GATEWAY COMMONS
ITHACA, NY

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STRUCTURAL OPTION
Spring 2008
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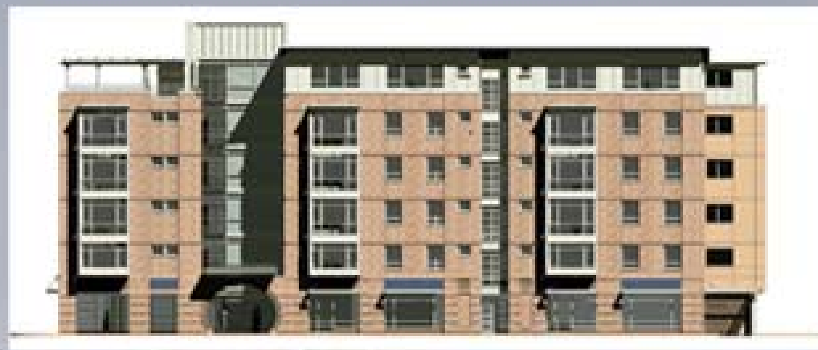
Gateway Commons
Ithaca, NY



General Project Info

Cost: *$7.2 million*
Size: *43,000 sq ft*
Stories: *6 above grade and 1 basement level*
Occupancy: *25 apartments and 2 retail spaces*
Occupant: *Ithaca Rentals & Renovation*
Construction: *December 2005 - April 2007*

Primary Project Team

Owner: *Gateway Commons, LLC*
Architect: *Holt Architects*
Structural Engineer: *Ryan-Biggs Associates*
General Contractor: *Northeast Construction*
Mechanical Designer: *Halco Mechanical*
Landscape Designer: *Trowbridge & Wolf*
Energy Consultant: *Taitem Engineering*
Masonry Contractor: *Casler Masonry*
Precast Plank Supplier: *Empire Precastors*

Lighting/Electrical

- Track lighting typical for appartments uses a 35w MRII Bi-pin base lamp
- Appartments use 120v duplex receptacles and 240v receptacles

Architecture

- Received LEED SILVER Certification
- Building shape made up of 2 rectangular forms
- Facade uses brick, glass, EIFS and metal paneling
- 2 retail spaces are located on first floor the rest of the floors are apartment spaces
- A rooftop garden is available to all residents
- Sthapatya Veda principles were used in the design

Structural

- Footings have been designed for a soil bearing pressure of 5,000 psf.
- Spread footing and spot footing foundation with strength of f'c = 3,000 psi
- 8" CMU bearing walls
- Floor system is constructed of 8" hollow core precase concrete planks

Mechanical

- Typical unit is conditioned by a MC QUAY heat pump with a 1.5 ton cooling capacity and on average a 24,000 BTU/hr heating capacity.
- Chase brings outside air to heat pump where it is mixed with recirculated air

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http://www.engr.psu.edu/ae/thesis/portfolios/2008/gjn113/

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Executive Summary

The Gateway Commons building in Ithaca, New York is a mixed-use development building being used for retail and residential apartments. It has a basement floor below grade and six floors above grade at a height of 62 feet. CMU walls supporting precast concrete hollow core planks make up the building structure. The building façade uses a combination of brick, an Exterior Insulation Finish System (EIFS), and metal panels. The apartment units are designed as luxury apartments. Construction of this project started in December of 2005 and was completed in April of 2007.

This report consists of a detailed study of an alternative structural system. The structural members: columns, girders, pan joist slab, footings and shear walls were all designed according to the loads applied and constraints that restricted the member sizes. Columns of size 14”x24”, girders of the size 14”x18” and 14”x16”, and a pan joist slab with a tops slab of 4.5” thick with 7”x10” joists were used in the structure redesign of the gravity force resisting system. 8” thick ordinary reinforced concrete shear walls were used as the lateral force resisting system.

Two breadth studies were preformed to validate the redesign of the structure. In the architecture breadth new structure was designed as an office building to show that the new structure allows for versatility in redesign of the architecture. The column layout on the new structure was superimposed on the existing architecture floor plan that the new structure is compatible with the existing architecture.

A construction management breadth was also completed for this project. The cost of the existing structure is $2,078,841. The cost of the new structure will be $1,293,136. The total cost savings of switching the structure from precast hollow core concrete planks on CMU walls to a concrete pan joist system is $785,705. A schedule comparison was also preformed and the new structure was able to be completed 79 day before the existing structure would have finished.

Introduction

Gateway Commons located in downtown Ithaca, New York is a LEED registered, $7.4 million, upscale, mixed use development containing two retail and 25 residential spaces. It offers unique and spacious apartments for mature living with finishes and features more commonly found in major metropolitan areas. It has a basement floor below grade and six floors above grade at a height of 62 feet. The total building area is 43,000 square feet.

The basement is used for storage and a mechanical room. The ground floor includes a one bedroom apartment and two retail spaces only one is occupied right now by Ithaca Coffee Company. The floors above include one, two, and three bedroom apartments and a roof garden on the sixth floor. The monthly cost of renting theses apartments will range from $1475 to $3295 depending on the size of the apartment. Construction for this project started in December of 2005 and was completed in April of 2007.

Location

Gateway Commons is located at 311 East Green Street in Ithaca, New York. The site is unique because it is within walking distance of Ithaca's downtown area as well as being adjacent to the Six Mile Creek Nature Area. Downtown Ithaca is a culturally rich scene with art, music, theater, cafes, shopping, and a business district. The Six Mile Creek Walk provides excellent opportunities for recreational activities. Gateway commons is highlighted in yellow on the map of Ithaca in Figure 1.

Figure 1 – maps

Primary Project Team

- **Building Owner/ Landlord:**
  Ithaca Rentals & Renovations
  http://www.ithaca-rentals.com/index.htm

- **Architect:**
  Holt Architects
  http://www.holt.com/

- **Structural Engineer:**
  Ryan-Biggs Associates
  http://www.ryanbiggs.com/

- **Mechanical Engineer:**
  Halco Mechanical
  http://www.halcoheating.com/

- **Electrical Engineer:**
  The Sparks Electric Co. Inc.

- **General Contractor:**
  Northeast Construction Services
  http://www.northeastconstruction.net/

- **Masonry Contractor:**
  Casler Masonry

- **Precast Plank Suppliers:**
  Empire Precasters

Architecture

Design and Functional Components:

The shape of the building is made up of two rectangular forms connected on their long sides. The first five stories have a façade of brick, EIFS, and glass. The sixth floor façade is composed of metal panel siding and glass, and acts as an ornamental cap for the building. The façade materials used on the first five stories was chosen to make a connection between the Gateway Commons building and the Gateway Center building, a pre-existing building located on the same site.

The basement is mainly storage and mechanical room space. The first floor is made up of 2 retail spaces and a 1 bedroom apartment. Most of this space is used as retail space, and there is an independent entrance into the residential portion of the building. On floors two through six, a corridor is located where the two rectangular forms connect with each other. On either side of that corridor there are apartments. The second through fifth floors are identical in their layout. Each floor includes (1) 3 bedroom apartment, (2) 2 bedroom apartments, and (2) 1 bedroom apartments. The sixth floor includes (2) 2 bedroom apartments, (1) luxury 2 bedroom apartment, (1) 1 bedroom apartment, and an outdoor terrace.

Building Envelope:

The building's wall structure is constructed of 8" CMU. Some walls have an exterior façade constructed of an EIFS (Exterior and Insulated Finishing Systems). Other walls have an exterior façade constructed of brick. This masonry system is made up of 3" XPS insulation against the CMU wall, an air space, and face brick connected to the CMU wall with wall ties. The windows on this project are aluminum framed windows with a U factor of 0.60 Btu/sq. ft. x h x deg F and a maximum air infiltration rate of 0.1 cfm/sq. ft. The sixth floor façade is made up of a 2" EPS insulation and metal siding. The roof structure is a hollow core concrete plank topped with 6" of PolyISO insulation and a membrane roof.

Mechanical

The typical unit is conditioned by a MC QUAY heat pump with a 1.5 ton cooling capacity and on average a 24,000 BTU/hr heating capacity. A chase brings outside air to the heat pumps where it is mixed with re-circulated air to meet the ventilation needs. An Energy Recovery Ventilator (ERV) was also incorporated into the mechanical design. The ERV is located on the roof and will be used to exchange the heat and humidity of the outgoing conditioned air with the incoming air. This reduces the amount of energy that is required to heat or cool the fresh air. There is also an EVAPCO cooling tower located on the roof with a GPM of 98, water in temperature of 102° F, and water out temperature of 90° F.

Lighting/Electrical

The electrical system in the Gateway Commons building operates under a simple radial system. An NYSEG pad mounted transformer brings one service line into the switchboard. The 2000 Amp 208Y/120 V switchboard distributes power to different panels throughout the building.

The building is mostly lit by fluorescent lighting. The apartments are lit by compact fluorescent lights and track lighting. In the apartments lights are operated by a standard wall box switch. In the public spaces lights are operated by occupancy sensors. The lighting design for the retail spaces will be finalized by the company that decide to rent the space.

Construction

The delivery method for the Gateway Commons project was a negotiated contract with Northeast Construction, the project's general contractor. The cost of the project amounted to $7.2 million. Construction of the building started in December of 2005 and was completed in April of 2007.

LEED Certification

The Gateway Commons project received a LEED Silver Certification. The interior air quality factors that helped obtain this rating are large operable windows that continuously supply fresh air to apartments. Cross ventilation and low voc carpets, paints, adhesives, and sealants also added to the interior air quality. Water efficiency factors that contributed to the silver certification are rainwater collection for watering plants, roof top gardens, low flow shower heads, and front load energy star washers. Overall energy use was cut down by the high Albedo roof that reduces heat island effect, energy star appliances, daylight sensors, and no ozone-depleting refrigerants. They also took advantage of the close proximity to mass transit, the use of bike racks, and green materials such as bamboo flooring and porous pavement.

Existing Structural System

Foundation

Between grid lines A and D, the basement floor slab-on-grade and loads from the concrete foundations walls are transferred onto strip footings with a 28-day strength of f'c = 3,000 psi. These strip footings sit on undisturbed indigenous soils composed of sand and gravel with an allowable bearing capacity of 5,000 psf . The slab-on-grade is 5" thick and reinforced with #4 bars at 16" on center spanning in both directions. The slab-on-grade has a concrete strength of f'c = 3,500 psi. The foundations walls will have a concrete strength of f'c = 3,000 psi or 4,000 psi depending on the type of wall. Between grid lines D and E the footings sit on a compacted structural fill that has an allowable bearing capacity of 5,000 psf. The slab on grade in this section is supported by the compacted structural fill and the foundation walls on grid lines D and E. It has the same thickness and reinforcing as the other slab on grade. The slab on grade in this section is 11'-4" higher than slab on grade between grid lines A and D.

There are also five concrete piers that are supported by spot footings on the north east corner of the building. The reason for these piers is to create the loggia. At the second floor a concrete beam spans across the piers to pick up the gravity loads and distribute them onto the piers.

Masonry Walls

The walls that are not considered part of the lateral system are wall type MW1. Unlike the concrete foundations walls these walls are constructed out of 8" thick concrete masonry units (CMU). These walls act as the gravity framing system and support the precast concrete hollow core floor planks that act as the flooring system. Between the first and second floors the walls are grouted solid. Between the second and third floors the walls are grouted at 2' on center. For the rest of the floors, wall type MW1 has vertical reinforcing of #5 at 4' on center. The walls are horizontally reinforced at 16" on center. A wall schedule describing this reinforcing can be found in Figure 2. The exterior walls on the north and part of the east and west sides have a brick façade that is supported by shelf angles at each floor. The exterior walls on the south and other part of the east and west sides carry an Exterior Insulation Finish System (EIFS) façade. A typical floor framing plan is shown in Figure 3. Building sections are shown in Figures 4 and 5 in order to give a better idea of the building structure.

WALL ~~LINTEL~~ SCHEDULE

MARK	VERTICAL REINFORCING	HORIZONTAL REINFORCING	REMARKS
MW1	#5 AT 4'-0"OC	STANDARD JOINT REINFORCING AT 16"OC	GROUT WALL SOLID 1ST-2ND FLOORS GROUT WALL AT 2'-0"OC 2ND-3RD FLOORS
MW2	#5 AT 4'-0"OC (TYPICAL) (6)#5 EACH END (1ST-2ND) (4)#5 EACH END (2ND-4TH) (2)#5 EACH END (4TH-ROOF)	STANDARD JOINT REINFORCING 1ST-2ND AND 6TH-ROOF, HEAVY DUTY JOINT REINFORCING AT 8"OC 2ND-6TH	GROUT WALL SOLID 1ST-2ND FLOORS
MW3	#5 AT 4'-0"OC (TYPICAL) (2)#5 EACH END	STANDARD JOINT REINFORCING 1ST-2ND AND 6TH-ROOF, HEAVY DUTY JOINT REINFORCING AT 8"OC 2ND-6TH	GROUT WALL SOLID 1ST-2ND FLOOR

NOTES:
1. UNLESS NOTED OTHERWISE ON PLAN, ALL WALLS ARE TYPE MW1.
2. MINIMUM REINFORCING REQUIREMENTS SHOWN ON A3/S506 APPLY TO ALL WALLS.
3. SEE F6/S506 FOR PLACEMENT OF VERTICAL BARS AT ENDS OF WALLS.

Figure 2 – Wall Schedule

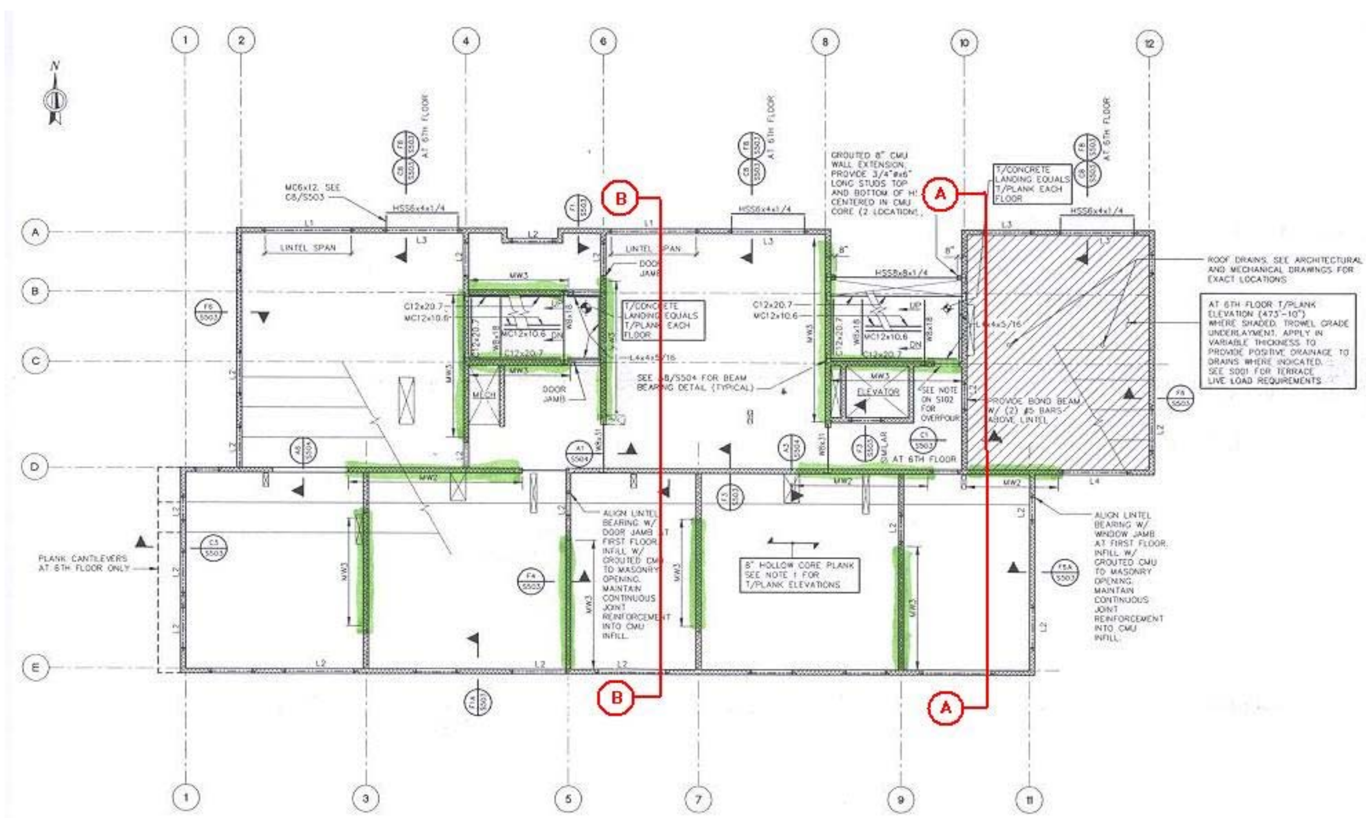


Figure 3 – Typical Framing Plan

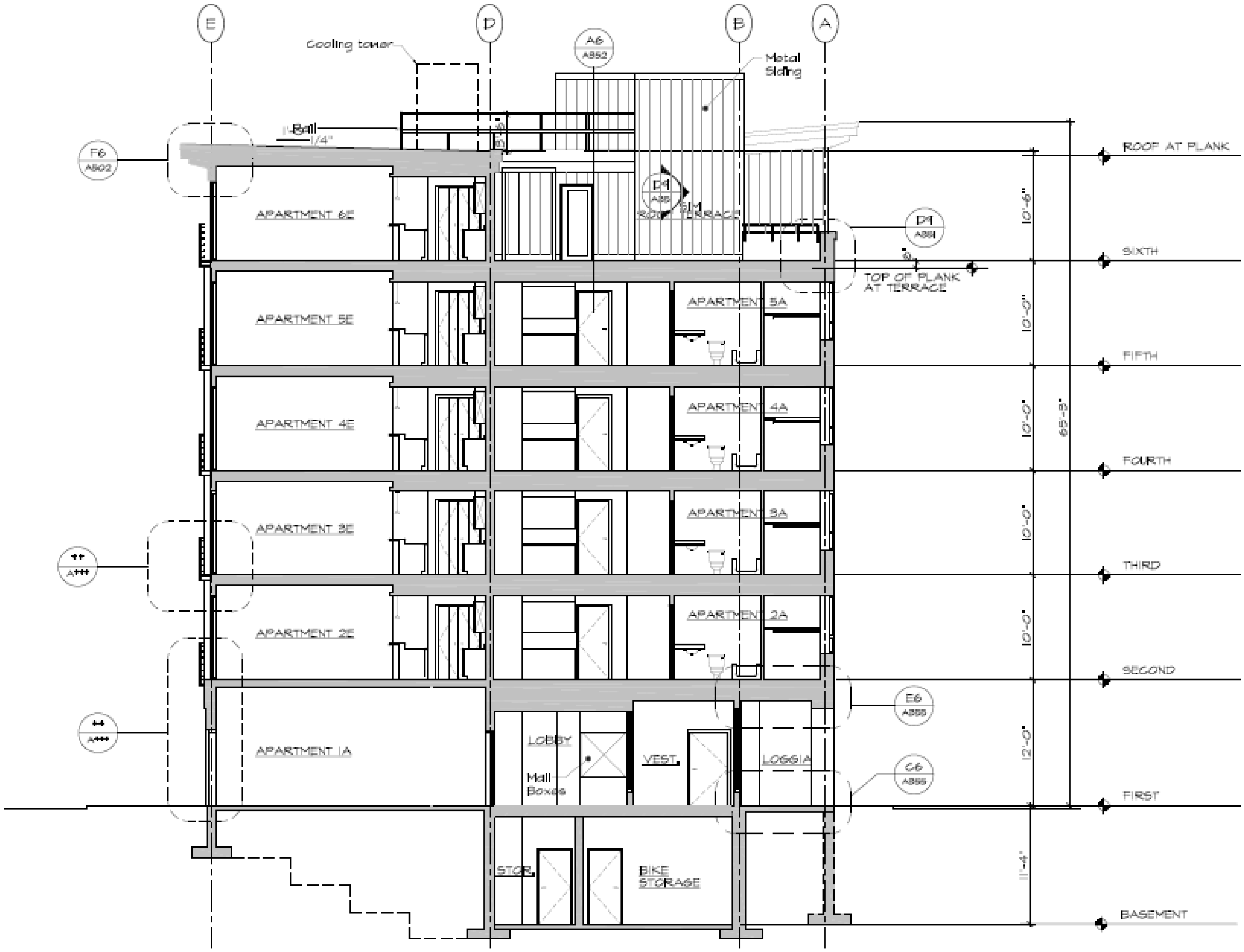


Figure 4 – Section A

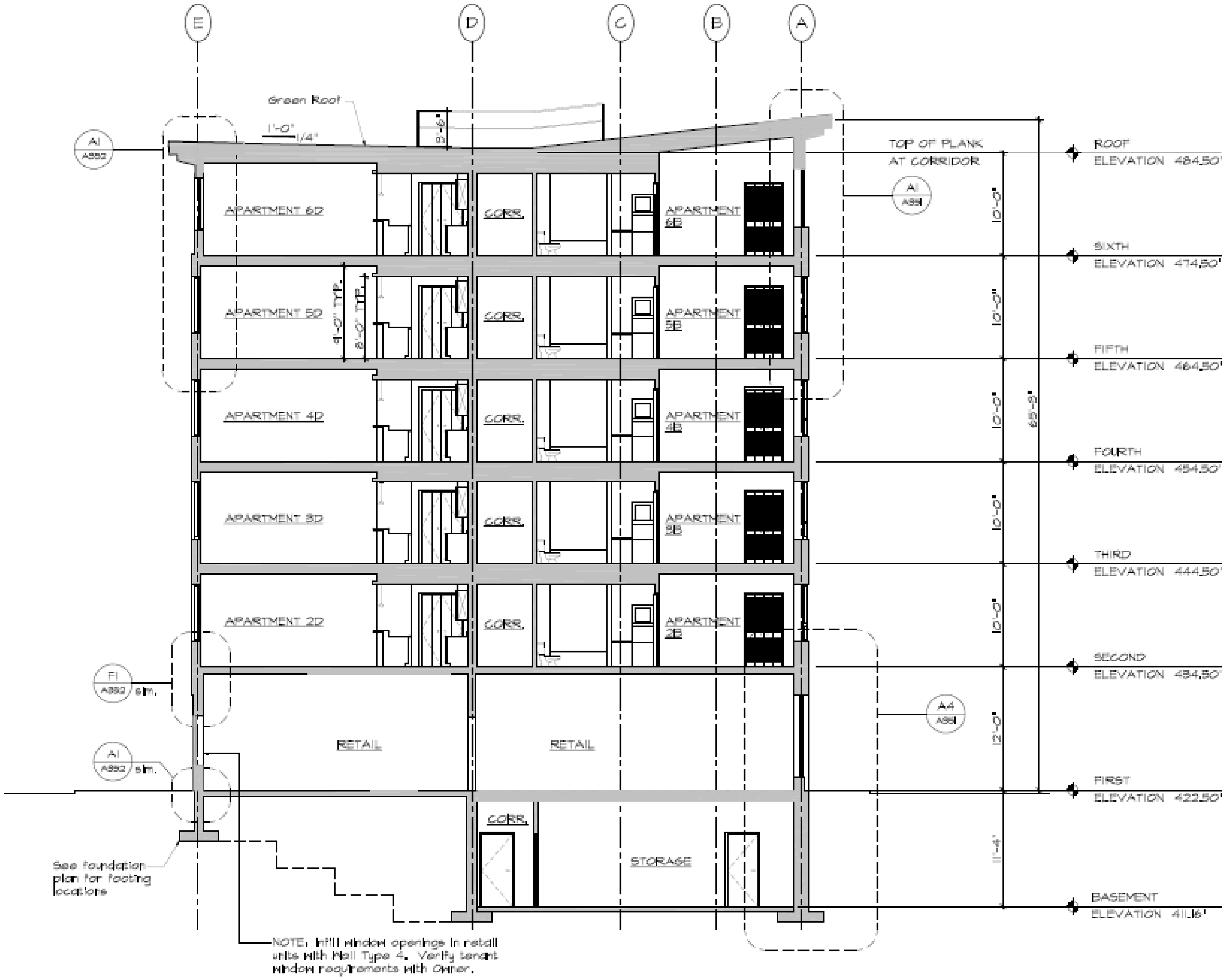


Figure 5 – Section B

Floor System

The primary flooring system for the elevated floors of the building is precast concrete hollow core planks. The planks span in the east/west direction. On the first floor the planks have a thickness of 10", but on floors two through six the plank thickness is 8". The planks on the first floor have a 2" thick concrete topping. All planks have a maximum width of 4' and are allowed to have a minimum width of 1'-6". Planks located at interior bearing partitions must be connected with a 6' long #3 bar or 5/16" diameter strand grouted into the keyway, as shown in Figure 6. Planks are often connected to exterior CMU walls with #4 dowels that are bent into the keyways, as shown in Figure 7. On the first floor, half of the floor is planks while the other half is a 5" thick slab on grade. The slab on grade described in the foundations section is the floor system for the basement.

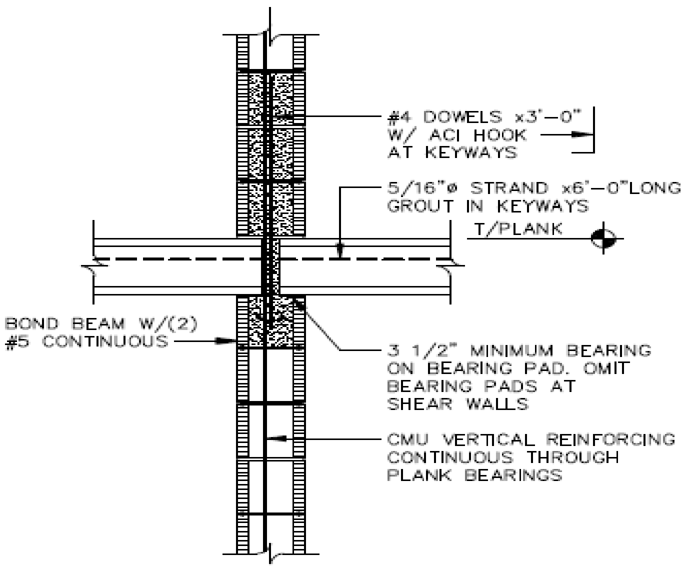


Figure 6 – Floor Planks at Interior Walls

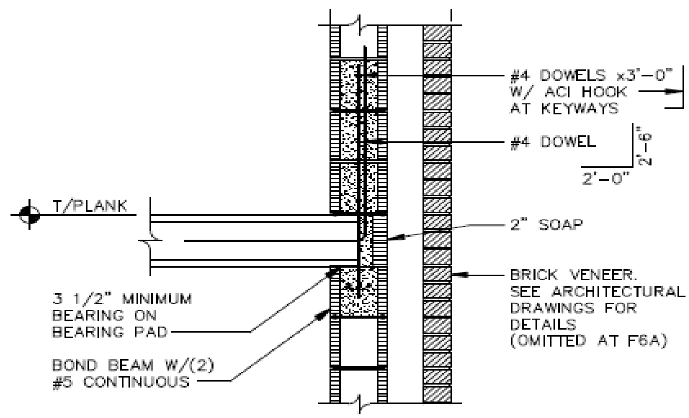


Figure 7 – Floor Planks at Exterior Walls

Roof

The roof structure uses the same 8" thick, precast, hollow core, concrete planks as used on the floors. At gridline D the roof begins to slope up toward the building's south end at ¼"/foot. Between gridline D and C the roof begins to slope up toward the building's north end at slightly larger slope. The building section in Figure 8 shows how the roof is sloped. The roof planks have a 2'-8" roof overhang. Two different steel shapes are used to support the planks at the overhang, a WT6x43.5 and an L6x6x1/2. There is also a roof terrace on the sixth floor that uses the same planks system as used by the typical floor system. There is no roof overhang on the sixth floor roof terrace.

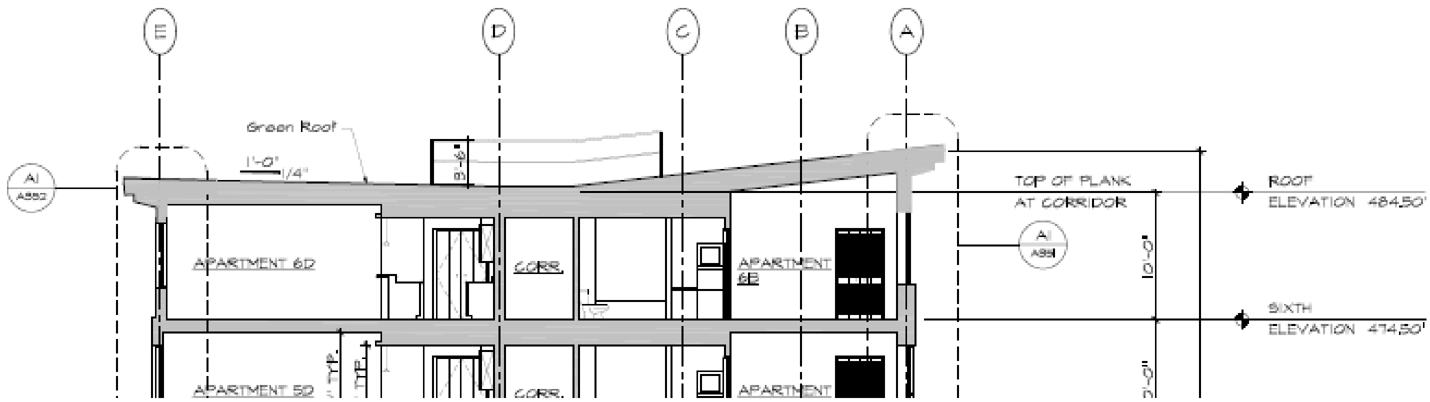


Figure 8 – Building Section for Roof

Lateral System

The structure is laterally supported by intermediate reinforced masonry shear walls in the N-S and E-W directions. Like the load bearing walls for the gravity framing system the shear walls are also 8" thick CMU walls. However, the shear walls are designed to resist the lateral loads due to seismic and wind forces. These lateral forces are distributed onto the shear walls through the rigid floor system of hollow core planks. There are two different shear wall types, MW2 and MW3. The shear walls are highlighted in green on the floor plan in Figure 2. The wall schedule in Figure 1 describes the reinforcing for both shear wall types. An ETABS generated model in Figure 9 shows the shear walls in red in plan and elevation views.

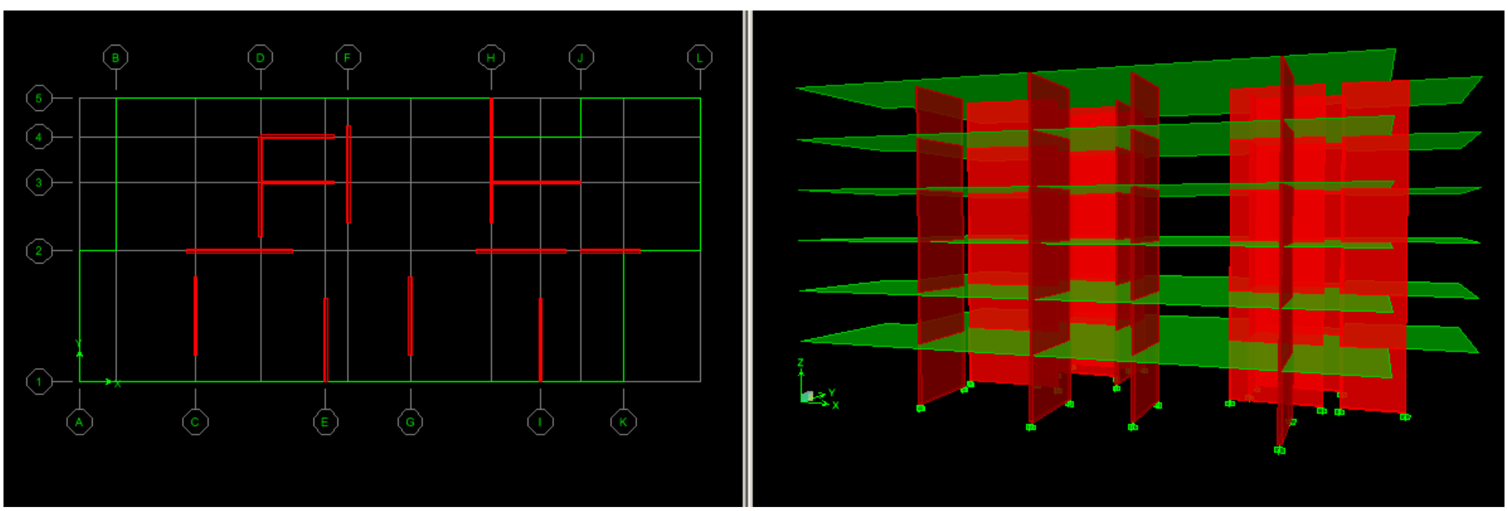


Figure 9 – ETABS Model

Problem Statement

The concrete hollow core floor plank on CMU walls structure of Gateway Commons is an excellent design for the building's use. It is a durable material and relatively inexpensive compared to steel and concrete structural systems. However, this is a very custom structure. Spaces are separated by load bearing walls and openings in the walls have to be coordinated with the architecture. This becomes a problem when a change to the buildings architecture becomes an issue. The interior load bearing walls would make it difficult to produce an effective redesign of the interior spaces.

If the owner felt that the Gateway Commons building could serve a better function than the current residential apartment design it would be almost impossible to redesign the interior for spaces that are different than the ones currently provided. Due to conditions that occur down the road the owner may want the building to be used to an office building or student housing. With the way the interior load bearing walls are laid out it would be impossible to come up with a logical design for these spaces.

An alternate structure would allow for a more versatile design. It should be determined if the added cost is worth the versatility in design.

Proposed Solution/Methods

A pan joist floor system supported by concrete girders and columns proved to be the best structure to fit in with the existing architecture and allow for an effective redesign of the architecture. Columns will have to be located to coincide with the existing architecture. The use of columns instead of walls creates an open floor plan with possibilities for a creative redesign of the architecture.

The lateral system in this design will be concrete shear walls. This design will allow for less shear walls than the current system. They will be placed around the stair towers so that they do not interfere with the open floor plan. Floor to floor height will also have to be taken into consideration due to the 65 feet above grade building height limitation. Edge beams will also have to be designed to support the brick façade.

PCAcolumn, PCAslab, SAP2000, and hand calculations will be used to design the structure for gravity loads. ETABS will be used to obtain the design values for the shear walls and the reinforcement for the walls will be designed by hand calculations and PCAcolumn. I hope to achieve the following goals by redesigning the structure of Gateway Commons:

- To better understand the design of concrete structures and the engineering design process in general
- To create a complete and economical structural design of Gateway Commons
- To compare the new structure to the existing hollow core floor plank on CMU walls structure
- To determine the cost and schedule of the new structure and determine if this redesign is economically feasible.
- To architecturally design the new structure for an office building to show that the new structure allows for versatility in architectural redesign.

Design Criteria

Design Procedure

A two-way concrete slab was first proposed as the system to be used in the redesign of the structure. After investigating this system a column layout that was compatible with the existing architecture could not be determined. It was clear that a one way concrete system would have to be used. A pan joist slab system was chosen because it works well for long span floors with relatively light loads. The slab was determined to span north-south and columns were placed so not to line up with door and windows. Girder sizes were determined by deflection criteria and architectural constraints. Shear walls were positioned around the stairs where previous shear walls were located.

After the structure had been laid out gravity loads were determined and the pan joist slab was designed using PCAslab. Loads on the girders from the slab and possibly exterior façade were determined. The girders were modeled in SAP2000 where pattern loading of the live load was used to determine the maximum design moments. Next, flexure and torsion reinforcing for the beams were designed by hand calculations. The SAP2000 model was used to determine the axial loads and moment acting on the columns. These factored values were used in PCAcolumn to design the columns. Spot footings for the columns were then designed.

After the gravity system had been designed seismic and wind loads were calculated. The ordinary reinforced concrete shear walls were modeled in ETABS. Axial, shear, and moment values were taken from the program and used to design the shear walls. Shear reinforcing was designed by hand and flexure reinforcement was designed with the help of PCAcolumn. Displacement values obtained from ETABS were checked against allowable displacement values.

Codes and References

This section lists the codes and reference materials that aided in designing both the gravity and lateral portions of the structure.

- ACI 318-05
- ASCE 7-05
- Design of Concrete Structures, by Arthur Nilson, David Darwin, and Charles Dolan
- Portland Cement Association's, Notes on ACI 318-05: Building Code Requirements for Structural Concrete
- Reinforced Concrete Mechanics and Design, by James MacGregor and James Wight

Materials

The tables in this section show the material properties of structural components that were used in the design of the structure.

Cast in Place Concrete	
Member	**28 Day Compressive Strength (f'c)**
Columns, girders, slabs, and shear walls	5,000 psi
Interior Slabs on Grade	3,500 psi
Footings	3,000 psi
Retaining Walls	4,000 psi

Structural Steel		
Material	**ASTM Standard**	**Fy (ksi)**
Reinforcing Bars	A 615, Grade 60	60

Loading Conditions

<u>Gravity Loads:</u>

The gravity load information for the existing structure was obtained from the general notes page of the building plans. These loads were used to design the gravity load bearing walls of the existing structure. Since the new structure will be able to be designed as an office building the live load for the floors is now required to meet 80 psf for office corridors.

Existing Structure: Concrete Hollow Core Planks on CMU Walls

Live Loads

First Floor…………………………….…...100 psf
Floors 2-6…………..……………………..…..40 psf
Sixth Floor Terrace……………………….…100 psf
Ground Snow load (Pg)………………......45 psf
Flat Roof Snow Load (Pf)……….……….....32 psf

Dead Loads: Construction

First Floor…………………………………....100 psf
Floors 2-6…………………….……….....70 psf
Green Roof or Roof Top Pavers…………..95 psf
Other Roof Areas…………………………….75 psf
CMU Walls……………………………….55 plf

Dead Loads: Superimposed

Mechanical Equipment……………………5 psf
Partition walls……………………………..10 psf

Dead Loads: Exterior Façade

Brick Façade……………………...……40 plf

New Structure: Pan Joist

Live Loads

First Floor…………………………….…...100 psf
Floors 2-6…………..……………………..…..80 psf
Sixth Floor Terrace……………………….…100 psf
Ground Snow load (Pg)………………......45 psf
Flat Roof Snow Load (Pf)……….……….....32 psf
Second Floor Roof Garden…………...….100 psf

Dead Loads: Construction

Floors 1-6…………………….………...90 psf

Dead Loads: Superimposed

Mechanical Equipment……………………5 psf
Partition walls……………………………..10 psf

Dead Loads: Exterior Façade

Brick Façade……………………...……40 plf

Lateral Loads:

Lateral loads acting on the building are the result of wind and seismic forces. Wind and seismic loads were calculated using methods from ASCE 7 – 05. For each lateral load, story forces are calculated which act at the center of mass of the floor. Wind loads were calculated for the north-south and east-west directions using Method 2-Analyitical Procedure from chapter 6 of ASCE 7 – 05. Wind forces control in the north-south direction of the building because there is a larger surface area for wind forces to act on. See Appendix A1 for more wind load calculations.

Seismic loads were calculated using chapters 11 and 12 of ASCE 7 – 05. Since Gateway Commons is in Seismic Design Category B several simplifications in the code were allowed. A few of the conditions that were allowed to be neglected were structural irregularities, redundancy, and torsional amplification. By changing the structure to concrete the weight of the structure decreases and the seismic base shear drops from 208 kips to 120 kips. See Appendix A2 for more seismic load calculations. The following is a summary of the lateral load findings.

Wind Loading

Basic Wind Speed	V = 90 mph
Importance Factor	I = 1
Exposure Category	B
Building Height	h = 66'
Building Classification	Rigid, Enclosed
Directionality Factor	Kd = 0.85
Topographic Factor	Kzt = 0.85
Velocity Pressure Coeff.	Kh = 0.874
Gust Effect Factor	G = 0.85
Internal Pressure Coeff.	GCpi = ± 0.18

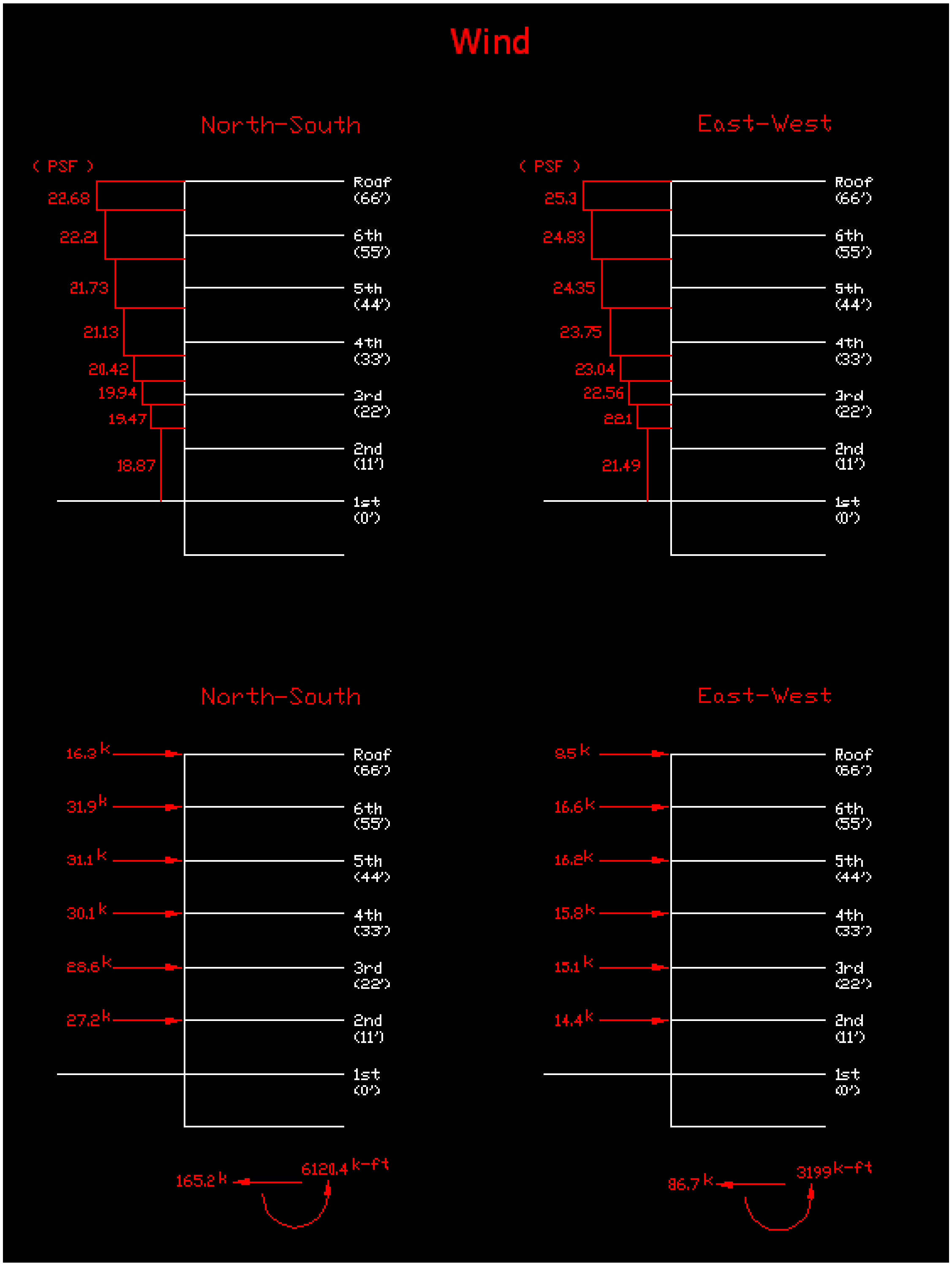


Figure 10 – Wind Pressures and Story Forces

Seismic Loading

Seismic Use Group	I
Site Class	D
Seismic Design Category	B
Importance Factor	I = 1
Spectral Response Acc.	S1 = 0.055, Ss = 0.159
Building Frame System	R = 5
Fundamental Period	Ta = 0.695
Seismic Response Coefficient	Cs = 0.015
Weight of the Building	W = 5516.8 kips
Seismic Base Shear	V = 83 kips

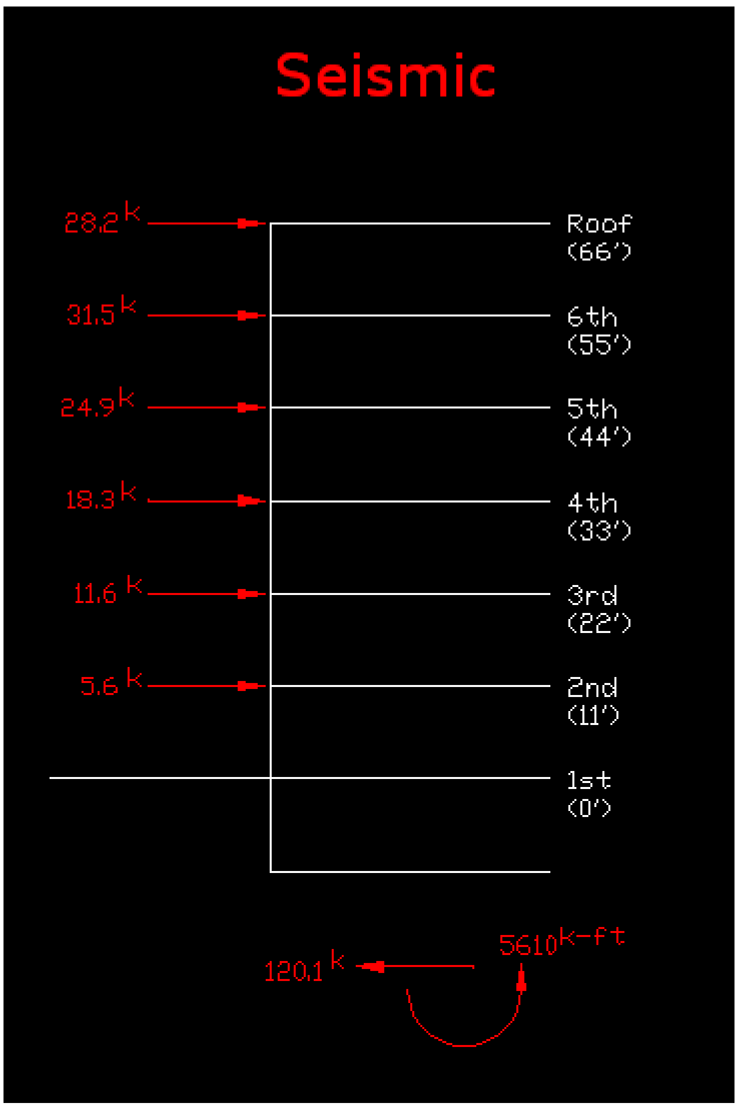


Figure 11 – Seismic Story Forces

Structural Depth

The redesigned structure of Gateway Commons is a pan joist slab supported by girders and columns. Ordinary reinforced concrete shear walls resist the laterals loading on the building. Floor framing plans for the first floor through the roof level along with a wall section are displayed in the following figures. Dimensions for the structure are shown on the first floor plan. In floors 3-6 the continuous beams are labeled as what they will be referred to throughout the report and the shear walls are labeled as well. The east shear walls only include the C shaped wall; the elevator walls are not included as shear walls. The only reason the second floor differs from floor 3-6 is because of the roof garden extending from the east shear walls. The following components of the structure will be discussed in this section of the report: pan joist slab, continuous beams, columns, shear walls, and footings.

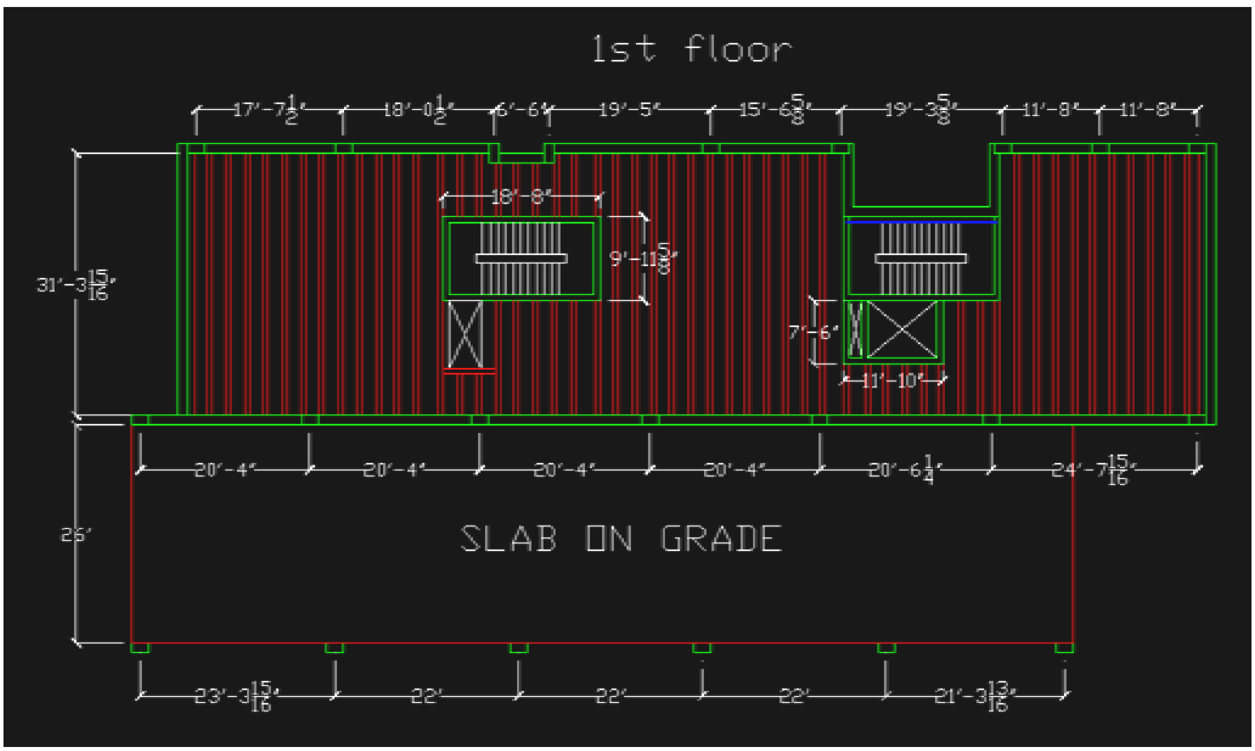


Figure 12 – First Floor Framing Plan

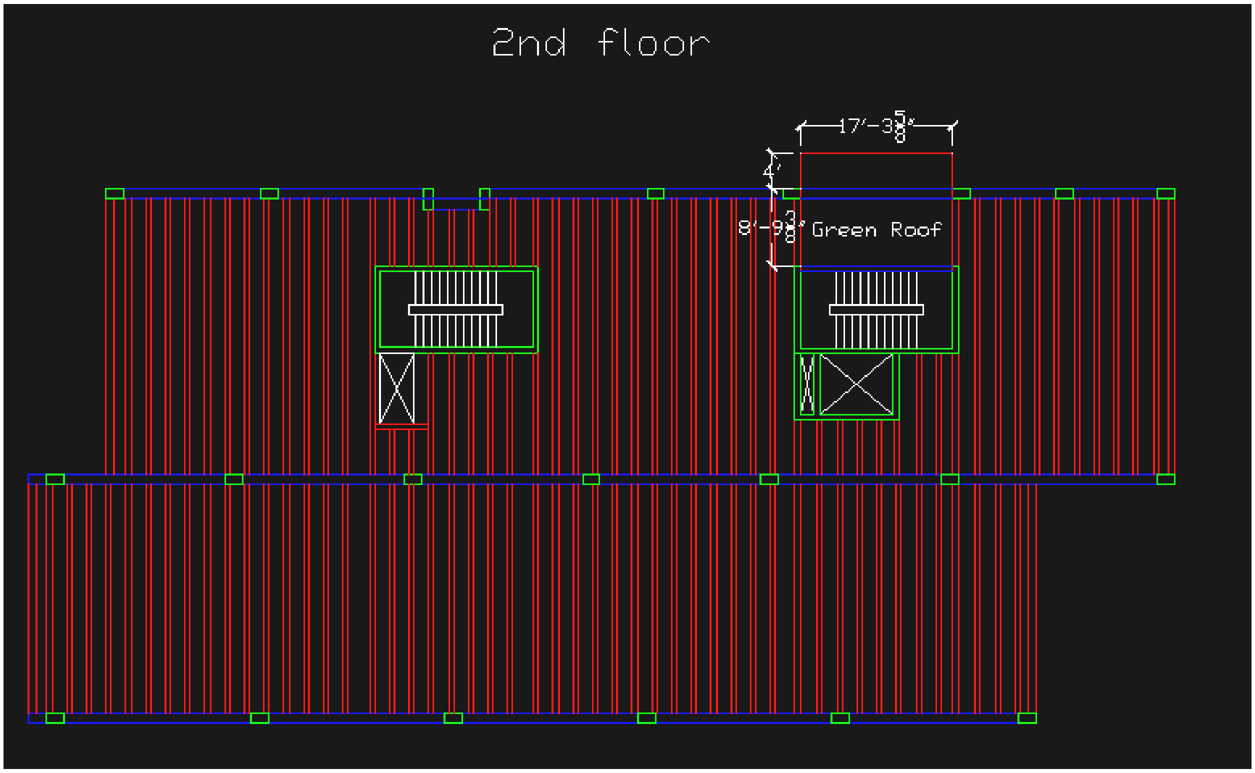


Figure 13 – Second Floor Framing Plan

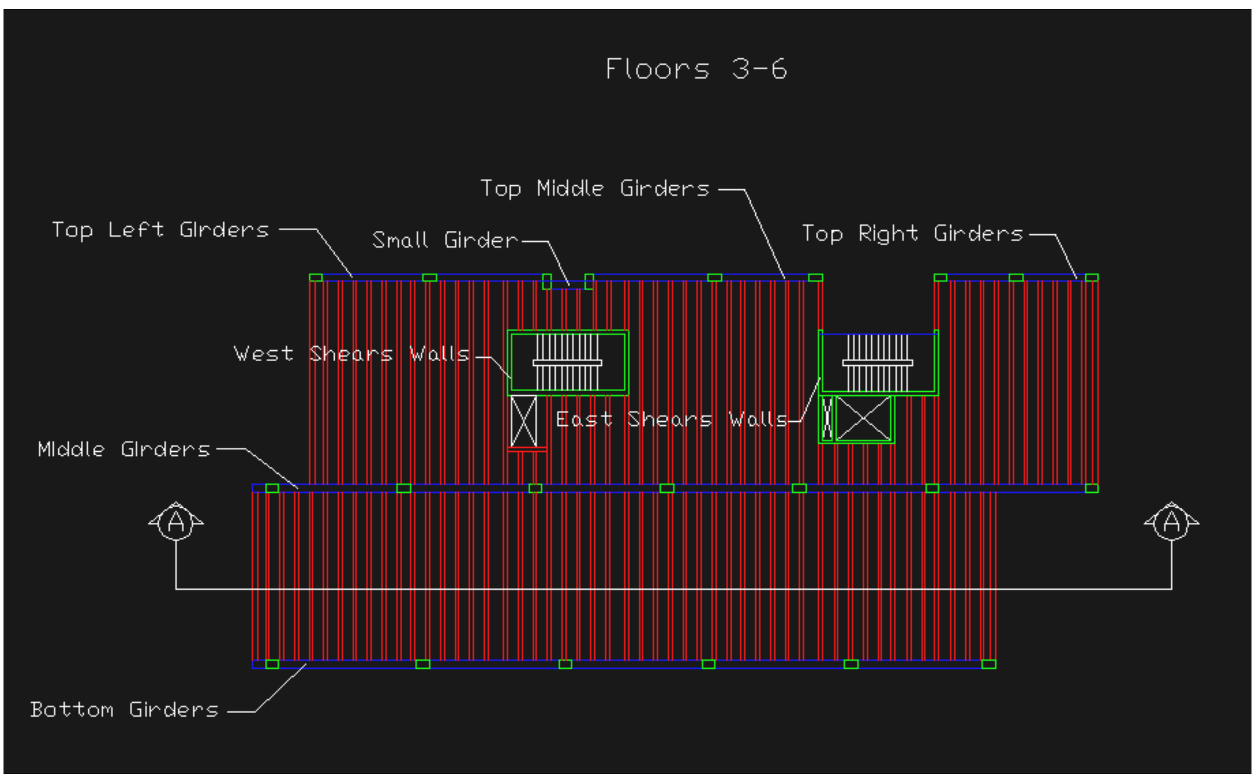


Figure 14 – Third Through Fourth Floor Framing Plan

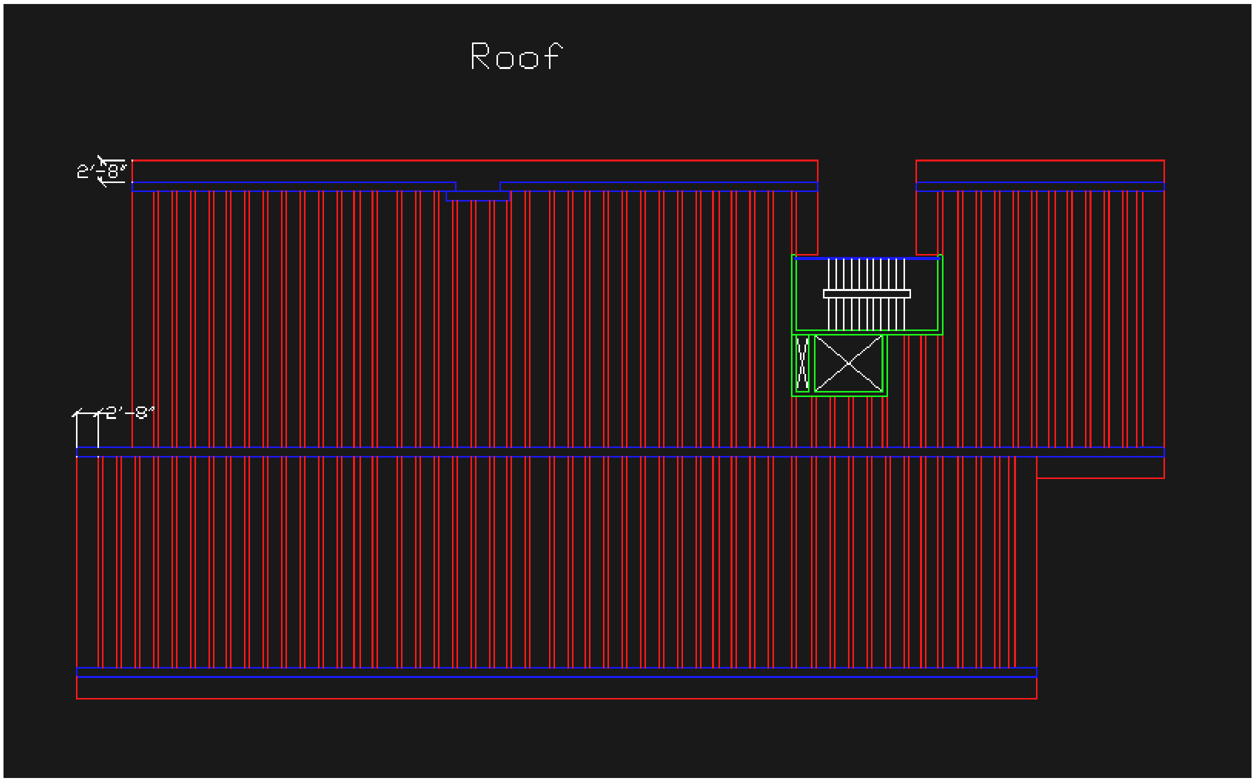


Figure 15 – Roof Framing Plan

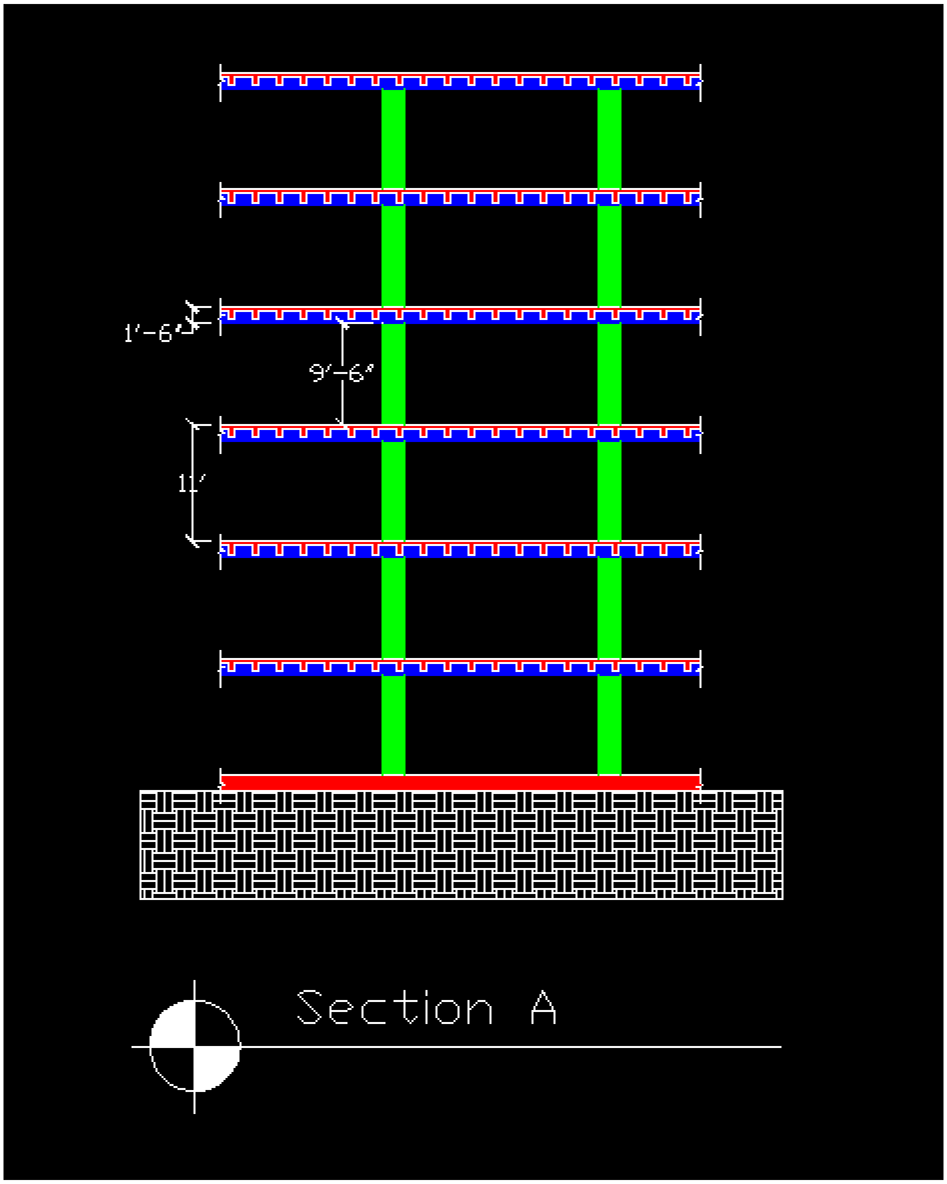
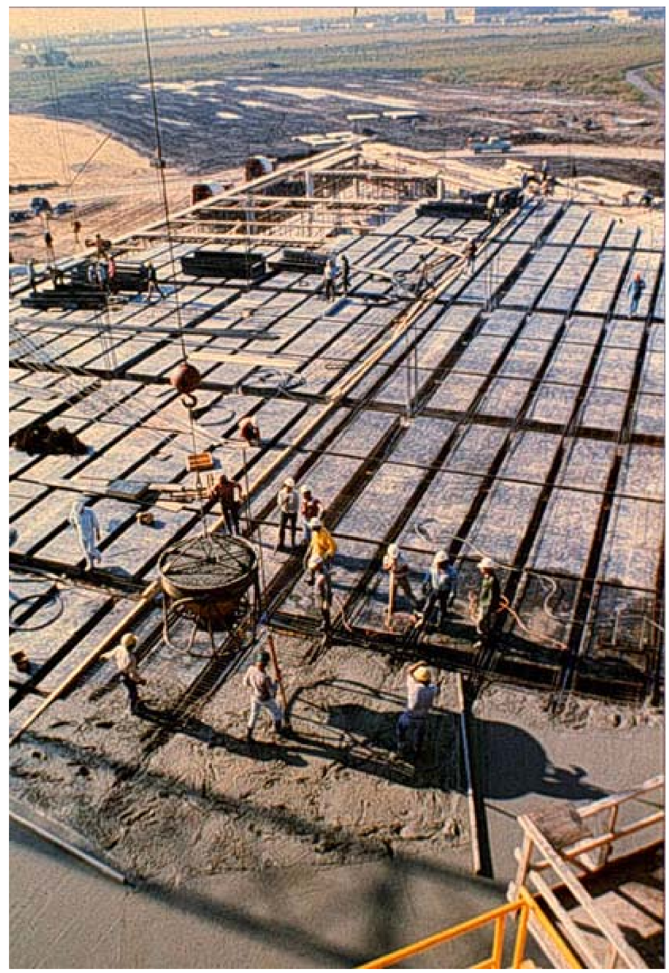


Figure 16– Wall Section

Pan Joist Slab

The structure used in the redesign of Gateway Commons is a concrete pan joist slab system supported by girders and columns and shear walls. This one way slab system, also known as a ribbed slab, is a slab supported by a series of closely spaced T-beams. Reinforcing for tension is placed in the joists and compression reinforcing is placed in the top slab. Distribution ribs running perpendicular to the joists are required for spans greater than 20'. These distribution ribs are 4" wide and the depth of the joists. The distribution ribs can be spaced at a maximum of 15'. These slabs are constructed using reusable metal pans with widths of either 20" or 30"; however specific distances between ribs can also be formed. In determining the dimensions of the slab the top slab thickness will be based on strength and fire protection requirements. The overall depth and rib thickness is determined by deflection and shear.

The top slab depth of 4.5" was determined due to fire resistance rating. This depth provides a 2 hour fire rated slab. The redesign of the structure will allow for the possibility of an office building design which requires 2 hour fire rated horizontal partitions according to Table 706.3.9 of IBC 2006. PCA slab was used to size the joists and design the reinforcement.



Representative design strips for larger parts of the slab are used in PCA slab to design the reinforcement while in smaller areas the whole area can be designed for in PCA slab. Representative design strips produce a conservative design with more reinforcement than is actually needed in the slab to make it function safely, but it simplifies the design process. The top reinforcement for the part of the slab that is being designed for by a representative design strip will use the bar size at the required spacing given by the PCA slab results for the representative design strip. The bottom reinforcement in the joists will be what the design results state.

For floors 2 through 6 a live load of 80 psf for office corridors and a superimposed dead load of 15 psf were applied to the slab. However the roof terrace at the 6th floor receives a live load of 100 psf. Figure 17 shows the design strips for floors 2 through 6 used in PCA slab. Figure 18 shows which parts of the slab will use the design of the different design strips.

It was determined that 7" wide and 10" deep joist spaced at 20" would be acceptable to hold up to deflection criteria and withstand the slab shear forces. The top and bottom reinforcement and cut off points for the bars were calculated by PCA slab. Appendix B contains the second through sixth floor PCA slab design results for the design strips that will be used to design the slab sections. There are also cross sectional cuts of the slab displaying where the bars will be placed and cut off.

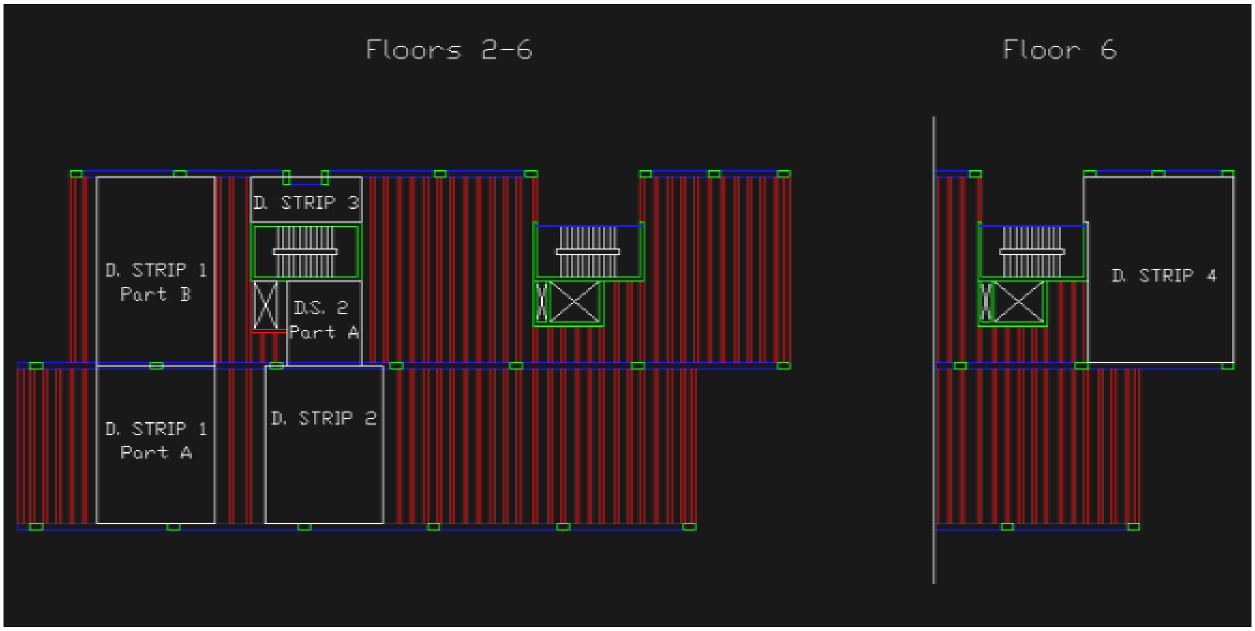


Figure 17– Design Strips Floor 2-6

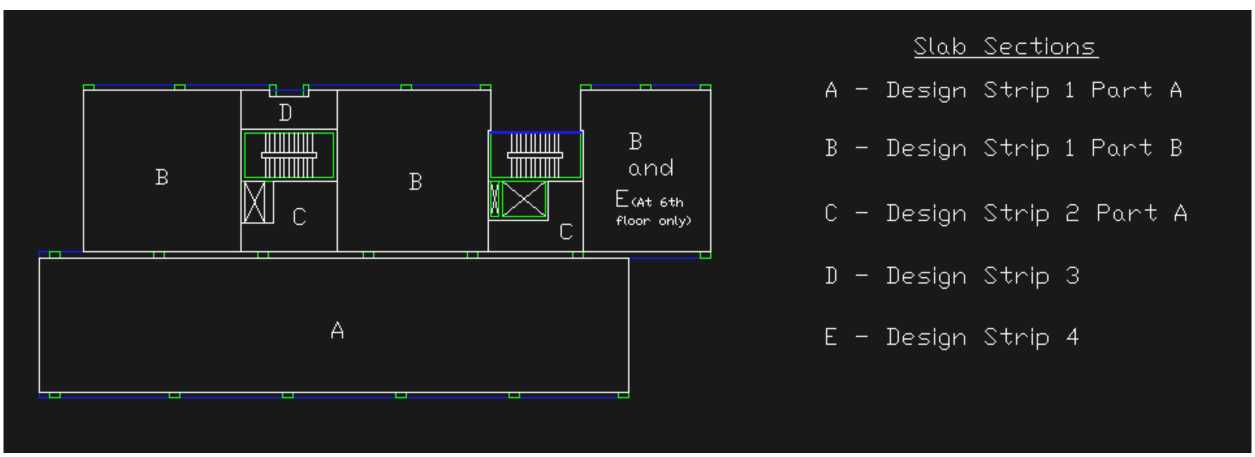


Figure 18– Slab Section Floor 2-6

The first floor is half slab on grade and half pan joist slab. The slab on grade is a 5" thick slab reinforced with #4 bars spaced at 16" in both directions. This is the same slab on grade used in the original design and since the loading on the 1st floor is still 100 psf live load this slab will be acceptable in this the redesign. The basement also uses the same 5" thick slab on grade reinforced with #4 bars spaced at 16" in both directions. This was the same slab on grade for the basement of the original design. The same pan joist slab dimensions that were used on floors 2 through 6 are also used on the first floor. Figure 19 shows the design strips for floors 1 used in PCA slab. Figure 20 shows which parts of the slab will use the design of the different design strips. PCA slab design results like the ones shown in Appendix B for floors 2-6 were also determined for the first floor slab.

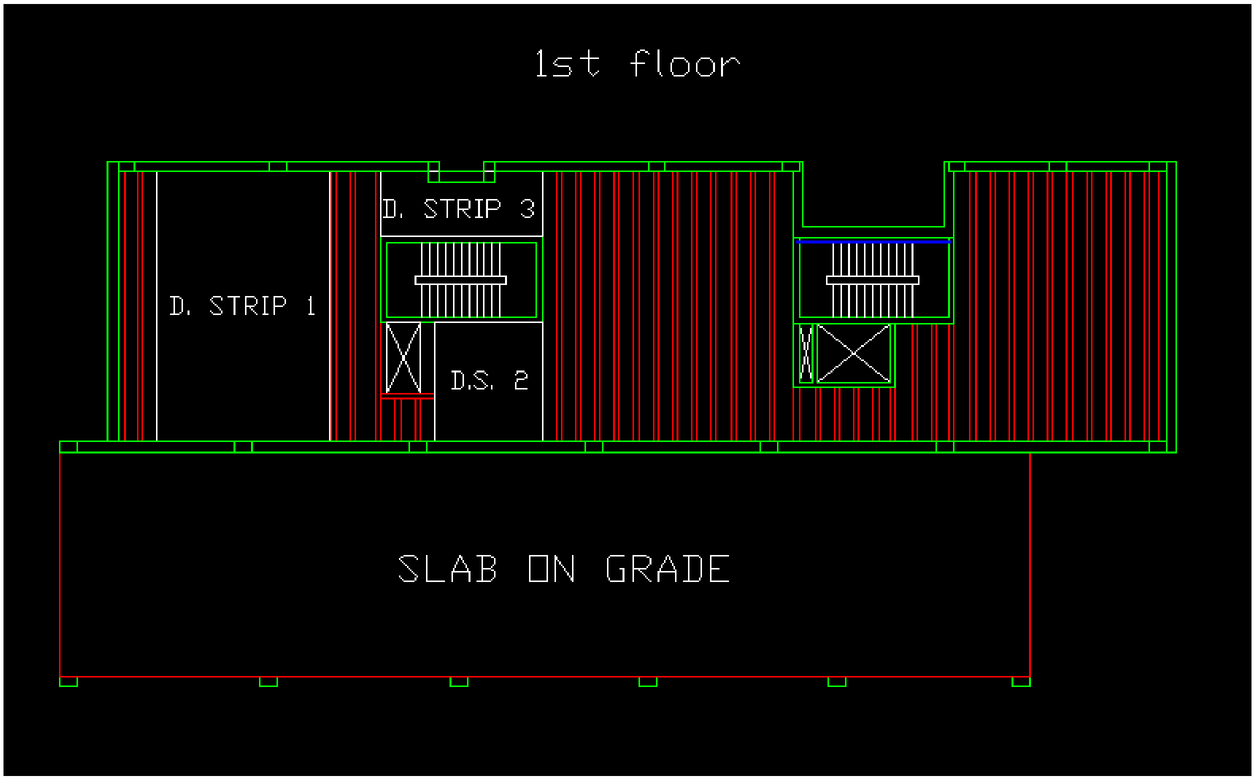


Figure 19– Design Strip 1st Floor

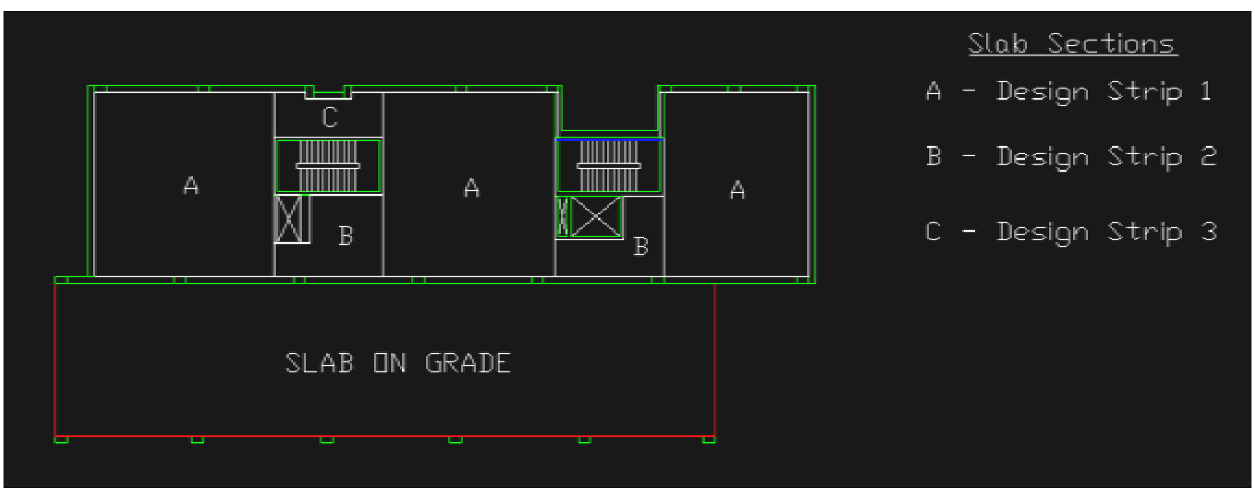


Figure 20– Slab Section 1st Floor

The same pan joist slab dimensions that were used on the other floors are also used on the roof. The roof snow load is 32 psf and the dead load is only from the mechanical loads, 5 psf. However, there are locations on the roof where there are heavier snow loads. Figure 21 shows which parts of the slab will use the design of the different design strips. Design Strip 2 Part A has a live load of 84 psf and Design Strip 3 Part A has a live load of 75 psf. Figure 22 shows which parts of the slab will use the design of the different design strips.

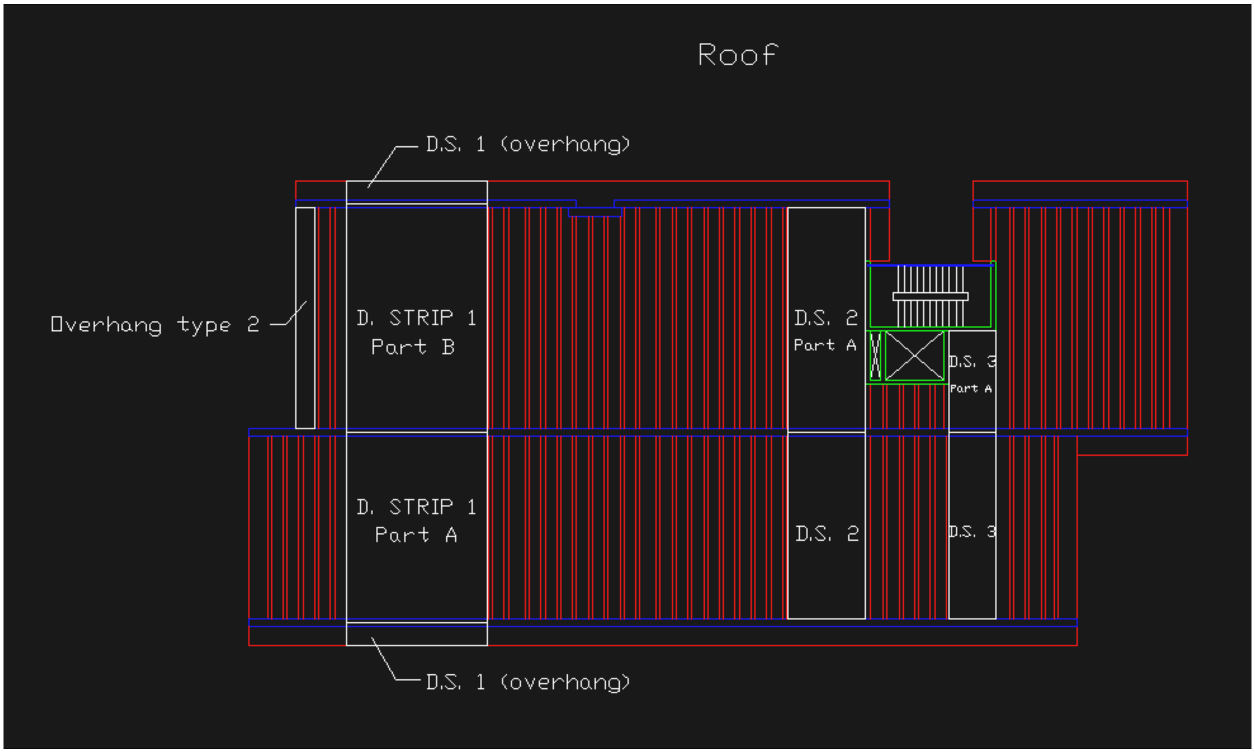


Figure 21– Design Strip Roof

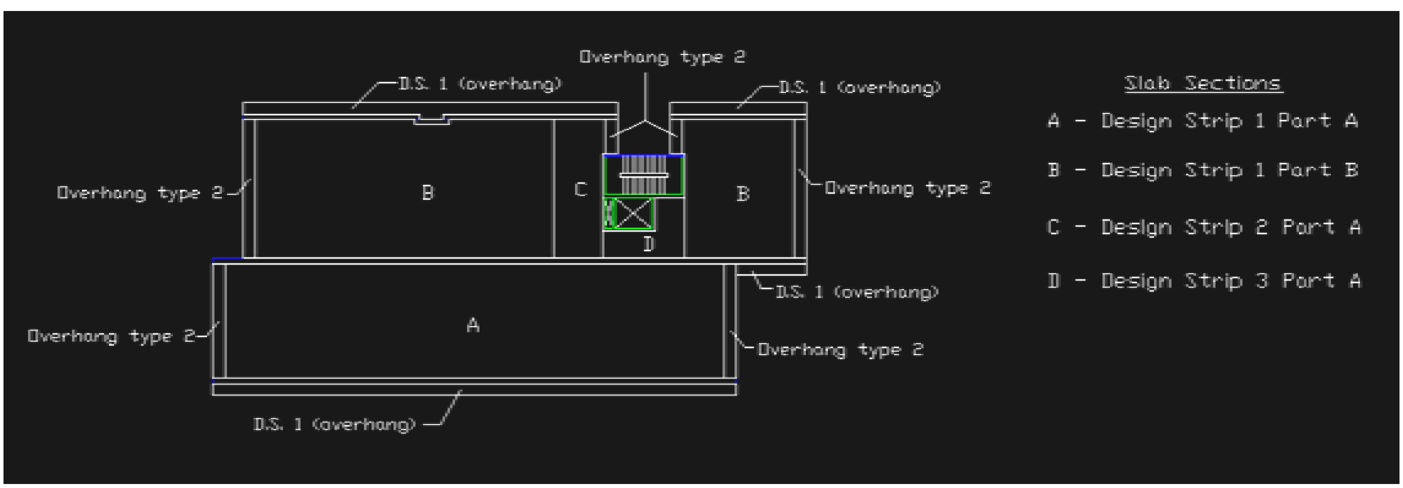


Figure 22– Slab Section Roof

Another design challenge was designing the roof overhangs. The overhangs are 6” thick concrete slabs that cantilever 2’-8” from the exterior spandrel beams. These overhanging slabs are labeled D.S. 1 (overhang) in Figure 22. In other locations the beams cantilever out 2’-8” past the columns and a 6” slab spans between the cantilevered beams to create the overhang. These overhanging slabs are labeled Overhang Type 2 in Figure 22. The overhangs were originally done with steel beams cantilevering to support the hollow core floor planks that acted as the overhang. PCA slab design results like the ones shown in Appendix B for floors 2-6 were also determined for the roof pan joist slab and overhangs. Also, in the existing design there is no roof over the 6th floor outdoor terrace. In the redesign the roof was continued over the 6th floor terrace. This will be discussed further in the architecture breadth.

A problem with this system is the difficulty of putting openings in the slab. Openings can be cored through the slab between the ribs and if the openings are too large to place in between the ribs then transfer ribs should be framed around the opening. In the redesign of the structure some of the openings of the existing structure are small but are not located in between ribs. Other openings are larger than the span between ribs and both types of openings had to be framed around. Figure 23 shows a typical floor framing plan with all of the openings and how they are framed. Appendix B.6 contains calculations for a design of framing around an opening.

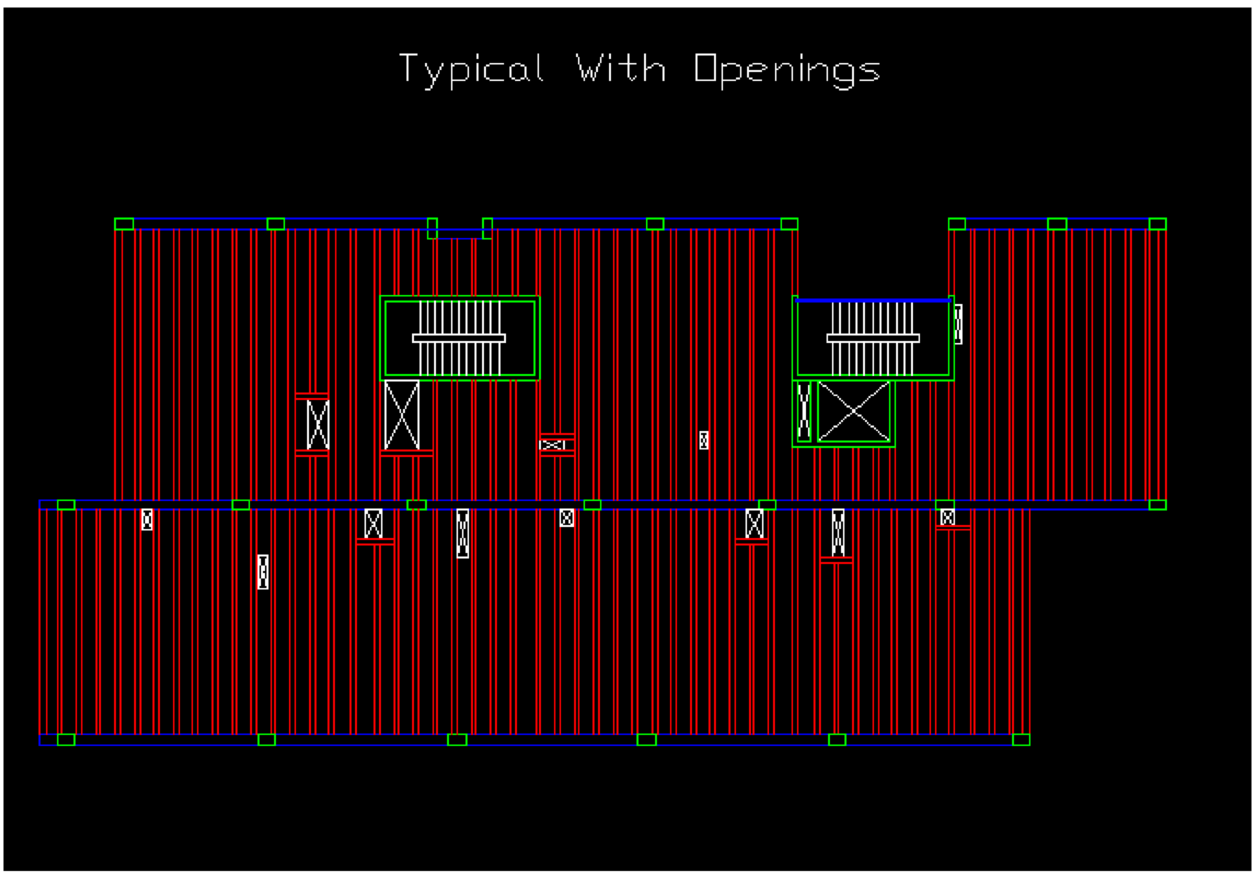


Figure 23– Typical Floor Framing Plan with Openings

The structure of the 2nd floor green roof had to be altered. Two beams were added, one between the east shear walls and one between the adjacent columns. These beams support the 4” thick concrete slab that cantilevers out 4’ from the structure. Appendix B.5 contains the PCA slab design results for the 2nd floor green roof.

Continuous Beams

The girders that directly support the pan joist slab are designed as continuous beams. This means that the reinforcement extends through the girders at the vertical supports. By extending the reinforcement like this it provides continuity from one member to the next through the support region. This continuity will cause loads on one span to spread to all the other spans. On simple spans when one span is loaded all the other ones will remain strait. For simple spans the design values, moments and shears, can be found from the loads acting on the span and the length of the member. Simple calculations such as M = (w*L^2)/8 and V = w*L/2 can be computed to find these values. Continuous beam are however statically indeterminate and not only the loads and member dimensions effect it’s analysis but also joint rotations. Because of the continuous beam’s statically indeterminate nature the design values will be determined by the use of a finite element analysis computer program named SAP2000. Pattern loading will be used to determine the correct design values.

As discussed above loads on one span will cause moments and shear forces on other spans. Dead loads will be applied continuously to all of the span. However, the live loads will not always be acting at the same time. When spans are loads every other span it creates large positive bending moments in the spans that are loaded. This loading case is shown in Figure 24. When spans are loaded right next to each other it creates the maximum negative bending moment at that support. This loading case is shown in Figure 25. This is how the loads will be applied to the SAP model to determine the design values.

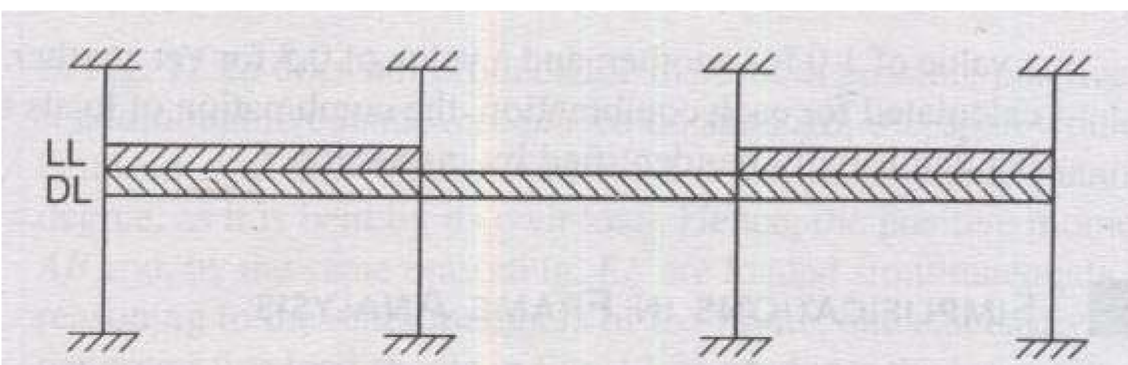


Figure 24– Pattern Loading for Positive Moment

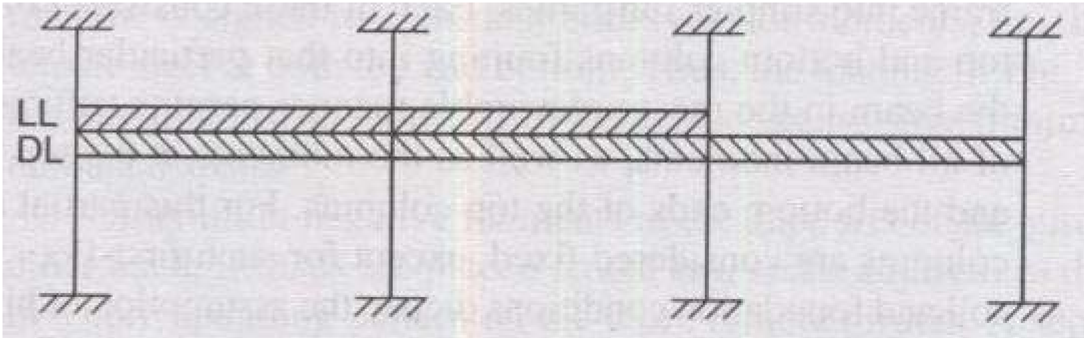


Figure 25– Pattern Loading for Negative Moment

For the Gateway Commons redesign, the tension reinforcement for all of the spans of a continuous beam will be designed for the largest positive bending moment acting on that continuous beam. The compression reinforcement at all the supports on a continuous beam will be designed for the largest negative moment acting on the continuous beam. This is a conservative approach for the flexure design of the girders but it will save time.

The sizes of the girders were determined based on deflection criteria and architectural constraints. As shown in Figure 26 for the middle girders, if columns and girders were placed inside the apartments the layout of fixtures and mechanical openings would have to be altered. Columns should not extend more than 6” into the hallways to allow for a 5 foot wide hallway. This means that the girders have the same requirements. The existing 8” thick wall between the corridor and the apartments and the 6” maximum hallway penetration allow for 14” wide middle girders. The top and bottom girders and columns will not affect the interior architecture by extending 6” into the rooms therefore 14” wide top and bottom girders are allowed. ACI table 9.5 shown in Figure 27 was used to calculate a beam height suitable for deflection. Top girders are allowed to have a minimum height of 12.3” and a height of 16” is chosen for a height. Middle girders are allowed a minimum height of 16.7” and a height of 18” is chosen. Bottom girders are allowed a minimum height of 15.8” and 16” is chosen as a height. In summary the top and bottom girders are 14”x16” and the middle girders are 14”x18”. In constructing a pan joist system it is preferred that the beams be the same depth as the slab but it is not necessary. If the beams were 14.5” deep than they would have to be wider to support the loads and this is not possible due to the architectural constraints.

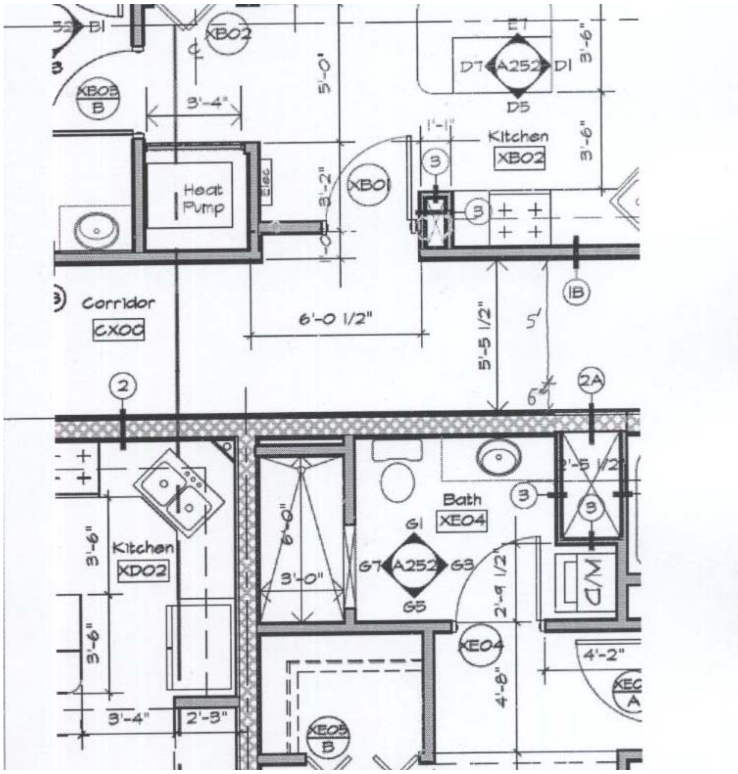


Figure 26– Architectural constraints

TABLE 9.5(a)—MINIMUM THICKNESS OF NONPRESTRESSED BEAMS OR ONE-WAY SLABS UNLESS DEFLECTIONS ARE CALCULATED

	Minimum thickness, *h*			
	Simply supported	One end continuous	Both ends continuous	Cantilever
Member	Members not supporting or attached to partitions or other construction likely to be damaged by large deflections.			
Solid one-way slabs	ℓ/20	ℓ/24	ℓ/28	ℓ/10
Beams or ribbed one-way slabs	ℓ/16	ℓ/18.5	ℓ/21	ℓ/8

Notes:
Values given shall be used directly for members with normalweight concrete (w_c = 145 lb/ft^3) and Grade 60 reinforcement. For other conditions, the values shall be modified as follows:
a) For structural lightweight concrete having unit weight, w_c, in the range 90-120 lb/ft^3, the values shall be multiplied by ($1.65 - 0.005w_c$) but not less than 1.09.
b) For f_y other than 60,000 psi, the values shall be multiplied by ($0.4 + f_y/100{,}000$).

Figure 27– ACI 318-05 Table 9.5

A SAP model of the continuous beams was created. The columns above and below the beams were modeled with fixed supports to create a frame as shown in Figure 28. The girders were defined as concrete beams with of their actual dimensions. The self weight of the girders was determined by SAP and the dead loads on the girders from the slabs were assigned to the beams, see Figure 29. Next, live loads were added to the beams in pattern loading. The pattern loading in Figure 30 will find the maximum negative moment at gridline x6. Next, the program analysis is run and a diagram of the moments acting on the girders can be displayed, as shown in Figure 31. The span adjacent to x6 that creates the largest moment can be clicked on to bring up a moment diagram of the span for a more clear view. The span to the right was chosen and a negative moment of 130.5 kip-ft acts on the girder at that support, as shown in Figure 32. This was done for all of the possible patterns of loading and the maximum negative and positive moments were chosen to design the girder. The girders for floors 2-6 would all be the same because they are the same size and loading. These girders will also be used for the roof because the roof loads are less than the floor loads. There are no beams on the first floor because the slab on the first floor frames into the retaining wall. This will be discussed further in the foundations section. Calculations for the girders can be found in Appendix C.

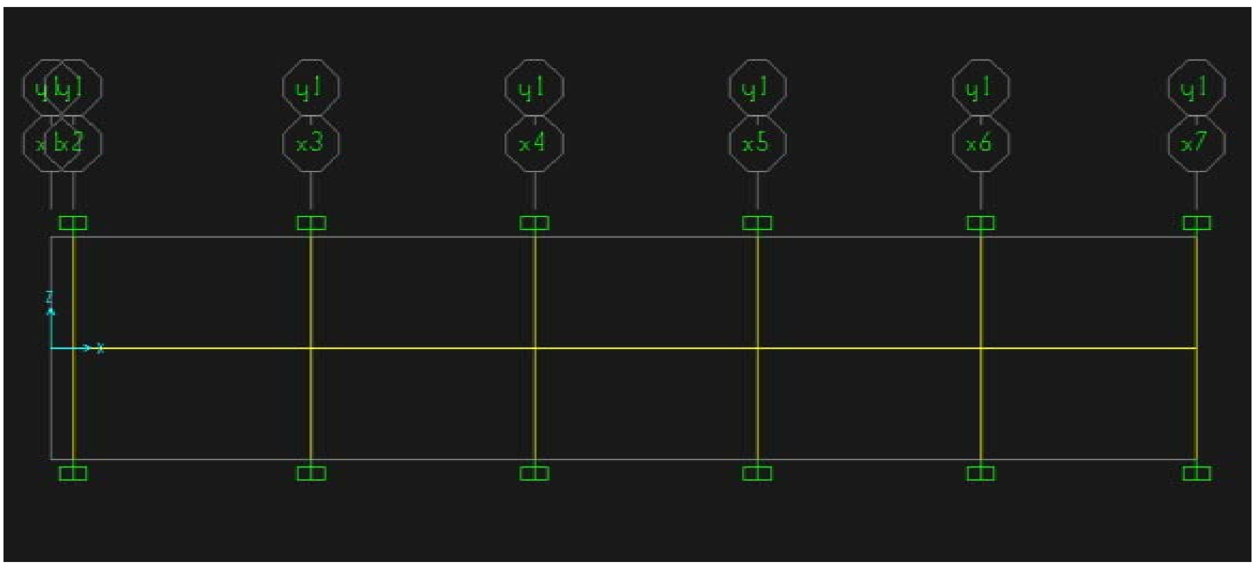


Figure 28– SAP Frame for Continuous Beam

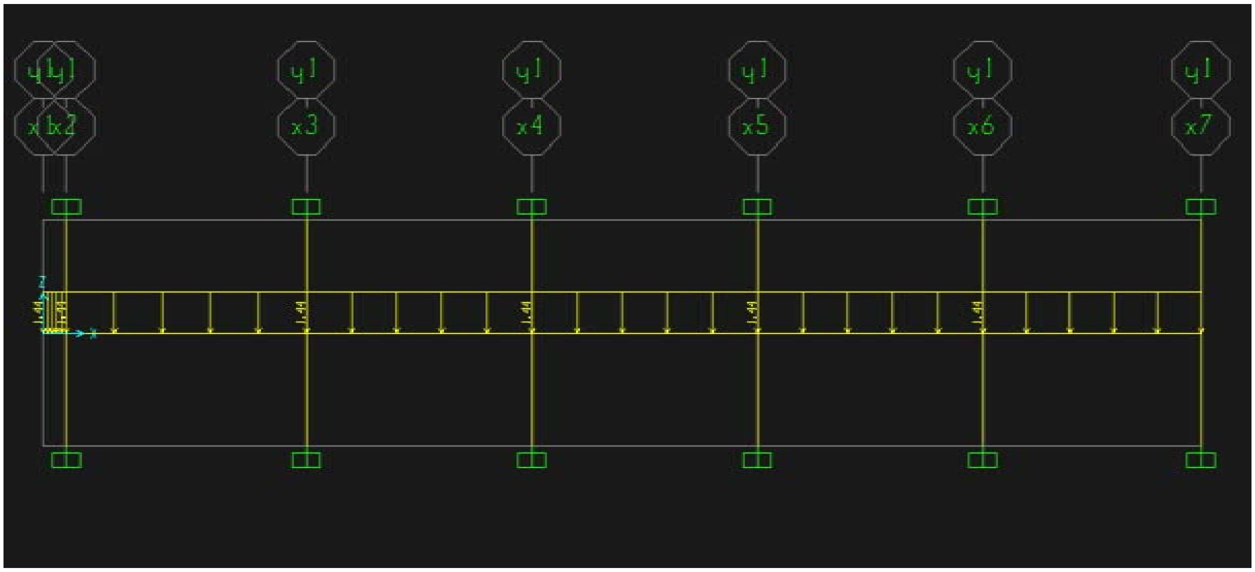


Figure 29– Dead Load on Continuous Beam

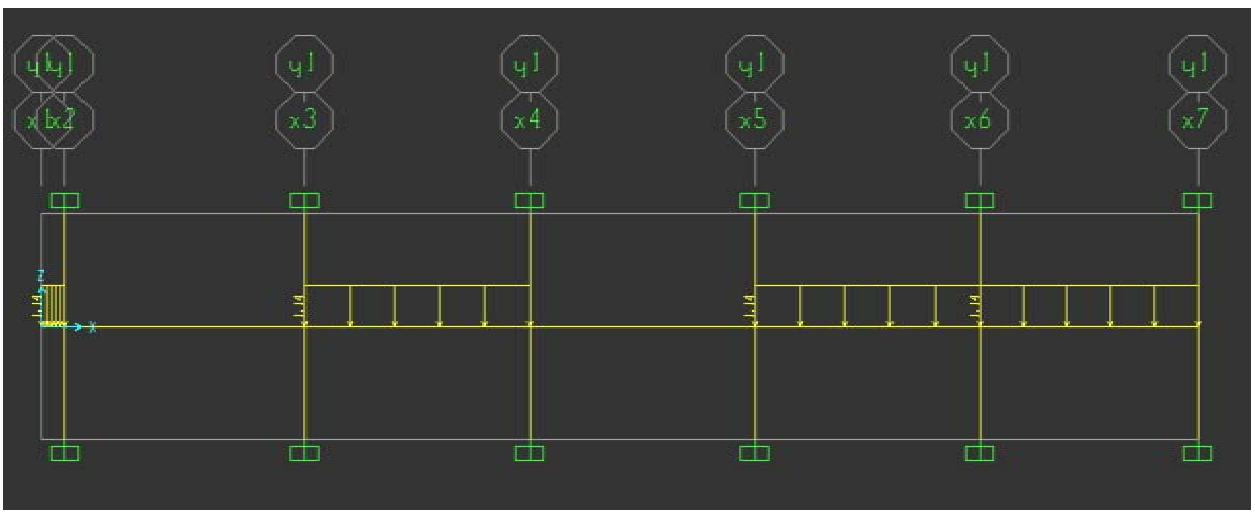


Figure 30– Pattern Live Loading on Continuous Beam

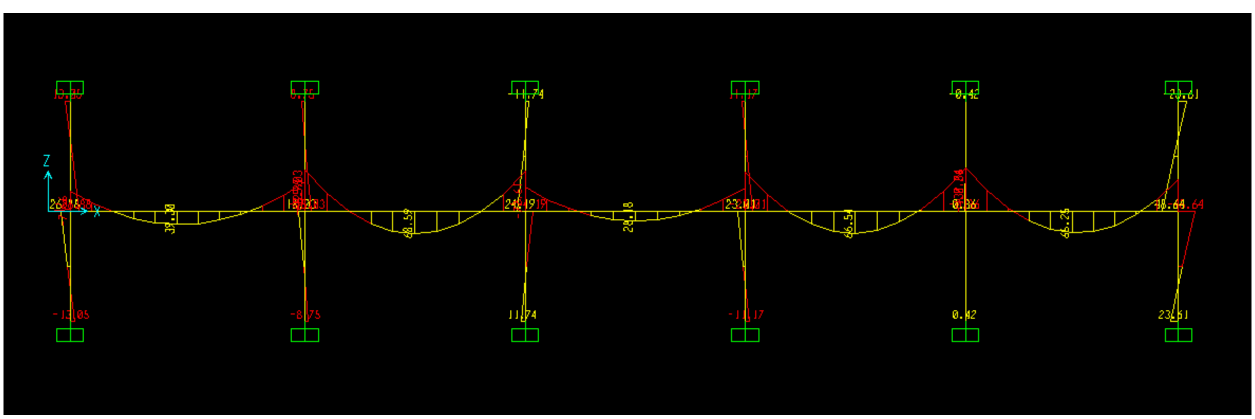


Figure 31– Moment Forces on Frame

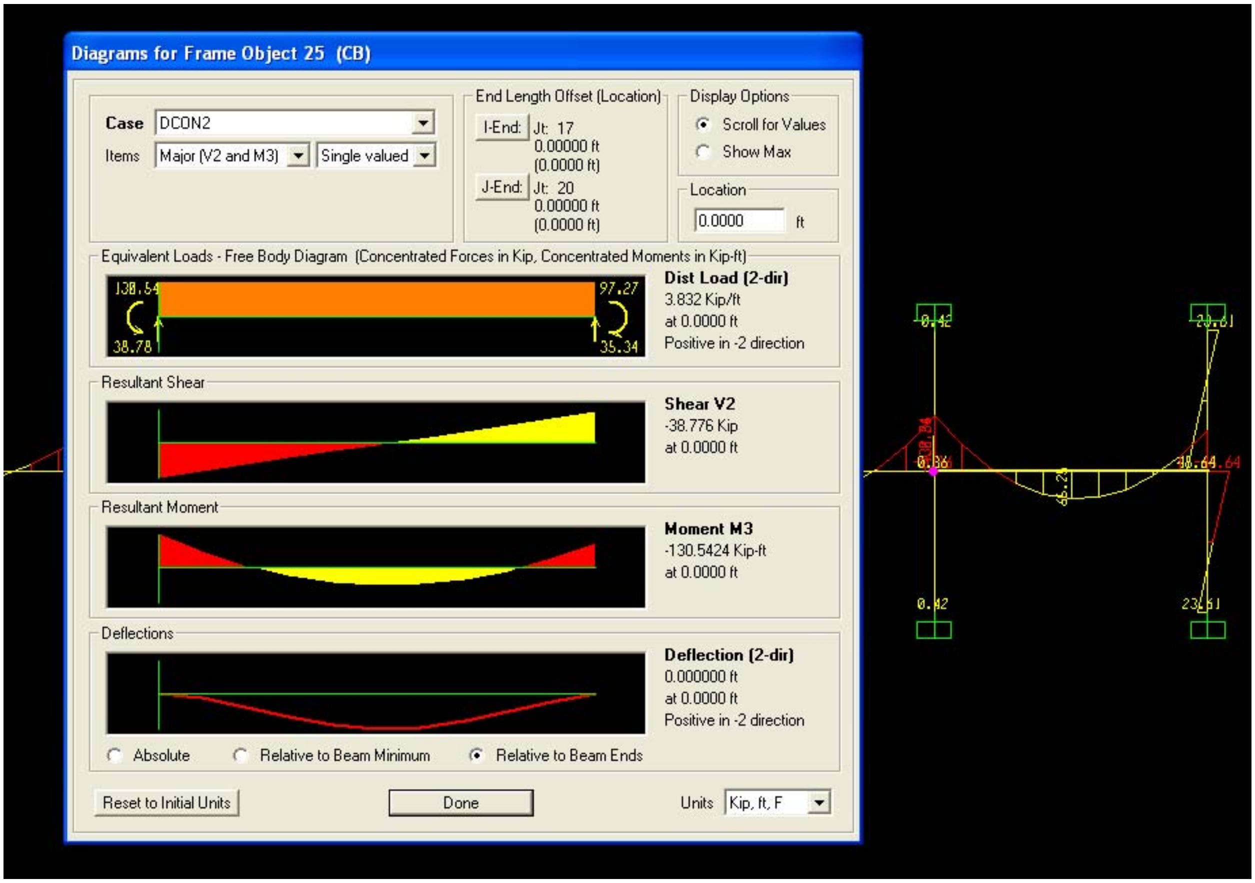


Figure 32– Moment Diagram on Single Span

The shear design for the girders was controlled by torsional effects specifically compatibility torsion. In calculating torsion moments on the girders are determined by using moment coefficients from ACI 318-05 8.3.3. The moment is used to determine Tu. Then Tth is calculated and if Tu>Tth than torsional reinforcement is required. The size of the girder is also checked to determine if it is big enough for torsion. For all of the girders torsional reinforcement was needed and the sections were large enough to resist the torsional forces. Calculations for torsion can be found in Appendix C.

Columns

To maintain a floor to floor height of 11' columns that support 18" deep girders will be 9'-6" while columns that support 16" deep girders will be 9'-8". The columns used in the redesign have a dimension of 14"x24". The constraints that allow for a maximum width of 14" are discussed in the Continuous Beam section above. The SAP models that were used for the girders were used to find the moment and axial forces on the columns. Live loads were set up like in

Figure 30 to determine the maximum moment on the column. The difference of the moments on either side of the column is determined and the resultant shear values in the beam diagram at the column face are taken as the axial values. These values can be easily determined by using beam diagram like in Figure 32.

PCA column is then used to design the reinforcement for the columns. Since the loads in SAP are already factored the moment and axial value will be input into PCA column as factored loads. The design option is chosen, column dimensions are input, the rebar at equal spacing function is selected, and tied confinement is selected. The program is run and the reinforcing for the column is show on the screen. An interaction diagram is also created plotting axial and moment values. If the point that the loading creates is inside the interaction diagram than the design will work. Many of the columns are reinforced with 4 #9 bars because the low moment and axial values on the columns. However, columns at the end of continuous beam spans do not have a beam on both sides to balance out the moment acting on the column and will have to be reinforced more heavily. A good example of this is the column on the right end of the middle span on the first floor. It is reinforced with 6 # 10 bars. Columns in the basement will be discussed in the foundations section of this report. Column compatibility with the existing architecture will be discussed in the architecture breadth.

Lateral Force Resisting System

The lateral force resisting system used in the redesign is 8” thick ordinary reinforced concrete shear walls. These shear walls are placed around the two stair towers. Each floor has an opening for a door in the east shear walls and west shear walls. The columns are not designed to resist the lateral loading therefore the shear walls act as the main lateral load resisting system and are designed to resist the total lateral load in both directions. The west shear walls contain two walls in the north-south direction and two walls in the east-west direction. The east shear walls contain two walls in the north-south direction and 1 wall in the east-west direction. The slab is connected to the shear walls and acts as a rigid diaphragm transferring the lateral loads onto the shear walls.

After lateral loads were determined for each story an ETABS model was created. This model was built for the purpose of finding the shear forces and moments of each shear wall. In order to do this, ETABS distributed the lateral forces acting at each story onto each shear wall according to their stiffness. These values along with axial loads were used to design the shear and flexure reinforcement for the shear walls.

The ETABS model was started by creating the shear walls and rigid diaphragm. The piers for the walls are labeled then the diaphragm and walls are meshed. Next the lateral loads are added to the program and applied to the walls. User defined wind and seismic forces are applied to each story for both directions. Earthquake forces at each floor can be determined by ETABS based on IBC calculations. The ETABS calculated earthquake forces were calculated and would be compared against the user defined loads. ETABS can also calculate wind forces based on ASCE 07. All 12 wind cases described in Figure 6.9 of ASCE are calculated by ETABS. These were calculated and compared to the user defined wind load output. After the loads were determined the program was run and output data for shear and moments on each shear wall at each story is displayed.

Two different models were created in ETABS, one for flexure design and one for shear design. For the shear design model each wall on a floor was assigned a different pier label as shown in Figure 33. After ETABS analyzes the structure shear forces for each wall were given and each wall was designed for shear reinforcement. The shear forces were due to wind and seismic forces. Shears due to live and dead load were considered negligible.

Openings in walls are set 8”away from an end of the wall. The wall then has two pieces, the large rectangular wall next to the opening and the small rectangular piece above the opening. In Figure 34 an elevation of the walls with the most openings is shown. The shear reinforcement for the large rectangular part of the wall (P1 or P6) will be designed for and that design will extend into the small rectangular part (S1 and S2). The opening will also be framed around with 2 # 5 bars per ACI 22.6.6.5. Where possible the bars will extend 24” past the corners of the wall and where it is not possible the bars will be bent 90 degrees and developed. Shear reinforcement will be designed by hand see Appendix X for calculations.

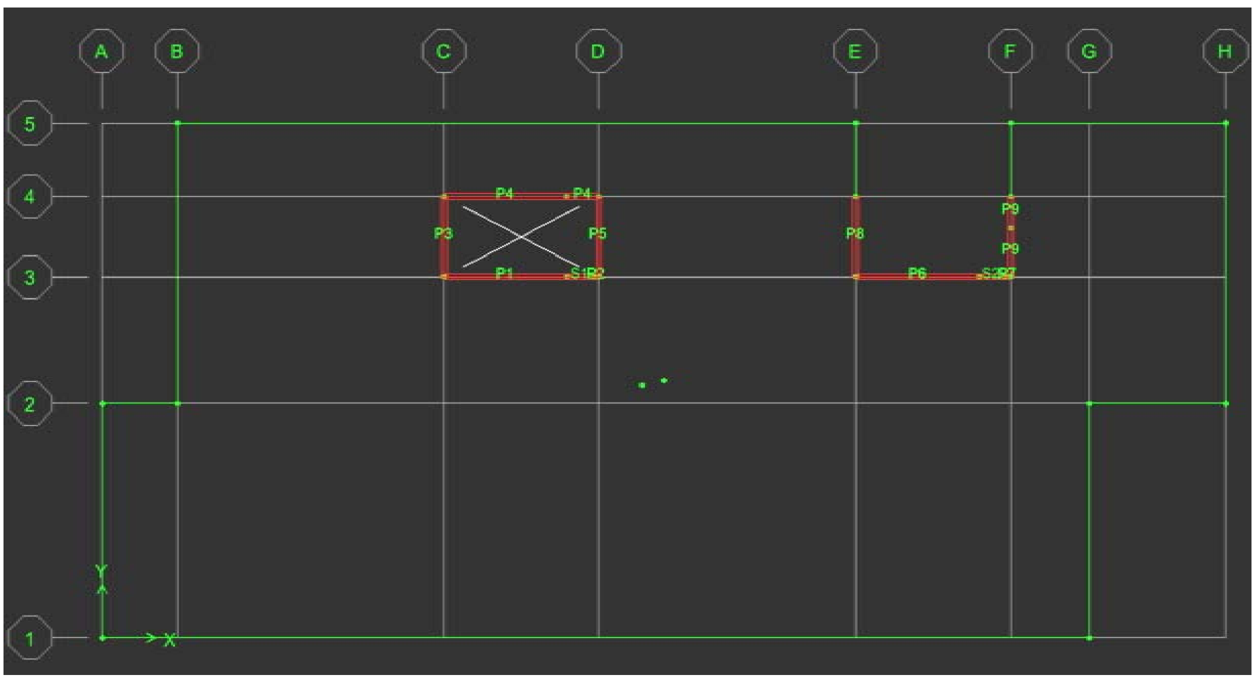


Figure 33– Shear Model Pier Label

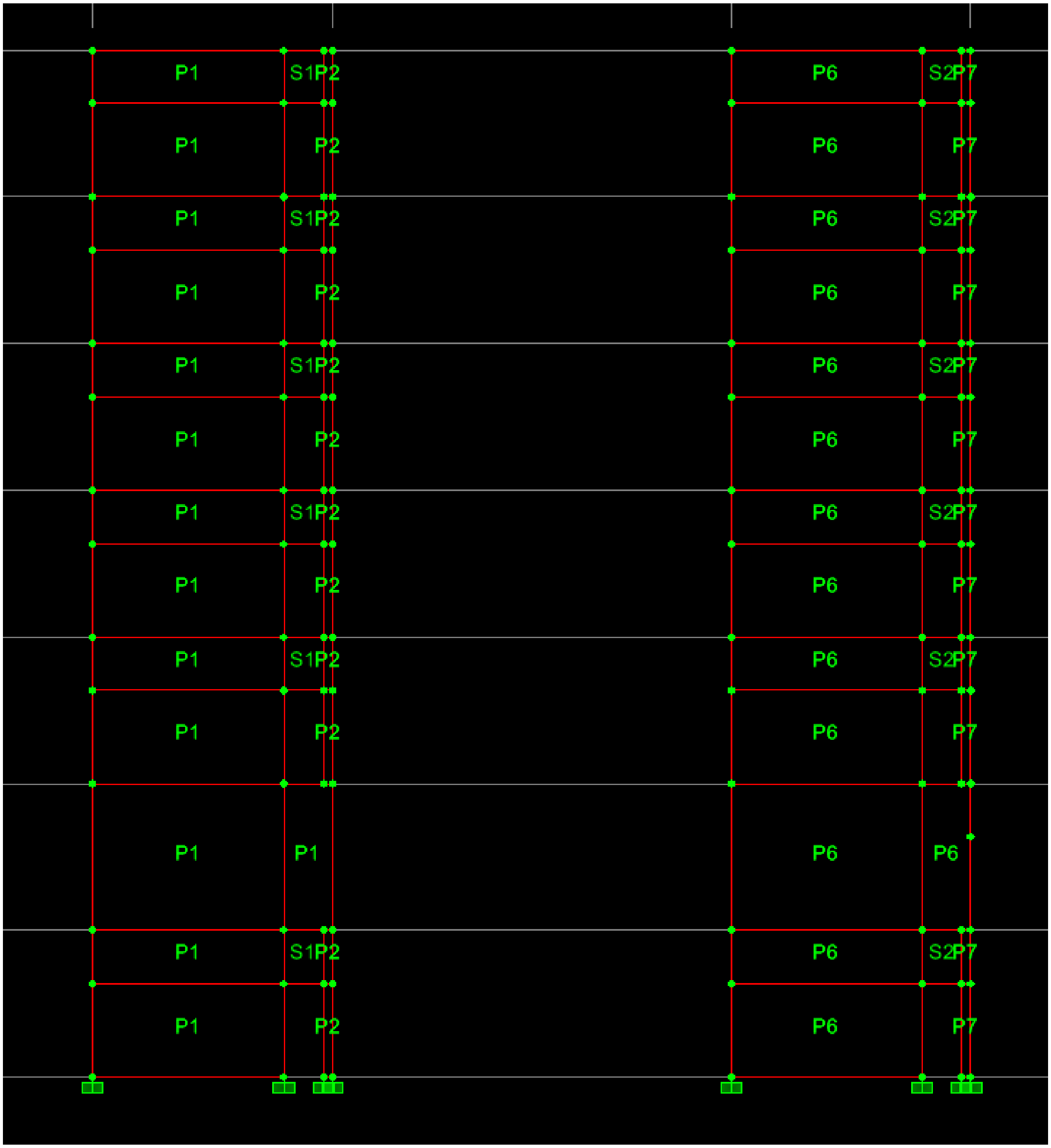


Figure 34– Shear Model Elevation 3

Torsion was considered when determining the shear forces on each wall. Torsion occurs because the lateral forces acting at the center of mass of the floor act eccentrically from the center of rigidity of the floor. This creates a twisting which cause shear forces in the shear walls. The centers of mass and rigidity are shown in Figure 35. Since the distance between the two centers is relatively close with an average distance of (1.02’,14.4’) between the two points, torsion forces were added to the direct shear for each wall and not considered by itself.

Story	Diaphragm	XCM	YCM	XCR	YCR
STORY7	D1	752.225	351.687	734.931	523.167
STORY6	D1	752.225	351.687	736.988	525.388
STORY5	D1	752.225	351.687	743.227	526.291
STORY4	D1	752.225	351.687	753.854	526.02
STORY3	D1	752.225	351.687	772.302	524.058
STORY2	D1	752.225	351.687	803.412	522.12
STORY1	D1	752.225	351.687	865.385	536.79

Figure 35– Centers of Mass and Rigidity

For the flexure model all of the west shear walls on each floor were labeled as the same pier (P1). Since the east shear walls are a C shape. On one side of the door opening is an L shaped wall and it was labeled the same pier for both walls that made up the L shape (P2). On the other side of the opening is a single shear wall (P3). Figure 36 shows the pier labels for the flexure model. The small rectangular piece above the opening will be treated the same way as it was in the shear design. PCA Column was used to design the shear walls for flexure. The program was set to inspection and the wall shape was drawn out. The vertical shear reinforcing was added to the wall. The wall shape of pier 2 with the added reinforcing is shown in Figure 36. The axial forces on the wall and the moments at the top and bottom due to wind and seismic were entered as service loads and load combinations were input into PCA column to factor the loads. The moments due to dead and live loads were considered negligible. If the wall failed when looking at the interaction diagram then larger bars or shorter spacing was used and the analysis was done again.

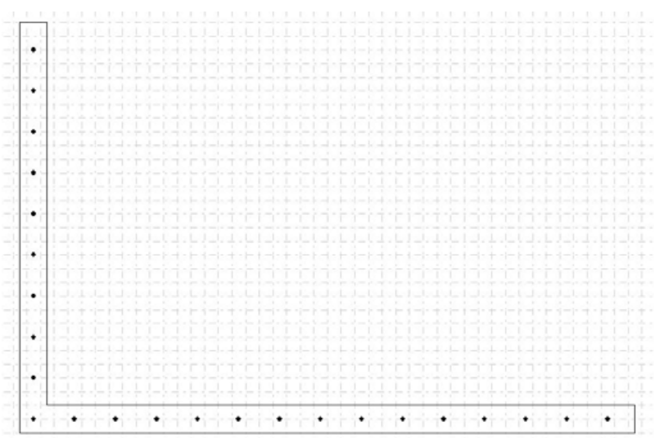


Figure 35– Shear wall section in PCA column

All of the horizontal reinforcement and most of the vertical reinforcement are two curtains of # 4 bars spaced at 18". The reinforcement is so minimal because of the relatively light lateral loading and the added stiffness due to walls spanning in opposite directions being connected with one another. Pier 3 was designed as an isolated shear wall and it was one of the only walls to see an increase in flexural reinforcement. See Appendix D for a summary of the shear and flexure design values and a summary of reinforcement.

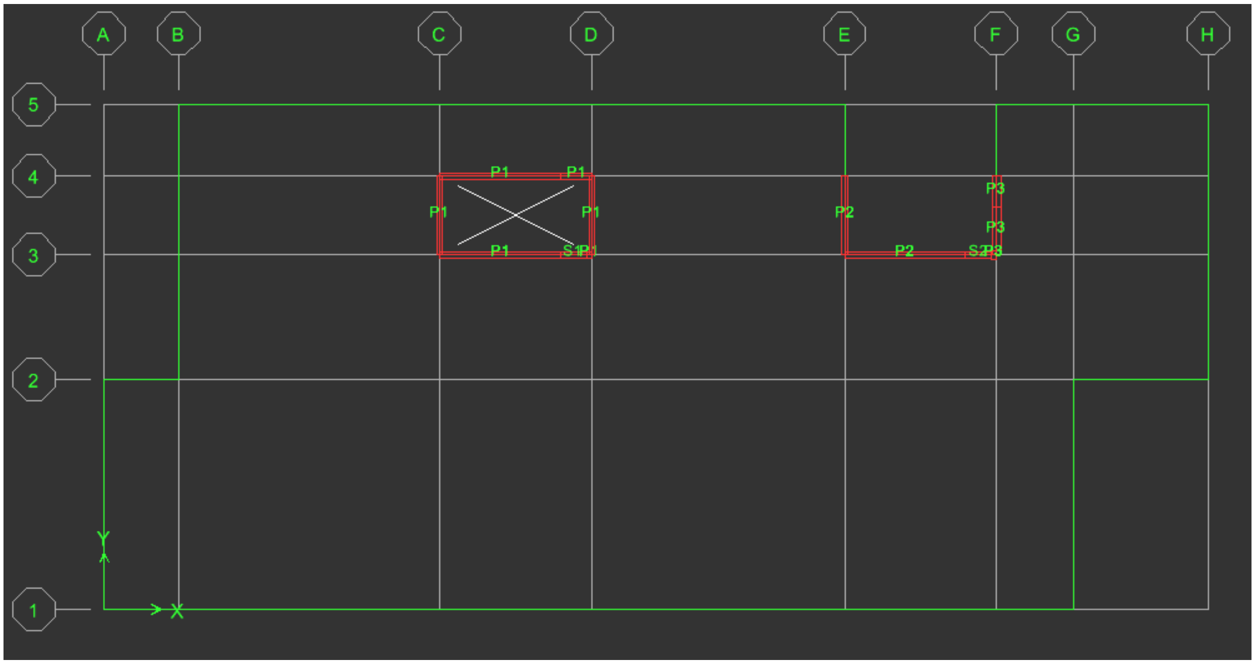


Figure 36– Flexure Model Pier Label

The allowable story displacement at the top of the building h/400 = 1.98”. All of the displacement values at the tops story were less than 1” therefore the displacement is acceptable.

Footings

The columns that support the bottom girders are supported by 9’x9’x3’ spread footings at the first floor. The foundation plan for the spread footings is shown in Figure 37. The retaining walls will be the same ones that were used in the existing design since the soil will be the same. The columns supporting the middle and top girders will be integrated with the retaining wall. Where the columns bear on the retaining walls the column reinforcing will continue through the retaining wall and a column sized section of the retaining wall will be designed as one. A check was done to make sure that the span of retaining wall between the columns would be able to support slab loads with the existing reinforcing. The loads were determined and a 1’ section of the wall was checked on PCA column. It proved that the retaining wall will be able to support the slab loads with the existing rebar design. The foundation plan for the strip footings is shown in Figure 38. Calculations for footings can be found in Appendix E.

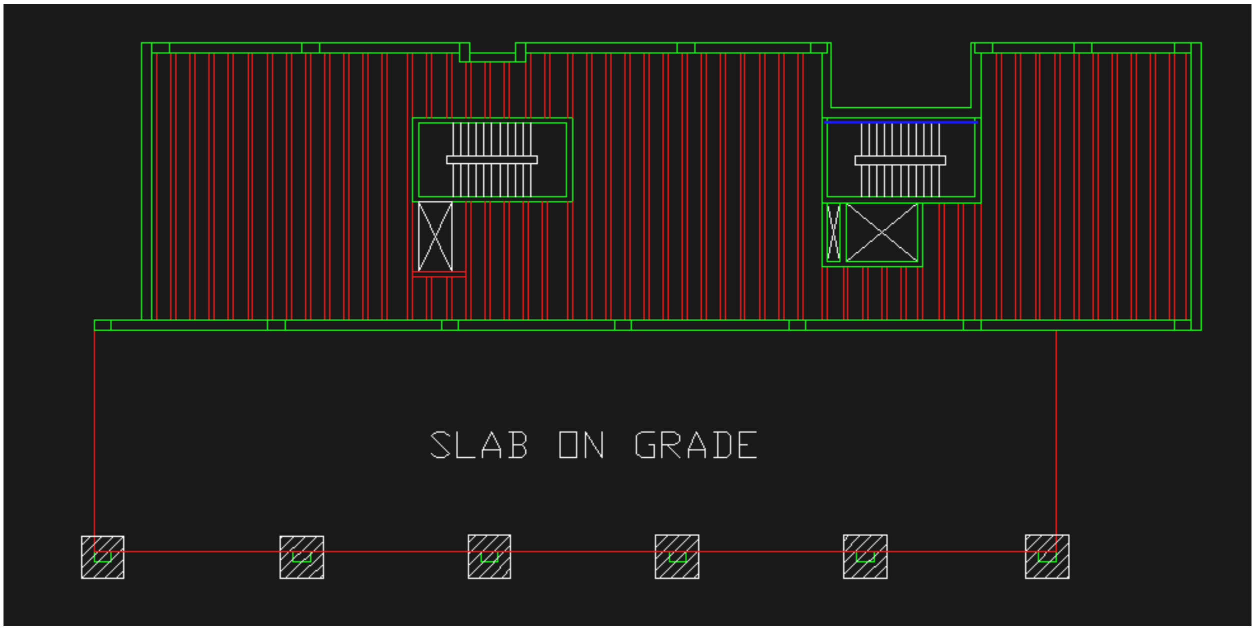


Figure 37– Spread Footing Foundation Plan

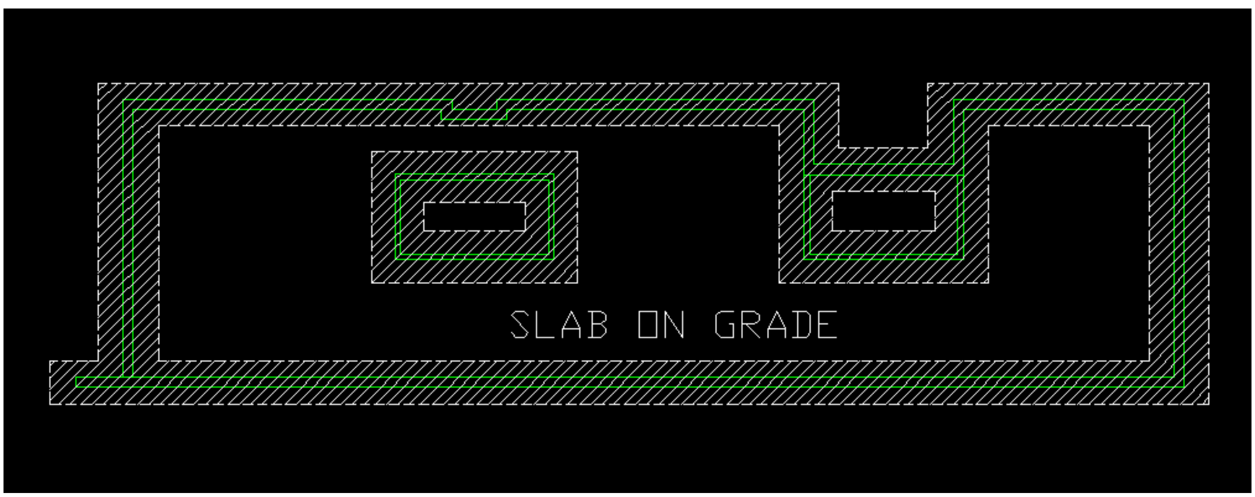


Figure 38– Strip Footing Foundation Plan

Architectural Breadth

The architecture of the existing building will be maintained with the change of structure. However, it was decided that there be a roof put over the 6^{th} floor roof terrace. This will create more space for a different building type to use when the architecture is redesigned. This area in the existing architecture can still be open to the environment by putting railings around the perimeter of the space and leaving it open to the outdoors instead of putting walls up. The area could still be used as a community gathering place and it would be able to get use during all kinds of weather conditions because there is now a roof. Structural members from the redesigned structure overlapping with the existing architecture can be found in Appendix F.

The building market in Ithaca is currently more profitable for housing than office building. However, if this changes or if another possibility arises that would be a profitable new use of the building the ability to redesign the building for a whole different use is now possible. The architecture of the building was redesigned as an office building to show that there are now new possibilities for versatility in architectural redesign that never existed with the old structure.

The office building redesign will be a mixed use building. Two retail spaces will be provided on the first floor along with a café. Floors 2-6 will be office spaces. Floors 2-5 will be broken into two separate units and the entire 6^{th} floor will be a single office unit. The basement will be used for storage and mechanical spaces. Bathrooms are in the same location on each floor to allow for piping runs in the same area. Each office unit will have a receptionist area, kitchen, conference room, storage, and office space. The exterior of the building will be all glass on the first floor and the exterior columns will be located outside of the glass walls. The rest of the floors will be brick façade with lots of window space. This amount of windows would not be possible with CMU bearing wall structure of the existing building. Floor plans and exterior elevations of the office building redesign can be found in Appendix F.

Construction Management Breadth

Changing a building's structure will affect the cost and schedule of a project. The cost of the new structure will be compared to the cost of the existing structure. Only the structure will be taken account in these costs. Cost and schedule information of the existing structure was provided by Northeast Construction Services. RS Means Facilities Construction Cost Data 2006 was used to get values for estimating the cost of the structure and scheduling information. Microsoft Project was used to put the schedule information together and create the schedule for the new structure.

Cost

The cost of the existing structure included labor and materials for concrete walks, concrete footings, cast in place foundation walls, slab on grade, elevator pit, cast in place masonry wall caps, concrete reinforcement, pre-cast concrete planks, masonry, and structural and miscellaneous steel. The price came to $2,078,841. The cost of the new structure will be $1,293,136. The total cost savings of switching the structure from precast hollow core concrete planks on CMU walls to a concrete pan joist system is $785,705. Additional information about the cost estimates can be found in Appendix G.

Schedule Impact

Both the existing and new structure set their starting dates for the construction of the structure at December 7, 2005. The existing structure was completed by October 4, 2006 and the new structure was completed by July 17, 2006. The new structure was able to be completed 79 day before the existing structure would have finished. Copies of the schedules can be found in Appendix G.

Conclusion

A thorough redesign of the structure of the Gateway Commons building was done with the main purpose being to create a structural system that will allow for more versatility in architecture redesign possibilities and minimally affect the existing architecture of the building. A one way concrete system was determined to be the best structure to complete this goal. A pan joist system was used based on the design criteria. This structure was compared to the existing hollow core concrete floor planks on CMU walls. The two systems were compared on construction cost, schedule impact, and versatility in architectural redesign.

Instead of having load bearing walls in various places throughout the structure and designing some of them for lateral resistance, concrete shear walls around the stair towers supplemented the large amount of masonry shear walls scattered throughout the structure. The design of the retaining walls was able to be used over but spread footings for the columns on the opposite side of the building had to be designed. The reduction of weight caused a reduction in the seismic forces acting laterally on the building however it was still found to control the design in the east-west direction.

The use of columns instead of walls will not only present a more open floor plan but will also allow for more versatility of the exterior façade. Lots of windows were used in the office space redesign. The structural bearing walls in the existing design would not be able to be removed to provide the window spaces used in the redesign or any other openings that an alternative design would require.

The use of the new structure with the existing architecture will change the 6th floor roof terrace but an alternative solution was discussed in the architecture breadth. The pan joist slab will cause an increase in floor to floor height and cause the building to be 6' taller than zoning allows. This would be able to be worked out with the Ithaca Board of Zoning Appeals and does not seem to be that big of a problem.

The new structure was able to be constructed for less than the existing one and the schedule for the structure was able to be reduced from the existing one. In this case, changing the building structure would be extremely economical. Not only the cost of the building is reduced but since it will be able to be built earlier it will be able to start making revenue sooner. Additionally, the potential for profit is larger now that the structure has the possibility of being redesigned for the most profitable use of the building.

ACKNOWLEDGEMETNS

I would like to mention the following people whose help throughout this project I have appreciated from the deepest level of my soul.

Professor Parfitt
Professor Hanagan

Thank you for your help during the past 2 semester.

Steve Reichwein

Your answers for all of my stupid little question I continuously had throughout this project added up to an enormous amount of help. On top of that I want to thank you for all of your help with my ETABS model. Thanks to you I finally have a comprehensive understanding of the program.

Jamie Manner

Thank you for all of your emotional support during the last couple of months. You've picked me up when I was down, calmed me down when I was stressed, and all from 200 miles away. You stuck with me when I was at my worst and make me the best I can be. I love you more than word can describe.

APPENDIX

APPENDIX A

A1. Wind

Wind Loads

$V = 90$ mph

Exposure B, Case 2
enclosed building

$K_d = 0.85$

$I = 1$

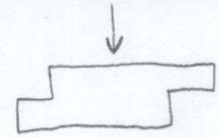
$K_{zt} = 1$

$G = 0.85$

$GC_{pi} = \pm 0.18$

$K_h = 0.874$ @ 66 ft

N-S



$\frac{L}{B} = \frac{60.8}{130.5} = 0.47$

direction	for use with	Cp
windward	q_z	0.8
Leeward	q_h	-0.5
Side	q_h	-0.3

$$q_z = 0.00256 (K_z)(K_{zt})(K_d)V^2 I \ [lb/ft^2]$$

$$= 0.00256 (K_z)(1)(0.85)(90)^2(1)$$

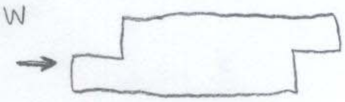
$$= 17.63 (K_z) \ [lb/ft^2]$$

$$q_h = 0.00256 (K_h)(K_{zt})(K_d)V^2 I$$

$$= 0.00256(0.874)(1)(0.85)(90)^2 1$$

$$= 15.4 \ [lb/ft^2]$$

E-W



$\frac{L}{B} = \frac{130.5}{60.8} = 2.15$

direction	for use with	Cp
windward	q_z	0.8
Leeward	q_h	-0.7
Side	q_h	-0.3

$p = q_z G C_p \pm q_h GC_{pi}$ windward

$p = q_h G C_p \pm q_h G C_{pi}$ Leeward & side

N-S

Windward $p = 17.63(K_z)(0.85)(0.8) + 15.4(0.18)$
$= 11.99 K_z + 2.77$

Side $p = 15.4(0.85)(-0.3) - 15.4(0.18)$
$= -6.7$

Leeward $p = 15.4(0.85)(-0.5) - 15.4(0.18)$
$= -9.32$

E-W

$p = 17.63(K_z)(0.85)(0.8) + 15.4(0.18)$
$= 11.99 K_z + 2.77$

$p = 15.4(0.85)(-0.3) - 15.4(0.18)$
$= -6.7$

$p = 15.4(0.85)(-0.7) - 15.4(0.18)$
$= -11.94$

NORTH-SOUTH

Z (ft)	Kz	Pwindward(psf)	Pside(psf)	Pleeward(psf)	Ptotal(psf)
0-15	0.57	9.553	-6.7	-9.32	18.873
20	0.62	10.148	-6.7	-9.32	19.468
25	0.66	10.624	-6.7	-9.32	19.944
30	0.7	11.1	-6.7	-9.32	20.42
40	0.76	11.814	-6.7	-9.32	21.134
50	0.81	12.409	-6.7	-9.32	21.729
60	0.85	12.885	-6.7	-9.32	22.205
70	0.89	13.361	-6.7	-9.32	22.681

EAST-WEST

Z (ft)	Kz	Pwindward(psf)	Pside(psf)	Pleeward(psf)	Ptotal(psf)
0-15	0.57	9.553	-6.7	-11.94	21.493
20	0.62	10.148	-6.7	-11.94	22.088
25	0.66	10.624	-6.7	-11.94	22.564
30	0.7	11.1	-6.7	-11.94	23.04
40	0.76	11.814	-6.7	-11.94	23.754
50	0.81	12.409	-6.7	-11.94	24.349
60	0.85	12.885	-6.7	-11.94	24.825
70	0.89	13.361	-6.7	-11.94	25.301

A2. Seismic

Seismic Load

Use Group – I

Site Class – D

Seismic Design Category – B

Importance factor – 1.0

$S_1 = 0.055,\ F_v = 2.4$

$S_s = 0.159,\ F_a = 1.6$

$S_{MS} = F_a(S_s) = 1.6(0.159) = 0.254$

$S_{M1} = F_v(S_1) = 2.4(0.055) = 0.132$

$S_{DS} = \frac{2}{3}(S_{MS}) = 0.169$

$S_{D1} = \frac{2}{3}(S_{M1}) = 0.088$

R = 5, building frame system, ordinary reinforced concrete shear walls.

$V = C_s W$

$T_a = C_t h_n^x = 0.016(66)^{0.9} = 0.695$

$C_u(T_a) = 0.695(1.7) = 1.18$

$C_s = \frac{S_{DS}}{R/I} = \frac{0.169}{5} = 0.034$ $\qquad C_s = 0.015$

$= \frac{S_{D1}}{T(R/I)} = \frac{0.088}{1.18(5)} = 0.015$ *

$= \frac{0.088(6)}{(1.18)^2(5)} = 0.076$

$W = 606 + 829.3(4) + 848 + 745.6 = 5516.8$

$V = C_s W = 0.015(5516.8) = \boxed{83\text{ k}}$

$$\Sigma w_x h_x = 606(66) + 829.3(22+33+44+55) + 848(11) = 177036$$

$$k = 1.1$$

$$C_{vx} = \frac{w_x h_x^k}{\Sigma w_x h_x}$$

floor 2

$$C_{v2} = \frac{848(11)^{1.1}}{177036} = 0.067 \qquad F = 0.067(83) = \boxed{5.6\text{ k}}$$

floor 3

$$C_{v3} = \frac{829.3(22)^{1.1}}{177036} = 0.14 \qquad F = 0.14(83) = \boxed{11.6\text{ k}}$$

floor 4

$$C_{v4} = \frac{829.3(33)^{1.1}}{177036} = 0.22 \qquad F = 0.22(83) = \boxed{18.3\text{ k}}$$

floor 5

$$C_{v5} = \frac{829.3(44)^{1.1}}{177036} = 0.3 \qquad F = 0.3(83) = \boxed{24.9\text{ k}}$$

floor 6

$$C_{v6} = \frac{829.3(55)^{1.1}}{177036} = 0.38 \qquad F = 0.38(83) = \boxed{31.5\text{ k}}$$

Roof

$$C_{vroof} = \frac{606(66)^{1.1}}{177036} = 0.34 \qquad F = 0.34(83) = \boxed{28.2\text{ k}}$$

APPENDIX B

B1. Floors 2-6 Design Strip 1

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==

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[2] DESIGN RESULTS

==

* Slab section A ⊠ Slab section B

Top Reinforcement:

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

Span	Zone	Width	Mmax	Xmax	AsMin	AsMax	SpReq	AsReq	Bars	
* 1	Left	20.00	113.77	0.583	3.553	16.065	12.000	1.907	20-#4	*5
	Middle	20.00	0.00	13.550	0.000	16.065	0.000	0.000	---	
	Right	20.00	421.54	26.517	3.553	16.065	5.581	7.458	43-#4	
⊠ 2	Left	20.00	477.56	0.583	3.553	16.065	5.581	8.542	43-#4	
	Middle	20.00	0.00	16.250	0.000	16.065	0.000	0.000	---	
	Right	20.00	219.09	31.917	3.553	16.065	12.000	3.737	20-#4	*5

NOTES:
*5 - Number of bars governed by maximum allowable spacing.

Top Bar Details:

Units: Length (ft)

	Left				Continuous		Right			
Span	Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
* 1	18-#4	9.14	2-#4	5.77	---		22-#4	11.28	21-#4	5.77
⊠ 2	22-#4	10.92	21-#4	6.85	---		18-#4	10.92	2-#4	6.85

Bottom Reinforcement:

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

Span	Width	Mmax	Xmax	AsMin	AsMax	SpReq	AsReq	Bars
* 1	20.00	181.83	11.216	3.553	68.531	5.052	3.027	16-#5
⊠ 2	20.00	289.13	17.817	3.553	68.531	5.052	4.833	16-#5

Bottom Bar Details:

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Units: Start (ft), Length (ft), As (in^2)

Span	Long Bars: Bars	Long Bars: Start	Long Bars: Length	Short Bars: Bars	Short Bars: Start	Short Bars: Length	Joist: Ribs	Joist: Bars/Rib	Joist: As/Rib
* 1	16-#5	0.00	27.10	---			8	2-#5	0.620
⊗ 2	16-#5	0.00	32.50	---			8	2-#5	0.620

Flexural Capacity:
==================

Units: From, To (ft), As (in^2), PhiMn (k-ft)

Span	From	To	AsTop	AsBot	PhiMn-	PhiMn+
1	0.000	0.583	4.00	4.96	-240.88	296.67
	0.583	4.771	4.00	4.96	-240.88	296.67
	4.771	5.771	3.60	4.96	-216.98	296.67
	5.771	8.142	3.60	4.96	-216.98	296.67
	8.142	9.142	0.00	4.96	0.00	296.67
	9.142	9.660	0.00	4.96	0.00	296.67
	9.660	13.550	0.00	4.96	0.00	296.67
	13.550	15.819	0.00	4.96	0.00	296.67
	15.819	16.819	0.00	4.96	0.00	296.67
	16.819	17.440	4.40	4.96	-264.74	296.67
	17.440	21.329	4.40	4.96	-264.74	296.67
	21.329	22.329	4.40	4.96	-264.74	296.67
	22.329	26.517	8.60	4.96	-512.66	296.67
	26.517	27.100	8.60	4.96	-512.66	296.67
2	0.000	0.583	8.60	4.96	-512.66	296.67
	0.583	5.797	8.60	4.96	-512.66	296.67
	5.797	6.851	4.40	4.96	-264.74	296.67
	6.851	9.870	4.40	4.96	-264.74	296.67
	9.870	10.924	0.00	4.96	0.00	296.67
	10.924	11.550	0.00	4.96	0.00	296.67
	11.550	16.250	0.00	4.96	0.00	296.67
	16.250	20.950	0.00	4.96	0.00	296.67
	20.950	21.576	0.00	4.96	0.00	296.67
	21.576	22.576	0.00	4.96	0.00	296.67
	22.576	25.649	3.60	4.96	-216.98	296.67
	25.649	26.649	3.60	4.96	-216.98	296.67
	26.649	31.917	4.00	4.96	-240.88	296.67
	31.917	32.500	4.00	4.96	-240.88	296.67

Slab Shear Capacity:
====================

Units: b, d (in), Xu (ft), PhiVc, Vu(kip)

Span	b	d	Vratio	PhiVc		Vu		Xu
* 1	64.96	13.44	1.000	101.84	>	73.00	√ ok	25.40
⊗ 2	64.96	13.44	1.000	101.84	>	83.03	√ ok	1.70

Maximum Deflections:
====================

Units: Dz (in)

Span	Dz(DEAD)	Dz(LIVE)	Dz(TOTAL)
* 1	-0.061	-0.056	-0.116
⊗ 2	-0.159	-0.397	-0.556

Material Takeoff:
=================

Reinforcement in the Direction of Analysis

Top Bars:	761.5 lb	<=>	12.78 lb/ft	<=>	0.639 lb/ft^2
Bottom Bars:	994.6 lb	<=>	16.69 lb/ft	<=>	0.834 lb/ft^2
Stirrups:	0.0 lb	<=>	0.00 lb/ft	<=>	0.000 lb/ft^2
Total Steel:	1756.1 lb	<=>	29.46 lb/ft	<=>	1.473 lb/ft^2
Concrete:	789.2 ft^3	<=>	13.24 ft^3/ft	<=>	0.662 ft^3/ft^2

*

Span length = 26'(12) = 312"

$\Delta_{LL} = L/360 = 0.87 > 0.056$

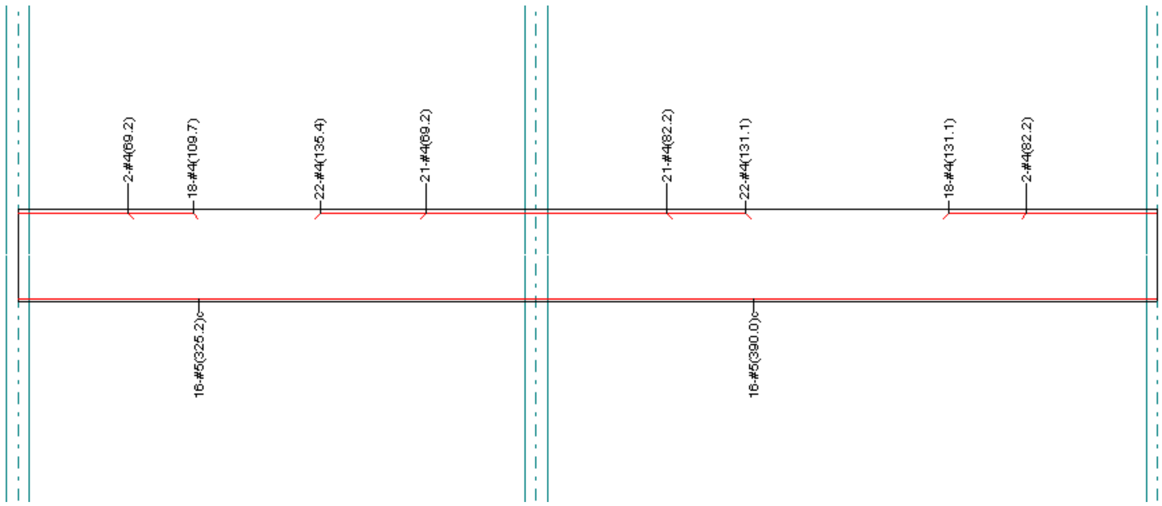
$\Delta_{TL} = L/240 = 1.3 > 0.116$

⊗

Span length = 31.33'(12) = 376"

$\Delta_{LL} = L/360 = 1.04 > 0.397$

$\Delta_{TL} = L/240 = 1.56 > 0.556$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22-#4(135.4)
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21-#4(82.2)
22-#4(131.1)
18-#4(131.1)
2-#4(82.2)
16-#5(325.2)
16-#5(390.0)

B2. Floors 2-6 Design Strip 2

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[2] DESIGN RESULTS

* = Slab Section C Design

Top Reinforcement:

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

Span	Zone	Width	Mmax	Xmax	AsMin	AsMax	SpReq	AsReq	Bars	
1	Left	20.00	149.55	0.583	3.553	16.065	12.000	2.521	20-#4	*5
	Middle	20.00	0.00	13.550	0.000	16.065	0.000	0.000	---	
	Right	20.00	268.99	26.517	3.553	16.065	10.000	4.628	24-#4	
* 2	Left	12.50	178.91	0.583	2.220	10.041	10.000	3.087	24-#4	
	Middle	12.50	54.98	5.162	2.220	10.041	12.500	0.918	12-#4	
	Right	12.50	5.06	9.088	2.220	10.041	12.500	0.083	12-#4	

NOTES:
*5 - Number of bars governed by maximum allowable spacing.

Top Bar Details:

Units: Length (ft)

	Left				Continuous		Right			
Span	Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	18-#4	9.14	2-#4	5.77	---		18-#4	9.14	6-#4	5.77
* 2	6-#4	4.90	6-#4	3.20	12-#4	14.00	---		---	

Bottom Reinforcement:

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

Span	Width	Mmax	Xmax	AsMin	AsMax	SpReq	AsReq	Bars
1	20.00	224.90	12.772	3.553	68.531	5.052	3.750	16-#5
* 2	12.50	22.43	14.000	2.220	42.832	5.052	0.371	10-#5

Bottom Bar Details:

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Units: Start (ft), Length (ft), As (in^2)

Span	Long Bars: Bars	Long Bars: Start	Long Bars: Length	Short Bars: Bars	Short Bars: Start	Short Bars: Length	Joist: Ribs	Joist: Bars/Rib	Joist: As/Rib
1	16-#5	0.00	27.10	---			8	2-#5	0.620
*2	10-#5	0.00	14.00	---			5	2-#5	0.620

Flexural Capacity:
==================

Units: From, To (ft), As (in^2), PhiMn (k-ft)

Span	From	To	AsTop	AsBot	PhiMn-	PhiMn+
1	0.000	0.583	4.00	4.96	-240.88	296.67
	0.583	4.771	4.00	4.96	-240.88	296.67
	4.771	5.771	3.60	4.96	-216.98	296.67
	5.771	8.142	3.60	4.96	-216.98	296.67
	8.142	9.142	0.00	4.96	0.00	296.67
	9.142	9.660	0.00	4.96	0.00	296.67
	9.660	13.550	0.00	4.96	0.00	296.67
	13.550	17.440	0.00	4.96	0.00	296.67
	17.440	17.958	0.00	4.96	0.00	296.67
	17.958	18.981	0.00	4.96	0.00	296.67
	18.981	21.329	3.60	4.96	-216.98	296.67
	21.329	22.352	3.60	4.96	-216.98	296.67
	22.352	26.517	4.80	4.96	-288.55	296.67
	26.517	27.100	4.80	4.96	-288.55	296.67
2	0.000	0.583	4.80	3.10	-286.72	185.42
	0.583	2.201	4.80	3.10	-286.72	185.42
	2.201	3.201	3.60	3.10	-215.96	185.42
	3.201	3.901	3.60	3.10	-215.96	185.42
	3.901	4.901	2.40	3.10	-144.58	185.42
	4.901	5.162	2.40	3.10	-144.58	185.42
	5.162	7.000	2.40	3.10	-144.58	185.42
	7.000	9.088	2.40	3.10	-144.58	185.42
	9.088	13.667	2.40	3.10	-144.58	185.42
	13.667	14.000	2.40	3.10	-144.58	185.42

Slab Shear Capacity:
====================

Units: b, d (in), Xu (ft), PhiVc, Vu(kip)

Span	b	d	Vratio	PhiVc		Vu	Xu
1	64.96	13.44	1.000	101.84		65.50	25.40
*2	40.60	13.44	1.000	63.65	>	32.18	1.70

√ok

Maximum Deflections:
====================

Units: Dz (in)

Span	Dz(DEAD)	Dz(LIVE)	Dz(TOTAL)
1	-0.090	-0.137	-0.227
*2	0.009	0.008	0.017

Span Length = 13.75' (12) = 165"

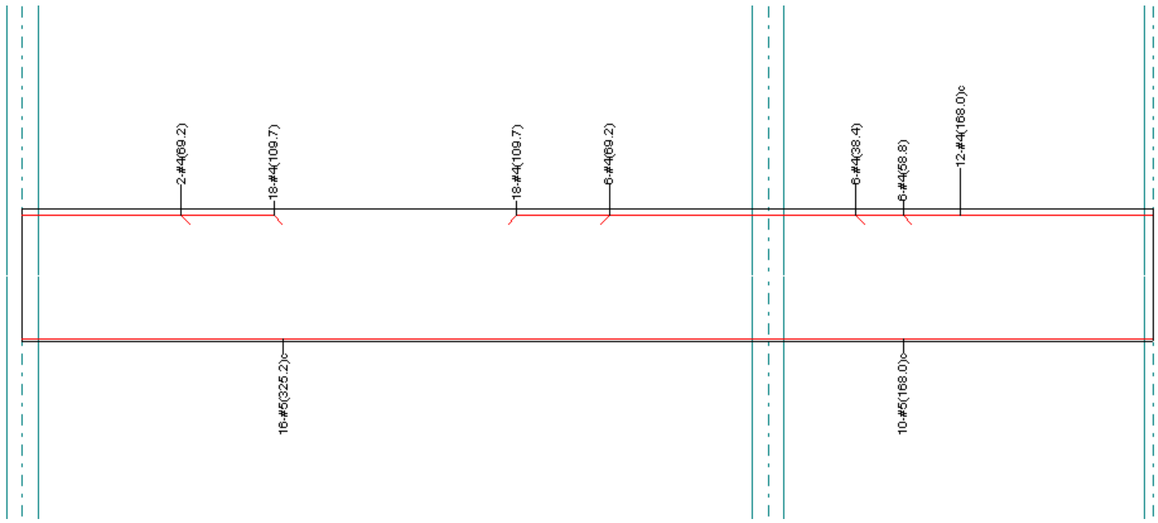
$\Delta_{LL} = \frac{L}{360} = 0.46 > 0.008$

$\Delta_{TL} = \frac{L}{240} = 0.69 > 0.017$

Material Takeoff:
=================

Reinforcement in the Direction of Analysis

Top Bars:	395.4 lb	<=>	9.62 lb/ft	<=>	0.551 lb/ft^2
Bottom Bars:	598.3 lb	<=>	14.56 lb/ft	<=>	0.834 lb/ft^2
Stirrups:	0.0 lb	<=>	0.00 lb/ft	<=>	0.000 lb/ft^2
Total Steel:	993.6 lb	<=>	24.18 lb/ft	<=>	1.386 lb/ft^2
Concrete:	475.7 ft^3	<=>	11.57 ft^3/ft	<=>	0.663 ft^3/ft^2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18-#4(109.7)
6-#4(69.2)
6-#4(38.4)
6-#4(58.8)
12-#4(168.0)c
16-#5(325.2)c
10-#5(168.0)c

B3. Floors 2-6 Design Strip 3

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===

===

[2] DESIGN RESULTS

===

Top Reinforcement:

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

Span	Zone	Width	Mmax	Xmax	AsMin	AsMax	SpReq	AsReq	Bars	
1	Left	18.00	1.65	0.583	3.197	16.065	12.000	0.027	18-#4	*5
	Middle	18.00	0.00	4.625	0.000	16.065	0.000	0.000	---	
	Right	18.00	11.71	8.667	3.197	16.065	12.000	0.193	18-#4	*5

NOTES:
*5 - Number of bars governed by maximum allowable spacing.

Top Bar Details:

Units: Length (ft)

Span	Left Bars	Left Length	Left Bars	Left Length	Continuous Bars	Continuous Length	Right Bars	Right Length	Right Bars	Right Length
1	16-#4	3.25	2-#4	2.20	---		16-#4	3.00	2-#4	1.95

Bottom Reinforcement:

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

Span	Width	Mmax	Xmax	AsMin	AsMax	SpReq	AsReq	Bars
1	18.00	31.25	4.258	3.197	61.965	5.167	0.515	16-#4

Bottom Bar Details:

Units: Start (ft), Length (ft), As (in^2)

Span	Long Bars Bars	Long Bars Start	Long Bars Length	Short Bars Bars	Short Bars Start	Short Bars Length	Joist Ribs	Joist Bars/Rib	Joist As/Rib
1	16-#4	0.00	9.00	---			8	2-#4	0.400

Flexural Capacity:

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Units: From, To (ft), As (in^2), PhiMn (k-ft)

Span	From	To	AsTop	AsBot	PhiMn-	PhiMn+
1	0.000	0.583	3.60	3.20	-216.79	192.89
	0.583	1.201	3.60	3.20	-216.79	192.89
	1.201	2.201	3.20	3.20	-192.89	192.89
	2.201	2.251	3.20	3.20	-192.89	192.89
	2.251	3.251	0.00	3.20	0.00	192.89
	3.251	3.412	0.00	3.20	0.00	192.89
	3.412	4.500	0.00	3.20	0.00	192.89
	4.500	5.837	0.00	3.20	0.00	192.89
	5.837	5.999	0.00	3.20	0.00	192.89
	5.999	6.999	0.00	3.20	0.00	192.89
	6.999	7.049	3.20	3.20	-192.89	192.89
	7.049	8.049	3.20	3.20	-192.89	192.89
	8.049	8.667	3.60	3.20	-216.79	192.89
	8.667	9.000	3.60	3.20	-216.79	192.89

Slab Shear Capacity:
===================

Units: b, d (in), Xu (ft), PhiVc, Vu(kip)

Span	b	d	Vratio	PhiVc		Vu		Xu
1	65.00	13.50	1.000	102.38	⊃	14.74	✓ok	7.54

Maximum Deflections:
===================

Units: Dz (in)

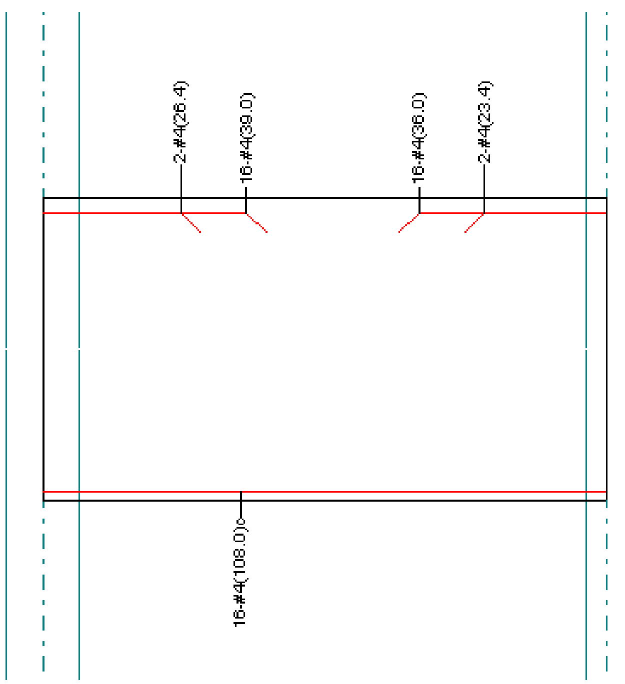
Span	Dz(DEAD)	Dz(LIVE)	Dz(TOTAL)
1	-0.002	-0.001	-0.003

deflections are acceptable by inspection

Material Takeoff:
================

Reinforcement in the Direction of Analysis

Top Bars:	72.4 lb	<=>	8.04 lb/ft	<=>	0.447 lb/ft^2
Bottom Bars:	96.2 lb	<=>	10.69 lb/ft	<=>	0.594 lb/ft^2
Stirrups:	0.0 lb	<=>	0.00 lb/ft	<=>	0.000 lb/ft^2
Total Steel:	168.6 lb	<=>	18.73 lb/ft	<=>	1.041 lb/ft^2
Concrete:	115.0 ft^3	<=>	12.77 ft^3/ft	<=>	0.710 ft^3/ft^2



B4. Floors 2-6 Design Strip 4

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==============================================================================================
[2] DESIGN RESULTS
==============================================================================================

Top Reinforcement:
==================
   Units: Width (ft), Mmax (k-ft), Xmax (ft), As (in^2), Sp (in)
   Span Zone      Width          Mmax     Xmax    AsMin    AsMax    SpReq    AsReq     Bars
   ---- ------ -------- ------------ -------- -------- -------- -------- -------- -------
      1 Left      25.20       363.49    0.583    4.029   22.089    9.450    6.249    32-#4
        Middle    25.20         0.00   16.250    0.000   22.089    0.000    0.000      ---
        Right     25.20       361.84   31.917    4.029   22.089    9.450    6.219    32-#4

Top Bar Details:
================
   Units: Length (ft)
        _____________Left______________   ___Continuous__   _____________Right_____________
   Span    Bars  Length    Bars  Length      Bars  Length      Bars  Length    Bars  Length
   ---- ------- ------- ------- -------   ------- -------   ------- ------- ------- -------
      1   21-#4   10.92   11-#4    6.85       ---              21-#4   10.92   11-#4    6.85

Bottom Reinforcement:
=====================
   Units: Width (ft), Mmax (k-ft), Xmax (ft), As (in^2), Sp (in)
   Span    Width         Mmax     Xmax    AsMin    AsMax    SpReq    AsReq     Bars
   ---- -------- ------------ -------- -------- -------- -------- -------- -------
      1    25.20       531.15   16.250    4.029   85.948    4.938    8.965    22-#6

Bottom Bar Details:
===================
   Units: Start (ft), Length (ft), As (in^2)
        _______Long Bars_______ ______Short Bars_______ _________Joist_________
   Span    Bars   Start  Length    Bars   Start  Length  Ribs Bars/Rib   As/Rib
   ---- ------- ------- ------- ------- ------- ------- ----- -------- --------
      1   22-#6    0.00   32.50     ---                     11     2-#6    0.880

Flexural Capacity:
==================
   Units: From, To (ft), As (in^2), PhiMn (k-ft)
   Span      From         To AsTop AsBot        PhiMn-       PhiMn+
```

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1	0.000	0.583	6.40	9.68	-384.50	572.77
	0.583	5.815	6.40	9.68	-384.50	572.77
	5.815	6.851	4.20	9.68	-253.30	572.77
	6.851	9.888	4.20	9.68	-253.30	572.77
	9.888	10.924	0.00	9.68	0.00	572.77
	10.924	11.550	0.00	9.68	0.00	572.77
	11.550	16.250	0.00	9.68	0.00	572.77
	16.250	20.950	0.00	9.68	0.00	572.77
	20.950	21.576	0.00	9.68	0.00	572.77
	21.576	22.607	0.00	9.68	0.00	572.77
	22.607	25.649	4.20	9.68	-253.30	572.77
	25.649	26.680	4.20	9.68	-253.30	572.77
	26.680	31.917	6.40	9.68	-384.50	572.77
	31.917	32.500	6.40	9.68	-384.50	572.77

Slab Shear Capacity:

Units: b, d (in), Xu (ft), PhiVc, Vu(kip)

Span	b	d	Vratio	PhiVc	Vu	Xu
1	89.26	13.38	1.000	139.29 >	106.04 ✓ok	1.70

Span length = 31.33 (12) = 376"

Maximum Deflections:

Units: Dz (in)

Span	Dz(DEAD)	Dz(LIVE)	Dz(TOTAL)
1	-0.256	-0.579	-0.834

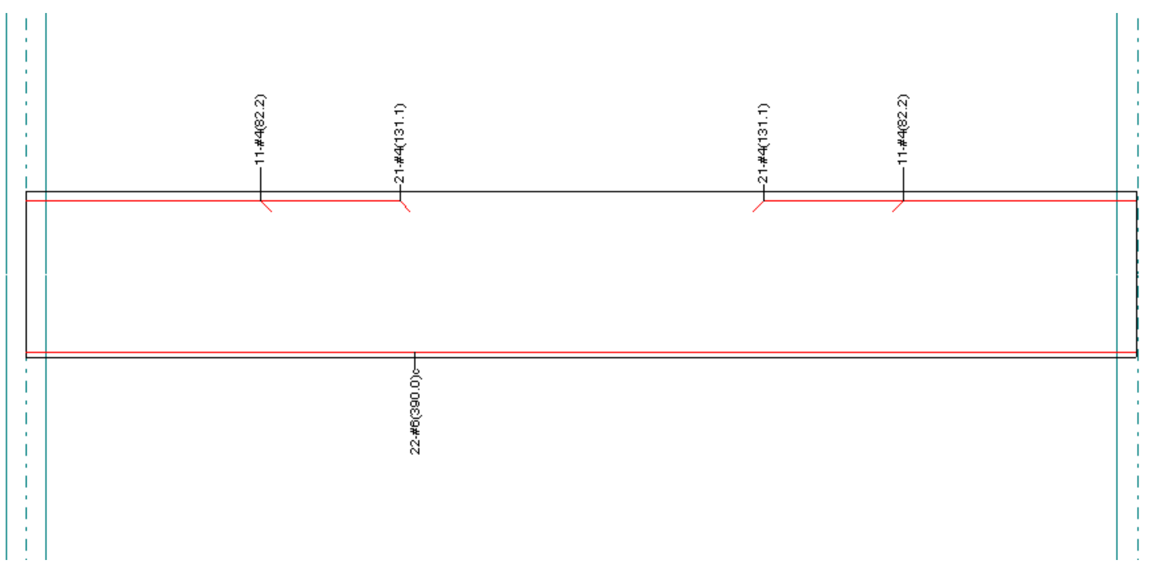
$\Delta_{LL} = L/360 = 1.04 > 0.579$

$\Delta_{TL} = L/240 = 1.56 > 0.834$

Material Takeoff:

Reinforcement in the Direction of Analysis

Top Bars:	407.2 lb	<=>	12.53 lb/ft	<=>	0.497 lb/ft^2
Bottom Bars:	1073.9 lb	<=>	33.04 lb/ft	<=>	1.311 lb/ft^2
Stirrups:	0.0 lb	<=>	0.00 lb/ft	<=>	0.000 lb/ft^2
Total Steel:	1481.1 lb	<=>	45.57 lb/ft	<=>	1.808 lb/ft^2
Concrete:	552.2 ft^3	<=>	16.99 ft^3/ft	<=>	0.674 ft^3/ft^2



B5. Design of Second Floor Green Roof Slab

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[2] DESIGN RESULTS

Top Reinforcement:

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

Span	Zone	Width	Mmax	Xmax	AsMin	AsMax	SpReq	AsReq	Bars	
1	Left	16.00	11.57	0.583	1.382	12.240	12.000	0.866	16-#4	*5
	Middle	16.00	0.00	4.000	0.000	12.240	0.000	0.000	---	
	Right	16.00	17.76	7.417	1.382	12.240	8.000	1.338	24-#4	*5
2	Left	16.00	24.14	0.583	1.382	12.240	8.000	1.829	24-#4	*5
	Middle	16.00	10.22	1.814	1.382	12.240	8.000	0.764	24-#4	*5
	Right	16.00	2.98	2.869	1.382	12.240	8.000	0.221	24-#4	*5

NOTES:
*5 - Number of bars governed by maximum allowable spacing.

Top Bar Details:

Units: Length (ft)

	Left				Continuous		Right			
Span	Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	8-#4	2.84	8-#4	1.95	---		12-#4	3.78	12-#4	1.95
2	---		---		24-#4	4.10	---		---	

Bottom Reinforcement:

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

Span	Width	Mmax	Xmax	AsMin	AsMax	SpReq	AsReq	Bars	
1	16.00	8.23	3.756	1.382	12.240	12.000	0.614	16-#4	*5
2	16.00	0.00	4.100	0.000	12.240	0.000	0.000	---	

NOTES:
*5 - Number of bars governed by maximum allowable spacing.

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Bottom Bar Details:

Units: Start (ft), Length (ft)

Span	Long Bars: Bars	Long Bars: Start	Long Bars: Length	Short Bars: Bars	Short Bars: Start	Short Bars: Length
1	8-#4	0.00	8.00	8-#4	2.75	5.25
2	---			---		

Flexural Capacity:

Units: From, To (ft), As (in^2), PhiMn (k-ft)

Span	From	To	AsTop	AsBot	PhiMn-	PhiMn+
1	0.000	0.583	3.20	1.60	-41.51	21.18
	0.583	0.951	3.20	1.60	-41.51	21.18
	0.951	1.839	1.60	1.60	-21.18	21.18
	1.839	1.951	0.00	1.60	0.00	21.18
	1.951	2.755	0.00	1.60	0.00	21.18
	2.755	2.839	0.00	1.60	0.00	21.18
	2.839	2.975	0.00	1.60	0.00	21.18
	2.975	3.755	0.00	1.60	0.00	21.18
	3.755	4.000	0.00	3.20	0.00	41.51
	4.000	4.220	0.00	3.20	0.00	41.51
	4.220	5.025	0.00	3.20	0.00	41.51
	5.025	5.220	0.00	3.20	0.00	41.51
	5.220	6.049	2.40	3.20	-31.45	41.51
	6.049	7.049	2.40	3.20	-31.45	41.51
	7.049	7.417	4.80	3.20	-60.99	41.51
	7.417	8.000	4.80	3.20	-60.99	41.51
2	0.000	0.583	4.80	0.00	-60.99	0.00
	0.583	1.814	4.80	0.00	-60.99	0.00
	1.814	2.050	4.80	0.00	-60.99	0.00
	2.050	2.869	4.80	0.00	-60.99	0.00
	2.869	4.100	4.80	0.00	-60.99	0.00

Slab Shear Capacity:

Units: b, d (in), Xu (ft), PhiVc, Vu(kip)

Span	b	d	Vratio	PhiVc		Vu		Xu
1	192.00	3.00	1.000	61.09	≥	13.27	✓ OK	7.17
2	192.00	3.00	1.000	61.09	≥	12.75	✓ OK	0.83

Maximum Deflections:

Units: Dz (in)

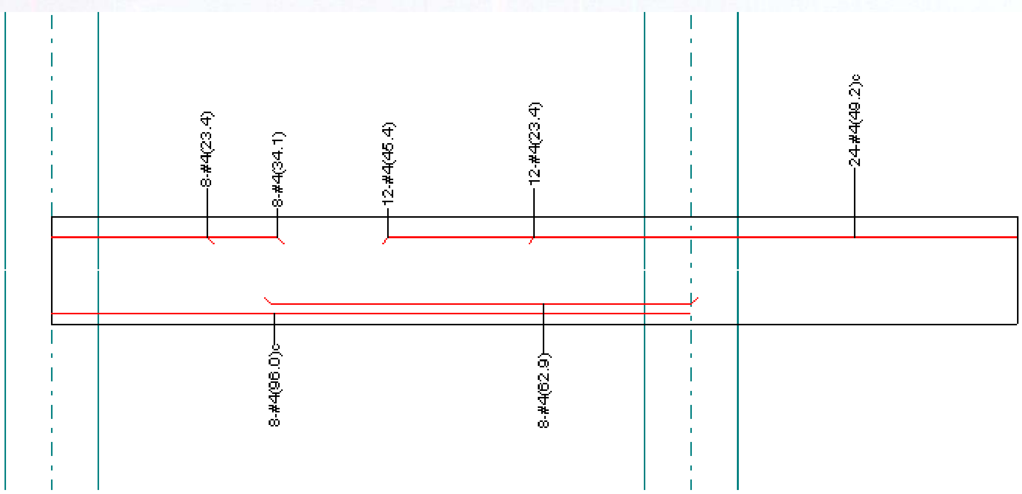
Span	Dz(DEAD)	Dz(LIVE)	Dz(TOTAL)
1	-0.003	-0.004	-0.007
2	-0.009	-0.013	-0.023

Deflections are acceptable by inspection

Material Takeoff:

Reinforcement in the Direction of Analysis

Top Bars:	137.3 lb	<=>	11.34 lb/ft	<=>	0.709 lb/ft^2
Bottom Bars:	70.8 lb	<=>	5.85 lb/ft	<=>	0.366 lb/ft^2
Stirrups:	0.0 lb	<=>	0.00 lb/ft	<=>	0.000 lb/ft^2
Total Steel:	208.0 lb	<=>	17.19 lb/ft	<=>	1.075 lb/ft^2
Concrete:	101.9 ft^3	<=>	8.42 ft^3/ft	<=>	0.526 ft^3/ft^2



B5. Design of Framing Around an Opening

Design around opening in Slab

Top

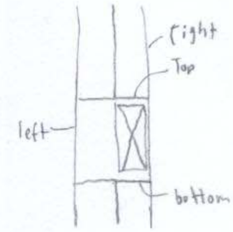
$DL = \frac{4.5}{12}\left(\frac{27}{12}\right)150 = 84.4\ \text{lb/ft}\ (9.55) = 806\ \text{lbs}$

$= 1.5\ \text{kips}$

$DL = \frac{10(7)}{144}(150) = 72.9\ \text{lb/ft}\ (9.55) = 696\ \text{lbs}$

$LL = 80\ \text{psf}\,(27/12) = 180\ \text{lb/ft}\ (9.55) = 1.7\ \text{kips}$

$\boxed{7\text{x}10\ \text{beam}}$



bottom

$DL = 84.4(2.55) + 72.9(2.55) = 0.4\ \text{k}$

$LL = 180(2.55) = 0.46\ \text{k}$

$\boxed{7\text{x}10\ \text{beam}}$

Left & right

Top $DL = 0.9\ \text{k}$
$LL = 0.85\ \text{k}$

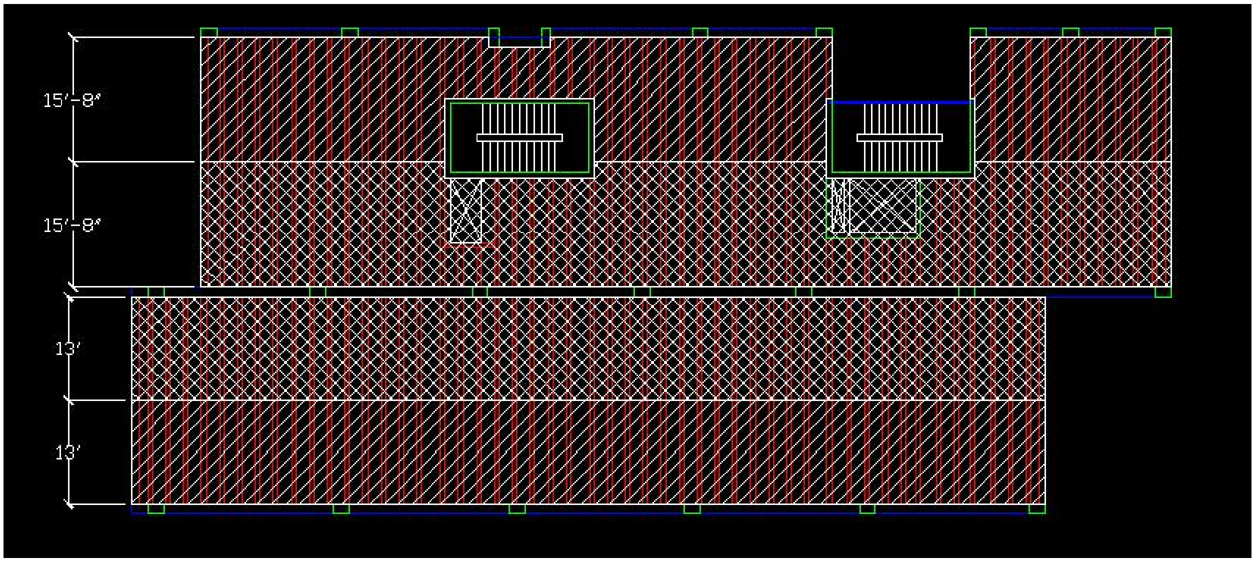
Bottom $DL = 0.35\ \text{k}$
$LL = 0.23\ \text{k}$

$\boxed{10\text{x}10\ \text{beam}}$

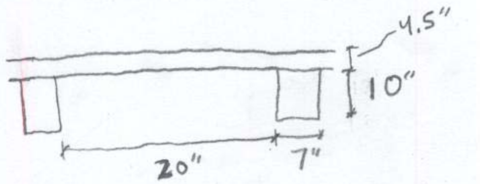
* PCA Slab was used to calculate the beam sizes for the loads determined above.

APPENDIX C

C1. Tributary Area on the Girders



C2. Dead Load of Slab Calculation



Dead Loading

$\frac{4.5}{12}(150) = 56.3(27/12) = 84.4$ PLF

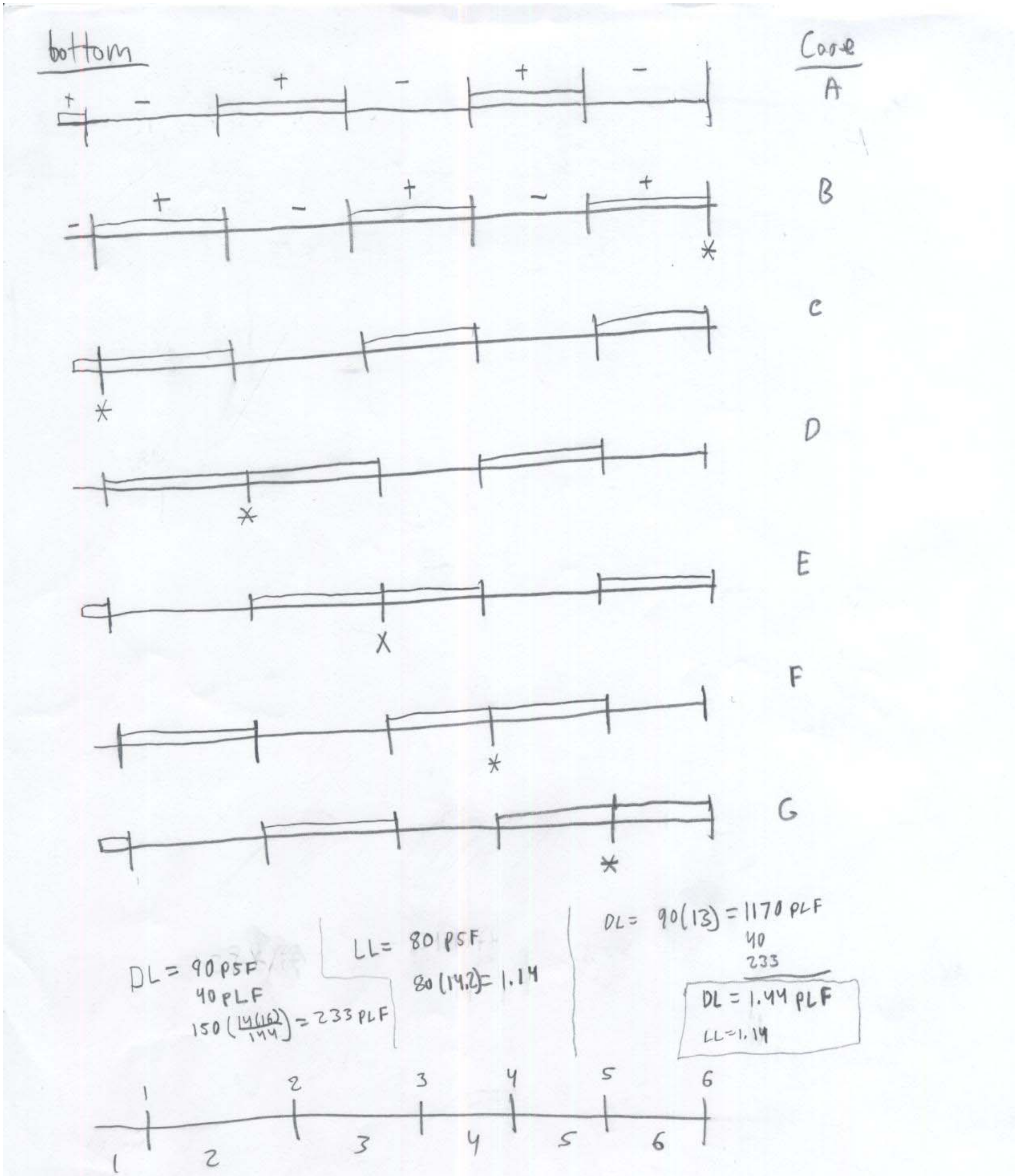
$\frac{10(7)}{144} = 0.486(150) = 72.9$ PLF

15 lbs/ft^2 $\left(\frac{27}{12}\right) = 45$ PLF

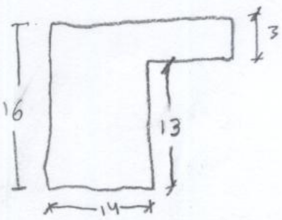
202.3 PLF

$\frac{202.3 \text{ PLF}}{\left(\frac{27}{12}\right)} = 90$ PSF

C3. Bottom Girder Calculations



Torsion – Span 2-6 (designed for span 6)

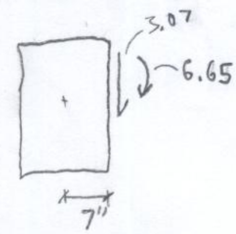


$d = 13.5''$

$DL = 90\,(1.2) = 108\ psf$ $= 0.236\ KSF$
$LL = 80\,(1.6) = 128\ psf$

– Shear on spandral from slab –

$\frac{wl}{2} = \frac{0.236\,(26)}{2} = 3.07\ k/ft$



– Moment on spandral –
from ACI 8.3.3, moment coefficient

$-Mu = \frac{W l_n^2}{24} = \frac{0.236\,(26)^2}{24} = 6.65\ ft\text{-}k/ft$

$\frac{7}{12}(3.07) = 1.8\ ft\text{-}k/ft$

$t = 1.8 + 6.65 = 8.45\ ft\text{-}k/ft$

$T_u = \frac{t l_n}{2} = \frac{8.45\,(21.33)}{2} = 90.1$

$T_{th} < T_u$ than torsion should be considered

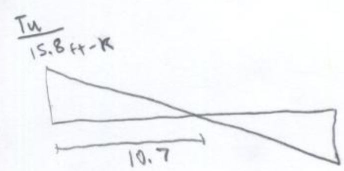
$T_{th} = \phi\sqrt{f'c}\,\frac{A_{cp}^2}{P_{cp}} = 0.75\sqrt{5000}\left(\frac{260^2}{84}\right) = 3.6 < 90.5$ torsion design is needed

flange projection $\Rightarrow 13$ or $3(4) = \underline{12}$

$A_{cp} = 14(16) + 12(3) = 260$

$P_{cp} = 16+14+13+12+3+12+14 = 84$

T_u – reduce torsion @ d from column face



$T_u = 4\,(T_{th}) = 14.2\ ft\text{-}K$

$\frac{10.7}{9.6} = \frac{x}{14.2}$

– Is the section big enough for torsion –

$\sqrt{\left(\frac{V_u}{b_w d}\right)^2 + \left(\frac{T_u P_h}{1.7 A_{oh}^2}\right)^2} \le \phi\,\frac{V_c}{b_w d} + 8\sqrt{f'c}$

$A_{oh} = (16-3.5)(14-3.5) = 131.3$

$P_h = (12.5 + 10.5)2 = 46$

from SAP

$\sqrt{\left(\frac{36{,}000}{14(13.5)}\right)^2 + \left(\frac{14.2\,(12{,}000)(46)}{1.7\,(131.3)^2}\right)^2} = 328\ psi$

$V_c = 2\sqrt{f'c}\,b_w d \;\therefore\; \phi(10\sqrt{f'c}) = 0.75\,(10\sqrt{5000}) = 530$

$328 < 530$

✓ OK

–Spacing–

$$A_T = \frac{T_u s}{2\phi A_o f_{yv} \cot\theta}$$

$$= \frac{15.8(12)s}{2(0.75)0.85(131.3)60} = 0.0189s$$

$$2A_T = 0.0378s$$

~~T = 16.7(2) ... = 1.8~~
~~20.4 − (2.155/12)~~
~~$T_u = \frac{1.8(20.4)}{2} = 18.36$ at column face~~

$$\phi V_c = 0.75(2)\sqrt{5000}(14)\left(\frac{13.5}{1000}\right) = 20$$

$$A_v = \frac{(V_u - \phi V_c)s}{\phi f_{y} d} = \frac{(41.6 - 20)s}{0.75(60)(13.5)} = 0.031s$$

$0 \le x \le 5.6$ ft

$$2A_t + A_v = 0.0378s\left(1 - \frac{x}{10.7}\right) + 0.031s\left(1 - \frac{x}{5.6}\right)$$

where $V_u = \phi V_c$

$5.61 \le x \le 10.2$

$$2A_t + A_v = 0.0378s\left(1 - \frac{x}{10.2}\right)$$

~~0.02278 + 0.033~~
~~0.01958 + 0.03039~~
~~0.01056~~ 0.0248
~~0.00133~~ 0.0199

use No. 4 stirrups, spacing @ 2' intervals

$S_0 = 6.8$ in
$S_2 = 7.9$ in
$S_4 = 11.93$ in
$S_6 = 23.4$
$S_8 = 49.1$

Max spacing

$d/2 = 6.75$

$\frac{P_h}{8} = 46/8 = 5.75$

Space bars @ 5.75"

Minimum Steel

$0.75\sqrt{5000}(14)6/60{,}000 = 0.074$

$50(14)6/60{,}000 = 0.07$

< 0.4 ✓ok

–required Longitudinal steel–

$$\frac{A_t}{s} = 0.0189\left(1 - \frac{1.13}{10.7}\right) = 0.0169$$

$$A_L = 0.0169(46) = 0.78$$

use $A_L = 0.77 in^2$

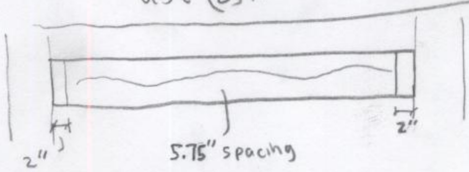
$0.77/3 = 0.26$

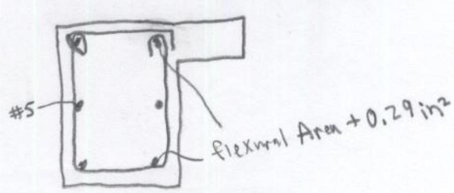
use (2) #5 at Middepth

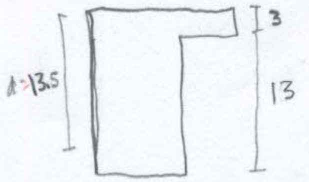
$$\frac{25(14)}{60{,}000} = 0.0058 \checkmark$$

$$A_{L,min} = \frac{5\sqrt{5000}(260)}{60(1000)} - 0.0169(46) = 0.77$$

The shear at the column face on the other side of the beam is less. Therefore, the stirrup design will be suitable to resist torsion for the other half of the beam.







Support 2 has the largest moment out of all 6 supports. All supports will be reinforced with the amount of steel used to reinforce support 2. The other support design moments are close enough to the moment at support 2 that this would not create a radical over design. This will save me time.

– Design Moment –

$M_u = 153.7$ k-ft

– reinforcement –

$$A_s = \frac{M_u(12)}{\phi f_y j d} = \frac{153.7(12)}{0.9(60)(0.875)(13.5)} = 2.89 \text{ in}^2$$

$$a = \frac{2.89(60)}{0.85(5)(14)} = 2.9$$

$$\frac{a}{d} = \frac{2.9}{13.5} = 0.21$$

$$\frac{a_b}{d} = 0.474 > 0.21 \therefore f_y = f_s$$

$$\frac{a_{TCL}}{d} = 0.3 > 0.21 \therefore \phi = 0.9$$

$$A_s = \frac{M_u(12)}{0.9(60)(13.5 - \frac{2.9}{2})} = 0.0184\, M_u = 2.83 \text{ in}^2$$

Torsion

$2.83 + 0.26 = 3.09$

use (4) #8 $\quad A_s = 3.16 \text{ in}^2$

As the reinforcement for the supports were designed for the largest design moment in the continuous beam the reinforcement in the span will be designed the same way.

$$b_{eff} \Rightarrow \frac{1}{12}(23.33)(12) = \underline{23.33}$$
$$6(3)+14 = 32$$
$$\frac{26.7(12)}{2} = 160$$

– Design moment –
$M_u = 81.6$ k-ft

$$A_s = \frac{81.6(12)}{0.9(60)(0.95)(13.5)} = 1.41$$

– assume $a < h_f$

$$a = \frac{1.41(60)}{0.85(5)(23.33)} \Rightarrow 0.85 < 3 \quad \text{o.k}$$

$$\frac{a}{d} = 0.056 \quad \therefore f_y = f_s,\ \phi = 0.9$$

$$A_s = \frac{M_u(12)}{0.9(60)\left(13.5 - \frac{0.85}{2}\right)} = 0.017\,M_u \Rightarrow 1.39\ in^2$$

~ torsion
$1.39 + 0.26 = 1.7\ in^2$

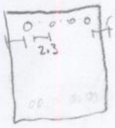
(2) #9 $\quad A_s = 2.0\ in^2$

Span 1 (Cantilever)

(2) #8 top steel will be used to resist the negative moment on the span.

* See rebar diagram for layout.

Development length



$d_b = 1.0$

$2.3 > d_b$

$1.5 > d_b$

use $\ell_d = \left(\dfrac{f_y \alpha \beta \lambda}{20\sqrt{f'_c}}\right) d_b$

top bars

$\ell_d = \dfrac{60{,}000(1.3)}{20\sqrt{5000}}(1) = 55 \text{ in}$

~~bottom bars~~

~~$\ell_d = \dfrac{60{,}000(1.3)}{20\sqrt{5000}}(0.875) = 49 \text{ in}$~~

Top bars

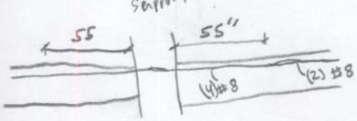
$A_s = \dfrac{M_u(12)}{0.9(60)(13.5 - \frac{2.89}{2})}$

$1.58 = \dfrac{M_u(12)}{0.9(60)(13.5 - \frac{2.89}{2})} \Rightarrow M_u = 99.9 \text{ k-ft}$

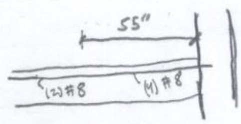
(2) #8

bar moment capacity

Support 2-5



Support 6



use $\ell_d = 55''$

$d = 15.5$

$12(d_b) = 12$

Moment capacity theoretical cut off point = 22.2

$22.2 + 15.5 = 37.7$

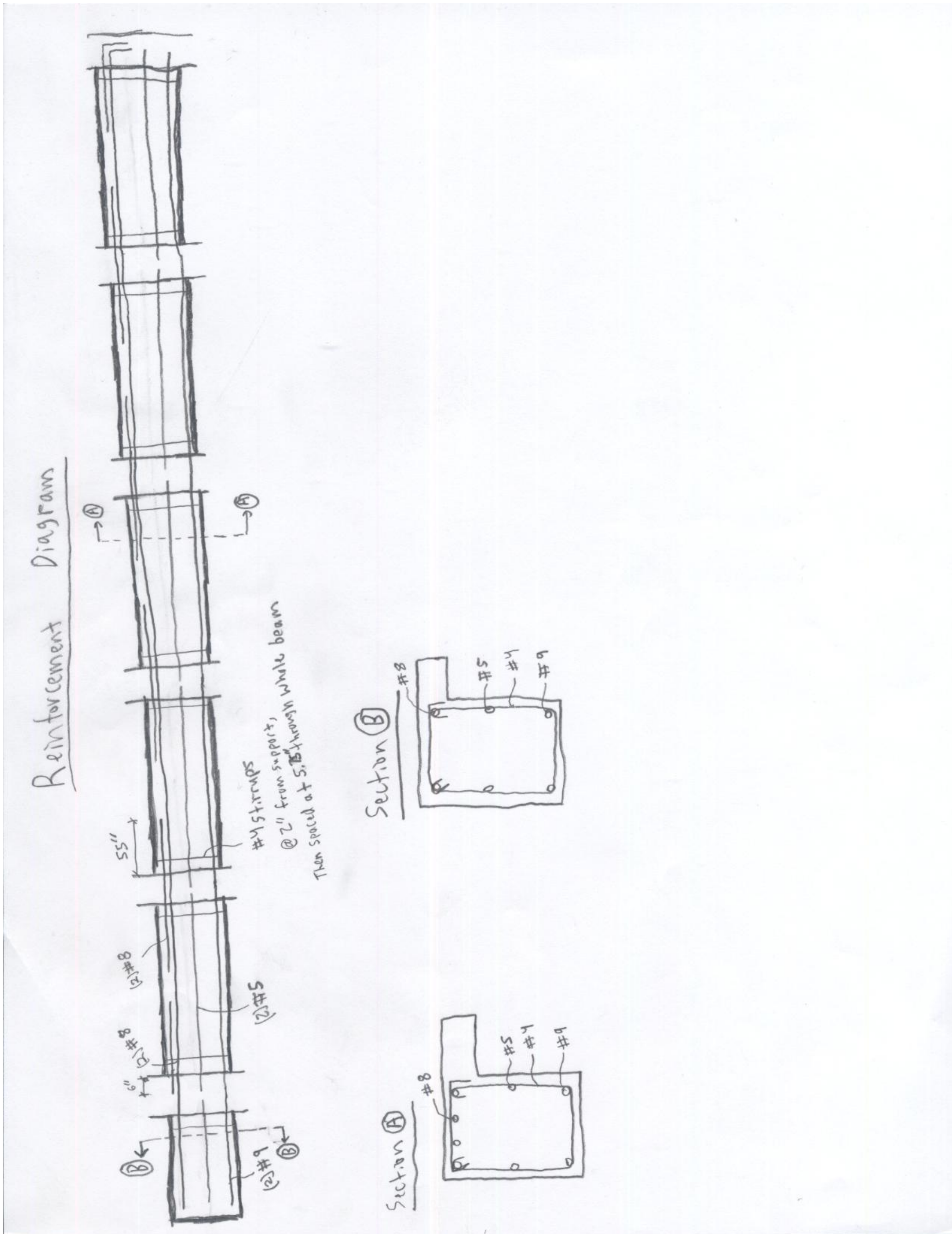
use $\ell_d = 55''$

Cut off = 18

$18 + 15.5 = 33.5$

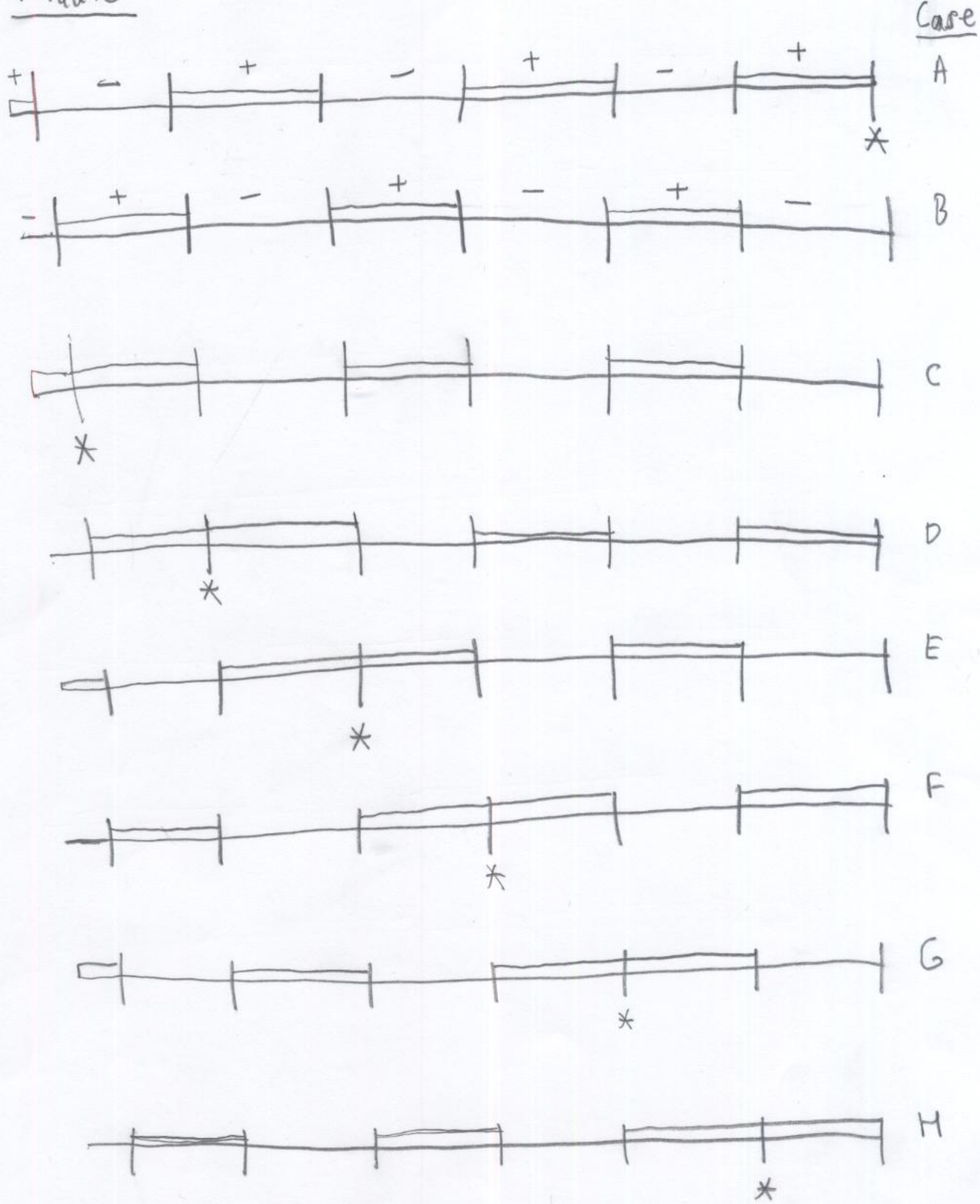
use $\ell_d = 55$

There will be no cut off for the bottom bars. The stirrups require 2 bars on the bottom of the beam to hook onto and there are only 2 bars on the bottom. There fore none can be cut.


Reinforcement Diagram
#4 stirrups
@ 2" from supports,
Then spaced at 5.5" through whole beam
55"
6"
(2)#8
(2)#5
(2)#9
Section A
#8
#5
#4
#9
Section B
#8
#5
#4
#9

C4. Middle Girder Calculations

Middle



DL
90 PSF
$\frac{14(18)}{144}(150) = 262.5$ PLF

LL
80 PSF

bottom
LL $80(14.2) = 1.14$ KLF
DL $90(13) = 1170$
262.5
1.43 KLF

Top
LL $80(16.9) = 1.35$ KLF
DL $90(15.8) = 1.42$ KLF

or
LL $80(6.9) = 0.55$ KLF
DL $90(6.9) = 0.62$ KLF

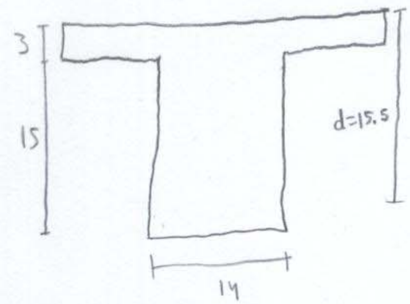
torsion

$w_u = 0.236$ KSF

– shear –

$\frac{w(L)}{2} = \frac{0.236(31.3)}{2} = 3.88$ $\qquad 3.88 - 3.22 = 0.66$

$\frac{wL}{2} = \frac{0.236(26)}{2} = 3.22$



moment

$M_u = \frac{w l_n^2}{10} = \frac{0.236(31.3)^2}{10} = 24.3$

$24.3 - 16.8 = 7.5$ ft-k/ft

$M_u = \frac{w l_n^2}{10} = \frac{0.236(26)^2}{10} = 16.8$

$A_{cp} = 14(18) + 12(3)2 = 324$

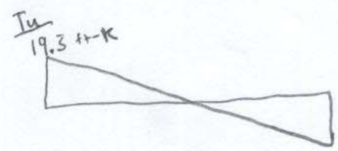
$P_{cp} = 14(2) + 15(2) + (12)(4) + 3(2) = 112$

$T_u = 0.75\sqrt{5000}\left(\frac{324^2}{112}\right) = 4.14 < 81$

$\frac{7}{12}(0.66) = 0.4$ ft-k/ft

$t = 7.9$

$T_u = \frac{7.9(18.3)}{2} = 81.1$ ft-k



reduce torsion to $\Longrightarrow T_u = 16.7$ ft-k

– Spacing –

$A_T = \frac{19.3(12)s}{2(0.75)(0.85)(152.3)60} = 0.0199s$

$t = \frac{16.7(2)}{18.3 - (2 \cdot \frac{15.5}{12})} = 2.03$

$T_u = \frac{2.03(18.3)}{2} = 19.3$

$2A_T = 0.0398s$

$A_v = \frac{(71.1 - 23.03)}{0.75(60)(15.5)}s = 0.069s$

$0 \le x \le 6.21$

$2A_T + A_v = 0.0398s\left(1 - \frac{x}{9.15}\right) + 0.069s\left(1 - \frac{x}{6.21}\right)$

$3 \le x \le 9.15$

$2A_T + A_v = 0.0398s\left(1 - \frac{x}{9.15}\right)$

use #4 stirrups

$S_d = 4.5$

$S_2 = 5.135$

$S_4 = 8.57$

$S_6 =$

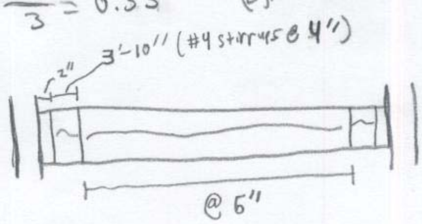
$S_8 =$

required A_L

$$\frac{A_t}{s} = 0.0199\left(1 - \frac{1.29}{9.17}\right) = 0.0171$$

$A_L = 0.86$

$$\frac{1.05}{3} = 0.35 \quad (2)\#5 \text{ @ middepth}$$



~~0.0379~~ 0.0546
~~0.0274~~ 0.0468
0.02456

Max spacing

6"

Min. Steel (Long.)

$$A_{L\,min} = \frac{5\sqrt{5000}\,(324)}{60(1000)} - 0.0171(50)$$

$A_{lmin} = 1.05$

Support I-7 (designed for support 6)

$M_u = 257.9$ k-ft

$$A_s = \frac{257.9(12)}{0.9(60)(0.875)(15.5)} = 4.23 \text{ in}^2$$

$$a = \frac{4.23(60)}{0.85(5)(14)} = 4.27$$

$$\frac{a}{d} = 0.28$$

$f_y = f_s \quad \phi = .9$

$$\cancel{c = \frac{0.328}{0.8} = 6.35}$$

$$\cancel{\varepsilon_t = \frac{0.003(15.5 - 6.35)}{6.35} = 0.0043} \quad \cancel{\therefore \phi = .84}$$

$$A_s = \frac{M_u(12)}{0.9(60)\left(15.5 - \frac{4.27}{2}\right)} = 0.01663\, M_u = 4.3 \text{ in}^2 + 0.35 = 4.65$$

(5) #9 $A_s = 5.00 \text{ in}^2$

All the spans will be designed for span 7's design moment, the largest span moment.

$b_{eff} = \frac{1}{4}(25)(12) = 75$

$16(3)+14 = \underline{62}$

$160+186 = 346$

$M_u = 129.2$ k-ft

$$A_s = \frac{129.2(12)}{0.9(60)(0.95)(15.5)} = 1.95 \text{ in}^2$$

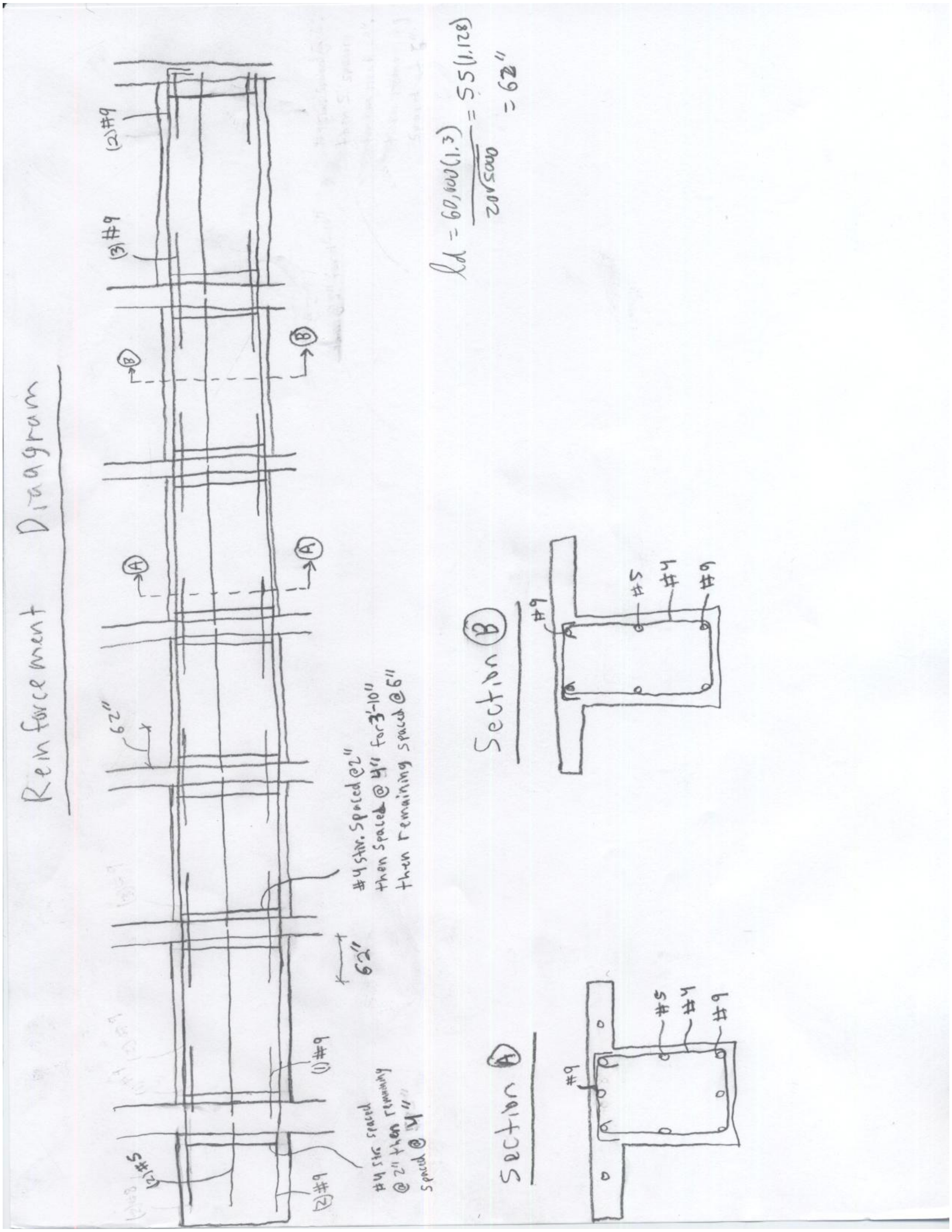
assume $a < h_f$

$$a = \frac{1.95(60)}{0.85(5)62} = 0.44$$

$\frac{a}{d} = 0.034 \quad f_y = f_s, \ \phi = 0.9$

$$A_s = \frac{M_u(12)}{0.9(60)(13.5 - \frac{0.44}{2})} = 0.0167 M_u = 2.16 \text{ in}^2 + 0.35 = 2.52$$

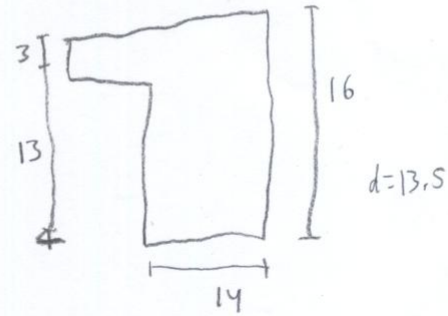
(3) #9 $A_s = 3 \text{ in}^2$


Reinforcement Diagram
(2)#9
(3)#9
(1)#9
(2)#5
62"
#4 stir. Spaced @2" then Spaced @ 4" for 3'-10" then remaining spaced @ 6"
$l_d = \frac{60{,}000(1.3)}{20\sqrt{5000}} = 55(1.128) = 62"$
Section A
Section B
#5
#4
#9

C5. Top Girder Calculations

Torsion

$W = 0.236$ KSF



Shear

$\frac{wl}{2} = 3.88$

$\frac{wl_n^2}{24} = 10.1$ K-ft/ft

$\frac{7}{12}(3.88) = 2.26$

$t = 12.36$

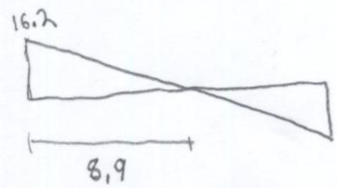
$T = \frac{12.36(17.8)}{2} = 110$

$T_{th} = 3.6$ ∴ Torsion needs to be considered

-reduced torsion-

$T_u = 14.2$

-section will be large enough-



$\frac{8.9}{7.8} = \frac{x}{14.2}$ $x = 16.2$

Spacing

$A_T = \frac{16.2(12)s}{2(0.75)(0.85)(152.3)60)} = 0.0167s$

$2A_T = 0.0334s$

$A_v = \frac{(40.6 - 20)s}{0.75(60)(13.5)} = 0.0339s$

$0 \le x \le 4.3$

$2A_T + A_v = 0.0334s\left(1 - \frac{x}{8.9}\right) + 0.0339s\left(1 - \frac{x}{4.3}\right)$

$0 \le x \le 8.9$

$2A_T + A_v = 0.0334s\left(1 - \frac{x}{8.9}\right)$

use #4 stirrups

$S_d = 7.38$

$S_2 = 9.11$

$S_3 =$

$S_4 =$

$S_6 =$

$S_8 =$

Max spacing

5.75"

0.025

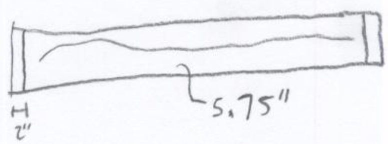
0.018

Longitudinal steel

$A_{L\,min} = 0.77$

$\frac{0.77}{3} = 0.26\,in^2$

use 2#5 @ middepth



Design for the supports

$M_u = 115.5$ k-ft

$$A_s = \frac{M_u(12)}{\phi(f_y)jd} = \frac{115.5(12)}{0.9(60)(0.875)(13.5)} = 2.17 \text{ in}^2$$

$$a = \frac{2.17(60)}{0.85(5)(14)} = 2.19 \text{ in}$$

$\frac{a}{d} = 0.16 \quad f_y = f_s, \phi = 0.9$

$$A_s = \frac{M_u(12)}{0.9(60)(13.5 - \frac{2.19}{2})} = 0.0179\, M_u = 2.07 \text{ in}^2 + 0.26 = 2.33 \text{ in}^2$$

(3) #8

$A_s = 2.37$

Design for the Slab

$b_{eff} = \frac{1}{12}(12)(17.8) = 17.8$

$6(3) + 14 = 32$

$M_u = 67.6$ k-ft

$$A_s = \frac{67.6(12)}{0.9(60)(0.95)(13.5)} = 1.17 \text{ in}^2$$

$$a = \frac{1.17(60)}{0.85(5)(17.8)} = 0.95$$

$\frac{a}{d} = 0.07 \quad f_y = f_s, \phi = 0.9$

$$A_s = \frac{M_u(12)}{0.9(60)(13.5 - \frac{0.9}{2})} = 0.017\, M_u = 1.15 \text{ in}^2 + 0.26 = 1.41 \text{ in}^2$$

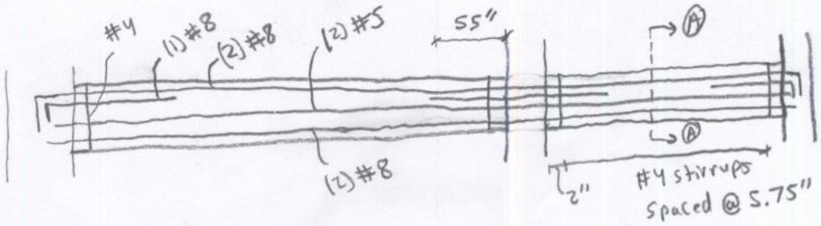
(2) #8

$A_s = 1.58 \text{ in}^2$

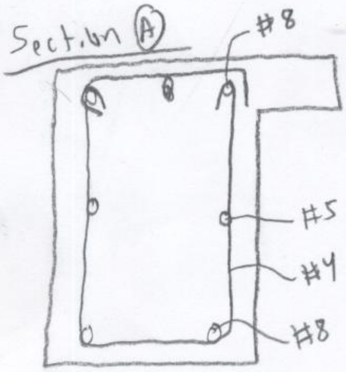
Cutoff pt.

55" for #8 bars

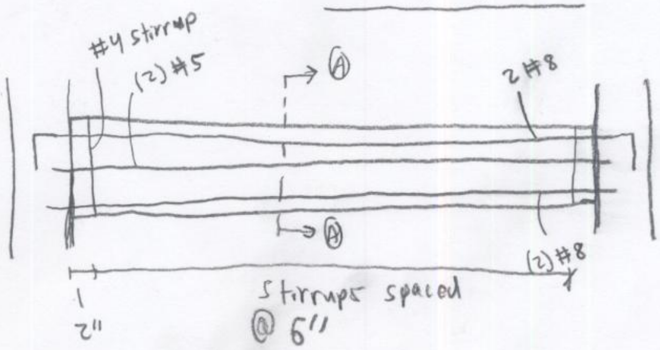
Top Left, Top Middle, Top Right



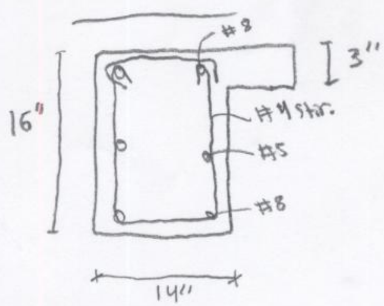
Section A



Small girder



Section A



Check flexural capacity for Beams for 6th floor roof terrace Loads

$DL = 90\text{ PSF} \Rightarrow 90(15.67) = 1.41\text{ KLF}$

$= 40\text{ PLF} \Rightarrow 0.04\text{ KLF}$

$= \frac{14(16)}{144}(150) = 233\text{ PLF} \Rightarrow 0.23\text{ KLF}$

1.69 KLF

$LL = 100\text{ PSF}(15.67) = 1.57\text{ KLF}$

Span moment $\Rightarrow 22.1\text{ k-ft}$

Support moment $\Rightarrow 42.4\text{ k-ft}$

Span

$$A_s = \frac{22.1(12)}{0.9(60)(0.95)(13.5)} = 0.38$$

$$a = \frac{0.38(60)}{0.85(5)(\underbrace{9.67}_{b_{eff}})} = 0.55$$

$$A_s = \frac{M_u(12)}{0.9(60)(13.5-\frac{0.55}{2})} = 0.0168 M_u = 0.37\text{ in}^2$$

Support

$$A_s = \frac{42.4(12)}{0.9(60)(0.875)(13.5)} = 0.8$$

$$a = \frac{0.8(60)}{0.85(5)(14)} = 0.81$$

$$A_s = \frac{M_u(12)}{0.9(60)(13.5-\frac{0.81}{2})} = M_u(0.01697) = 0.72\text{ in}^2$$

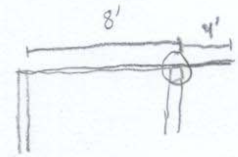
these beams were origionally overdesigned because they use the same reinforcement as the other top girders that have longer spans and larger moments. Due to the over design the origional design is suitable to withstand the 100 PSF live load.

2nd floor Green roof beam

green roof DL = 95 psf (12) = 1.14 KLF

beam DL = $\frac{14(16)}{144}(150) = 0.233$ KLF

$\Big\} = 1.37$ KLF



LL = 100 psf (12) = 1.2 KLF

Span moment = 57.3 k-ft

Support = 85.2 k-ft

Span

$$A_s = \frac{57.3(12)}{0.9(60)(.95)(13.5)} = 1.0 \text{ in}^2$$

$$a = \frac{(1)60}{0.85(5)(17.33)} = 0.81$$

$$A_s = \frac{M_u(12)}{0.9(60)(13.5 - \frac{0.81}{2})} = 0.97 \text{ in}^2$$

Support

$$A_s = \frac{85.2(12)}{0.9(60)(.875)(13.5)} = 1.6$$

$$a = \frac{1.6(60)}{0.85(5)(14)} = 1.61$$

$$A_s = \frac{M_u(12)}{0.9(60)(13.5 - \frac{1.61}{2})} = 1.49$$

The design for the other top girders will be suitable for this beam.

APPENDIX D

D1. Shear Wall - Shear Design Values

SHEAR WALL-SHEAR DESIGN VALUES

Story	Pier	Load	Loc	V2
STORY7	P1	EQX22	Bottom	47.49
STORY7	P1	WIND1-5	Bottom	30.63
STORY5	P1	EQX22	Bottom	81.42
STORY5	P1	WIND1-11	Bottom	48.55
STORY3	P1	EQX22	Bottom	96.48
STORY3	P1	WIND1-11	Bottom	56.28
STORY2	P1	EQX2	Bottom	87.48
STORY2	P1	WIND1-11	Bottom	53.94

Pier 1	Load combo	
Story	1.6W	1.0E
6	49.008	47.49
4	77.68	81.42
2	90.048	96.48
1	86.304	87.48

Story	Pier	Load	Loc	V2
STORY6	P6	EQX22	Top	17.79
STORY6	P6	WINDEW	Top	7.44
STORY5	P6	EQX22	Bottom	41.27
STORY5	P6	WINDEW	Bottom	18.53
STORY2	P6	EQX22	Top	67.32
STORY2	P6	WINDEW	Top	41.26
STORY1	P6	EQX22	Top	39.41
STORY1	P6	WINDEW	Top	23.79

Pier 6	Load combo	
Story	1.6W	1.0E
6	11.904	17.79
4	29.648	41.27
2	66.016	67.32
1	38.064	39.41

Story	Pier	Load	Loc	V2
STORY6	P3	EQYE12	Top	30.36
STORY6	P3	WIND1-9	Top	-22.65
STORY4	P3	EQY22	Top	55.06
STORY4	P3	WIND1-6	Top	43.58
STORY2	P3	EQY22	Top	64.35
STORY2	P3	WIND1-6	Top	59.35
STORY1	P3	EQY22	Top	68.83
STORY1	P3	WIND1-6	Top	61.48

Pier 3	Load combo	
Story	1.6W	1.0E
6	-36.24	30.36
4	69.728	55.06
2	94.96	64.35
1	98.368	68.83

Story	Pier	Load	Loc	V2
STORY6	P5	EQYE11	Top	34.14
STORY6	P5	WIND1-5	Top	30.92
STORY4	P5	EQY21	Top	50.07

Pier 5	Load combo	
Story	1.6W	1.0E
6	49.472	34.14
4	68.256	50.07

STORY4	P5	WINDNS	Top	42.66
STORY2	P5	EQY21	Top	53.14
STORY2	P5	WINDNS	Top	57.1
STORY1	P5	EQY21	Top	43.84
STORY1	P5	WINDNS	Top	50.08

2	91.36	53.14
1	80.128	43.84

Story	Pier	Load	Loc	V2
STORY6	P4	EQX22	Top	-38.93
STORY6	P4	WIND1-5	Top	-41.09
STORY4	P4	EQX22	Top	-32.28
STORY4	P4	WIND1-5	Top	-50.66
STORY3	P4	WIND1-5	Bottom	-53.1
STORY2	P4	EQY22	Top	34.61
STORY1	P4	EQX21	Top	31.23
STORY1	P4	WIND1-10	Top	33.18

Pier 4	Load combo	
Story	1.6W	1.0E
6	-65.744	-38.93
4	-81.056	-32.28
2	-84.96	34.61
1	53.088	31.23

Story	Pier	Load	Loc	V2
STORY6	P8	EQY22	Top	8.95
STORY6	P8	WINDNS	Top	5.96
STORY4	P8	EQY21	Top	19.34
STORY4	P8	WINDNS	Top	18.02
STORY2	P8	EQY21	Top	33.08
STORY2	P8	WINDNS	Top	36.26
STORY1	P8	EQY21	Top	21.28
STORY1	P8	WINDNS	Top	24.05

Pier 8	Load combo	
Story	1.6W	1.0E
6	9.536	8.95
4	28.832	19.34
2	58.016	33.08
1	38.48	21.28

Story	Pier	Load	Loc	V2
STORY6	P9	EQY21	Top	7.93
STORY6	P9	WINDNS	Top	5.48
STORY4	P9	EQY21	Top	17.97
STORY4	P9	WINDNS	Top	15.88

Pier 9	Load combo	
Story	1.6W	1.0E
6	8.768	7.93
4	25.408	17.97
2	29.552	18.71

STORY3	P9	EQY21	Bottom	18.71	1	63.984	37.75
STORY3	P9	WINDNS	Bottom	18.47			
STORY1	P9	EQX22	Top	37.75			
STORY1	P9	WIND1-11	Top	39.99			

D2. Shear Wall - Flexure Design Values

Story	Pier	Load	Loc	P (Kips)	M3 (Kip-in)
STORY7	P1	DEAD	Top	-0.21	
STORY7	P1	DEAD	Bottom	-57.98	
STORY7	P1	EQX22	Top		38.772
STORY7	P1	EQXE11	Bottom		140.204
STORY7	P1	EQYE11	Top	-2.63	
STORY7	P1	WIND1-5	Top	-2.35	
STORY7	P1	WIND1-10	Top		-22.697
STORY7	P1	WINDEW	Bottom		36.237
STORY7	P1	EQY21	Top		34.676
STORY7	P1	EQY22	Bottom		-5.245
STORY7	P1	WIND1-11	Bottom		20.758
STORY7	P1	WIND1-11	Top		33.742

Axial-Dead	Axial-Live
34.7	12.3

Story	Pier	Load	Loc	P (Kips)	M3 (Kip-in)
STORY6	P1	DEAD	Top	-57.08	
STORY6	P1	DEAD	Bottom	-115.97	
STORY6	P1	EQX21	Bottom		486.683
STORY6	P1	EQX22	Top		184.316
STORY6	P1	WINDEW	Top		55.528
STORY6	P1	WINDEW	Bottom		176.055
STORY6	P1	EQYE11	Top	-3.23	
STORY6	P1	WIND1-11	Top	-2.45	
STORY6	P1	WIND1-8	Bottom		82.226
STORY6	P1	WIND1-11	Top		63.35
STORY6	P1	EQY21	Top		44.208
STORY6	P1	EQYE11	Bottom		-17.706

Axial-Dead	Axial-Live
129.7	28.3

Story	Pier	Load	Loc	P (Kips)	M3 (Kip-in)
STORY5	P1	DEAD	Top	-113.95	
STORY5	P1	DEAD	Bottom	-173.95	
STORY5	P1	EQY21	Top	-6.54	
STORY5	P1	WINDNS	Top	-4.94	
STORY5	P1	EQX22	Top		517.627
STORY5	P1	EQX21	Bottom		1016.494
STORY5	P1	WINDEW	Top		195.866
STORY5	P1	WINDEW	Bottom		422.006
STORY5	P1	EQYE12	Bottom		42.009

Axial-Dead	Axial-Live
224.7	44.3

STORY5	P1	EQY21	Top		60.506
STORY5	P1	WIND1-8	Top		124.001
STORY5	P1	WIND1-8	Bottom		200.211

Story	Pier	Load	Loc	P (Kips)	M3 (Kip-in)
STORY4	P1	DEAD	Top	-170.8	
STORY4	P1	DEAD	Bottom	-231.94	
STORY4	P1	EQX21	Top		1033.25
STORY4	P1	EQX21	Bottom		1686.013
STORY4	P1	EQY21	Top	-10.71	
STORY4	P1	WINDNS	Top	-8.97	
STORY4	P1	WINDEW	Top		437.465
STORY4	P1	WINDEW	Bottom		770.043
STORY4	P1	EQY22	Top		122.49
STORY4	P1	EQY22	Bottom		85.071
STORY4	P1	WIND1-8	Top		258.717
STORY4	P1	WIND1-8	Bottom		366.007

Axial-Dead	Axial-Live
319.7	60.3

Story	Pier	Load	Loc	P (Kips)	M3 (Kip-in)
STORY3	P1	DEAD	Top	-227.55	
STORY3	P1	DEAD	Bottom	-289.92	
STORY3	P1	EQX21	Top		1680.953
STORY3	P1	EQX21	Bottom		2443.944
STORY3	P1	EQY21	Top	-15.39	
STORY3	P1	WINDNS	Top	-14.03	
STORY3	P1	WINDEW	Top		774.482
STORY3	P1	WINDEW	Bottom		1208.278
STORY3	P1	WIND1-12	Bottom		589.737
STORY3	P1	WIND1-8	Top		440.906
STORY3	P1	EQY22	Top		208.017
STORY3	P1	EQY22	Bottom		136.237

Axial-Dead	Axial-Live
415	76.3

Story	Pier	Load	Loc	P (Kips)	M3 (Kip-in)
STORY2	P1	DEAD	Top	-276.97	
STORY2	P1	DEAD	Bottom	-347.91	
STORY2	P1	EQX21	Top		2329.709
STORY2	P1	EQX21	Bottom		3220.339
STORY2	P1	EQY22	Top	41.35	
STORY2	P1	WINDNS	Top	39.45	
STORY2	P1	WINDEW	Top		1151.351
STORY2	P1	WINDEW	Bottom		1707.071

Axial-Dead	Axial-Live
502	92.3

STORY2	P1	WIND1-12	Top		442.797
STORY2	P1	WIND1-12	Bottom		861.321
STORY2	P1	EQY21	Top		-389.722
STORY2	P1	EQY22	Bottom		222.83

Story	Pier	Load	Loc	P (Kips)	M3 (Kip-in)
STORY1	P1	DEAD	Top	-334.4	
STORY1	P1	DEAD	Bottom	-405.89	
STORY1	P1	EQX21	Top		3063.414
STORY1	P1	EQX21	Bottom		4160.1
STORY1	P1	EQY22	Top	-51.4	
STORY1	P1	WINDNS	Top	-52.34	
STORY1	P1	WINDEW	Top		1621.566
STORY1	P1	WINDEW	Bottom		2293.478
STORY1	P1	WIND1-8	Top		1010.813
STORY1	P1	WIND1-12	Bottom		1184.667
STORY1	P1	EQY22	Top		606.314
STORY1	P1	EQY22	Bottom		303.492

Axial-Dead	Axial-Live
597	112.3

Story	Pier	Load	Loc	P (Kips)	M3 (Kip-in)
STORY7	P2	WINDEW	Top	0.57	5.75808333
STORY7	P2	WIND1-9	Bottom		31.9799167
STORY7	P2	EQXE11	Top	1.86	18.7016667
STORY7	P2	EQX22	Bottom		72.49725
STORY7	P2	EQY22	Top		5.62241667
STORY7	P2	EQY22	Bottom		52.1355
STORY7	P2	WIND1-6	Bottom		26.5341667
STORY7	P2	WIND1-12	Top		-66.865

Axial-Dead	Axial-Live
52	6.2

Story	Pier	Load	Loc	P (Kips)	M3 (Kip-in)
STORY6	P2	DEAD	Top	-26.11	
STORY6	P2	EQXE11	Top	11.62	
STORY6	P2	WIND1-12	Top	4.24	
STORY6	P2	WINDNS	Top		40.1080833
STORY6	P2	WINDNS	Bottom		98.4599167
STORY6	P2	WIND1-9	Top		32.05225

Axial-Dead	Axial-Live
104.5	16.4

STORY6	P2	WIND1-9	Bottom		101.639333
STORY6	P2	EQX22	Top		29.0229167
STORY6	P2	EQX22	Bottom		211.39475
STORY6	P2	EQY22	Top		68.0895833
STORY6	P2	EQY22	Bottom		160.572417

Story	Pier	Load	Loc	P (Kips)	M3 (Kip-in)
STORY5	P2	DEAD	Top	-52.25	
STORY5	P2	WINDEW	Top	10.87	
STORY5	P2	EQX22	Top	28.85	131.432417
STORY5	P2	EQX22	Bottom		355.500083
STORY5	P2	EQY22	Top		174.135583
STORY5	P2	EQY22	Bottom		313.56525
STORY5	P2	WINDNS	Top		120.443083
STORY5	P2	WINDNS	Bottom		236.022583
STORY5	P2	WIND1-7	Top		94.81825
STORY5	P2	WIND1-7	Bottom		211.96475

Axial-Dead	Axial-Live
157.1	26.6

Story	Pier	Load	Loc	P (Kips)	M3 (Kip-in)
STORY4	P2	DEAD	Top	-78.47	
STORY4	P2	EQX22	Top	54.33	
STORY4	P2	WINDEW	Top	22.51	
STORY4	P2	EQX21	Top		243.942667
STORY4	P2	EQX21	Bottom		505.3035
STORY4	P2	EQY21	Top		321.229917
STORY4	P2	EQY21	Bottom		505.272833
STORY4	P2	WINDNS	Top		261.862167
STORY4	P2	WINDNS	Bottom		438.702583
STORY4	P2	WIND1-7	Top		200.336667
STORY4	P2	WIND1-7	Bottom		365.709083

Axial-Dead	Axial-Live
209.7	36.8

Story	Pier	Load	Loc	P (Kips)	M3 (Kip-in)
STORY3	P2	DEAD	Top	-104.69	
STORY3	P2	EQX22	Top	85.61	
STORY3	P2	WINDEW	Top	38.15	
STORY3	P2	EQX21	Top		383.917917
STORY3	P2	EQX21	Bottom		719.148167

Axial-Dead	Axial-Live
262.3	47

STORY3	P2	EQY21	Top		524.763083
STORY3	P2	EQY21	Bottom		740.358167
STORY3	P2	WINDNS	Top		459.616083
STORY3	P2	WINDNS	Bottom		682.1095
STORY3	P2	WIND1-7	Top		344.132583
STORY3	P2	WIND1-7	Bottom		560.9295

Story	Pier	Load	Loc	P (Kips)	M3 (Kip-in)
STORY2	P2	DEAD	Top	-141.04	
STORY2	P2	EQX22	Top	109.23	
STORY2	P2	WINDNS	Top	70.06	824.209167
STORY2	P2	WINDNS	Bottom		1405.61667
STORY2	P2	WIND1-7	Top		661.263
STORY2	P2	WIND1-7	Bottom		1018.25125
STORY2	P2	EQX21	Top		844.8465
STORY2	P2	EQX21	Bottom		994.667
STORY2	P2	EQY21	Top		882.79125
STORY2	P2	EQY21	Bottom		1426.59108

Axial-Dead	Axial-Live
327.2	60.5

Story	Pier	Load	Loc	P (Kips)	M3 (Kip-in)
STORY1	P2	DEAD	Top	-153.75	
STORY1	P2	EQX22	Top	187.33	
STORY1	P2	WIND1-11	Top	97.34	
STORY1	P2	EQX21	Top		426.716
STORY1	P2	EQX21	Bottom		883.45225
STORY1	P2	EQY21	Top		1140.7985
STORY1	P2	EQY21	Bottom		1414.47608
STORY1	P2	WINDNS	Top		1118.89992
STORY1	P2	WINDNS	Bottom		1417.32367
STORY1	P2	WIND1-10	Top		726.757083
STORY1	P2	WIND1-7	Bottom		1020.80975

Axial-Dead	Axial-Live
366.4	73.2

Story	Pier	Load	Loc	P (kips)	M3 (Kip-in)
STORY7	P3	WIND1-9	Bottom		28.214
STORY7	P3	WINDEW	Top		2.71141667
STORY7	P3	EQX22	Bottom		29.7111667
STORY7	P3	EQXE11	Top	-2.27	9.8395

Axial-Dead	Axial-Live
20.8	1.5

STORY7	P3	EQYE12	Bottom		26.6099167
STORY7	P3	EQY22	Top		5.47108333
STORY7	P3	WIND1-11	Bottom		32.8599167
STORY7	P3	WIND1-12	Top	-0.95	4.11308333

Story	Pier	Load	Loc	P (kips)	M3 (Kip-in)
STORY6	P3	DEAD	Top	-11.08	
STORY6	P3	EQXE11	Top	-11.39	
STORY6	P3	WIND1-12	Top	-4.83	
STORY6	P3	WIND1-6	Bottom		48.8428333
STORY6	P3	WIND1-11	Top		36.0165833
STORY6	P3	EQX22	Top		46.2925833
STORY6	P3	EQX22	Bottom		22.6746667
STORY6	P3	EQYE12	Top		14.8464167
STORY6	P3	EQY22	Bottom		76.2545
STORY6	P3	WIND1-9	Top		24.9449167
STORY6	P3	WIND1-9	Bottom		48.2865833

Axial-Dead	Axial-Live
37.9	2.5

Story	Pier	Load	Loc	P (kips)	M3 (Kip-in)
STORY5	P3	DEAD	Top	-21.95	
STORY5	P3	EQXE12	Top	-27.39	
STORY5	P3	WINDNS	Top	-11.62	
STORY5	P3	WIND1-6	Top		40.6458333
STORY5	P3	WINDNS	Bottom		105.786417
STORY5	P3	EQY22	Top		57.9020833
STORY5	P3	EQYE11	Bottom		145.21925
STORY5	P3	EQX22	Top		50.18075
STORY5	P3	EQXE12	Bottom		17.9400833
STORY5	P3	WIND1-9	Top		46.5875
STORY5	P3	WIND1-7	Bottom		61.6719167

Axial-Dead	Axial-Live
54.8	3.5

Story	Pier	Load	Loc	P (kips)	M3 (Kip-in)
STORY4	P3	DEAD	Top	-32.72	
STORY4	P3	EQY22	Top		118.924583
STORY4	P3	EQY21	Bottom		261.135833
STORY4	P3	EQX22	Top	-52.03	24.071
STORY4	P3	EQXE12	Bottom		70.65875
STORY4	P3	WINDNS	Top	-24.68	76.58725
STORY4	P3	WINDNS	Bottom		214.998917
STORY4	P3	WIND1-7	Bottom		109.55675
STORY4	P3	WIND1-9	Top		61.7929167

Axial-Dead	Axial-Live
71.5	4.5

Story	Pier	Load	Loc	P (kips)	M3 (Kip-in)
STORY3	P3	DEAD	Top	-43.38	
STORY3	P3	EQX22	Top	-82.25	
STORY3	P3	WINDNS	Top	-42.41	174.219583
STORY3	P3	WINDNS	Bottom		339.574917
STORY3	P3	WIND1-7	Top		97.38525
STORY3	P3	WIND1-10	Bottom		182.29775
STORY3	P3	EQXE12	Top		31.5930833
STORY3	P3	EQX22	Bottom		125.27575
STORY3	P3	EQY21	Top		215.983167
STORY3	P3	EQY21	Bottom		376.54875

Axial-Dead	Axial-Live
88.2	5.5

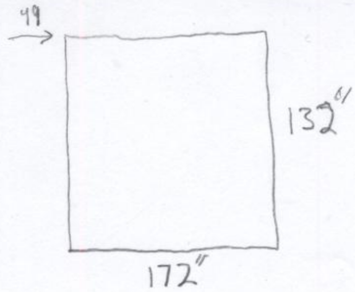
Story	Pier	Load	Loc	P (kips)	M3 (Kip-in)
STORY2	P3	DEAD	Top	-52.59	
STORY2	P3	WINDEW	Top	-25.73	
STORY2	P3	EQX22	Top	-53.8	
STORY2	P3	WIND1-7	Top		23.0703333
STORY2	P3	WIND1-10	Bottom		107.235333
STORY2	P3	EQY21	Top		48.6855
STORY2	P3	EQY21	Bottom		187.525917
STORY2	P3	EQXE12	Top		5.49675
STORY2	P3	EQX22	Bottom		115.811917
STORY2	P3	WINDNS	Top		40.9428333
STORY2	P3	WINDNS	Bottom		181.975417

Axial-Dead	Axial-Live
105.6	9.8

Story	Pier	Load	Loc	P (kips)	M3 (Kip-in)
STORY1	P3	DEAD	Top	-61.29	
STORY1	P3	EQX22	Top	-178.94	
STORY1	P3	WIND1-11	Top	-116.04	
STORY1	P3	EQY21	Top		339.40875
STORY1	P3	EQY21	Bottom		736.67125
STORY1	P3	EQXE12	Top		27.8415
STORY1	P3	EQX22	Bottom		350.238417
STORY1	P3	WINDNS	Top		323.54625
STORY1	P3	WINDNS	Bottom		714.367667
STORY1	P3	WIND1-10	Top		196.561667
STORY1	P3	WIND1-10	Bottom		496.739167

Axial-Dead	Axial-Live
120.3	11.3

D3. Sample Shear Wall- Shear Reinforcing Calculation



Pier 1 – Story 6 ~~8~~ 7

$V_u = 49\,k$

$t = 8$

$f'_c = 5000$ psi

$f_y = 60{,}000$ psi

Maximum Shear strength

$\phi V_n = \phi 10\sqrt{f'_c}\, t d$

$d = 0.8 l_w = 0.8(172'') = 137.6''$

$\phi V_n = 0.75(10)\sqrt{5000}\,(8)\dfrac{(137.6)}{1000} = 583.79 > 49$ ✓ ok

Shear Strength provided by concrete V_c

Critical section – $\dfrac{l_w}{2} = \dfrac{172}{2} = 86$

– $\dfrac{H_w}{2} = \dfrac{132}{2} = 66$ critical section @ 66"

$V_c = 3.3\sqrt{f'_c}\, t d$

$= 3.3\sqrt{5000}\,(8)\dfrac{(137.6)}{1000} = \boxed{257 \text{ kips}}$ governs

or

$V_c = \left[0.6\sqrt{f'_c} + \dfrac{L_w(1.25)\sqrt{f'_c}}{\dfrac{M_u}{V_u} - \dfrac{L_w}{2}}\right] t d = \left[0.6\sqrt{f'_c} + \dfrac{172(1.25)(\sqrt{5000})}{66 - 86}\right]\dfrac{8(137.6)}{1000} = -790$ kips

$M_u = (132 - 66)V_u$

$= 66 V_u$

$V_u = 49$ kips, $\phi V_c/2 = \dfrac{0.75(257)}{2} = 96.4$

$V_u < \phi V_c/2$

$\therefore$ use $\rho_h = 0.0025$

Shear reinforcement design

- Horizontal reinforcement -

Maximum Spacing $\geq \begin{cases} L_w/5 = \frac{172}{5} = 86 \\ 3t = 3(8) = 24 \\ 18'' \text{ governs} \end{cases}$

$P_h = \frac{A_v}{A_g}$

$\frac{\overset{2\#4}{2(0.2)}}{8(18)} = 0.0028 > 0.0025$

Horizontal reinf
use 2 curtains of #4 bars spaced at 18"

- Vertical reinforcement -

$P_n = 0.0025$ ∴ use 2 curtains of #4 bars spaced @ 18"

D4. Shear Wall- Reinforcement Summary

SHEAR REINFORCEMENT

		Horizontal		Vertical	
Pier	Story	Bar Size	Spacing	Bar Size	Spacing
P1	7	# 4	18"	# 4	18"
P1	6	# 4	18"	# 4	18"
P1	5	# 4	18"	# 4	18"
P1	4	# 4	18"	# 4	18"
P1	3	# 4	18"	# 4	18"
P1	2	# 4	18"	# 4	18"
P1	1	# 4	18"	# 4	18"
P3	7	# 4	18"	# 4	18"
P3	6	# 4	18"	# 4	18"
P3	5	# 4	18"	# 4	18"
P3	4	# 4	18"	# 4	18"
P3	3	# 4	18"	# 4	18"
P3	2	# 4	18"	# 4	18"
P3	1	# 4	18"	# 4	18"
P4	7	# 4	18"	# 4	18"
P4	6	# 4	18"	# 4	18"
P4	5	# 4	18"	# 4	18"
P4	4	# 4	18"	# 4	18"
P4	3	# 4	18"	# 4	18"
P4	2	# 4	18"	# 4	18"
P4	1	# 4	18"	# 4	18"
P5	7	# 4	18"	# 4	18"
P5	6	# 4	18"	# 4	18"
P5	5	# 4	18"	# 4	18"
P5	4	# 4	18"	# 4	18"
P5	3	# 4	18"	# 4	18"
P5	2	# 4	18"	# 4	18"
P5	1	# 4	18"	# 4	18"
P8	7	# 4	18"	# 4	18"
P8	6	# 4	18"	# 4	18"
P8	5	# 4	18"	# 4	18"
P8	4	# 4	18"	# 4	18"
P8	3	# 4	18"	# 4	18"
P8	2	# 4	18"	# 4	18"
P8	1	# 4	18"	# 4	18"

P6	7	# 4	18"	# 4	18"
P6	6	# 4	18"	# 4	18"
P6	5	# 4	18"	# 4	18"
P6	4	# 4	18"	# 4	18"
P6	3	# 4	18"	# 4	18"
P6	2	# 4	18"	# 4	18"
P6	1	# 4	18"	# 4	18"
P9	7	# 4	18"	# 4	18"
P9	6	# 4	18"	# 4	18"
P9	5	# 4	18"	# 4	18"
P9	4	# 4	18"	# 4	18"
P9	3	# 4	18"	# 4	18"
P9	2	# 4	18"	# 4	18"
P9	1	# 4	18"	# 4	18"

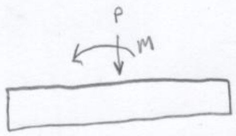
FLEXURE REINFORCEMENT

Pier	Story	Bar Size	Spacing
P1	7	# 4	18"
P1	6	# 4	18"
P1	5	# 4	18"
P1	4	# 4	18"
P1	3	# 4	18"
P1	2	# 4	18"
P1	1	# 4	18"
P2	7	# 4	18"
P2	6	# 4	18"
P2	5	# 4	18"
P2	4	# 4	18"
P2	3	# 4	18"
P2	2	# 5	12"
P2	1	# 5	12"
P3	7	# 4	18"
P3	6	# 4	18"
P3	5	# 4	12"
P3	4	# 4	12"
P3	3	# 5	12"
P3	2	# 5	12"
P3	1	# 5	12"

APPENDIX E

E1. Foundation Design

Footing design



$P = 284.6\ K$ $\quad q_a = 5\ ksi$

$M = 123.7$

$\frac{M}{P} = e \Rightarrow \frac{123.7}{284.6} = 0.43'$

$B^3 \; 5\ ksi = \frac{284.6(B^3)}{B^2} + \frac{123.7(6)\cancel{B^3}}{\cancel{B^3}}$

$(5)B^3 = (284.6)B + 742.2$

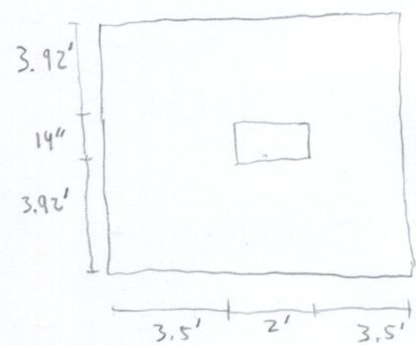
$5B^3 - 284.6B - 742.2 = 0$

$B = 8.61 \cong 9'$

$\frac{B}{6} = \frac{9}{6} = 1.5' > 0.43'$ ∴ inside kern

$B' = 9 - 2(0.43) = 8.14$

$A' = 8.14'(9)' = 73.26\ ft^2$



Overturning

$M_{over} = 123.7$

$M_{resist} = 284.6(4.5') = 1280.7$

$FS = \frac{M_{resist}}{M_{over}} = \frac{1281}{124} = 10 > 2$ ✓ok

Wide beam shear

$q_u = \frac{284.6}{73.26} = 3.88$

$3.88(3.5 - d/12)1' = \frac{(0.75)(2)\sqrt{3000}(12)d}{1000}$

$13.6 - 0.32(d) = 0.98(d)$

$d \geq 10.4''$

Punching Shear

$\phi V_c = 164$

$$4d^2 + 2d(b + c) = \frac{P_u}{\phi V_c}$$

$$4d^2 + 2d(14 + 24) = \frac{284{,}600}{164}$$

$$4d^2 + 2d(38) = 1735 \longrightarrow d^2 + 19d - 1735 = 0$$

$$d \geq 33''$$

$$h = 33 + 3 + 5/8 = 36.625''$$

$$h = 38'' \quad d = 38 - 3 - 5/8 = 34.375''$$

flexural

$l = 3.5$

$$M_u = \frac{3.88(3.5)^2}{2} = 23.8^{\ '-k}$$

$$a = \frac{A_s(60)}{0.85(3)(12)} = \frac{1.96 A_s}{2} = 0.98$$

$$\frac{23.8(12)}{60(0.9)} = 34.375 A_s - 0.98 A_s^2$$

$$5.3 = 34.375 A_s - 0.98 A_s^2$$

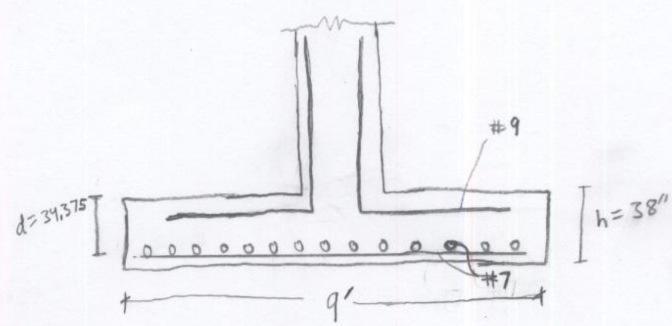
$$A_s = 0.15 \text{ in}^2/\text{ft}$$

Shrinkage + Temp

$$\frac{0.15}{12(38)} = 0.0003 < 0.0018 \quad \text{no good}$$

$$0.0018(12)(38) = 0.82$$

$$\#7 @ 8'' \quad A_s = 0.90 \text{ in}^2/\text{ft}$$



Footings – Retaining wall

90 psf + 5 psf = 95 psf   95(15.67) = 1489 plf

Ribr/slab   mech

LL = 100 psf   100(15.67) = 1567 plf

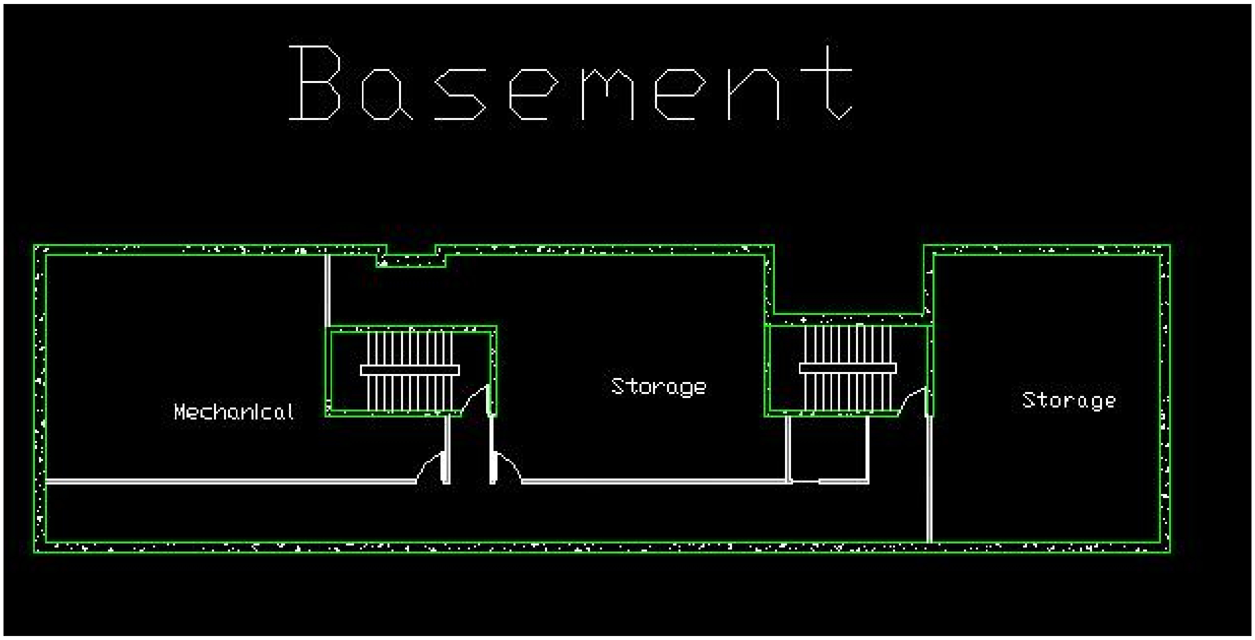
1.2(1489) + 1.6(1567) = 4.3 klf (1' section) = 4.3 k

∴ 4.3 k on 1 ft section of retaining wall from slab.

PCA Col. used to check this 1 ft section of retaining wall for the existing retaing wall design with the new floor loads. Check proved that the existing reinforcing in the retaining wall will be able to support the slab loads

APPENDIX F

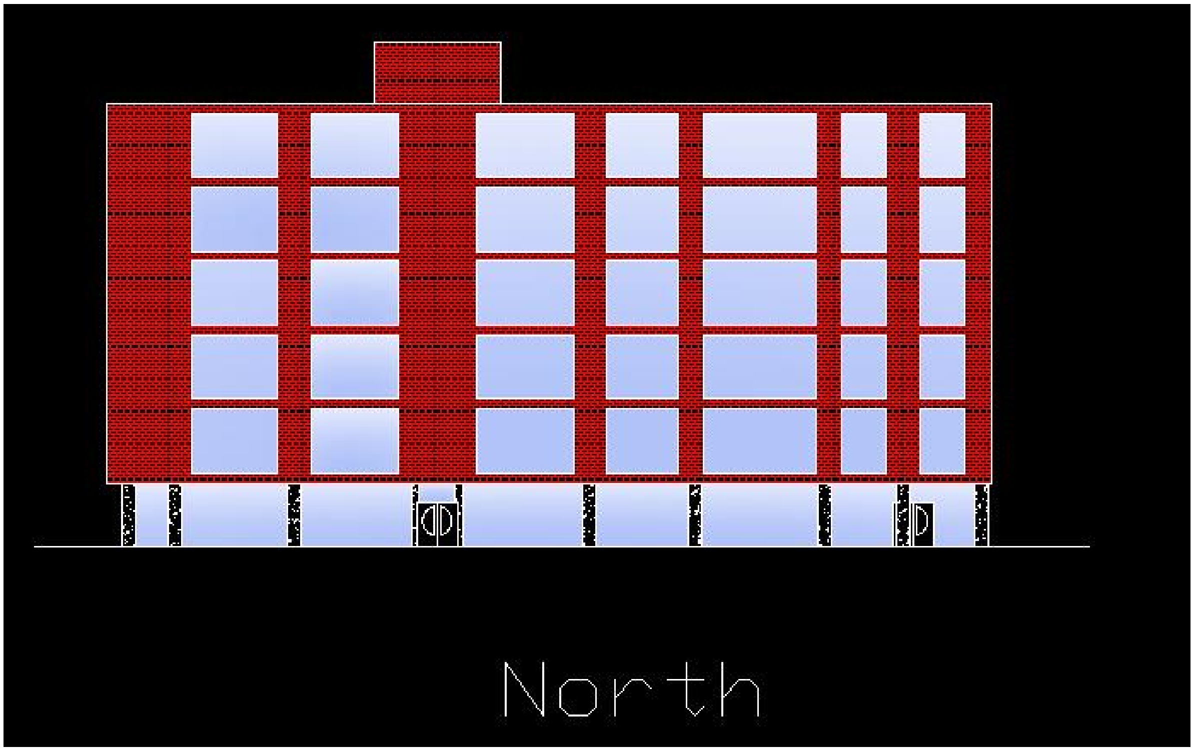
F.1 Architecture Redesign-Floor Plans

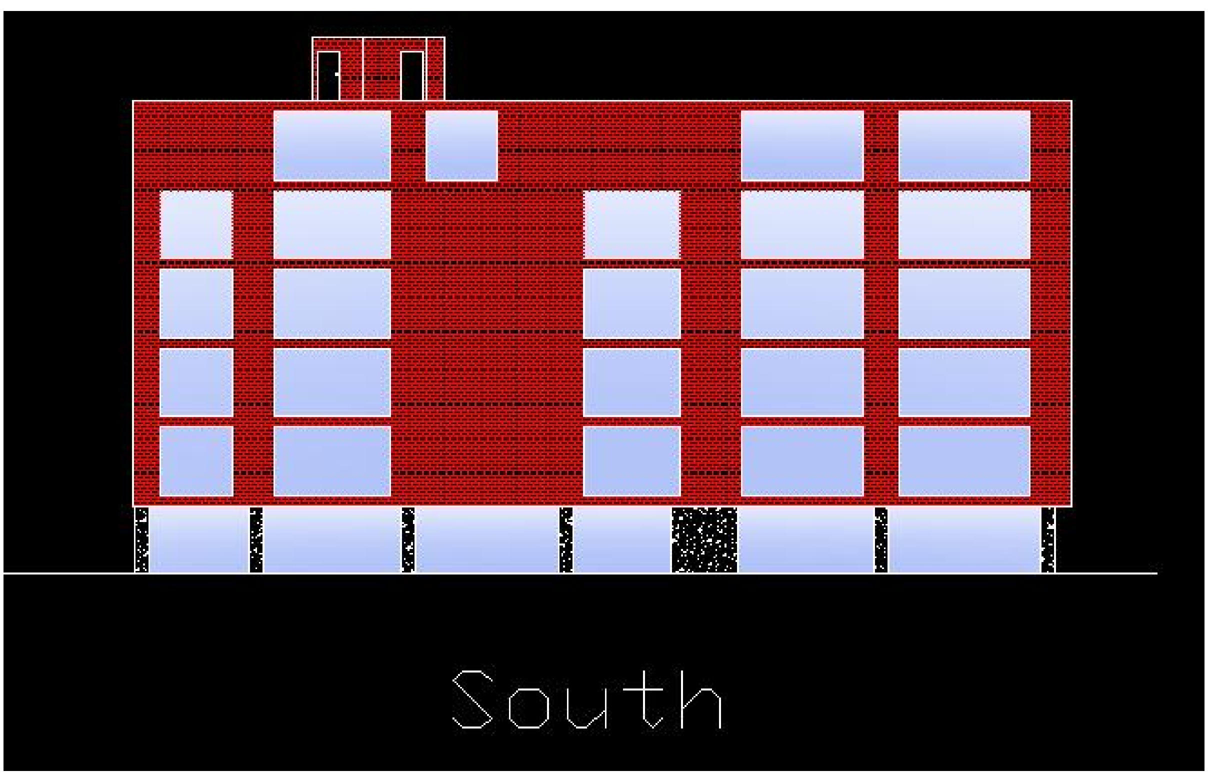


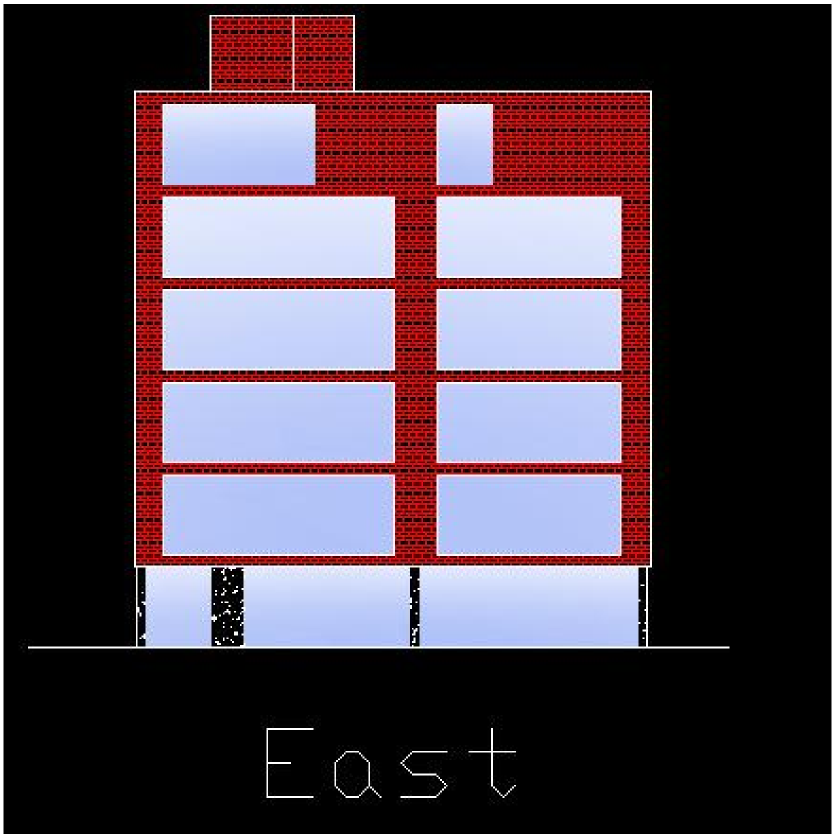

2nd-5th Floor
Con. Room
Office Space
Recp.
Office Space
Strg.
Kitchen
Corridor
Kitchen
Recp.
Office Space
Office Space
Con. Room
Storage

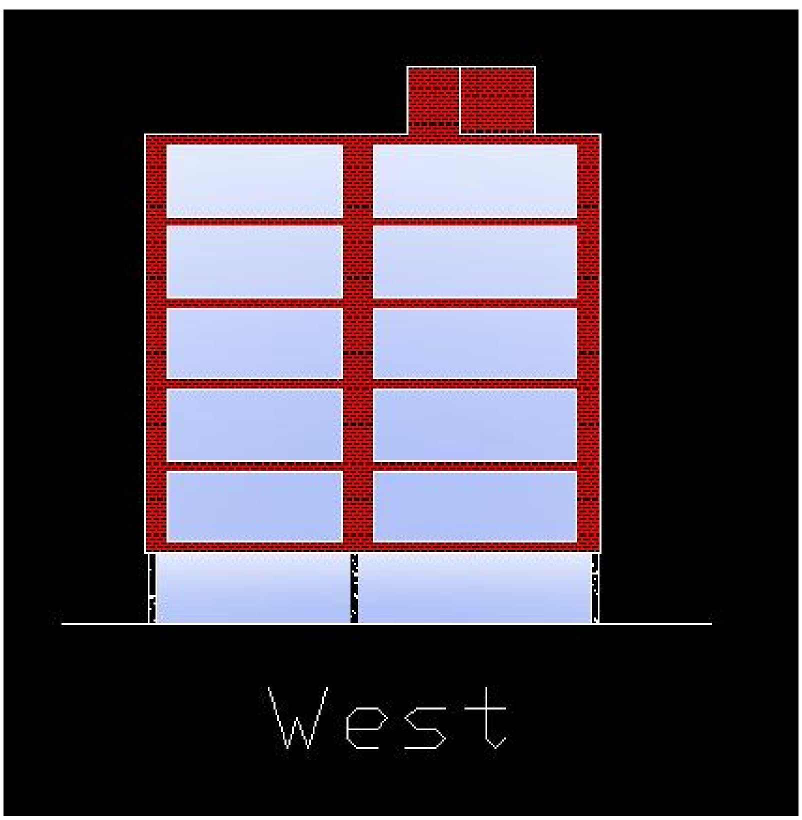

6th Floor
Office Space
Office Space
Office Space
Kitchen
Reception
Office Space
Conference Room
Storage

F.2 Architecture Redesign-Elevations

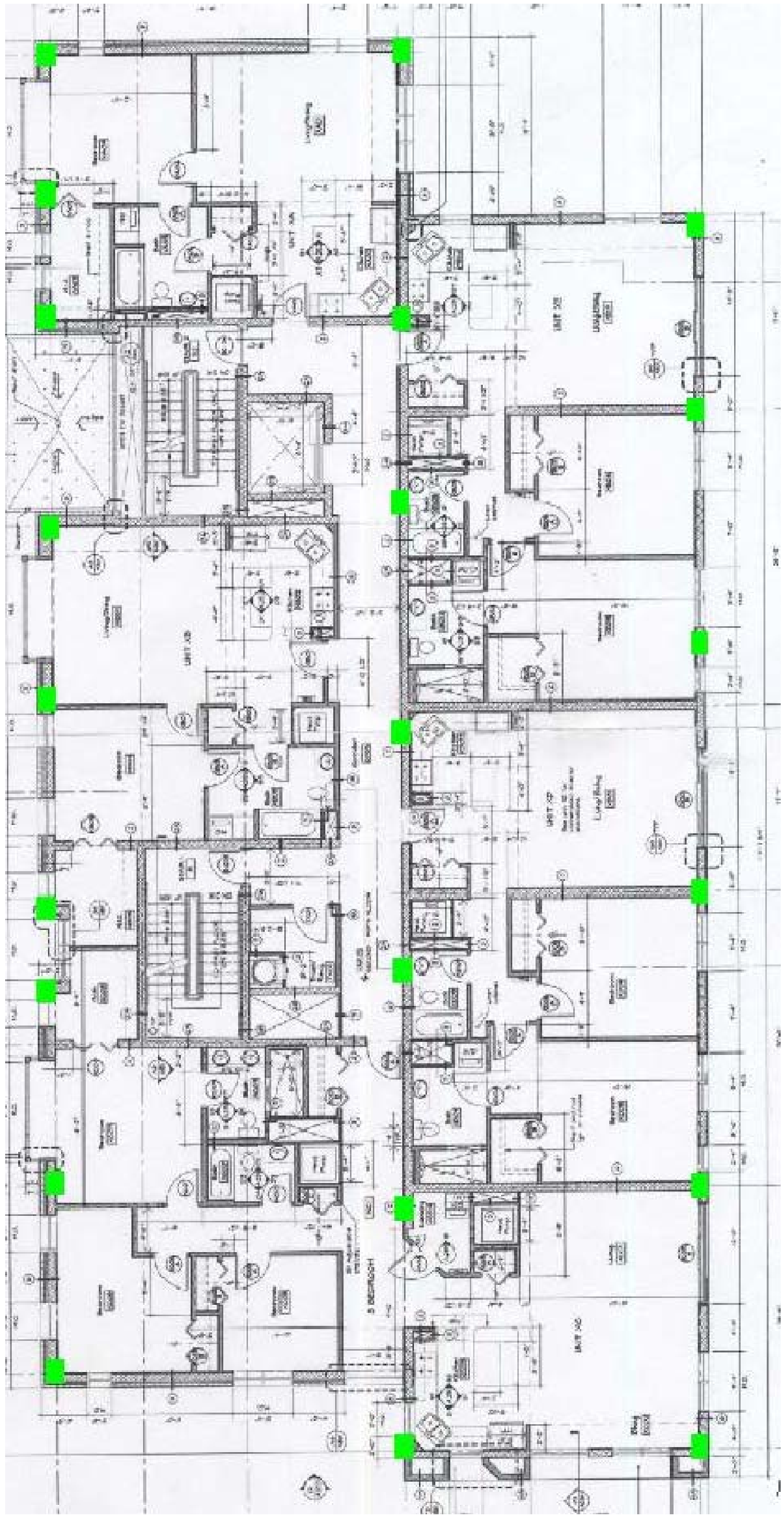





East


West

F.3 Existing Architecture with Columns of New Structure



APPENDIX G

G.1 Cost of Existing Structure

Gateway Commons
Ithaca, New York

- Concrete Walks = $66,052
- Concrete Footings, Cast-In-Place Foundation Walls, Slab-On-Grade, and Elevator Pit = $302,681
- Cast-In-Place Masonry Wall Caps = $12,600
- Concrete Reinforcement = $ 65,920
- Pre-Cast Concrete Plank = $483,678
- Masonry = $830,041
- Structural & Misc. Steel = $317,869

G.2 Cost of New Structure

Cost Estimate

Slab

Bay sizes in the building are not typical but they can be roughly estimated as 26'x22'. RS Means Square Costs was used to determine the cost of the pan joist slab. A value for cost/S.F. was determined by interpolating between a 20'x20' and a 30'x30' bay. It was determined that a cost/SF of 17.8 \$/SF will be used for the slab.

SF for typical Floor

$26(114.67) + 31.33(30.6) + (31.33)(29.9) + 31.33(25.33) + 18.67(7.6) + 18.67(13.8)$
$+ 13.8(7.5) + 11.2(6.8) = 7030 \text{ SF}$

Floor 2 – Roof $\Rightarrow 6(7030) = 42{,}180$ SF

Roof over stairs $\Rightarrow 18.67(10) = 186.7$ SF

2nd Floor green roof $\Rightarrow 17.33(12.75) = 221$ SF

roof overhang $\Rightarrow 2.67(30.67 + 84.5 + 31.33 + 26 + 7.6 + 7.6 + 118 + 31.33 + 15.85 + 26)$
$= 1011$ SF

1st Floor $\Rightarrow 31.33(79.2) - (10)(18.67) + 31.33(25.33) + 7.5(13.8) + 11.2(6.8)$
$= 3268$

Total slab area $= 3268 + 1011 + 221 + 186.7 + 42{,}180 = 46{,}867$ SF

Cost $= 46{,}867(17.8) = \boxed{834{,}232\ \$}$

Beams

RS Means Facilities was used to determine the total construction cost of the Girders. The average girder span is 20'. Girder span in RS Means are 10' and 25' so cost values for a 20' span beam will be interpolated.

ft	\$/cy
10	1125
20	x — 1042 \$/cy
25	1000

Top & bottom beams => 14" x 16"

$$\frac{14(16)}{144} = 1.56 \text{ SF} (79.2 + 25.33 + 112.67) = 339 \text{ CFT}$$

Middle beams => 14" x 18"

$$\frac{14(18)}{144} = 1.75 (130.5) = 228 \text{ CFT}$$

$$567 \text{ CFT} (6 \text{ Floors}) = 3402 \text{ CFT}$$

2nd Floor Green Roof => $\frac{14(16)}{144} = 1.56(17.33) = 27 \text{ CFT}$

$$\frac{6(8)}{144}(17.33) = 6 \text{ CFT}$$

$$\text{Total Volume} = 6 + 27 + 3402 = \frac{3435 \text{ CFT}}{27} = 127.2 \text{ cy}$$

$$\text{Cost} = 127.2(1042) = \boxed{132{,}542\$}$$

Retaing Walls

The Cost of Retaing walls from RS Means Facilities Construction Cost Data is 247 \$/cy.

$\frac{14''}{12}(11') = 12.8\text{SF}$ $(129.7 + 33.67 + 124.2 + 6.55 + 6.55 + 32.5) = 4265\text{CFT}$

Volume $= \frac{4265}{27} = 158$ cy

Cost $= 158(247) = \boxed{39{,}026\$}$

Slab on Grade

RS Means Building Cost Construction Data has a cost for 4" and 6" SOG Costs. The cost for Gateway Commons' 5" SOG will be interpolated.

4 221
5 x — 204 \$/cy
6 187

$26'(112.67) + 24.67(31.33) + 13.75(18.67) + 29.2(31.33) + 30.6(31.33)$
$+ 7.6(18.67) + 18.67(13.75) = 6231\text{SF}\left(\frac{5}{12}\right) = \frac{2596\text{ CFT}}{27} = 96\text{cy}$

Cost $= 204(96) = \boxed{19{,}584\$}$

Shear Walls – Labor

The Cost of 8" Wall Placed by pump is 40.5 \$/cy according to RS Means Facilities. This price does not account for Concrete, formwork, and rebar only Labor.

Length $= 10(4) + 17.33(3) = 92$ ft

height $= 11'/\text{Floor}(7) = 77$ ft

Roof $\Rightarrow 10(2) + 17.33 = 37.33$ ft

$92(77)(0.67) = 4746$ CFT

$37.33(9')(0.67) = 225$ CFT

$= \frac{4971\text{CFT}}{27}$

$= 184$ cy

Labor Cost $= 40.5(184) = \boxed{7{,}452\$}$

Shear Wall – Formwork

RS Means Facilities gives a price for Formwork on wall between 8'–16' as 8.6 \$/Square foot of Contact Area.

SFCA = 11'(18.67(3) + 40) = 1056 SFCA/Floor (7 Floors) = 7392 SFCA

For Roof => 10(2) + 18.67 = 38.67 SFCA

7431 SFCA

Formwork Cost = 7431 (8.6) = $\boxed{63{,}907\,\$}$

Shear Wall – Concrete

Shear walls have an f'c = 5000 psi. The Cost of 5000 psi concrete equals 106 \$/cy.

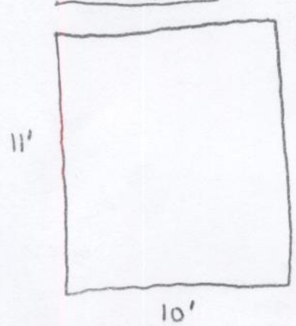
Volume = 184 cy

Cost of Material = 184(106) = $\boxed{19{,}504\,\$}$

Shear Wall – Rebar

The Cost of Reinforcing for walls is 0.85 \$/Lb according to RS Means Facilities.

Wall type 1



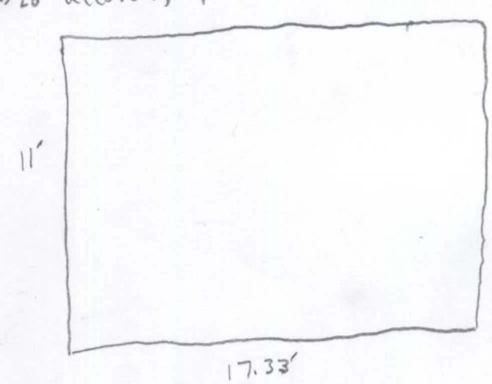
#4 @ 18"

$\frac{18}{12} = 1.5'$

$\frac{11'}{1.5'}$ = 7 rows of 2 curtains of #4 bars Horizontally

$\frac{10'}{1.5'}$ = 6 rows of 2 curtains of #4 bars Vertically

7(2)(10') = 140'
6(2)(11') = 132'
= 272 ft/wall (30) = 8160'   (Wall type 1)



#4 @ 18" same horizontal spacing

$\frac{17.33}{1.5}$ = 11 rows of 2 curtains of #4 bars Vertically

7(2)(17.33) = 243'
11(2)(11) = 242'
= 485 ft/wall (22) = 10,670'   (wall type 2)

Cost of Rebar = 0.668 lb/ft (8160 + 10,670) = 12,578 lb × 0.85 \$/lb
= $\boxed{10{,}692\,\$}$

Footing

Since there is over 50cy of spread footings 280 \$/cy will be the cost of the spread footings. The strip footing will cost 340 \$/cy. Both these values were determined by RS Means Facilities Construction Cost Data.

Spread Footing => $9(9)(38/12) = 257\ \text{CFT}/27 = 9.5\ \text{cy}\ (6) = 57\ \text{cy}$

Continuous Footing => $[24(15.3) - 3.3(12)] + [15.75(24.5) - (12)(4.75)] + 5.67(5)$
$+ 32.5(7) + 5(115.9) + 7(32.5) + 5(32.8) + 7(5.2) + 86.67(5)$
$= 327.6 + 329 + 1697 = 2354\ \text{SF}\ (1.5') = 3531\ \text{CFT}/27$
$= 131\ \text{cy}$

Cost = 57cy (280) = 15,960 \$
= 131cy (340) = 44,540 \$

Total Footing cost = 60,500 \$

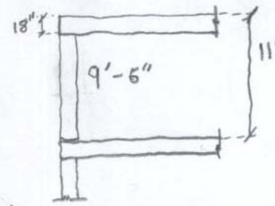
Total cost of building structure = 834,232 + 166,197 + 132,542 + 39,026 + 19,584 + 7452 + 63,907 + 19,504 + 10,692

= 1,293,136 \$

Columns

RS Means Facilities Construction Cost Data was used to determine the full cost of building Columns. Columns used in the building are 14"x24" the equal area of a square column is an 18"x18" column. Since RS Means gives data for 16"x16" and 24"x24" values for 18"x18" were interpolated. Average reinforcement was used.

16 sq	1450
18 sq	x ~ 1369 \$/cy
24 sq	1125



$$\text{Volume} = 9.5'\left(\frac{14\times 24}{144}\right) = \frac{22\text{ CFT}}{27} = 0.82\text{ cy/col}$$

22 col. (6 floors) = 132
16 Columns (for basement) = 16
148 col.

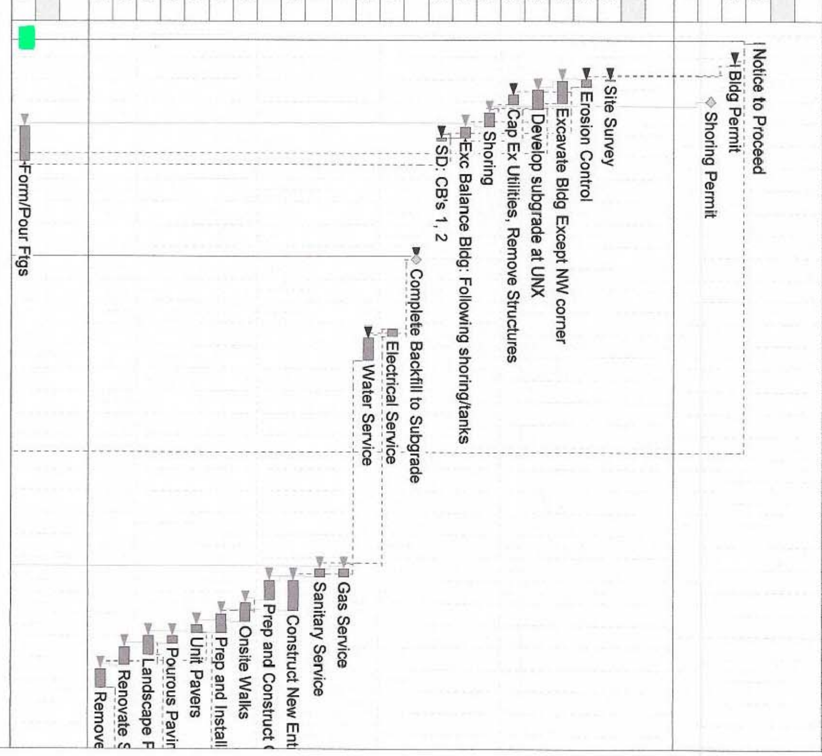
$$\text{Volume} = 0.82(148) = 121.4\text{ cy}$$

$$\text{Cost} = 1369(121.4) = \boxed{166{,}197\ \$}$$

G.3 Schedule of Existing Structure

NEW BUILDING AT GATEWAY COMMONS — NORTHEAST CONSTRUCTION SERVICES

Act ID	Area	Description	Original Duration	Early Start	Early Finish
GC General					
GN1000		Notice to Proceed	1	14OCT05 *	14OCT05
GN1010		Bldg Permit	1	28OCT05 *	28OCT05
GN1020		Shoring Permit	0	21NOV05 *	
GN1030		Occupy Bldg	0		02FEB07
GC- Sitework					
SW1000		Site Survey	1	07NOV05 *	07NOV05
SW1010		Erosion Control	5	07NOV05 *	11NOV05
SW1050		Excavate Bldg Except NW corner	10	08NOV05	21NOV05
SW1052		Develop subgrade at UNX	10	14NOV05	25NOV05
SW1020		Cap Ex Utilities, Remove Structures	4	17NOV05 *	22NOV05
SW1030		Shoring	7	28NOV05	06DEC05
SW1060		Exc Balance Bldg: Following	5	07DEC05	13DEC05
SW1040		SD: CB's 1, 2	2	14DEC05	15DEC05
SW1070		Complete Backfill to Subgrade	0	28FEB06	
SW1100		Electrical Service	3	13APR06 *	17APR06
SW1080		Water Service	10	19APR06 *	02MAY06
SW1090		Gas Service	5	11SEP06	15SEP06
SW1110		Sanitary Service	5	11SEP06	15SEP06
SW1120		Construct New Entrances and DOT	15	18SEP06	06OCT06
SW1130		Prep and Construct Onsite Curb	10	18SEP06	29SEP06
SW1180		Onsite Walks	10	02OCT06	13OCT06
SW1140		Prep and Install Onsite Paving	10	09OCT06	20OCT06
SW1170		Unit Pavers	5	16OCT06	20OCT06
SW1150		Pourous Paving	5	23OCT06	27OCT06
SW1200		Landscape Prep and Plantings	10	23OCT06	03NOV06
SW1190		Renovate State St Driveway	10	30OCT06	10NOV06
SW1160		Remove Original Driveway and	10	13NOV06	24NOV06
GC Shell Construction					
Basement (A-D x 1-12)					
SH1030	BASE	Form/Pour Ftgs	15	07DEC05	28DEC05



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Finish date	02FEB07
Data date	29SEP05
Page number	1A

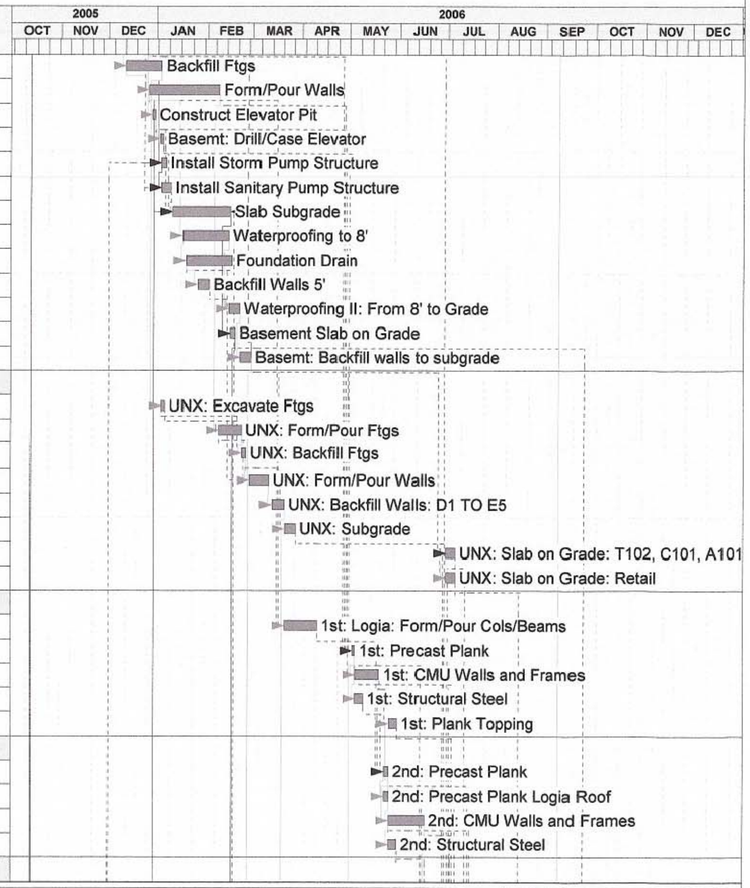
CONTRACT SCHEDULE- REV 061206

Early bar
Progress bar
Critical bar
Summary bar
Start milestone point
Finish milestone point

NEW BUILDING AT GATEWAY COMMONS

NORTHEAST CONSTRUCTION SERVICES

Act ID	Area	Description	Original Duration	Early Start	Early Finish
SH1070	BASE	Backfill Ftgs	15	12DEC05	03JAN06
SH1090	BASE	Form/Pour Walls	30	27DEC05	07FEB06
SH1060	BASE	Construct Elevator Pit	2	29DEC05	30DEC05
SH1155	BASE	Basemt: Drill/Case Elevator	2	03JAN06	04JAN06
SH1000	BASE	Install Storm Pump Structure	3	04JAN06	06JAN06
SH1010	BASE	Install Sanitary Pump Structure	4	04JAN06	09JAN06
SH1150	BASE	Slab Subgrade	25	11JAN06	14FEB06
SH1110	BASE	Waterproofing to 8'	20	17JAN06	13FEB06
SH1125	BASE	Foundation Drain	20	19JAN06	15FEB06
SH1140	BASE	Backfill Walls 5'	5	26JAN06	01FEB06
SH1300	BASE	Waterproofing II: From 8' to Grade	5	14FEB06	20FEB06
SH1160	BASE	Basement Slab on Grade	3	15FEB06	17FEB06
SH1310	BASE	Basemt: Backfill walls to subgrade	5	21FEB06	27FEB06
Slab on Grade Area (D-E x 1-11)					
SH1170	UNX	UNX: Excavate Ftgs	2	04JAN06	05JAN06
SH1180	UNX	UNX: Form/Pour Ftgs	10	08FEB06	21FEB06
SH1240	UNX	UNX: Backfill Ftgs	3	22FEB06	24FEB06
SH1200	UNX	UNX: Form/Pour Walls	10	27FEB06	10MAR06
SH1210	UNX	UNX: Backfill Walls: D1 TO E5	6	13MAR06	20MAR06
SH1270	UNX	UNX: Subgrade	5	21MAR06	27MAR06
SH1280	UNX	UNX: Slab on Grade: T102, C101,	3	30JUN06 *	05JUL06
SH1282	UNX	UNX: Slab on Grade: Retail	3	30JUN06	05JUL06
First Floor					
SH1610	1ST	1st: Logia: Form/Pour Cols/Beams	15	21MAR06	10APR06
SH1290	1ST	1st: Precast Plank	2	03MAY06 *	04MAY06
SH1320	1ST	1st: CMU Walls and Frames	11	05MAY06	19MAY06
SH1630	1ST	1st: Structural Steel	3	05MAY06	09MAY06
SH1330	1ST	1st: Plank Topping	3	26MAY06	30MAY06
Second Floor					
SH1350	2ND	2nd: Precast Plank	3	23MAY06 *	25MAY06
SH1620	2ND	2nd: Precast Plank Logia Roof	3	23MAY06	25MAY06
SH1360	2ND	2nd: CMU Walls and Frames	16	26MAY06	16JUN06
SH1640	2ND	2nd: Structural Steel	3	26MAY06	30MAY06
Third Floor					



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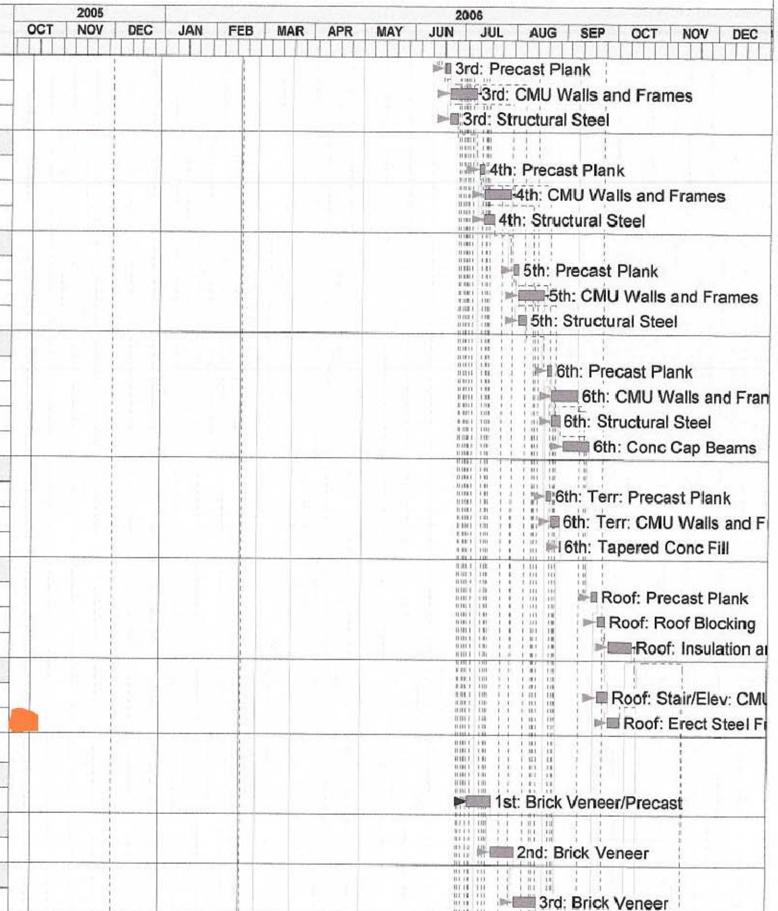
CONTRACT SCHEDULE- REV 061206

Early bar
Progress bar
Critical bar
Summary bar
Start milestone point
Finish milestone point

NEW BUILDING AT GATEWAY COMMONS

NORTHEAST CONSTRUCTION SERVICES

Act ID	Area	Description	Original Duration	Early Start	Early Finish
SH1380	3RD	3rd: Precast Plank	3	19JUN06	21JUN06
SH1400	3RD	3rd: CMU Walls and Frames	11	22JUN06	07JUL06
SH1650	3RD	3rd: Structural Steel	3	22JUN06	26JUN06
Fourth Floor					
SH1410	4TH	4th: Precast Plank	3	10JUL06	12JUL06
SH1430	4TH	4th: CMU Walls and Frames	12	13JUL06	28JUL06
SH1660	4TH	4th: Structural Steel	4	13JUL06	18JUL06
Fifth Floor					
SH1440	5TH	5th: Precast Plank	3	31JUL06	02AUG06
SH1460	5TH	5th: CMU Walls and Frames	12	03AUG06	18AUG06
SH1670	5TH	5th: Structural Steel	3	03AUG06	07AUG06
Sixth Floor					
SH1470	6TH	6th: Precast Plank	3	21AUG06	23AUG06
SH1490	6TH	6th: CMU Walls and Frames	12	24AUG06	08SEP06
SH1680	6TH	6th: Structural Steel	3	24AUG06	28AUG06
SH1502	6TH	6th: Conc Cap Beams	12	31AUG06	15SEP06
6th Floor Terrace					
SH1482	6TER	6th: Terr: Precast Plank	3	21AUG06	23AUG06
SH1492	6TER	6th: Terr: CMU Walls and Frames	3	24AUG06	28AUG06
SH1472	6TER	6th: Tapered Conc Fill	1	29AUG06	29AUG06
Main Roof					
SH1500	ROOF	Roof: Precast Plank	3	18SEP06	20SEP06
SH1590	ROOF	Roof: Roof Blocking	3	21SEP06	25SEP06
SH1600	ROOF	Roof: Insulation and Roofing	10	28SEP06	11OCT06
Stair Penthouse					
SH1570	PH	Roof: Stair/Elev: CMU Walls, Frames	5	21SEP06	27SEP06
SH1580	PH	Roof: Erect Steel Framing and	5	28SEP06	04OCT06
GC Exterior Skin					
First Floor					
SK1080	1ST	1st: Brick Veneer/Precast	10	05JUL06 *	18JUL06
Second Floor					
SK1090	2ND	2nd: Brick Veneer	10	19JUL06	01AUG06
Third Floor					
SK1100	3RD	3rd: Brick Veneer	10	02AUG06	15AUG06



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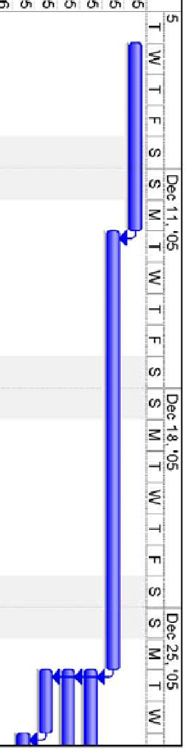
Data date	29SEP05
Page number	3A

CONTRACT SCHEDULE- REV 061206

Early bar
Progress bar
Critical bar
Summary bar
Start milestone point
Finish milestone point

G.4 Schedule of New Structure

ID	story	Task Name	Duration	Start
1	BASE	form/pour strip footings	4 days	Wed 12/7/05
2	BASE	backfill footings	10 days	Tue 12/13/05
3	BASE	form/pour shear walls	5 days	Tue 12/27/05
4	BASE	form/pour retaining walls	30 days	Tue 12/27/05
5	BASE	construct elevator pit	2 days	Tue 12/27/05
6	BASE	drill/case elevator	2 days	Thu 12/29/05
7	BASE	install storm pump structure	3 days	Mon 1/2/06
8	BASE	install sanitary pump structure	4 days	Thu 1/5/06
9	BASE	slab subgrade	25 days	Wed 1/11/06
10	BASE	waterproofing to 8'	6 days	Tue 2/7/06
11	BASE	foundation drain	20 days	Wed 1/11/06
12	BASE	backfill walls 5'	5 days	Wed 2/15/06
13	BASE	waterproofing from 8' to grade	5 days	Wed 2/15/06
14	BASE	basement slab on grade	2 days	Wed 2/8/06
15	BASE	backfill walls to subgrade	5 days	Wed 2/22/06
16	1ST	excavate spread footings	2 days	Wed 3/1/06
17	1ST	form/pour spread footings	2 days	Fri 3/3/06
18	1ST	backfill footings	3 days	Tue 3/7/06
19	1ST	form/pour 1st floor slab	3 days	Wed 3/1/06
20	1ST	subgrade	5 days	Fri 3/10/06
21	1ST	slab on grade	3 days	Fri 3/17/06
22	1ST	form/pour columns	4 days	Mon 3/6/06
23	1ST	form/pour shear walls	4 days	Mon 3/6/06
24	2ND	form/pour beams	4 days	Wed 3/22/06
25	2ND	form/pour 2nd floor slab	6 days	Tue 3/28/06
26	2ND	form/pour columns	4 days	Wed 4/5/06
27	2ND	form/pour shear walls	4 days	Wed 4/5/06
28	2ND	form/pour 2nd floor green roof	1 day	Tue 3/28/06
29	3RD	form/pour beams	4 days	Tue 4/11/06
30	3RD	form/pour 3rd floor slab	6 days	Mon 4/17/06
31	3RD	form/pour columns	4 days	Tue 4/25/06
32	3RD	form/pour shear walls	4 days	Tue 4/25/06
33	4TH	form/pour beams	4 days	Mon 5/1/06
34	4TH	form/pour 4th floor slab	6 days	Fri 5/5/06
35	4TH	form/pour columns	4 days	Mon 5/15/06
36	4TH	form/pour shear walls	4 days	Mon 5/15/06
37	5TH	form/pour beams	4 days	Fri 5/19/06
38	5TH	form/pour 5th floor slab	6 days	Thu 5/25/06
39	5TH	form/pour columns	4 days	Fri 6/2/06



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ID		story	Task Name	Duration	Start
40		5TH	form/pour shear walls	4 days	Fri 6/2/06
41		6TH	form/pour beams	4 days	Thu 6/8/06
42		6TH	form/pour 6th floor slab	6 days	Wed 6/14/06
43		6TH	form/pour columns	4 days	Thu 6/22/06
44		6TH	form/pour shear walls	4 days	Thu 6/22/06
45		ROOF	form/pour beams	4 days	Wed 6/28/06
46		ROOF	form/pour roof slab	7 days	Tue 7/4/06
47		ROOF	insulation and roofing	10 days	Tue 7/18/06
48		ROOF	stair/elevator walls	3 days	Thu 7/13/06

5					Dec 11, '05							Dec 18, '05							Dec 25, '05			
T	W	T	F	S	S	M	T	W	T	F	S	S	M	T	W	T	F	S	S	M	T	W

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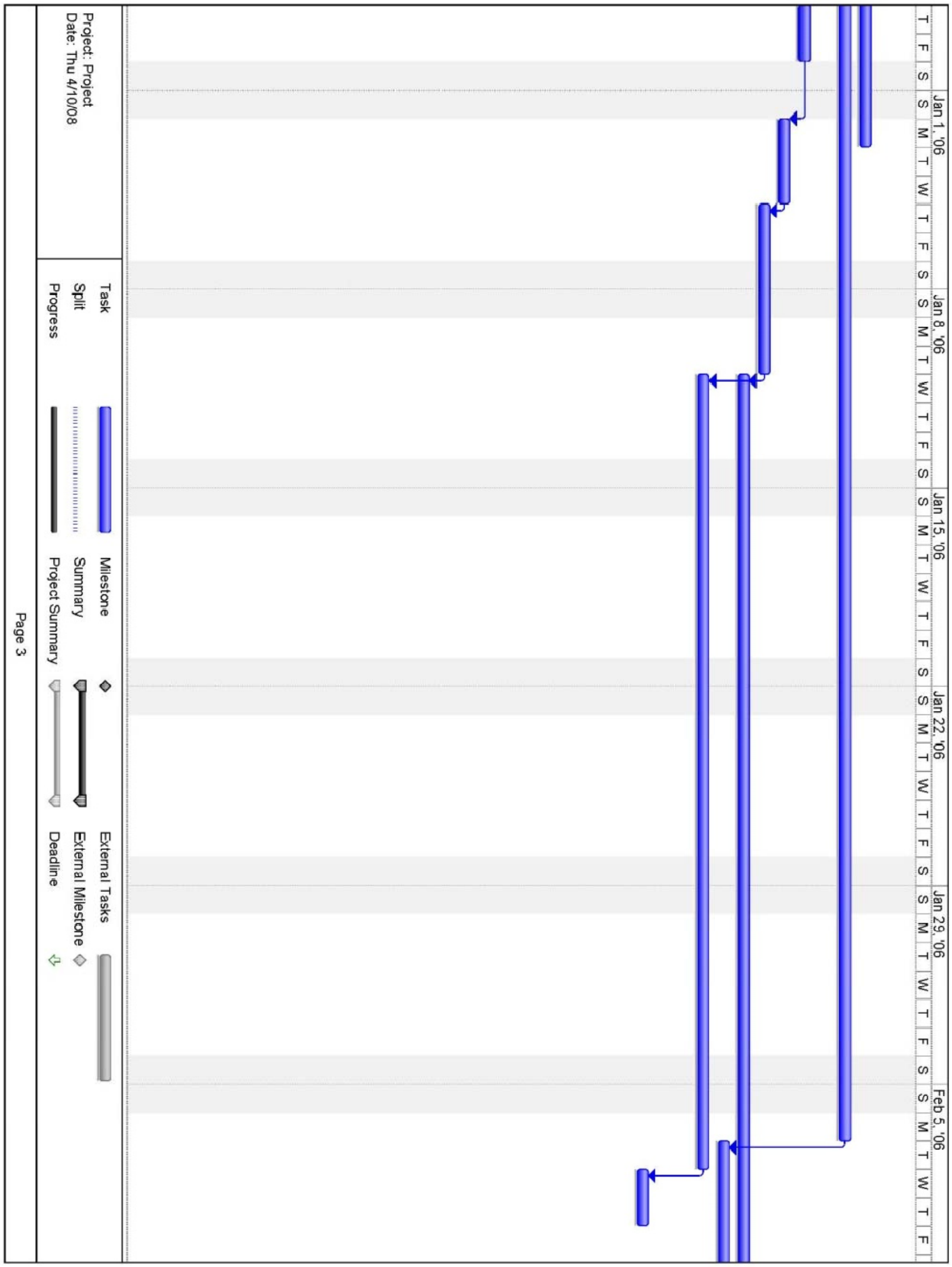
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Jan 15, '06
Jan 22, '06
Jan 29, '06
Feb 5, '06
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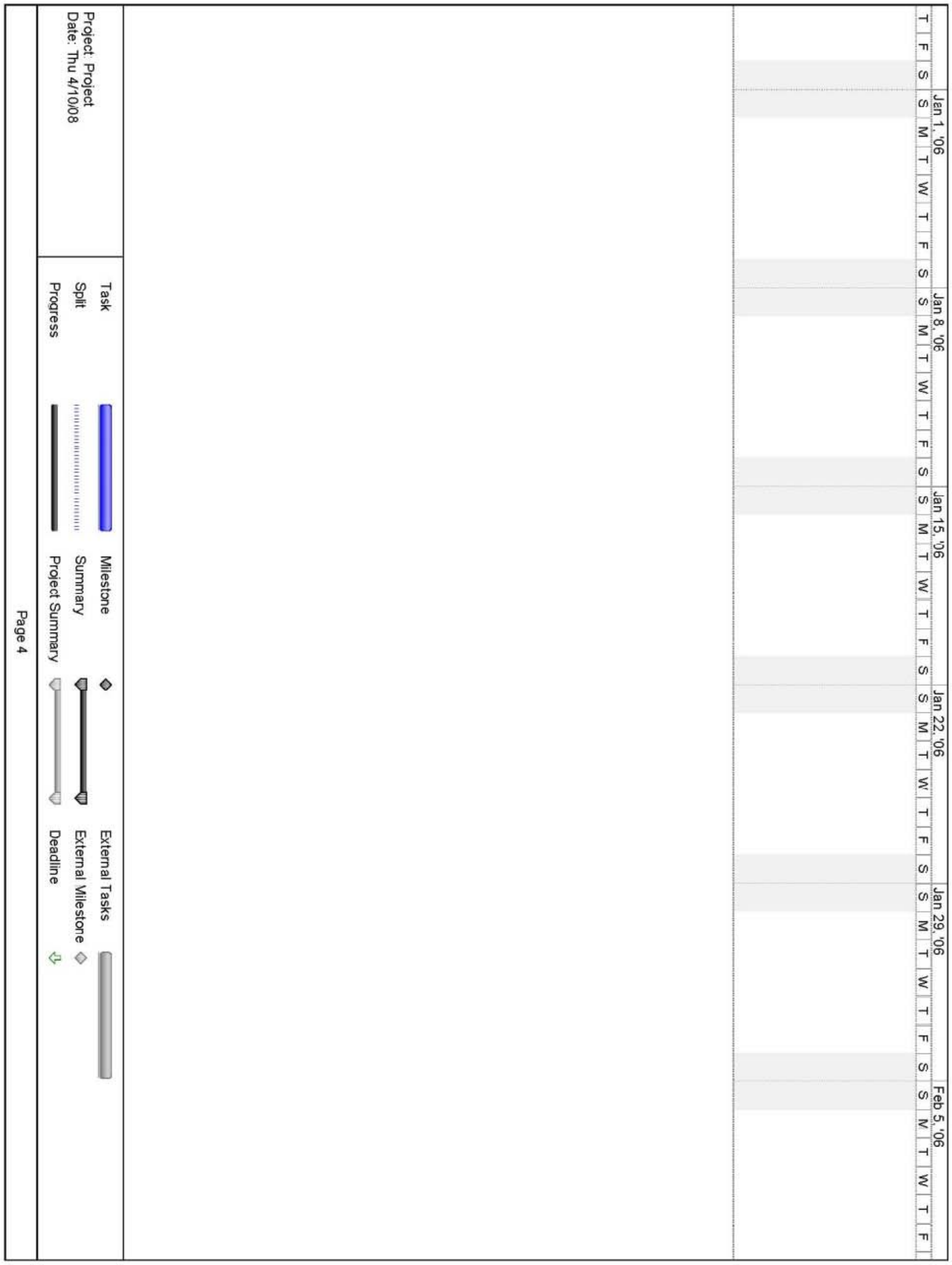
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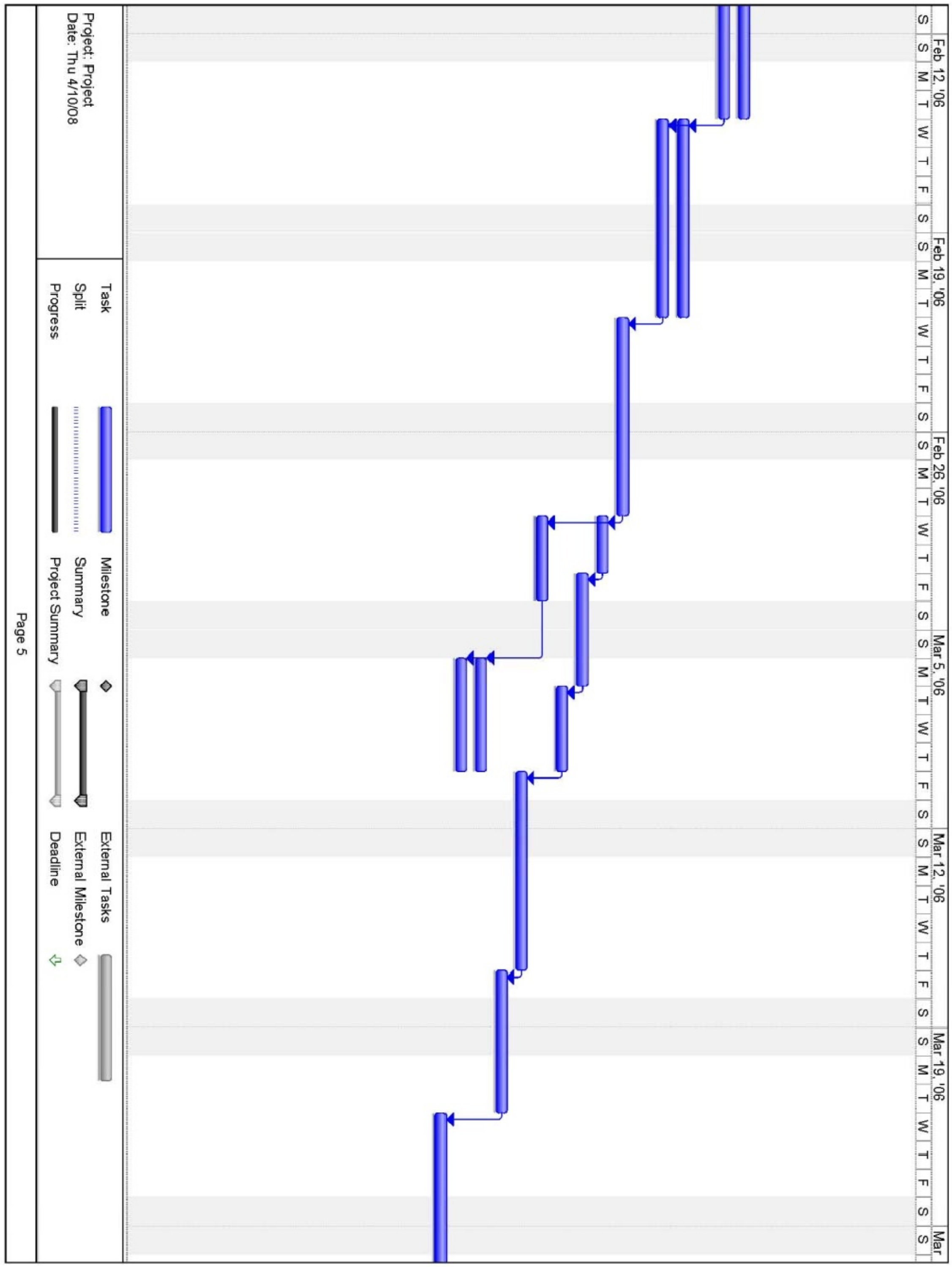
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Mar 12, '06
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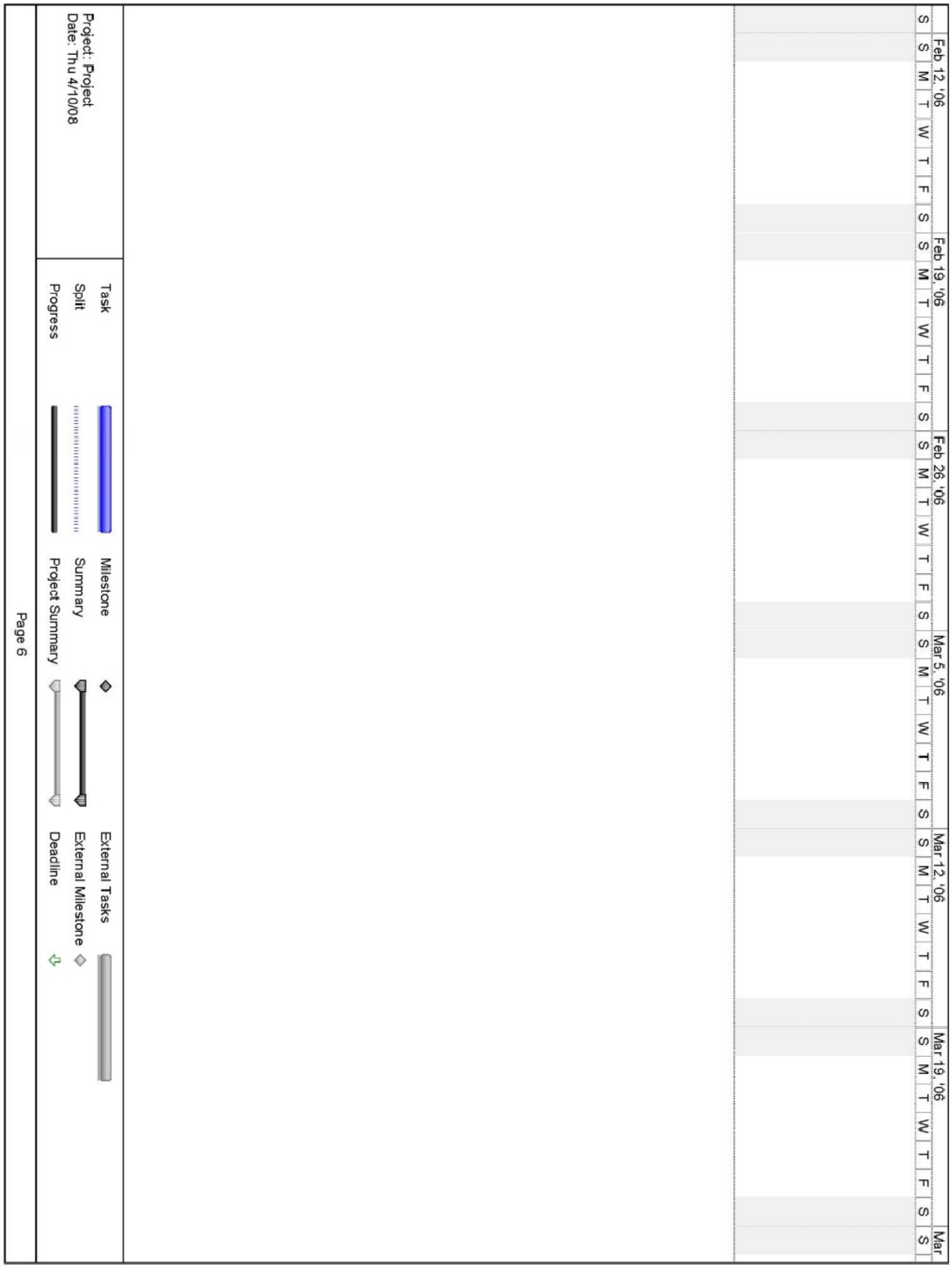
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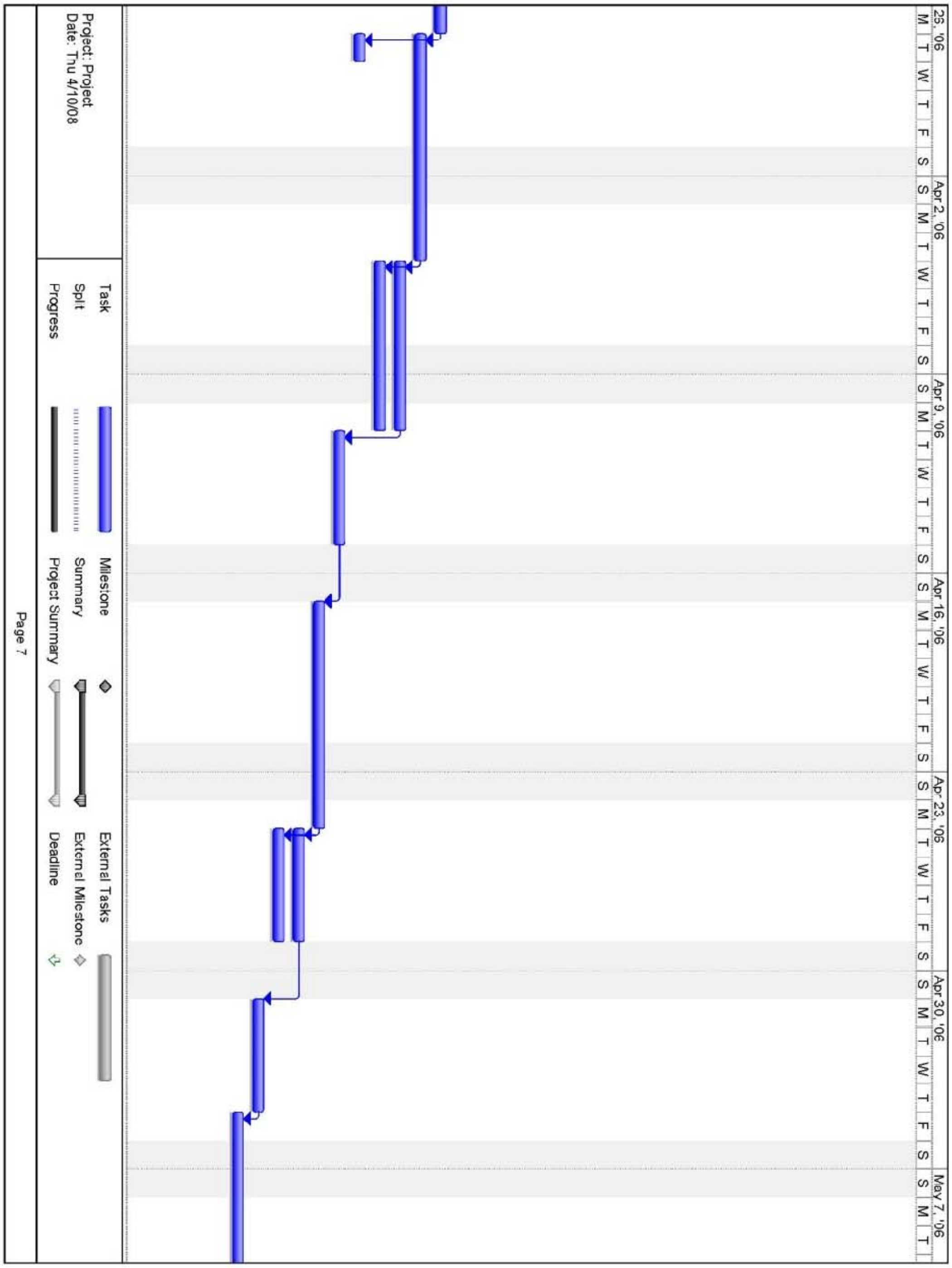
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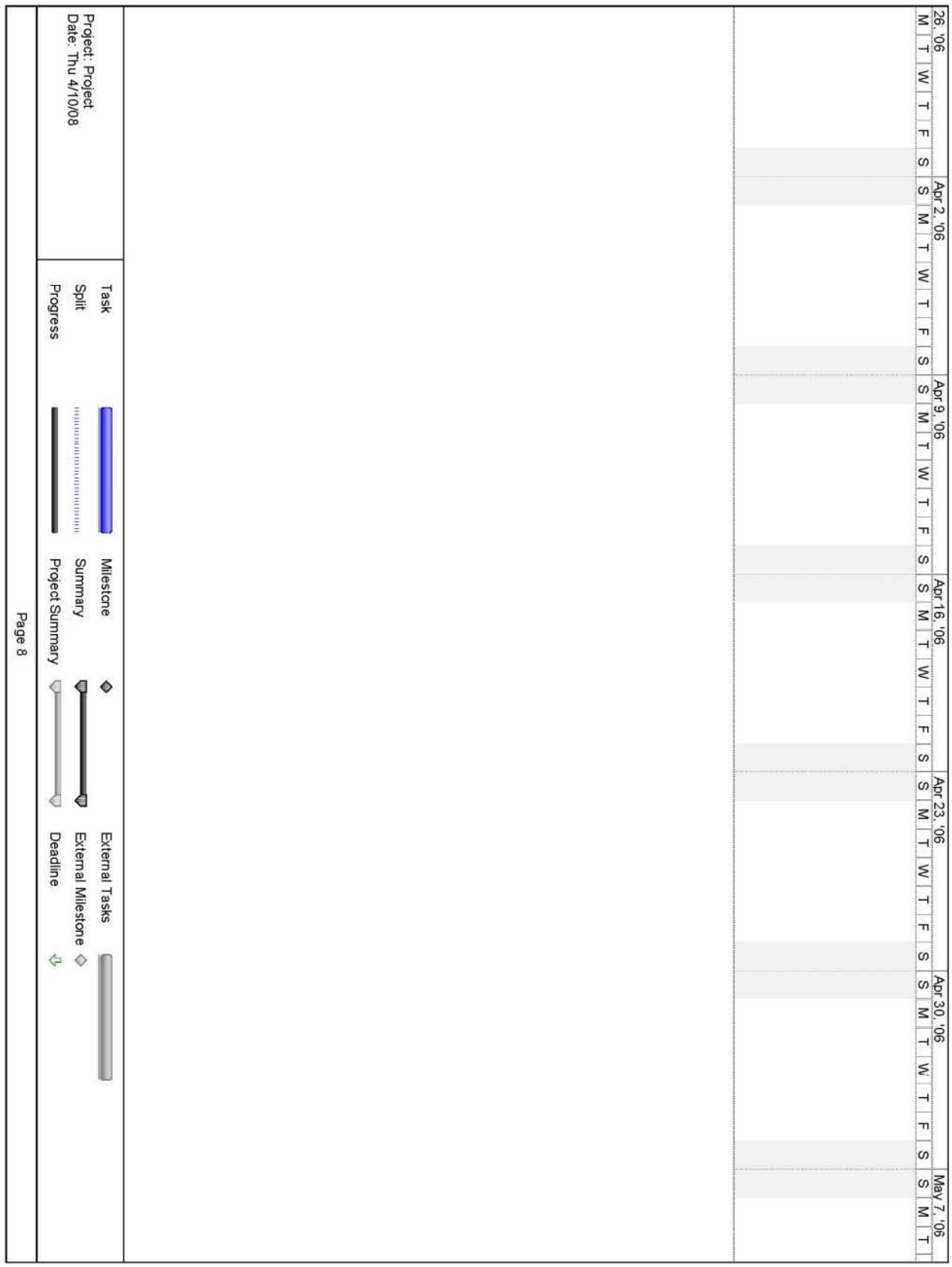
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Apr 30, '06
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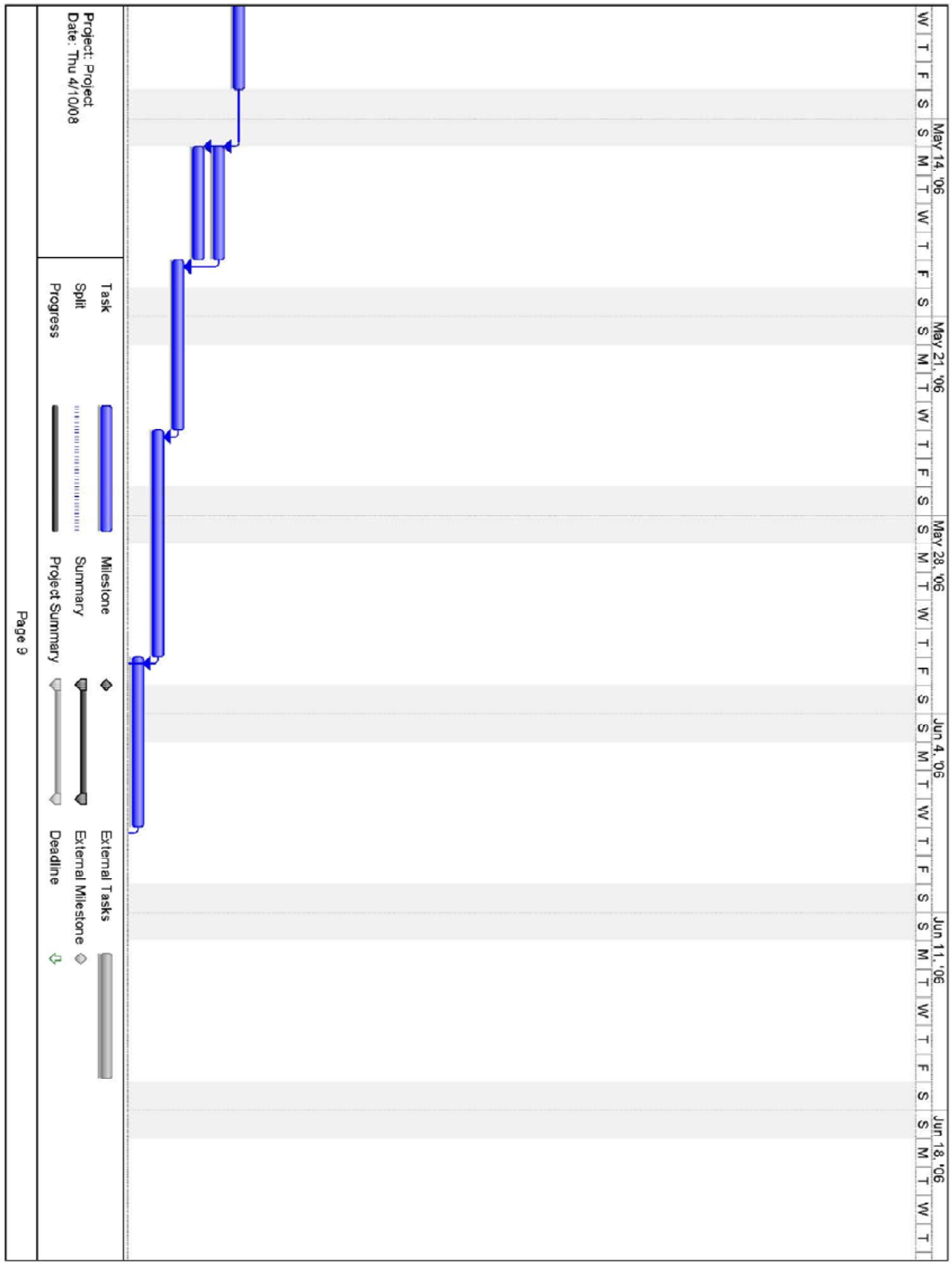
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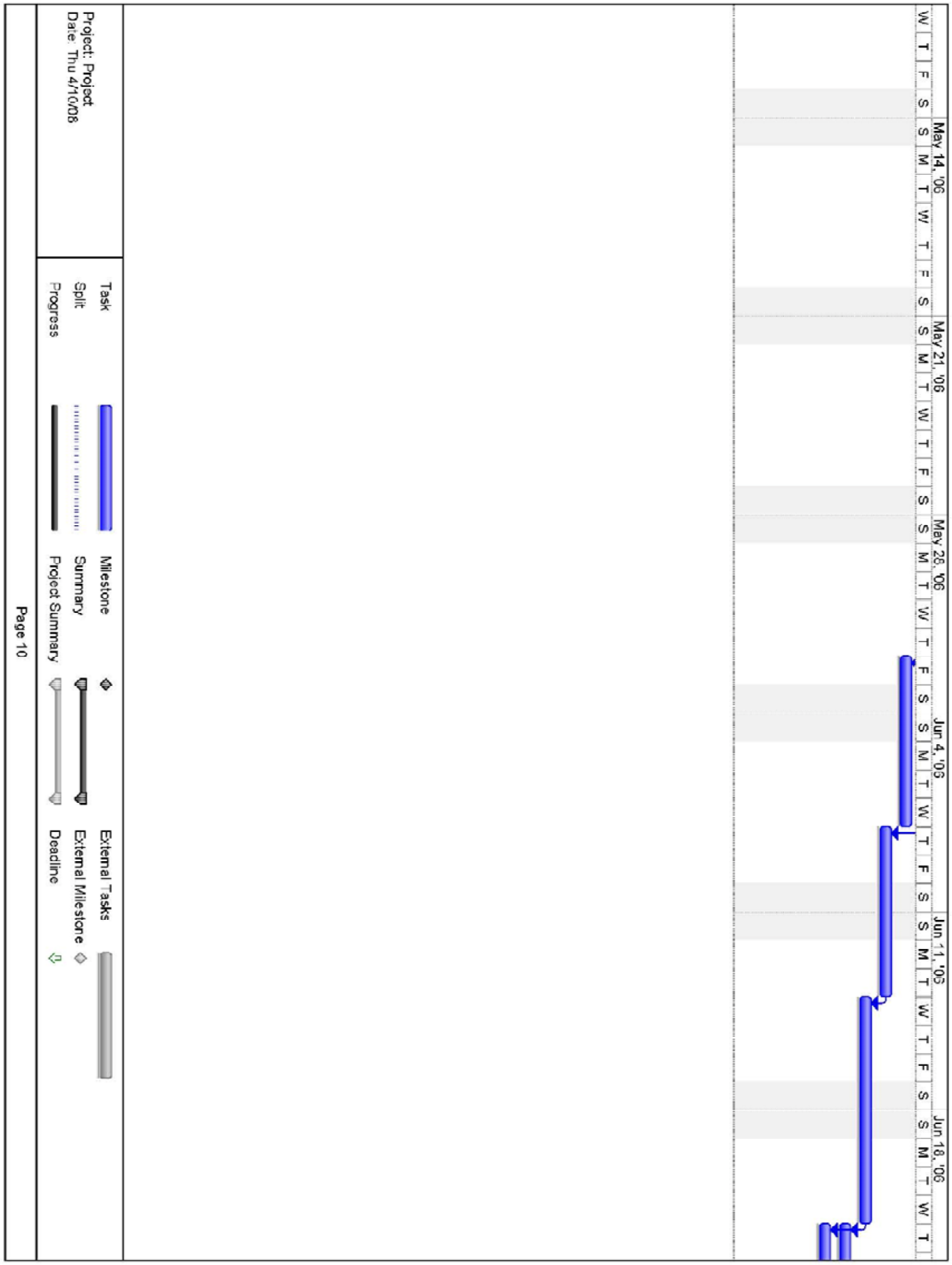
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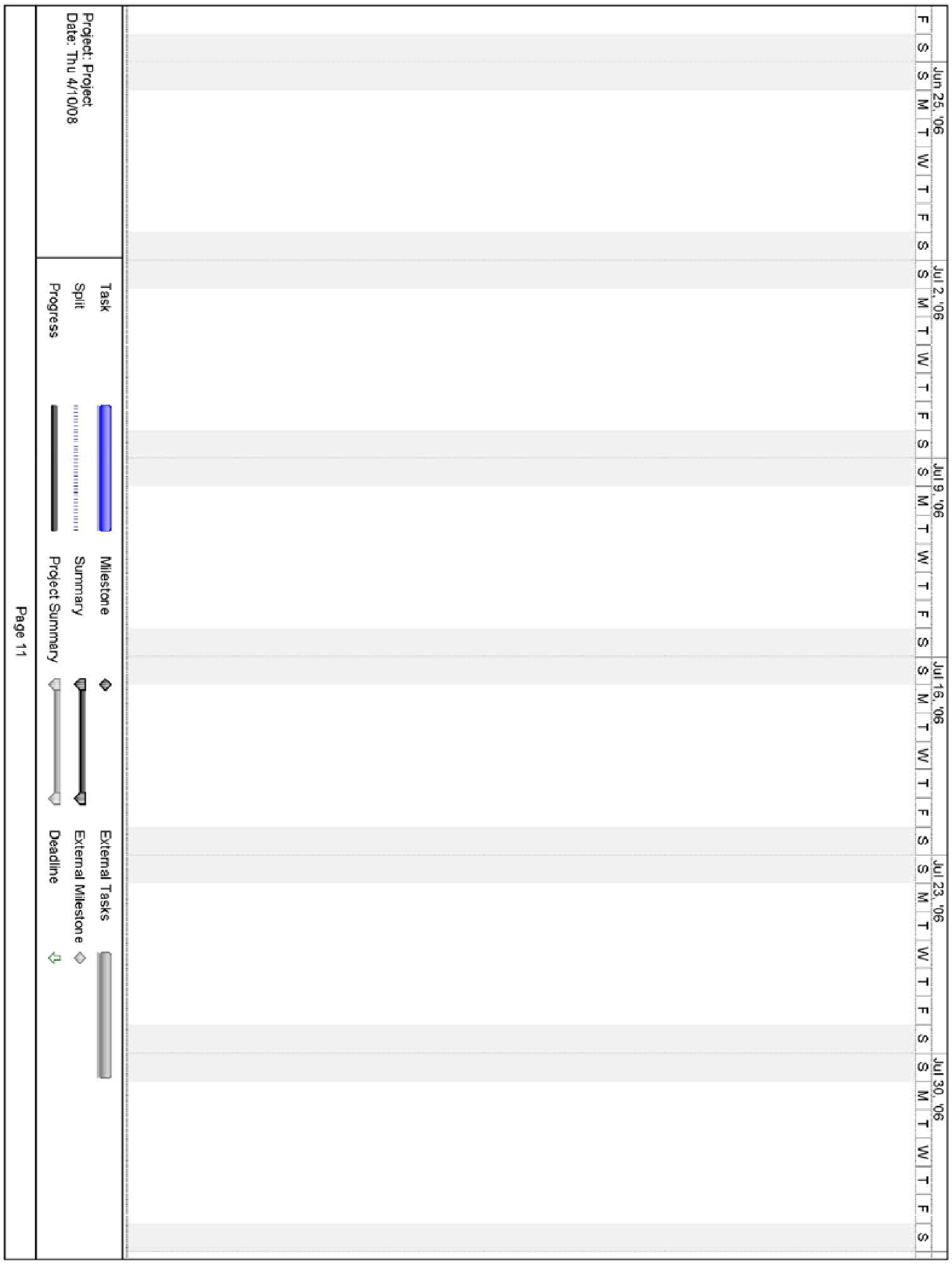
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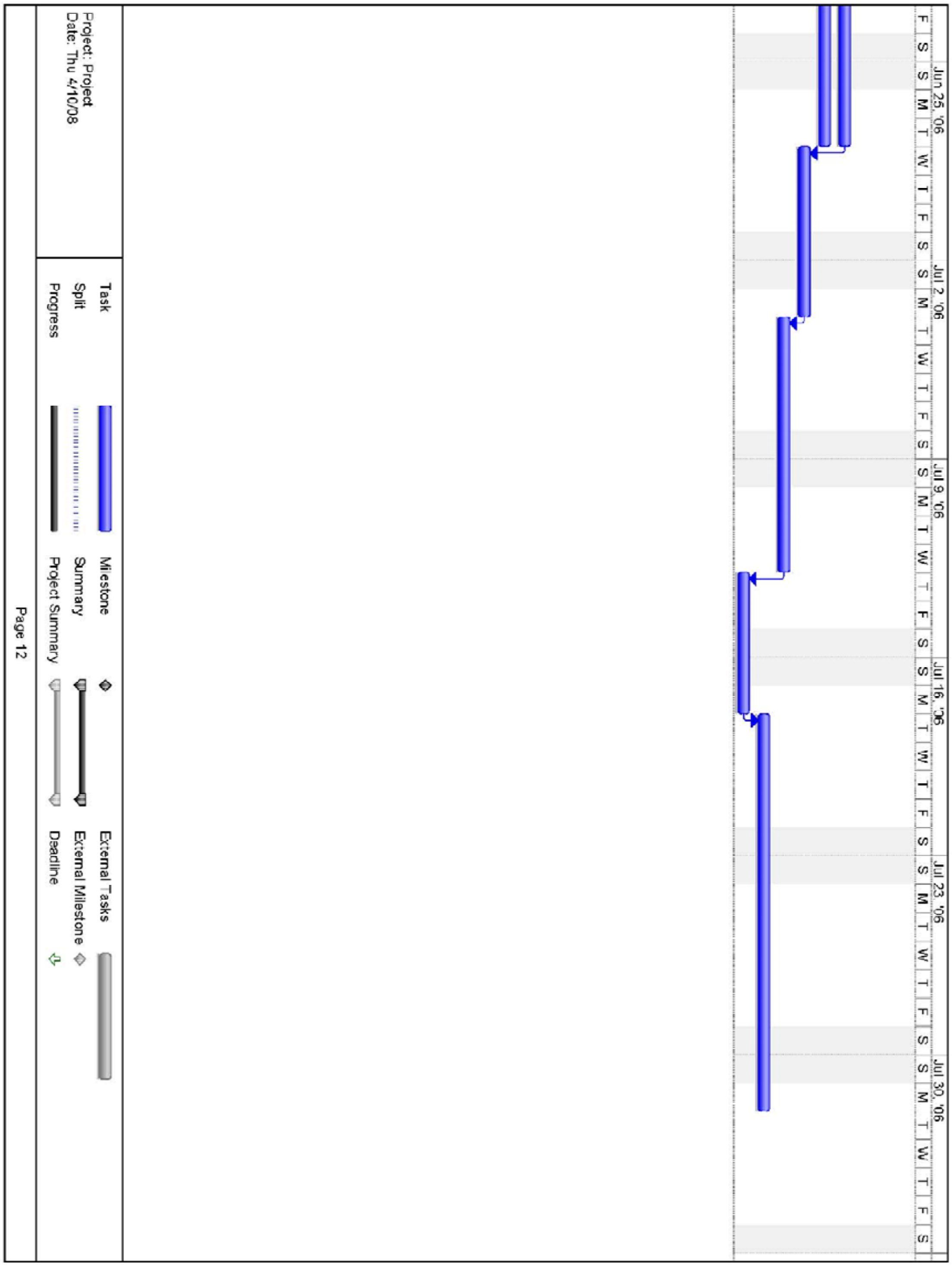
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