Final Thesis Report

The University Medical Center of Princeton

4/4/2012
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Structural Option
The Pennsylvania State University

University Medical Center at Princeton

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http://www.engr.psu.edu/ae/thesis/portfolios/2012/AJB449/index.html

General Information

- Construction Cost: $250 Million
- Building Occupant Name: Business Group B, Institutional Group I-2
- Size (S.F.): 800,000
- Stories Above Grade: 6 Stories
- Delivery Method: Design-Bid-Build

Project Team

- Owner: Princeton University
- Construction Manager: Turner Construction
- Architects: HOK & Hiller Architecture
- Structural Engineer: O'Donnell & Naccartato
- MEP Engineer: Birdsall Services Group

Structural

- Foundation: Spread footings with load bearing masonry walls
- Superstructure: Steel Framing with composite metal decking
- Lateral Structure: Moment Framing in the East/West Direction & Braced in the North/South Direction Perimeters

M.E.P.

- There are 17 Air Handling Units in UMCP
- Steam humidifiers in patient spaces
- CAV Units in patient's rooms
- VAV Units In every room
- Steam heat supplied by Princeton's Energy Plant

Electrical

- 13.2 kV electrical service to the building
- Two bus systems, one at 1600 Amp, 3 phase, 4 wires. The other bus is at 1200 Amp, 3 phase, 4 wires
- UMCP runs on a 277/480Volt system

Architecture

This six story tall building has a long and curving body that encases the parking lot to draw people into the building. The body is a curtain wall that will provide a view to the outside for all the patients, and it is framed with aluminum reliefs and metal panels. The West and East elevations have a CMU ground face with a brick façade on the top floors.

Executive Summary

The University Medical Center of Princeton (UMCP) is a seven story, 92' tall building that services the medical needs for Princeton students and the members of the surrounding community in Plainsboro, NJ. The superstructure is composed of a steel framing system with composite deck, and the lateral system is designed with a combination of braced frames and moment frames.

This thesis was based on the investigation of a changing UMCP to a reinforced concrete superstructure. The same column layout was used for the redesign. The lateral system changed the steel moment frames to concrete moment frames, and braced frames to concrete shear walls. The lateral system was designed by the loads and deflection from the third wind case determined from ASCE 7-10. All of the structural members were designed by iterating through a compiled spreadsheet of slab, beams, girders, and columns. The redesigned and the existing structure are adequate for serviceability issues, but it was determined that concrete structures are more proficient in vibration concerns.

Since time and money are very important in this market and in general, a cost and schedule analysis was established for both the existing structure and the suggested structure. It was determined that the raw material for the reinforced concrete and placement was $94,322.28 cheaper than the steel design. After overhead and profit the concrete structure was $786,922.71 more than the steel structure. Also, while comparing the two schedules of tasks showed that the concrete structure would take approximately 100 days longer than the steel system.

Making the building LEED certified was another option taken into account by trying to improve the UMCP building. Adding a green roof was gave an extra 3000 square feet that the occupants can enjoy which would be accessed from the second floor. This green roof would increase the budget by approximately $555,000 in initial cost, but there is much payback that comes with a green roof. Also, the roof of the seventh story would implement a cooling roof, which decreases the heat island effect and cuts down on cooling costs in the summer. Other green practices were incorporated into the building, plus the existing HVAC system and curtain wall helped come close to possibly getting a LEED certification for UMCP.

The proposed design would be feasible if you are willing to increases the construction cost plus increasing the length of the schedule. Also, if you implement the sustainability design you can gain an extra 3000 square feet of outdoor space, and save money on the lifecycle cost of the building.

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Acknowledgements

First, I would like to thank the Penn State Faculty for pushing me past my yield point, almost to my breaking point, but changing my knowledge and outlook on life and engineering forever. I couldn't be happier with the professors I had throughout the past years, and I am honored to say that I am going to be a Penn State Alumni.

Second, by no means could I have made it this far without losing my mind if it wasn't for my AE friends. My colleagues are some of the smartest people I have ever met, and I am happy to have them in my network for the professional world. I cherish more than anything how much fun we had just goofing around during are study time, while doing homework, or just doing nothing at all.

Lastly, but definitely not least, I want to thank my family most of all for all of their support they have given me. If it wasn't for all of you I would not be where I am today. I have learned so much from all of the good and bad example my brothers and sister have made throughout the past many years, and I am trying very hard to follow in your extremely successful footsteps. Also, my Mom and Dad, you two are truly the best parents any kid could ever have.

Special Thanks To…

- Professor Parfitt
- Professor Holland
- Dr. Linda Hanagan
- Jeff Cerquetti, M. CE, P.E., Vice President Facilities Structures & Coastal Engineering, JMT
- Andy Verrengia, P.E., LEED AP, Project Engineer, Atlantic Engineering Services
- Turner Construction
- The University Medical Center of Princeton University

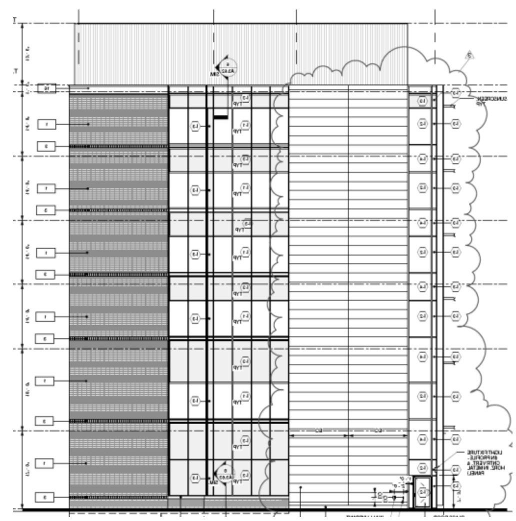
Building Introduction

Princeton University Medical Center was in a big need of change. The rapid growth of people plus the outdated building design and equipment were the main reasons to upgrade their old medical center.

The University Medical Center at Princeton (UMCP) will also be joining the Pebble Project. Pebble Project is a research effort between The Center for Health Design and selected healthcare providers to measure the layout and design of a hospital and how it can increase quality care and economic performance. The design of this building is not just for looks, but to help operate a hospital in a healthy and efficient manner.

Figure 1: UMCP Site Location Shown in Blue Satellite Photo Courtesy of Google Maps

This six story tall building has a long and curving body that encases the parking lot to draw people into the building. Lighting is not going to be an issue during the day as the glass curtain wall is used on the south face of the building. Furthermore, it will provide a view to the outside for all the patients and workers in the building. The curtain wall is framed with aluminum reliefs and metal panels. The West and East elevations have a CMU ground face with a brick façade on the top floors, and there are very few windows since these walls are framed with steel bracing. The mechanical equipment is encased in 13.5’ parapets. Floors two through six almost mimic each other in framing and room layout. The entrance of the building has a wide atrium open to the second floor with interior wood shading panels. The overall design of the building is simple, sleek, and efficient.



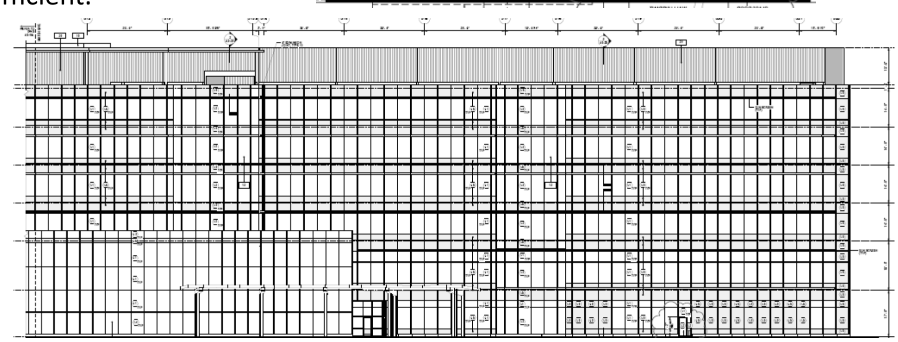


Figure 2: East & South Building Elevations Drawings Courtesy of Turner Construction

Structural Overview

The foundation plan for the University Medical Center is built on 4” to 5” Slab-On-Grade basement floor with interior concrete piers stabilizing wide flange columns, and an exterior 2’ thick foundation wall partially incasing mini tension piles. The design of the superstructure is primarily steel framing. The framed floors consist of a 3 span 3 ¼” lightweight concrete composite decking system with composite steel framing. Roof decking is type B 1 ½” galvanized metal deck, and 6 ½” normal weight concrete composite metal deck for the roof Penthouse area. There is also a massive curtain wall spanning the South end of the curving building, but this will not be analyzed in this technical report.

FOUNDATIONS

According to drawing S3.01 all the subgrade footings were poured under the supervision of a registered Soils Engineer. The capacity of the soils, shown in the boring test specifications, came out to be 4,000psf and 8,000psf for the compacted/native soils (medium-dense/stiff) and decomposed bedrock respectively. The spread footings support wide flange columns, varying from W10x54 to W14x311, to anchor the superstructure (Refer to Figure 3 for more detail). The spacing for the foundation columns is not consistent throughout the basement, which that is the reason for the varying column sizes. Figure 3 shows a typical spread footing supporting a steel column. Outlying the basement is a 2’ thick foundation wall with mini tension piles that relives up to 150kips of tension from the concrete bearing wall.

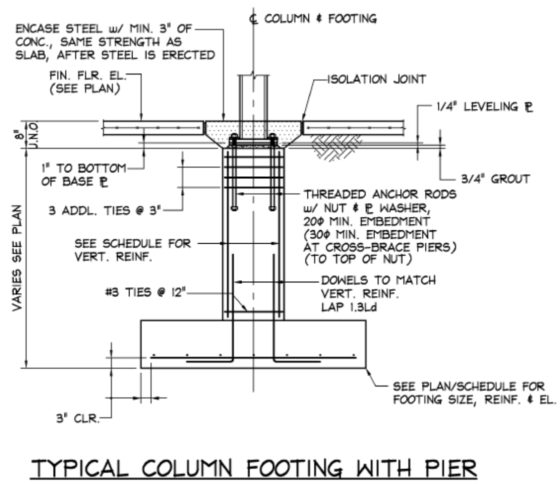


Figure 3: Typical Column Footing with Pier Drawing, Courtesy of Turner Construction

Concrete Strengths:

- 3,000psi- Spread Footings, Wall Footings, Foundation Wall, & Retaining Walls
- Minimum of 3,000psi- Piers-match wall strength
- 3,500psi- Slab-On-Grade and Slab-On-Deck

Rebar Design:

- ASTM A615- Deformed Bars Grade 60
- ASTM A185- Welded Wire Fabric

FLOOR & FRAMING SYSTEMS

A typical beam spanning in the North/South direction, consists of a 26’ span then a 15’ span, and finally back to a 26’ span. The East/West girders span 29 ½’ typically and Appendix 1 helps better understand the layout of the building. Floors two through six do not change in design other than the column thickness, all of the floors use a 3 span 3 ¼” lightweight concrete composite decking. This creates a one-way composite flooring system connected to composite beams. Even though the first floor has an additional atrium, the decking is still consistent to the floors above. Figure 4 shows the wide flange beams used in each span.

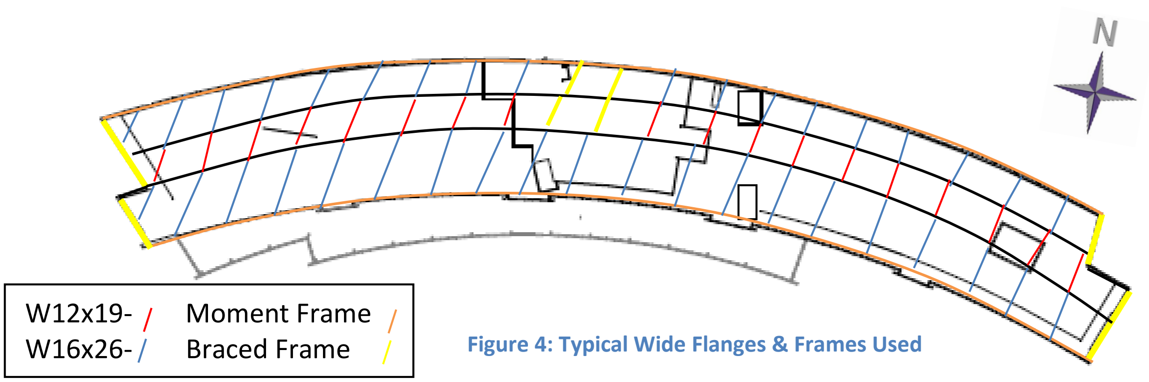


Figure 4: Typical Wide Flanges & Frames Used

The infill beams are usually at a spacing of 9.8’ and they range from W16x26 for the 26’ spans or W12x19 for the 15’ spans. The girders typically span 29.5’ and vary from W24x55 on the exterior girders to W21x44 on the interior girders. These composite beams use ¾” bolts to help anchor the decking. The typical bays then come out to be either 29.5’x26’ or 29.5’x15’. There are also two transfer beams on the on column lines N2 and S3 to account for columns that do not line up on the first to second floor.

Steel Design:

- ASTM A992- Wide Flanges
- ASTM A500- Rectangular/Square Hollow Structural Sections Grade B, Fy=46ksi
- ASTM A500 or ASTM A53- Steel Pipe Type E or S Grade B
- ASTM F1554- Anchor Rods Grade 55

Lateral Systems

The UMCP lateral systems design was comprised of typical steel moment frames in the East/West direction and steel concentrically braced frames in the North and South direction. Those framing systems only occurred on the perimeter of the building. Around the elevator shaft is another place where the design is concentrically braced. The lateral forces will travel into the composite deck, and then through the wide flange beams or HSS braces into the columns to the piers to then dissipate into the ground.

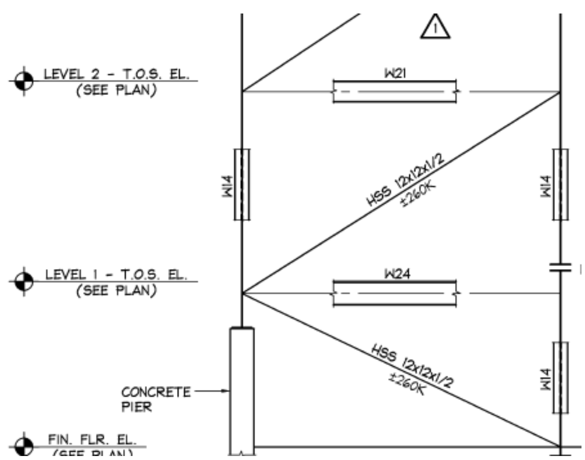


Figure 5: Typical Braced Frame, Courtesy of Turner Construction

Codes/Means Used

This building fit into an Occupancy Category III. Any Hospital/Medical Center needs to be designed with an Occupancy Category III as a safety factor.

Original design codes used on this building were:

- 2006 International Building Code (IBC) with New Jersey Uniform Construction Code
- 2006 International Mechanical Code (IMC)
- 2005 National Electric Code (NEC) with local amendments
- 2006 International Energy Conservation Code with other local amendments
- 2006 International Fuel Gas Code with local amendments
- New Jersey Department of Health and Senior Services - "Licensing Standards for Hospitals, N.J.A.C 8.43G" and the 2006 Edition - "Guidelines for Design and Construction of Hospital and Health Care Facilities."

Design codes/means used for thesis designs and calculations:

- ASCE 7-10 Minimum Design Loads for Buildings and other Structures
- American Institute of Steel Construction, 14th Edition AISC Steel Construction Manual
- 2008 Vulcraft Steel Roof & Floor Deck Manual
- Building Code Requirements for Structural Concrete, ACI 318-08
- Facility Guidelines Institute
- Concrete Reinforcing Steel Institute
- Green Building and LEED Core Concepts Guide, First Edition
- LEED Green Associate
- RSMeans 2012

Proposal Objectives

Depth Topic

The gravity system for the redesigned building will consist of a solid one way slap with beams supported by concrete square columns. The lateral system design will consist of changing the braced frame walls to shear walls, and all the steel moment frames to concrete moment frames. To analyze the lateral System in more detail a 3-D model will be represented in ETABs. Changing the structure to concrete will create a much heavier mass, which in turn will create more of an effect due to seismic force. There are many advantages of having a concrete structure as opposed to a steel structures.

Changing the design to a solid one way slab should limit the deflection and vibration in UMCP due to the extra mass of the concrete. This will create a more comfortable atmosphere for the patients due to less vibration and better noise control (in both sound transmission and impact noise); performance in surgery rooms could also improve due to the same enhancements. A more in-depth research on vibration control in hospital surgery rooms will need to be conducted to make sure the needs of the hospital are met.

Also, the concrete does not need to be fireproofed, and by keep the same column layout the floor to ceiling height could decrease. Therefore, lifecycle costs of the hospital should decrease. A cost and schedule comparison will be completed to determine which framing system is more cost and time effective. The formwork and schedule of the project would impact the cost as well. Reusing formwork should maintain a low project cost.

Breadth Topic 1- Construction impact and Cost analysis

There will be a great impact on the project cost and scheduling for the redesign of the building. Erecting steel and placing concrete will require different construction scheduling due to the placing of the formwork and waiting for the concrete to cure. Therefore, an accurate schedule of the critical path of the redesign will be created for the new construction process. The cost of the redesign will include items such as base material cost, labor teams, additional or eliminated work days, and formwork. For that reason, an analysis of the new cost and schedule will be necessary to compare with the existing design. RS Means 2010 will be used to conclude the final project cost.

Breadth Topic 2- Sustainability

A green roof will be added on top of the atrium roof which will be accessible for the patients on the second floor. This will be an enjoyable additional architectural space, as well as a step into the future of sustainability. A check of the column sizes must be done to make sure the added weight of the roof will be supported. Water retention will be another issue that will have to be taken into design consideration. Further research on xeriscaping must be done to see what type of plants should be used on the roof. This project is not LEED certified, but with some green additions i.e. solar panels, gray water reuse, water efficient toilets/sinks, and day lighting the project could be certified. The cost of the project will increase, but if it is done right a green building, overtime, saves money and helps the environment.

Structural Depth

The main scope of the structural depth is focused on the redesign of the University Medical Center at Princeton from a structural steel superstructure to a concrete superstructure. The same column layout will be used in the design to keep the architectural flow of the existing floor layout. The gravity system will be designed as a one-way slab with beams. As for the main lateral force resisting system there will be concrete moment frames in the long direction or East/West direction, and shear walls in the short direction or North/South direction. The concrete moment frames will replace the steel moment frames, and the shear walls will replace the braced frames. The design should be adequate for strength and serviceability requirements such as drift, deflection, and vibration concerns for the health care facility's needs.

Gravity System

Live Loads

The live loads were taken from ASCE 7-10 to determine what loads were going to be applied to the structure for hospital's occupancy type. Live load reduction was used for the beam, girder, and column design because there influence areas were greater than 400 square feet. Though there were multiple occupancy rooms in the building the influence areas for the majority of the members would impede on a corridor, so in all of the hand calculations a live load of 80 psf was applied to be conservative. The table below shows the live loads for a hospital.

Hospital Live Loads from ASCE 7-10 Ch. 4	
Occupancy/Use	**Uniform Load**
Patient Rooms:	40 psf
Operating Rooms:	60 psf
Corridors Above 1st Floor:	80 psf
Corridors and Lobbies on 1st Floor:	100 psf
Roofs Used for Gardens:	100 psf

Table 1: Live Loads

Dead Loads

A superimposed dead load of 35 psf was applied for the design of the gravity and lateral system. These elements are assumed to be fastened directly to the slab or other structural elements, and the load is spread over the full area of the floor. The elements include various MEP systems, ACT tiles, certain hospital equipment, other finishes, and collateral to be conservative. For the dead loads in the design refer to the table below.

Material	Dead Loads
Normal Weight of Concrete:	150 pcf
Structural Steel	490 pcf
Superimposed Dead Load:	35 psf

Table 2: Dead Loads

Slab Design

The reinforced concrete slab was designed with accordance of the Concrete Reinforcing Steel Institute, chapter seven. The one-way design table was used with a ρ=0.005 and a span of 14.5' with a factored superimposed load of 170psf, which was taking form the controlling load combo of 1.2D+1.6L. This lead to a slab thickness of 6.5" concrete slab that could handle up to 203psf load with bottom reinforcement of #7 rebar spaced at 11" and top reinforcement of #4 rebar spaced at 12" on center. Appendix 2 has the table used for the slab design, and it takes into account deflection.

Slab Design	
Bottom Reinforcement:	#7 spaced at 11 inches
Top Reinforcement:	#4 spaced at 12 inches
Temperature & Shrinkage:	#3 spaced at 9 inches
Slab Weight:	81 psf

Table 3: Slab Design

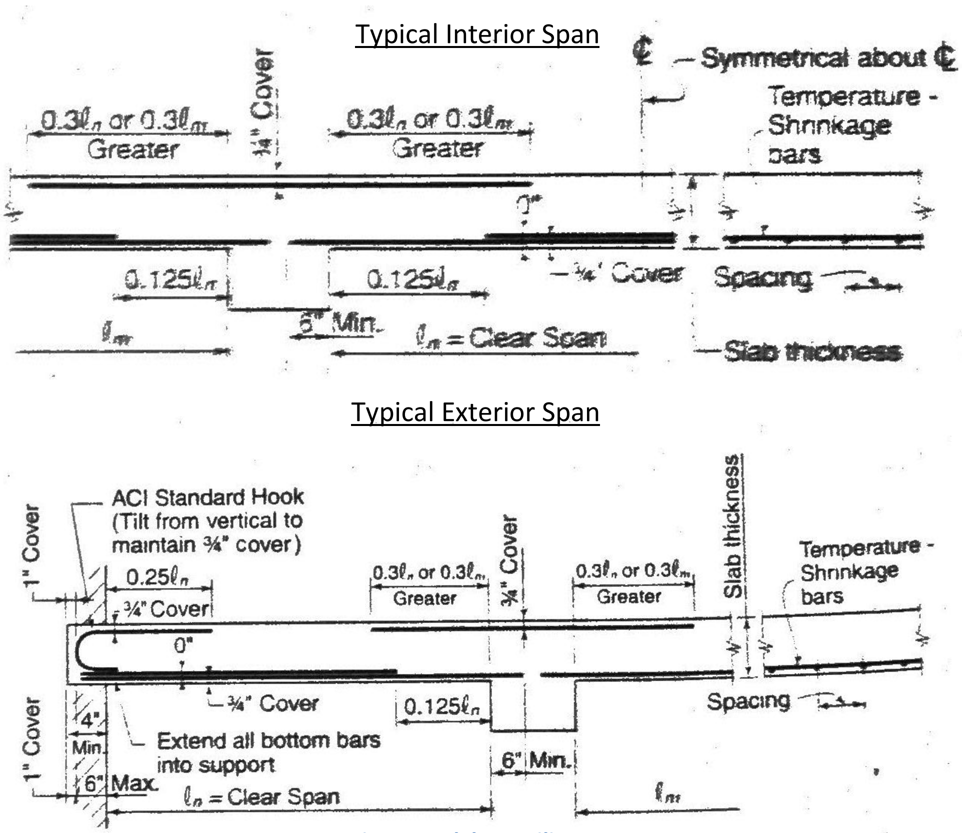


Figure 6: Slab Detailing

Beam Design

The beams were designed perpendicular to the one-way, 6.5" slab. The beams are placed in line with the columns and also split the column line which is displayed in the figure below. This allows the beams to hold a tributary spacing of 14.75' because the columns are spaced at 29.5' on center. The dead and live loads that were stated previously were used to find the size and amount of steel reinforcement during a flexural analysis of the beams. To make sure the depth of the beam was adequate for deflection, h>l/18.5, taken from table 9.5 in ACI 318-08. This deflection equation is for a one end continuous beam to be conservative because this gives the biggest depth. After iterating through hand calculations the adequate beam size was determined to be a 10x20 with five #8 rebar and #3 stirrups for the edge beams (B2) and a 10x20 with four #8 rebar and #3 stirrups for the interior beams (B1). The beams have two rows of reinforcement. The table below shows the design details, and Appendix 3 shows the full hand calculations.

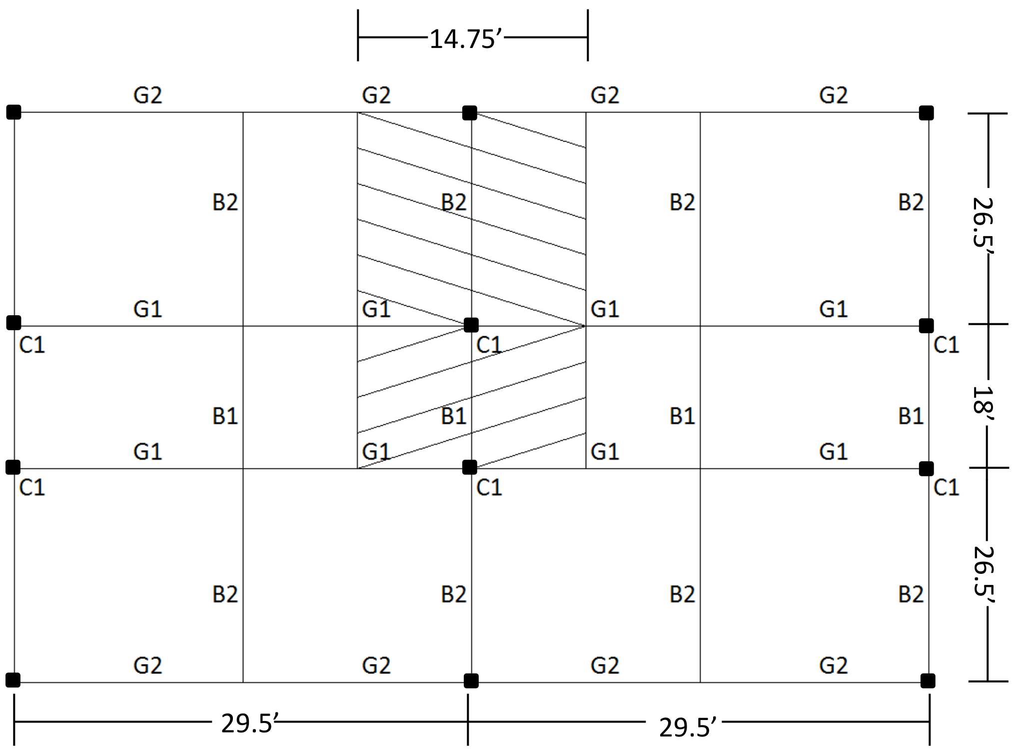


Figure 7: Beam Layout with Tributary Area

Beam Design, B1 & B2	
Section Size for B1 & B2:	10x20
Steel Reinforcement, B1:	(4) # 8 rebar & # 3 Stirrups
Steel Reinforcement, B2:	(5) # 8 rebar & # 3 Stirrups
Weight:	141 plf
f'c:	4 ksi
fy:	60 ksi

Table 4: Gravity Beam Design

Girder Design

To design the girder, the same approach was taken that was used for the beam design. The girders are parallel with span of the slab. These girders are set in line with the interior columns, and span 29.5' with a spacing of 26.5' to the exterior girder and 18' to the interior girder. The figure below shows the tributary area of the gravity girder. The same dead and live loads from the beam design are applied to the girder, but over a bigger tributary area. There is also a point load from the beam at the center of the span that acts as a dead load that adds to the moment. After running the calculations the most efficient typical girder design is the same section as the beams at 10x20, but with a different reinforcement with seven #8 rebar and #3 stirrups. There are a couple spans that are 32' long, these were designed with a different section at 12x20 and reinforced with nine #8 and #3 stirrups. The girders are designed with two rows of reinforcement, and the table below shows the design details of the girders. Appendix 4 shows the hand calculations that determined the size and reinforcement of the girder.

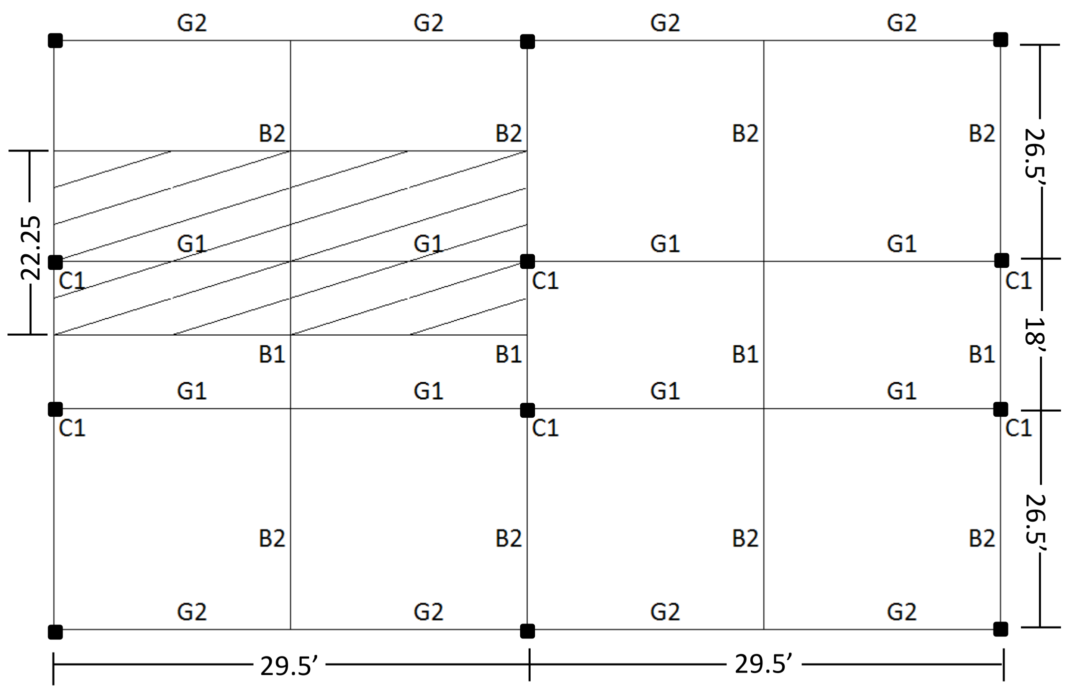


Figure 8: Girder Layout with Tributary Area

Girder Design, G1	
Typical Section Size:	10x20
Section Size for 32' Span:	12x20
Typical Steel Reinforcement:	(7) # 8 rebar & # 3 Stirrups
Steel Reinforcement for 32' Span:	(9) # 8 rebar & # 3 Stirrups
Typical Weight:	141 plf
Weight for 32' Span:	169 plf
f'c:	4 ksi
fy:	60 ksi

Table 5: Gravity Girder Design

COLUMN DESIGN

All of the interior columns are the same size, even with the 32' spans, for continuity and simplicity of the design. The design is based off of the bottom column because it has to carry the load of the seven stories of columns above it. The figure below shows the tributary area of a typical column. The column design had to fit an interaction diagram containing pure axial, pure bending, and balance point loads. If the actual axial and actual moment load is outside of this curve the column will fail. After finishing the hand calculations the column size and reinforcement was checked with spColumn. The final result of the column came to be a 20x20 with twelve #10 rebar and with a 2.5" clear cover, and the columns that had varying spans had a reinforcement of sixteen #10 rebar. Appendix 5 has the hand calculation for this design and spColumn check.

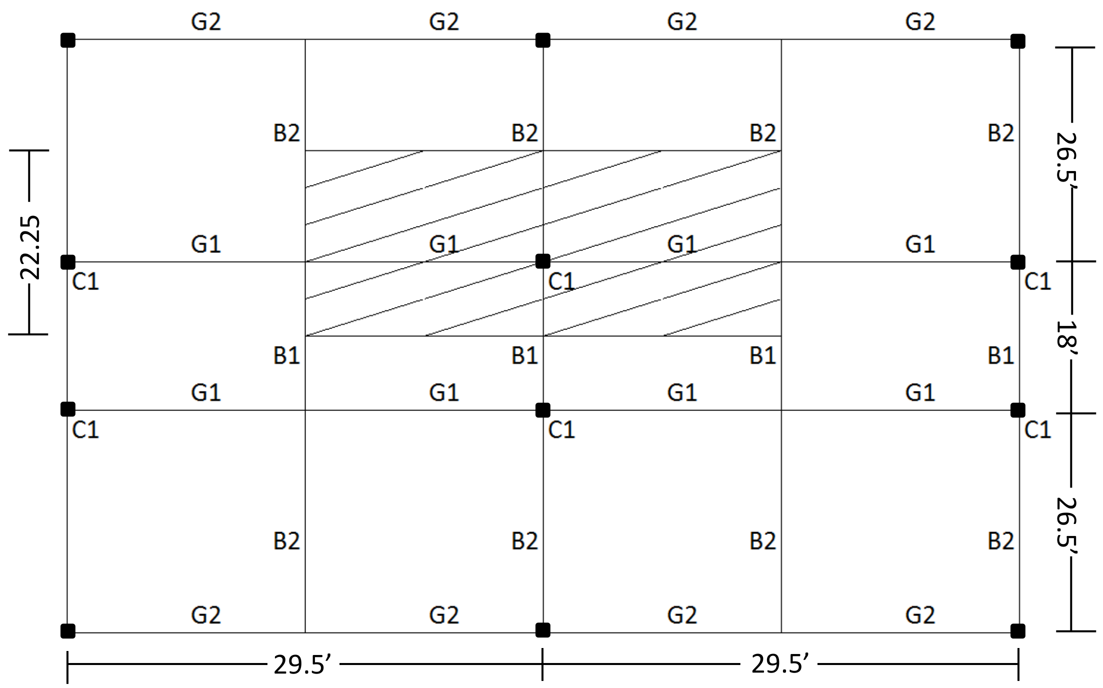


Figure 9: Column Layout with Tributary Area

Column Design, C1	
Section Size:	20x20
Typical Steel Reinforcement:	(12) # 10 rebar
32' Spacing Reinforcement:	(16) # 10 rebar
f'c:	4 ksi
fy:	60 ksi

Table 6: Gravity Column Design

Vibration Concern

The Facility Guidelines Institute states that an operating room should stay under a footfall vibration peak of 4000 micro-inches/second which is approximately 50 steps per minute. It is engineering judgment that most operating rooms will be below 50 steps per minute, but it is the adjacent rooms/corridors that are the problem because the rush of patients in and out of the room. When designing a steel system to be less than 4000 micro-inches/second a vibration concern would very critical because there steel is prone to vibrating at a 4000 to 2000 micro-inches/second. Most concrete gravity systems do not need to be checked for vibration concerns until 1000 micro-inches/second. Even then it tends to be a little murky to determine the vibration in a concrete slab, and there are not many references to check this criterion. There are ways to check it for a steel system, but there will be no results for the proposed structure to compare to. It is known throughout the engineering industry that concrete slabs are damper than steel, and work much better in any vibration concern. The original design is probably fine to comply with the 4000 micro-inches/second, but the proposed design will work better in vibration. Appendix 6 shows the table referenced for the operating room guidelines.

Gravity Design Advantages & Disadvantages

The design was kept very simple to help keep constructing the structure fast and easy. Also, the forms can be reused from floor to floor because the majority of the members are the same size for each floor. The original girder depth is an 18” deep wide flange with infill beams spaced at 9’. The new total depth is 20” with infill beams spaced at 14.25’ on center. The plenum space has grown 2 inches which will help with mechanical system design, but this will decrease the floor to ceiling height. Also, the floor weight increased, not by much, but it still has a bigger impact in seismic design which will be discussed later in the report. This system cuts the cost of fireproofing because concrete is fireproof by itself. Also, the vibration will be less in the concrete design for the reason that concrete is more massive, making the floor system damper.

Lateral System

ETABS Model

An ETABs model was constructed to make sure the strength and serviceability criteria is adequate for the proposed lateral design. Since there was a separation joint in the original structure only the bigger structure was modeled. So if the bigger structure is adequate then the smaller structure would be acceptable as well. The self-weight multiplier was changed to zero, so it would act as diaphragm system. Also the cracking moment of inertia was applied to both girders and columns in the moment frame. The columns had a 0.7Ig multiplier and the girders had a 0.35Ig multiplier which was taken from Chapter 10.10.4.1 in ACI 318-08 to account for cracking. The mass of the diaphragm was taken by the weight of the gravity system and other superimposed dead load because that is all that affects the lateral system. The mass was found for a typical bay is equal to 6.5E-5Kips/ft^2. An end offset of 0.5 was applied to ensure that cracking would ensue in the concrete as well. The model was modeled as pin connection at the bottom of the columns because it is expensive hard to create a fixed end constraint. The next two sections will get into the wind and seismic loads that were calculated and applied to the structure. After that the sections show the design of the moment frames (green and yellow) and shear walls (red) shown in the figure above.

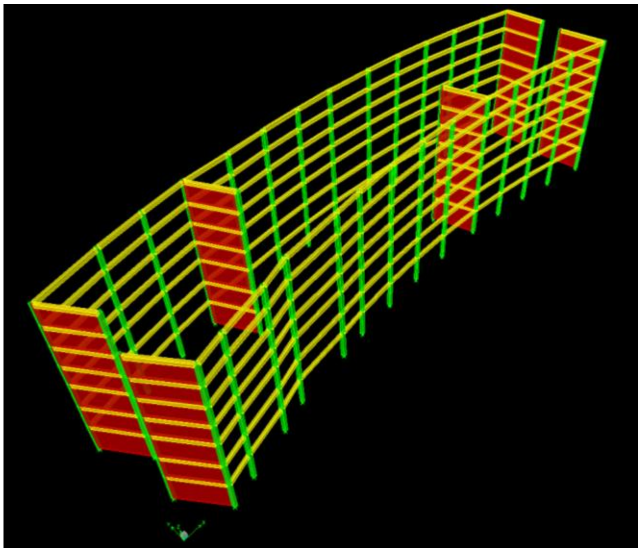


Figure 10: ETABs 3D Model

Wind Loads

For the wind load calculations the MWFRS directional procedure was used to determine the lateral loads, and the equations used to perform this method were taken from ASCE7-10 chapter 27. It turned out to be that the UMCP building is a flexible structure. All supporting calculations and applied load cases can be found in Appendix 7.

A diagram showing the wind pressure coming from North/South and East/West for those facades is shown below in figure 11 and figure 12. According to ASCE7-10 the parapets also needed to be taken as a separate practice, and are not included in the figures below. Through these calculations, the base shear for the East/West and North/South came out to be 1601kips and 1054kips, respectively. It was proven that the greater the area the more base shear will occur in the building. The allowable drift is determined by an engineering rule of thumb of (story height)/400. The drifts were taken from the ETABS model during load case three, shown in Appendix 7 taken form ASCE7-10, because that is where the most drift happens in both directions, the tables below conveys that the structure is adequate for drift serviceability.

Windward Pressure North/South											
floor	z	li	kz	q	windward, p		Windward Pressure	Windward Force	Leward, p	Leward Force	Floor Hight
1	0	85	0.85	26.63	18.18 (+/-)	4.79	22.98	72.26	23.61	74.26	0
2	17	85	0.87	27.26	18.61 (+/-)	4.91	23.52	152.28	23.61	152.88	17
3	35	85	1.01	31.65	21.61 (+/-)	5.70	27.30	161.63	23.61	139.78	18
4	49	85	1.085	34.00	23.21 (+/-)	6.12	29.33	151.93	23.61	122.30	14
5	63	85	1.142	35.78	24.43 (+/-)	6.44	30.87	159.91	23.61	122.30	14
6	77	85	1.198	37.54	25.63 (+/-)	6.76	32.38	167.75	23.61	122.30	14
roof	91	85	1.242	38.92	26.57 (+/-)	7.01	33.57	86.96	23.61	61.15	14
							Σ	880.47		720.72	

Critical Variables Found for Wind Analysis

B:	370.00	G:	0.85
g_q:	3.40	n_a:	0.75
gv:	3.40	I_z:	0.18
c:	0.20	Q:	0.80
z(bar):	54.60	N_1:	3.36
L_z:	552.98	R_n:	0.07
b(bar):	0.65	β:	0.02
ά:	0.15	g_R:	4.12
Vz(bar):	123.61	$R_{h,n}$:	2.54
l:	500.00	$R_{L,n}$:	7.94
ε:	0.20	$R_{B,n}$:	10.33
h:	91	R_h:	0.32
L:	85	R_L:	0.12
V:	120.00	R_B:	0.09
α:	9.5	R:	0.24
z_g:	900		

L_{EFF}	Base Shear	Over Turning Moment
1350	**1601.19 k**	**82767 k-ft**

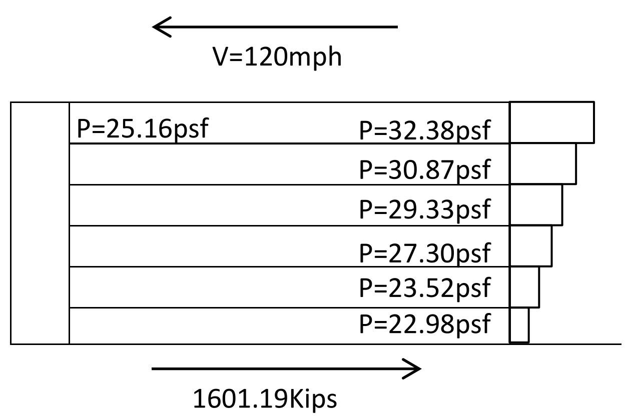


Figure 11: North/South Wind Analysis

Wind Drift North/South Direction (Y)					
Story	Allowable Drift	Check Y-Dir.	X-Dir.	Y-Dir.	Total Drift
1	**0.51**	OK	0.0002	0.0002	**0.0003**
2	**0.54**	OK	0.0004	0.0002	**0.0005**
3	**0.42**	OK	0.0006	0.0003	**0.0007**
4	**0.42**	OK	0.0009	0.0003	**0.0009**
5	**0.42**	OK	0.0014	0.0002	**0.0014**
6	**0.42**	OK	0.0020	0.0002	**0.0021**
Roof	**0.42**	OK	0.0043	0.0002	**0.0043**

Table 7: North/South Wind Story Drift

Windward Pressure East/West										
floor	z	li	kz	q	windward, p	Windward Pressure	Windward Force	Leward, p	Leward Force	Floor Hight
1	0	370	0.85	26.63	19.91 (+/-) 4.79	24.70	17.85	25.16	18.18	0
2	17	370	0.872	27.31	20.41 (+/-) 4.92	25.33	164.01	25.16	37.43	17
3	35	370	1.015	31.79	23.77 (+/-) 5.72	29.49	174.58	25.16	34.22	18
4	49	370	1.089	34.13	25.51 (+/-) 6.14	31.65	163.97	25.16	29.94	14
5	63	370	1.148	35.98	26.90 (+/-) 6.48	33.37	172.88	25.16	29.94	14
6	77	370	1.198	37.53	28.06 (+/-) 6.76	34.81	180.34	25.16	29.94	14
roof	91	370	1.241	38.88	29.06 (+/-) 7.00	36.06	21.46	25.16	14.97	14
						Σ	877.22		176.45	

Critical Variables Found for Wind Analysis

B:	85.00
g_q:	3.40
gv:	3.40
c:	0.20
z(bar):	54.60
L_z:	552.98
b(bar):	0.65
ά:	0.15
Vz(bar):	123.61
l:	500.00
ε:	0.20
h:	91
L:	370
V:	120.00
α:	9.5
z_g:	900

G:	0.93
n_a:	0.75
I_z:	0.18
Q:	0.87
N_1:	3.36
R_n:	0.07
β:	0.02
g_R:	4.12
$R_{h,n}$:	2.54
$R_{L,n}$:	34.59
$R_{B,n}$:	2.37
R_h:	0.32
R_L:	0.03
R_B:	0.33
R:	0.43

L_{EFF}	Base Shear	Over Turning Moment
310	**1053.67 k**	**52517.79 k-ft**

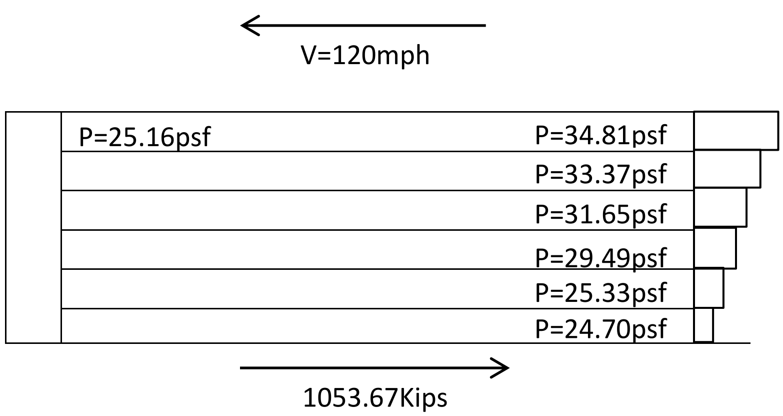


Figure 12: East/West Wind Analysis

Wind Drift East/West Direction (X)					
Story	Allowable Drift	Check X-Dir.	X-Dir.	Y-Dir.	Total Drift
1	**0.51**	OK	0.0004	0.0000	**0.0004**
2	**0.54**	OK	0.0009	0.0001	**0.0009**
3	**0.42**	OK	0.0017	0.0001	**0.0017**
4	**0.42**	OK	0.0025	0.0001	**0.0025**
5	**0.42**	OK	0.0041	0.0002	**0.0041**
6	**0.42**	OK	0.0058	0.0002	**0.0059**
Roof	**0.42**	OK	0.0277	0.0010	**0.0277**

Table 8: East/West Wind Story Drift

Seismic Loads

For the seismic design process, ASCE7-10 chapter 12 was referenced to make sure that conservative standards were met by code. The USGS Earthquake Ground Motion Parameter Application was used to find the seismic response coefficients (S_1 and S_s) for Plainsboro, New Jersey. Since all of the floors have the same gravity system, each floor weighs the same amount. The roof weighs more due to the fact that the mechanical equipment is so heavy. Also, the response modification factor value, R, is equal to 3.0 because none of my systems were design as a "Special System." The seismic design category of the building was determined as "B" from table 11.6-1. The tables below also shows the drifts that were found in ETABs with the allowable drift of the building, and it came out to be adequate for serviceability concerns. The allowable drift found in table 12.12-1 ASCE 7-10 is equal to 0.015 x (story height). The drifts taken from ETABs have to be adjusted by code by multiplying the drift by the (story height) x C_D/I. The story shear forces and the calculations for determining these values are located in Appendix 8.

Seismic Loading

		$T=$	1.003	s
		$k=$	1.250	
Mass/Area:	6.56E-05 kip	$C_S=$	0.039	
Floor Wieght:	4,889.05 kips	$V_b=$	1,144.04	kips

Floor	$h_{i\,(ft.)}$	h (ft)	w (kips)	w^*h^k	C_{VX}	f_i (kips)
Roof	14	91	29,334	8,244,749.59	0.300	344
6	14	77	29,334	6,690,970.82	0.244	279
5	14	63	29,334	5,206,566.46	0.190	217
4	14	49	29,334	3,802,951.87	0.139	158
3	18	35	29,334	2,497,242.89	0.091	104
2	17	17	29,334	1,012,597.57	0.037	42
		Σ	176,005.80	27,455,079.19		1,144.04

Overturning Moment: 78,524.09

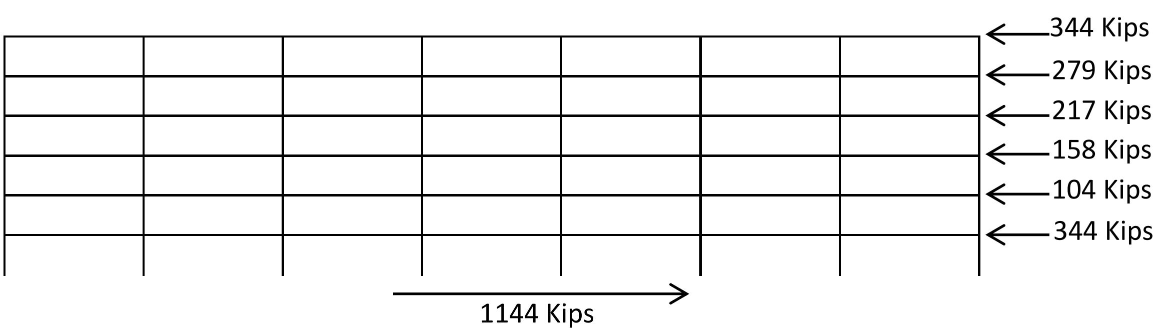


Figure 13: Seismic Analysis

Seismic Drift East/West Direction (X- Direction), I=1.25 & C_D=2.5					
Story	Allowable Drift	Check X-Dir.	X-Dir.	Y-Dir.	Total Drift
Roof	**2.52**	**OK**	0.648	0.030	**0.648**
6	**2.52**	**OK**	1.043	0.041	**1.044**
5	**2.52**	**OK**	1.413	0.052	**1.414**
4	**2.52**	**OK**	1.785	0.064	**1.787**
3	**2.52**	**OK**	2.436	0.081	**2.437**
2	**3.24**	**OK**	3.197	0.110	**3.199**
1	**3.06**	**OK**	2.927	0.186	**2.933**

Table 9: East/West Seismic Story Drift

Seismic Drift North/South Direction (Y-Direction), I=1.25 & C_D=3.0					
Story	Allowable Drift	Check Y-Dir.	X-Dir.	Y-Dir.	Total Drift
Roof	**2.52**	**OK**	0.016	0.016	**0.023**
6	**2.52**	**OK**	0.026	0.017	**0.031**
5	**2.52**	**OK**	0.035	0.017	**0.039**
4	**2.52**	**OK**	0.044	0.016	**0.047**
3	**2.52**	**OK**	0.060	0.015	**0.062**
2	**3.24**	**OK**	0.098	0.015	**0.099**
1	**3.06**	**OK**	0.185	0.011	**0.185**

Table 10: North South Seismic Story Drift

Shear Wall Design

The shear walls were only designed in the short direction, and are all the same length. This means each shear wall was designed to be identical. The max shear force was taken from the ETABs model. The wall was designed so the wall could resist the force in shear and flexure failure, and the calculations could be found in Appendix 9. The shear walls was designed to be 8" thick with horizontal reinforcement of #3 rebar at 10" spacing and vertical reinforcement with #3 rebar at 12" spacing. The flexure reinforcement was designed with four #9 rebar.

Shear Wall Design	
Horizontal Reinforcement:	#3 rebar spaced at 10"
Vertical Reinforcement:	#3 rebar spaced at 12"
Flexural Reinforcement:	(4) #9
Thickness:	8 inches

Table 11: Shear Wall Design

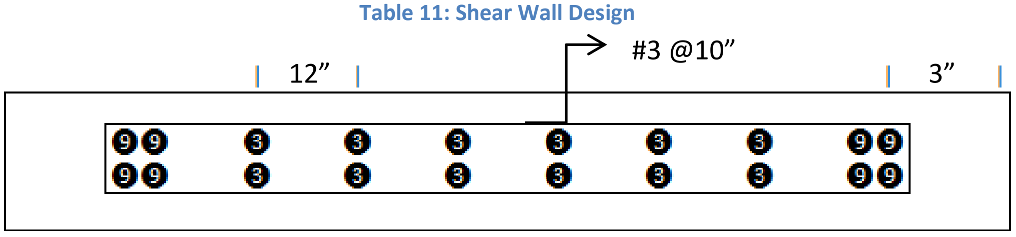


Figure 14: Shear wall Design

MOMENT FRAME DESIGN

The girder in the moment frame was designed almost the same way as the gravity girder was, but there was an additional moment added that was taken from the ETABs model. Also, the controlling load combination was 1.2D+1.0W+L. The hand calculations were perfected on a spreadsheet to get the most efficient section and reinforcement. The girder section size turned out to be an 18x30, but the reinforcement changed per floor because the lateral load decreases per floor. Appendix 10 shows the detail of the spreadsheets. The columns in the moment frame were designed like the gravity columns were, but with a max moment added from ETABs for each floor. The column changes its square dimension on almost every floor. The reinforcement changes in almost every floor as well. The reinforcement ratio always stays less than 4.0% reinforcement, as a rule of thumb. The columns were checked with spColumn. The figures below shows the tributary are of the girders and columns in the moment frame. The table on the next page lays out the section and reinforcement for each girder and column.

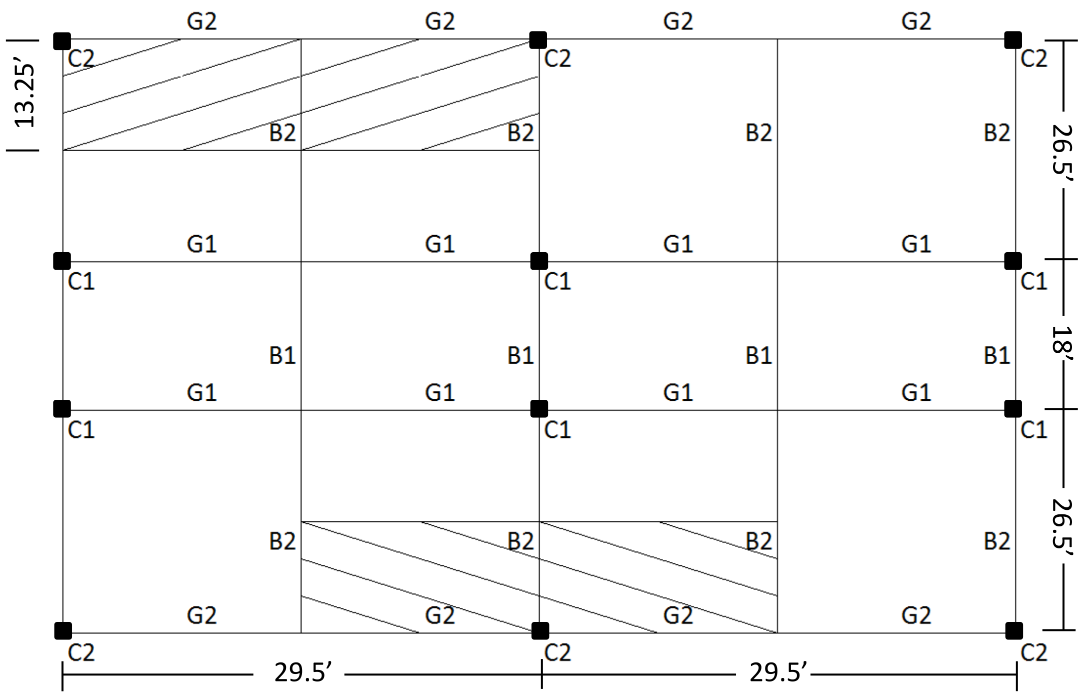


Figure 15: Lateral Column & Girder Layout with Tributary Area

Column Design, C2		Girder Design, G2	
1st Floor Section Size:	26x26	**1st Floor Section Size:**	18x30
1st Floor Reinforcement:	(16) # 10 rebar	**1st Floor Reinforcement:**	(13) # 8 rebar
2nd Floor Section Size:	24x24	**2nd Floor Section Size:**	18x30
2nd Floor Reinforcement:	(12) # 8 rebar	**2nd Floor Reinforcement:**	(9) # 8 rebar
3rd Floor Section Size:	20x20	**3rd Floor Section Size:**	18x30
3rd Floor Reinforcement:	(12) # 8 rebar	**3rd Floor Reinforcement:**	(7) # 8 rebar
4th Floor Section Size:	18x18	**4th Floor Section Size:**	18x30
4th Floor Reinforcement:	(8) # 8 rebar	**4th Floor Reinforcement:**	(5) # 8 rebar
5th Floor Section Size:	16x16	**5th Floor Section Size:**	18x30
5th Floor Reinforcement:	(8) # 8 rebar	**5th Floor Reinforcement:**	(4) # 8 rebar
6th Floor Section Size:	14x14	**6th Floor Section Size:**	18x30
6th Floor Reinforcement:	(4) # 8 rebar	**6th Floor Reinforcement:**	(3) # 8 rebar
7th Floor Section Size:	12x12	**7th Floor Section Size:**	18x30
7th Floor Reinforcement:	(4) # 8 rebar	**7th Floor Reinforcement:**	(3) # 8 rebar
f'c:	4 ksi	**f'c:**	4 ksi
fy:	60 ksi	**fy:**	60 ksi
ρ:	% < 4.0% O.K.	**ρ:**	% < 4.0% O.K.

Table 12: Moment Frame Design

Lateral Design Advantages

Each girder in the moment frame and shear wall is designed with the same section throughout the building, so this is advantageous in construction because it is simple and the formwork is reusable. The original steel moment frame design was only 26" deep, so this means that we gained 4" of plenum space decreasing the floor to ceiling height which is a weakness in this design. Not all the braced frames were switched to shear walls because there was no need for the extra stiffness which would save money in cost and construction time. Concrete is fire proof, so this saves money compared to the steel structure that needs to be fireproofed. Also, Concrete is cheaper than steel, and the cost analysis breadth will go into detail relating the pros and cons of the construction and cost of the concrete design.

Construction Impact & Costs Analysis Breadth

To help compare the new structure design to the existing structure a cost analysis for both systems was prepared. RS Means 2010 was used to quantify the impact of the cost difference by switching from steel to concrete. Also, a simplified construction schedule was developed to display what design has the best impact on overall time of the completion. These two analyses will determine what design is more cost and time efficient.

Cost Estimate

A detailed estimate of the existing and proposed design of the superstructure was compiled using RSMeans 2010. The foundation was not redesigned, so that was left out of both cost estimates. The existing structure included the steel framing members (beams, columns, and girders), metal decking, concrete slab, concrete finish, fireproofing, and curb edging. The detailed spreadsheet of the total cost and total cost with overhead and profit (O&P) can be found in Appendix 11.

RSMeans was referenced to tabulate the proposed structure to stay consistent with the cost of the original design. Both of the cost estimates were factored for location. The Cost analysis for the redesign included 400 psi concrete, pumping and placing the slab, shear walls, beams, girders, and columns, concrete finish of the slab, all reinforcement, and all form work. The form work was tabulated for a reuse factor, so it was able to be used for multiple uses. A breakdown of the cost analysis can be found in Appendix 11.

Through the cost analysis it was determined that the redesign is about three-quarters of a million dollars more than the proposed design for the total with O&P. The total without O&P for the proposed design is about one hundred thousand dollars less than the original design. The true numbers are tabulated in the table below for a better comparison.

Cost Analysis		
	Total	**Total With Overhead & Profit**
Existing Structure:	$ 5,972,968.56	**$ 7,030,233.51**
Proposed Concrete Structure:	$ 5,878,646.28	**$ 7,817,156.22**
Cost Difference:	$ 94,322.28 (Saved)	**$ 786,922.71 (Gained)**

Table 13: Cost Analysis

Project Schedule

The modifications of the original design were found to have a significant impact on the completion time of the project. Since there are many different tasks that go into constructing a concrete structure than a steel structure the two rough schedules were prepared for comparison. The downfall for constructing a concrete structure is waiting for the concrete to cure before constructing the floors above. Steel construction has no waiting time after you erect the members, so you can have multiple tasks going on at the same time.

The daily output for each task was tabulated by the crew specified in the RSMeans. It was assumed that it takes eight days for the concrete to reach enough strength to construct the framing for the next floor above. The existing structure schedule and the proposed schedule can be found in Appendix 12. The start time for both designs started in November 2011. It took approximately 100 more days to construct the proposed concrete structure than the original steel design. Typically it does take longer to erect a concrete structure than it does to erect a steel structure. These are ideal schedules that do not including any unforeseen issues that typically do happen on a jobsite.

Conclusion

The results of this breadth indicated that the existing building is cheaper to construct with overhead and profit, but the raw material is cheaper for the proposed concrete structure. Though the cost estimate was a rather rough detailed estimate, it still shows that this design is overall more expensive. The $786,922.71 increase is just a drop in the bucket for a $300 million project.

The scheduling analysis showed that the existing structure would be built almost four months faster than the concrete structure. If time constraints are an issue for the owner, then this design would not be ideal to use. No time constrains have been given, but normally each project should be constructed as fast as possible.

Sustainability Breadth

To become LEED certified the building must accumulate 45-49 points based on a credit system governed by the United States Green Building Council (USGBC). Each credit is allocated by points based on the relative importance of the building-related impacts that it addresses. The Green Building and LEED Core Concepts Guide will be referenced for dictate the actual accreditation for each innovative design added to the building. The credits will be determined by studies taken from the USGBC on previous projects. The USGBC decides what rating the building will actually receive.

GREEN ROOF DESIGN

A green roof life cycle can last two or even three times longer than a conventional roof. Depending on the plant selection the green roof does not require watering and can absorb up to 70% of storm water. The native plants that will be used in the xeriscaping of the green roof are Canadian Serviceberry, White Baneberry, and Common Yarrow.

The green roof will need to be designed with a roof-repelling membrane, which is about $10-$15 per square foot plus the green roof system: curbing, drainage layer, filter cloth, and a growing medium that costs about $15-$30 per square foot. The total green roof material plus installation for and accessible green roof will be about $125-$185 per square foot. Also, the weight of the roof with a 4" growing medium is tabulated as 45 psf of dead load, and the live load taken from ASCE is 100 psf. The green roof will take up about 3000 square feet making the green roof cost approximately $555,000. That may seem like a lot of money, but it is beneficial if you gain 3000 square feet that the patients can access to get out of a hospital atmosphere to get a breath of fresh air. Plus all the benefits a green roof adds cutting down on cooling and heating, especially since there is a full glass façade beneath the green roof in the atrium.

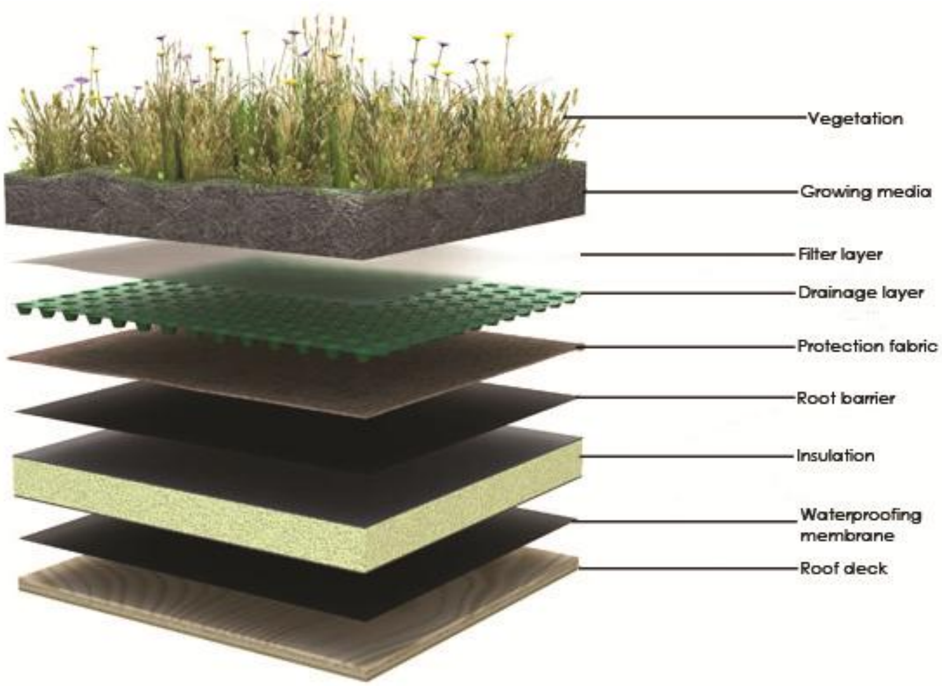


Figure 16: Green Roof Design, Courtesy of DC Green Works

The green roof added additional weight to the atrium roof, so a design of the extension of the one way slab with beams was designed to support the system that needs to be checked. The Beams and girders were able to stay the same size as the original gravity system, but with less rebar. The columns changed to 16x16 with eight #9 rebar. The structure design can be found in Appendix 13.

Green Roof Affects & Savings		
Criteria	**Savings from Conventional Design**	**Credit**
Energy :	57 kBTU/sf = 171,000 kBtu annually	2
Electricity:	28000 MWh Reduction over 30 Years	2
Water Efficiency:	Uses 75% of Storm Water, No need for Irrigation	2

Table 14: Green Roof Savings

Since a green roof cost a lot to construct per square foot a green roof was not designed for the actual roof on the seventh story. A cool roof with a reflective covering will be applied to the roof creating less of a heat island effect and reducing the heat impact in the building itself lowering cooling costs.

Indoor Strategies

The UMCP building has an efficient HVAC system helping them come closer to a LEED certification by using all outdoor air. Also, the glass curtain wall helps cut down lighting cost, and the wood louvers aid in reducing solar gain. The table below states the indoor strategies implemented into the building to gain LEED credits.

Indoor Strategies		
Use	**Savings from Conventional Design**	**Credit**
Low Flow Toilets/Facets:	67% Water Savings	10
Light bulb Use/ Light Sensors:	70% Electricity Savings	10
Recycling Bins/Source Reuse:	Bettering the Environment	3
Green Cleaning:	Bettering Indoor Air Quality	2

Table 15: Green Indoor Design Strategies

Synergies

A synergy implies the two individual parts can work together to create more than just the sum of the two credits. An example would be a water heater wouldn't have to heat as much water out of a low flow shower head/sink because less water is pouring out per minute reducing the cost of the heat and reducing the cost of the water, as well as helping the environment by reducing emissions.

Conclusion

There are a total of 31 credits not including the cool roof; this alone is not enough to become lead certified. If you implement synergies into the accreditation the building could become close to being LEED Certified. Furthermore, if you account for the existing sustainable attributes could bring the project closer to a LEED Certified building. Overall, the green roof with the other green advancements added to the project would increases the cost of the project, but in the long run the savings could be paid back within the decade. This would also become a better place for sick patients to reside because there would be fewer emissions produced from the building with a higher indoor quality environment. The green roof could brighten ones day by taking them out of the hospital to an outdoor environment, but still keeping them close to the safety of the hospital.

Conclusion

The focus of this report was to weigh the pros and cons of redesigning the superstructure from structural steel to reinforced concrete. A cost and schedule analysis was also taken into account to help justify if the proposed solution would be better or worse than the original.

The redesign of the building was determined to be a 6.5" reinforced one way slab, with gravity beams and girders at a size of 10x20, but with different reinforcement at four #8 and seven #8 rebar consecutively. Also, the typical gravity columns stayed the same size throughout the building at 20X20 square columns. The restructure of the lateral system was determined to have 18x30 girders with varying reinforcement per floor, and varying square columns and reinforcement per floor for the moment frames in the East/West direction. The shear walls in the North/South direction are all 26' long and were designed the same throughout the each floor at 8" thick with vertical reinforcement of #3 rebar spaced at 12" on center, horizontal reinforcement of #3 rebar spaced at 10" on center, and flexural reinforcement of four # 9 rebar. All the criteria was met for strength due to this design as well as serviceability such as drift, deflection, and vibration.

The cost analysis determined that the raw materials are cheaper than the raw steel materials. With overhead and profit of the reinforced concrete project was determined to be $786,923. If this is a low budget project then a reinforced concrete structure might not be feasible, but in that amount of increase in cost compared to the actual full cost of the project is not that big of a difference in the whole scheme of things. There were two schedules that were constructed to compare which structure would be erected faster. The concrete structure took an extra 100 days for the completion of the assembly. Since most projects want to be done as fast as possible the steel structure would be ideal, but if there were no time constraints there would be no reason for the construction of the concrete building not to be used.

The breadth for becoming LEED certified included the design of a green roof and implementing other sustainable techniques. The green roof would increase the project cost by approximately $555,000. This increase in money is detrimental in the beginning, but has a lot of payback cost to it throughout the buildings lifecycle. The other green strategies used throughout the building would increase the cost in the project as well, but they too have an effect on payback as well as bettering the environment. If the budget could have been increased the use of more sustainable techniques would better the lifecycle cost of the building, and could possibly make UMCP LEED Certified by the USGBC.

Overall, the results of this thesis had a great impact on the system and lifespan of the building, which would better the patients stay at the UMCP. These designs and strategies ended up costing more money, but with the sustainability techniques the building would have a lot of payback. Also, the time span of the construction would increase dramatically. If time and budget were not an issue the redesign would be adequate.

References

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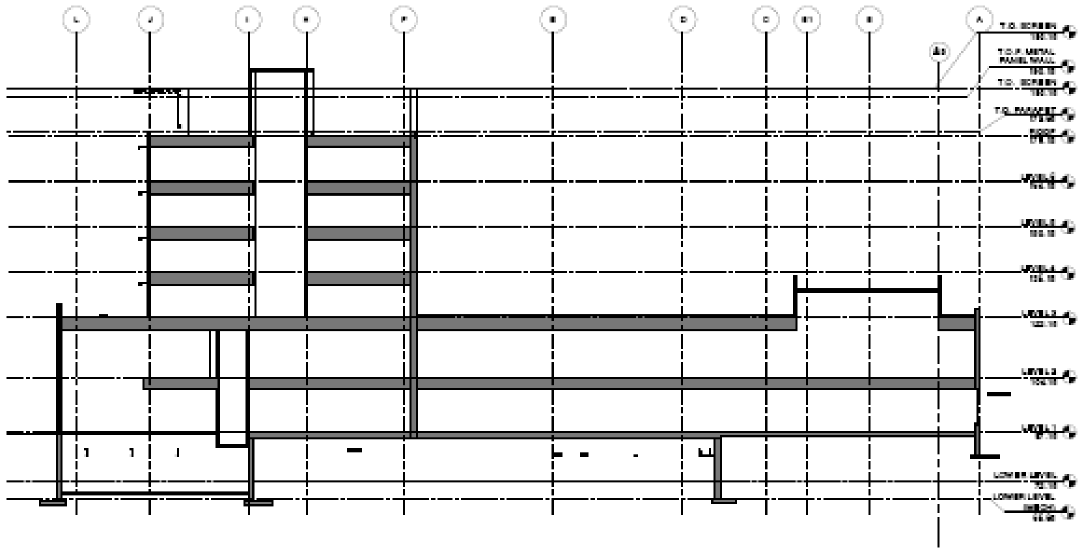
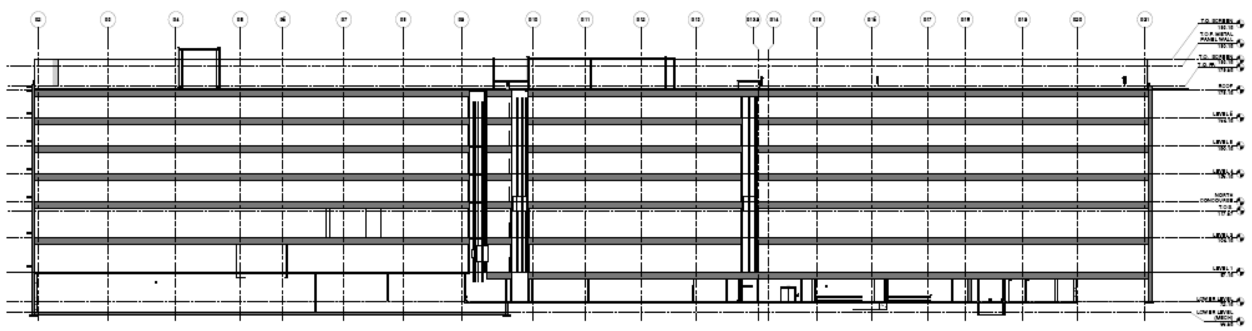
Appendices

Appendix 1: Architectural Sections & Plans



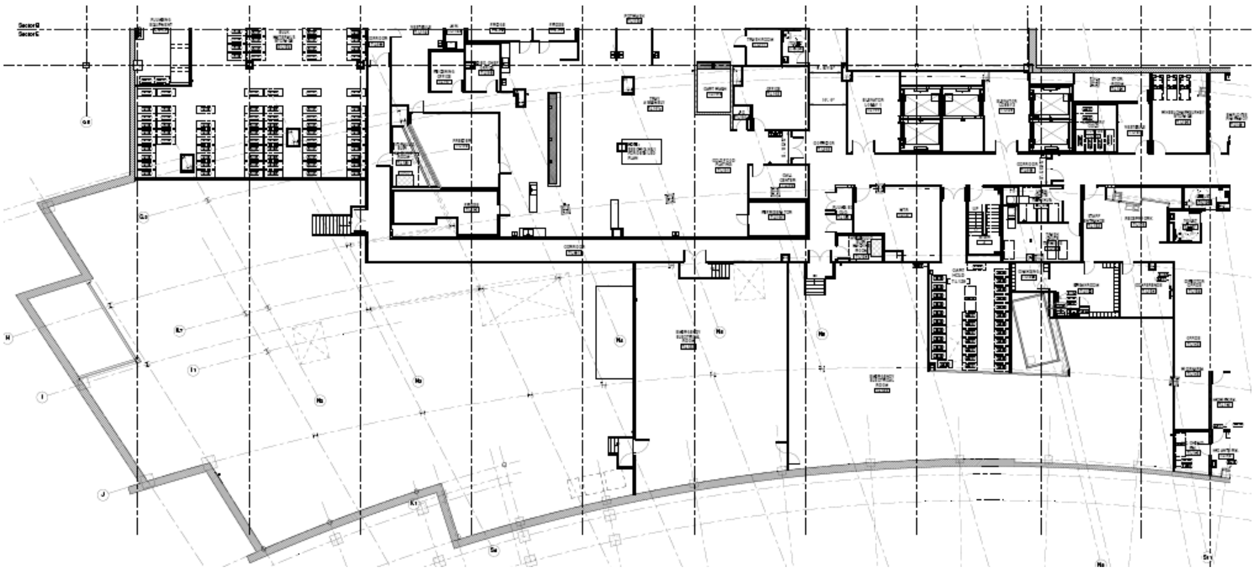
East/West Section

Courtesy of Turner Construction



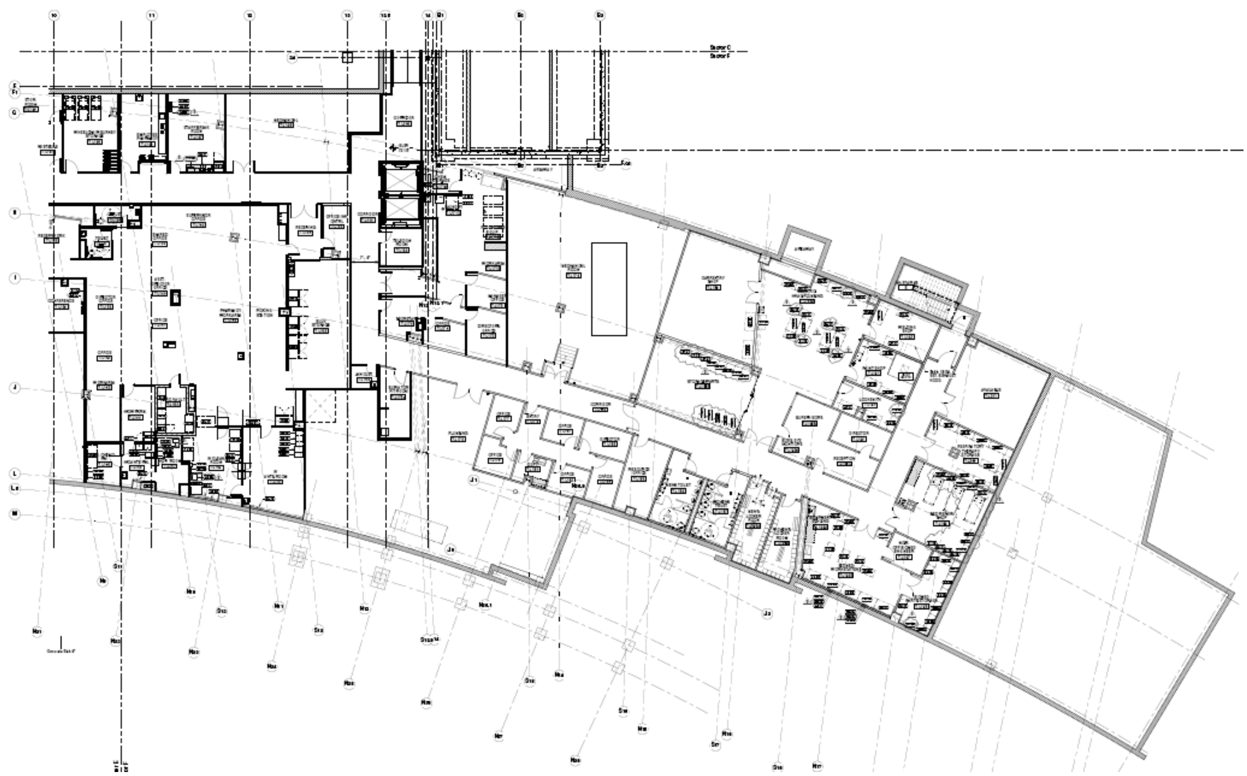
North/South Section

Courtesy of Turner Construction



TYPICAL WEST END FLOOR PLAN

COURTESY OF TURNER CONSTRUCTION



TYPICAL WEST END FLOOR PLAN

COURTESY OF TURNER CONSTRUCTION

Appendix 2: Slab Design

SOLID ONE-WAY SLABS—SINGLE SPAN

f'_c = 4,000 psi

Grade 60 Bars

Bottom Steel for + M_u

$\rho \approx 0.0050$

ckness (in.)	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10
ttom Bars	#4	#4	#4	#4	#5	#5	#5	#5	#6	#6	#6	#6	#6
Spacing (in.)	12	11	10	8	12	11	10	9	12	11	11	10	9
p Bars	#3	#3	#3	#4	#4	#4	#4	#4	#4	#4	#4	#4	#4
Spacing (in.)	12	12	12	12	12	12	12	12	12	12	12	12	12
S Bars	#3	#3	#3	#3	#3	#3	#3	#3	#3	#3	#3	#4	#4
Spacing (in.)	11	11	11	11	10	9	8	8	7	7	6	11	11
eas of Steel (in.²/ft) Bottom	0.200	0.218	0.240	0.300	0.310	0.338	0.372	0.413	0.440	0.480	0.480	0.528	0.587
ab Wt. (psf)	50	56	63	69	75	81	88	94	100	106	113	119	125
eel Wt. (psf)	1.25	1.31	1.38	1.73	1.83	1.96	2.15	2.29	2.48	2.61	2.72	2.84	3.04
LEAR SPAN	FACTORED USABLE SUPERIMPOSED LOAD (psf)												
6'-0"	510	661	841										
6'-6"	425	553	705										
7'-0"	359	467	598	860	982								
7'-6"	305	398	511	738	844								
8'-0"	260	342	440	639	730	889							
8'-6"	224	295	381	556	637	776	943						
9'-0"	193	256	332	487	558	682	830						
9'-6"	167	223	290	429	492	602	734	898					
10'-0"	145	194	254	379	435	534	652	799	917				
10'-6"	126	170	223	336	386	475	582	714	821	973			
11'-0"	109	149	197	299	344	424	521	641	737	875	938		
11'-6"	95	131	174	266	307	380	467	577	664	790	847		
12'-0"	82	114	153	238	274	341	421	520	600	715	766	911	
12'-6"	71	100	135	212	246	306	379	471	544	649	696	828	991
13'-0"	61	87	119	190	220	276	343	427	493	590	633	755	905
13'-6"	52	76	105	170	198	249	310	387	449	538	577	690	828
14'-0"	44	66	92	152	178	225	281	352	409	491	527	631	759
14'-6"		57	81	136	159	203	255	321	373	449	482	579	698
15'-0"		49	71	122	143	183	231	292	341	411	441	531	642
15'-6"		41	61	109	128	165	210	266	311	377	405	488	592
16'-0"			53	97	115	149	190	243	285	346	372	450	546
16'-6"			45	86	102	134	172	222	261	318	341	414	504
17'-0"				77	91	121	156	202	239	292	314	382	466
17'-6"				68	81	109	142	185	218	268	288	352	432
18'-0"				59	72	97	128	168	200	247	265	325	400
18'-6"				52	63	87	115	153	183	227	244	300	370
19'-0"				45	55	77	104	139	167	208	224	277	343
19'-6"					48	68	93	127	152	191	206	256	318
20'-0"					41	60	83	115	139	176	189	236	295

Note: See Fig. 7-1 for reinforcing bar details.

*Service loads corresponding to 1/1.6 of the tabulated superimposed load results in calculated immediate deflection of 1/360 span.

"H" – Use hooked or headed bars.

Concrete Reinforcing Steel Institute					
Clear Span:	14.5	feet	1.2D+1.6L	170	psf
Thickness:	6.5	inch	Factored Load:		
f'c:	4	ksi	ṕ:	0.005	
Bar Grade:	60				
Concrete Slab Design					
Bottom Bars:	#7	spaced at	11	inch	
Top Bars:	#4	spaced at	12	inch	
T-S Bars	#3	spaced at	9	inch	
Ares of Steel:	0.655	in^2			
slab Wt.:	81	psf			

Total Weight for All Slab Reinforcement			
Number of Rebar Spaced Across the Slab	Length	Weight	
77	617.5	342.2	ton
71	617.5	102.5	ton
	Total:	**444.7**	ton

Appendix 3: Gravity Beam Design

	f'c:	4	ksi	Clear Cover:	1.5	inch						
	fy:	60	ksi	Conc. Wieght:	150	pcf						
	Slab, t:	6.5	inch	Stirrup Size:	#	3	Stirrup Diameter:	0.357	inch			
	Misc. Dead Load:	35	psf	Bar Size:	#	8	Bar Diameter, d_b:	1.000	inch			
	Live Load:	80	psf	# of bars, n:	5		Area of Steel, A_b:	0.79	in^2			
	β_1:	0.85		ε_u:	0.003		Area of Steel, A_s:	3.95	in^2			
Beam Design, B1							Tributary Area:	391	ft^2			
	Span:	26.5	feet	Two Row	Yes		Influence Area:	782	ft^2	Live Load Reduction		
	Spaced:	14.75	feet	Reinforcement?			L_L=Max of	0.4				
								.25+15/√(K_{LL}*A_T)=		0.79	L_L:	62.92
Spacing,S:	Max of d_b, 1", 3/4A_b			b_{min}=2*Cc + n*d_b + 2d_{st} + (n-1)*S								
	d_b:	1.0	inch	b_{min}:	7.71	inch				Total Factored Weight, Wu		
	3/4A_b:	0.6	inch							Dead Load: Misc. dead + Slab		
	1":	1.0	inch	h_{min}>l/18.5 (ACI 318-08)		Table 9.5 min h>l/18.5					116.25	psf
				h_{min}:	17.19	inch	min h:	17.2		Wu=1.2D +1.6L		
	S:	1.0	inch							240.17004	psf	
Try a:												
	b:	10	inch	OK	d=h-d_b/2-Cc			a=A_s*fy/(.85*f'c*b_{eff})			c=a/β_1	
	h:	20	inch	OK	d:	17		a:	0.88	Rectangular	c:	1.03
										Section		
b_{eff}= min	b*16*h_f	1040			ε_t=ε_u(d-c)/c							
	Trib width	177			ε_t:	0.05	Φ=0.9					
	.25L	79.5										
	b_{eff}:	79.5	inch									
Check Flexure, ΦMn>Mu					Cheak Shear, Vn>Vu		Girder width, b:	10	inch			
Mu=Wu*Ln^2/8					Vu=Wu*Ln/2							
	Mu:	291.7	kip-ft		Vu:	46.9	Kip					
ΦMn=0.9*$A_s$fy(d-a/2)					Vn=10*(f'c)^(1/2)*b*d							
	ΦMn:	294.4	kip-ft	OK	Vn:	126491	Kip	OK				
Use Member Size:		10	x	20								
	Beam Wieght:	141	plf									

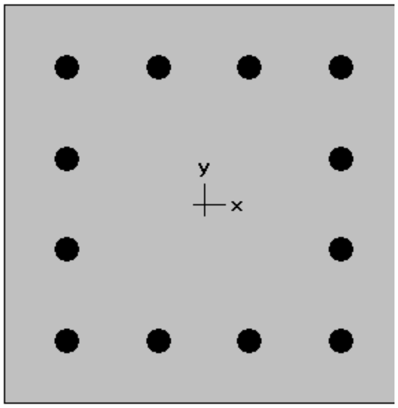
	f'c:	4	ksi	Clear Cover:	1.5	inch						
	fy:	60	ksi	Conc. Wieght:	150	pcf						
	Slab, t:	6.5	inch	Stirrup Size:	#	3	Stirrup Diameter:	0.357	inch			
	Misc. Dead Load:	35	psf	Bar Size:	#	8	Bar Diameter, d_b:	1.000	inch			
	Live Load:	80	psf	# of bars, n:	4		Area of Steel, A_b:	0.79	in^2			
	β_1:	0.85		ε_u:	0.003		Area of Steel, A_s:	3.16	in^2			
Beam Design, B2							Tributary Area:	266	ft^2			
	Span:	18	feet	Two Row	Yes		Influence Area:	531	ft^2	Live Load Reduction		
	Spaced:	14.75	feet	Reinforcement?			L_L=Max of	0.4				
								$.25+15/\sqrt{(K_{LL}*A_T)}$=		0.90	L_L:	72.08
Spacing,S:	Max of d_b, 1", $3/4A_b$			$b_{min} = 2*Cc + n*d_b + 2d_{st} + (n-1)*S$								
d_b:	1.0	inch		b_{min}:	6.71	inch				Total Factored Weight, Wu		
$3/4A_b$:	0.6	inch								Dead Load: Misc. dead + Slab		
1":	1.0	inch		h_{min}>l/18.5 (ACI 318-08)			Table 9.5 min h>l/18.5				116.25	psf
				h_{min}:	11.68	inch	min h:	11.7		Wu=1.2D +1.6L		
S:	1.0	inch								254.8209	psf	
Try a:												
b:	10	inch	OK		$d=h-d_b/2-Cc$			$a=A_s*fy/(.85*f'c*b_{eff})$			$c=a/\beta_1$	
h:	20	inch	OK		d:	17		a:	1.03	Rectangular	c:	1.21
										Section		
b_{eff}= min	$b*16*h_f$	1040			$\varepsilon_t=\varepsilon_u(d-c)/c$							
	Trib width	177			ε_t:	0.04	Φ=0.9					
	.25L	54										
b_{eff}:	54	inch										
Check Flexure, ΦMn>Mu					Cheak Shear, Vn>Vu			Girder width, b:	0	inch		
Mu=Wu*Ln^2/8					Vu=Wu*Ln/2							
Mu:	152.2	kip-ft			Vu:	33.8	Kip					
ΦMn=0.9*$A_s$fy(d-a/2)					Vn=10*(f'c)^(1/2)*b*d							
ΦMn:	234.4	kip-ft	OK		Vn:	126491	Kip	OK				
Use Member Size:		**10**	x	**20**								
	Beam Wieght:	141	plf									

Appendix 4: Gravity Girder Design

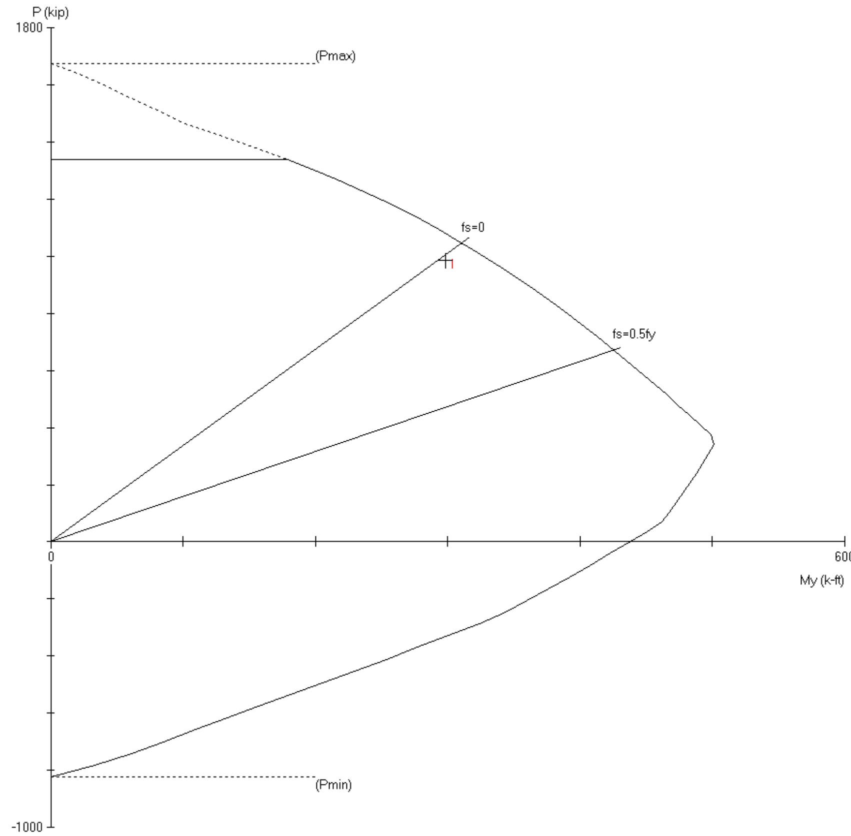
	Floor:	All	/	7								
	f'c:	4	ksi	Clear Cover:	1.5	inch						
	fy:	60	ksi	Conc. Wieght:	150	pcf						
	Slab, t:	6.5	inch	Stirrup Size:	#	3	Stirrup Diameter:	0.357	inch			
	Misc. Dead Load:	35	psf	Bar Size:	#	8	Bar Diameter, d_b:	1.000	inch			
	Live Load:	80	psf	# of bars, n:	7		Area of Steel, A_b:	0.79	in^2			
	β_1:	0.85		ε_u:	0.003		Area of Steel, A_s:	5.53	in^3			
	Typical											
	Girder Design, G1	(gravity)					Tributary Area:	656	ft^2			
	Span:	29.5	feet	Two Row Reinforcement?	Yes		Influence Area:	2626	ft^2	Use Live Load Reduction		
	Spaced Left:	26.5	feet				L_r=Max of	0.4				
	Spaced Right:	18	feet					.25+15/√(K_{LL}*A_T)=		0.54	L_r:	43.42
Spacing,S	Max of d_b, 1", 3/4A_b			b_{min}=2*Cc + n*d_b + 2d_{st} + (n-1)*S								
d_b:	1.0	inch		b_{min}:	9.71	inch	Total Factored Weight, Wu					
3/4A_b:	0.6	inch					Dead Load: Misc. dead + Slab				Beam Wieght	
1":	1.0	inch		h_{min}>l/18.5 (ACI 318-08)					116	psf	141	plf
				h_{min}:	19.14	inch		Wu=1.2D +1.6L			Pu=1.2D	
S:	1.0	inch						209	psf		3755	pounds
Try a												
b:	10	inch	OK		d=h-d_b/2-Cc			a=A_s*fy/(.85*f'c*b_{eff})			c=a/β_1	
h:	20	inch	OK		d:	17		a:	1.10	Rectangula r Section	c:	1.30
b_{eff}= min	b*16*h_f	1040			ε_t=ε_u(d-c)/c							
	Trib width	318			ε_t:	0.04	Φ=0.9					
	.25L	88.5										
b_{eff}:	88.5	inch										
Check Flexure, ΦMn>Mu					Cheak Shear, Vn>Vu		Column width:		20	inch		
Mu=0.107*Ln^2+Pu*Ln/8, continuous + point load					Vu=Wu*Ln/2+Pu/2							
Mu:	398	kip-ft			Vu:	84	Kip					
ΦMn=0.9*$A_s$fy(d-a/2)					Vn=10*(f'c)^(1/2)*b*d							
ΦMn:	409	kip-ft	OK		Vn:	126491	Kip	OK				
Use Member Size:		**10**	**x**	**20**								
	Beam Wieght:	141	plf									

	Floor:	All	/	7								
	f'c:	4	ksi	Clear Cover:	1.5	inch						
	fy:	60	ksi	Conc. Wieght:	150	pcf						
	Slab, t:	6.5	inch	Stirrup Size:	#	3	Diameter:	0.357	inch			
Misc. Dead Load:		35	psf	Bar Size:	#	8	meter, d_b:	1.000	inch			
	Live Load:	80	psf	# of bars, n:	9	Area of Steel, A_b:		0.79	in^2			
	β_1:	0.85		ε_u:	0.003	Area of Steel, A_s:		7.11	in^3			
Long Span												
Girder Design, G1		(gravity)					Tributary Area:	712	ft^2			
	Span:	32	feet	Two Row Reinforcement?	Yes		Influence Area:	2848	ft^2	Use Live Load Reduction		
	Spaced Left:	26.5	feet				L_r=Max of	0.4				
	Spaced Right:	18	feet					$.25+15/\sqrt{(K_{LL}*A_T)}=$		0.53	L_r:	42.49
Spacing,S	Max of d_b, 1", $3/4A_b$			$b_{min}=2*Cc + n*d_b + 2d_{st} + (n-1)*S$								
d_b:	1.0	inch		b_{min}:	11.71	inch	Total Factored Weight, Wu					
$3/4A_b$:	0.6	inch					Dead Load: Misc. dead + Slab				Beam Wieght	
1":	1.0	inch		h_{min}>l/21 (ACI 318-08)					116	psf	141	plf
				h_{min}:	18.29	inch	Wu=1.2D +1.6L				Pu=1.2D	
S:	1.0	inch						207	psf		3755	pounds
Try a												
b:	12	inch	OK		d=h-d_b/2-Cc			a=A_s*fy/(.85*f'c*b_{eff})			c=a/β_1	
h:	20	inch	OK		d:	17		a:	1.31	Rectangular	c:	1.54
b_{eff}= min	b*16*h_f	1248			$\varepsilon_t=\varepsilon_u$(d-c)/c							
	Trib width	318			ε_t:	0.03	Φ=0.9					
	.25L	96										
b_{eff}:	96	inch										
Check Flexure, ΦMn>Mu					Cheak Shear, Vn>Vu		Column width:		20	inch		
Mu=0.107*Ln^2+Pu*Ln/8, continuous + point load					Vu=Wu*Ln/2+Pu/2							
Mu:	469	kip-ft			Vu:	90	Kip					
ΦMn=0.9*$A_s$fy(d-a/2)					Vn=10*(f'c)^(1/2)*b*d							
ΦMn:	523	kip-ft	OK		Vn:	151789	Kip	OK				
Use Member Size:		**12**	**x**	**20**								
	Beam Wieght:	169	plf									

Appendix 5: Gravity Column Design



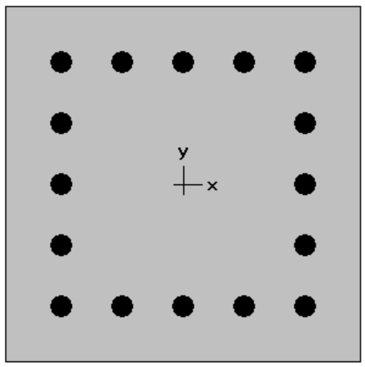
20 x 20 in
3.81% reinf.



MATERIAL:
=========
f'c = 4 ksi
Ec = 3605 ksi
fc = 3.4 ksi
Beta1 = 0.85
fy = 60 ksi
Es = 29000 ksi

SECTION:
========
Ag = 400 in^2
Ix = 13333.3 in^4
Iy = 13333.3 in^4
Xo = 0 in
Yo = 0 in

REINFORCEMENT:
===============
12 #10 bars @ 3.810%
As = 15.24 in^2
Confinement: Tied
Clear Cover = 2.50 in
Min Clear Spacing = 3.31 in

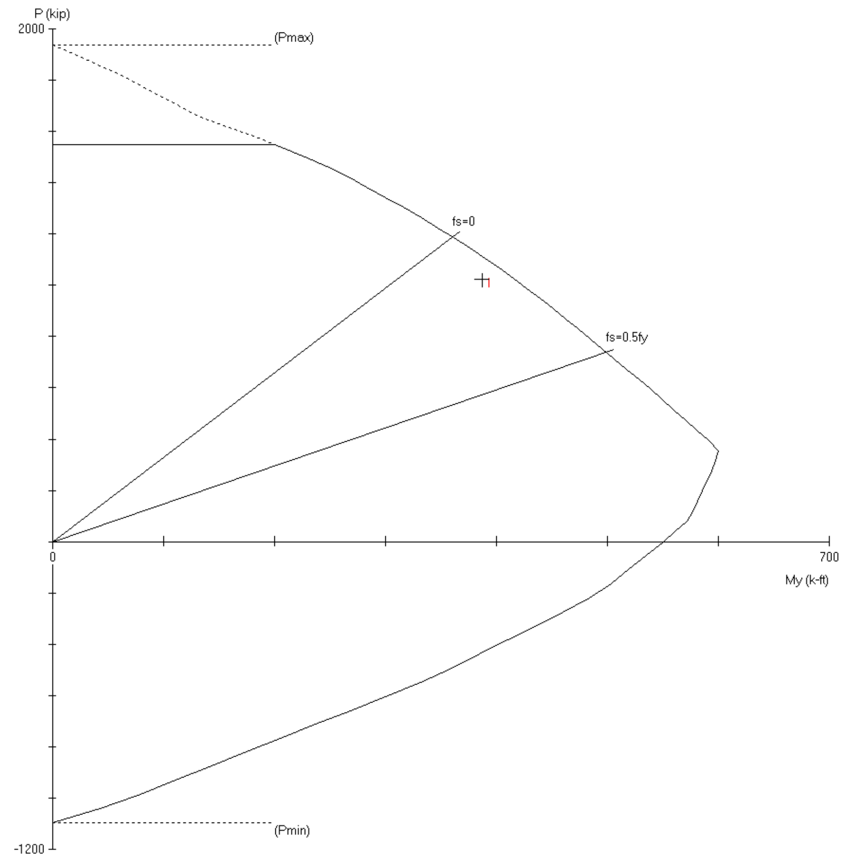


20 x 20 in
5.08% reinf.

MATERIAL:
=========
f'c = 4 ksi
Ec = 3605 ksi
fc = 3.4 ksi
Beta1 = 0.85
fy = 60 ksi
Es = 29000 ksi

SECTION:
========
Ag = 400 in^2
Ix = 13333.3 in^4
Iy = 13333.3 in^4
Xo = 0 in
Yo = 0 in

REINFORCEMENT:
==============
16 #10 bars @ 5.080%
As = 20.32 in^2
Confinement: Tied
Clear Cover = 2.50 in
Min Clear Spacing = 2.16 in



Appendix 6: Vibration Table

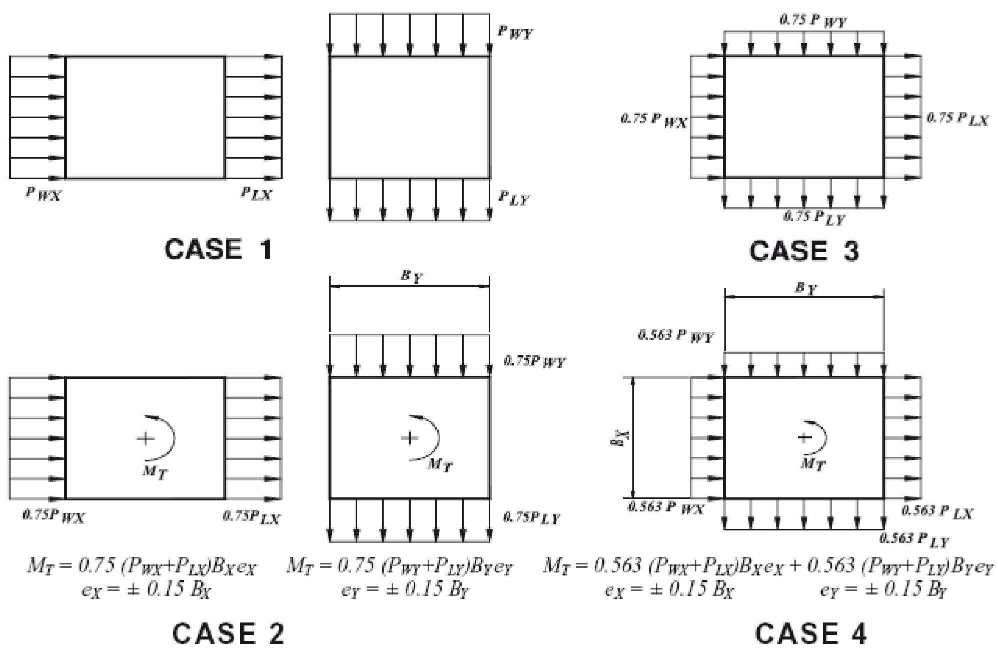
Table 1.2-5

Maximum Limits on Footfall Vibration in Health Care Facilities

Space Type	Footfall Vibration Peak Velocity (micro-in/s)
Patient rooms and other patient areas	4000
Operating and other treatment rooms	4000
Administrative areas	8000
Public circulation areas	8000

Appendix 7: Wind Calculations

Main Wind Force Resisting System – Part 1		All Heights
Figure 27.4-8	Design Wind Load Cases	



Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes:

1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of 27.4.1 and 27.4.2 as applicable for building of all heights.
2. Diagrams show plan views of building.
3. Notation:

   P_{WX}, P_{WY}: Windward face design pressure acting in the x, y principal axis, respectively.
   P_{LX}, P_{LY}: Leeward face design pressure acting in the x, y principal axis, respectively.
   $e\ (e_X, e_Y)$: Eccentricity for the x, y principal axis of the structure, respectively.
   M_T: Torsional moment per unit height acting about a vertical axis of the building.

Alex Burg | Final Thesis Design | Wind Loads | 1/1

Short Direction

Exposure C

V = 120 mph (fig 26.5-1B)

I = 1.0 (Table 1.5-2)

kd = 0.85 (Table 26.6-1)

GCpi = ± 0.18 (Enclosed Building)

$k_{zt} = 1.0$ (26.8.2)

Li = 370'

$L_{eff} = \frac{\Sigma h_i L_i}{\Sigma h_i} = 310$

· 91' < 300' } approximate
· 91' < 4(310) } frequency applies, n_a

$n_a = \frac{43.5}{h^{0.9}}$ (concrete moment resisting frame)

$n_a = \frac{43.5}{91^{0.9}} = \boxed{0.75} < 1.0$ Flexible

Gust Factor

$$G = 0.925\left(\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right)$$

$$= 0.925\left(\frac{1 + 1.7(0.18)\sqrt{3.4^2(.87)^2 + (4.12)^2(0.43)^2}}{1 + 1.7(3.4)(0.18)}\right)$$

G = 0.93

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = .2\left(\frac{33}{54.6}\right)^{1/6} = 0.18$

$Q = \sqrt{\frac{1}{1 + .63\left(\frac{B + h}{L_{\bar{z}}}\right)^{0.63}}} = .87$

$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 552$

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

$g_R = 4.12$

$V_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}}\left(\frac{88}{60}\right)V = 123.61$

$N_1 = \frac{n_a L_{\bar{z}}}{V_{\bar{z}}} = 3.36$

$R_n = 0.07$ $\beta = 0.02$

$R = 0.43$

Velocity Exposure Coefficient

$K_z = 2.01(z/z_g)^{2/\alpha}$

$= 2.01(17/900)^{2/9.5}$

$K_z = 0.87$ ← first floor only

Velocity Pressure, q_z

$q_z = 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d V^2$

$= 0.00256(0.87)(.85)(1)(120)^2$

$= 27.31$

Wind Pressure, P

Refer to Wind Analysis Spread Sheet

Windward Pressure North/South										
floor	z	li	kz	q	windward, p	Windward Pressure	Windward Force	Leward, p	Leward Force	Floor Hight
1	0	85	0.85	26.63	18.18 (+/-) 4.79	22.98	72.26	23.61	74.26	0
2	17	85	0.87	27.26	18.61 (+/-) 4.91	23.52	152.28	23.61	152.88	17
3	35	85	1.01	31.65	21.61 (+/-) 5.70	27.30	161.63	23.61	139.78	18
4	49	85	1.085	34.00	23.21 (+/-) 6.12	29.33	151.93	23.61	122.30	14
5	63	85	1.142	35.78	24.43 (+/-) 6.44	30.87	159.91	23.61	122.30	14
6	77	85	1.198	37.54	25.63 (+/-) 6.76	32.38	167.75	23.61	122.30	14
roof	91	85	1.242	38.92	26.57 (+/-) 7.01	33.57	86.96	23.61	61.15	14
						Σ	880.47		720.72	

L_{EFF}	Base Shear	Over Turning Moment
1350	**1601.19 k**	**82767 k-ft**

B:	370.00
g_q:	3.40
gv:	3.40
c:	0.20
z(bar):	54.60
L_z:	552.98
b(bar):	0.65
ά:	0.15
Vz(bar):	123.61
l:	500.00
ε:	0.20
h:	91
L:	85
V:	120.00
α:	9.5
z_g:	900

G:	0.85
n_a:	0.75
I_z:	0.18
Q:	0.80
N_1:	3.36
R_n:	0.07
β:	0.02
g_R:	4.12
$R_{h,n}$:	2.54
$R_{L,n}$:	7.94
$R_{B,n}$:	10.33
R_h:	0.32
R_L:	0.12
R_B:	0.09
R:	0.24

Case 1, P	Case 2, Mt	Case 3, P	Case 4, Mt	Case 4, P
147	1939	110	1893	82
305	1962	229	1916	172
301	2119	226	2073	170
274	2204	206	2155	154
282	2268	212	2220	159
290	2331	218	2276	163
148	2380	111	2323	83

Windward Pressure East/West											
floor	z	li	kz	q	windward, p		Windward Pressure	Windward Force	Leward, p	Leward Force	Floor Hight
1	0	370	0.85	26.63	19.91 (+/-)	4.79	24.70	17.85	25.16	18.18	0
2	17	370	0.872	27.31	20.41 (+/-)	4.92	25.33	164.01	25.16	37.43	17
3	35	370	1.015	31.79	23.77 (+/-)	5.72	29.49	174.58	25.16	34.22	18
4	49	370	1.089	34.13	25.51 (+/-)	6.14	31.65	163.97	25.16	29.94	14
5	63	370	1.148	35.98	26.90 (+/-)	6.48	33.37	172.88	25.16	29.94	14
6	77	370	1.198	37.53	28.06 (+/-)	6.76	34.81	180.34	25.16	29.94	14
roof	91	370	1.241	38.88	29.06 (+/-)	7.00	36.06	21.46	25.16	14.97	14
							Σ	877.22		176.45	

L_{EFF}	Base Shear	Over Turning Moment
310	**1053.67 k**	**52517.79 k-ft**

B:	85.00	G:	0.93
g_q:	3.40	n_a:	0.75
gv:	3.40	I_z:	0.18
c:	0.20	Q:	0.87
z(bar):	54.60	N_1:	3.36
L_z:	552.98	R_n:	0.07
b(bar):	0.65	β:	0.02
ά:	0.15	g_R:	4.12
Vz(bar):	123.61	$R_{h,n}$:	2.54
l:	500.00	$R_{L,n}$:	34.59
ε:	0.20	$R_{B,n}$:	2.37
h:	91	R_h:	0.32
L:	370	R_L:	0.03
V:	120.00	R_B:	0.33
α:	9.5	R:	0.43
z_g:	900		

Case 1, P	Case 2, Mt	Case 3, P	Case 4, Mt	Case 4, P
36	477	27	1893	20
201	483	151	1916	113
209	523	157	2073	118
194	543	145	2155	109
203	560	152	2220	114
210	574	158	2276	118
36	585	27	2323	21

Appendix 8: Seismic Calculations

Alex Burg | Final Thesis Design | Seismic Loads

$S_S = 0.31$ (usgs) $F_a = 1.45$ (11.4-1)
$S_1 = 0.064$ (usgs) $F_v = 2.4$ (11.4-2)

$S_{MS} = F_a S_S = 1.45(0.31) = \underline{0.45}$ $S_{M1} = F_v S_1 = 2.4(0.064) = \underline{0.15}$

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.45) = \underline{0.3}$ $S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.15) = \underline{0.1}$

Design Category = B => C-1

$I = 1.25$ Table 1.5-2

$R = 3$ Concrete (Non-Special)

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

$h_n = 91'$

$T_a = C_t h_n^x = 0.02(91)^{.75} = 0.59$ $T_L = 6$ sec (Fig 22-2) $C_u = 1.7$ (table 12.8-1)

$T = C_u T_a = 1.7(0.59) = 1.003$

$T > T$ use $C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \frac{0.1}{1.003\left(\frac{3}{1.25}\right)} = 0.042$

$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{0.3}{\left(\frac{3}{1.25}\right)} = 0.125$

$C_s = 0.042 > 0.01$ ∴ ✓
$C_s = 0.042 < 0.125$ ∴ ✓
so
$C_s = 0.039$

Base Shear

$V = C_s W$
$V = \frac{0.039(32.551E6)}{1{,}000}$
$\boxed{V = 1{,}269.5 \text{ kip}}$

Assuming floors 2-7 are equal weight
$W = (9''/12'')150\,pcf + 20\,psf + 15\,psf = 172.5\,psf$
Area $= 370' \times 85' = 31{,}450\ ft^2$
$W_{total} = 31{,}450(172.5)(6\ floors) = 32.551E6$

Refer To Seismic Analysis spreadsheet

Seismic Loading

Mass/Area: 6.56E-05 kip
Floor Wieght: 4,889.05 kips

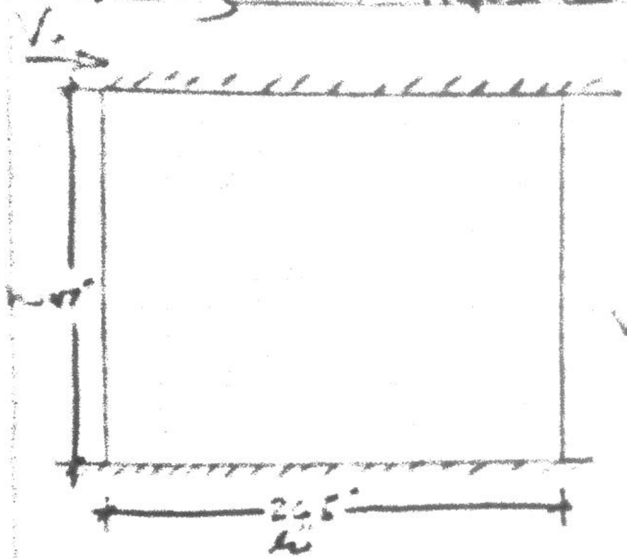
$T=$ 1.003 s
$k=$ 1.250
$C_S=$ 0.039
$V_b=$ 1,144.04 kips

Floor	$h_{i\,(ft.)}$	h (ft)	w (kips)	$w*h^k$	C_{VX}	f_i (kips)
Roof	14	91	29,334	8,244,749.59	0.300	344
6	14	77	29,334	6,690,970.82	0.244	279
5	14	63	29,334	5,206,566.46	0.190	217
4	14	49	29,334	3,802,951.87	0.139	158
3	18	35	29,334	2,497,242.89	0.091	104
2	17	17	29,334	1,012,597.57	0.037	42
		Σ	176,005.80	27,455,079.19		1,144.04

Overturning Moment: 78,524.09

Appendix 9: Shear Wall Design

Alex Burg — Proposed Shear Wall Design



$f'_c = 4 \text{ ksi}$
$E_c = 57\sqrt{f'_c} = 3,605 \text{ ksi}$
$E_{s} = 29{,}000 \text{ ksi}$
$d = 0.8 l_w = 21.2'$

$V_u = E_{wind} = V_{u1} + V_{u2} + V_{u3}$
$= 9 + 23 + V_{u3}$
$V_u = 252^k$

Check Max. Permitted Shear Strength $V_u < \phi V_n$

Try $t_w = 12''$

$\phi V_n = \phi 10 \sqrt{f'_c}\, t\, d$
$= 0.75(10)\sqrt{4000}(12)(21.2 \times 12)/1000 = 1448^k > 252^k$ ✓

Try $t_w = 8''$

$\phi V_n = 965^k$ ∴ ✓

Check Shear Strength provided by V_c

Critical Section at: $l_w/2 = 26.5/2 = 14.75'$
min $h_w/2 = 17/2 = \underline{8.5'}$ ← governs

$t = 12''$ $V_c = 2\sqrt{f'_c}\, t\, d$
$= 2\sqrt{4000}(12)(21.2 \times 12)/1000 = 386^k$

$t = 8''$ $V_c = 257^k$

Alex Burg | Proposed Solution | Shear Wall Design 3/4

Required Steel:

If $V_u > \frac{1}{2} \phi V_c$ According to ACI 11.9.9 use Horizontal Reinforcement

t=12"

$\frac{1}{2}(.75)(386) = \underline{145^k}$ ∴ Needs Horizontal Reinforcemen

t=8"

$\frac{1}{2}(.75)(257) = \underline{96}^k$ ∴ Needs Horizontal Reinforcement

$\underline{V_u \le \phi V_n = \phi(V_c + V_s)}$

t=12" $252 = (386 + V_s)0.75 \Rightarrow V_s = -50^k$ ∴

t=8" $V_s = 79^k$

t=12" $V_s = \frac{A_v \cdot f_y \cdot d}{s} \Rightarrow \frac{A_v}{s} = \frac{V_s}{f_y d} = \frac{-50}{60(2)\cdot 2 \times 12)} = 0.00327$

t=8" $V_s = 0.005$

t=8"

try (2)#3 $s = \frac{2(0.11)}{0.005} = 44"$

try (2)#3 @ 36"

$\rho_t = \frac{A_v}{s(t)} = \frac{2(0.11)}{36(8)} = 0.000764 < 0.0025$ ∴ NG

try (2)#3 @ 12"

$\rho_t = \frac{2(0.11)}{12(8)} = 0.0022 < 0.0025$ ∴ NG

try (2) #(3) @ 10"

$\rho_t = \frac{2(0.11)}{10(8)} = 0.00275 < 0.0025$ ∴ OK

Alex Burg | Proposed Solution | Shear Wall Design

Max Spacing:

$$s \le \begin{cases} l_w/5 = (26.5)(12)/5 = 63.6'' \\ 3t = 3(8) = 24'' \\ 18'' \leftarrow \text{governs} \end{cases} \quad \text{min}$$

$10'' < 18''$ ✓

$\boxed{\text{Use (2) \#3 @ 10'' for Horizontal Shear}}$

Vertical Shear Rienforcement

$$\rho_l = \frac{A_v}{s \cdot h} \ge 0.0025 + 0.5(2.5 - h_w/l_w)(\rho_t - 0.0025)$$

$$= 0.0025 + 0.5(2.5 - 17/26.5)(0.00275 - 0.0025)$$

$$= 0.0027 > 0.0025 \quad \therefore \text{ OK}$$

Max spacing:

$$\begin{cases} l_w/3 = 26.5(12)/3 = 106'' \\ 3(t) = 24'' \\ 18'' \leftarrow \text{governs} \end{cases} \quad \text{min}$$

$$\rho_l = \frac{A_v}{sh} = 0.00273 \Rightarrow s = \frac{A_v}{0.00273(t)}$$

try (2) #3 $\Rightarrow s = \frac{2(.11)}{0.00273(8)} = 10.2''$

$\boxed{\text{Use (2) \#3 @ 12'' for Vertical Shear}}$

Design for Flexure:

$$M_u = V_u(h_w) = 252^k(17') = 4{,}284^{k'}$$

★ Assume Tension Controlled ★

$$M_n = A_s f_y (d - a/2) = A_s f_y jd$$

$$C = T \Rightarrow 0.85 f'_c a b = A_s f_y \qquad jd = .9d = (.9)(21.2)(12) = 229''$$

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

$$4{,}284(12000) \le .9(A_s)(60000)(229)$$

$$A_s \ge 4.16 \text{ in}^2$$

$$.85(4000)(a)(8) = (4.16)(60000) \Rightarrow a = 9.18''$$

Alex Burg | Proposed Solution | Shear Wall Design 4/4

$jd = d - a/2$

$= (21.2 \times 12) - 9.18/2 = 249''$

$4{,}284(12000) = 0.9\, A_s (60{,}000)\, 249$

$A_s = 3.8\ in^2$

Try (4) #9 $\quad A_s = 4\ in^2$

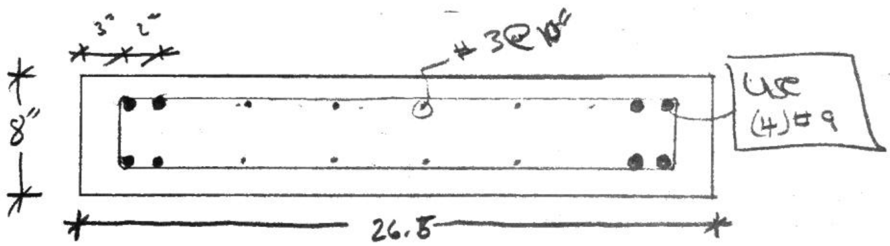
$C = T; \quad 0.85 f'_c a b = A_s f_y$

$$a = \frac{4(60)}{.85(4)(8)} = 8.8$$

$$c = a/\beta_1 = \frac{8.8}{.85} = 10.4''$$

$$\varepsilon_t = \varepsilon_u\left(\frac{d-c}{c}\right) = 0.003\left(\frac{(21.2 \times 12) - 10.4}{10.4}\right)$$

$= 0.07 > 0.005 \therefore$ OK ✓ Tension Controlled



Appendix 10: Moment Frame Design

	Floor:	1	/	7								
	f'c:	4	ksi	Clear Cover:	1.5	inch						
	fy:	60	ksi	Conc. Wieght:	150	pcf						
	Slab, t:	6.5	inch	Stirrup Size:	#	3	Stirrup Diameter:	0.357	inch			
	Misc. Dead Load:	35	psf	Bar Size:	#	8	Bar Diameter, d_b:	1.000	inch			
	Live Load:	80	psf	# of bars, n:	13		Area of Steel, A_b:	0.79	in^2			
	β_1:	0.85		ε_u:	0.003		Area of Steel, A_s:	10.27	in^3			
Girder Design, G2		(Lateral)					Tributary Area:	391	ft^2			
	Span:	29.5	feet	Two Row Reinforcement?	Yes		Influence Area:	1564	ft^2	Use Live Load Reduction		
	Spaced Left:	26.5	feet				L_L=Max of	0.4				
	Spaced Right:	0	feet					$.25+15/\sqrt{(K_{LL}*A_T)}$=		0.63	L_L:	50.35
Spacing,S	Max of d_b, 1", 3/4A_b			$b_{min}=2*Cc + n*d_b + 2d_{st} + (n-1)*S$								
d_b:	1.0	inch		b_{min}:	15.71	inch	Total Factored Weight, Wu					
3/4A_b:	0.6	inch					Dead Load: Misc. dead + Slab				Beam Wieght	
1":	1.0	inch		h_{min}>l/18.5 (ACI 318-08)					116	psf	141	plf
				h_{min}:	19.14	inch	Wu=1.2D +1.0W+L				Pu=1.2D	
S:	1.0	inch						190	psf		2236	plf
Try a												
b:	18	inch	OK		$d=h-d_b/2-Cc$			$a=A_s*fy/(.85*f'c*b_{eff})$			$c=a/\beta_1$	
h:	30	inch	OK		d:	27		a:	2.05	Rectangular Section	c:	2.41
b_{eff}= min	$b*16*h_f$	1872			$\varepsilon_t=\varepsilon_u(d-c)/c$							
	Trib width	318			ε_t:	0.03	Φ=0.9					
	.25L	88.5										
b_{eff}:	88.5	inch										
Check Flexure, ΦMn>Mu					Cheak Shear, Vn>Vu			Column width:	26	inch		
Mu=0.107*Ln^2+Pu*Ln/8, Etabs+continuous+point load					Vu=Wu*Ln/2+Pu/2							
Mu:	1182	kip-ft		Combo Controls	Vu:	75	Kip			Etabs Moment	973	kip-ft
ΦMn=0.9*$A_s$fy(d-a/2)					Vn=10*(f'c)^(1/2)*b*d							
ΦMn:	1200	kip-ft	OK		Vn:	341526	Kip	OK				
Use Member Size:		**18**	**x**	**30**								
	Beam Wieght:	441	plf									

	Floor:	2	/	7								
	f'c:	4	ksi	Clear Cover:	1.5	inch						
	fy:	60	ksi	Conc. Wieght:	150	pcf						
	Slab, t:	6.5	inch	Stirrup Size:	#	3	Stirrup Diameter:	0.357	inch			
	Misc. Dead Load:	35	psf	Bar Size:	#	8	Bar Diameter, d_b:	1.000	inch			
	Live Load:	80	psf	# of bars, n:	9		Area of Steel, A_b:	0.79	in^2			
	β_1:	0.85		ε_u:	0.003		Area of Steel, A_s:	7.11	in^3			
Girder Design, G2		(Lateral)					Tributary Area:	391	ft^2			
	Span:	29.5	feet	Two Row Reinforcement?	Yes		Influence Area:	1564	ft^2	Use Live Load Reduction		
	Spaced Left:	26.5	feet				L_i=Max of	0.4				
	Spaced Right:	0	feet					$.25+15/\sqrt{(K_{LL}*A_T)}$=		0.63	L_i:	50.35
Spacing,S	Max of d_b, 1", 3/4A_b			$b_{min}=2*Cc + n*d_b + 2d_{st} + (n-1)*S$								
d_b:	1.0	inch		b_{min}:	11.71	inch	Total Factored Weight, Wu					
3/4A_b:	0.6	inch					Dead Load: Misc. dead + Slab				Beam Wieght	
1":	1.0	inch		h_{min}>l/18.5 (ACI 318-08)				116	psf		141	plf
				h_{min}:	19.14	inch	Wu=1.2D +1.0W+L				Pu=1.2D	
S:	1.0	inch						190	psf		2236	plf
Try a												
b:	18	inch	OK		$d=h-d_b/2-Cc$			$a=A_s*fy/(.85*f'c*b_{eff})$			$c=a/\beta_1$	
h:	30	inch	OK		d:	27		a:	1.42	Rectangular Section	c:	1.67
b_{eff}= min	$b*16*h_f$	1872			$\varepsilon_t=\varepsilon_u(d-c)/c$							
	Trib width	318			ε_t:	0.05	Φ=0.9					
	.25L	88.5										
b_{eff}:	88.5	inch										
Check Flexure, ΦMn>Mu					Cheak Shear, Vn>Vu		Column width:		26	inch		
Mu=0.107*Ln^2+Pu*Ln/8, Etabs+continuous+point load					Vu=Wu*Ln/2+Pu/2							
Mu:	833	kip-ft		Combo Controls	Vu:	75	Kip			Etabs Moment	624	kip-ft
ΦMn=0.9*$A_s$fy(d-a/2)					Vn=10*(f'c)^(1/2)*b*d							
ΦMn:	841	kip-ft	OK		Vn:	341526	Kip	OK				
Use Member Size:		**18**	**x**	**30**								
	Beam Wieght:	441	plf									

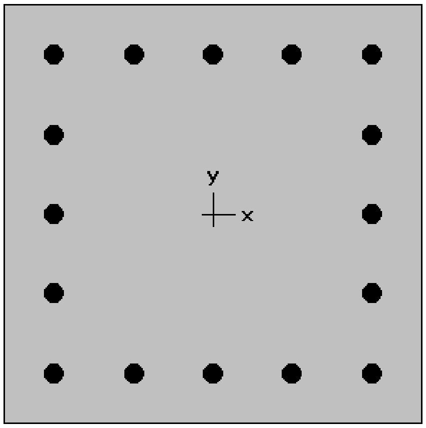
	Floor:	3	/	7								
	f'c:	4	ksi	Clear Cover:	1.5	inch						
	fy:	60	ksi	Conc. Wieght:	150	pcf						
	Slab, t:	6.5	inch	Stirrup Size:	#	3	Stirrup Diameter:	0.357	inch			
	Misc. Dead Load:	35	psf	Bar Size:	#	8	Bar Diameter, d_b:	1.000	inch			
	Live Load:	80	psf	# of bars, n:	7		Area of Steel, A_b:	0.79	in^2			
	β_1:	0.85		ε_u:	0.003		Area of Steel, A_s:	5.53	in^3			
Girder Design, G2		(Lateral)					Tributary Area:	391	ft^2			
	Span:	29.5	feet	Two Row Reinforcement?	Yes		Influence Area:	1564	ft^2	Use Live Load Reduction		
	Spaced Left:	26.5	feet				L_L=Max of	0.4				
	Spaced Right:	0	feet					$.25+15/\sqrt{(K_{LL}*A_T)}$=		0.63	L_L:	50.35
Spacing,S	Max of d_b, 1", 3/4A_b			$b_{min}=2*Cc + n*d_b + 2d_{st} + (n-1)*S$								
d_b:	1.0	inch		b_{min}:	9.71	inch	Total Factored Weight, Wu					
3/4A_b:	0.6	inch					Dead Load: Misc. dead + Slab				Beam Wieght	
1":	1.0	inch		h_{min}>l/18.5 (ACI 318-08)					116	psf	141	plf
				h_{min}:	19.14	inch	Wu=1.2D +1.0W+L				Pu=1.2D	
S:	1.0	inch						190	psf		2236	plf
Try a												
b:	18	inch	OK		$d=h-d_b/2-Cc$			$a=A_s*fy/(.85*f'c*b_{eff})$			$c=a/\beta_1$	
h:	30	inch	OK		d:	27		a:	1.10	Rectangular Section	c:	1.30
b_{eff}= min	$b*16*h_f$	1872			$\varepsilon_t=\varepsilon_u(d-c)/c$							
	Trib width	318			ε_t:	0.06	Φ=0.9					
	.25L	88.5										
b_{eff}:	88.5	inch										
Check Flexure, ΦMn>Mu					Cheak Shear, Vn>Vu		Column width:		26	inch		
Mu=0.107*Ln^2+Pu*Ln/8, Etabs+continuous+point load					Vu=Wu*Ln/2+Pu/2							
Mu:	616	kip-ft		Combo Controls	Vu:	75	Kip			Etabs Moment	407	kip-ft
ΦMn=0.9*$A_s$fy(d-a/2)					Vn=10*(f'c)^(1/2)*b*d							
ΦMn:	658	kip-ft	OK		Vn:	341526	Kip	OK				
Use Member Size:		**18**	**x**	**30**								
	Beam Wieght:	441	plf									

	Floor:	4	/	7								
	f'c:	4	ksi	Clear Cover:	1.5	inch						
	fy:	60	ksi	Conc. Wieght:	150	pcf						
	Slab, t:	6.5	inch	Stirrup Size:	#	3	Stirrup Diameter:	0.357	inch			
	Misc. Dead Load:	35	psf	Bar Size:	#	8	Bar Diameter, d_b:	1.000	inch			
	Live Load:	80	psf	# of bars, n:	5		Area of Steel, A_b:	0.79	in^2			
	β_1:	0.85		ε_u:	0.003		Area of Steel, A_s:	3.95	in^3			
Girder Design, G2		(Lateral)					Tributary Area:	391	ft^2			
	Span:	29.5	feet	Two Row	No		Influence Area:	1564	ft^2	Use Live Load Reduction		
	Spaced Left:	26.5	feet	Reinforcement?			L_r=Max of	0.4				
	Spaced Right:	0	feet					.25+15/√(K_{LL}*A_T)=		0.63	L_L:	50.35
Spacing,S	Max of d_b, 1", 3/4A_b			b_{min}=2*Cc + n*d_b + 2d_{st} + (n-1)*S								
d_b:	1.0	inch		b_{min}:	12.71	inch	Total Factored Weight, Wu					
3/4A_b:	0.6	inch					Dead Load: Misc. dead + Slab				Beam Wieght	
1":	1.0	inch		h_{min}>l/18.5 (ACI 318-08)					116	psf	141	plf
				h_{min}:	19.14	inch	Wu=1.2D +1.0W+L				Pu=1.2D	
S:	1.0	inch						190	psf		2236	plf
Try a												
b:	18	inch	OK		d=h-d_b/2-Cc			a=A_s*fy/(.85*f'c*b_{eff})			c=a/β_1	
h:	30	inch	OK		d:	28		a:	0.79	Rectangular Section	c:	0.93
b_{eff}= min	b*16*h_f	1872			ε_t=ε_u(d-c)/c							
	Trib width	318			ε_t:	0.09	Φ=0.9					
	.25L	88.5										
b_{eff}:	88.5	inch										
Check Flexure, ΦMn>Mu					Cheak Shear, Vn>Vu		Column width:		26	inch		
Mu=0.107*Ln^2+Pu*Ln/8, Etabs+continuous+point load					Vu=Wu*Ln/2+Pu/2							
Mu:	465	kip-ft		Combo Controls	Vu:	75	Kip			Etabs Moment	256	kip-ft
ΦMn=0.9*$A_s$fy(d-a/2)					Vn=10*(f'c)^(1/2)*b*d							
ΦMn:	491	kip-ft	OK		Vn:	341526	Kip	OK				
Use Member Size:		**18**	**x**	**30**								
	Beam Wieght:	441	plf									

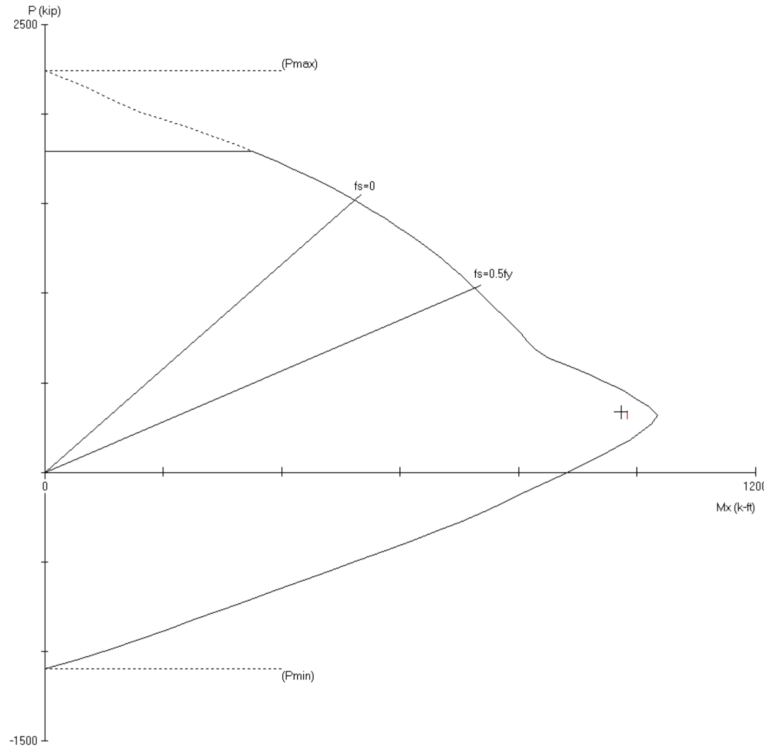
	Floor:	5	/	7								
	f'c:	4	ksi	Clear Cover:	1.5	inch						
	fy:	60	ksi	Conc. Wieght:	150	pcf						
	Slab, t:	6.5	inch	Stirrup Size:	#	3	Stirrup Diameter:	0.357	inch			
	Misc. Dead Load:	35	psf	Bar Size:	#	8	Bar Diameter, d_b:	1.000	inch			
	Live Load:	80	psf	# of bars, n:	4		Area of Steel, A_b:	0.79	in^2			
	β_1:	0.85		ε_u:	0.003		Area of Steel, A_s:	3.16	in^3			
Girder Design, G2		(Lateral)					Tributary Area:	391	ft^2			
	Span:	29.5	feet	Two Row Reinforcement?	No		Influence Area:	1564	ft^2	Use Live Load Reduction		
	Spaced Left:	26.5	feet				L_r=Max of	0.4				
	Spaced Right:	0	feet					.25+15/√(K_{LL}*A_T)=		0.63	L_r:	50.35
Spacing,S	Max of d_b, 1", 3/4A_b			b_{min}=2*Cc + n*d_b + 2d_{st} + (n-1)*S								
d_b:	1.0	inch		b_{min}:	10.71	inch	Total Factored Weight, Wu					
3/4A_b:	0.6	inch					Dead Load: Misc. dead + Slab				Beam Wieght	
1":	1.0	inch		h_{min}>l/18.5 (ACI 318-08)					116	psf	141	plf
				h_{min}:	19.14	inch		Wu=1.2D +1.0W+L			Pu=1.2D	
S:	1.0	inch						190	psf		2236	plf
Try a												
b:	18	inch	OK		d=h-d_b/2-Cc			a=A_s*fy/(.85*f'c*b_{eff})			c=a/β_1	
h:	30	inch	OK		d:	28		a:	0.63	Rectangular Section	c:	0.74
b_{eff}= min	b*16*h_f	1872			ε_t=ε_u(d-c)/c							
	Trib width	318			ε_t:	0.11	Φ=0.9					
	.25L	88.5										
b_{eff}:	88.5	inch										
Check Flexure, ΦMn>Mu					Cheak Shear, Vn>Vu		Column width:		26	inch		
Mu=0.107*Ln^2+Pu*Ln/8, Etabs+continuous+point load					Vu=Wu*Ln/2+Pu/2							
Mu:	357	kip-ft		Combo Controls	Vu:	75	Kip			Etabs Moment	148	kip-ft
ΦMn=0.9*$A_s$fy(d-a/2)					Vn=10*(f'c)^(1/2)*b*d							
ΦMn:	394	kip-ft	OK		Vn:	341526	Kip	OK				
Use Member Size:		**18**	**x**	**30**								
	Beam Wieght:	441	plf									

	Floor:	6	/	7								
	f'c:	4	ksi	Clear Cover:	1.5	inch						
	fy:	60	ksi	Conc. Wieght:	150	pcf						
	Slab, t:	6.5	inch	Stirrup Size:	#	3	Stirrup Diameter:	0.357	inch			
Misc. Dead Load:		35	psf	Bar Size:	#	8	Bar Diameter, d_b:	1.000	inch			
	Live Load:	80	psf	# of bars, n:	3		Area of Steel, A_b:	0.79	in^2			
	β_1:	0.85		ε_u:	0.003		Area of Steel, A_s:	2.37	in^3			
Girder Design, G2		(Lateral)					Tributary Area:	391	ft^2			
	Span:	29.5	feet	Two Row Reinforcement?	No		Influence Area:	1564	ft^2	Use Live Load Reduction		
	Spaced Left:	26.5	feet				L_L=Max of	0.4				
	Spaced Right:	0	feet					$.25+15/\sqrt{(K_{LL}*A_T)}$=		0.63	L_L:	50.35
Spacing,S	Max of d_b, 1", $3/4A_b$			$b_{min}=2*Cc+n*d_b+2d_{st}+(n-1)*S$								
d_b:	1.0	inch		b_{min}:	8.71	inch	Total Factored Weight, Wu					
$3/4A_b$:	0.6	inch					Dead Load: Misc. dead + Slab				Beam Wieght	
1":	1.0	inch		h_{min}>l/18.5 (ACI 318-08)					116	psf	141	plf
				h_{min}:	19.14	inch	Wu=1.2D +1.0W+L				Pu=1.2D	
S:	1.0	inch						190	psf		2236	plf
Try a												
b:	18	inch	OK		d=$h-d_b/2$-Cc			$a=A_s*fy/(.85*f'c*b_{eff})$			$c=a/\beta_1$	
h:	30	inch	OK		d:	28		a:	0.47	Rectangular Section	c:	0.56
b_{eff}= min	$b*16*h_f$	1872			$\varepsilon_t=\varepsilon_u(d-c)/c$							
	Trib width	318			ε_t:	0.15	Φ=0.9					
	.25L	88.5										
b_{eff}:	88.5	inch										
Check Flexure, ΦMn>Mu					Cheak Shear, Vn>Vu		Column width:		26	inch		
Mu=0.107*Ln^2+Pu*Ln/8, Etabs+continuous+point load					Vu=Wu*Ln/2+Pu/2							
Mu:	279	kip-ft		Combo Controls	Vu:	75	Kip			Etabs Moment	70	kip-ft
ΦMn=0.9*$A_s$fy(d-a/2)					Vn=10*(f'c)^(1/2)*b*d							
ΦMn:	296	kip-ft	OK		Vn:	341526	Kip	OK				
Use Member Size:		**18**	**x**	**30**								
	Beam Wieght:	441	plf									

Floor:	7	/	7									
f'c:	4	ksi		Clear Cover:	1.5	inch						
fy:	60	ksi		Conc. Wieght:	150	pcf						
Slab, t:	6.5	inch		Stirrup Size:	#	3	Stirrup Diameter:	0.357	inch			
Misc. Dead Load:	35	psf		Bar Size:	#	8	Bar Diameter, d_b:	1.000	inch			
Live Load:	80	psf		# of bars, n:	3		Area of Steel, A_b:	0.79	in^2			
β_1:	0.85			ε_u:	0.003		Area of Steel, A_s:	2.37	in^3			
Girder Design, G2	(Lateral)						Tributary Area:	391	ft^2			
Span:	29.5	feet		Two Row Reinforcement?	No		Influence Area:	1564	ft^2	Use Live Load Reduction		
Spaced Left:	26.5	feet					L_r=Max of	0.4				
Spaced Right:	0	feet						$.25+15/\sqrt{(K_{LL}*A_T)}$=		0.63	L_r:	50.35
Spacing,S	Max of d_b, 1", $3/4A_b$			$b_{min}=2*Cc+n*d_b+2d_{st}+(n-1)*S$								
d_b:	1.0	inch		b_{min}:	8.71	inch	Total Factored Weight, Wu					
$3/4A_b$:	0.6	inch					Dead Load: Misc. dead + Slab				Beam Wieght	
1":	1.0	inch		h_{min}>l/18.5 (ACI 318-08)					116	psf	141	plf
				h_{min}:	19.14	inch	Wu=1.2D +1.0W+L				Pu=1.2D	
S:	1.0	inch						190	psf		2236	plf
Try a												
b:	18	inch	OK		d=h-d_b/2-Cc			a=A_s*fy/(.85*f'c*b_{eff})			c=a/β_1	
h:	30	inch	OK		d:	28		a:	0.47	Rectangular Section	c:	0.56
b_{eff}= min	b*16*h_f	1872			$\varepsilon_t=\varepsilon_u(d-c)/c$							
	Trib width	318			ε_t:	0.15	Φ=0.9					
	.25L	88.5										
b_{eff}:	88.5	inch										
Check Flexure, ΦMn>Mu					Cheak Shear, Vn>Vu		Column width:		26	inch		
Mu=0.107*Ln^2+Pu*Ln/8, Etabs+continuous+point load					Vu=Wu*Ln/2+Pu/2							
Mu:	253	kip-ft		Combo Controls	Vu:	75	Kip			Etabs Moment	44	kip-ft
ΦMn=0.9*$A_s$fy(d-a/2)					Vn=10*(f'c)^(1/2)*b*d							
ΦMn:	296	kip-ft	OK		Vn:	341526	Kip	OK				
Use Member Size:	18	x	30									
Beam Wieght:	441	plf										
					Total Reinforcment Wieght in Lateral Girders:				309.7	Ton		



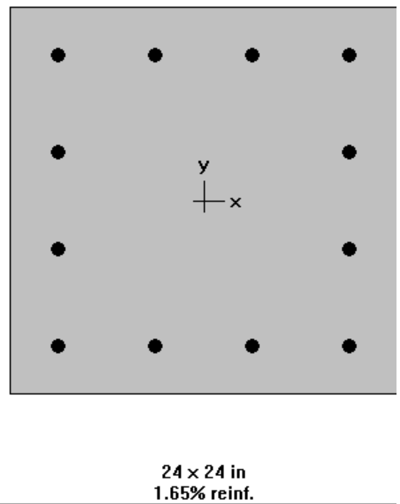
26 x 26 in
3.01% reinf.

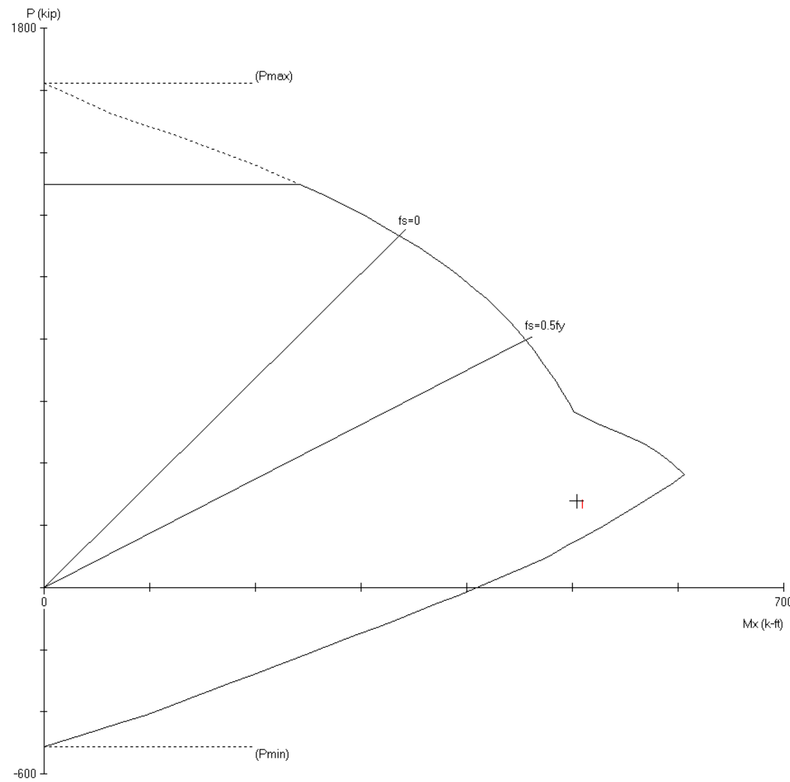


MATERIAL:
=========
f'c = 4 ksi
Ec = 3605 ksi
fc = 3.4 ksi
Beta1 = 0.85
fy = 60 ksi
Es = 29000 ksi

SECTION:
========
Ag = 676 in^2
Ix = 38081.3 in^4
Iy = 38081.3 in^4
Xo = 0 in
Yo = 0 in

REINFORCEMENT:
==============
16 #10 bars @ 3.006%
As = 20.32 in^2
Confinement: Tied
Clear Cover = 2.50 in
Min Clear Spacing = 3.66 in

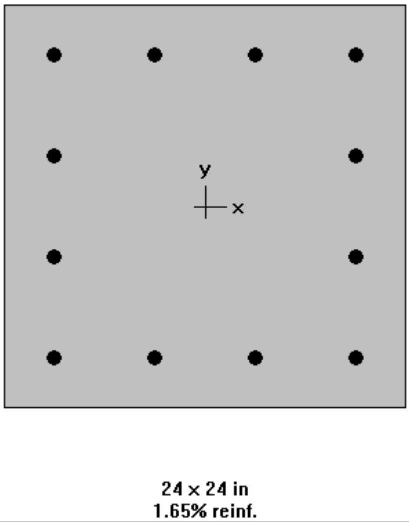


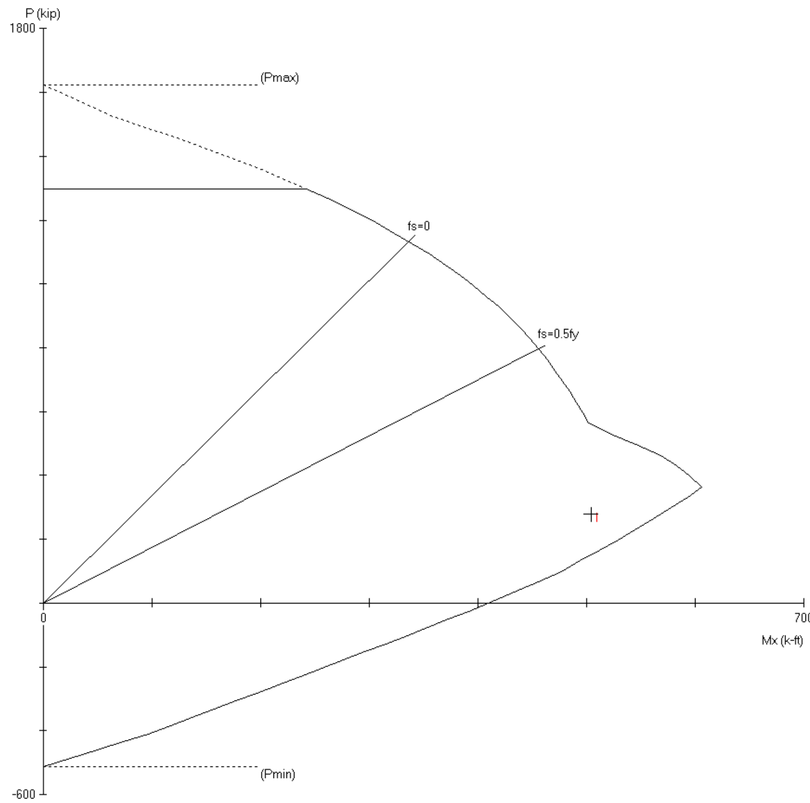


MATERIAL:
=========
f'c = 4 ksi
Ec = 3605 ksi
fc = 3.4 ksi
Beta1 = 0.85
fy = 60 ksi
Es = 29000 ksi

SECTION:
========
Ag = 576 in^2
Ix = 27648 in^4
Iy = 27648 in^4
Xo = 0 in
Yo = 0 in

REINFORCEMENT:
==============
12 #8 bars @ 1.646%
As = 9.48 in^2
Confinement: Tied
Clear Cover = 2.50 in
Min Clear Spacing = 5.00 in

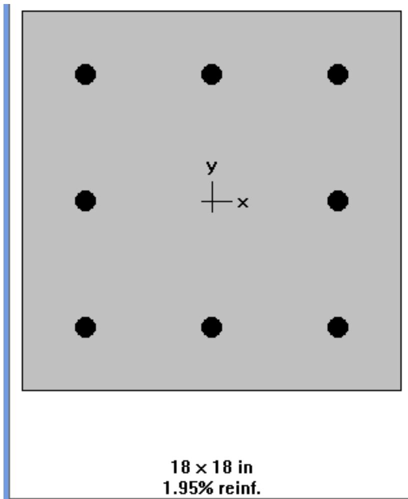




MATERIAL:
=========
f'c = 4 ksi
Ec = 3605 ksi
fc = 3.4 ksi
Beta1 = 0.85
fy = 60 ksi
Es = 29000 ksi

SECTION:
========
Ag = 576 in^2
Ix = 27648 in^4
Iy = 27648 in^4
Xo = 0 in
Yo = 0 in

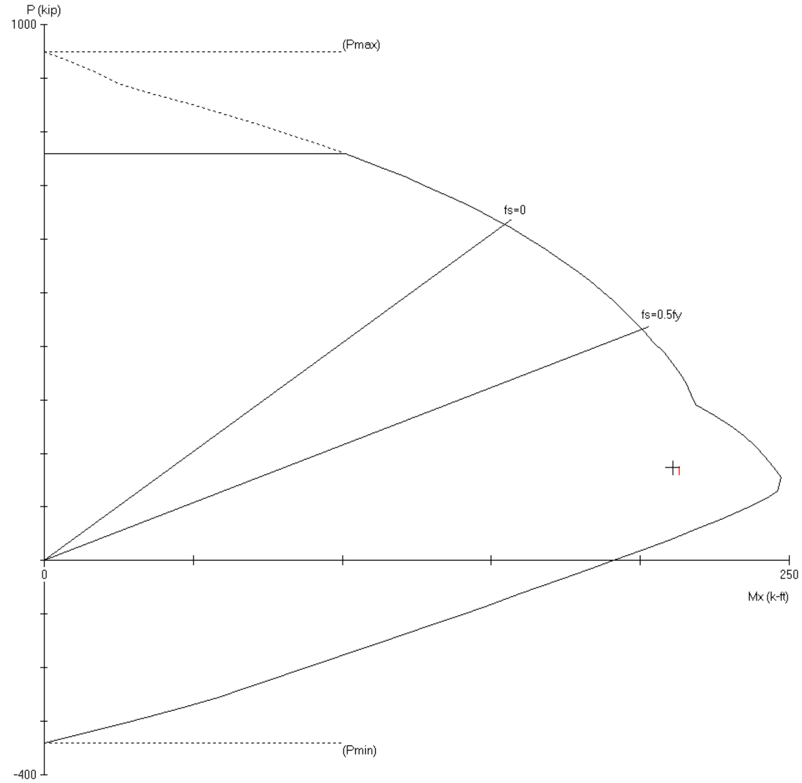
REINFORCEMENT:
===============
12 #8 bars @ 1.646%
As = 9.48 in^2
Confinement: Tied
Clear Cover = 2.50 in
Min Clear Spacing = 5.00 in

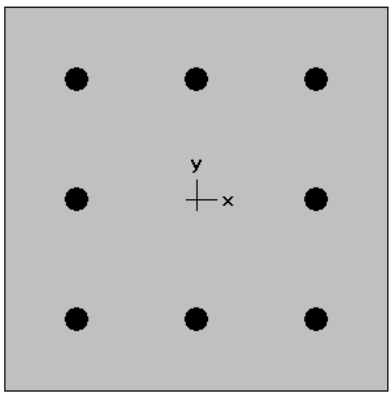


MATERIAL:
=========
f'c = 4 ksi
Ec = 3605 ksi
fc = 3.4 ksi
Beta1 = 0.85
fy = 60 ksi
Es = 29000 ksi

SECTION:
========
Ag = 324 in^2
Ix = 8748 in^4
Iy = 8748 in^4
Xo = 0 in
Yo = 0 in

REINFORCEMENT:
==============
8 #8 bars @ 1.951%
As = 6.32 in^2
Confinement: Tied
Clear Cover = 2.50 in
Min Clear Spacing = 5.00 in



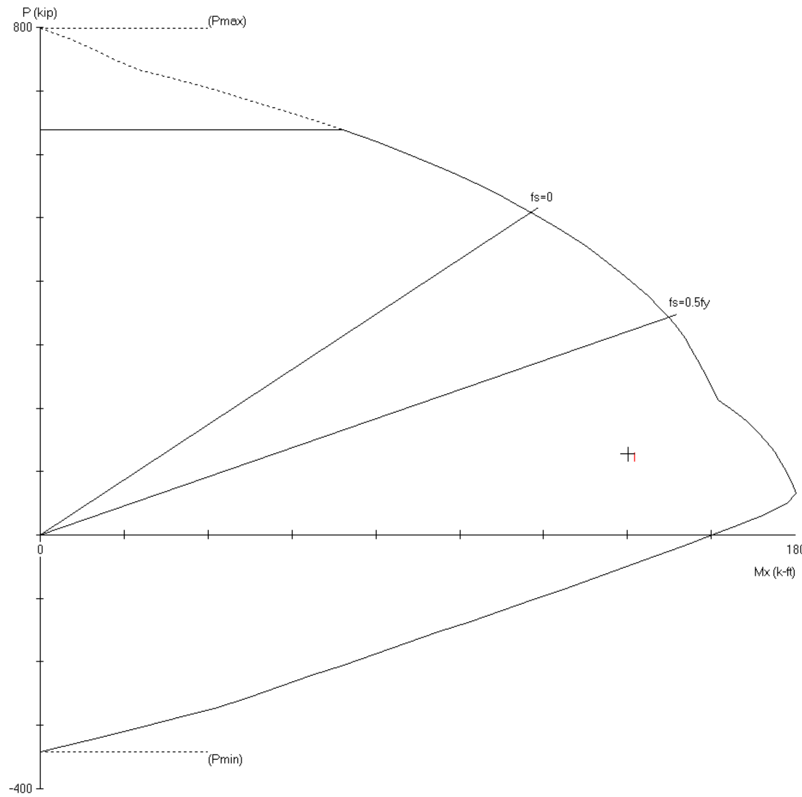


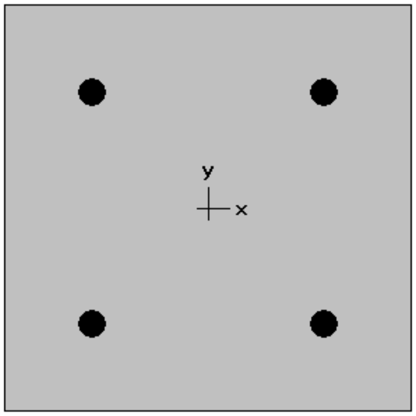
16 x 16 in
2.47% reinf.

MATERIAL:
=========
f'c = 4 ksi
Ec = 3605 ksi
fc = 3.4 ksi
Beta1 = 0.85
fy = 60 ksi
Es = 29000 ksi

SECTION:
========
Ag = 256 in^2
Ix = 5461.33 in^4
Iy = 5461.33 in^4
Xo = 0 in
Yo = 0 in

REINFORCEMENT:
==============
8 #8 bars @ 2.469%
As = 6.32 in^2
Confinement: Tied
Clear Cover = 2.50 in
Min Clear Spacing = 4.00 in



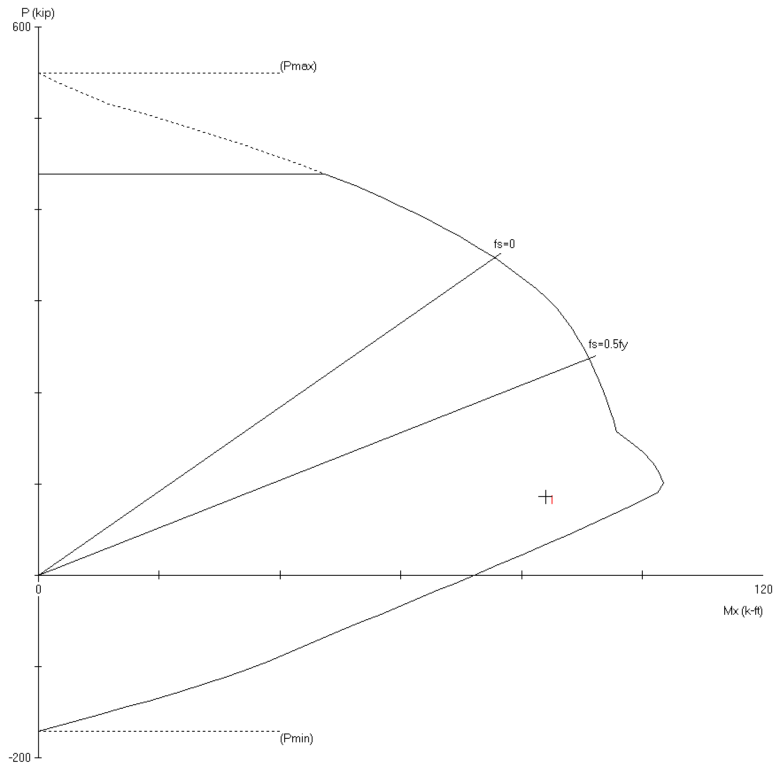


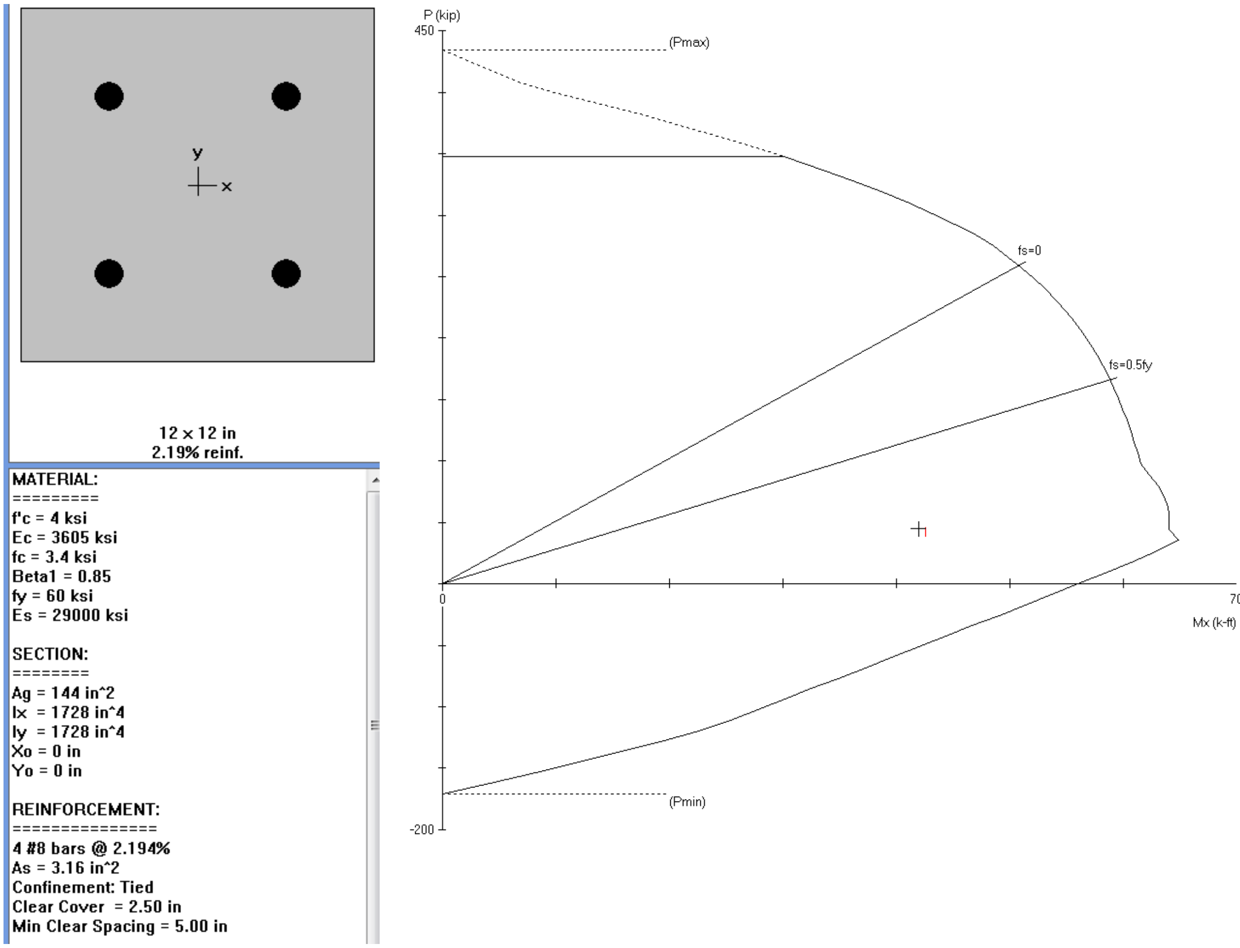
14 x 14 in
1.61% reinf.

MATERIAL:
=========
f'c = 4 ksi
Ec = 3605 ksi
fc = 3.4 ksi
Beta1 = 0.85
fy = 60 ksi
Es = 29000 ksi

SECTION:
========
Ag = 196 in^2
Ix = 3201.33 in^4
Iy = 3201.33 in^4
Xo = 0 in
Yo = 0 in

REINFORCEMENT:
==============
4 #8 bars @ 1.612%
As = 3.16 in^2
Confinement: Tied
Clear Cover = 2.50 in
Min Clear Spacing = 7.00 in




y
x
12 x 12 in
2.19% reinf.
MATERIAL:
=========
f'c = 4 ksi
Ec = 3605 ksi
fc = 3.4 ksi
Beta1 = 0.85
fy = 60 ksi
Es = 29000 ksi
SECTION:
========
Ag = 144 in^2
Ix = 1728 in^4
Iy = 1728 in^4
Xo = 0 in
Yo = 0 in
REINFORCEMENT:
===============
4 #8 bars @ 2.194%
As = 3.16 in^2
Confinement: Tied
Clear Cover = 2.50 in
Min Clear Spacing = 5.00 in
P (kip)
450
(Pmax)
fs=0
fs=0.5fy
0
70
Mx (k-ft)
(Pmin)
-200

Appendix 11: Cost Analysis

Existing Steel Structure

	Project Number/Item		Size	Unit	Material	Labor	Equipment	Total	Total Incl. O&P	Location	Amount	Total(No O&P)	Total (w/ O&P)
Steel Decking	05 31 13.50	5900	18 Gauge	S.F.	$ 1.80	$ 0.40	$ 0.05	$ 2.25	$ 2.80	All Stories	306,894.00	$ 690,511.50	$ 859,303.20
Deck FireProofing	07 81 16.10	100	1" thick	S.F.	$ 0.53	$ 0.22	$ 0.04	$ 0.79	$ 0.99	All Stories	306,894.00	$ 242,446.26	$ 303,825.06
3" Slab Pumped	03 31 15.70	1400	pumped	C.Y.	-	$ 12.50	$ 5.70	$ 18.20	$ 27.50	All Stories	2,838.77	$ 51,665.60	$ 78,066.16
4000psi Concrete	03 31 05.35	300	3" Slab	C.Y.	$ 103.00	-	-	$ 103.00	$ 113.00	All Stories	2,838.77	$ 292,393.26	$ 320,780.95
Concrete Finish	03 35 29.30	125	Bull Float	S.F.	-	$ 0.35	-	$ 0.35	$ 0.57	All Stories	306,894.00	$ 107,412.90	$ 174,929.58
Curb Edging	05 12 23.12	1501	12" Channel	L.F.	$ 28.00	$ 7.40	-	$ 35.40	$ 43.50	All Stories	1,377.00	$ 48,745.80	$ 59,899.50
Steel Beam	05 12 23.75	1300	W12x19	L.F.	$ 22.89	$ 1.93	$ 1.83	$ 26.65	$ 30.64	All Stories	7,560.00	$ 201,446.51	$ 231,663.49
Beam FireProofing	07 81 16.10	400	1" thick	S.F.	$ 0.53	$ 0.43	$ 0.09	$ 1.05	$ 1.39	All Stories	30,240.00	$ 31,752.00	$ 42,033.60
Steel Beam	05 12 23.75	1300	W16x26	L.F.	$ 31.50	$ 1.70	$ 1.61	$ 34.81	$ 39.50	All Stories	22,292.50	$ 776,001.93	$ 880,553.75
Beam FireProofing	07 81 16.10	400	1" thick	S.F.	$ 0.53	$ 0.43	$ 0.09	$ 1.05	$ 1.39	All Stories	89,170.00	$ 93,628.50	$ 123,946.30
Steel Girder	05 12 23.75	4900	W24x55	L.F.	$ 66.50	$ 2.29	$ 1.58	$ 70.37	$ 79.00	All Stories	14,840.00	$ 1,044,290.80	$ 1,172,360.00
Girder FireProofing	07 81 16.10	400	1" thick	S.F.	$ 0.53	$ 0.43	$ 0.09	$ 1.05	$ 1.39	All Stories	118,720.00	$ 124,656.00	$ 165,020.80
Steel Column	05 12 23.17	7350	W14x99	L.F.	$ 89.50	$ 1.72	$ 1.63	$ 92.85	$ 104.00	Top 2 Stories	3,744.00	$ 347,630.40	$ 389,376.00
Column FireProofing	07 81 16.10	800	1" thick	S.F.	$ 1.13	$ 0.93	$ 0.19	$ 2.25	$ 2.98	All Stories	17,472.00	$ 39,312.00	$ 52,066.56
Steel Column	05 12 23.17	7400	W14x120	L.F.	$ 145.00	$ 1.77	$ 1.67	$ 148.44	$ 165.00	Mid 2 Stories	3,744.00	$ 555,759.36	$ 617,760.00
Column FireProofing	07 81 16.10	800	1" thick	S.F.	$ 1.13	$ 0.93	$ 0.19	$ 2.25	$ 2.98	All Stories	17,472.00	$ 39,312.00	$ 52,066.56
Steel Column	05 12 23.17	7450	W14x176	L.F.	$ 213.00	$ 1.86	$ 1.76	$ 216.62	$ 239.00	Bot 2 Stories	3,430.00	$ 743,006.60	$ 819,770.00
Column FireProofing	07 81 16.10	800	1" thick	S.F.	$ 1.13	$ 0.93	$ 0.19	$ 2.25	$ 2.98	All Stories	16,006.67	$ 36,015.00	$ 47,699.87
												$ 5,972,968.56	**$ 7,030,233.51**

Location Factor:	1.1
Floor Area	
43842	ft^2
Building Area	
306894	ft^2
Concrete Volume	
2,838.77	C.Y.

Proposed Concrete Structure

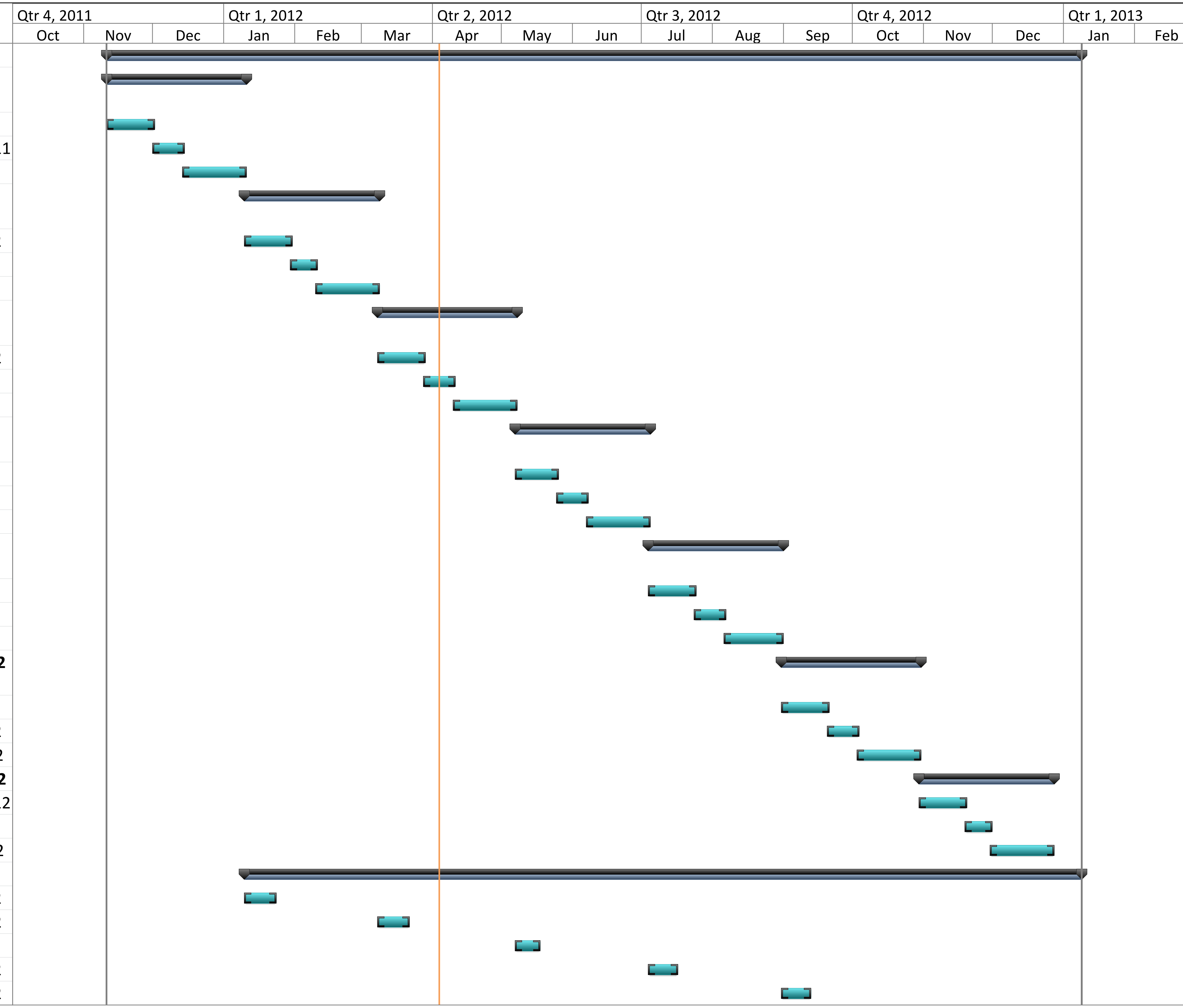
	Project Number/Item		Size	Unit	Material	Labor	Equipment	Total	Total Incl. O&P	Location	Amount	Total(No O&P)	Total (w/ O&P)
4000psi Concrete	03 31 05.35	300		C.Y.	$ 103.00	-	-	$ 103.00	$ 113.00	All Stories	16,740.67	$ 1,724,288.73	$ 1,891,695.40
Concrete Finish	03 35 29.30	125	Bull Float	S.F.	-	$ 0.35	-	$ 0.35	$ 0.57	All Stories	43,842.00	$ 15,344.70	$ 24,989.94
Concrete Slab	03 31 05.70	1400	6.5" Slab	C.Y.	-	$ 10.95	$ 5.00	$ 15.95	$ 23.50	All Stories	16,740.67	$ 267,013.64	$ 393,405.68
Slab Reinforcing	03 21 10.60	400		Ton	$ 850.00	$ 385.00	-	$ 1,235.00	$ 1,625.00	All Stories	445.00	$ 549,575.00	$ 723,125.00
Slab Form	03 11 13.35	1150	4 use	SFCA	$ 1.32	$ 2.48	-	$ 3.80	$ 5.60	All Stories	306,894.00	$ 1,166,197.20	$ 1,718,606.40
Edge Form	03 11 13.35	7000	4 use	L.F.	$ 0.12	$ 1.84	-	$ 1.96	$ 3.22	All Stories	9,639.00	$ 18,892.44	$ 31,037.58
Concrete Beam	03 31 05.70	200	10x20	C.Y.	-	$ 19.45	$ 8.90	$ 28.35	$ 42.50	All Stories	4,142.00	$ 117,425.70	$ 176,035.00
Beam Reinforcing	03 21 10.60	150		Ton	$ 800.00	$ 415.00	-	$ 1,215.00	$ 1,600.00	All Stories	132.00	$ 160,380.00	$ 211,200.00
Beam Form	03 11 13.20	1150	4 use	SFCA	$ 1.09	$ 3.08	-	$ 4.17	$ 6.38	All Stories	16,567.00	$ 69,029.17	$ 105,614.63
Concrete Girder	03 31 05.70	200	10x20	C.Y.	-	$ 19.45	$ 8.90	$ 28.35	$ 42.50	All Stories	1,639.00	$ 46,465.65	$ 69,657.50
G1 Reinforcing	03 21 10.60	150		Ton	$ 800.00	$ 415.00	-	$ 1,215.00	$ 1,600.00	All Stories	77.00	$ 93,555.00	$ 123,200.00
G1 Form	03 11 13.20	1150	4 use	SFCA	$ 1.09	$ 3.08	-	$ 4.17	$ 6.38	All Stories	7,204.00	$ 30,016.67	$ 45,925.50
Concrete Girder	03 31 05.70	200	18x30	C.Y.	-	$ 19.45	$ 8.90	$ 28.35	$ 42.50	All Stories	4,425.00	$ 125,448.75	$ 188,062.50
G2 Reinforcing	03 21 10.60	150		Ton	$ 800.00	$ 415.00	-	$ 1,215.00	$ 1,600.00	All Stories	310.00	$ 376,650.00	$ 496,000.00
G2 Form	03 11 13.20	1150	4 use	SFCA	$ 0.91	$ 4.41	-	$ 5.32	$ 8.40	All Stories	12,968.00	$ 68,989.76	$ 108,931.20
Concrete Column	03 31 05.70	800	20x20	C.Y.	-	$ 19.05	$ 8.70	$ 27.75	$ 41.00	Top 2 Stories	384.00	$ 10,656.00	$ 15,744.00
C1 Reinforcing	03 21 10.60	250		Ton	$ 1,175.00	$ 510.00	-	$ 1,685.00	$ 2,175.00	All Stories	94.00	$ 158,390.00	$ 204,450.00
C1 Form	03 11 13.25	6650	4 use	SFCA	$ 0.62	$ 3.22	-	$ 3.83	$ 6.08	All Stories	13,347.00	$ 51,163.50	$ 81,194.25
Concrete Column	03 31 05.70	800	24x24-14x14	C.Y.	-	$ 15.88	$ 14.50	$ 30.38	$ 34.17	Top 2 Stories	425.00	$ 12,909.38	$ 14,520.83
C2 Reinforcing	03 21 10.60	400		Ton	$ 850.00	$ 385.00	-	$ 1,235.00	$ 1,625.00	All Stories	85.00	$ 104,975.00	$ 138,125.00
C2 Form	03 11 13.25	6650	4 use	SFCA	$ 0.74	$ 3.86	-	$ 4.60	$ 7.30	All Stories	16,016.00	$ 73,673.60	$ 116,916.80
Concrete Shear Wall	03 31 05.70	800	8" Thick	C.Y.	-	$ 14.25	$ 0.68	$ 14.93	$ 24.50	All Stories	357.00	$ 5,330.01	$ 8,746.50
Wall Reinforcing	03 21 10.60	400		Ton	$ 760.00	$ 281.00	-	$ 1,041.00	$ 1,325.00	All Stories	94.00	$ 97,854.00	$ 124,550.00
Wall Form	03 11 13.85	2550	4 use	SFCA	$ 0.59	$ 3.52		$ 4.11	$ 6.55	All Stories	14,469.00	$ 59,467.59	$ 94,771.95
												$ 5,878,646.28	**$ 7,817,156.22**

Location Factor:	1.1
Floor Area	
43842	ft^2
Building Area	
306894	ft^2
Concrete Volume	
16,740.67	C.Y.

Difference in Cost	$ (94,322.28)	$ 786,922.71

Appendix 12: Schedule Analysis

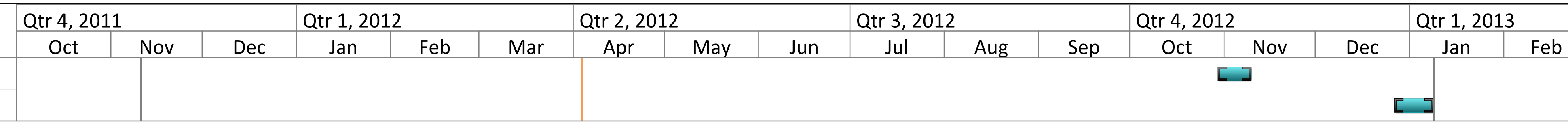
ID		Task Mode	Task Name	Duration	Start	Finish
1			**Steel Superstructure**	**303 days**	**Fri 11/11/11**	**Tue 1/8/13**
2			**1st Floor Steel Structure**	**43 days**	**Fri 11/11/11**	**Tue 1/10/12**
3			Set Steel	15 days	Fri 11/11/11	Thu 12/1/11
4			Detail Steel	10 days	Thu 12/1/11	Wed 12/14/11
5			Install Decking	20 days	Wed 12/14/11	Tue 1/10/12
6			**2nd Floor Steel Structure**	**43 days**	**Tue 1/10/12**	**Thu 3/8/12**
7			Set Steel	15 days	Tue 1/10/12	Mon 1/30/12
8			Detail Steel	10 days	Mon 1/30/12	Fri 2/10/12
9			Install Decking	20 days	Fri 2/10/12	Thu 3/8/12
10			**3rd Floor Steel Structure**	**43 days**	**Thu 3/8/12**	**Mon 5/7/12**
11			Set Steel	15 days	Thu 3/8/12	Wed 3/28/12
12			Detail Steel	10 days	Wed 3/28/12	Tue 4/10/12
13			Install Decking	20 days	Tue 4/10/12	Mon 5/7/12
14			**4th Floor Steel Structure**	**43 days**	**Mon 5/7/12**	**Wed 7/4/12**
15			Set Steel	15 days	Mon 5/7/12	Fri 5/25/12
16			Detail Steel	10 days	Fri 5/25/12	Thu 6/7/12
17			Install Decking	20 days	Thu 6/7/12	Wed 7/4/12
18			**5th Floor Steel Structure**	**43 days**	**Wed 7/4/12**	**Fri 8/31/12**
19			Set Steel	15 days	Wed 7/4/12	Tue 7/24/12
20			Detail Steel	10 days	Tue 7/24/12	Mon 8/6/12
21			Install Decking	20 days	Mon 8/6/12	Fri 8/31/12
22			**6th Floor Steel Structure**	**43 days**	**Fri 8/31/12**	**Tue 10/30/12**
23			Set Steel	15 days	Fri 8/31/12	Thu 9/20/12
24			Detail Steel	10 days	Thu 9/20/12	Wed 10/3/12
25			Install Decking	20 days	Wed 10/3/12	Tue 10/30/12
26			**Roof Steel Structure**	**43 days**	**Tue 10/30/12**	**Thu 12/27/12**
27			Set Steel	15 days	Tue 10/30/12	Mon 11/19/12
28			Detail Steel	10 days	Mon 11/19/12	Fri 11/30/12
29			Install Decking	20 days	Fri 11/30/12	Thu 12/27/12
30			**Concrete Pour**	**261 days**	**Tue 1/10/12**	**Tue 1/8/13**
31			Pour 1st Floor	10 days	Tue 1/10/12	Mon 1/23/12
32			Pour 2nd Floor	10 days	Thu 3/8/12	Wed 3/21/12
33			Pour 3rd Floor	9 days	Mon 5/7/12	Thu 5/17/12
34			Pour 4th Floor	9 days	Wed 7/4/12	Mon 7/16/12
35			Pour 5th Floor	9 days	Fri 8/31/12	Wed 9/12/12



Project: Existing Schedule
Date: Wed 4/4/12

Task	Project Summary	Inactive Milestone	Manual Summary Rollup
Split	External Tasks	Inactive Summary	Manual Summary
Milestone	External Milestone	Manual Task	Start-only
Summary	Inactive Task	Duration-only	Finish-only
Deadline	Progress		

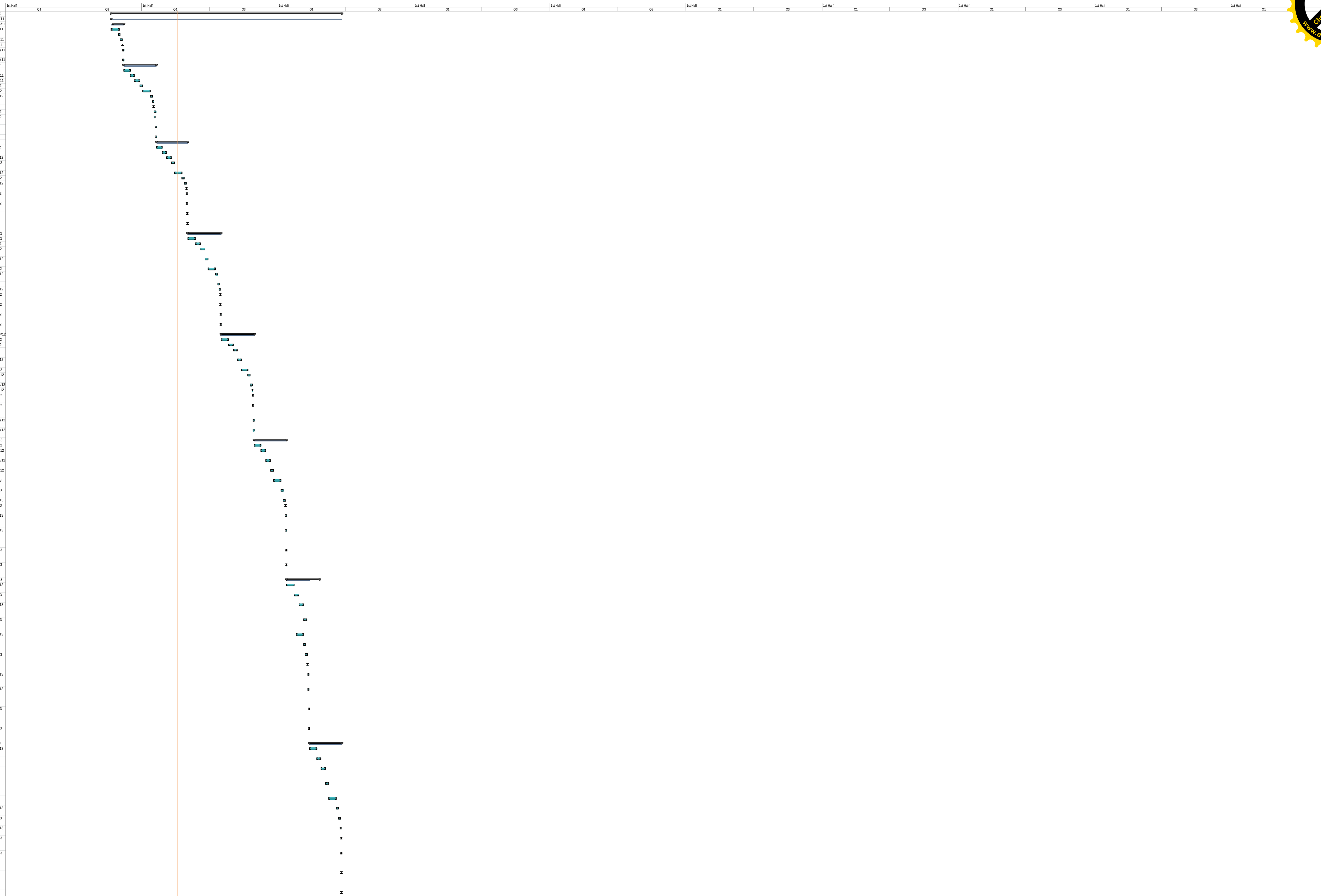
ID		Task Mode	Task Name	Duration	Start	Finish
36			Pour 6th Floor	9 days	Tue 10/30/12	Fri 11/9/12
37			Pour Roof	9 days	Thu 12/27/12	Tue 1/8/13



Project: Existing Schedule
Date: Wed 4/4/12

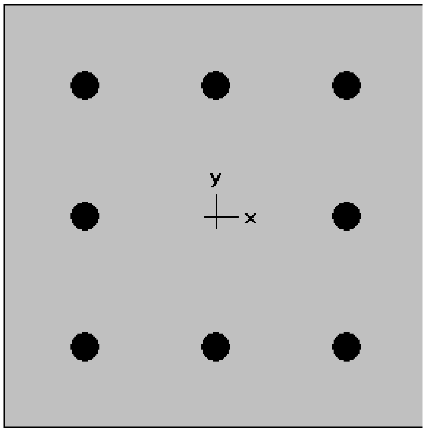
Task	Project Summary	Inactive Milestone	Manual Summary Rollup	Deadline
Split	External Tasks	Inactive Summary	Manual Summary	Progress
Milestone	External Milestone	Manual Task	Start-only	
Summary	Inactive Task	Duration-only	Finish-only	

ID	Task Mode	Task Name	Duration	Start	Finish
1		Conc...erStructure	444 days	Tue 10/11/11	Fri 6/21/13
2		...uperStructure	1 day	Tue 10/11/11	Tue 10/11/11
3		...nd Floor	21 days	Mon 10/17/11	Mon 11/14/11
4		...me Columns	17 days	Tue 10/11/11	Wed 11/2/11
		Reinforce Columns	5 days	Mon 10/31/11	Fri 11/4/11
		Frame Walls	5 days	Fri 11/4/11	Thu 11/10/11
		Reinforce Walls	2 days	Thu 11/10/11	Fri 11/11/11
		Place Concrete in Columns	2 days	Fri 11/11/11	Mon 11/14/11
9		Place Concrete in Walls	2 days	Fri 11/11/11	Mon 11/14/11
10		First Floor	64 days	Mon 11/14/11	Thu 2/9/12
11		Frame Slab	15 days	Mon 11/14/11	Fri 12/2/11
12		Rienforce Slab	9 days	Thu 12/1/11	Tue 12/13/11
13		Frame Beam & Girder	12 days	Mon 12/12/11	Tue 12/27/11
14		Rienforce Beam & Girder	7 days	Tue 12/27/11	Wed 1/4/12
15		Frame Columns	15 days	Wed 1/4/12	Tue 1/24/12
16		Reinforce Columns	5 days	Tue 1/24/12	Mon 1/30/12
17		Frame Walls	5 days	Mon 1/30/12	Fri 2/3/12
18		Reinforce Walls	2 days	Thu 2/2/12	Fri 2/3/12
19		Place Concrete in Slab	4 days	Fri 2/3/12	Wed 2/8/12
20		Place Concrete in Beams & Girder	2 days	Fri 2/3/12	Mon 2/6/12
21		Place Concrete in Columns	2 days	Wed 2/8/12	Thu 2/9/12
22		Place Concrete in Walls	2 days	Wed 2/8/12	Thu 2/9/12
23		Second Floor	61 days	Fri 2/10/12	Fri 5/4/12
24		Frame Slab	12 days	Fri 2/10/12	Sat 2/25/12
25		Rienforce Slab	10 days	Sat 2/25/12	Thu 3/8/12
26		Frame Beam & Girder	10 days	Thu 3/8/12	Wed 3/21/12
27		Rienforce Beam & Girder	7 days	Wed 3/21/12	Thu 3/29/12
28		Frame Columns	15 days	Thu 3/29/12	Wed 4/18/12
29		Reinforce Columns	5 days	Wed 4/18/12	Tue 4/24/12
30		Frame Walls	5 days	Tue 4/24/12	Mon 4/30/12
31		Reinforce Walls	2 days	Mon 4/30/12	Tue 5/1/12
32		Place Concrete in Slab	2 days	Tue 5/1/12	Wed 5/2/12
33		Place Concrete in Beams & Girder	2 days	Tue 5/1/12	Wed 5/2/12
34		Place Concrete in Columns	2 days	Wed 5/2/12	Thu 5/3/12
35		Place Concrete in Walls	2 days	Thu 5/3/12	Fri 5/4/12
36		Third Floor	64 days	Fri 5/4/12	Wed 8/1/12
37		Frame Slab	15 days	Fri 5/4/12	Thu 5/24/12
38		Rienforce Slab	10 days	Thu 5/24/12	Wed 6/6/12
39		Frame Beam & Girder	10 days	Wed 6/6/12	Tue 6/19/12
40		Rienforce Beam & Girder	7 days	Tue 6/19/12	Wed 6/27/12
41		Frame Columns	15 days	Wed 6/27/12	Tue 7/17/12
42		Reinforce Columns	5 days	Tue 7/17/12	Mon 7/23/12
43		Frame Walls	5 days	Mon 7/23/12	Fri 7/27/12
44		Reinforce Walls	2 days	Fri 7/27/12	Mon 7/30/12
45		Place Concrete in Slab	2 days	Mon 7/30/12	Tue 7/31/12
46		Place Concrete in Beams & Girder	2 days	Mon 7/30/12	Tue 7/31/12
47		Place Concrete in Columns	2 days	Tue 7/31/12	Wed 8/1/12
48		Place Concrete in Walls	2 days	Tue 7/31/12	Wed 8/1/12
49		Fourth Floor	64 days	Wed 8/1/12	Mon 10/29/12
50		Frame Slab	15 days	Wed 8/1/12	Tue 8/21/12
51		Rienforce Slab	10 days	Tue 8/21/12	Mon 9/3/12
52		Frame Beam & Girder	10 days	Mon 9/3/12	Fri 9/14/12
53		Rienforce Beam & Girder	7 days	Fri 9/14/12	Mon 9/24/12
54		Frame Columns	15 days	Mon 9/24/12	Fri 10/12/12
55		Reinforce Columns	5 days	Fri 10/12/12	Thu 10/18/12
56		Frame Walls	5 days	Thu 10/18/12	Wed 10/24/12
57		Reinforce Walls	2 days	Wed 10/24/12	Thu 10/25/12
58		Place Concrete in Slab	2 days	Thu 10/25/12	Fri 10/26/12
59		Place Concrete in Beams & Girder	2 days	Thu 10/25/12	Fri 10/26/12
60		Place Concrete in Columns	2 days	Fri 10/26/12	Mon 10/29/12
61		Place Concrete in Walls	2 days	Fri 10/26/12	Mon 10/29/12
62		Fifth Floor	64 days	Mon 10/29/12	Thu 1/24/13
63		Frame Slab	15 days	Mon 10/29/12	Fri 11/16/12
64		Rienforce Slab	10 days	Fri 11/16/12	Thu 11/29/12
65		Frame Beam & Girder	10 days	Thu 11/29/12	Wed 12/12/12
66		Rienforce Beam & Girder	7 days	Wed 12/12/12	Thu 12/20/12
67		Frame Columns	15 days	Thu 12/20/12	Wed 1/9/13
68		Reinforce Columns	5 days	Wed 1/9/13	Tue 1/15/13
69		Frame Walls	5 days	Tue 1/15/13	Mon 1/21/13
70		Reinforce Walls	2 days	Mon 1/21/13	Tue 1/22/13
71		Place Concrete in Slab	2 days	Tue 1/22/13	Wed 1/23/13
72		Place Concrete in Beams & Girder	2 days	Tue 1/22/13	Wed 1/23/13
73		Place Concrete in Columns	2 days	Wed 1/23/13	Thu 1/24/13
74		Place Concrete in Walls	2 days	Wed 1/23/13	Thu 1/24/13
75		Sixth Floor	64 days	Thu 1/24/13	Tue 4/23/13
76		Frame Slab	15 days	Thu 1/24/13	Wed 2/13/13
77		Rienforce Slab	10 days	Wed 2/13/13	Tue 2/26/13
78		Frame Beam & Girder	10 days	Tue 2/26/13	Mon 3/11/13
79		Rienforce Beam & Girder	7 days	Mon 3/11/13	Tue 3/19/13
80		Frame Columns	15 days	Tue 2/19/13	Mon 3/11/13
81		Reinforce Columns	5 days	Mon 3/11/13	Fri 3/15/13
82		Frame Walls	5 days	Fri 3/15/13	Thu 3/21/13
83		Reinforce Walls	2 days	Thu 3/21/13	Fri 3/22/13
84		Place Concrete in Slab	2 days	Fri 3/22/13	Mon 3/25/13
85		Place Concrete in Beams & Girder	2 days	Fri 3/22/13	Mon 3/25/13
86		Place Concrete in Columns	2 days	Mon 3/25/13	Tue 3/26/13
87		Place Concrete in Walls	2 days	Mon 3/25/13	Tue 3/26/13
88		Roof	64 days	Tue 3/26/13	Fri 6/21/13
89		Frame Slab	15 days	Tue 3/26/13	Mon 4/15/13
90		Rienforce Slab	10 days	Mon 4/15/13	Fri 4/26/13
91		Frame Beam & Girder	10 days	Fri 4/26/13	Thu 5/9/13
92		Rienforce Beam & Girder	7 days	Thu 5/9/13	Fri 5/17/13
93		Frame Columns	15 days	Fri 5/17/13	Thu 6/6/13
94		Reinforce Columns	5 days	Thu 6/6/13	Wed 6/12/13
95		Frame Walls	5 days	Wed 6/12/13	Tue 6/18/13
96		Reinforce Walls	2 days	Tue 6/18/13	Wed 6/19/13
97		Place Concrete in Slab	2 days	Wed 6/19/13	Thu 6/20/13
98		Place Concrete in Beams & Girder	2 days	Wed 6/19/13	Thu 6/20/13
99		Place Concrete in Columns	2 days	Thu 6/20/13	Fri 6/21/13
100		Place Concrete in Walls	2 days	Thu 6/20/13	Fri 6/21/13

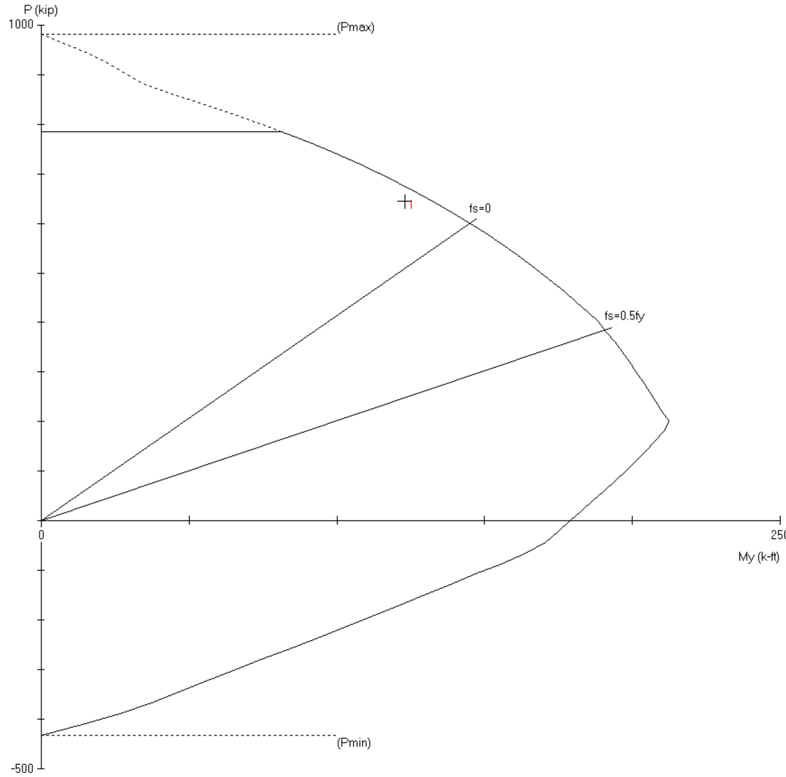


Appendix 13: Green Roof Structure

	f'c:	4	ksi	Clear Cover:	1.5	inch						
	fy:	60	ksi	Conc. Wieght:	150	pcf						
	Slab, t:	6.5	inch	Stirrup Size:	#	3	Diameter:	0.357	inch			
	Misc. Dead Load:	45	psf	Bar Size:	#	8	ameter, d_b:	1.000	inch			
	Live Load:	100	psf	# of bars, n:	4		Area of Steel, A_b:	0.79	in^2			
	β_1:	0.85		ε_u:	0.003		Area of Steel, A_s:	3.16	in^2			
Green Roof Beam Design							Tributary Area:	295	ft^2			
	Span:	20	feet	Two Row Reinforcement?	Yes		Influence Area:	590	ft^2	Live Load Reduction		
	Spaced:	14.75	feet				L_L=Max of	0.4				
								$.25+15/\sqrt{(K_{LL}*A_T)}$=		0.87	L_L:	86.75
Spacing,S:	Max of d_b, 1", $3/4A_b$			$b_{min}=2*Cc+n*d_b+2d_{st}+(n-1)*S$								
	d_b:	1.0	inch	b_{min}:	6.71	inch				Total Factored Weight, Wu		
	$3/4A_b$:	0.6	inch							Dead Load: Misc. dead + Slab		
	1":	1.0	inch	h_{min}>l/18.5 (ACI 318-08)		able 9.5 min h>l/18.5					126.25	psf
				h_{min}:	12.97	inch	min h:	13.0		Wu=1.2D +1.6L		
	S:	1.0	inch							290.306	psf	
Try a:												
	b:	10	inch	OK	$d=h-d_b/2-Cc$			$a=A_s*fy/(.85*f'c*b_{eff})$			$c=a/\beta_1$	
	h:	20	inch	OK	d:	17		a:	0.93	Rectangular	c:	1.09
b_{eff}= min	$b*16*h_f$	1040			$\varepsilon_t=\varepsilon_u(d-c)/c$							
	Trib widt	177			ε_t:	0.04	Φ=0.9					
	.25L	60										
	b_{eff}:	60	inch									
Check Flexure, ΦMn>Mu					Cheak Shear, Vn>Vu		Girder width, b:		12	inch		
Mu=Wu*Ln^2/8					Vu=Wu*Ln/2							
	Mu:	193.2	kip-ft		Vu:	42.8	Kip					
$\Phi Mn=0.9*A_s fy(d-a/2)$					Vn=10*(f'c)^(1/2)*b*d							
	ΦMn:	235.1	kip-ft	OK	Vn:	126491	Kip	OK				
Use Member Size:		**10**	x	**20**								
	Beam Wieght:	141	plf									



16 x 16 in
3.13% reinf.



MATERIAL:
=========
f'c = 4 ksi
Ec = 3605 ksi
fc = 3.4 ksi
Beta1 = 0.85
fy = 60 ksi
Es = 29000 ksi

SECTION:
========
Ag = 256 in^2
Ix = 5461.33 in^4
Iy = 5461.33 in^4
Xo = 0 in
Yo = 0 in

REINFORCEMENT:
==============
8 #9 bars @ 3.125%
As = 8 in^2
Confinement: Tied
Clear Cover = 2.50 in
Min Clear Spacing = 3.81 in

	f'c:	4	ksi		Clear Cover:	1.5	inch						
	fy:	60	ksi		Conc. Wieght:	150	pcf						
	Slab, t:	6.5	inch		Stirrup Size:	#	3	Diameter:	0.357	inch			
	Misc. Dead Load:	45	psf		Bar Size:	#	8	ameter, d_b:	1.000	inch			
	Live Load:	100	psf		# of bars, n:	7		Area of Steel, A_b:	0.79	in^2			
	β_1:	0.85			ε_u:	0.003		Area of Steel, A_s:	5.53	in^2			
Green Roof Girder Design													
								Tributary Area:	590	ft^2			
	Span:	29.5	feet		Two Row Reinforcement?	Yes		Influence Area:	1180	ft^2	Live Load Reduction		
	Spaced:	20	feet					L_L=Max of	0.4				
									$.25+15/\sqrt{(K_{LL}*A_T)}$=		0.69	L_L:	68.67
Spacing,S:	Max of d_b, 1", 3/4A_b				b_{min} =2*Cc + n*d_b + 2d_{st} + (n-1)*S								
	d_b:	1.0	inch		b_{min}:	9.71	inch				Total Factored Weight, Wu		
	3/4A_b:	0.6	inch								Dead Load: Misc. dead + Slab		
	1":	1.0	inch		h_{min}>l/18.5 (ACI 318-08)		able 9.5 min h>l/18.5					126.25	psf
					h_{min}:	19.14	inch	min h:	19.1		Wu=1.2D +1.6L		
	S:	1.0	inch								261.367	psf	
Try a:													
	b:	18	inch	OK		d=h-d_b/2-Cc			a=A_s*fy/(.85*f'c*b_{eff})			c=a/β_1	
	h:	30	inch	OK		d:	27		a:	1.10	Rectangular	c:	1.30
b_{eff}= min	b*16*h_f	1872				ε_t=ε_u(d-c)/c							
	Trib width	240				ε_t:	0.06	Φ=0.9					
	.25L	88.5											
	b_{eff}:	88.5	inch										
Check Flexure, ΦMn>Mu					Cheak Shear, Vn>Vu			Girder width, b:		0	inch		
Mu=Wu*Ln^2/8					Vu=Wu*Ln/2								
	Mu:	568.6	kip-ft		Vu:	77.1	Kip						
ΦMn=0.9*$A_s$fy(d-a/2)					Vn=10*(f'c)^(1/2)*b*d								
	ΦMn:	658.2	kip-ft	OK	Vn:	341526	Kip	OK					
Use Member Size:		18	x	30									
	Beam Wieght:	441	plf										

Appendix 14: LEED References

Table I. Functional Unit—30% Green Roof Replacement on Typical Urban Building Stock

Building type	Number of households	Conditioned space per household (sq ft)	Average number of floors	Annual energy use (Mill BTU/HH resid; kBTU/sf comm)	Total roof area (1,000 sq ft)	Total replaced roofing (1,000 sq ft)
Single-family detached	3,000	2,500	1.5	59	5,000	1,500
Single-family attached	500	1,800	2	59	450	140
Multifamily, 2–4 units	500	800	3	51	130	40
Multifamily, >5 units	1,400	700	5	18	200	60
Commercial	—	3,400	5	57	680	200

Units	\$/MT	\$/kWh	\$/kgal
Market value	\$21.47	\$0.0982	\$2.27
Reference	Capoor and Ambrosi (2008)[6]	Energy Information Administration (2009)[18]	Fisher et al. (2008)[19]

Table IV. Costs, Energy Used, and GHGs Released from Producing and Replacing 30% of Existing Roofs with Green Roofs in a Typical Urban Neighborhood over 30 Years

Building type	Roofing replaced (1,000 sq ft)	Private costs (\$1,000)			Energy used (MWh)		GHGs released (MT CO2 eq)	
		Materials	Construction	Total	Materials	Construction	Materials	Construction
Single family	1,600	(\$5,100)	(\$7,600)	(\$13,000)	(59)	(0.41)	(19,000)	(3,000)
Multifamily	100	(\$690)	(\$690)	(\$1,400)	(5.9)	(0.042)	(1,800)	(270)
Commercial	200	(\$1,400)	(\$2,200)	(\$3,600)	(15)	(0.14)	(4,600)	(840)
All	1,900	(\$7,200)	(\$10,000)	(\$17,000)	(79)	(0.59)	(25,000)	(4,100)

Table V. Reduced Electricity Use from Green Roof Installation over a 30-Year Planning Horizon

Building type	Electricity use reductions (MWh)				Private benefits	Public benefits
	Private	Social				
	Direct energy savings	UHI energy savings	CSO energy savings	Total	Market value of energy savings (\$1,000)	Market value of energy savings (\$1,000)
Single family	4,700	67,000	530	67,000	\$210	\$7,200
Multifamily	790	11,000	32	11,000	\$34	\$1,200
Commercial	3,500	28,000	67	28,000	\$150	\$3,100
All categories	9,100	110,000	640	110,000	\$390	\$12,000

Table VI. Greenhouse Gas Reductions from Green Roof Installation over a 30-Year Planning Horizon

Building type	GHG reductions (MT CO2 eq)					Public benefits
	Direct energy mitigation	UHI energy mitigation	CSO energy mitigation	Sequestered	Total GHG mitigation	Market value of CO2 mitigated (\$1,000)
Single family	3,300	47,000	370	390	51,000	\$630
Multifamily	530	7,700	20	24	8,300	\$130
Commercial	2,300	19,000	46	49	21,000	\$340
All categories	6,100	74,000	436	470	81,000	\$1,100