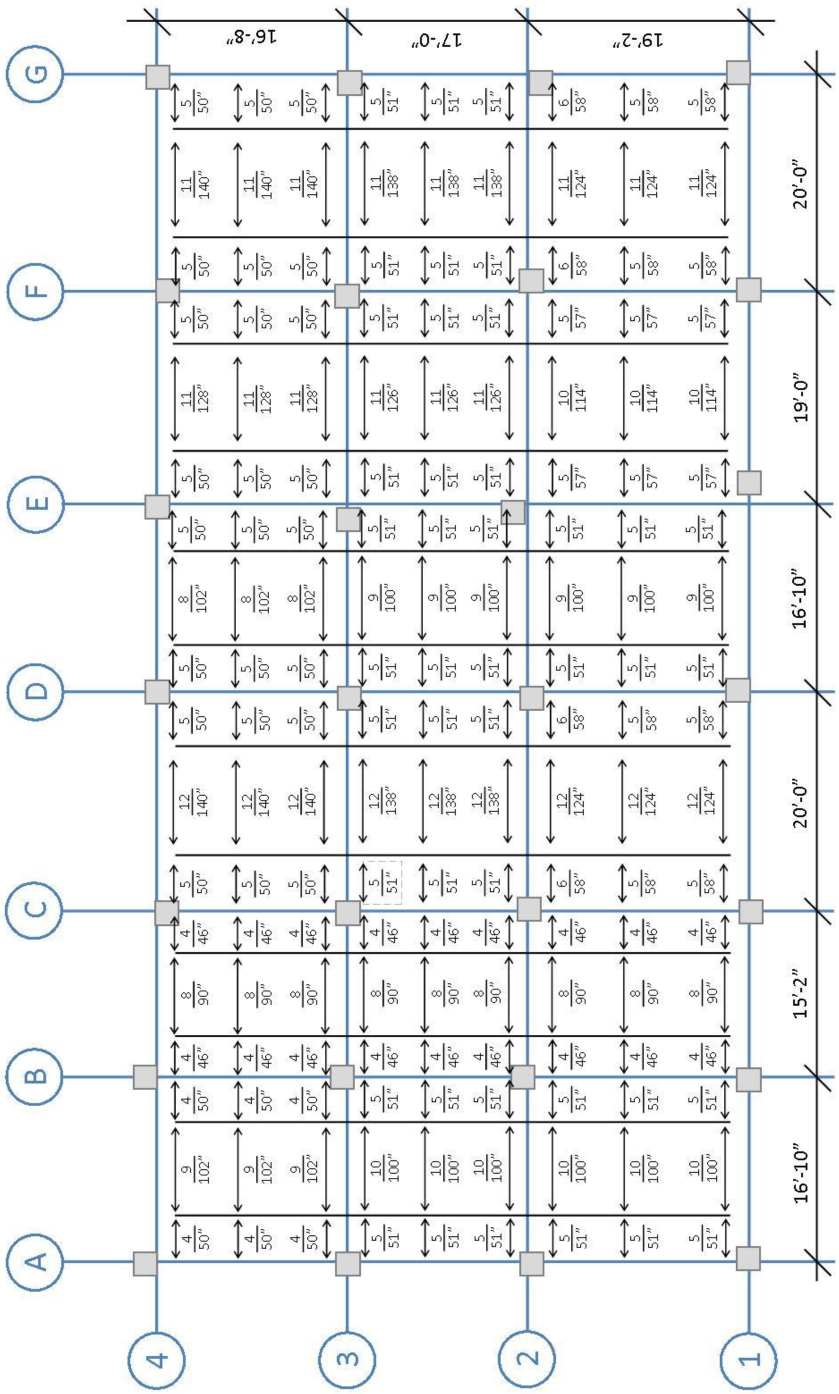


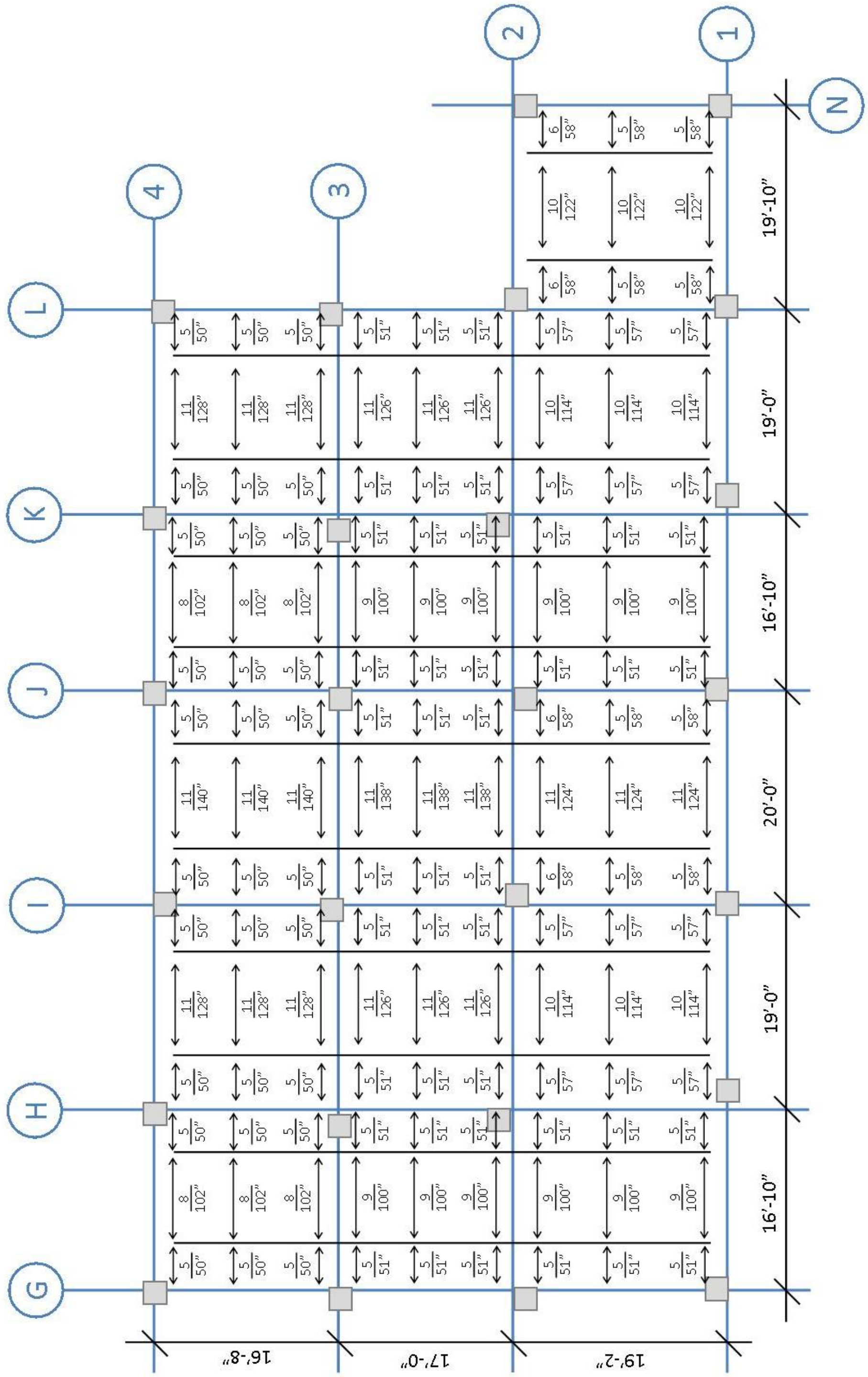


Fourth Floor

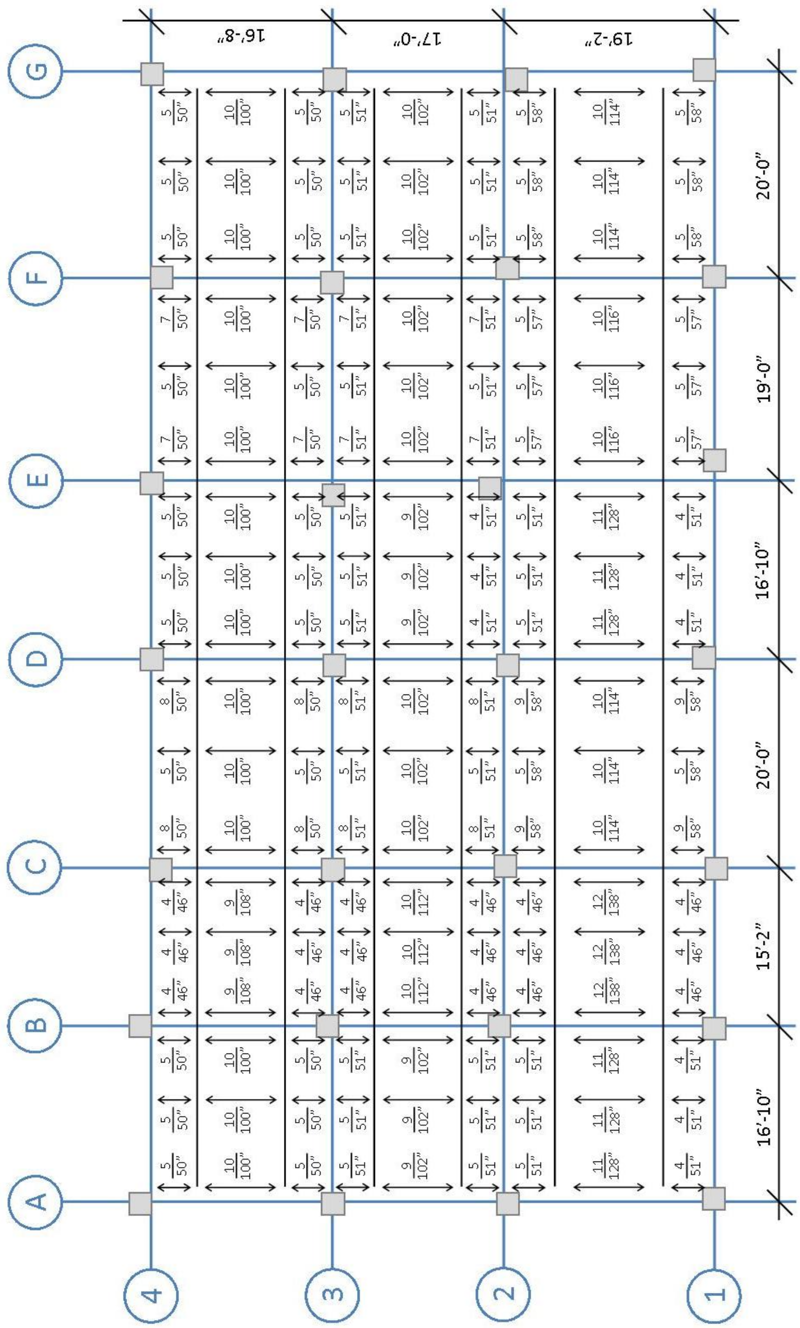


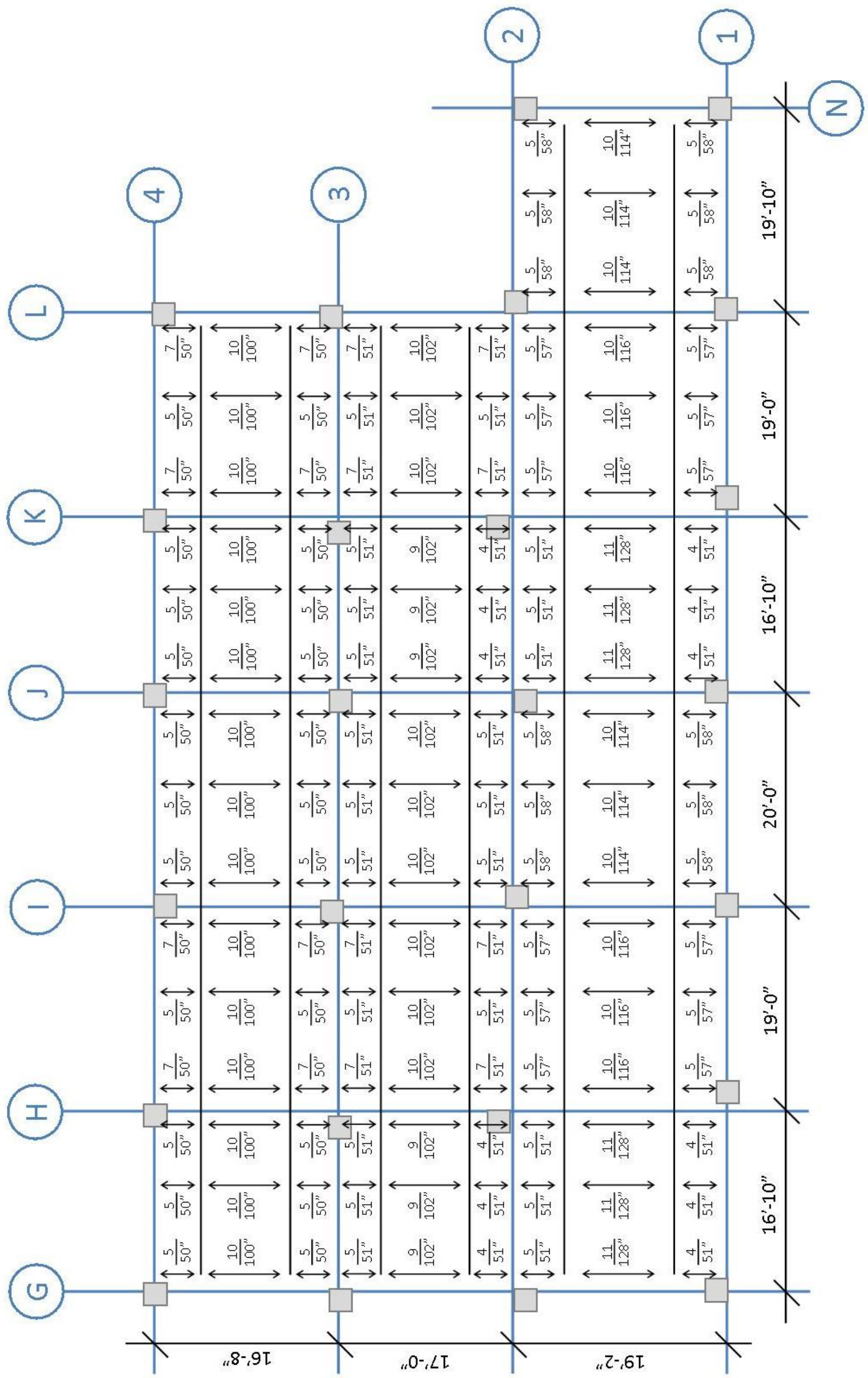

1
2
3
4
N
L
K
J
I
H
G
19'-10"
19'-0"
16'-10"
20'-0"
19'-0"
16'-10"
16'-8"
17'-0"
19'-2"


G
F
E
D
C
B
A
4
3
2
1
20'-0"
19'-0"
16'-10"
20'-0"
15'-2"
16'-10"



Mechanical Penthouse

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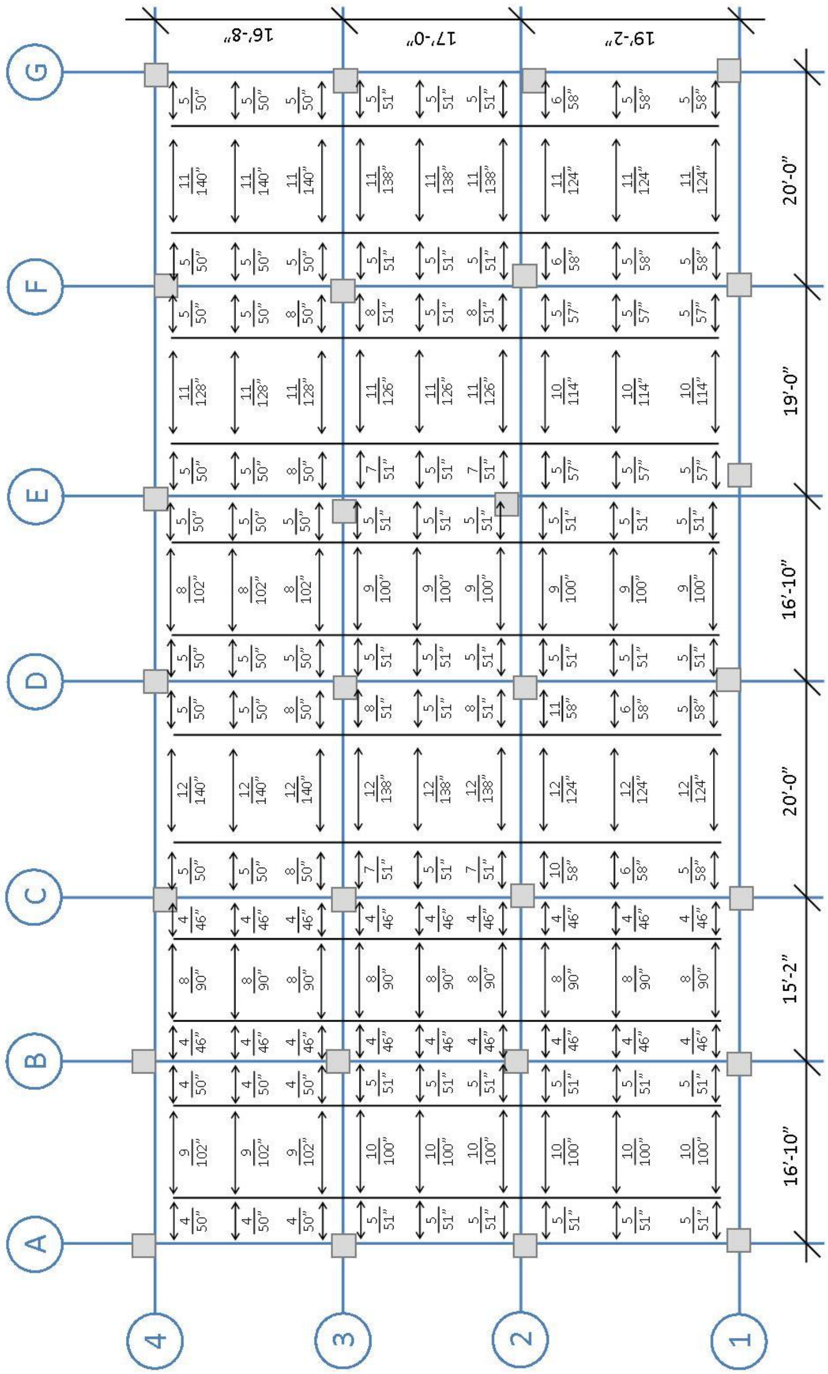
19'-2"



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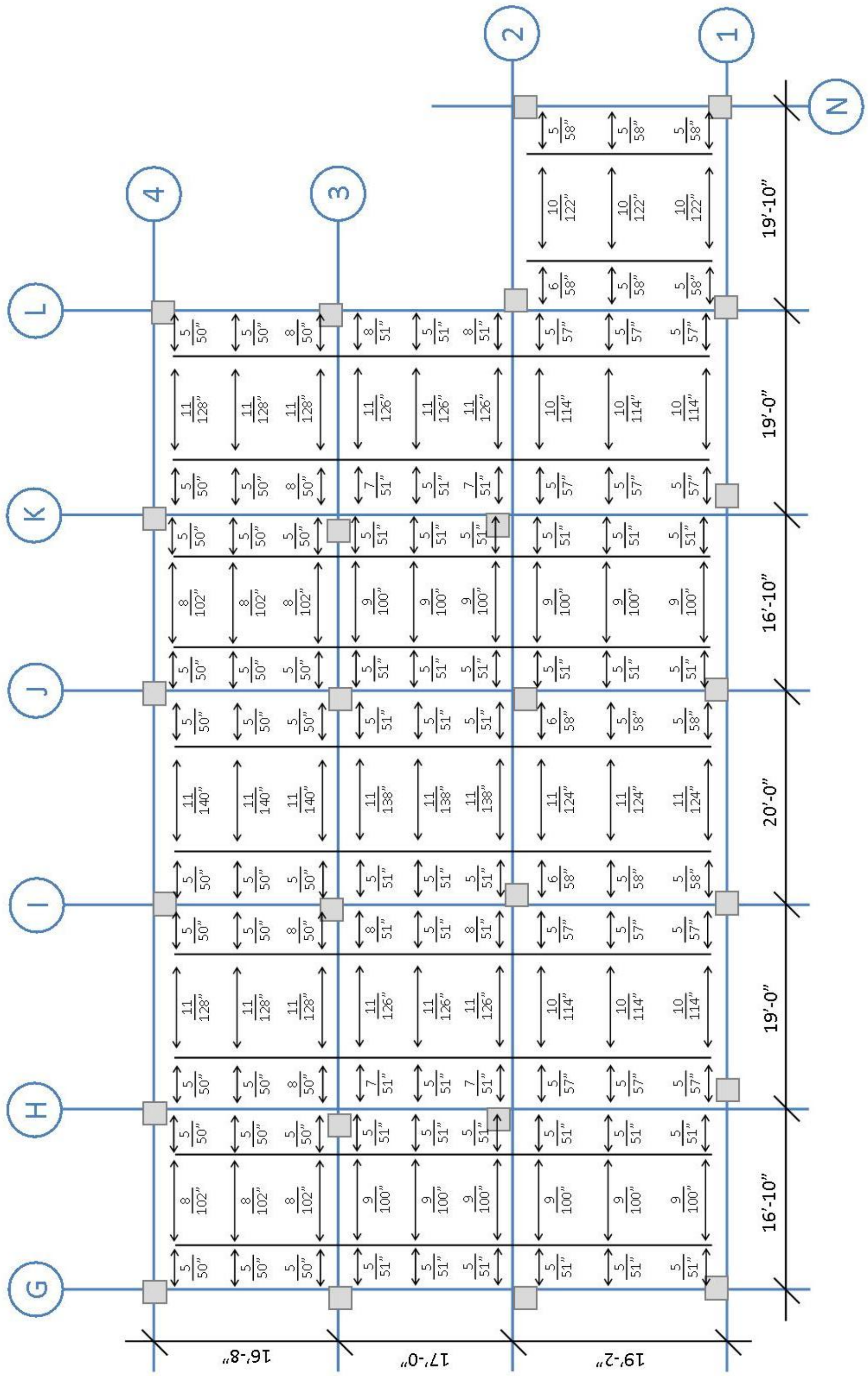

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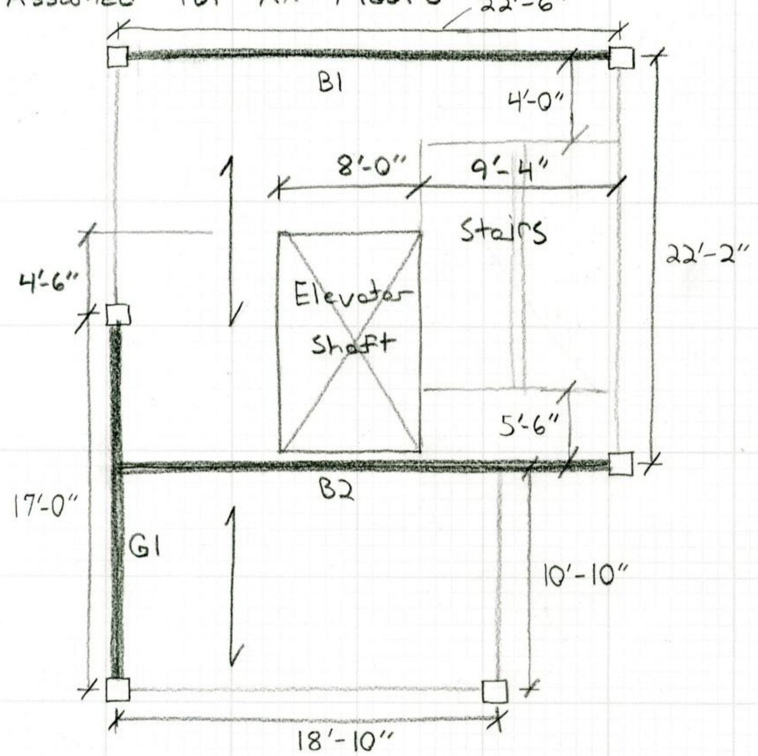
19'-2"

Appendix I: Stairwell Corner Analysis

MULKEY ENGINEERS & CONSULTANTS
P.O. Box 33127 • Raleigh, NC 27636-3127
Phone: (919) 851-1912 • Fax: (919) 851-1918

PROJECT NAME Corner Analysis
PROJECT NO. SHEET 1 OF 4
SUBJECT
PREPARED BY DATE CHECKED BY DATE

Assumed For All Floors



→ Dead Loads:

Slab Weight: 106 psf
MEP: 10 psf
SDL: 5 psf

→ Live Load:

Stairway: 100 psf (Non-Reducible)

ME-02

MULKEY ENGINEERS & CONSULTANTS

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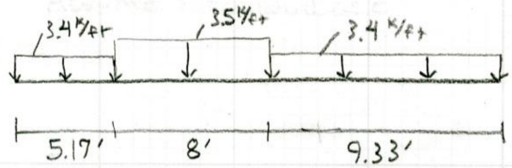
Phone: (919) 851-1912 • Fax: (919) 851-1918

PROJECT NAME: Corner Analysis
PROJECT NO. ____ SHEET 2 OF 4
SUBJECT ____
PREPARED BY ____ DATE ____ CHECKED BY ____ DATE ____

→ Beam B1



$\omega_D = 1.34$ K/ft = 1.39 K/ft = 1.34 K/ft

$\omega_L = 1.11$ K/ft = 1.15 K/ft = 1.11 K/ft

$\omega_t = 1.2\omega_D + 1.6\omega_L$

= 3.4 K/ft = 3.5 K/ft = 3.4 K/ft

└ use worst case to design beam

$$M_u = \frac{\omega_u l_n^2}{8} = \frac{3.5(22.5 - 1.67)^2}{8} \times 1.1 = 209 \text{ k-ft}$$

└ self wt estimate of 10%

Estimate size: $bd^2 = 20M_u$, Try $b = \frac{4}{5}d$

$d^3 = 20(209)(5/4)$

$\rightarrow d = 17.3''$

$h = d + 2.5$ use $h = 20''$ $b = 14''$

$bd^2 = 4288 \text{ in}^3$

Self Wt Effects:

$\omega_{sw} = \frac{14(20)}{144} \times 150 = 291.7$ plf

$\omega_u = 3500 + 1.2(292) = 3850$ plf

$M_u = 3.85 \times 20.83^2/8 = 209$ k-ft

$20 \times 209 = 4180 < 4288$ ✓ ok

Required steel

$A_s = \frac{M_u}{4d} = \frac{209}{4(17.5)}$

$= 3.0 \text{ in}^2$

Use (3) #4

Use b=14", h=20" with (3) #9

ME-02

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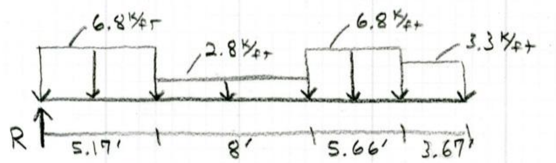
PROJECT NAME Corner Analysis
PROJECT NO. SHEET 3 OF 4
SUBJECT
PREPARED BY DATE CHECKED BY DATE

→ Beam B2

For Girder G1:
From $\Sigma M = 0$,
$R = 56.5^k$



$w_D = 2.7$ k/ft = 1.3 k/ft = 2.7 k/ft = 1.3 k/ft
$w_L = 2.2$ k/ft = 1.1 k/ft = 2.2 k/ft = 1.1 k/ft
$w_T = 6.8$ k/ft = 2.8 k/ft = 6.8 k/ft = 3.3 k/ft
└ use worst case to design beam

$$M_u = \frac{w_u l_n^2}{8} = \frac{6.8(22.5-1.67)^2}{8} \times 1.1 = 406 \text{ k-ft}$$

Estimate size

$d^3 = 20(406)(5/4)$ $h = d + 2.5$

$\rightarrow d = 21.7''$

Use $h = 25''$ $b = 18''$ $bd^2 = 4113 \text{ in}^3$

Self Wt Effects:

$$w_{sw} = \frac{18(25)}{144} \times 150 = 464 \text{ plf}$$

$$w_u = 6800 + 1.2(469) = 7363 \text{ plf}$$

$$M_u = \frac{7.363(20.83^2)}{8} = 399 \text{ k-ft}$$

$20 \times 399 = 7980 < 9113$
✓ok

Required Steel

$$A_s = \frac{M_u}{4d} = \frac{399}{4(22.5)} = 4.433$$ use (5) #9

Use b=18", h=25" with (5) #9

ME-02

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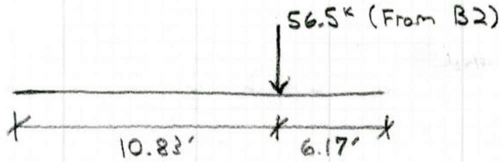
PROJECT NAME Corner Analysis
PROJECT NO. SHEET 4 OF 4
SUBJECT
PREPARED BY DATE CHECKED BY DATE

→ Girder G1



$$M_u = \frac{Pab}{l} = \frac{56.5(6.17-\frac{1.67}{2})(10.83-\frac{1.67}{2})}{(17-1.67)}$$

$$= 196.5^{k\text{-}ft} \times 1.1 = 216^{k\text{-}ft}$$

Estimate size

$bd^2 = 20 M_u$

↳ use same d as B2

$b(22.5)^2 = 20(216) \rightarrow b = 8.533$

use $b = 12''$ for cover reasons

$bd^2 = 6075 \text{ in}^3$

Self Wt Effects:

$w_{sw} = \frac{12(25)}{144} \times 150 = 313 \text{ plf}$

$M_u = 216 + \frac{1.2(.313)(17-1.67)^2}{8}$

$= 227^{k\text{-}ft}$ $20 \times 227 = 4540 < 6075$ ✓ok

Required Steel

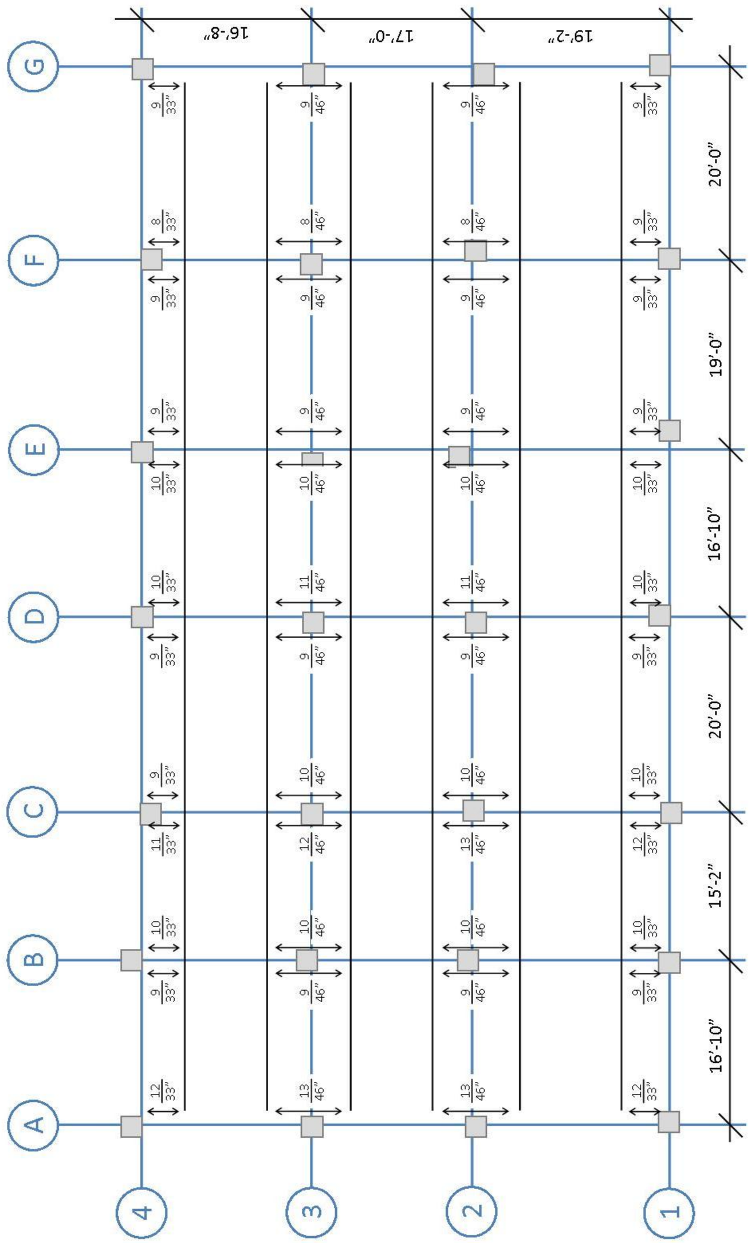
$A_s = \frac{M_u}{4d} = \frac{227}{4(22.5)} = 2.52$ use (3) #9

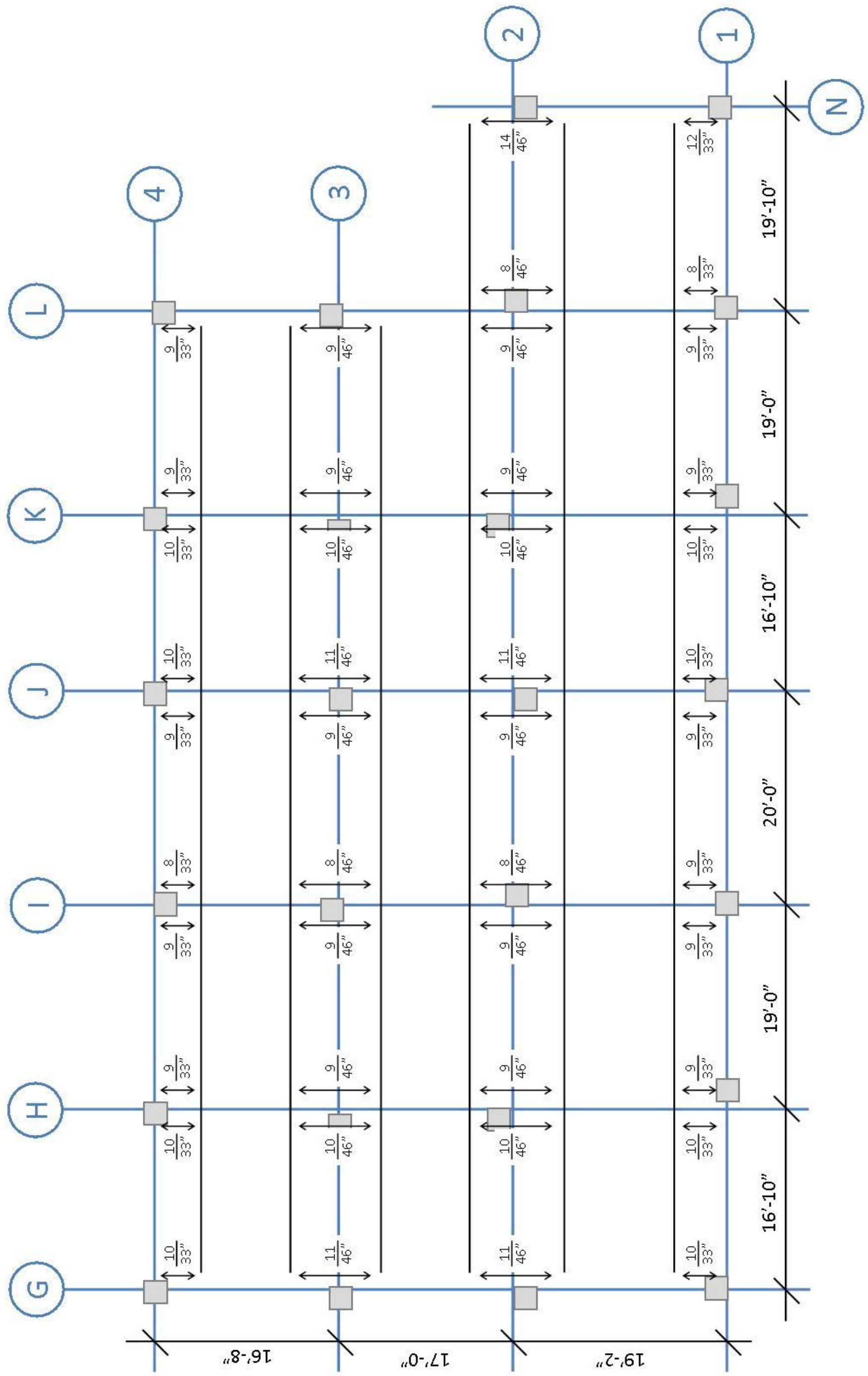
Use $b = 12''$, $h = 25''$ with (3) #9

ME-02

Appendix J: Lateral System Reinf

Second Floor

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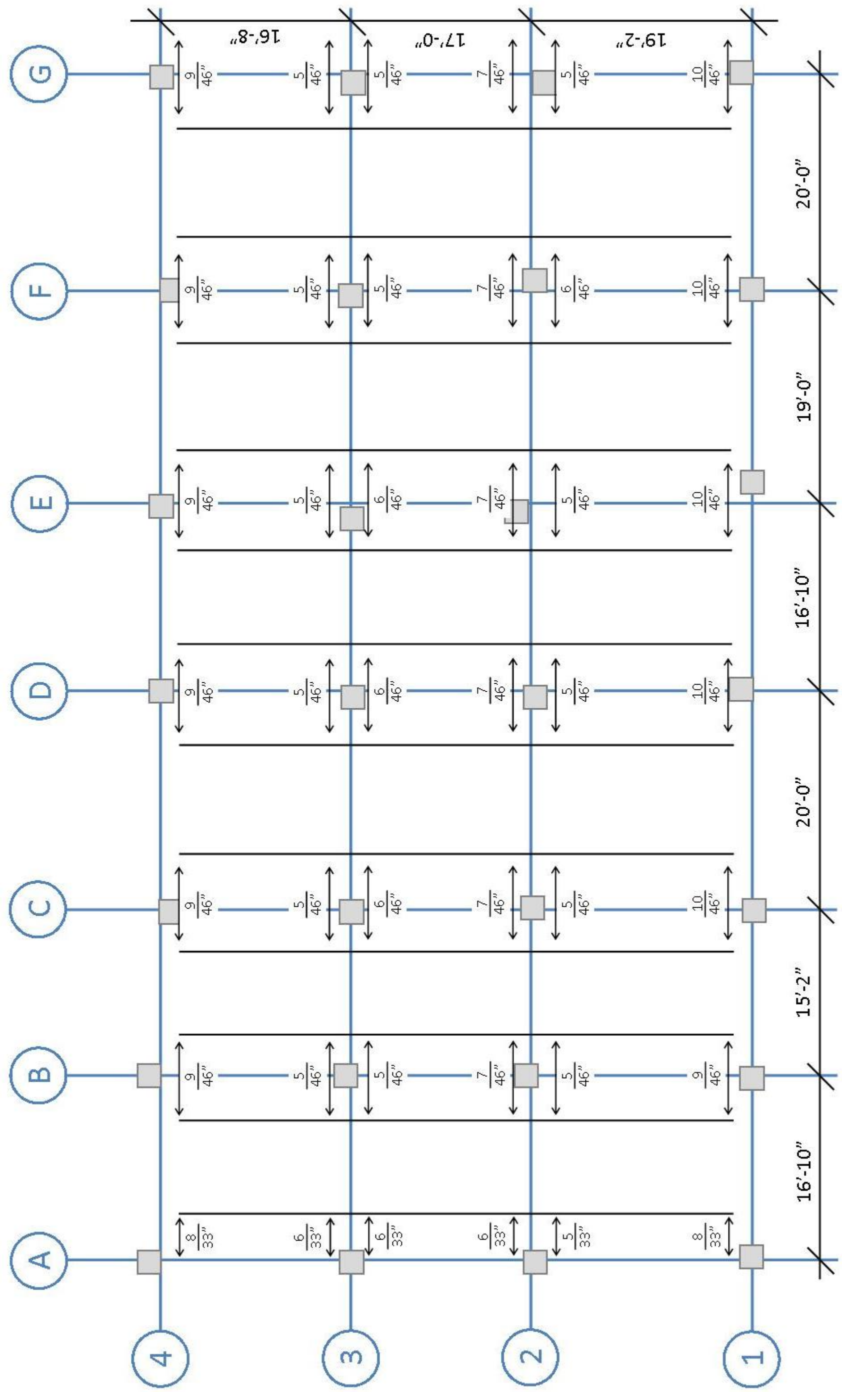

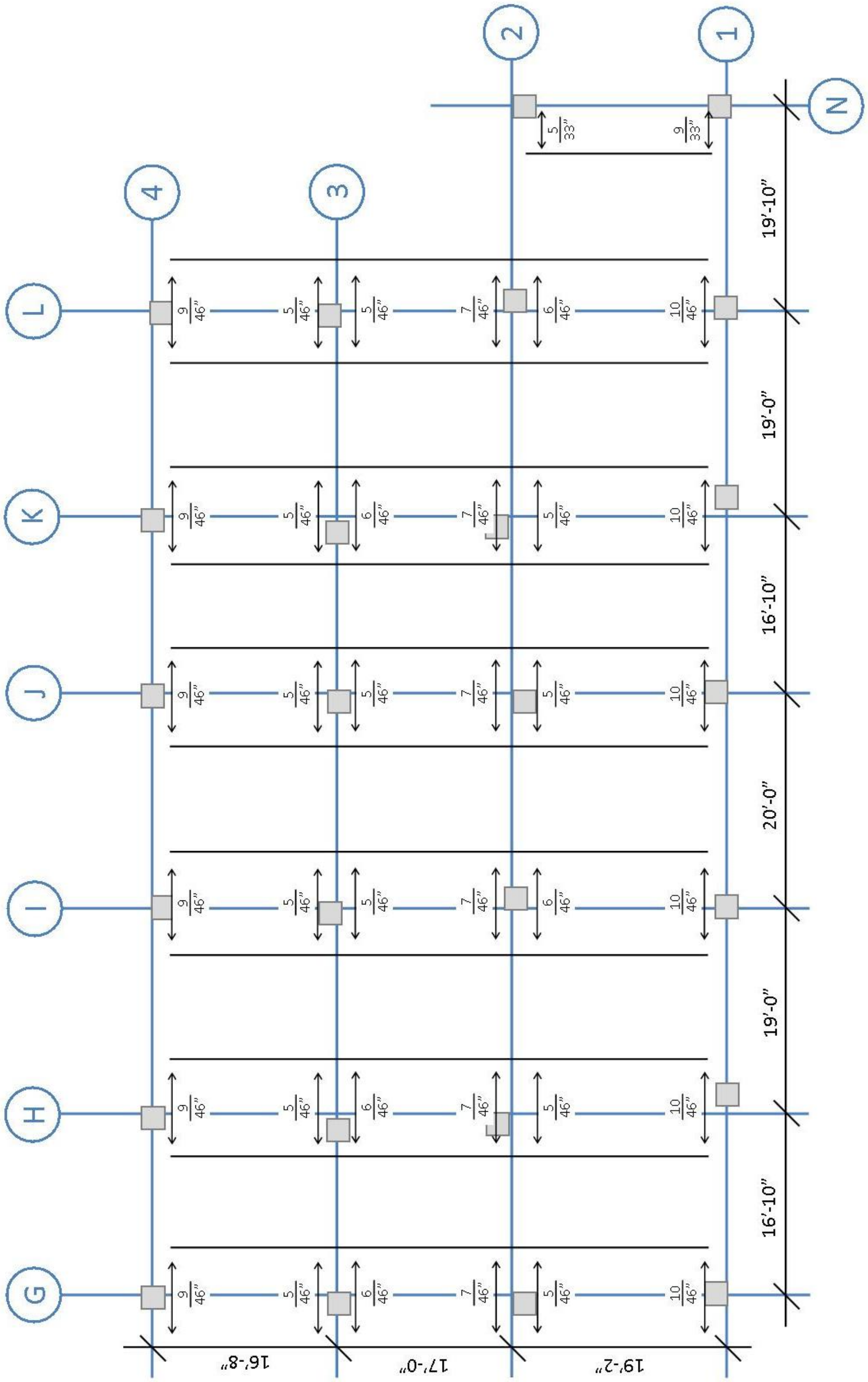
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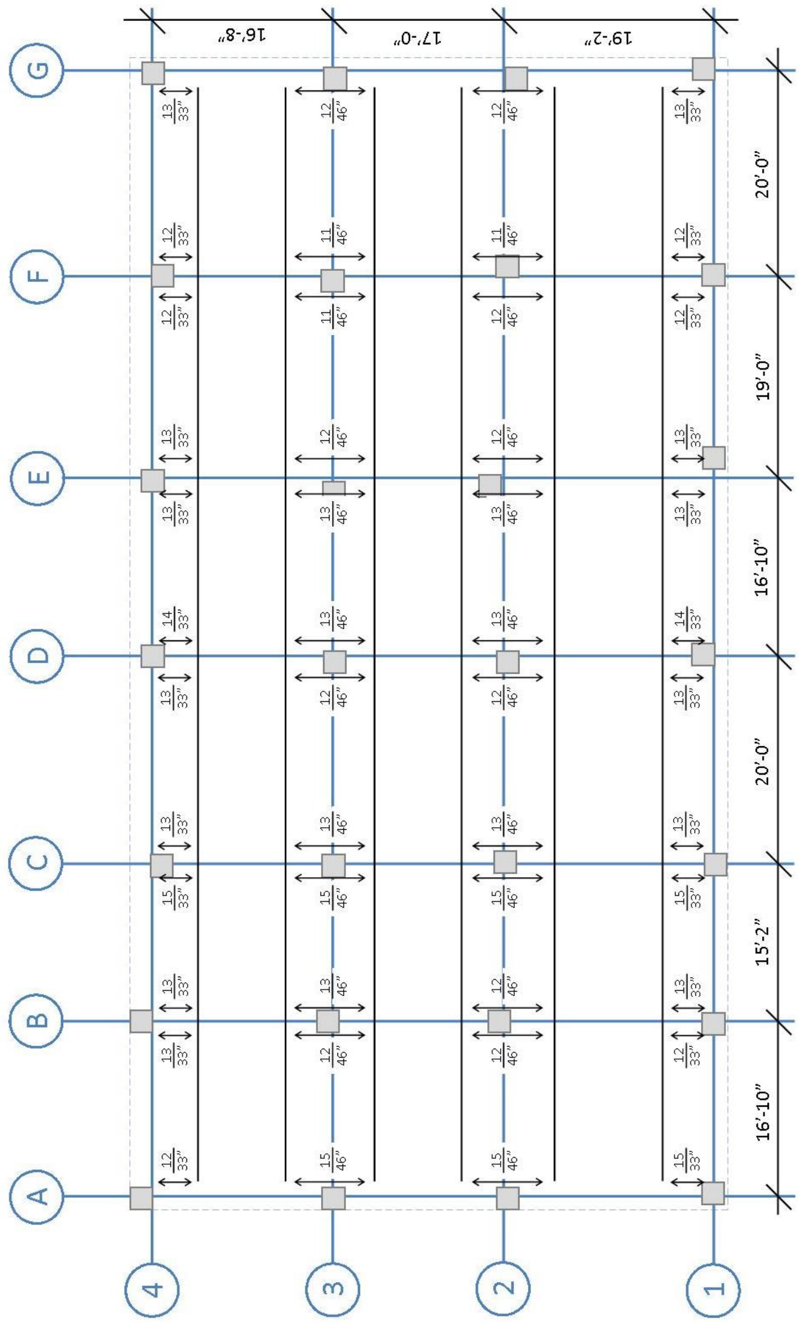
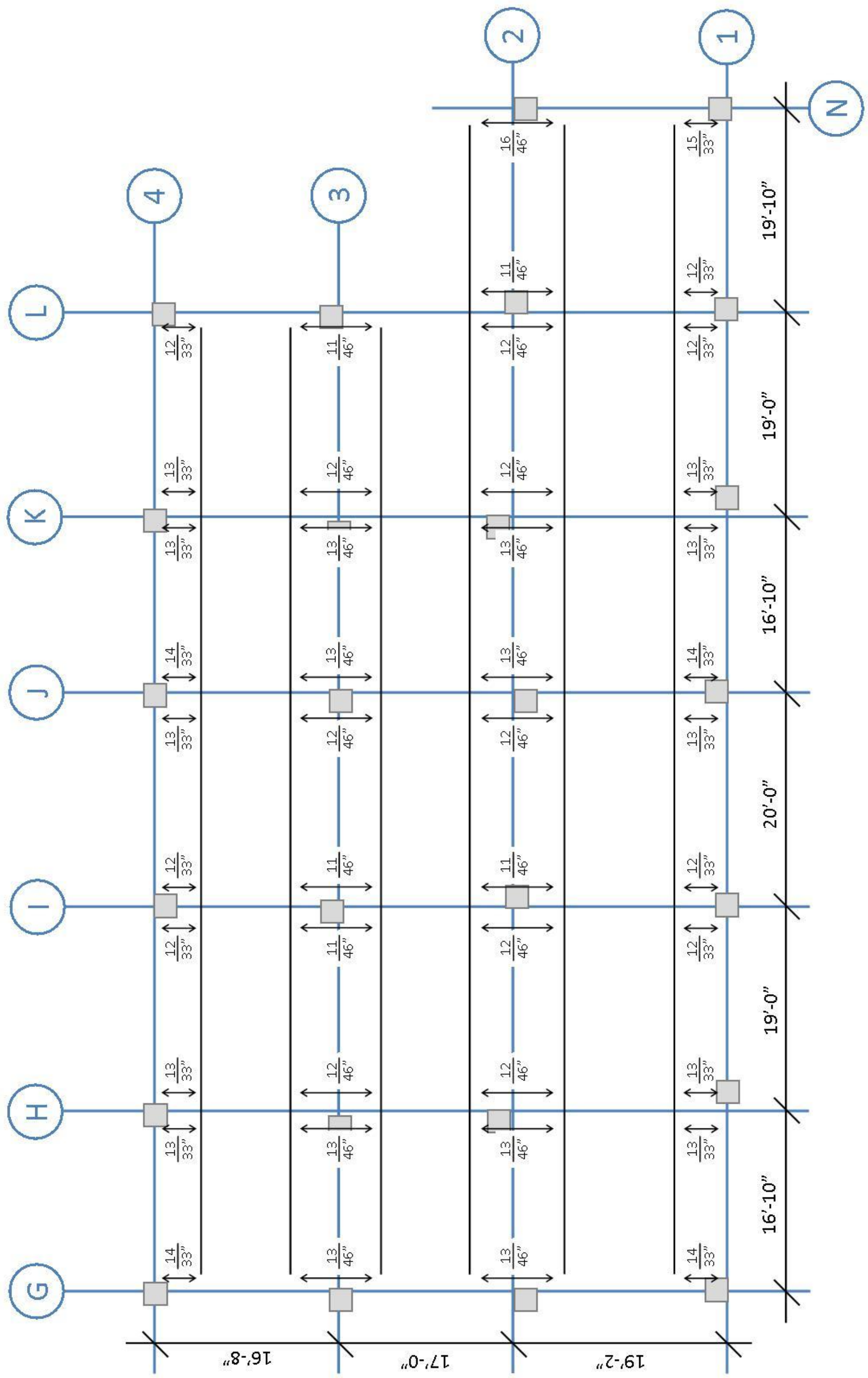
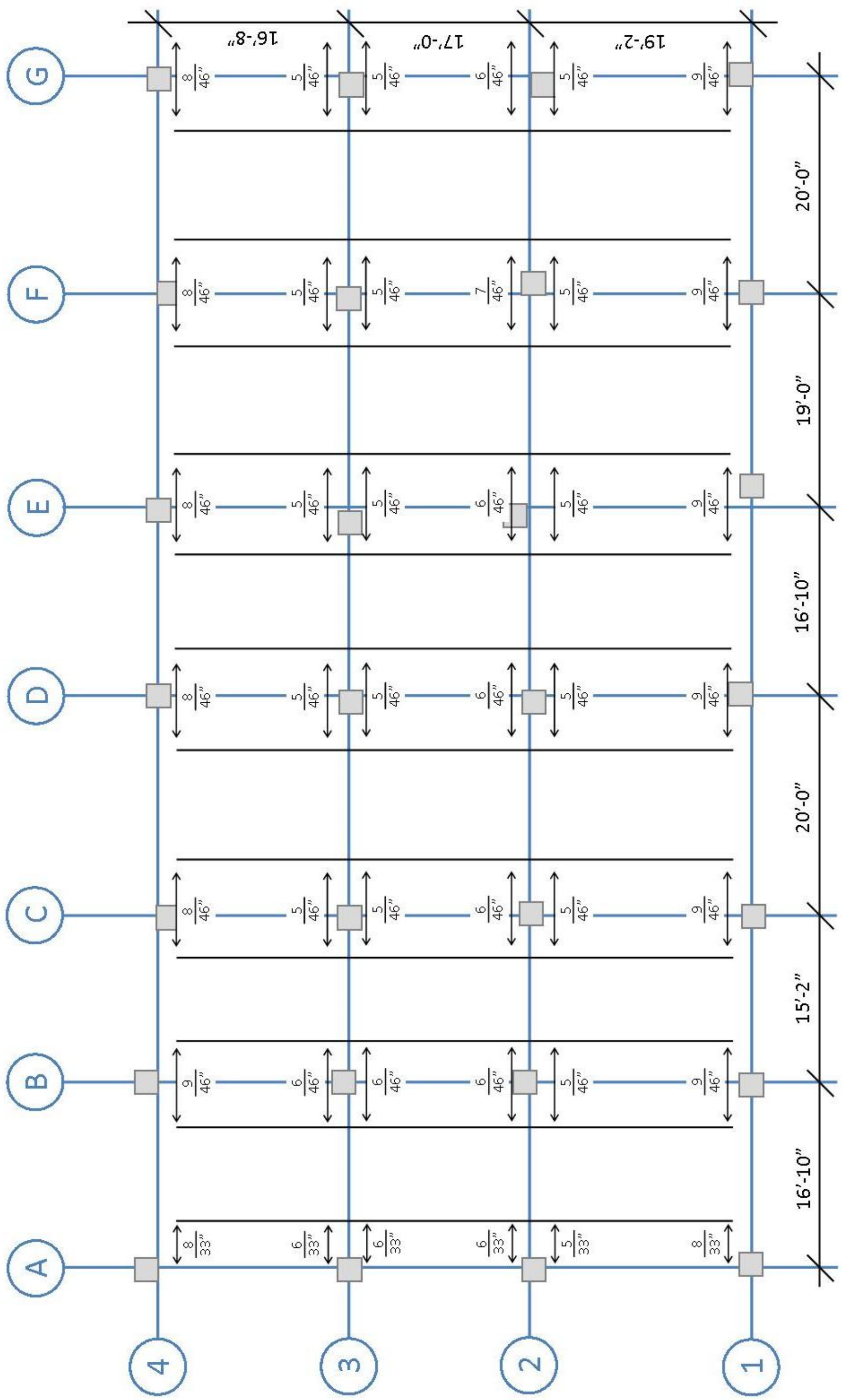
Third Floor



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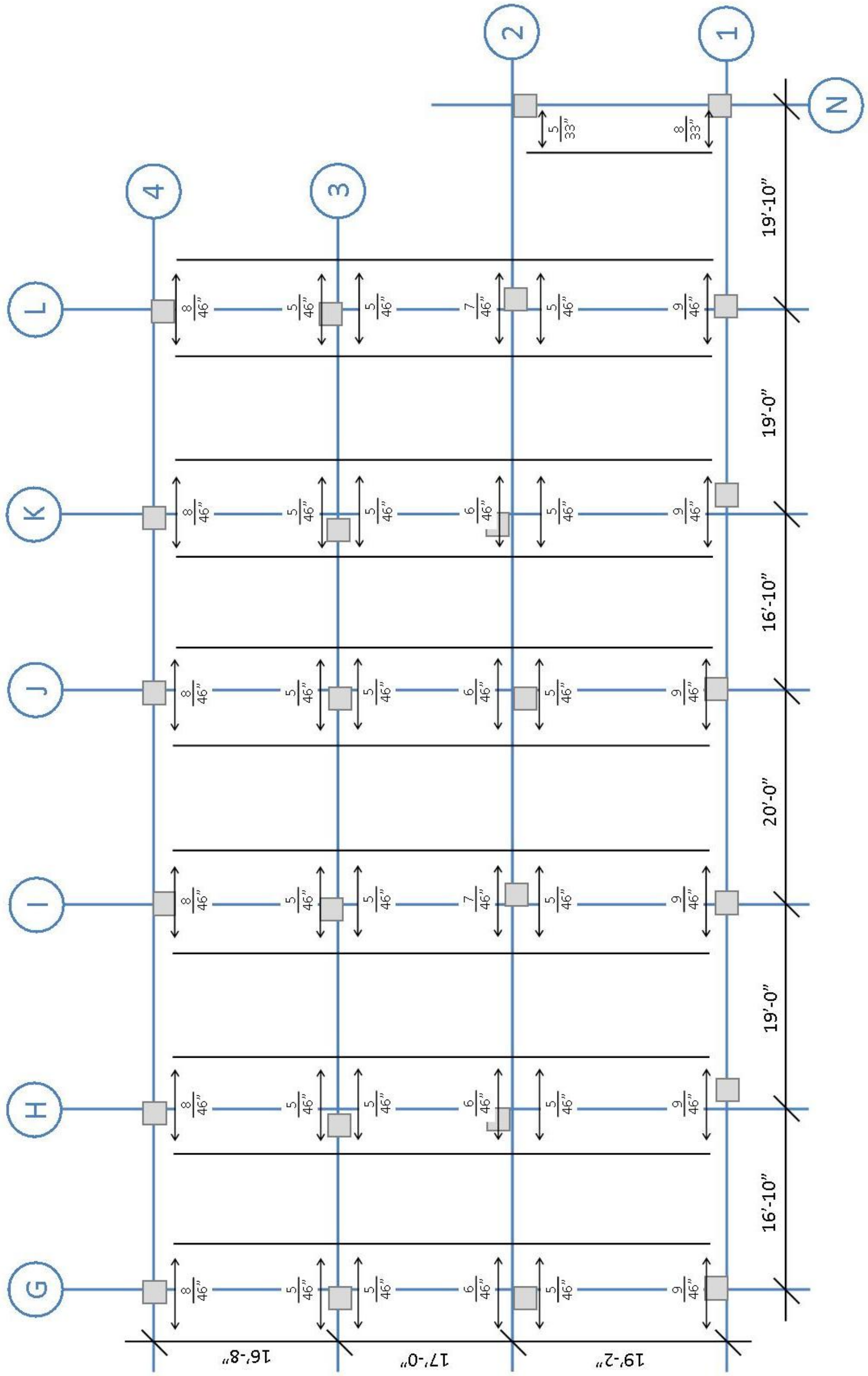
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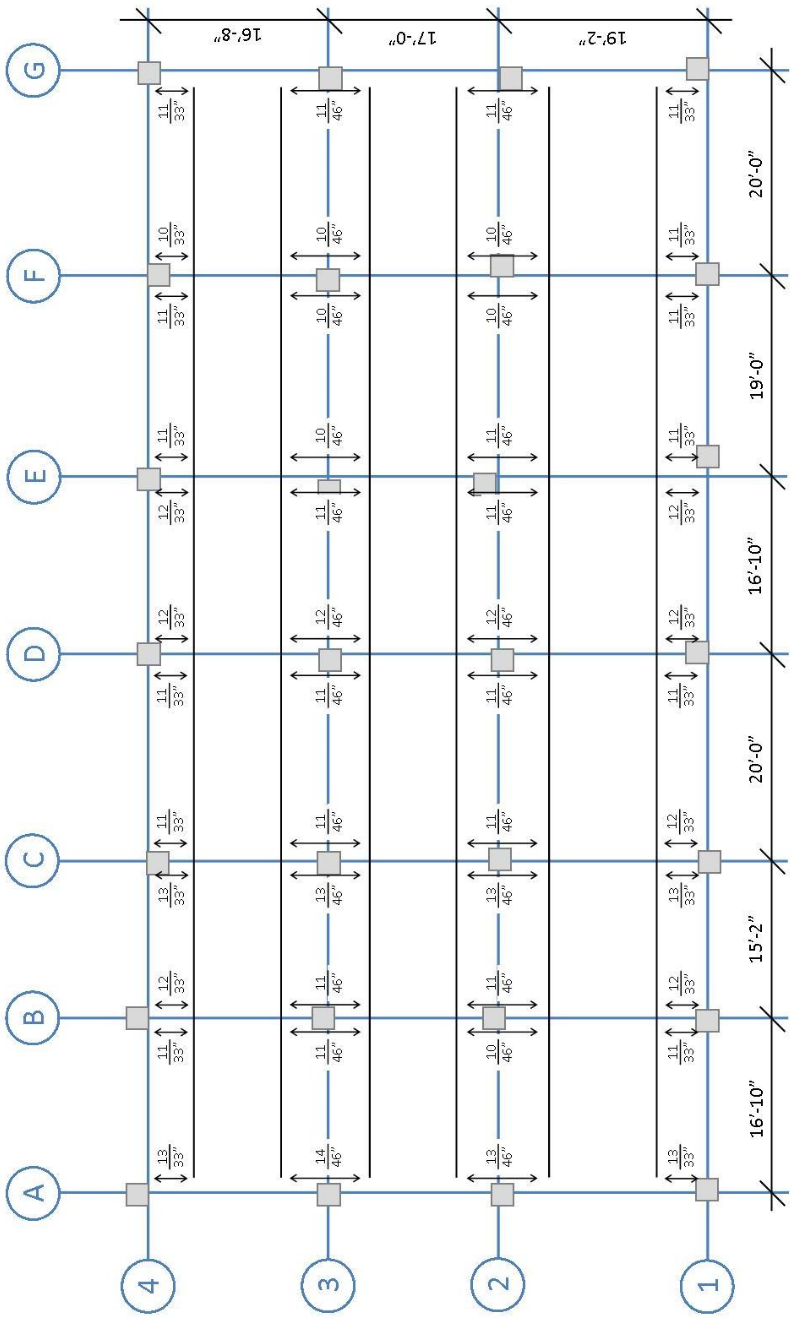
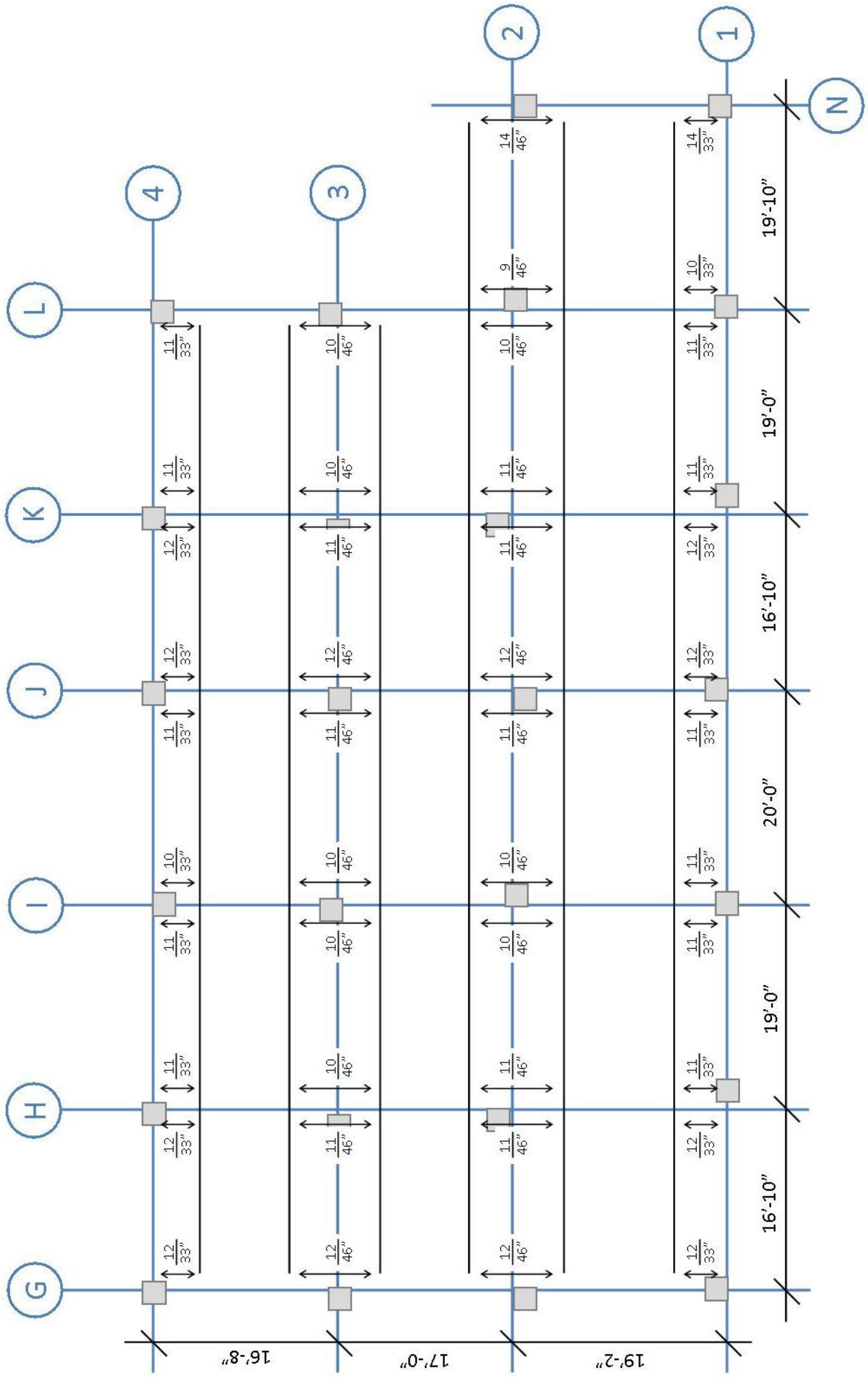
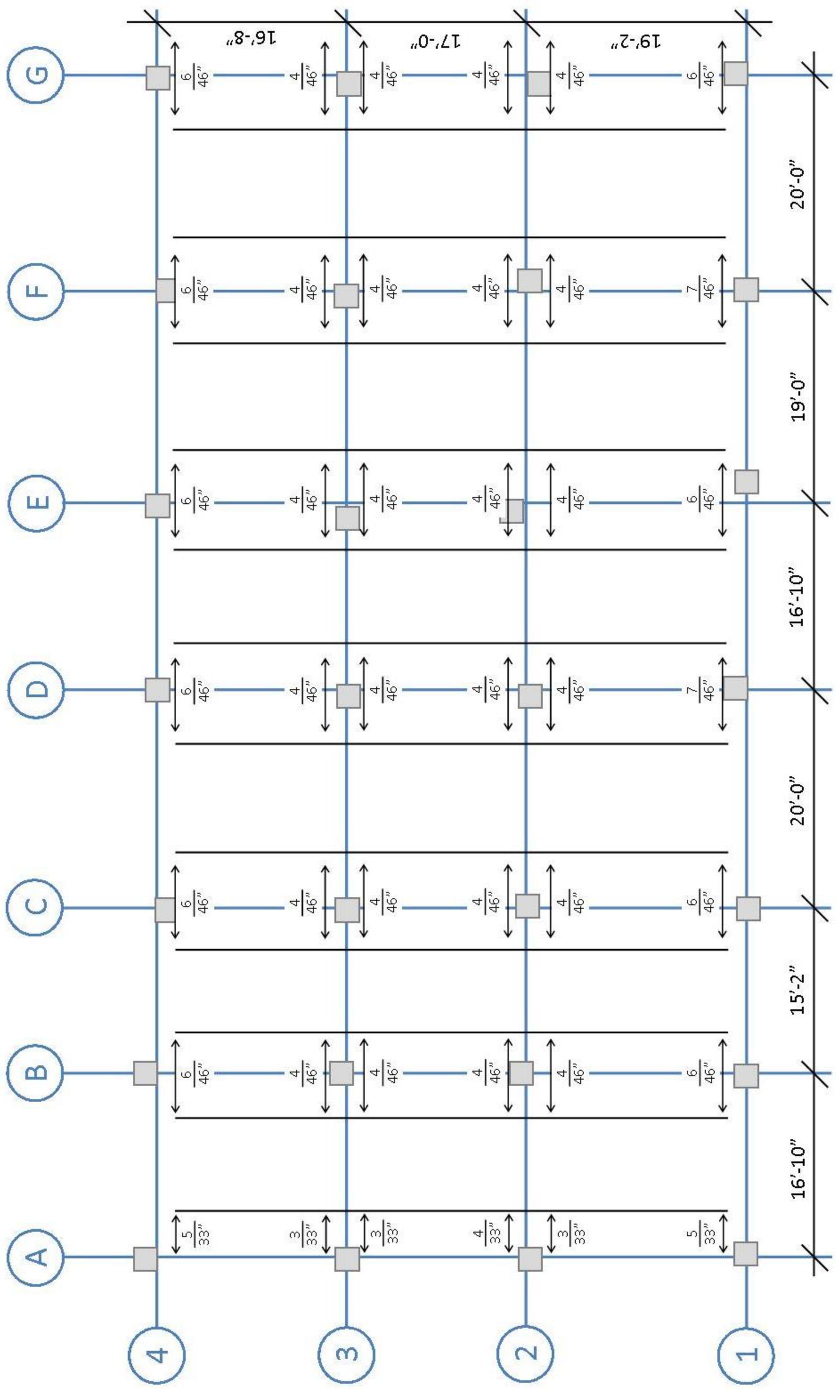
Fourth Floor



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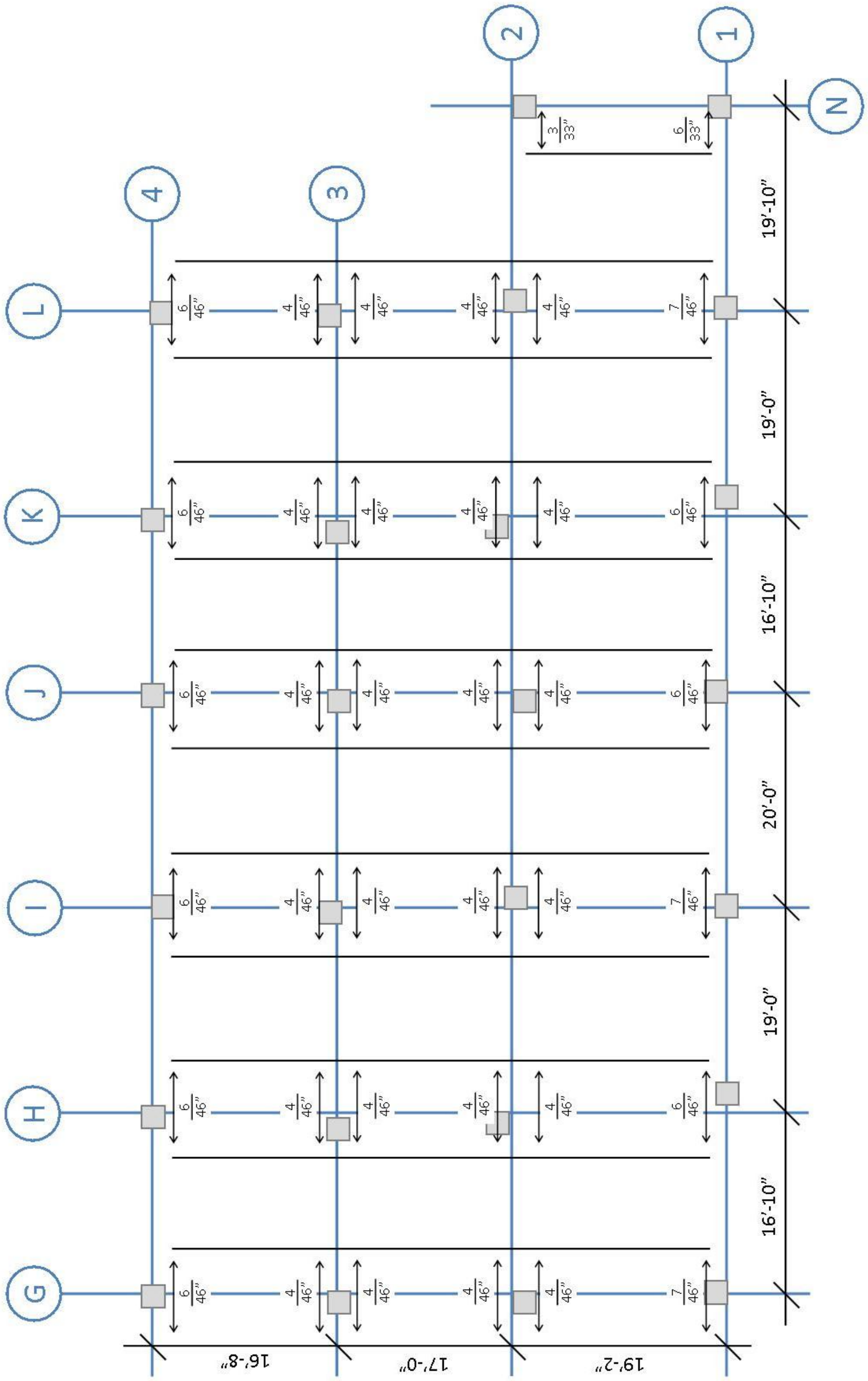
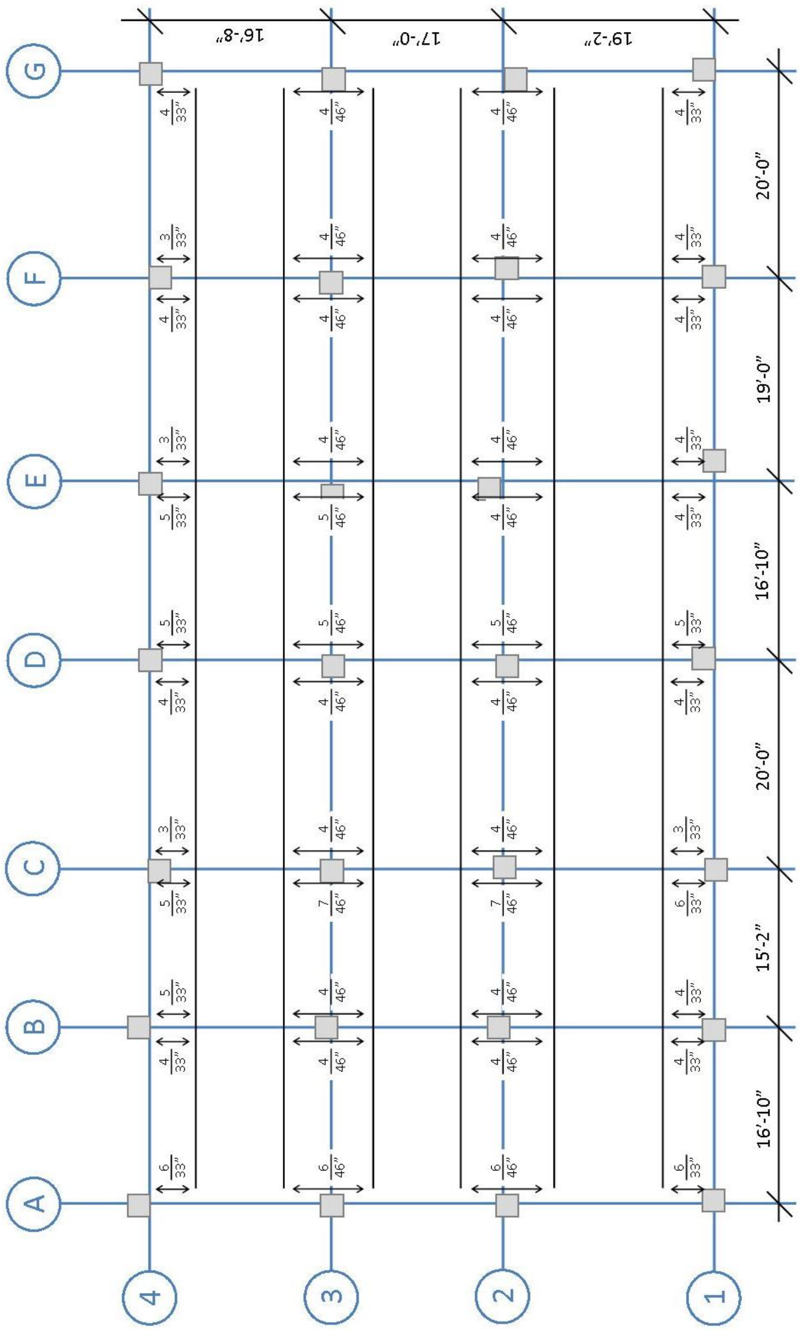
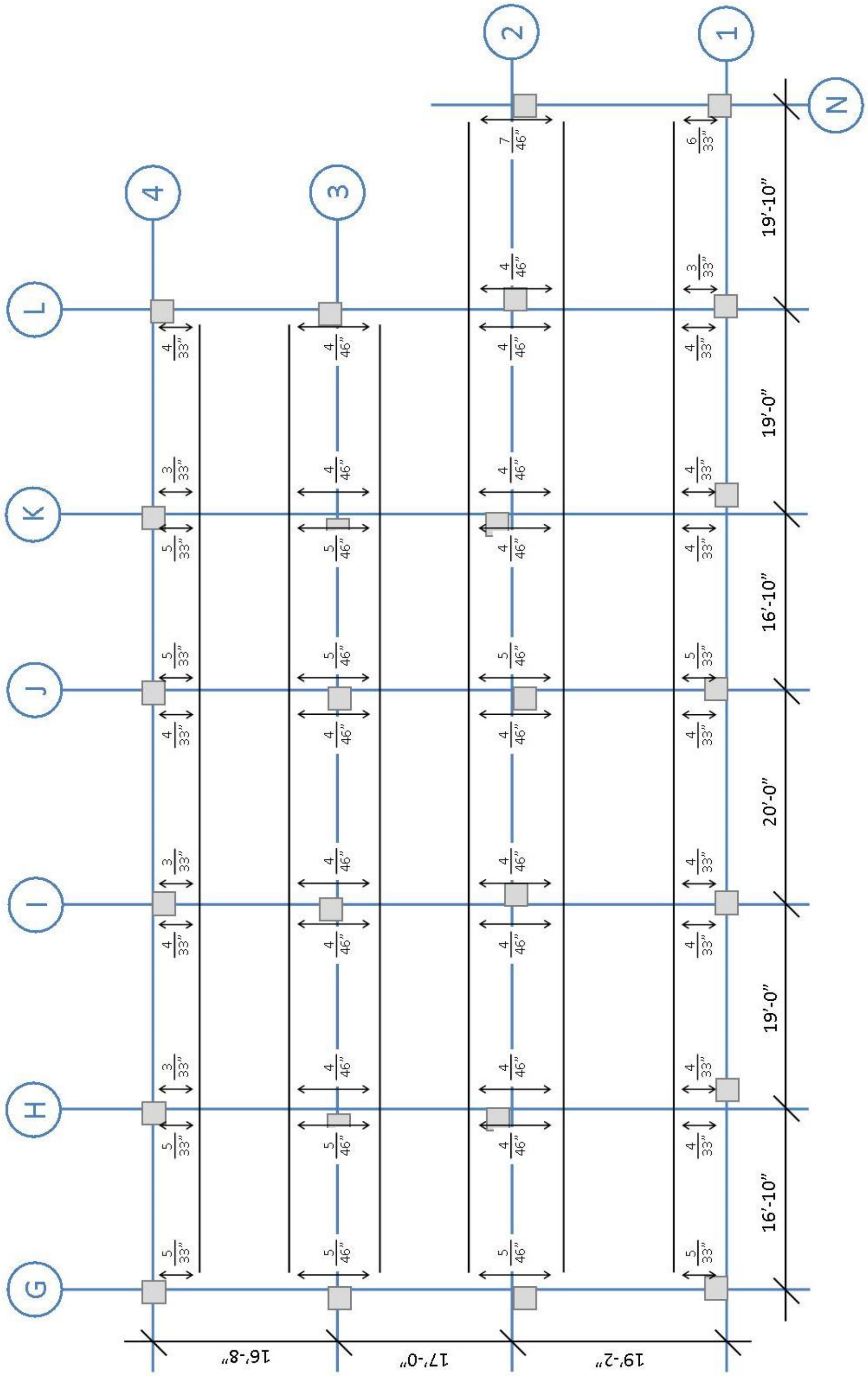


Mechanical Penthouse

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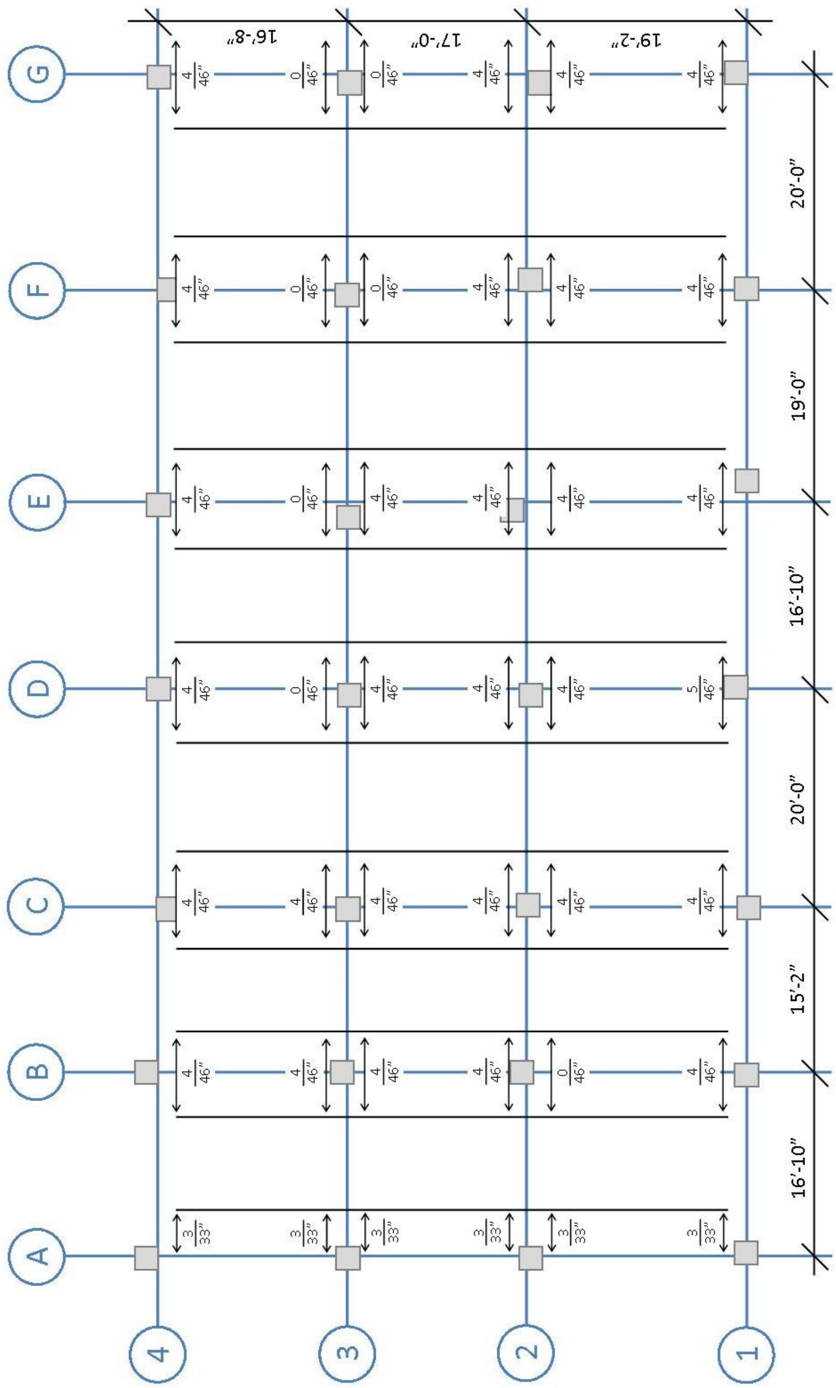
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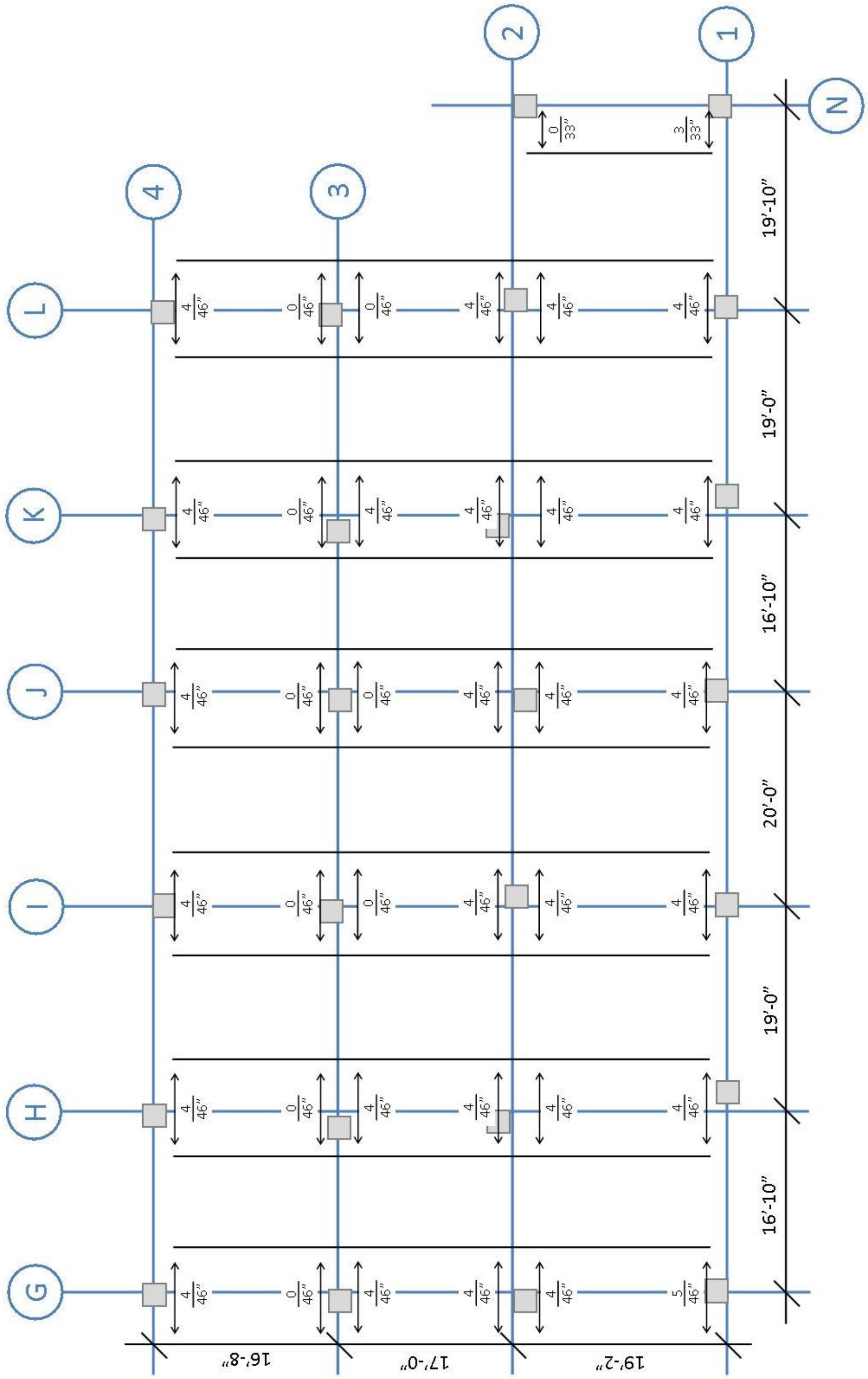

Appendix K: Cost Analysis

Existing Building - Steel and Wood Frame

Item	Size	Levels	Amount	Unit	Material	Labor	Equipment	Total	Over+Prof	Total	Total Ov+Pr
Steel Decking	18 Gauge	2nd, 3rd	23562	S.F.	1.80	0.40	0.05	2.25	2.80	53014.50	65973.60
Deck FireProofing	1" thick	2nd, 3rd	23562	S.F.	0.53	0.22	0.04	0.79	0.99	18613.98	23326.38
3.25" Slab Pumped	pumped	2nd, 3rd	236	C.Y.	-	16.20	5.70	21.90	31.00	5170.83	7319.44
4000 psi Concrete	3.25" Slab	2nd, 3rd	236	C.Y.	103.00	-	-	103.00	113.00	24319.42	26680.53
Concrete Finish	Bull Float	2nd, 3rd	23562	S.F.	-	0.35	-	0.35	0.57	8246.70	13430.34
Steel Beam (1)	W16x31	2nd	493	L.F.	37.50	2.84	1.79	42.13	48.45	20770.09	23885.60
Fire Proofing (1)	1" thick	2nd	1972	S.F.	0.53	0.43	0.09	1.05	1.39	2070.60	2741.08
Steel Beam (2)	W16x26	2nd	483	L.F.	31.50	2.55	1.61	35.66	40.50	17223.78	19561.50
Fire Proofing (2)	1" thick	2nd	1932	S.F.	0.53	0.43	0.09	1.05	1.39	2028.60	2685.48
Steel Beam (3)	W21x44	3rd	754	L.F.	53.00	3.47	1.65	58.12	66.50	43822.48	50141.00
Fire Proofing (3)	1" thick	3rd	3016	S.F.	0.53	0.43	0.09	1.05	1.39	3166.80	4192.24
Steel Beam (4)	W18x35	3rd	575	L.F.	42.50	3.85	1.83	48.18	55.00	27703.50	31625.00
Fire Proofing (4)	1" thick	3rd	2300	S.F.	0.53	0.43	0.09	1.05	1.39	2415.00	3197.00
Steel Girder (1)	W24x55	2nd, 3rd	856	L.F.	66.50	3.33	1.58	71.41	80.50	61126.96	68908.00
Fire Proofing (1)	1" thick	2nd, 3rd	6848	S.F.	0.53	0.43	0.09	1.05	1.39	7190.40	9518.72
Steel Column	W10x68	Up to 3rd	1791	L.F.	82.50	2.60	1.63	86.73	96.50	155333.43	172831.50
Fire Proofing	1" thick	Up to 3rd	8358	S.F.	1.13	0.93	0.19	2.25	2.98	18805.50	24906.84
Wood Framing	2x6, 10'	3rd, 4th, 5th	1790	L.F.	3.96	7.40	0.00	11.36	15.75	20334.40	28192.50
									Total:	$ 484,969.33	$ **571,588.23**

Location Factor:	0.987	
Floor Area:	11781	ft^2
Concrete Volume:	236	C.Y.

Proposed Building - Concrete Frame

Item	Size	Location	Amount	Unit	Material	Labor	Equipment	Total	Over+Prof	Total	Total Ov+Pr
4000 psi Concrete		All	1492	C.Y.	103.0	-	-	103.0	113.0	153632.2	168548.0
Concrete Finish	Bull Float	All	47124	S.F.	-	0.4	-	0.4	0.6	16493.4	26860.7
Concrete Slab	8.5"	All	1492	C.Y.	-	11.0	5.0	16.0	23.5	23790.6	35052.0
Slab Reinforcing		All	23	Ton	850.0	385.0	-	1235.0	1625.0	28405.0	37375.0
Slab Formwork	4 use	All	47124	SFCA	1.3	2.5	-	3.8	5.6	179071.2	263894.4
Column (Concrete)	20x20	Top 2	257	C.Y.	-	19.1	8.7	27.8	41.0	7118.8	10517.9
Column Reinforcing		All	43	Ton	1175.0	510.0	-	1685.0	2175.0	72455.0	93525.0
Column Formwork	4 use	All	199680	SFCA	0.6	3.2	-	3.8	6.1	765440.0	1214720.0
									Total:	$ 1,230,202.95	$ **1,826,436.50**

Location Factor:	0.987	
Floor Area	11781	ft^2
Building Area	47124	ft^2
Concrete Volume	1492	C.Y.