FINAL REPORT

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Structural Option

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The Primary Health Network's Medical Office Building
Sharon, PA

General Information

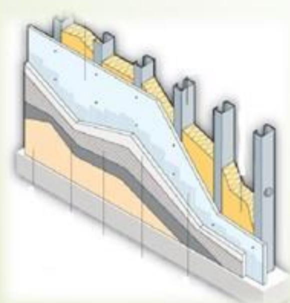
Height: 82ft
Size: 78,000 sq. ft.
Cost: $10 million
Construction: November 2014-January 2016
Project Delivery Method: Design-Build

Project Team

Owner: The Primary Health Network
Architect: John N Guitza Associates, Inc.
Structural Engineer: Taylor Structural Engineers
MEP Engineer: BDA Engineering
Construction Manager: Hudson Construction
Civil Engineer: Professional Service Industries, Inc.

Architecture

The primary architectural goal was to create a modern look with a strong focus on economy. This was accomplished by methods such as incorporating an exterior finish/insulation system (E.I.F.S. shown below).



Mechanical System

Variable Air Volume system comprised of (2) 65 ton units and (1) 30 ton unit

Lighting and Electrical Systems

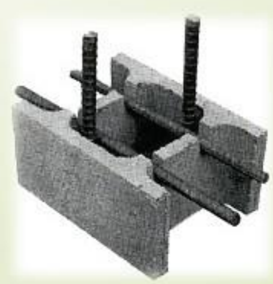
(5) 120/208V 3 Phase panel boards
(6) 480/277V 3 Phase panel boards

Low voltage dual technology occupancy sensors are used to increase efficiency

Structural System

Foundation: Concrete spread and Mat footings
Gravity: Steel columns and wide flange girders, steel bar joists, and normal weight concrete on metal deck floors

Lateral: 3 Ivany block shear walls
(Ivany Block Pictured below)



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Executive Summary

A New Medical Office Building for The Primary Health Network in Sharon, Pa will serve to help revitalize a community that hasn't seen new construction in 46 years. The 78,000 sq. ft. building will be located between Pitt and E Silver streets near the Shenango River. Construction began in November 2014 and is expected to be completed by January of 2016.

The following report contains an overview of the building site, size, architecture and structure in the first portion. An alternate solution to the structural framing of the building is offered and then explored in detail. A two way flat slab with drop panels and edge beams was designed for strength and serviceability requirements using spSlab and verified with hand calculations. These slabs are supported by concrete columns modeled in spColumn and verified with hand calculations.

The existing lateral system consists of Ivany Block masonry shear walls which were redesigned as concrete shear walls. The lateral system was modeled using ETABS 2013. The redesign focused heavily on keeping the original column layout with marked exceptions. The change to a concrete system resulted in drastically increased lateral loads due to seismic forces, these loads were calculated by ETABS and verified by hand.

Sharon, Pa hasn't had a commercial construction project since 1969. This gap in construction results in an even more pronounced gap in architecture. The new medical office building has to be modern enough to breathe new life into the city while acknowledging the surrounding buildings in order to mesh well with the community. The building's façade was redesigned in order to better accomplish these goals. The building and site were modeled using Revit 2015.

The Primary Health Network had a very tight budget for this project; efficiency played a leading role in all aspects of design. The change in building structure as well as the change in building façade result in an equivalent change in building cost which must be accounted for to determine the feasibility of the redesign. A cost comparison of the existing structural system to the structural redesign was completed using RS Means Facility Cost Data 2015. A Similar cost comparison was made between the existing and redesigned building facades.

The change in building material will also affect the building construction period. A building construction schedule was created for the redesigned structural system only using Microsoft Project by referencing the information found in RS Means Facility Cost Data 2015.

The redesign was found to reduce the overall structural depth while meeting all strength and serviceability requirements. The redesign increased the overall building cost primarily due to the redesign of the building façade.

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Acknowledgements

I would like to extend a special thank you to the following individuals:

From Taylor Structural Engineers

- Brian Hermiller for his quick, pertinent responses to my myriad of questions and inquiries for information

- Dirk Taylor for giving me the option of several buildings as well as his advice on which to choose

From the Penn State AE Faculty

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- Thomas Boothby for his light hearted attitude and assistance during the second half of my senior thesis

- The remaining AE Faculty for their willingness to answer my occasional questions

My Family and Friends for providing the occasional and necessary distractions

Building Introduction

Figure 1 – Elevation
Image courtesy of Taylor Structural Engineers

The Primary Health Network's Medical Office Building, as shown in Figure 1, will be located between Pitt and E Silver streets near the Shenango River in Sharon, Pa as denoted in red on Figure 2. The building will be 5 stories above grade, four elevated floors and a roof comprising a total building height of 85 feet. The tentative construction period is November 2014-August 2016, the demolition of existing structures on the site is included in this timeframe. The approximate building cost of $10 million will provide 78,000 square feet of occupant space. The building façade is an exterior insulation finishing system in combination with a glazing system. The E.I.F.S. was chosen for its economic efficiency while the glazing serves the purpose of giving the building modern aesthetics.

Figure 2 – Site Map

Image courtesy of Taylor Structural Engineers

Structural System Overview

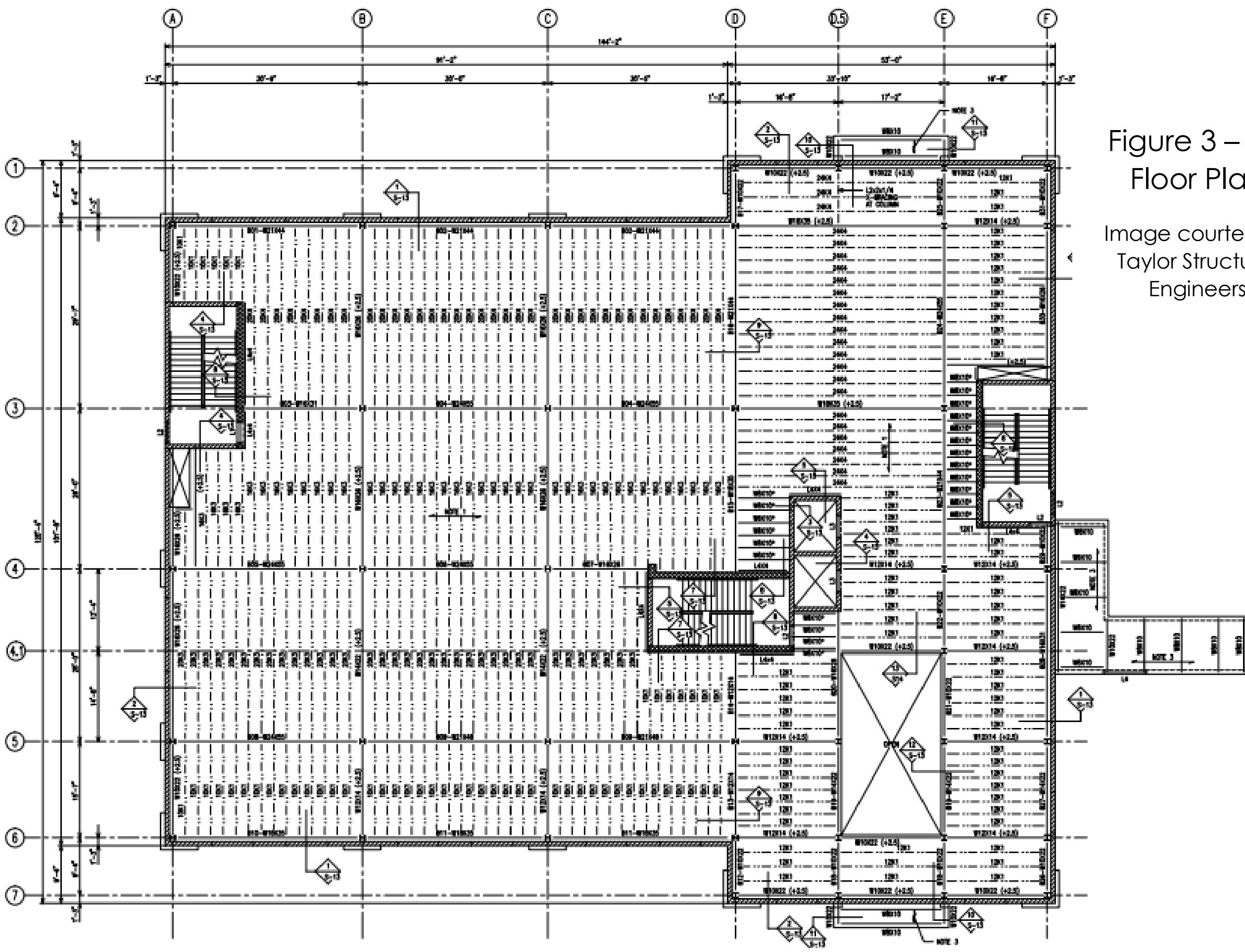


Figure 3 – 2nd Floor Plan

Image courtesy of Taylor Structural Engineers

The Primary Health Network's Medical Office Building in Sharon, Pa is primarily a steel framed structure. Steel columns and rolled steel girders comprise the gravity support system as seen in Figure 3 above. The four elevated floors consist of concrete on metal deck supported by steel bar joists. The roof structure is comprised of an adhered membrane on rigid insulation supported by metal deck. Fully grouted Ivany block masonry walls encasing the three main stairs comprise the lateral force resisting system for the building. The building first floor is supported by a reinforced concrete slab-on-grade while the remaining building load is transferred through the columns to reinforced concrete footings.

Design Codes and Standards

Below is a list of all applicable building codes and standards used in design.

- International Building Code 2009
  - NOTE: IBC 2012 selected for wind load calculations
- American National Standards Institute 2006
- American Society of Civil Engineers 7-05
  - ASCE 7-10 for wind calculations
- American Concrete Institute 318-08
- American Institute of Steel Construction
  - Structural Steel Buildings 2005

Materials

The following tables give the material properties of all major structural components used in the building design.

Table 1.1 – Steel Properties

Shape	ASTM	Grade	Fy(ksi)
Beams and Girders	A992	50	50
Plates and Bars	A36	-	36
Steel HSS	A500	B	46
Pipe	A53	B	30
Columns	A992	50	50
Bolts	A325	-	-

Table 1.2 – Concrete Properties

	Minimum Strength(ksi)	Weight (pcf)	Max Water/Cement Ratio
Mat Footings	4	144	0.50
All Other Foundation	3	144	0.50
Interior Slabs	4	144	0.45
Exterior Slabs	4	144	0.40

Table 1.3 – Masonry Properties

	Minimum Strength(ksi)	ASTM
Hollow Units	1.5	C90
Solid Units	1.5	C90
Ivany Block	3	-
Standard Mortar Above Grade	3	C270 Type S
Standard Mortar Below Grade	3	C270 Type M
Mortar for Ivany Block	3	C270 Type M

Typical Bay

A typical bay in this building is roughly 30'x30' with the joists spanning north to south on the western half of the building and east to west on the eastern half. A typical bay is shown in Figure 7 below, a typical floor plan can be seen in Figure 8 on the next page. Steel columns support the floor and roof structures. Figure 6 – Details the typical spliced connection at the third floor level where column sizes are reduced. All columns are W10's with weights ranging from 33 to 60 plf. At the third floor level the columns are spliced with the majority being decreased to W10x33's.

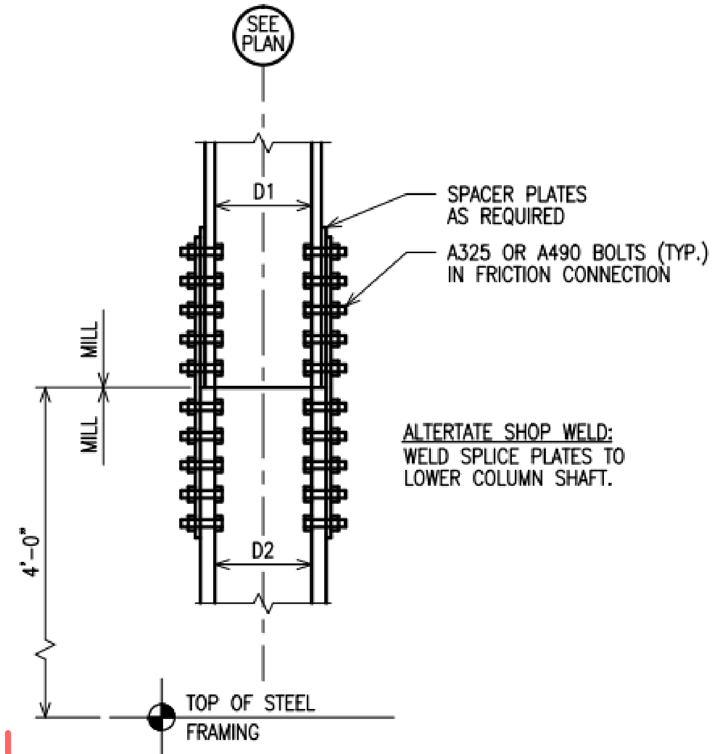


Figure 6 – Typical Column Splice

S-10 Typical Bolted Column Splice

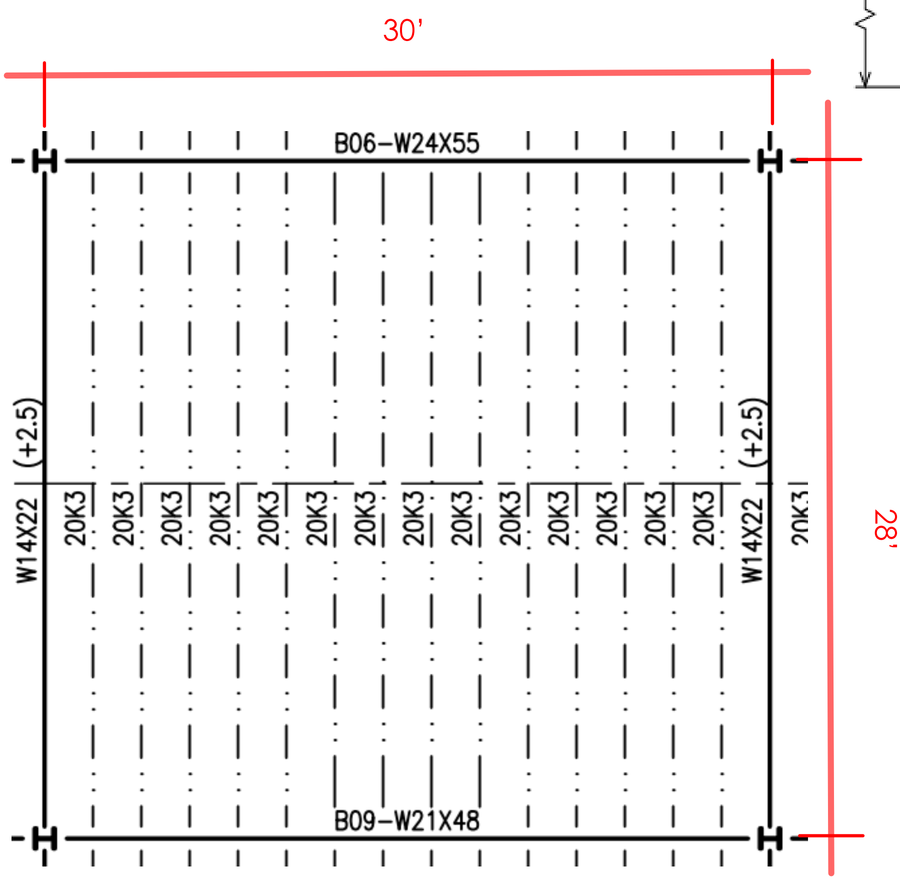


Figure 7 – Typical Bay

S-2 Second Floor Plan

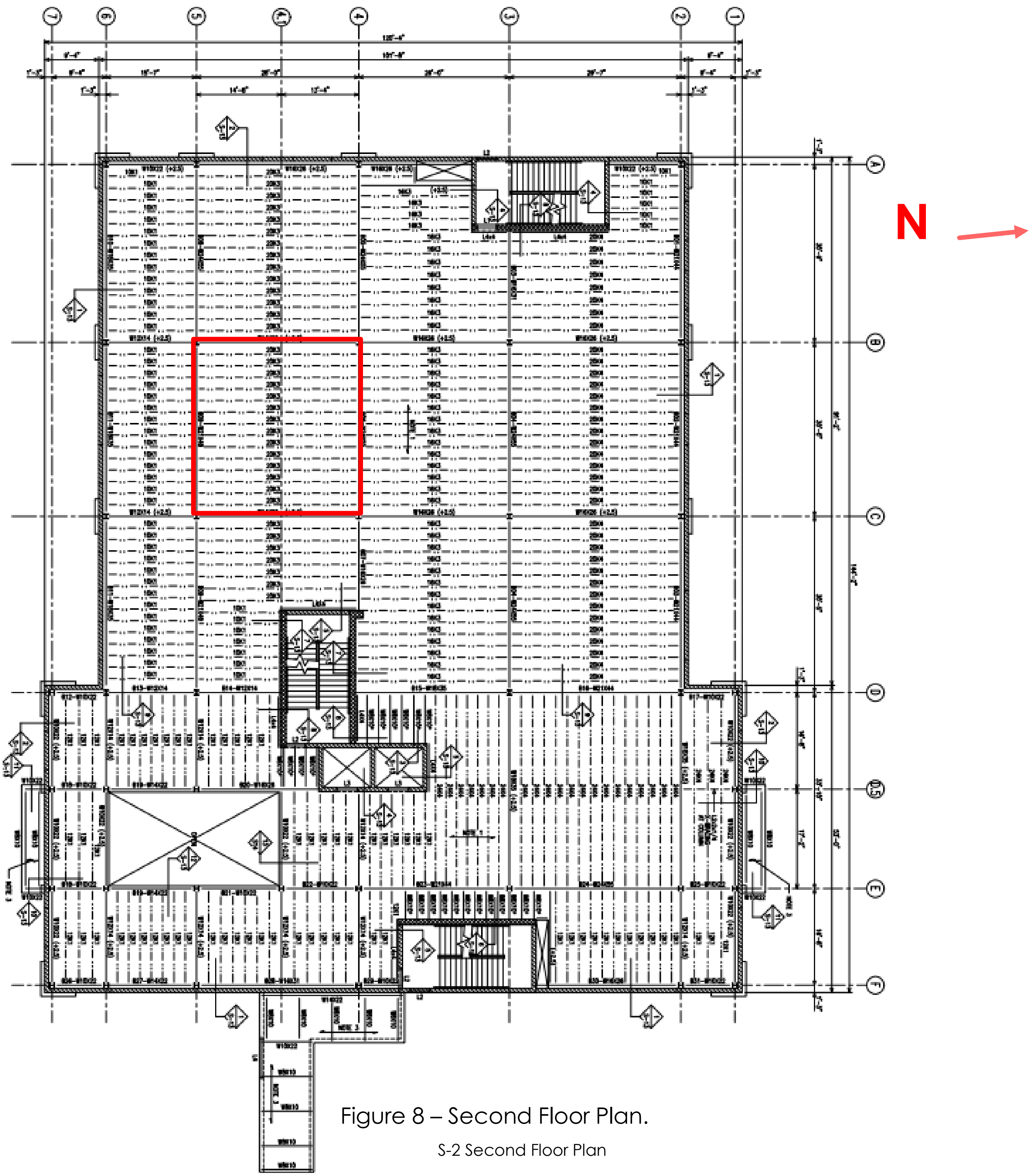


Figure 8 – Second Floor Plan.

S-2 Second Floor Plan

Floor System

The Medical Office Building's floor system consists of normal weight concrete on 19/32" 26 gage galvanized form deck. K series steel bar joists of various sizes ranging from 10 inch to 24 inch depth support the floor deck. These joist are in turn supported by wide flange sections with similar variances in depth. In areas where joist span direction changes HSS sections are used to maintain deck elevation consistent with joist seat height as noted in Figure 9 below.

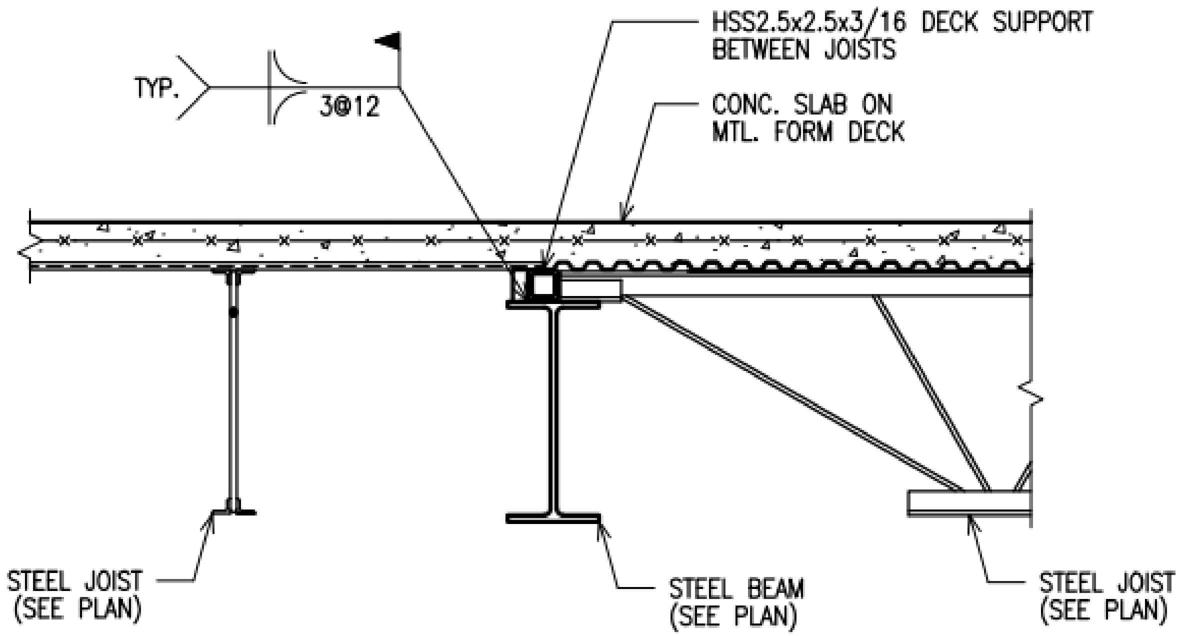


Figure 9 – Typical Framing Detail
S-13 Section 9

Building Lateral System

The main lateral force resisting system in the Primary Health Networks Medical Office Building is Ivany block shear walls. Ivany block is a concrete masonry unit which, when fully grouted, provides similar performance as an f'c=3ksi cast in place concrete shear wall system with significant cost savings. Ivany block gains another advantage over typical CMU blocks in the placement of reinforcement; Ivany block has slots for rebar allowing for a consistent "d" value to be used in flexural calculations, as shown in Figure 10. Ivany block shear walls partially encase the three stair towers as shown in red on Figure 11 below.

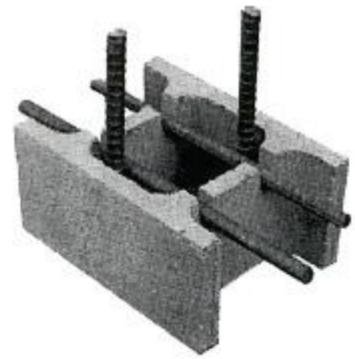


Figure 10 – (Source: koltcz.com)
Ivany Block with Reinforcing

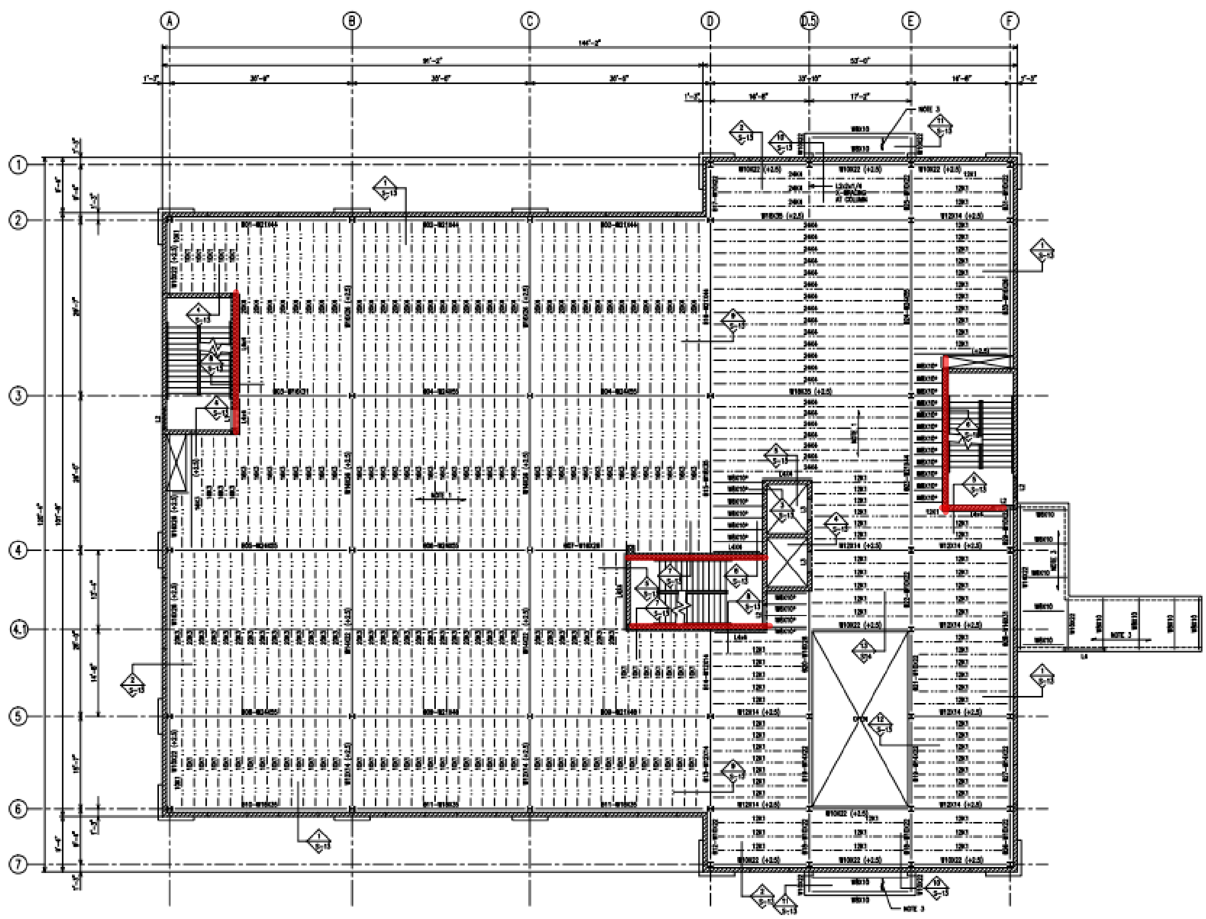


Figure 11
S-2 Second Floor Plan

Shear wall considerations

Lateral loads enter the building through the façade and transfer through girders and tie-beams to ultimately be taken by the Ivany Shear walls. These shear walls which rest on mat footings extend vertically to the roof level. The shear wall located on the western side of the building has openings in the wall at each floor level, this restricted the flexural capacity of the wall by decreasing its depth by 4 feet. The vertical and horizontal bars are #4 spaced at 16" on center. The flexural reinforcing consists of twelve #6 bars spaced at 8" on center up to the third floor where a 28" overlap splices into twelve #5 bars at the same spacing.

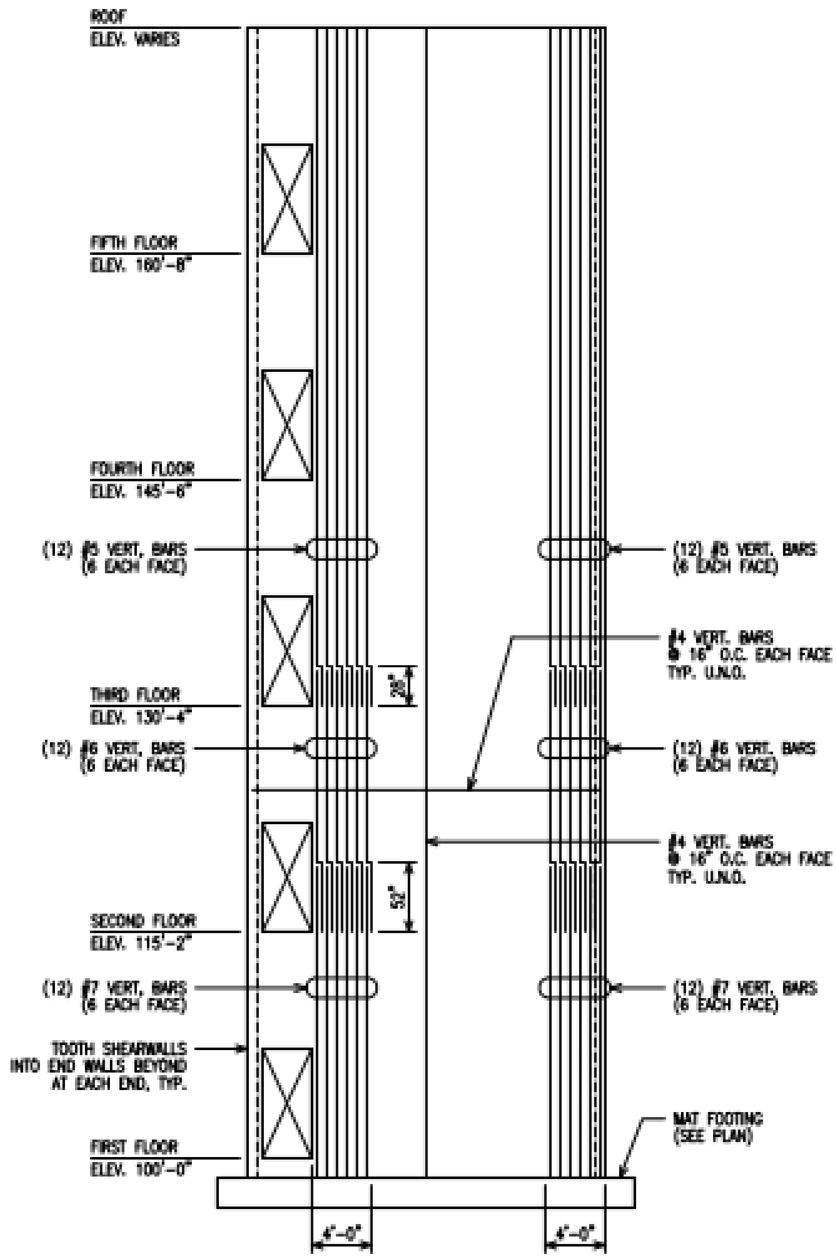


Figure 12
S-11 Shearwall notes

Foundation Design

Greenleaf development services conducted a site survey. Their geotechnical report showed that the soil had a bearing capacity of 2500 psf. This was the basis for the design of the buildings footings. The overall design ideology for the foundation was to keep a shallow profile of individual and spread footings resting on the soil.

All interior columns rest on individual concrete spread footings, a section of which is shown in Figure 13. Exterior columns rest on a continuous concrete wall footing. The ivany block walls sit on mat footings as can be seen in Figure 14.

Figure 13 – Spread Footing
S-12 Section 2

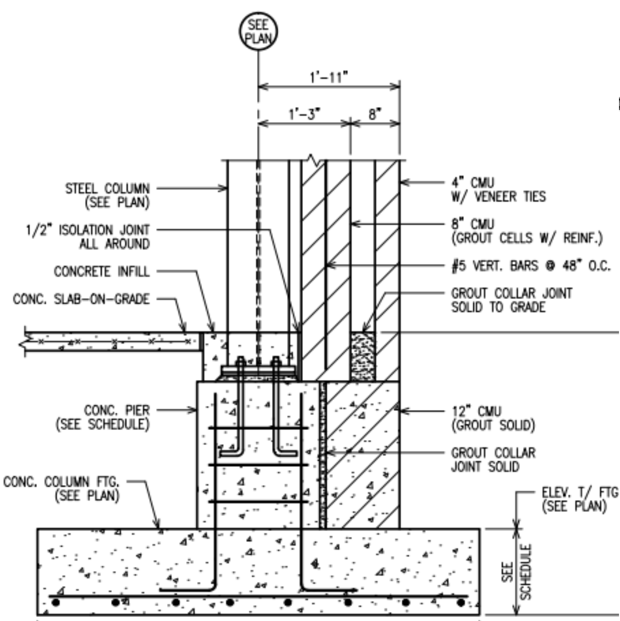
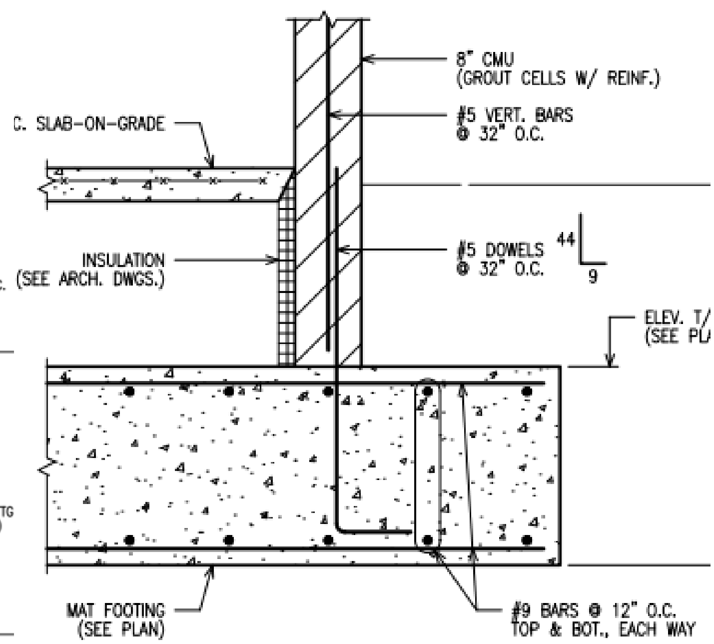


Figure 14 – Mat Footing
S-12 Section 4



Load Path

This section discusses the manner in which forces are transferred and distributed through the building structure ultimately leading to their dissipation.

Gravity

Gravity Loads in The Primary Health Networks Medical Office Building are received by the concrete floor deck which transfers the load to the steel bar joists. The bar joists transfer the load into the wide flange steel girders which bring the load to steel columns. From there the load is transferred down into spread footings which ultimately dissipate the force into the soil.

Lateral Loads

Wind forces are received by the building façade and then transferred into exterior girders. The lateral loading continues through the floor diaphragm, comprised of concrete on metal deck, to the Ivany block shear walls. These shear walls transfer the energy into the foundations and ultimately the soil.

Design Loads

In the design of The Primary Health Network's Medical Office Building two different codes were used to determine design loads. All gravity loads were determined using ASCE 7-05, whereas the lateral forces were determined using ASCE 7-10.

Dead Loads

The floor dead load was taken as 50 psf to account for the concrete deck, steel joists and girders, MEP and a false ceiling. 20 psf was used as the roof dead load, the reduction due to an adhered membrane being used instead of concrete on the roof deck.

Live Loads

All of the floors were designed for a 100psf live load typically used for lobbies or first floor corridors instead of the typical office live load listed in ASCE 7-05. This allows for flexibility in future changes to the floor layout. A roof live load of 35 psf controlled over the ground snow load rating of 25 psf. This design choice was likely made to account for additional mechanical equipment as well as snow drift where the roof level changes.

Lateral Loads

Wind loads were calculated using ASCE 7-10 with a building category II, exposure B and a 115mph base wind speed. The building was designed using seismic design category A, site class B and use group 1.

Joint Details

In the Medical Office Building typical connections include joist to girder, girder to column, joist to block wall and deck to block wall. The first of these two connection types are to be detailed by the steel fabricator, as such this section will focus on the remaining two.

Typical joist to block wall connection

Steel bar joists and steel girders transfer loads into the Ivany block walls via ½" Plates with two ½" dia. By 6" headed studs. Figure 15 below shows a joist seat sitting on the plate supporting the joist floor system. The concrete deck is flush to the wall with a ½" isolation joint.

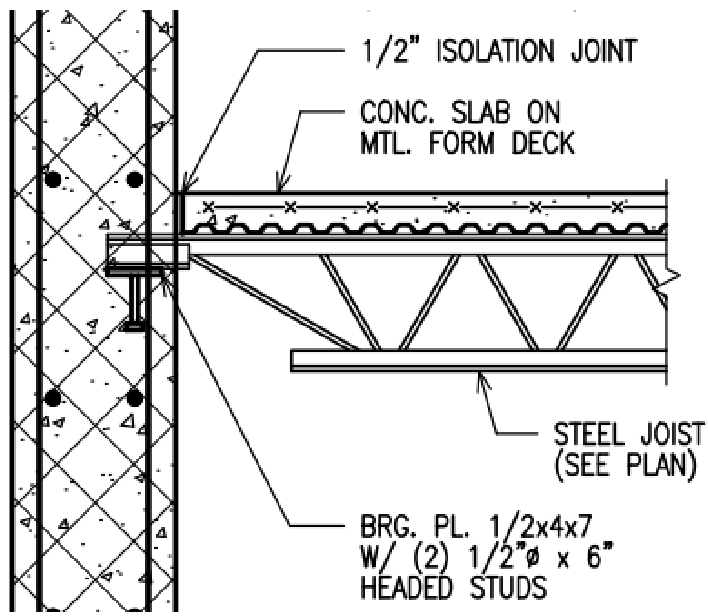


Figure 15 – Typical Joist to block wall connection
S-13 Section 4

Typical concrete deck to block wall connection

Where the concrete on metal deck meets the masonry block walls in an unsupported condition a 4"x4"x1/4" steel angle is fastened to the block wall in order to support the deck via ¾" dia. hilti sleeve anchors spaced at 16" on center. This type of fastener has a casing that expands as the connection is tightened. This is shown in Figure 16 below.

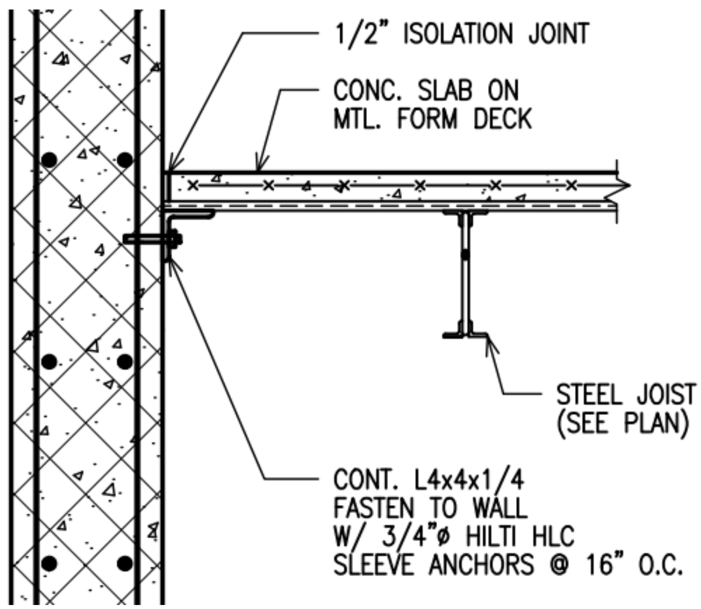


Figure 16
S-13 Section 8

Lateral Loads

Modeling Process

Ram Structural Systems was initially chosen as the modeling program due to familiarity. After modeling part of the gravity system it was determined that model was becoming unnecessarily over-complicated. To simplify the model only the lateral system was modeling using ETABS 2013. The four masonry shear walls were modeled using the properties associated with 3000psi f'm, 2700ksi E masonry. The cracked section modifier for concrete of 0.7 from ASCE7-10 was used for the full height of all walls. The fully grouted masonry walls will exhibit similar performance to 3000psi f'c concrete and therefore the concrete section of ASCE7-10 can be used. The walls were to the rigid floor diaphragm that was created at each level. The weight of the floor structure was included in all previously calculations and therefore the floor diaphragms were modeled as having no mass. The walls were modeled as fully fixed at the base level.

The wind loads were taken from technical report II and applied at the center of each diaphragm in its respective direction as a point load assuming the rigid diaphragm will distribute the load based on stiffness. The corrected seismic loads from technical report II were applied to the buildings center of mass as point loads at each floor level.

Center of Rigidity

Story	Diaphragm	Mass X lb-s²/ft	Mass Y lb-s²/ft	XCM ft	YCM ft	Cumulative X lb-s²/ft	Cumulative Y lb-s²/ft	XCCM ft	YCCM ft	XCR ft	YCR ft
Roof	D1	41.54	41.54	90.6919	64.2778	41.54	41.54	90.6919	64.2778	89.4399	45.5115
Story4	D1	166.16	166.16	82.6111	58.8232	207.7	207.7	84.2273	59.9141	88.9321	45.5194
Story3	D1	166.16	166.16	82.6111	58.8232	373.86	373.86	83.509	59.4293	87.9765	45.539
Story2	D1	166.16	166.16	82.6111	58.8232	540.02	540.02	83.2327	59.2428	85.9811	45.5744

The center of rigidity for the structure is highlighted in red above. The center of rigidity in the x direction moves to the right by a total distance of 3.46' over the height of the structure, this is a 2.4% difference and can be considered negligible. The reason for the change in XCR is due to the differing lengths of shear wall effective in this direction. Shear wall 1 (as seen in plan above) is only 19' in length whereas shear wall 4 is 24' in length. Rigidity is a factor of displacement, which is based heavily on wall length.

The center of rigidity was calculated by hand at the roof level in order to consider ultimate displacements and to consider the highest value eccentricity. The calculated value for the center of rigidity was found to be XCR=89.8ft and YCR=45.5ft. This gives an error value of 0.4% in the x direction, and a value of 0.03% error in the y direction. Supporting hand calculations can be found in Appendix A.

Wind Loads

Wind loads on the building were calculated in accordance with ASCE 7-10 chapter 27 for Main Wind-Force Resisting Systems using the Directional Procedure. This method was deemed most viable due to the buildings regular geometry and low overall height. The controlling wind direction was case 1 per ASCE 7-10 chapter 27.4-8. This method gave resulting wind pressures as shown in figure 15, with a maximum base shear occurring in the building North-South direction with a value of 304 kips. The overall building dimensions were simplified for the procedure to the dimensions shown in Figure 17 below. All hand calculations are included in Appendix A.

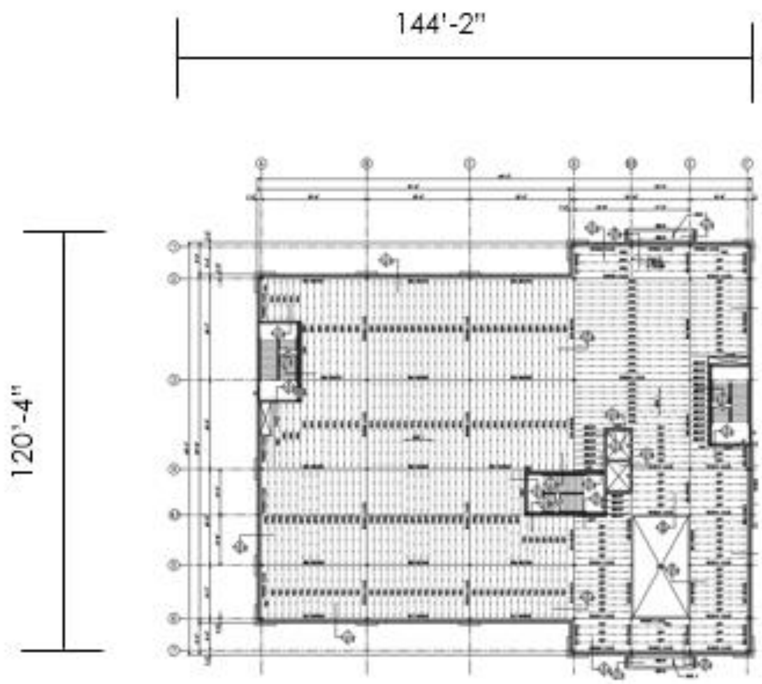
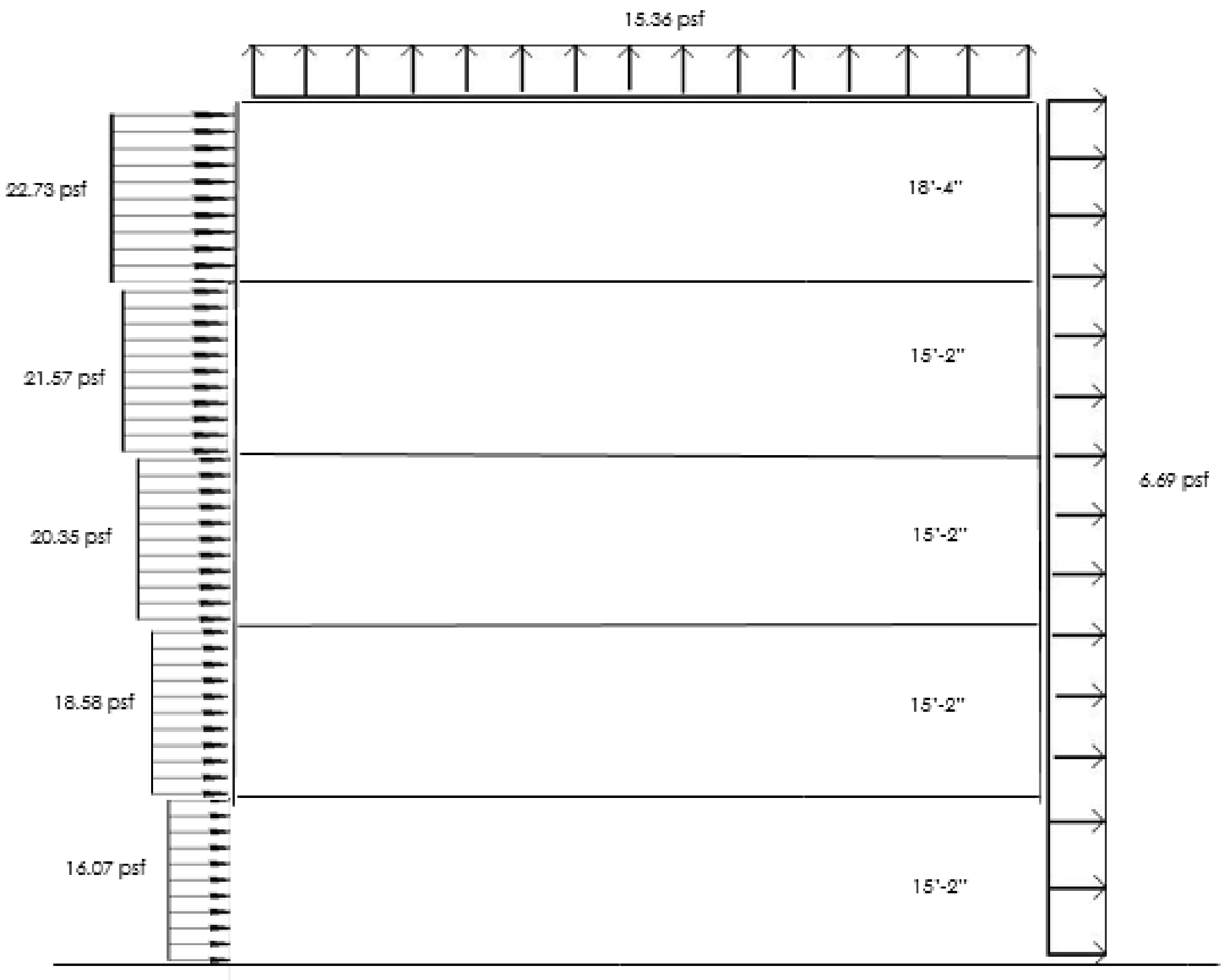


Figure 17
Simplified building Dimensions

Figure 18 - Wind Loading Diagram



Base Shear N-S direction

V=22.75psf(15.167'*144.167')+25.27 psf (15.167'*144.167')+27.04 psf (15.167'*144.167')+28.26 psf (15.167'*144.167')+29.42 psf (18.33'*144.167')

V=304kips

Base Shear E-W direction

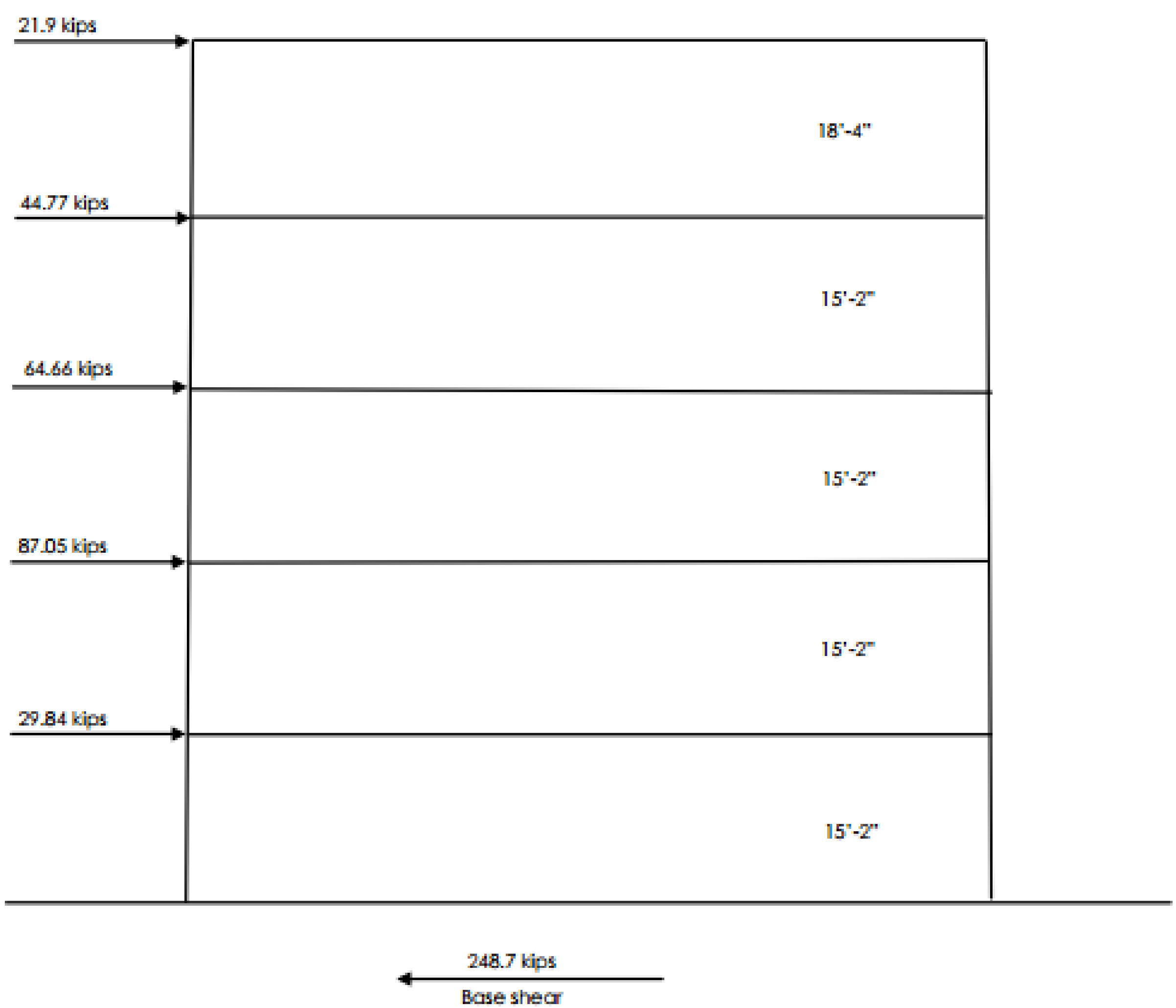
V=22.75psf(15.167'*120.33')+25.27 psf (15.167'*120.33')+27.04 psf (15.167'*120.33')+28.26 psf (15.167'*120.33')+29.42 psf (18.33'*120.33')

V=254kips

Seismic Loads

Seismic calculations were determined in accordance with ASCE 7-10 Chapters 11 and 12, using the Equivalent Lateral Force Procedure. The building weight, W, was estimated by hand in order to determine the building base shear. A full set of hand calculations can be found in Appendix A. The seismic story and base shears as calculated can be found on Figure 19 below.

Seismic Loading Diagram - Figure 19



Comparison of Lateral Forces

In order to establish whether wind or seismic forces would control the lateral design, the resulting shears and overturning moments were compared. For the Primary Health Networks Medical Office Building wind loading in the North-South direction controlled both in base shear and overturning moment. Supporting hand calculations can be found in Appendix A.

Story	Wind in the X	Wind in the Y	Seismic
Roof	64.33kip	53.69kip	29.84kip
Four	61.79kip	51.58kip	87.05kip
Three	59.13kip	49.35kip	64.66kip
Two	55.25kip	46.12kip	44.77kip
One	49.74kip	41.52kip	21.90kip

Overturning Moment (ft-k)	Wind in the X	Wind in the Y	Seismic
	10,877	9,079	9,634

Problem Statement and Proposed Alternate Solution

Structural Depth

The Primary Health Network's medical office building in Sharon, Pa meets all applicable code standards for strength and serviceability per technical reports I-IV. The current steel framing design consists of wide flange columns and girders supporting a concrete on metal deck with steel bar joist floor system. This system was requested by the building architect. However, alternative framing systems explored in technical report III provided the potential for a more efficient design. The building will be redesigned to demonstrate to the architect the value of an alternate design. The most promising of the three previously explored alternate systems was a two-way flat plate. The average bay size of roughly 30'x30' lends itself perfectly to two-way concrete design. Technical report III concluded that a two-way concrete system would have a shallower structural depth, cost less per square foot, and provide a greater overall fire rating. The main disadvantage of the two-way flat plate slab investigated in technical report III was the increased column size, this can be greatly reduced by the incorporation of drop panels. The floor and roof systems will be redesigned as two-way flat slabs with drop panels. The redesign will tentatively utilize all existing column locations to help maintain the existing building layout. All loading conditions from the original design will be used. The floor and roof designs will be created using programs such as spSlab and spColumn and then spot checked with hand calculations. All structural members will be designed to ACI318-11 specifications.

The existing lateral system is a reinforced type of concrete masonry called ivany block, which when fully grouted claims to have similar performance to concrete with an f'c of 3000psi. The redesigned lateral system will be comprised of concrete shear walls with an f'c of 3000psi located in the same locations as the current lateral system. The redesign will challenge the claim by attempting to achieve similar performance with less material than the original design. The new concrete shear wall system will be modeled in ETABS. The change in material for the buildings superstructure will have a significant effect on construction.

Architecture Breadth

The current design of the building's façade involves the use of an external insulation finishing system coupled with a glazing system. This provides a stark contrast to the brick façade that has become common place in downtown Sharon. The façade will be redesigned to incorporate common motifs of downtown Sharon such as brick in combination with more modern looks such as glazing systems. The original design focused heavily on cost efficiency, as such all cost implications of the new façade system will be considered and compared. The new façade will then be created and rendered in Revit.

Construction Management Breadth

By changing the structure from steel to concrete the construction timeline will change dramatically. The construction of formwork and concrete curing time will need to be taken into account, as well as temperature considerations for pouring concrete. The site is located in an urban center and as such logistics will need special considerations. A detailed concrete construction schedule will be developed for the redesign in order to account for both site existing conditions and new structural demands. The change in materials will also affect the project cost. This will be investigated through a cost estimate comparison between the as built and redesigns.

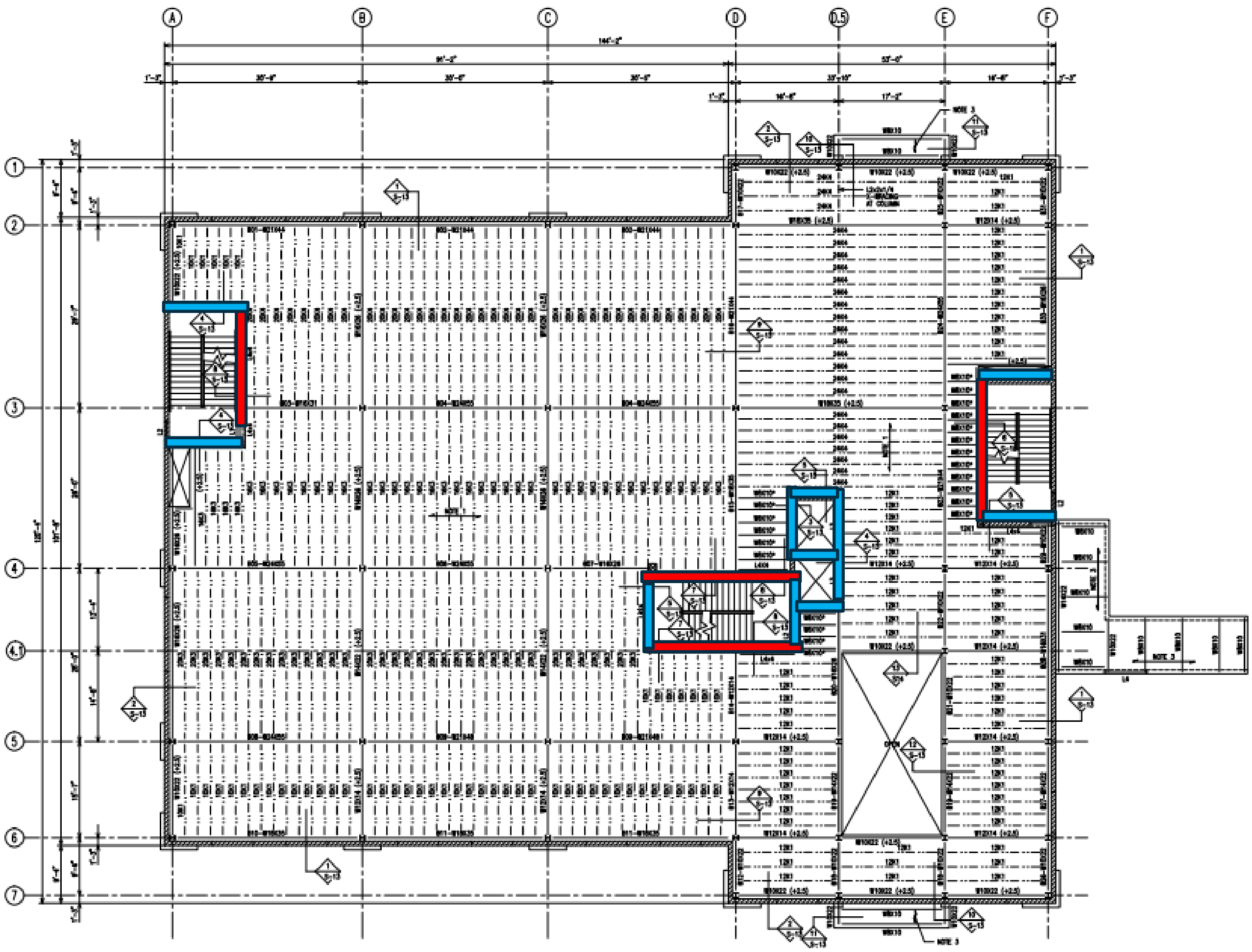
Structural Depth

Gravity System

Introduction

A preliminary design for the gravity system was created through the use of the *Concrete Reinforcing Steel Institute Design Handbook (CRSI)* table 10-27. Assuming an average span of 30 feet, 4000psi f'c concrete, with a superimposed load of 100psf CRSI suggests a 10 inch slab with 10ft square, 8.25 inch deep drop panels supported by 12 in square columns. All elevated floor slabs were designed using an 80 psf live load for corridors above the first floor to allow for flexibility in future renovations. An additional 20psf superimposed dead load was added to account for mechanical, electrical and plumbing equipment. The floorplan footprint does not change above the second floor, as such only two slabs were modeled for all elevated floor slabs. The roof slab will use the same design as similar floor slabs. Slab openings for M.E.P. equipment were not modeled since the project drawings did not show any openings in the floor system greater than 3 inches which is considered negligible. The intent of this thesis project was to redesign the buildings primary superstructure from a primarily steel framed structure into a reinforced concrete structure. The redesign focuses on retaining the original buildings design layout where possible, as such column locations and interior partition locations match the original layout unless noted otherwise. The original design had the stair towers encased in masonry. Sections of the masonry were designed as shear walls, while the remaining portions functioned as masonry bearing walls as seen in figure 20 below. The redesign considers the masonry bearing walls as interior partitions and as such will not include a redesign. These bearing walls will be used as supports for the floor structure was in the original design. All masonry shear walls will be redesigned to concrete in the lateral portion of the report.

Figure 20 - Existing Masonry Shear Walls & Masonry Infill Walls



Masonry Shear Walls are highlighted in red

Masonry Infill Walls are highlighted in blue

Slab design

All elevated floor slabs were modeled using spSlab. spSlab implements the equivalent frame method as outlined in ACI 318-11 for elevated two-way concrete slabs. All spSlab models used effective (cracked) sections for deflection calculations including a long term deflection load duration of 60 months. Initial designs in spSlab using the recommendations from the CRSI design handbook listed previously resulted in punching shear failures at roughly half of all column locations. It was determined that this shear failure was being caused by insufficient capacity for the transfer of moment in the column to slab connections resulting in the excess moment being transferred through shear. An initial redesign aimed at mitigating these excess moments by reducing the stiffness share for failing columns resulting in an increased moment on previously adequate columns. This method succeeded in reducing the loads on previously failing columns but resulted in more net failures than the original design.

The next design increased column sizes to 18" square while reducing drop panel dimensions to 9' square width and 8" depth. This change coupled with modifying the stiffness share in trouble locations succeeded in mitigating previous punching shear problems at all interior locations. The exterior columns still experienced failures due to punching shear which resulted in the addition of slab edge beams.

The edge beams were sized to the drop panel depth and column width in order to increase constructability and reduce required formwork. In locations where a sufficiently long span met a comparatively short span the drop panels were shortened to 1/6 the short span length in the shorter span direction.

The equivalent frame for column lines F3-F7 resulted in a deep beam between the supporting masonry infill wall and column F4 coupled with a 28' span from column F4 to F5. The negative moments created at column F4 continuing into the deep beam required reinforcing exceeding minimum spacing requirements. Column F4 (noted in green on figure 18) was moved to column line F4.1(Noted in Blue on Figure 18) in order to mitigate the excessive negative moments as well as the need for any "deep beam" provisions listed in ACI 318-11.

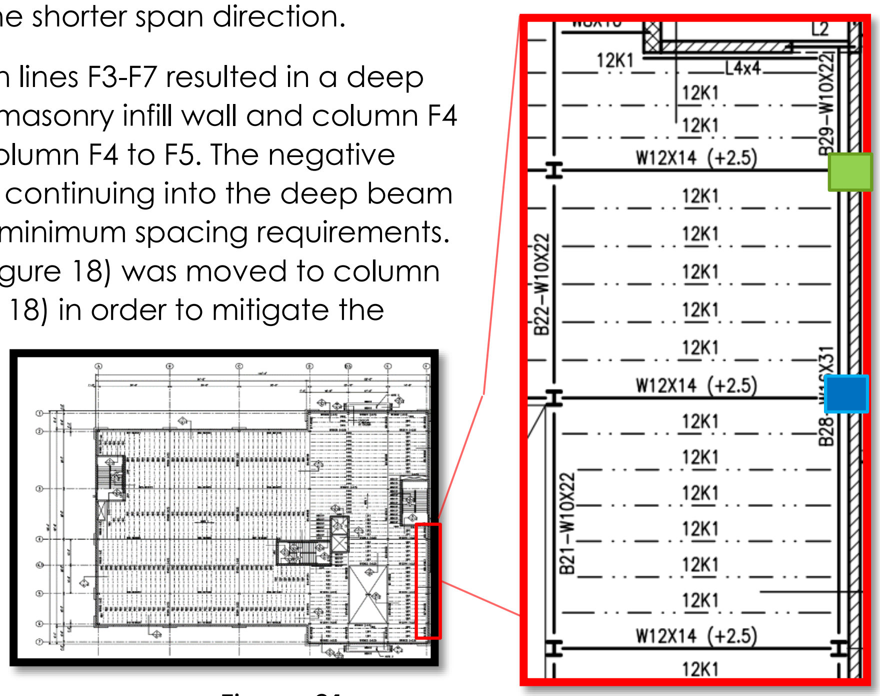


Figure 21

Figure 22 - Equivalent frames in the East-West Direction

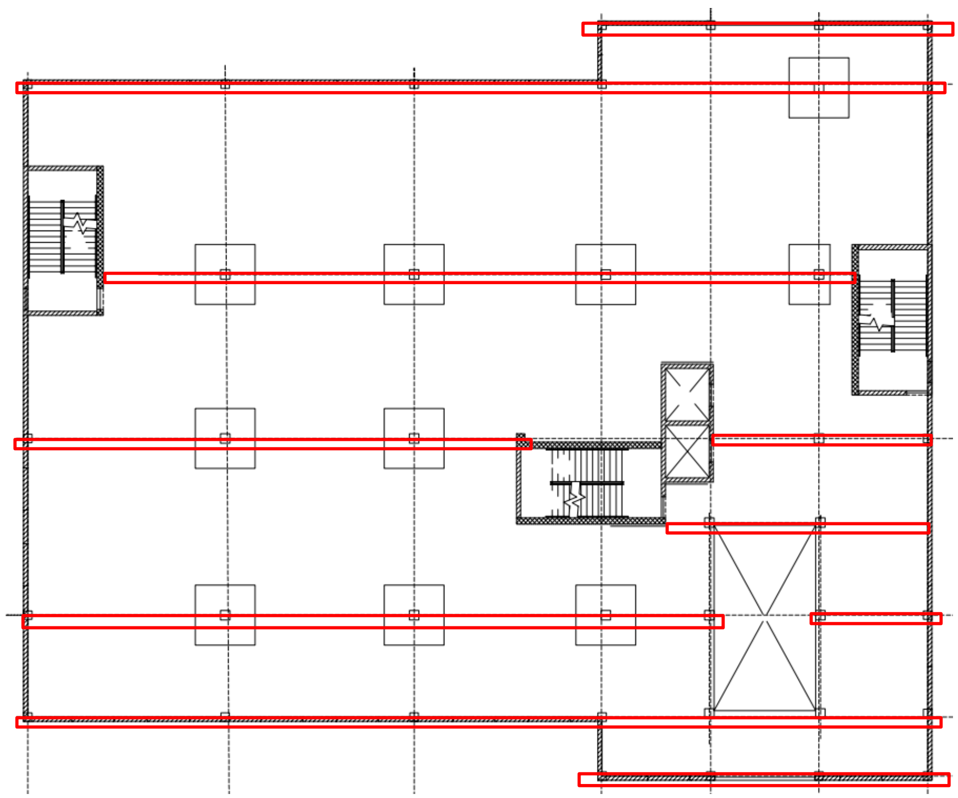


Figure 22 above represents the locations where equivalent frames were modeled using spSlab. Each rectangle represents an individual frame. Figure 23 on the following page is a representation of the frame created from column line 2A-2F.

Figure 23 – Equivalent frame modeled in spSlab along column lines 2A-2F.

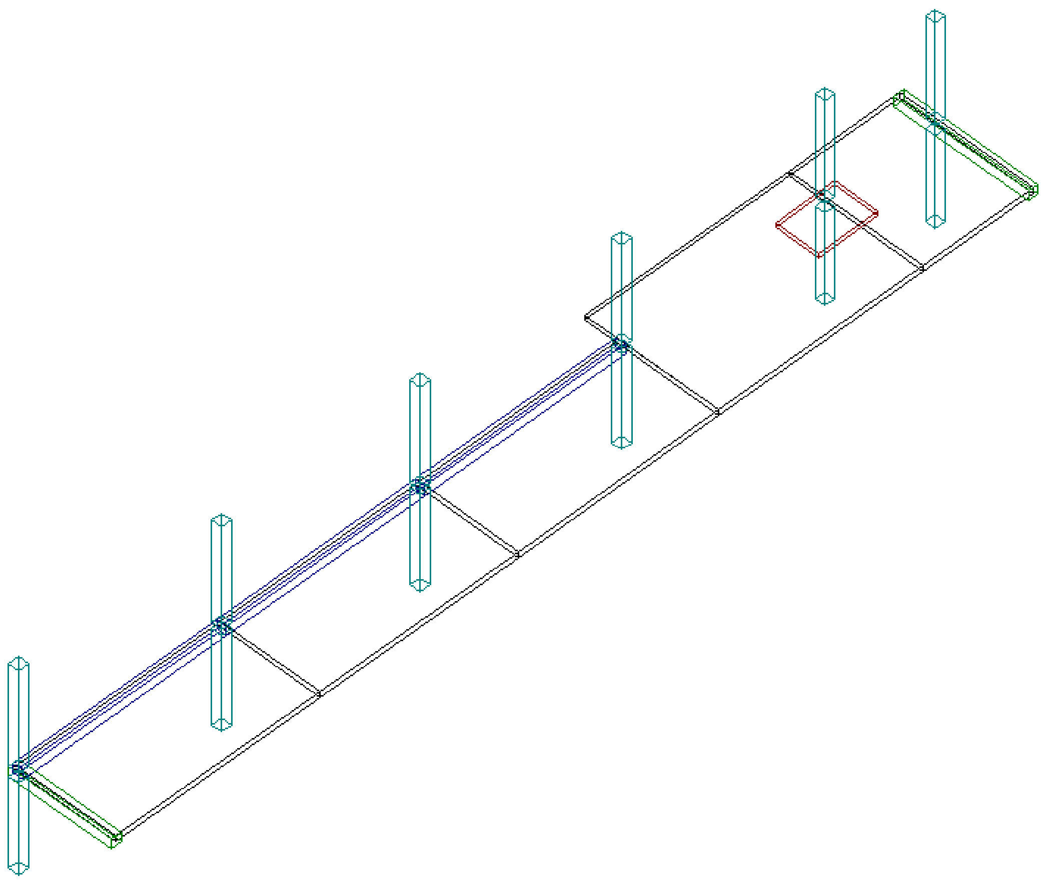
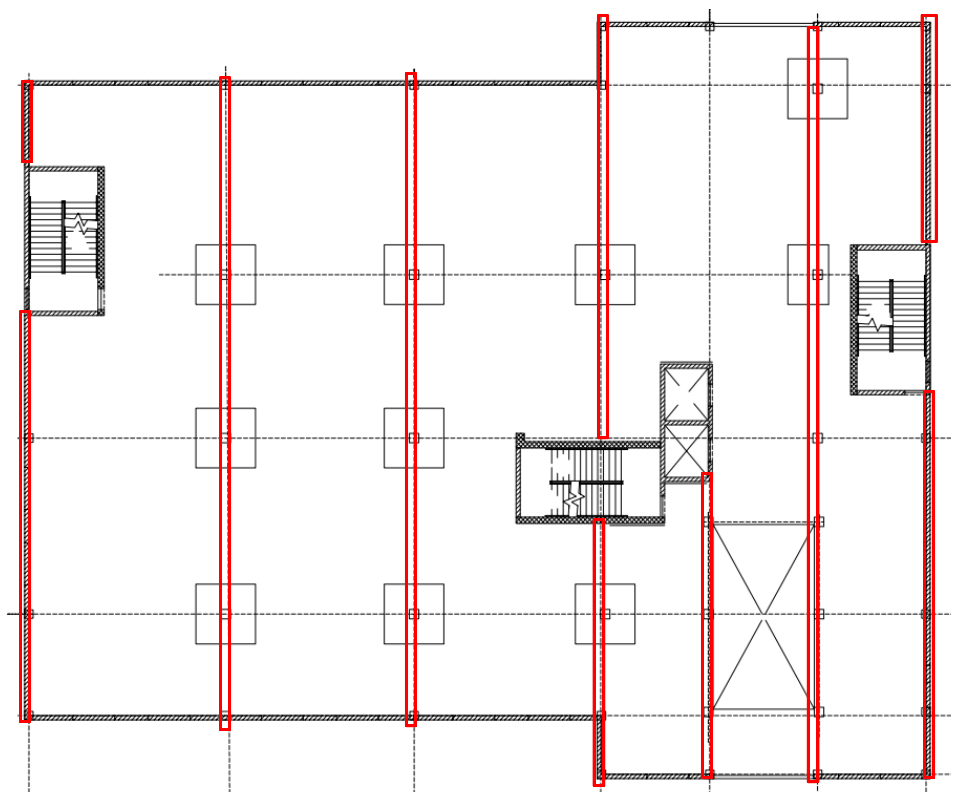
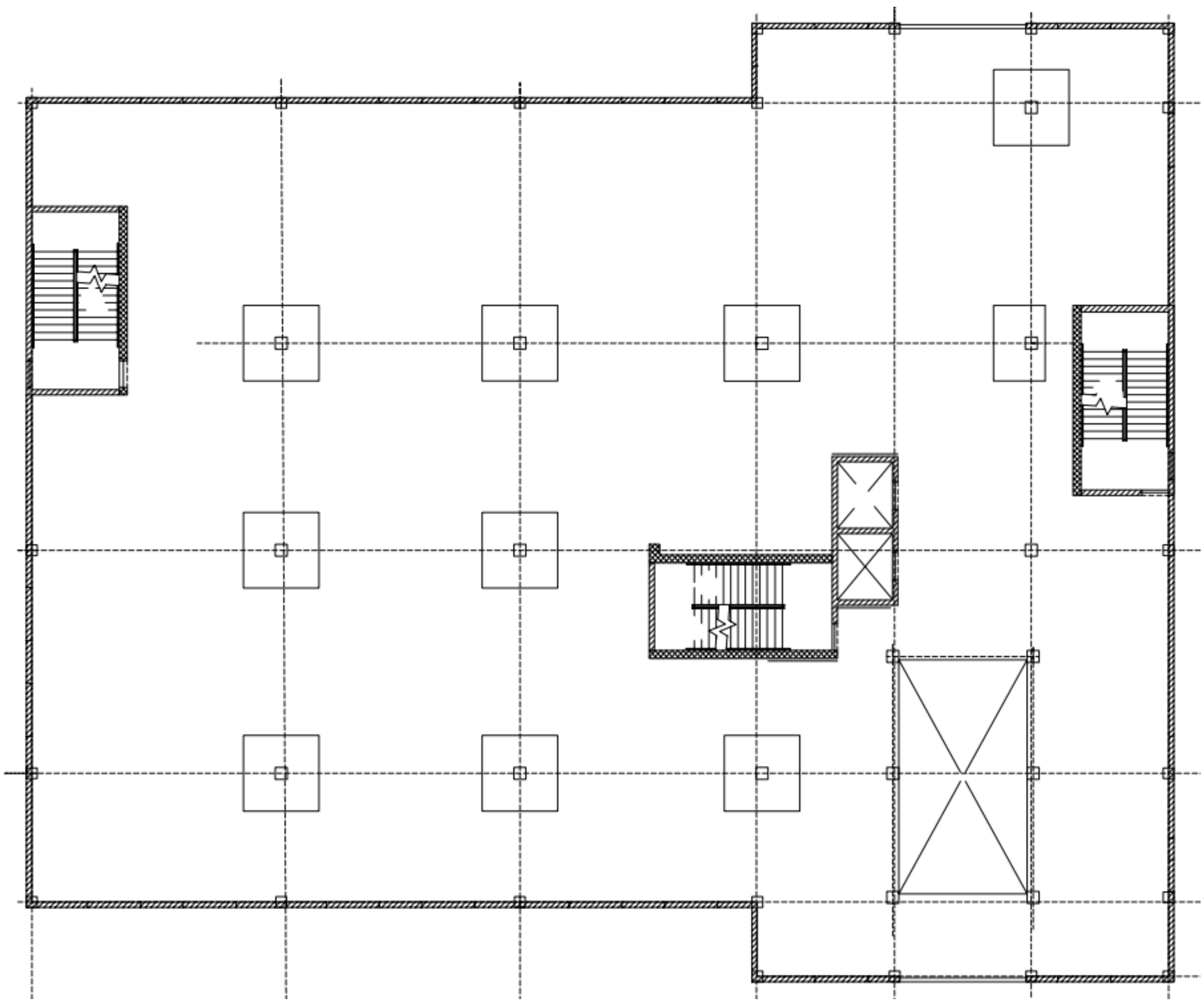


Figure 24 - Equivalent frames in the North-South Direction



Figures 22 and 24 show the equivalent frames created for the second floor plan. The second floor plan features a large opening between column lines 4.1 to 5 and D.5 to E (Can be seen in Figures 22 and 24). To create a layout for the remaining elevated floor slabs all frames which intersect with the opening were remodeled in spSlab.

Figure 25 – Drop Panel Locations



The above figure demonstrates the locations of drop panels on a typical floor plan.

Reinforcement Layout

An initial reinforcement layout was generated from spSlab. This layout proved to be highly impractical in terms of constructability due to frequent variances in both bar size and bar spacing between bays as seen in the sample output from two adjoining bays below (figures 26 & 27).

Figure 26 – spSlab reinforcement output (column line 4A-4C)

Top Bar Details
================

Units: Length (ft)

Span	Strip	Left Bars	Left Length	Left Bars	Left Length	Continuous Bars	Continuous Length	Right Bars	Right Length	Right Bars	Right Length
1	Column	10-#5	10.40	---		---		36-#5	11.68	35-#5	6.60
	Middle	10-#5	7.19	---		---		22-#5	11.68	---	
2	Column	31-#5	10.16	30-#5	6.45	10-#5	30.00	13-#5	10.16	13-#5	6.45
	Middle	12-#5	7.02	---		10-#5	30.00	4-#5	7.02	---	
3	Column	13-#5	6.55	13-#5	3.70	10-#5	16.25	---		---	
	Middle	---		---		14-#5	16.25	---		---	

Bottom Reinforcement
====================

Units: Width (ft), Mmax (k-ft), Xmax (ft), As (in^2), Sp (in)

Span	Strip	Width	Mmax	Xmax	AsMin	AsMax	AsReq	SpProv	Bars
1	Column	13.50	260.95	11.715	2.916	23.958	7.452	6.480	25-#5
	Middle	13.50	173.96	11.715	2.916	23.958	4.880	10.125	16-#5
2	Column	13.50	132.74	15.750	2.916	23.958	3.693	13.500	12-#5
	Middle	13.50	88.49	15.750	2.916	23.958	2.441	16.200	10-#5 *3
3	Column	8.13	54.10	9.750	1.755	14.419	1.493	16.250	6-#5 *3
	Middle	18.88	36.07	9.750	4.077	33.496	0.984	16.179	14-#5 *3

NOTES:
*3 - Design governed by minimum reinforcement.

Figure 27 – spSlab reinforcement output (column line 5A-5D.5)

Top Reinforcement

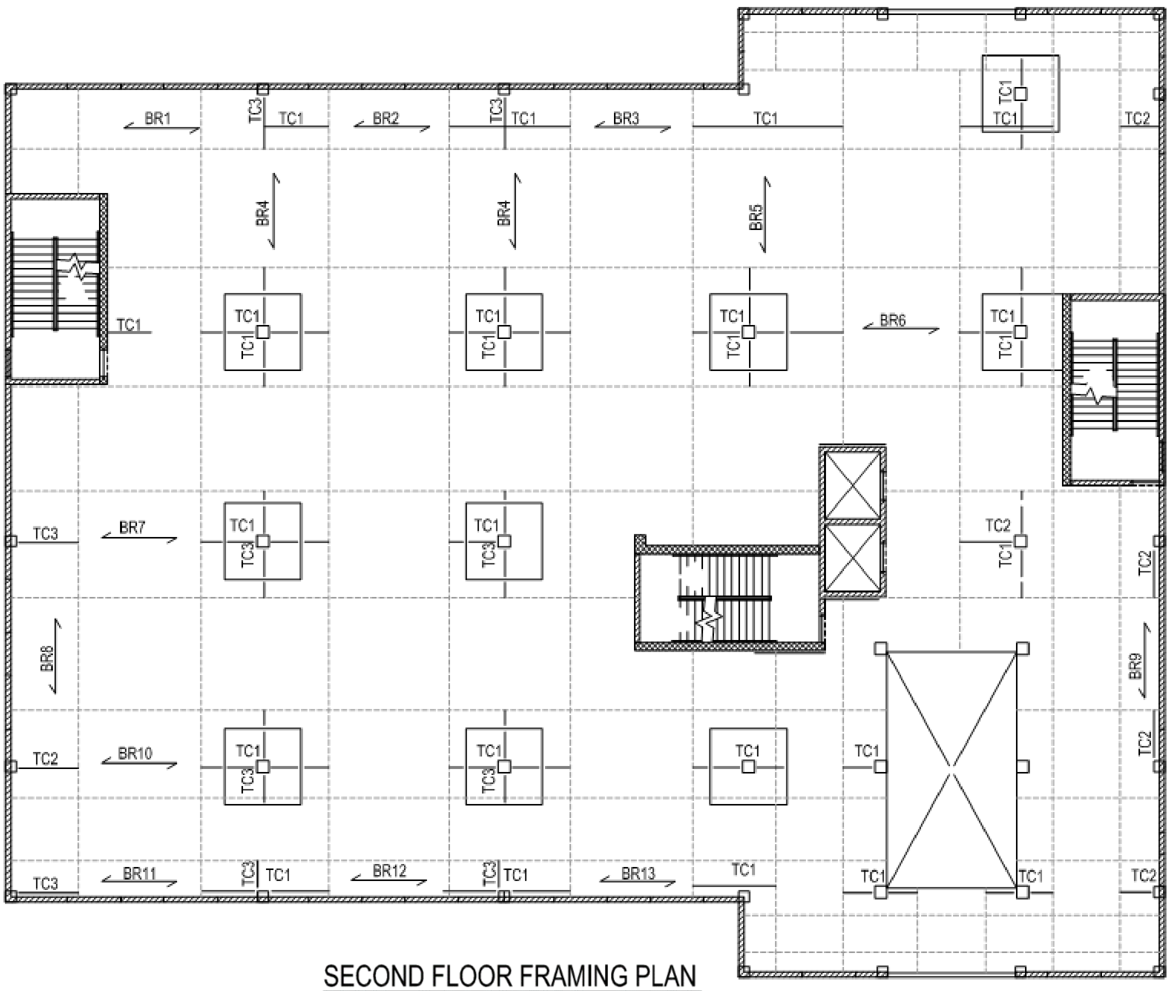
Units: Width (ft), Mmax (k-ft), Xmax (ft), As (in^2), Sp (in)

Span	Strip	Zone	Width	Mmax	Xmax	AsMin	AsMax	AsReq	SpProv	Bars
1	Column	Left	10.90	219.50	0.750	2.354	19.344	6.283	6.229	21-#5
		Midspan	10.90	0.00	15.375	0.000	19.344	0.000	0.000	---
		Right	10.90	496.61	30.000	2.354	19.344	15.444	2.616	50-#5
	Middle	Left	10.90	7.17	0.750	2.354	19.344	0.195	16.350	8-#5 *3
		Midspan	10.90	0.00	15.375	0.000	19.344	0.000	0.000	---
		Right	10.90	165.54	30.000	2.354	19.344	4.673	8.175	16-#5
2	Column	Left	10.90	438.16	0.750	2.354	19.344	13.364	2.616	50-#5
		Midspan	10.90	9.08	10.725	2.354	19.344	0.247	16.350	8-#5 *3
		Right	10.90	372.64	29.250	2.354	19.344	11.136	3.354	39-#5
	Middle	Left	10.90	146.05	0.750	2.354	19.344	4.103	8.175	16-#5
		Midspan	10.90	3.03	10.725	2.354	19.344	0.082	16.350	8-#5 *3
		Right	10.90	124.21	29.250	2.354	19.344	3.471	10.062	13-#5
3	Column	Left	10.90	400.60	0.750	2.354	19.344	12.074	3.354	39-#5
		Midspan	10.90	0.00	15.210	0.000	19.344	0.000	0.000	---
		Right	8.07	348.57	29.670	1.743	14.317	10.746	2.766	35-#5
	Middle	Left	10.90	133.53	0.750	2.354	19.344	3.740	10.062	13-#5
		Midspan	10.90	0.00	15.210	0.000	19.344	0.000	0.000	---
		Right	13.73	116.19	29.670	2.966	24.370	3.221	14.981	11-#5
4	Column	Left	8.07	298.09	0.750	1.743	14.317	8.990	2.766	35-#5
		Midspan	8.07	94.22	6.059	1.743	14.317	2.635	10.757	9-#5
		Right	8.07	15.82	15.920	1.743	14.317	0.432	10.757	9-#5 *3
	Middle	Left	13.73	99.37	0.750	2.966	24.370	2.746	14.981	11-#5 *3
		Midspan	13.73	19.79	6.059	2.966	24.370	0.539	16.479	10-#5 *3
		Right	13.73	0.52	15.920	2.966	24.370	0.014	16.479	10-#5 *3

NOTES:
*3 - Design governed by minimum reinforcement.

In order to create a more feasible layout a reinforcement grid was implemented over the entirety of the slab for bottom reinforcing bars. 40% coverage was determined to be the optimal balance between constructability and structural efficiency. All required areas of reinforcement for the frames modeled in spSlab were brought into an Excel spreadsheet from which the reinforcement area covering 40% of frames was calculated. This calculated area proved to be the value for minimum reinforcement. The reinforcement grid chosen was #5 bars spaced at 12 inches on center in each direction. The additional reinforcing area required for each bay was then calculated, any additional area required less than 0.04 in^2/ft. was considered negligible. All additional reinforcement used were #7 bars. #7 bars were implemented in order to clearly differentiate on site between the bars used for the grid and the add. bars. All top reinforcement will be #5 bars with varying spacing requirements. A Full reinforcement layout can be seen in Figure 28 below.

Figure 28 – Reinforcement layout



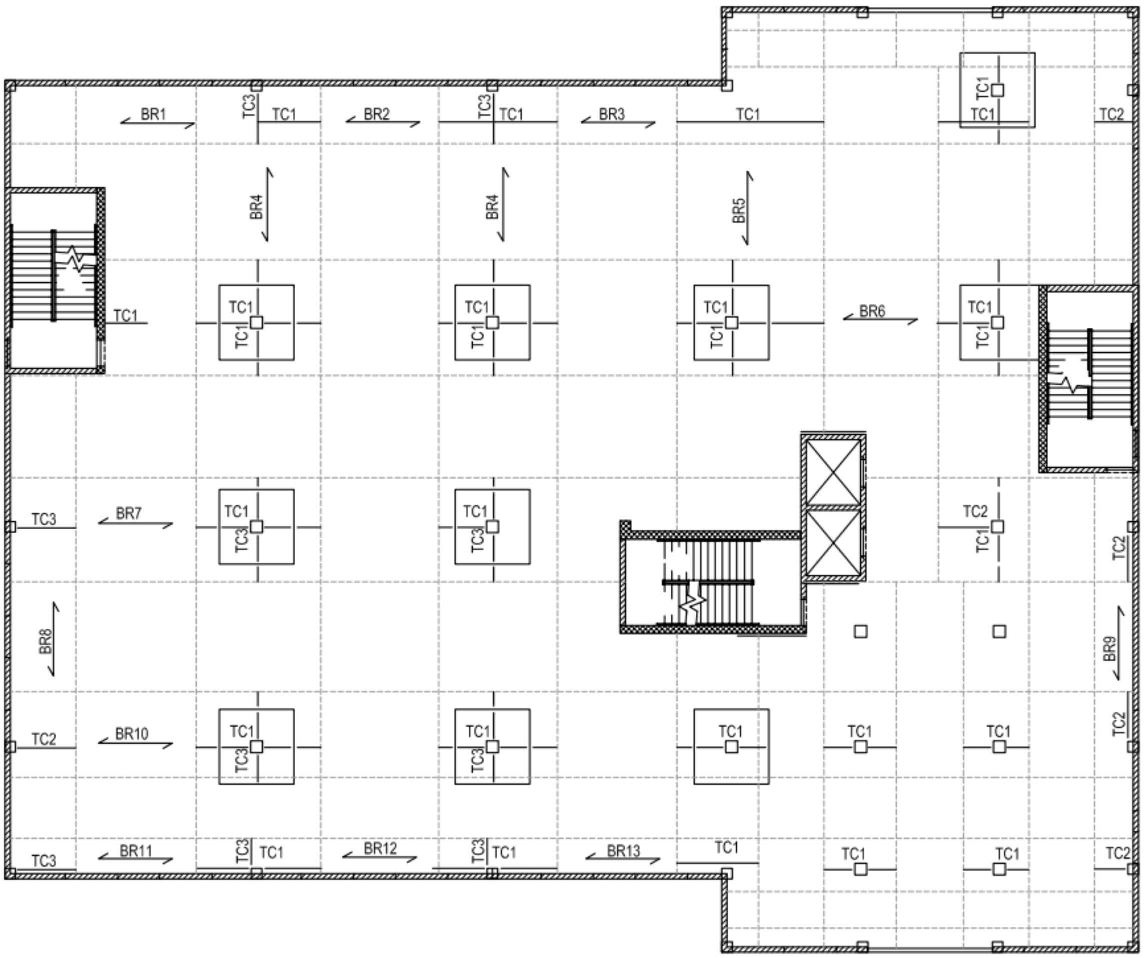
REINFORCING SCHEDULE

CALLOUT	LAYER	REINFORCING DETAIL
BR1	BOTTOM	(4) #7 BARS
BR2	BOTTOM	(2) #7 BARS
BR3	BOTTOM	(2) #7 BARS
BR4	BOTTOM	(3) #7 BARS
BR5	BOTTOM	(3) #7 BARS
BR6	BOTTOM	(4) #7 BARS
BR7	BOTTOM	(4) #7 BARS
BR8	BOTTOM	(2) #7 BARS
BR9	BOTTOM	(1) #7 BARS
BR10	BOTTOM	(3) #7 BARS
BR11	BOTTOM	(3) #7 BARS
BR12	BOTTOM	(2) #7 BARS
BR13	BOTTOM	(2) #7 BARS
TC1	TOP	(2) #7 BARS
TC2	TOP	(1) #7 BARS
TC3	TOP	(3) #7 BARS

SECOND FLOOR FRAMING PLAN
SCALE: 1/8" = 1'-0"

PLAN NOTES:

1. SLAB CONSTRUCTION IS 10" NORMAL WEIGHT CONCRETE OF 4000 PSI COMPRESSIVE STRENGTH WITH 60,000 PSI REINFORCING STEEL.
2. BOTTOM MAT OF REINFORCING WILL BE #5@12" O.C. IN EACH DIRECTION CONTINUOUS. ADDITIONAL BOTTOM REINFORCING IN REINFORCING SCHEDULE AS NOTED ON PLAN AND SHALL RUN FROM COLUMN TO COLUMN.
3. TOP MAT OF REINFORCING WILL BE #5@12" O.C. IN EACH DIRECTION ADDITIONAL TOP REINFORCING IN REINFORCING SCHEDULE AS NOTED ON PLAN



REINFORCING SCHEDULE

CALLOUT	LAYER	REINFORCING DETAIL
BR1	BOTTOM	(4) #7 BARS
BR2	BOTTOM	(2) #7 BARS
BR3	BOTTOM	(2) #7 BARS
BR4	BOTTOM	(3) #7 BARS
BR5	BOTTOM	(3) #7 BARS
BR6	BOTTOM	(4) #7 BARS
BR7	BOTTOM	(4) #7 BARS
BR8	BOTTOM	(2) #7 BARS
BR9	BOTTOM	(1) #7 BARS
BR10	BOTTOM	(3) #7 BARS
BR11	BOTTOM	(3) #7 BARS
BR12	BOTTOM	(2) #7 BARS
BR13	BOTTOM	(2) #7 BARS
TC1	TOP	(2) #7 BARS
TC2	TOP	(1) #7 BARS
TC3	TOP	(3) #7 BARS

TYPICAL FLOOR FRAMING PLAN

SCALE: 1/8" = 1'-0"

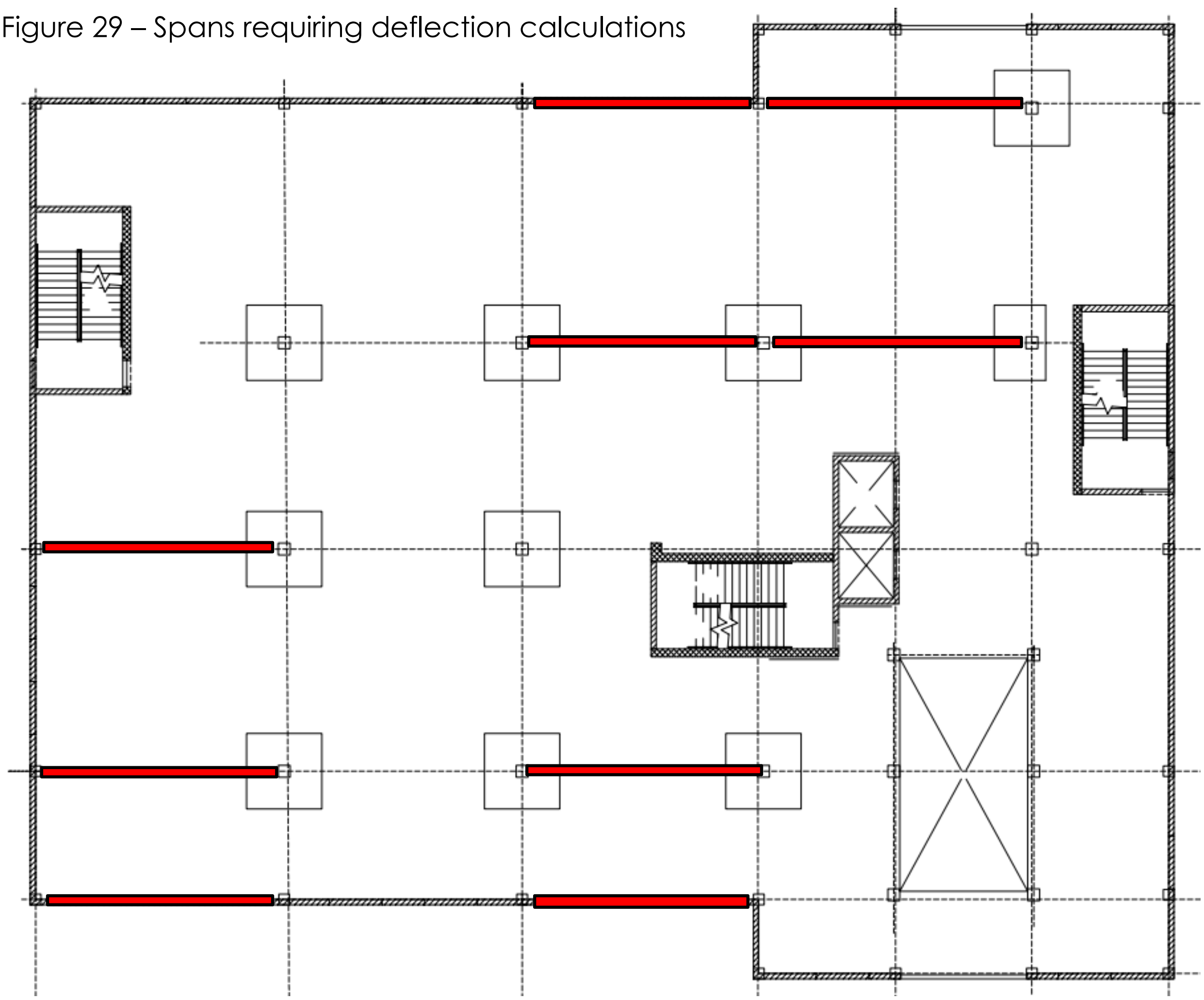
PLAN NOTES:

1. SLAB CONSTRUCTION IS 10" NORMAL WEIGHT CONCRETE OF 4000 PSI COMPRESSIVE STRENGTH WITH 60,000 PSI REINFORCING STEEL.
2. BOTTOM MAT OF REINFORCING WILL BE #5@12" O.C. IN EACH DIRECTION CONTINUOUS. ADDITIONAL BOTTOM REINFORCING IN REINFORCING SCHEDULE AS NOTED ON PLAN AND SHALL RUN FROM COLUMN TO COLUMN.
3. TOP MAT OF REINFORCING WILL BE #5@12" O.C. IN EACH DIRECTION ADDITIONAL TOP REINFORCING IN REINFORCING SCHEDULE AS NOTED ON PLAN

Deflections

ACI 318-11 governs the minimum thickness of slabs in chapter 9. The minimum thickness for a two way slab with drop panels using 60 ksi reinforcing is l/36 for interior panels as well as exterior panels with edge beams per ACI 318-11 Table 9.5(c). Therefore a 10" slab may have up to a 30' span. Slabs having a thickness less than this minimum shall be permitted where computed deflections do not exceed the limits provided in Table 9.5(b) per ACI 318-11 9.5.3.4. Table 9.5(b) requires a deflection limitation of l/360 due to immediate live load only for floors not supporting or attached to non-structural elements not likely to be damaged by large deflections as well as a limitation of l/240 due to dead and live loads including long term loads for floors supporting or attached to non-structural elements not likely to be damaged by large deflections. Figure 29 below highlights the spans requiring deflection calculations.

Figure 29 – Spans requiring deflection calculations



Deflections of long spans along column line 2

```
Maximum Instantaneous Deflections - Direction of Analysis
---------------------------------------------------------
Units:  D (in), Ig (in^4)
          ___________Frame___________ _____________________________Strips______________________________
Span    Ddead     Dlive    Dtotal Strip               Ig    LDF  Ratio    Ddead     Dlive    Dtotal
---- -------- -------- -------- ------- ------------ ------ ------ -------- -------- --------
   1    0.452    0.397    0.849 Column         19085  0.800  1.138    0.514    0.452    0.966
                                Middle          7400  0.200  0.734    0.332    0.291    0.623
   2    0.082    0.184    0.265 Column         19085  0.761  1.083    0.088    0.199    0.287
                                Middle          7400  0.239  0.876    0.072    0.161    0.232
   3    0.148    0.227    0.375 Column         19085  0.761  1.082    0.161    0.246    0.406
                                Middle          7400  0.239  0.877    0.130    0.199    0.329
   4    0.152    0.138    0.290 Column         12900  0.675  1.062    0.162    0.147    0.308
                                Middle          7400  0.325  0.892    0.136    0.123    0.259
   5   -0.011   -0.008   -0.019 Column          8335  0.726  1.768   -0.020   -0.014   -0.034
                                Middle         11965  0.274  0.465   -0.005   -0.004   -0.009

Maximum Long-term Deflections - Direction of Analysis
-----------------------------------------------------
Time dependant factor for sustained loads = 2.000
Units:  D (in)
          ____________________Column Strip_____________________ ____________________Middle Strip_____________________
Span    Dsust Lambda      Dcs   Dcs+lu     Dcs+l   Dtotal    Dsust Lambda      Dcs   Dcs+lu     Dcs+l   Dtotal
---- -------- ------ -------- -------- -------- -------- -------- ------ -------- -------- -------- --------
   1    0.514  2.000    1.027    1.479    1.479    1.993    0.331  2.000    0.663    0.954    0.954    1.286
   2    0.088  2.000    0.177    0.376    0.376    0.464    0.071  2.000    0.143    0.304    0.304    0.375
   3    0.161  2.000    0.321    0.567    0.567    0.727    0.130  2.000    0.260    0.459    0.459    0.589
   4    0.162  2.000    0.323    0.470    0.470    0.631    0.136  2.000    0.271    0.394    0.394    0.530
   5   -0.020  2.000   -0.040   -0.054   -0.054   -0.073   -0.005  2.000   -0.010   -0.014   -0.014   -0.019
```

Deflections						
Column Line	Span	Allowable live	Actual Live	Allowable Total	Actual Total	Design Result
2	3	1.01"	0.406"	1.53"	0.73"	Pass
2	4	1.13"	0.308"	1.69"	0.63"	Pass

Deflections of long spans along column line 3

Maximum Instantaneous Deflections - Direction of Analysis

Units: D (in), Ig (in^4)

	Frame			Strips						
Span	Ddead	Dlive	Dtotal	Strip	Ig	LDF	Ratio	Ddead	Dlive	Dtotal
1	0.016	0.009	0.025	Column	9750	0.738	2.103	0.034	0.018	0.052
				Middle	18050	0.262	0.404	0.007	0.003	0.010
2	0.096	0.059	0.155	Column	13900	0.675	1.350	0.130	0.079	0.209
				Middle	13900	0.325	0.650	0.062	0.038	0.101
3	0.059	0.047	0.106	Column	13900	0.675	1.350	0.080	0.064	0.144
				Middle	13900	0.325	0.650	0.038	0.031	0.069
4	0.222	0.349	0.570	Column	13900	0.675	1.350	0.299	0.471	0.770
				Middle	13900	0.325	0.650	0.144	0.227	0.371
5	-0.007	-0.006	-0.013	Column	2750	0.738	7.455	-0.054	-0.042	-0.096
				Middle	25050	0.262	0.291	-0.002	-0.002	-0.004

Maximum Long-term Deflections - Direction of Analysis

Time dependant factor for sustained loads = 2.000

Units: D (in)

	Column Strip						Middle Strip					
Span	Dsust	Lambda	Dcs	Dcs+lu	Dcs+l	Dtotal	Dsust	Lambda	Dcs	Dcs+lu	Dcs+l	Dtotal
1	0.034	2.000	0.069	0.087	0.087	0.121	0.007	2.000	0.013	0.017	0.017	0.023
2	0.130	2.000	0.259	0.339	0.339	0.468	0.062	2.000	0.125	0.163	0.163	0.226
3	0.080	2.000	0.160	0.223	0.223	0.303	0.038	2.000	0.077	0.108	0.108	0.146
4	0.299	2.000	0.598	1.069	1.069	1.368	0.144	2.000	0.288	0.515	0.515	0.659
5	-0.054	2.000	-0.108	-0.150	-0.150	-0.205	-0.002	2.000	-0.004	-0.006	-0.006	-0.008

Deflections						
Column Line	Span	Allowable live	Actual Live	Allowable Total	Actual Total	Design Result
3	3	1.01"	0.14"	1.53"	0.30"	Pass
3	4	1.13"	0.770"	1.69"	1.37"	Pass

Deflection of long spans along column line 4

```
Maximum Instantaneous Deflections - Direction of Analysis
---------------------------------------------------------
Units:  D (in), Ig (in^4)
     ___________Frame___________ ____________________________Strips___________________________
Span    Ddead     Dlive    Dtotal Strip              Ig    LDF  Ratio     Ddead     Dlive    Dtotal
---- --------- --------- --------- ------- ------------ ------ ------ --------- --------- ---------
   1     0.225     0.432     0.657 Column        13500  0.731  1.462     0.329     0.632     0.960
                                   Middle        13500  0.269  0.538     0.121     0.232     0.353
   2     0.075     0.055     0.130 Column        13500  0.675  1.350     0.102     0.074     0.176
                                   Middle        13500  0.325  0.650     0.049     0.036     0.085
   3    -0.005    -0.004    -0.009 Column         8125  0.738  2.451    -0.012    -0.009    -0.021
                                   Middle        18875  0.262  0.375    -0.002    -0.001    -0.003

Maximum Long-term Deflections - Direction of Analysis
-----------------------------------------------------
Time dependant factor for sustained loads = 2.000
Units:  D (in)
     ___________________Column Strip_____________________ ____________________Middle Strip_____________________
Span    Dsust Lambda       Dcs    Dcs+lu      Dcs+l    Dtotal     Dsust Lambda       Dcs    Dcs+lu     Dcs+l    Dtotal
---- --------- ------ --------- --------- --------- --------- --------- ------ --------- --------- --------- ---------
   1     0.329  2.000     0.657     1.289     1.289     1.618     0.121  2.000     0.242     0.474     0.474     0.595
   2     0.102  2.000     0.203     0.277     0.277     0.379     0.049  2.000     0.098     0.133     0.133     0.182
   3    -0.012  2.000    -0.025    -0.034    -0.034    -0.046    -0.002  2.000    -0.004    -0.005    -0.005    -0.007
```

Deflections						
Column Line	Span	Allowable live	Actual Live	Allowable Total	Actual Total	Design Result
4	1	1.02"	0.96"	1.54"	1.62"	Fail

spSlab calculates a deflection greater than the permissible per ACI 318-11 due to an inability to accurately model the masonry bearing wall along column line 3. The masonry bearing wall not only shortens the tributary area of column A4 by extending down past column line 3, but also provides a support condition extending nearly half the span length. Therefore by inspection the bay passes deflection criteria as the calculated value is only marginally higher than the allowable.

Deflection of long spans along column line 5

Maximum Instantaneous Deflections - Direction of Analysis

Units: D (in), Ig (in^4)

	Frame			Strips						
Span	Ddead	Dlive	Dtotal	Strip	Ig	LDF	Ratio	Ddead	Dlive	Dtotal
1	0.167	0.194	0.361	Column	10900	0.730	1.459	0.244	0.283	0.527
				Middle	10900	0.270	0.541	0.090	0.105	0.195
2	0.052	0.037	0.089	Column	10900	0.675	1.350	0.071	0.049	0.120
				Middle	10900	0.325	0.650	0.034	0.024	0.058
3	0.097	0.057	0.154	Column	10900	0.675	1.350	0.131	0.077	0.208
				Middle	10900	0.325	0.650	0.063	0.037	0.100
4	-0.007	-0.004	-0.010	Column	8067.5	0.730	1.972	-0.013	-0.008	-0.021
				Middle	13732.5	0.270	0.429	-0.003	-0.002	-0.004

Maximum Long-term Deflections - Direction of Analysis

Time dependant factor for sustained loads = 2.000

Units: D (in)

	Column Strip						Middle Strip					
Span	Dsust	Lambda	Dcs	Dcs+lu	Dcs+l	Dtotal	Dsust	Lambda	Dcs	Dcs+lu	Dcs+l	Dtotal
1	0.244	2.000	0.488	0.771	0.771	1.015	0.090	2.000	0.181	0.286	0.286	0.376
2	0.071	2.000	0.141	0.191	0.191	0.261	0.034	2.000	0.068	0.092	0.092	0.126
3	0.131	2.000	0.262	0.339	0.339	0.469	0.063	2.000	0.126	0.163	0.163	0.226
4	-0.013	2.000	-0.026	-0.034	-0.034	-0.047	-0.003	2.000	-0.006	-0.007	-0.007	-0.010

Deflections						
Column Line	Span	Allowable live	Actual Live	Allowable Total	Actual Total	Design Result
5	1	1.02"	0.57"	1.54"	1.02"	Pass
5	3	1.13"	0.21"	1.69"	0.46"	Pass

Deflection of long spans along column line 6

```
Maximum Instantaneous Deflections - Direction of Analysis
---------------------------------------------------------
Units:  D (in), Ig (in^4)
     ___________Frame____________ ___________________________Strips_____________________________
Span    Ddead    Dlive   Dtotal Strip            Ig    LDF  Ratio    Ddead    Dlive   Dtotal
---- -------- -------- -------- ------- ------------ ------ ------ -------- -------- --------
   1    0.177    0.233    0.410 Column      14689.4  0.814  1.083    0.192    0.252    0.444
                                Middle         3900  0.186  0.933    0.165    0.217    0.382
   2    0.079    0.089    0.168 Column      14689.4  0.789  1.050    0.083    0.093    0.177
                                Middle         3900  0.211  1.058    0.084    0.094    0.178
   3    0.131    0.184    0.315 Column      14689.4  0.788  1.048    0.137    0.193    0.330
                                Middle         3900  0.212  1.064    0.140    0.196    0.335
   4   -0.006    0.003   -0.008 Column         6235  0.675  1.350   -0.008    0.005   -0.011
                                Middle         6235  0.325  0.650   -0.004    0.002   -0.005
   5    0.003   -0.001    0.003 Column      11562.3  0.868  1.181    0.003   -0.001    0.004
                                Middle         2710  0.132  0.769    0.002   -0.001    0.003
   6    0.020    0.011    0.031 Column         6235  0.724  1.447    0.029    0.016    0.045
                                Middle         6235  0.276  0.553    0.011    0.006    0.017

Maximum Long-term Deflections - Direction of Analysis
-----------------------------------------------------
Time dependant factor for sustained loads = 2.000
Units:  D (in)
     ___________________Column Strip___________________ ___________________Middle Strip___________________
Span    Dsust Lambda      Dcs   Dcs+lu    Dcs+l   Dtotal    Dsust Lambda      Dcs   Dcs+lu    Dcs+l   Dtotal
---- -------- ------ -------- -------- -------- -------- -------- ------ -------- -------- -------- --------
   1    0.191  2.000    0.383    0.635    0.635    0.826    0.165  2.000    0.330    0.547    0.547    0.712
   2    0.083  2.000    0.166    0.260    0.260    0.343    0.084  2.000    0.168    0.262    0.262    0.346
   3    0.137  2.000    0.275    0.468    0.468    0.605    0.140  2.000    0.279    0.475    0.475    0.614
   4   -0.016  2.000   -0.032   -0.027   -0.027   -0.043   -0.008  2.000   -0.015   -0.013   -0.013   -0.021
   5    0.005  2.000    0.010    0.009    0.009    0.014    0.003  2.000    0.007    0.006    0.006    0.009
   6    0.029  2.000    0.057    0.073    0.073    0.102    0.011  2.000    0.022    0.028    0.028    0.039
```

Deflections						
Column Line	Span	Allowable live	Actual Live	Allowable Total	Actual Total	Design Result
5	1	1.02"	0.44"	1.54"	0.82"	Pass
5	3	1.13"	0.34"	1.69"	0.61"	Pass

Column Design

The redesign focused heavily on keeping the existing column layout, only changing one column location as noted earlier. The original design had a floor to floor height of 15'-2" for floors 1-4 with a fifth floor height of 18'-2". The redesign regularized the floor to floor height to 15'-6" typical giving an overall building height of 77'-6" as compared to the original buildings two varying heights of 73'-4" and 79'-0". Column geometry and reinforcement layout will not change throughout the height of the column and as such all columns were designed at base level. This design decision was made due to the buildings low height and live loads. Four column locations were investigated for design in order to balance constructability and structural efficiency. These locations included two interior columns and two exterior column. Out of the interior columns, one location was selected for having the largest tributary area while the second location was selected for having large tributary widths in two direction coupled with small tributary widths in the opposing directions likely leading to biaxial bending. The exterior column locations were both selected for having large tributary widths, one of these was also a corner column which could also experience the effects of biaxial bending. All columns were designed as 18" square using spColumn. spColumn produced the same reinforcement layout for both exterior columns, and produced similar reinforcement layouts for each of the interior columns; as such the redesign will utilize one layout for all exterior columns and one layout for all interior columns. All four column designs produced by spColumn were verified by hand plotting a minimum of two points on the column interaction diagram. These calculations are available in Appendix B.

Exterior Column Design

Exterior columns will have 4 #9 tied vertical bars providing a reinforcement utilization of 1.235% as seen in Figure 30. Figure 31 below represents the column interaction diagram for exterior columns.

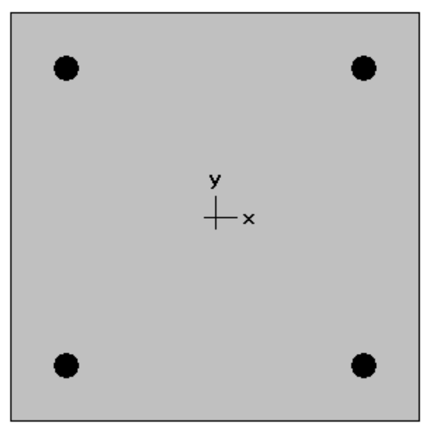
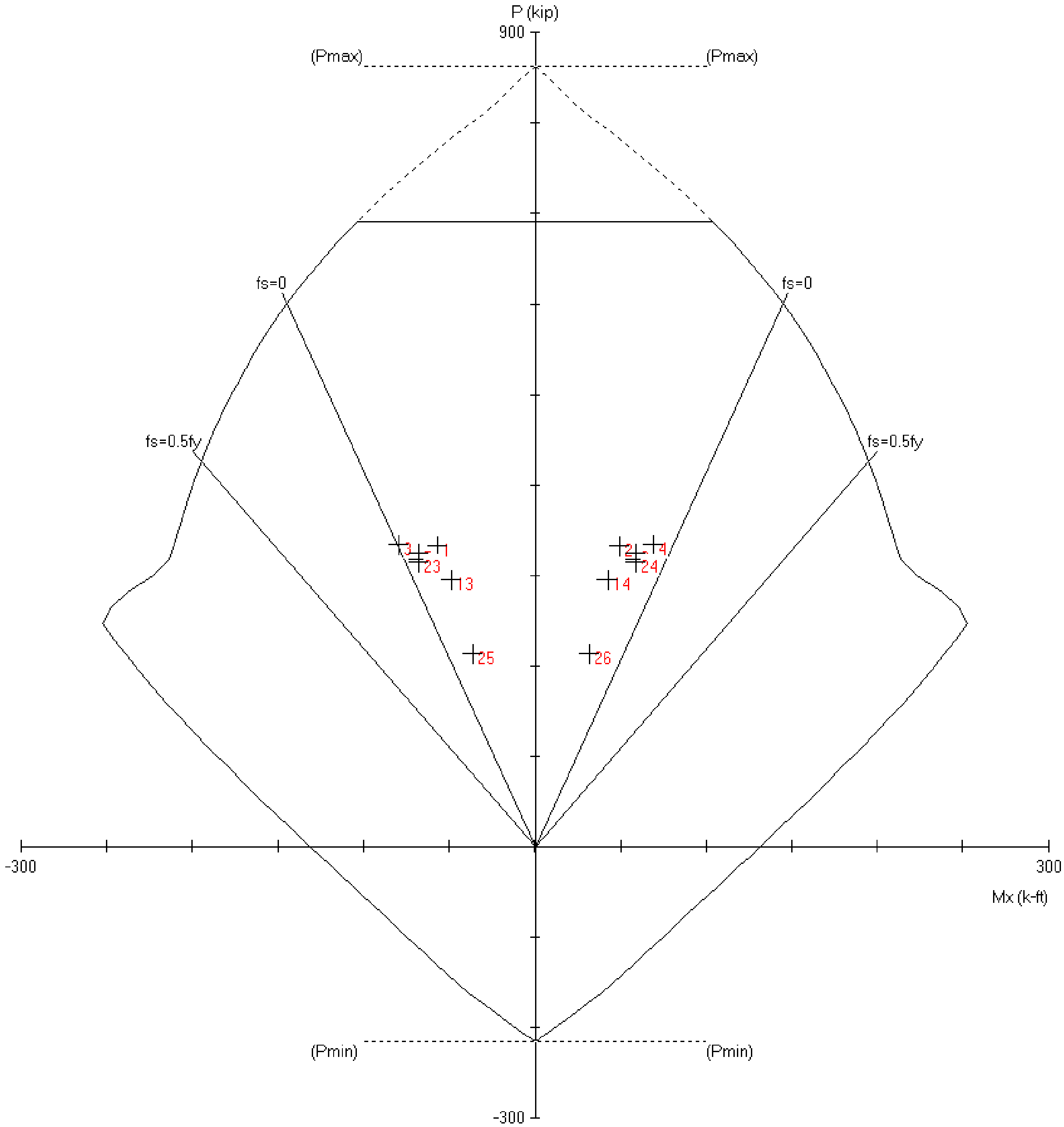


Figure 30 – Exterior Column

Figure 31 – Exterior Column Interaction Diagram



Interior Column Design

Interior columns will have 16 #9 tied vertical bars providing a reinforcement utilization of 4.94% as seen in Figure 33. Figure 34 below represents the column interaction diagram for interior columns.

Figure 33 – Interior Column Design

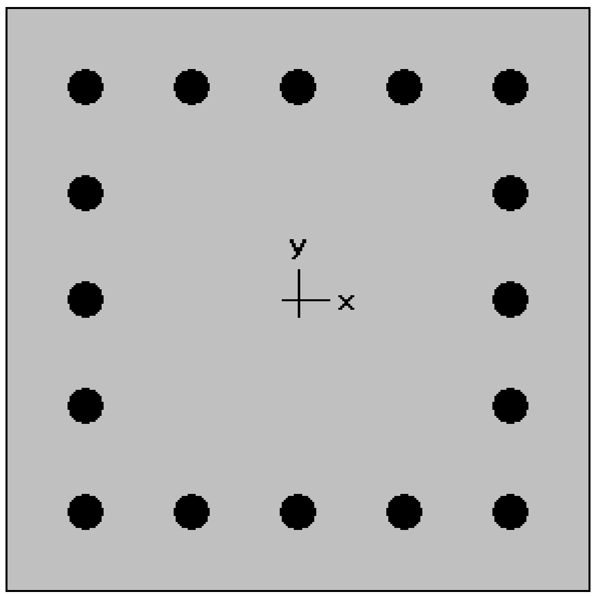
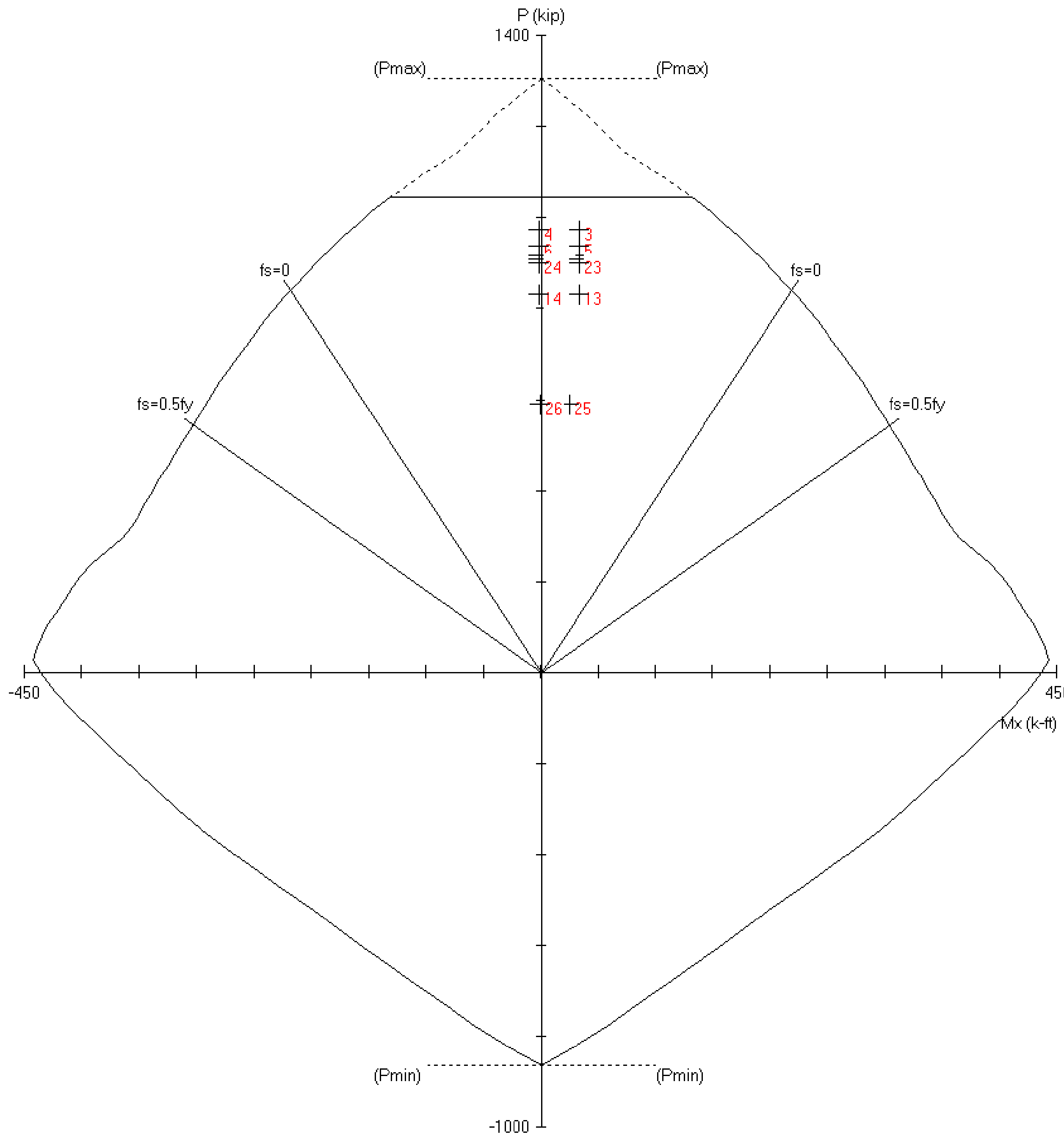


Figure 34 – Interior Column Interaction Diagram



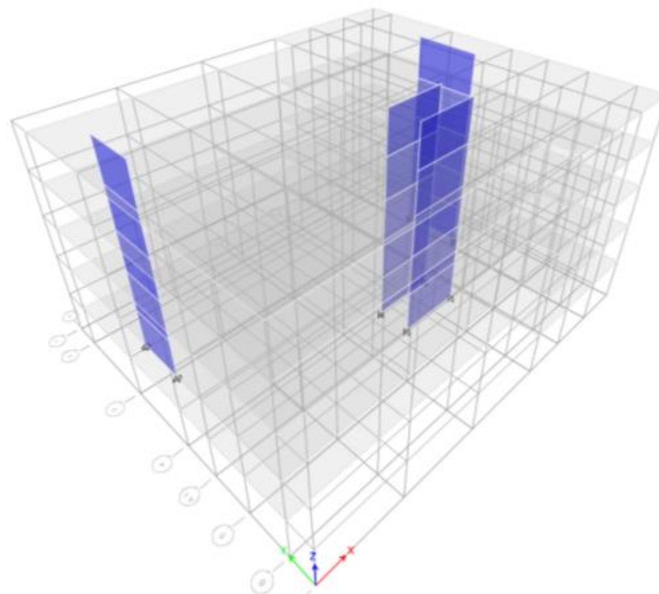
Non-Sway

Slenderness effects were ignored in the design of both interior and exterior columns. Slenderness effects are permitted to be neglected if the design meets the criteria in ACI 318-11 section 10.10.1. This section has criteria dependent on whether or not members are braced against sidesway. It also shall be permitted to assume a story within a structure is nonsway if the section 10.10.5.2 is met. The column design for The Primary Health Networks new Medical Office Building in Sharon, PA met all requirements permitting slenderness effects to be neglected. All relevant calculations can be found in Appendix B.

Shear Wall Design

ETABS® 2013
Integrated Building Design Software

The original design featured Ivany block masonry shear walls that also acted as masonry bearing walls. As previously mentioned the redesign intends to retain the original plan layout within reason and as such will keep the original shear wall locations. The redesigned shear walls will also function as concrete bearing walls in order to retain the original number of columns. The proposed solution intended to compare the efficiency of the existing masonry shear walls to the redesigned concrete shear walls. This is not feasible due to an increase in the concrete compressive strength, and more importantly a drastic increase in overall building weight due to the change from steel to concrete. All concrete shear walls were modeled using ETABS 2013. Discrepancies between hand calculated seismic loads and loads from ETABS stem from the building period. Hand calculations used the approximate period where as ETABS calculated the exact period which led to a difference in the Seismic Response Coefficient (Cs). The exact period calculated by ETABS was used in design.



Design & Modeling Assumptions:

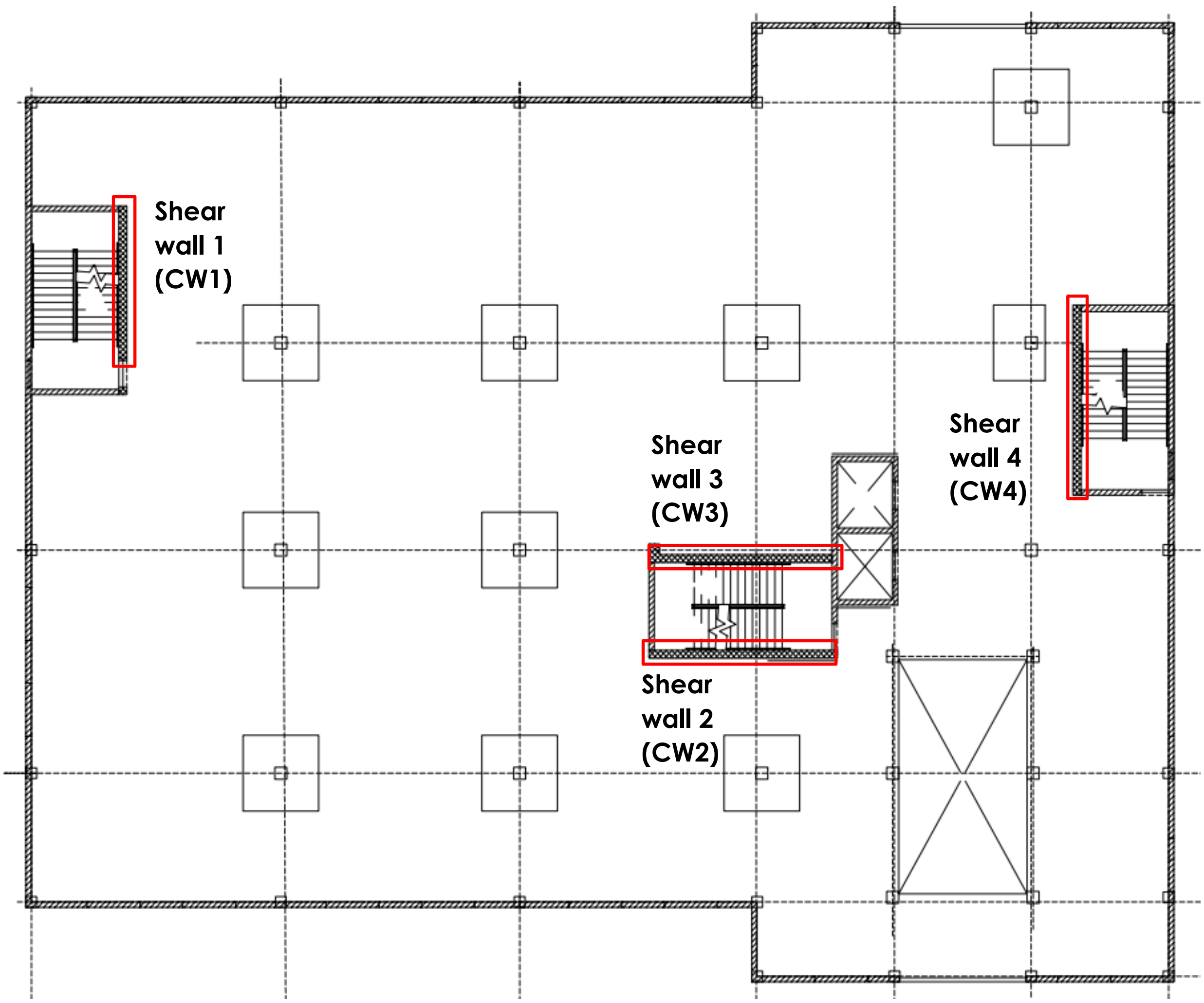
- Cracked sections
- Thin shells
- 12" thick
- Fully fixed at base level

Due to the increased building weight seismic loads controlled the lateral design. Seismic loads were calculated in ETABS per ASCE 7-10 and can be found in Appendix B. Each floor was modeled as a diaphragm having zero mass. The building mass at each floor level was then added as a point mass at the floor levels center of mass. The buildings seismic loads were calculated by hand using ASCE 7-10 and compared to the loads determined using ETABS to verify the model, all relevant seismic calculations can be found in Appendix B.

Shear Wall Reinforcement Design

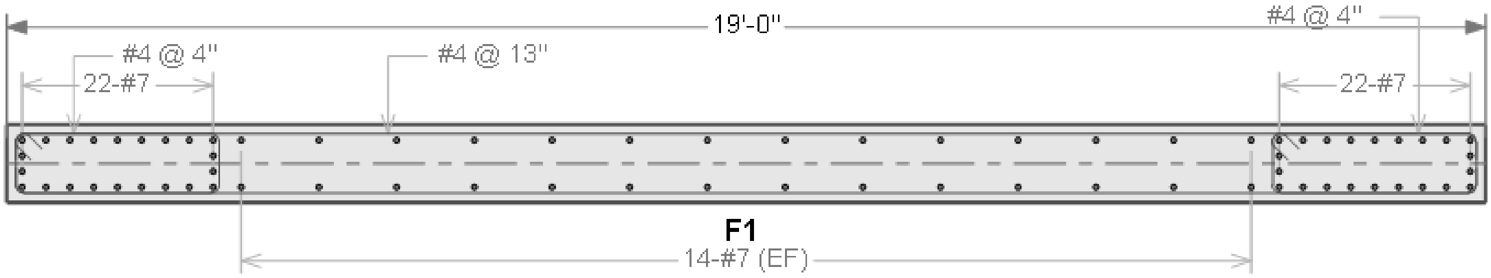
The reinforcement layout was designed using the simplified C&T method in ETABS. Figure 35 below shows the shear wall layout.

Figure 35 – Shear wall locations



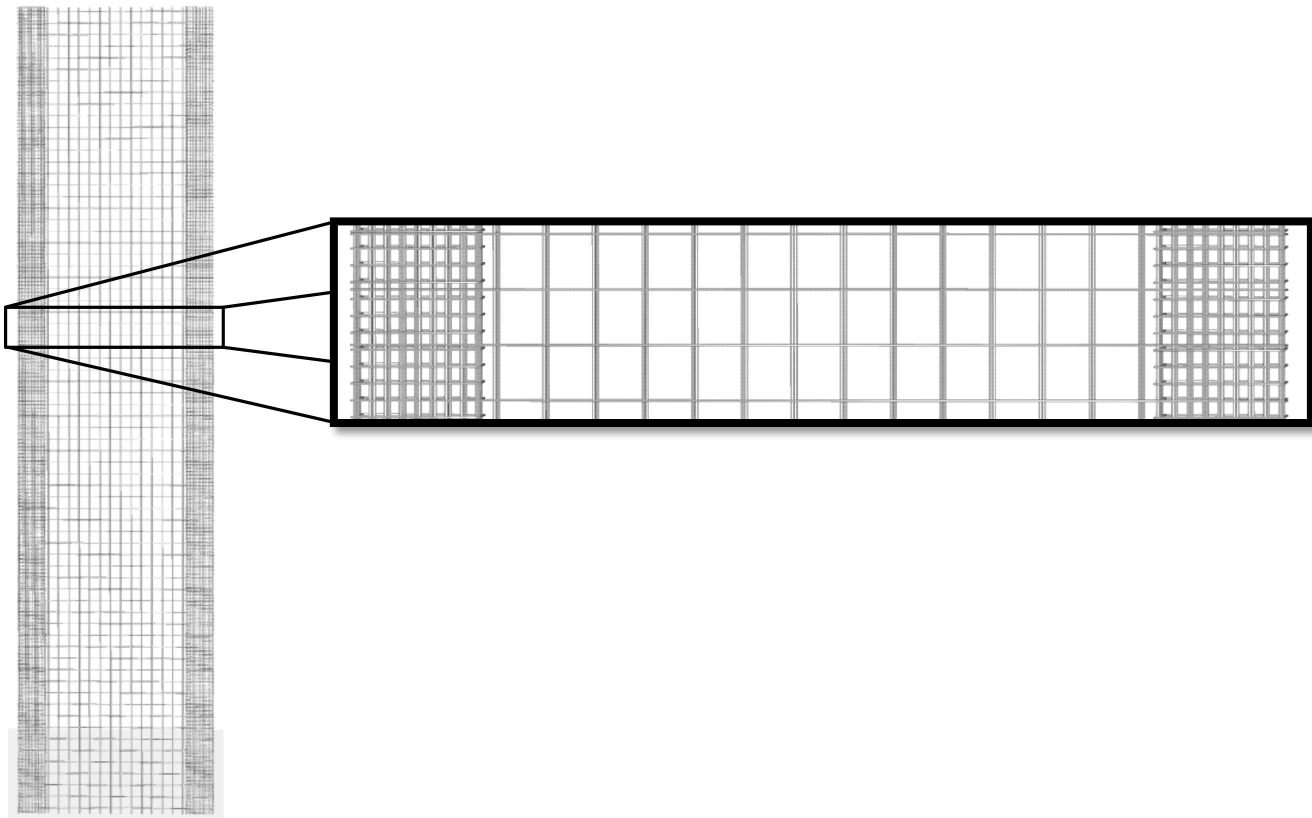
Shear Wall 1 (CW1)

Figure 36 – CW1 reinforcement layout



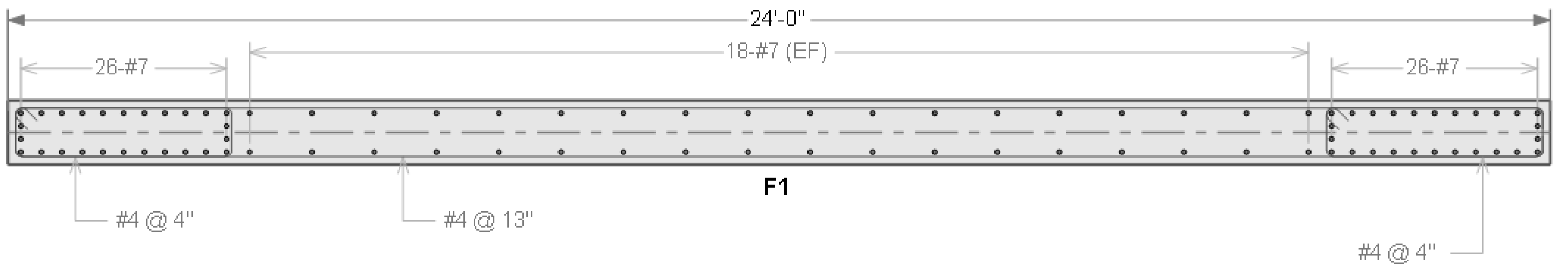
As shown in figure 36 above shear wall 1 has horizontal reinforcing of #4 bars at 13" on center coupled with a grid of 14 #7 bars vertical throughout. The main flexural reinforcing consists of 22 # 7 bars each side tied with #4 bars at 4" on center. This layout is consistent throughout the full wall height as shown in Figure 37 below.

Figure 37 – CWI Elevation and reinforcement detail



Shear Wall 2 (CW2)

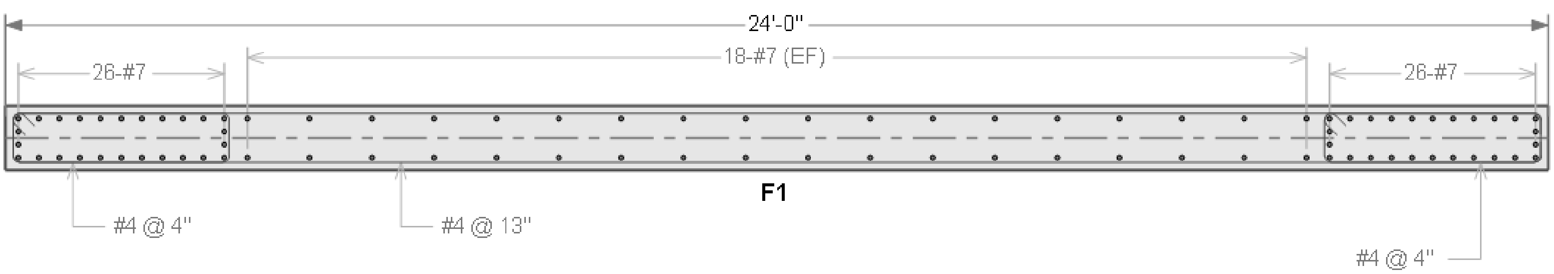
Figure 38 – CW2 reinforcement layout



As shown in figure 38 above shear wall 2 has horizontal reinforcing of #4 bars at 13" on center coupled with a grid of 18 #7 bars vertical throughout. The main flexural reinforcing consists of 26 # 7 bars each side tied with #4 bars at 4" on center.

Shear Wall 3 (CW3)

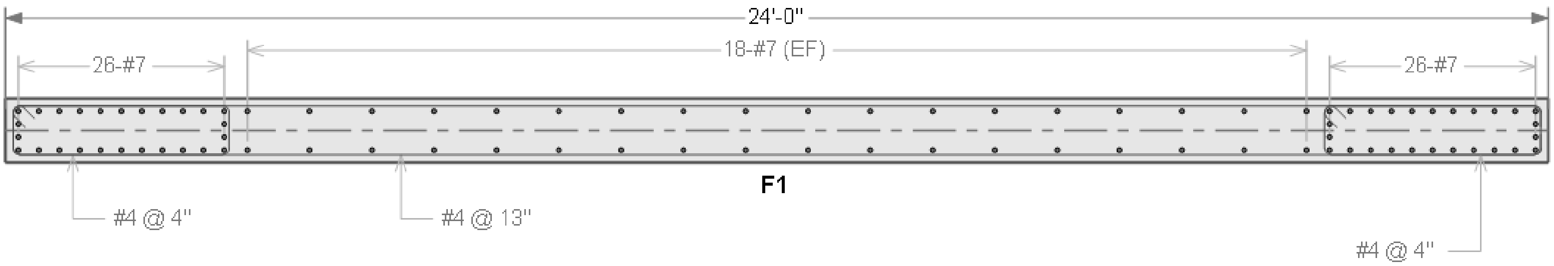
Figure 39 – CW3 reinforcement layout



As shown in figure 39 above shear wall 3 has horizontal reinforcing of #4 bars at 13" on center coupled with a grid of 18 #7 bars vertical throughout. The main flexural reinforcing consists of 26 # 7 bars each side tied with #4 bars at 4" on center.

Shear Wall 4 (CW4)

Figure 40 – CW4 reinforcement layout



As shown in figure 40 above shear wall 4 has horizontal reinforcing of #4 bars at 13" on center coupled with a grid of 18 #7 bars vertical throughout. The main flexural reinforcing consists of 26 # 7 bars each side tied with #4 bars at 4" on center.

P-Delta effects

P-delta effects on stories are not required to be considered where the stability coefficient as determined by ASCE 7-10 12.8-16 is less than 0.10.

$$\theta = \frac{P_x \Delta I_e}{V_x h_{sx} C_d} \qquad (12.8\text{-}16)$$

To determine if p-delta calculations were necessary equation 12.8-16 was applied to the worst case location, story 1.

$$\theta = \frac{10427 * 0.000611 * 1}{315.337 * 15.5 * 4}$$

Θ = 0.00033 < 0.10

Permitted to neglect P-Delta Effects

Structural Summary

The redesign consists of 10" two way slabs with drop panels and edge beams. Drop panels are typically 18" thick and 9' square. Edge beams are 18" wide and 18" deep. The slabs were modeled using spSlab; columns were modeled using spColumn. All columns are 18" square. All concrete has a compressive strength of 4000 psi. The lateral system redesign of concrete shear walls was modeled using ETABS 2013 and kept the geometry of the existing lateral system.

The structural redesign meets all requirements for strength and serviceability. The overall structural depth was reduced from an average of 30" to 18", a reduction of 40%.

Architecture Breadth

Figure 41 – Existing Façade

Source: sharonherald.com

The Background

The new medical office building for The Primary Health network will be the first commercial construction project in Sharon since 1969. The project, as rendered in Figure 41, is intended to help revitalize the town and a major challenge will be bringing modern architecture that also acknowledges the surrounding buildings. Downtown Sharon is dominated by brick facades with glazed storefronts and as such it was necessary to find a more modern material that could also compliment the surrounding architecture. A number of materials were considered, including brick, concrete, glazing systems, synthetics and terra cotta. The materials were compared to typical buildings in downtown Sharon and it was determined that a combination of terra cotta panels and glass curtain wall would best compliment the surrounding buildings while breathing fresh life into the area.

Figure 42 – Tsinghua Law Library

www.archdaily.com

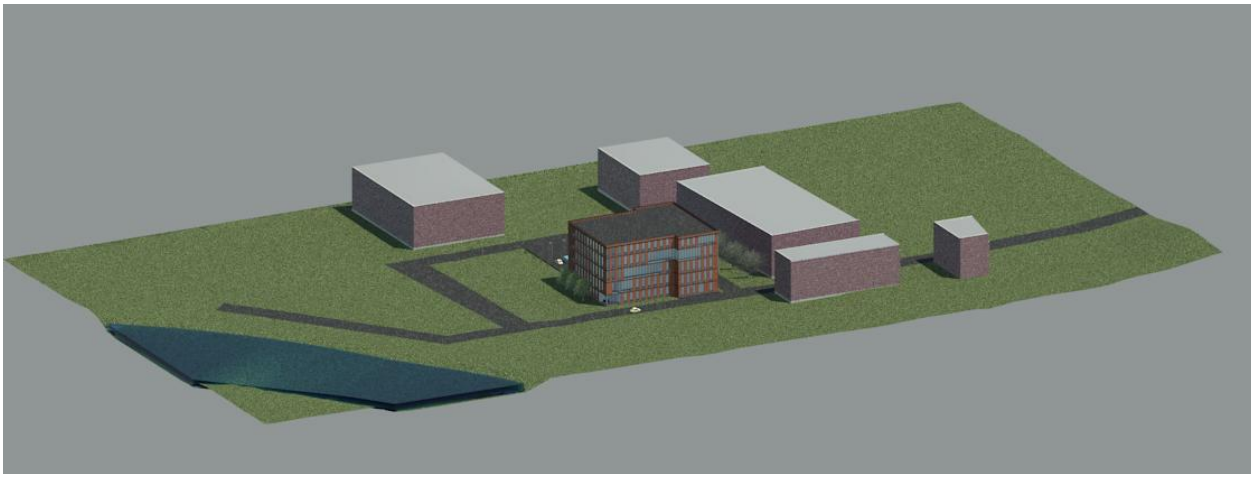
Figure 43 – Diana Center at Barnard College

www.flickr.com

The Inspiration

The Tsinghua Law Library and the Diana Center at Barnard College (Figures 42 & 43 above) provided inspiration for the redesign. Both buildings have a strong fundamental concept of the mixing solid and void. This concept inspires wonder as the buildings appear to not be structurally sound. The mixing of solid and void also stands to represent the mixing of new and old in the city of Sharon. The building will be recognized regardless of situation in the small town since it's the first new construction in 46 years; as such to not acknowledge the vast gap would create an unsettling atmosphere. The gap in construction, in architectural advance, is represented by the void and is being encompassed by the modern materials and shapes represented by the solid.

Figure 44 – Site Model

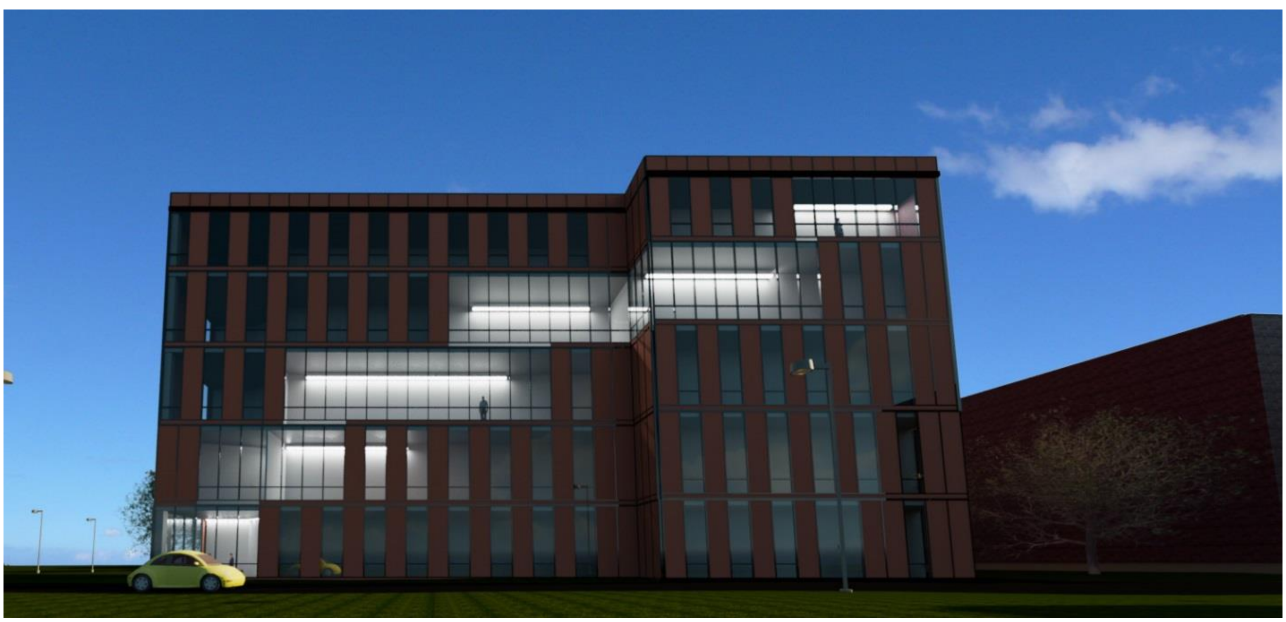


The Process

Revit 2015 was chosen as the modeling software due to its flexibility in design and ability to integrate with other programs. The building site was modeled between W State Street and E Silver Street from the Shenango River to N Railroad Street. The sites topography was brought into Revit from Google Earth to accurately represent the contours of the area. The buildings main faces in terms of entrance and sight lines are looking to the north and east respectively, as such all buildings large enough to be seen from the locations previously mentioned were modeled generically as blocks having brick facades. Streets, parking lots, and landscapes within these views were also modeled to give a more realistic feel. The full building site model can be seen in Figure 44 Above.

Figure 45 – North East View

Figure 46 – North View



The Model

The buildings main architectural components are terra cotta panels coupled with a glass curtain wall. The vertical strips are intended to increase the buildings perceived height in an attempt to inspire ambition. The main architectural feature is the diagonal glass strip that steps up each floor starting at the bottom west corner of the south façade climaxing at the top east corner. The strip when lit at night creates the illusion of the void discussed previously, allowing for the remaining solid sections to create the illusion of enclosure. The increased glass area allows for more daylight into the spaces as well as giving the building an overall lighter appearance as compared to the original façade.

Construction Management Breadth

Background

The new medical office building for The Primary Health Network was a project driven by cost. The budget for the project was small and tight, as such efficiency was paramount in every aspect of design. To determine if the redesign is truly feasible a cost comparison between the changes in structural system as well as façade must be accounted for. Furthermore the change from steel to concrete could drastically effect the construction timeline. The existing lateral system of masonry shear walls is fully grouted and has the same dimensions as the redesigned concrete shear wall system, as such the cost difference between the systems can be considered negligible and was not included in the cost comparison.

Cost Estimate

RS Means 2015 Facilities Cost Data was used to estimate the cost of the new structural system. The components included in the estimate were all concrete slabs, beams, columns, rebar, finishing, placement, formwork, and concrete material. The estimated costs of the slabs was taken from section 03-30 1950 for elevated slabs with 30' spans having a load of 125 psf. This line item includes formwork with an average of four uses, grade 60 rebar, Portland cement type 1, placement and finishing of the slabs. The line item for columns was found by linearly interpolating between references 03-30 0820 and 03-30 0920, columns 16"x16" and 24"x24" respectively, to obtain values for 18"x18" columns with average reinforcing between 2-3%. The line item for beams was taken from section 03-30 0350.

Component	crew	Unit	total including o&p	Total (Unit)	Location Modifier	Expected Cost
Elevated slab[2]	c-14b	C.Y.	635	2515	0.889	$1,419,755.23
Columns (18x18)	c-14a	C.Y.	1712.5	252	0.889	$383,647.95
beam	c-14a	C.Y.	1250	116	0.889	$128,905.00
Total						$1,932,308.18

The location multiplier was also taken from RS Means Facility Cost Data 2015 for New Castle Pa, the closest listed location. The cost estimate for the existing building was obtained from John N Gruitza associates and can be found in Appendix D. The line items relevant to the steel structure were taken from the estimate and can be seen below.

Steel System		
Component	Line #	Value
Structural Steel	25	$1,029,286
Misc. Steel	26	$192,500
Interior Columns	62	$23,259
Exterior Columns	63	$64,726
Structural Studs	49	$408,700
Total		$1,718,471

Structural Cost Comparison

Figure 47 - Structural Cost Comparison

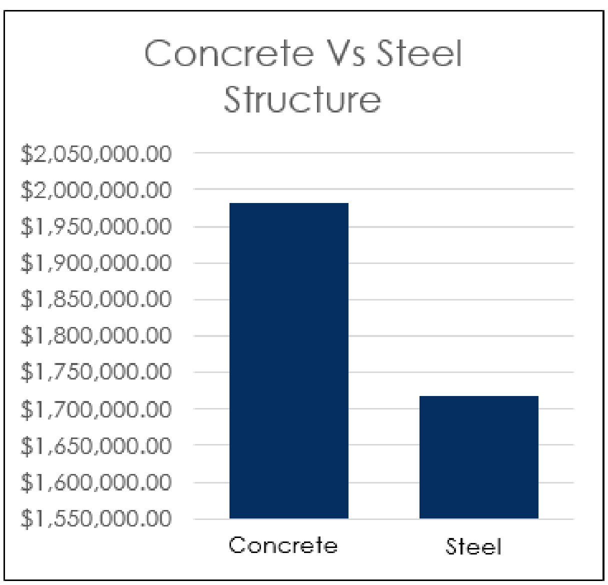


Figure 47 shows the relative costs of each structural system. It was determined that the change to a concrete structure would result in an approximate increase of 12.44% in building cost for structural systems.

Architectural Cost Comparison

RS Means Facility Cost Data 2015 was also used to determine the cost of the new architectural façade. The façade components included a terra cotta panel system reference 04-21 0750 and an architectural glazing system reference 08-44 0150. These line items include all required fasteners and labor costs.

Façade						
Component	crew	Unit	total including o&p	Total (Unit)	Location Modifier	Expected Cost
Terra Cotta	d-8	s.f.	11.9	16419.6	0.889	$195,393.24
Glazing	h-1	s.f.	72.5	24629.808	0.889	$1,785,661.08
Total						$1,981,054.32

The line items relevant to the architectural façade were taken from the existing cost estimate and can be seen below.

Façade		
Component	Line #	Value
EFIS	38	$161,129
Brick	40	$307,987
Windows	43	$401,500
Total		$870,616

Façade Cost Comparison

Figure 48 – Façade Cost Comparison

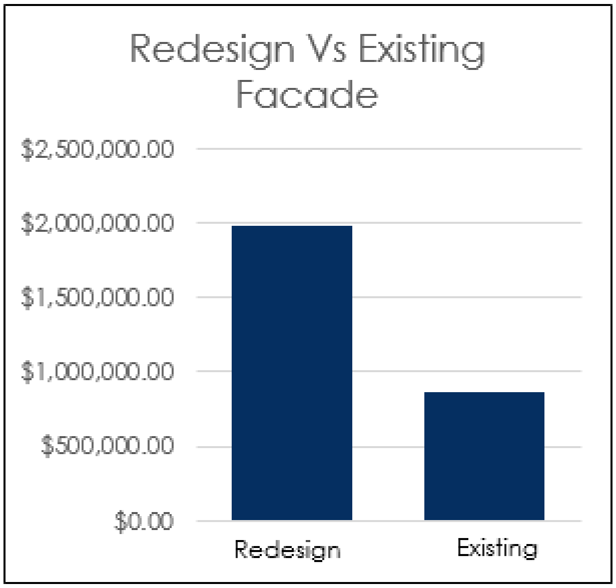


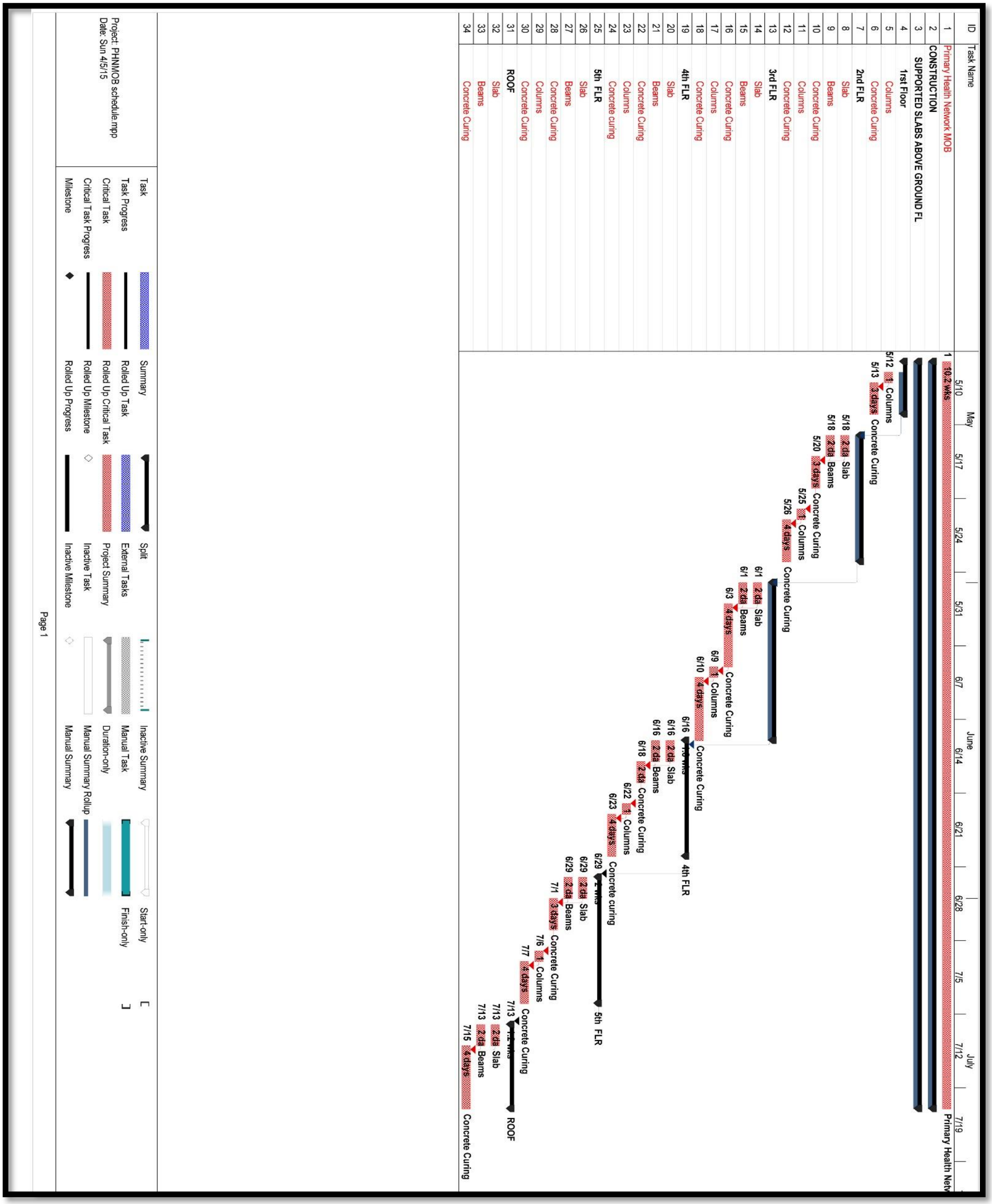
Figure 48 shows the relative costs of each façade system. It was determined that the architectural redesign would result in an approximate increase of 127.54% in building cost for façade systems.

Summary

The overall change in building costs from the redesigns would result in an increase of $1,324,275. Due to the cost sensitive nature of the project this increase is unacceptable. The structural redesign resulted in a minor increase to the budget and is considered a feasible option. The architectural changes to the project resulted in a comparatively large increase to the budget and as such cannot be considered a feasible redesign.

Construction Schedule

A construction schedule for the redesigned concrete structure was created using Microsoft Project 2013. The schedule includes all structural items listed previously in the cost estimate. Durations were calculated based on the daily output from the recommended crews for each section. The crew recommended for concrete line item slabswas "c-14b." The crew recommended for both columns and beams was "c-14a." Crew c-14b consists of 1 Carpenter Foreman (outside), 16 Carpenters, 4 Rodmen, 2 Laborers, 2 Cement Finishers, 1 Equipment Operator (Medium sized), 1 Gas Engine Vibrator, and 1 Concrete Pump (Small). Crew c-14a has one less Cement Finisher than Crew c-14b. Due to the large size of the recommended crew, only one crew was implemented for each task. The total duration for each item was then broken down into a per floor basis. The edge beams should be poured integrally with the slab, as such slab and beam durations were considered as one duration even though they are listed as individual line items. The duration for slab + beams for each floor is just under two days, the duration for columns is only one day. The concrete will need a minimum of four days curing time before it can support work on the next floor level; as such concrete curing was added into the schedule so that there is a minimum of four days between the pours of respective elements. The existing projects construction schedule could not be attained, as such no comparison between schedules can be made; because of this the project start date was set to May 11, 2015 to provide optimal conditions for concrete curing. This eliminates the need to heat or protect the concrete during the curing process. Because the schedule only incorporates the concrete structure the critical path follows the construction schedule exactly.



ID
Task Name
Primary Health Network MOB
CONSTRUCTION
SUPPORTED SLABS ABOVE GROUND FL
1st Floor
Columns
Concrete Curing
2nd FLR
Slab
Beams
Concrete Curing
Columns
Concrete Curing
3rd FLR
Slab
Beams
Concrete Curing
Columns
Concrete Curing
4th FLR
Slab
Beams
Concrete Curing
Columns
Concrete curing
5th FLR
Slab
Beams
Concrete Curing
Columns
Concrete Curing
ROOF
Slab
Beams
Concrete Curing
Project: PHNMOB schedule.mpp
Date: Sun 4/5/15
Task
Task Progress
Critical Task
Critical Task Progress
Milestone
Summary
Rolled Up Task
Rolled Up Critical Task
Rolled Up Milestone
Rolled Up Progress
Split
External Tasks
Project Summary
Inactive Task
Inactive Milestone
Inactive Summary
Manual Task
Duration-only
Manual Summary Rollup
Manual Summary
Start-only
Finish-only
Page 1

Conclusion

The report contains an overview of the building site, size, architecture and structure in the first portion. An alternate solution to the structural framing of the building is offered and then explored in detail. A two way flat slab with drop panels and edge beams was designed for strength and serviceability requirements using spSlab and verified with hand calculations. These slabs are supported by concrete columns modeled in spColumn and verified with hand calculations.

The existing lateral system consists of Ivany Block masonry shear walls which were redesigned as concrete shear walls. The lateral system was modeled using ETABS 2013. The redesign focused heavily on keeping the original column layout with marked exceptions. The change to a concrete system resulted in drastically increased lateral loads due to seismic, these loads were calculated by ETABS and verified by hand.

The structural redesign met all requirements for strength and serviceability while also reducing the overall structural depth by 40%.

Sharon, Pa hasn't had a commercial construction project since 1969. This gap in construction results in an even more pronounced gap in architecture. The new medical office building has to be modern enough to breathe new life into the city while acknowledging the surrounding buildings in order to mesh well with the community. The building's façade was redesigned in order to better accomplish these goals. The building and site were modeled using Revit 2015.

The Primary Health Network had a very tight budget for this project; efficiency played a leading role in all aspects of design. The change in building structure as well as the change in building façade result in an equivalent change in building cost which must be accounted for to determine the feasibility of the redesign. The structural redesign resulted in a 12.44% increase in building cost, while the façade redesign resulted in a 127.54% increase.

A building construction schedule was created for the redesigned structural system only using Microsoft Project by referencing the information found in RS Means Facility Cost Data 2015.

The structural redesign reduced the overall structural depth with only minimal impact on cost; therefore it is a feasible design. The change in façade resulted in a drastic increase in cost and therefore is not a feasible design with the buildings current budget.

Appendix A

Center of Rigidity Calculations

Determine Center of Rigidity

All shear walls will be treated as cantilevers

$G_{masonry} \approx 0.4E \qquad E_m = 900(3{,}000) = 2700 \text{ ksi}$

Determine Shear wall Stiffness

$$k = \frac{E}{4\left(\frac{h}{b}\right)^3 + 3\left(\frac{h}{b}\right)}$$

$$k_1 = \frac{2700}{4\left(\frac{960}{228}\right)^3 + 3\left(\frac{960}{228}\right)} = 8.67 \text{ k/in}$$

$$k_2 = \frac{2700}{4\left(\frac{960}{288}\right)^3 + 3\left(\frac{960}{288}\right)} = 17.07 \text{ k/in}$$

$$k_3 = k_4 = k_2 = 17.07 \text{ k/in}$$

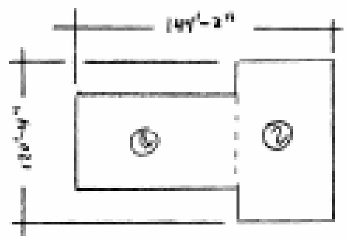
Center of Rigidity

Element	effective Direction	Dist from ref. Datum x	Dist from ref. Datum y	Rx	Ry	RxY	RyX
k_1	Y	11.5 ft	—	0	8.67	—	99.7
k_2	X	—	51.5 ft	17.07	0	879.1	—
k_3	X	—	39.5 ft	17.07	0	674.3	—
k_4	Y	129.5 ft	—	0	17.07	—	2210.6
Sum				34.15	25.74	1553.4	2310.3

$$\boxed{X_r = \frac{2310.3}{25.74} = 89.8 \text{ ft}} \qquad \boxed{Y_r = \frac{1553.4}{34.15} = 45.5 \text{ ft}}$$

Center of Mass

Assuming mass is uniformly distributed throughout the building, and assigning an arbitrary value of 1 psf to the structure the C.O.M. was calculated as follows



$$\bar{y} = \frac{120.167}{2} = 60.1'$$

$$\bar{x} = \frac{(9287)(45.75') + (6386)(118')}{(9287 + 6386)}$$

$$\bar{x} = 75.2'$$

Eccentricity

$$e = C.O.R - C.O.M$$

$$e_x = 89.8' - 75.2' = \underline{14.6'}$$

$$e_y = 45.5' - 60.1' = \underline{14.6'}$$

Torsional Rigidity (J)

$$J = \Sigma R_i d_i^2$$

$$J = (8.67)(78.3)^2 + (17.07)(6^2) + 17.07(6)^2 + 17.07(37.7)^2$$

$$= 81,288 \ \tfrac{k}{in} \ ft^2$$

Wind Loads

PENNSTATE Engineering 1855

CLASS: SECTION:
SHEET NO: OF
DESIGNED BY: DATE:
JOB NAME:

Wind Analysis

ASCE 7-10 Chapter 27

1.) risk category II Table 1.4-1

2) $V_u = 115$ mpH Figure 26.5-1A
$I = 1.0$

3) $K_d = 0.85$ Table 26.6-1

exposure B Section 26.7

$K_{zt} = 1.0$ Table 26.8-1

Gust effect, $G = 0.85$ Section 26.9

Enclosed Structure Section 26.10

$GC_{pi} = +/- 0.18$ Table 26.11-1

4) $K(15') = 0.57$ Table 27.3-1

$K(30') = 0.70$

$K(45') = 0.79$

$K(60') = 0.85$

$K(75') = 0.91$

$K(85') = 0.95$

Building natural frequency will be determined using eq 26.9-4 $n_a = \frac{75}{75'} = 1 \rightarrow$ Rigid

PENNSTATE Engineering

CLASS: SECTION:
SHEET NO: OF
DESIGNED BY: DATE:
JOB NAME:

5) Q_z or Q_h $\quad q_h = 0.00256(k_z)(k_{zt})(k_d)(V^2)$ $\quad$ EQ 27.3-1

$k_{zt} = 1.0$

$q_{15'} = 0.00256(0.85)(0.57)(115psf)^2 = 16.4psf$

$q_{30'} = 0.00256(0.85)(0.70)(115psf)^2 = 20.14psf$

$q_{45'} = 0.00256(0.85)(0.79)(115psf)^2 = 22.73psf$

$q_{60'} = 0.00256(0.85)(0.85)(115psf)^2 = 24.46psf$

$q_{75'} = 0.00256(0.85)(0.91)(115psf)^2 = 26.19psf$

$q_{85'} = 0.00256(0.85)(0.95)(115psf)^2 = 27.34psf$

6) $C_p = 0.8$ for windward walls $\quad$ Figure 27.4-1

$C_p = -0.5$ for leeward walls

$\hookrightarrow L/B = \frac{144'}{120'} = 1.2 \quad C_p = -.46$ by interpolation.
use -0.5 for all directions

Roof

$q = .00256(0.93)(0.85)(115)^2 = 26.7psf$

$p = 26.7[-0.9-0.18] = 28.8psf$

net $= 0.7w - 0.6D = 20.16 - 4.8 = 15.36psf$

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DESIGNED BY: DATE:
JOB NAME:

7) $+GC_{pi}$ Case

@ H = 15' 27.4-1

$P_{windward} = (16.4\,psf)(0.85)(0.8) - (27.3)(0.18) = 6.24\,psf$

$P_{leeward} = (27.3)(0.85)(-0.5) - (27.3)(0.18) = -16.52\,psf$

$\Sigma P = 22.76\,psf$

@ H = 30'

$P_{windward} = (20.1\,psf)(0.85)(0.8) - (27.3)(0.18) = 8.75\,psf$

$P_{leeward} = (27.3)(0.85)(-0.5) - (27.3)(0.18) = -16.52\,psf$

$\Sigma P = 25.27\,psf$

@ H = 45'

$P_{windward} = (22.7\,psf)(0.85)(0.8) - (27.3)(0.18) = 10.52\,psf$

$P_{leeward} = (27.3)(0.85)(-0.5) - (27.3)(0.18) = -16.52\,psf$

$\Sigma P = 27.04\,psf$

@ H = 60'

$P_{windward} = (24.5)(0.85)(0.8) - (27.3)(0.18) = 11.75\,psf$

$P_{leeward} = (27.3)(0.85)(-0.5) - (27.3)(0.18) = -16.52\,psf$

$\Sigma P = 28.27\,psf$

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OF
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@ H = 75'

$P_{windward} = (26.2\,psf)(0.85)(0.8) - (27.3)(0.18) = 12.90\,psf$

$P_{leeward} = (27.3\,psf)(0.85)(-0.5) - (27.3)(0.18) = -16.52\,psf$

$\Sigma P = 29.42\,psf$

@ H = 85'

$P_{windward} = (27.3\,psf)(0.85)(0.8) - (27.3)(0.18) = 13.65\,psf$

$P_{leeward} = (27.3\,psf)(0.85)(-0.5) - (27.3)(0.18) = -16.52\,psf$

$\Sigma P = 30.17\,psf$

$-GC_{pi}$ Case

@ H = 15'

$P_{windward} = (16.4\,psf)(0.85)(0.8) + (27.3)(0.18) = 16.07\,psf$

$P_{leeward} = (27.3)(0.85)(-0.5) + (27.3)(0.18) = -6.69\,psf$

$\Sigma P = 22.75\,psf$

@ H = 30'

$P_{windward} = (20.1\,psf)(0.85)(0.8) + (27.3)(0.18) = 18.58\,psf$

$P_{leeward} = (27.3)(0.85)(-0.5) + (27.3)(0.18) = -6.69\,psf$

$\Sigma P = 25.27\,psf$

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JOB NAME:

@ H=45'

$P_{windward} = (22.7psf)(0.85)(0.8) + (27.3)(0.18) = 20.35 \text{ psf}$

$P_{leeward} = (27.3)(0.85)(-0.5) + (27.3)(0.18) = -6.69 \text{ psf}$

$\Sigma P = 27.04 \text{ psf}$

@ H=60'

$P_{windward} = (24.5psf)(0.85)(0.8) + (27.3)(0.18) = 21.57 psf$

$P_{leeward} = (27.3)(0.85)(-0.5) + (27.3)(0.18) = -6.69 psf$

$\Sigma P = 28.26 psf$

@ H=75'

$P_{windward} = (26.2psf)(0.85)(0.8) + (27.3)(0.18) = 22.73 psf$

$P_{leeward} = (27.3)(0.85)(-0.5) + (27.3)(0.18) = -6.69 psf$

$\Sigma P = 29.42 psf$

@ H=85'

$P_{windward} = (27.3psf)(0.85)(0.8) + (27.3)(0.18) = 23.48 psf$

$P_{leeward} = (27.3psf)(0.85)(-0.5) + (27.3)(0.18) = -6.69 psf$

$\Sigma P = 30.17 psf$

Seismic Loads

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Seismic Analysis

Risk Category II — Section 11.6

Seismic Importance Factor = 1.0

Seismic Site Class – assume C

$S_s = 0.170$ — Figure 22-1

$S_1 = 0.055$ — Figure 22-2

$F_a = 1.6$ — Table 11.4-1

$F_v = 2.4$ — Table 11.4-2

$S_{DS} = F_a S_s (2/3) = 0.181$ — 11.4-3

$S_{D1} = F_v S_1 (2/3) = 0.088$ — 11.4-4

Seismic Design Category B — Tables 11.6-1, 11.6-2

The building has Intermediate reinforced Masonry Shear walls

$R = 4.0$ — Table 12.2-1

$T_a = 0.02(75)^{0.75} = 0.51$ — 12.8-7

$C_t = 0.02$ $x = 0.75$ — Table 12.8-2

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JOB NAME:

$$C_s = \frac{0.181}{\left(\frac{4.0}{1.0}\right)} = 0.045 \qquad 12.8\text{-}2$$

$$C_{s\,max} = \frac{0.088}{0.51\left(\frac{4.0}{1.0}\right)} = 0.043 \qquad 12.8\text{-}3 \quad \checkmark$$

$$C_{s\,min} = 0.044(0.181)(1.0) = 0.008 \qquad 12.8\text{-}5 \quad \checkmark$$

$$C_s = 0.045$$

Seismic weight w

Roof = 347^k from tech II

floor Dead load = 55psf

flooring = 2psf
Slab-on-Deck = 35psf
Steel = 10psf
MEP = 8psf

$$= \frac{(55)(120.37)(144.147)(4)}{1000} = 3806^k$$

Exterior wall load

$$\frac{\overset{w}{(10psf)}\overset{h}{(79')}\overset{L}{(529')}}{1000} = 418^k$$

Shear wall loads

$$\frac{\overset{w}{(133psf)}\overset{H}{(79')}\overset{L}{(91')}}{1000} = 956^k$$

Total Seismic weight, w = 5527^k

Vertical Distribution

$$V = C_s W = (0.045)(5527^k) = 248.7^k$$

$$C_{vx} = \left[\frac{W_x h_x^k}{\Sigma W_x h_x^k}\right] V \qquad K = 1 \text{ from tech II}$$

	W_i	h_x	$W_i h_x$	C_{vx}
Floor 2	1295^k	15'-2"	19,640	0.088
Floor 3	1295^k	30'-4"	39,282	0.18
Floor 4	1295^k	45'-6"	58,923	0.26
Floor 5	1295^k	60'-8"	78,563	0.35
Roof	347^k	75'	26,025'	0.12
Sum			222,433	

Story Forces

Floor 2 = 21.9^k

Floor 3 = 44.77^k

Floor 4 = 64.66^k

Floor 5 = 87.05^k

Roof = 29.84^k

Appendix B

Slenderness Effects

Determine if story is non-sway (Story One)

eq 10-10 from ACI 318-11

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

From Etabs

$\Sigma P_u = 791^k$ $\Delta_o = 0.117$ in

$V_{us} = 315^k$ $l_c = 15.5' - 18/12 = 14'$

$$Q = \frac{791(0.117)}{315(14')} = 0.021 \leq 0.05 \checkmark$$

Therefore use equation (10-7) section 10.10.1

$$\frac{k l_u}{r} \leq 34 - 12(M_1/M_2) \leq 40$$

$k = 1.0$ pinned $l_u = 15.5 - (18/12) = 14'$

$r = 0.3(18/12) = 0.45$ per 10.10.1.2

$$\frac{k l_u}{r} = \frac{1(14)}{0.45} = 31.1 \leq 34 \checkmark$$

Slenderness effects can be neglected

Corrections to Seismic Analysis

Seismic Loads

Calculate Seismic weight W

Floors

Slab = $\frac{10}{12}(150)[(91.5' \times 101.5) + (53' \times 120')]$

$= 1956^k$

Drop Panels = $\frac{8}{12}(150)(9^2)(10 \text{ drop panels})$

$= 81^k$

Edge Beams = $\frac{8}{12}(150)(\frac{18}{12})(91' + 9.5' + 53' + 120' + 57' + 9.5' + 91' + 101')$

$= 79.2$

total = $2116.2(5) = 10{,}581^k$

Columns = $(\frac{18}{12})^2(150)(77.5')(38) = 994^k$

Curtain wall = $(15psf)(77.5)(528 ft) = 613.8^k$

$W = 12{,}189^k$

Base Shear

$V = C_s W = 0.043(12{,}189^k) = 524^{kips}$

Vertical Distribution

$$C_{vx} = \left[\frac{w_x h_x^k}{\Sigma w_x h_x^k}\right] V \qquad k = 1 \text{ from tech II}$$

	W_i (k)	h_x (ft)	$w_i h_x$	C_{vx}
Floor 2	2636.36	15'-6"	40,858	0.074
Floor 3	2438	31'-0"	75,578	0.136
Floor 4	2438	46'-6"	113,367	0.205
Floor 5	2438	62'-0"	151,156	0.273
Roof	2239	77'-6"	173504	0.312
Total			554,463	

Story Forces

Floor 2 = 38.7^k

Floor 3 = 71.3^k

Floor 4 = 107k

Floor 5 = 143^k

Roof = 163^k

Column Load Takedowns

Column Load Take down (A2) (Exterior corner)

Influence Area = (30'-9")(29-7") = $\frac{910 \text{ sq ft}}{\text{floor}}$

= 910 (4) = 3638.25 sq ft

Live Load reduction

$LL = \left[0.25 + \sqrt{\frac{15}{3638}}\right] = 0.31 \Rightarrow$ use 0.40

Estimate Dead Loads

Slab load = $(10/12)(150)(\frac{910}{4})(5) = 142^{kip}$

Beam load = $(18/12)(30.1)(1.5)(5)(150) = 16.2^{k}$

Column self wt = 26k from previous Calc

Superimposed Dead load = $(20)(\frac{910}{4})(4) = 18.2^{k}$

exterior wall load = $(15)(77.5)(30.1) = 35^{k}$

237.4^{k}

Estimate Live loads

$(0.4)(80)(\frac{910}{4})(4) = 29.1^{k}$

Estimate Snow Loads

$(0.7)(40)(\frac{910}{4}) = 6.37^{k}$

Column Load Takedown (C2) (Generic Exterior)

Influence Area $= (30 + 30'\text{-}5")(29'\text{-}7") = 1787 \frac{\text{sq ft}}{\text{Floor}}$

$= 1787 (4 \text{ floors}) = 7149 \text{ sq ft}$

Live Load Reduction

$$LL = \left[0.25 + \sqrt{\frac{15}{7149}}\right] = 0.29 \Rightarrow \text{use } 0.40$$

Estimate Dead Loads

Slab load $= (10/12)(150)\left(\frac{1787}{4}\right)(5) = 279$ Kips

Beam load $= (8/12)(30)(1.5)(5)(150) = 22.5^k$

Column self wt $= 26$ kip from previous calcs

Super imposed dead $= (20)\left(\frac{1787}{4}\right)(4) = 35.7^k$

exterior wall Load $= (15)(77.5)(30.2) = 35.1^k$

403.3 kip

Estimate Live Loads

$(0.4)(80)\left(\frac{1787}{4}\right)(4) = 57.2$ kips

Estimate Snow Loads

$(0.7)(40)\left(\frac{1787}{4}\right) = 12.5$ kips

Column Load Takedown (E2) (Interior Biaxial)

Influence Area = (33'-10" + 16'-8")(24'-7" + 9'-4") = 1965 sq ft / floor

= 1965 (4 floors) = 7861 sq ft

Live load reduction

$$LL = \left[0.25 + \sqrt{\frac{15}{7861}}\right] = 0.29 \Rightarrow \text{use } 0.4$$

Estimate total Dead Load

Slab load = (10/12)(150)(1965/4)(5 floors) = 307 kips

drop panel load = (8/12)(150)(9.2)(5) = 40.5 kips

Column self weight = (1.5²)(150)(225) = 76 kip

Superimposed dead load = 20(1965/4)(5) = 39 kip

412.625 kip

Estimate total Live Load

$$(0.4)(80)\left(\frac{1965}{4}\right)(4) = 62.88 \text{ kip}$$

Estimate Snow load

$$(0.7)(40)\left(\frac{1965}{4}\right) = 13.76 \text{ kip}$$

Column Load Takedown (C3) (Generic Interior)

Influence Area $= (29'\text{-}7" + 26'\text{-}0")(30'\text{-}0" + 30'\text{-}5") = 3358 \frac{\text{sq ft}}{\text{floor}}$

$= 3358(4 \text{ floors}) = 13432$ sq ft

Live load reduction

$$LL = \left[0.25 + \sqrt{\frac{15}{13432}}\right] = 0.38 \Rightarrow \text{use } 0.40$$

Estimate floor dead load

$$\frac{10"}{12"}(150) = 125 \text{ psf}$$

Reduced Live Load

$$0.4(80) = 32 \text{ psf}$$

Total floor Service load

$$157 \text{ psf}\left(\frac{3358}{4}\right) = 131.8 \text{ kips}$$

Add drop pane weight

$$(8/12)(150)(9'\times 9') = 8.1 \text{ kips}$$

Add Column Selfweight $= (1.5^2)(77.5)(150) = 26$ kip

Total Axial service load

$$(131.8 + 8.1)(5) = 700 \text{ kip}$$

Superimposed dead $= \left(\frac{3358}{4}\right)(4)(20) = 67$ kip

Column Interaction Verifications

Verify Column Interaction Diagram

Typical Interior

Axial Strength, P_o

$$P_o = 0.85 f'_c (bh - \Sigma A_{si}) + \Sigma A_{si} f_{si}$$

$b = 18_{in}$, $h = 18_{in}$ $\quad A_s = (1 in^2)(16) = 16 in^2$ $\quad f'_c = 4 ksi$

$\varepsilon_c = \varepsilon_s = 0.003 \Rightarrow \frac{f_y}{E} = \frac{50}{29{,}000} = 0.00172$

$.003 > .00172$ ✓ Steel is yielding

$\varepsilon_s = 50 ksi$

$$P_o = 0.85(4)[18(18) - 16] + [16(50)]$$

$$P_o = 1947.2 \text{ kips}$$

$\varepsilon_t \leq 0.002 \rightarrow \Phi = 0.65$

$\phi P_n = 0.65(1947.2) = 1267^k$

$\phi P_n = 1280$ kips from spColum.

$1267^k \approx 1280^k$ ✓

Pure Tension, T_o

$f_s = -f_y$

$T_o = A_s \cdot f_y = 16\text{in}^2(-60) = -960$

Pure Tension = Tension Controlled $\Rightarrow \phi = 0.9$

$\phi t_o = 0.9(-960) = -864^k$

$\phi t_o = -865^k$ from spColumn

$-864^k \approx -865^k$ ✓

Typical Exterior

Axial Strength, P_o

$$P_o = 0.85 f'c (bh - \Sigma A_{si}) + \Sigma A_{si} f_{si}$$

$b = 18$ $h = 18$ $A_s = (1in^2)(4) = 4in^2$ $f'c = 4ksi$

$\varepsilon_c = \varepsilon_s = 0.003 \rightarrow \frac{f_y}{E} = \frac{60}{29000} = 0.00206$

$0.003 > .00206$ ✓ Steel is yielding

$\varepsilon_s = 60$ ksi

$P_o = 0.85(4)(18 \times 18 - 4) + 4(60)$

$= 1328^k$

$\varepsilon_t \leq .002 \rightarrow \phi = 0.65$

$\phi P_n = 0.65(1328) = 863.2^k$

$\phi P_n = 862^k$ from spColumn

$862^k \approx 863.2^k$ ✓

Pure Tension, T_o

$f_s = -f_y$ $T_o = A_s \cdot f_y = (4)60 = -240^k$

Pure Tension = Tension controlled $\rightarrow \phi = 0.9$

$\phi T_o = 0.9(-240) = -216^k$

$\phi T_o = -217^k$ from spColumn

$-217^k \approx -216^k$ ✓

Appendix C

Output from ETABS

Structure Data

Table 1.7 - Centers of Mass and Rigidity

Story	Diaphragm	Mass X lb-s²/ft	Mass Y lb-s²/ft	XCM ft	YCM ft	Cumulative X lb-s²/ft	Cumulative Y lb-s²/ft	XCCM ft	YCCM ft	XCR ft	YCR ft
Roof	D1	66682.69	66682.69	73.711	58.9203	66682.69	66682.69	73.711	58.9203	89.9327	45.3546
Story4	D1	72813.34	72813.34	73.7215	58.9164	139496.04	139496.04	73.7165	58.9183	89.5765	45.3794
Story3	D1	72813.34	72813.34	73.7215	58.9164	212309.38	212309.38	73.7182	58.9176	88.891	45.4267
Story2	D1	72813.34	72813.34	73.7215	58.9164	285122.72	285122.72	73.719	58.9173	87.3384	45.4953
Story1	D1	78809.85	78809.85	73.7198	58.9164	363932.57	363932.57	73.7192	58.9171	83.2859	45.6349

Table 1.8 - Mass Summary by Diaphragm

Story	Diaphragm	Mass X lb-s²/ft	Mass Y lb-s²/ft	Mass Moment of Inertia kip-ft-s²	X Mass Center ft	Y Mass Center ft
Roof	D1	66682.69	66682.69	177439.173	73.711	58.9203
Story4	D1	72813.34	72813.34	193621.9816	73.7215	58.9164
Story3	D1	72813.34	72813.34	193621.9816	73.7215	58.9164
Story2	D1	72813.34	72813.34	193621.9816	73.7215	58.9164
Story1	D1	78809.85	78809.85	209578.9756	73.7198	58.9164

Table 1.9 - Mass Summary by Story

Story	UX lb-s²/ft	UY lb-s²/ft	UZ lb-s²/ft
Roof	72802.94	72802.94	0
Story4	79210.44	79210.44	0
Story3	79210.44	79210.44	0
Story2	79210.44	79210.44	0
Story1	85206.95	85206.95	0
Base	411	411	0

ASCE 7-10 Auto Seismic Load Calculation

This calculation presents the automatically generated lateral seismic loads for load pattern Seismic-x according to ASCE 7-10, as calculated by ETABS.

Direction and Eccentricity

Direction = Multiple

Eccentricity Ratio = 5% for all diaphragms

Structural Period

Period Calculation Method = Program Calculated

Coefficient, C_t [ASCE Table 12.8-2]	C_t = 0.02ft
Coefficient, x [ASCE Table 12.8-2]	x = 0.75
Structure Height Above Base, h_n	h_n = 77.5 ft
Long-Period Transition Period, T_L [ASCE 11.4.5]	T_L = 8 sec

Factors and Coefficients

Response Modification Factor, R [ASCE Table 12.2-1]	R = 4
System Overstrength Factor, Ω_0 [ASCE Table 12.2-1]	Ω_0 = 2.5
Deflection Amplification Factor, C_d [ASCE Table 12.2-1]	C_d = 4
Importance Factor, I [ASCE Table 11.5-1]	I = 1

Ss and S1 Source = User Specified

Mapped MCE Spectral Response Acceleration, S_s [ASCE 11.4.1] S_s = 0.17g

Mapped MCE Spectral Response Acceleration, S_1 [ASCE 11.4.1] S_1 = 0.055g

Site Class [ASCE Table 20.3-1] = D - Stiff Soil

Site Coefficient, F_a [ASCE Table 11.4-1]	F_a = 1.6
Site Coefficient, F_v [ASCE Table 11.4-2]	F_v = 2.4

Seismic Response

MCE Spectral Response Acceleration, S_{MS} [ASCE 11.4.3, Eq. 11.4-1]	$S_{MS} = F_a S_S$	S_{MS} = 0.272g
MCE Spectral Response Acceleration, S_{M1} [ASCE 11.4.3, Eq. 11.4-2]	$S_{M1} = F_v S_1$	S_{M1} = 0.132g
Design Spectral Response Acceleration, S_{DS} [ASCE 11.4.4, Eq. 11.4-3]	$S_{DS} = \frac{2}{3} S_{MS}$	S_{DS} = 0.181333g
Design Spectral Response Acceleration, S_{D1} [ASCE 11.4.4, Eq. 11.4-4]	$S_{D1} = \frac{2}{3} S_{M1}$	S_{D1} = 0.088g

Equivalent Lateral Forces

Seismic Response Coefficient, C_s [ASCE 12.8.1.1, Eq. 12.8-2] $C_s = \frac{S_{DS}}{(\frac{R}{I})}$

[ASCE 12.8.1.1, Eq. 12.8-3] $C_{s,max} = \frac{S_{D1}}{T(\frac{R}{I})}$

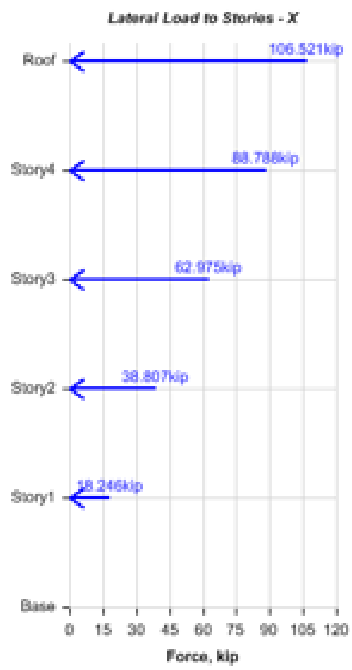
[ASCE 12.8.1.1, Eq. 12.8-5] $C_{s,min} = \max(0.044 S_{DS} I, 0.01) = 0.01$

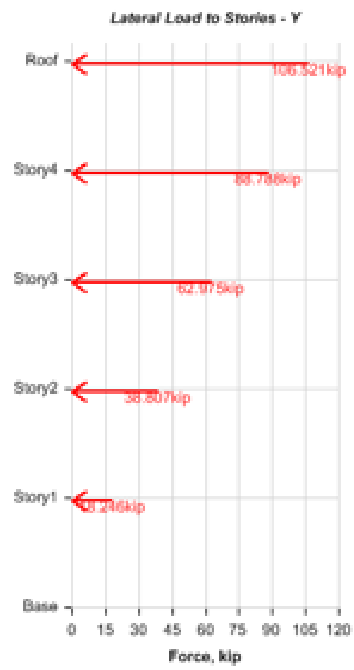
$C_{s,min} \le C_s \le C_{s,max}$

Calculated Base Shear

Direction	Period Used (sec)	C_s	W (kip)	V (kip)
X	0.888	0.024772	12729.38	315.3369
Y	0.888	0.024772	12729.38	315.3369
X + Ecc. Y	0.888	0.024772	12729.38	315.3369
Y + Ecc. X	0.888	0.024772	12729.38	315.3369
X - Ecc. Y	0.888	0.024772	12729.38	315.3369
Y - Ecc. X	0.888	0.024772	12729.38	315.3369

Applied Story Forces





Story	Elevation	X-Dir	Y-Dir
	ft	kip	kip
Roof	77.5	106.521	0
Story4	62	88.788	0
Story3	46.5	62.975	0
Story2	31	38.807	0
Story1	15.5	18.246	0
Base	0	0	0

Story	Elevation	X-Dir	Y-Dir
	ft	kip	kip
Roof	77.5	0	106.521
Story4	62	0	88.788
Story3	46.5	0	62.975
Story2	31	0	38.807
Story1	15.5	0	18.246
Base	0	0	0

Appendix D

Existing Cost Estimate

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Page 1

To(OWNER): Primary Health Network
P.O. Box 716
Sharon, PA 16146

From: Hudson Construction, Inc.
1625 Dutch Lane
Hermitage, PA 16148

Project: New Building
118 Vine Avenue
Sharon, PA 16146

Via(Architect):

Application No: 2
Invoice No: 1107
Period To: 2/5/2015

Architect's
Project No:
Invoice Date: 2/5/2015
Contract Date:

CHANGE ORDER SUMMARY	ADDITIONS	DEDUCTIONS
Approved previous months	0.00	0.00
Approved this month	0.00	0.00
TOTALS	0.00	0.00
Net change by change orders	0.00	

1. ORIGINAL CONTRACT SUM.......$	14,657,624.00
2. Net change by Change Orders.......$	0.00
3. CONTRACT SUM TO DATE(Line 1 +/- 2).......$	14,657,624.00
4. TOTAL COMPLETED & STORED TO DATE.......$	1,380,840.98
5. RETAINAGE.......$	138,084.10
6. TOTAL EARNED LESS RETAINAGE.......$ (Line 4 less Line 5)	1,242,756.88
7. LESS PREVIOUS CERTIFICATES FOR PAYMENT.......$ (Line 6 from prior Certificate)	482,593.45
8. SALES TAX.......$	0.00
9. CURRENT PAYMENT DUE.......$	760,163.43
10. BALANCE TO FINISH, PLUS RETAINAGE.......$ (Line 3 less Line 6)	13,414,867.12

A ITEM NO.	B DESCRIPTION OF WORK	C SCHEDULED VALUE	D WORK COMPLETED FROM PREV. APPLICATION (D+E)	E WORK COMPLETED THIS PERIOD	F MATERIALS PRESENTLY STORED (Not in D or E)	G TOTAL COMPLETED AND STORED TO DATE (D+E+F)	% G/C	H BALANCE TO FINISH (C-G)	I RETAINAGE
01	GENERAL CONDITIONS	408,439.00	24,506.34	49,012.68	0.00	73,519.02	18	334,919.98	7,351.90
02	BUILDING PERMIT	28,817.00	28,817.00	0.00	0.00	28,817.00	100	0.00	2,881.70
03	POWER COMPANY FEES	27,500.00	27,500.00	0.00	0.00	27,500.00	100	0.00	2,750.00
04	RENOVATE PARKING GARAGE	812,900.00	0.00	634,062.00	0.00	634,062.00	78	178,838.00	63,406.20
05	RELOCATE UTILITIES	308,000.00	206,360.00	101,640.00	0.00	308,000.00	100	0.00	30,800.00
06	BUILDING DEMOLITION	48,000.00	48,000.00	0.00	0.00	48,000.00	100	0.00	4,800.00
07	SITE DEMOLITION	32,132.00	9,639.60	9,639.60	0.00	19,279.20	60	12,852.80	1,927.92
08	EROSION CONTROL	4,675.00	0.00	0.00	0.00	0.00	0	4,675.00	0.00
09	SITEWORK	250,739.00	0.00	25,073.90	0.00	25,073.90	10	225,665.10	2,507.39
10	STORM SYSTEM	18,801.00	0.00	0.00	0.00	0.00	0	18,801.00	0.00
11	SANITARY SYSTEM	1,716.00	0.00	0.00	0.00	0.00	0	1,716.00	0.00
12	WATER SERVICE	22,000.00	0.00	0.00	0.00	0.00	0	22,000.00	0.00
13	PHONE AND CABLE CONDUITS	6,600.00	0.00	5,280.00	0.00	5,280.00	80	1,320.00	528.00
14	SITE CONCRETE	82,974.00	0.00	0.00	0.00	0.00	0	82,974.00	0.00
15	ASPHALT PAVING	32,441.00	0.00	0.00	0.00	0.00	0	32,441.00	0.00
16	PATCH PAVING	8,580.00	0.00	0.00	0.00	0.00	0	8,580.00	0.00
17	TRAFFIC SIGNS	660.00	0.00	0.00	0.00	0.00	0	660.00	0.00
18	DUMPSTER ENCLOSURE	18,505.00	0.00	0.00	0.00	0.00	0	18,505.00	0.00
19	FLAGPOLES	3,300.00	0.00	0.00	0.00	0.00	0	3,300.00	0.00
20	LANDSCAPING	39,414.00	0.00	0.00	0.00	0.00	0	39,414.00	0.00
21	CONCRETE	577,517.00	0.00	0.00	0.00	0.00	0	577,517.00	0.00
22	CONCRETE REBAR	64,784.00	32,392.00	0.00	0.00	32,392.00	50	32,392.00	3,239.20
23	MASONRY	634,700.00	0.00	0.00	0.00	0.00	0	634,700.00	0.00
24	MASONRY REBAR	35,417.00	0.00	0.00	0.00	0.00	0	35,417.00	0.00
25	STRUCTURAL STEEL	1,029,286.00	0.00	10,292.86	0.00	10,292.86	1	1,018,993.14	1,029.29
26	MISC STEEL	192,500.00	0.00	9,625.00	0.00	9,625.00	5	182,875.00	962.50
27	ROUGH CARPENTRY	54,275.00	0.00	0.00	0.00	0.00	0	54,275.00	0.00
28	FINISH CARPETNRY	23,785.00	0.00	0.00	0.00	0.00	0	23,785.00	0.00
29	CASEWORK	213,902.00	0.00	0.00	0.00	0.00	0	213,902.00	0.00
30	FOUNDATION INSULATION	4,352.00	0.00	0.00	0.00	0.00	0	4,352.00	0.00
31	INSULATION BEHING SIDING	35,696.00	0.00	0.00	0.00	0.00	0	35,695.00	0.00
32	RIGID INSULATION 1ST AND 2ND FLOOR	21,516.00	0.00	0.00	0.00	0.00	0	21,516.00	0.00
33	FACED INSULATION	23,667.00	0.00	0.00	0.00	0.00	0	23,667.00	0.00
34	VAPOR BARRIER	5,588.00	0.00	0.00	0.00	0.00	0	5,588.00	0.00
35	WATERPROOF ELEVATOR PIT	2,156.00	0.00	0.00	0.00	0.00	0	2,156.00	0.00
36	JOINT SEALANTS	29,416.00	0.00	0.00	0.00	0.00	0	29,416.00	0.00
37	MEMBRANE ROOFING	145,469.00	0.00	0.00	0.00	0.00	0	145,469.00	0.00
38	EFIS	161,129.00	0.00	0.00	0.00	0.00	0	161,129.00	0.00
39	METAL FLASHING EXTERIOR	27,018.00	0.00	0.00	0.00	0.00	0	27,018.00	0.00
40	BRICK SIDING	307,987.00	0.00	0.00	0.00	0.00	0	307,987.00	0.00
41	HM AND WOOD DOORS	118,500.00	0.00	0.00	0.00	0.00	0	118,500.00	0.00

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A	B	C	D	E	F	G		H	I
ITEM NO.	DESCRIPTION OF WORK	SCHEDULED VALUE	WORK COMPLETED: FROM PREV. APPLICATION (D+E)	WORK COMPLETED: THIS PERIOD	MATERIALS PRESENTLY STORED (Not in D or E)	TOTAL COMPLETED AND STORED TO DATE (D+E+F)	% G/C	BALANCE TO FINISH (C-G)	RETAINAGE
42	DOOR LABOR	62,719.00	0.00	0.00	0.00	0.00	0	62,719.00	0.00
43	ALUMINUM ENTRANCES AND WINDOWS	401,500.00	0.00	0.00	0.00	0.00	0	401,500.00	0.00
44	GLASS RAIL	32,653.00	0.00	0.00	0.00	0.00	0	32,653.00	0.00
45	DRIVE THRU WINDOW	3,658.00	0.00	0.00	0.00	0.00	0	3,658.00	0.00
46	METAL STUDS AND DRYWALL	758,718.00	0.00	0.00	0.00	0.00	0	758,718.00	0.00
47	SOUND INSULATION	33,660.00	0.00	0.00	0.00	0.00	0	33,660.00	0.00
48	LEAD LINED DRYWALL	16,500.00	0.00	0.00	0.00	0.00	0	16,500.00	0.00
49	STRUCTURAL STUDS	408,700.00	0.00	0.00	0.00	0.00	0	408,700.00	0.00
50	EXTERIOR SHEATHING	36,736.00	0.00	0.00	0.00	0.00	0	36,736.00	0.00
51	CARPET AND VCT	234,300.00	0.00	0.00	0.00	0.00	0	234,300.00	0.00
52	EPOXY FLOORS	18,870.00	0.00	0.00	0.00	0.00	0	18,870.00	0.00
53	CERAMIC	106,177.00	0.00	0.00	0.00	0.00	0	106,177.00	0.00
54	ACCOUSTICAL CEILINGS	193,046.00	0.00	0.00	0.00	0.00	0	193,046.00	0.00
55	WOOD CEILING	7,581.00	0.00	0.00	0.00	0.00	0	7,581.00	0.00
56	PAINTING	120,466.00	0.00	0.00	0.00	0.00	0	120,466.00	0.00
57	TOILET PARTITIONS	6,949.00	0.00	0.00	0.00	0.00	0	6,949.00	0.00
58	BATHROOM ACCESSORIES	22,611.00	0.00	0.00	0.00	0.00	0	22,611.00	0.00
59	SPECIMEN PASS THRUS	330.00	0.00	0.00	0.00	0.00	0	330.00	0.00
60	FIRE EXT	3,118.00	0.00	0.00	0.00	0.00	0	3,118.00	0.00
61	INTERIOR SIGNS	4,980.00	0.00	0.00	0.00	0.00	0	4,950.00	0.00
62	INTERIOR COLUMNS	23,259.00	0.00	0.00	0.00	0.00	0	23,259.00	0.00
63	EXTERIOR COLUMNS	64,728.00	0.00	0.00	0.00	0.00	0	64,726.00	0.00
64	CANOPY	30,000.00	0.00	0.00	0.00	0.00	0	30,000.00	0.00
65	ELEVATORS	221,885.00	0.00	0.00	0.00	0.00	0	221,885.00	0.00
66	SPRINKLERS	180,950.00	0.00	0.00	0.00	0.00	0	180,950.00	0.00
67	PLUMBING	462,000.00	0.00	0.00	0.00	0.00	0	462,000.00	0.00
68	DIG FOR PLUMBERS	15,000.00	0.00	0.00	0.00	0.00	0	15,000.00	0.00
69	HVAC	1,402,500.00	0.00	0.00	0.00	0.00	0	1,402,500.00	0.00
70	ELECTRIC	1,760,000.00	0.00	0.00	0.00	0.00	0	1,760,000.00	0.00
71	UNFINISHED SPACE FIT OUT 37,628 X $34.00	866,408.00	0.00	0.00	0.00	0.00	0	866,408.00	0.00
72	ELECTRIC FIT OUT	374,000.00	0.00	0.00	0.00	0.00	0	374,000.00	0.00
73	PLUMBING FIT OUT	136,896.00	0.00	0.00	0.00	0.00	0	136,896.00	0.00
74	HVAC FIT OUT	622,906.00	0.00	0.00	0.00	0.00	0	622,906.00	0.00
75	BOND	159,000.00	159,000.00	0.00	0.00	159,000.00	100	0.00	15,900.00
	Totals	14,657,624.00	536,214.94	844,626.04	0.00	1,380,840.98	9	13,276,783.02	138,084.10