Oklahoma University Children's Medical Office Building Final Report

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Structural

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Oklahoma University Children's Medical Office Building

Jonathan Ebersole | Structural Option
http://www.engr.psu.edu/ae/thesis/portfolios/2014/jme5193/index.html

Project Team

- Owner: University Hospitals Trust
- Construction Manager: Flintco, Inc.
- Project Architect: Miles Associates
- Design Architect: Hellmuth, Obata, and Kassabaun, Inc.
- Structural Engineer: Zahl-Ford
- MEP Engineer: ZRHD, P.C.
- Civil Engineer: Smith, Roberts, Baldischwiler, Inc.

General Information

- Location: 1200 North Children's Avenue, Oklahoma City, Oklahoma
- Occupancy: Office
- Size: 320,000 gsf
- Height: 12 Stories for a total of 172 ft.
- Construction Dates: February 2007-Spring of 2009
- Building Cost: $59,760,000
- Delivery Method: Design-Bid-Build

Architecture

- Exterior Façade comprised of brick Veneer with large glass curtain wall on the front face of the building
- Supports Hospital with additional office space, exam rooms, and labs
- Membrane roof system with rigid insulation and light weight insulating concrete

Structural Design

- Reinforced concrete columns and beams
- 10" thick flat slab system with drop panels
- Concrete shear walls located in elevator shafts and stairwells
- Drilled pier foundation with a minimum bearing capacity of 45 KSF

Mechanical Design

- 7,500 CFM Air Handling unit occupies each floor
- Heat Exchanger is used to heat water before entering the heating coil
- 850 CFM fans are used to pressurize the stairwells

Lighting/Electrical Design

- Service voltage is 480/277 V, three phase, with 4 wires
- Voltage reduced to 120/208V, three phase, with 4 wires and supplied to each panel box
- Fluorescent lamps are used throughout the building to save energy costs

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Executive Summary

Oklahoma University Children's Medical Office Building is an office building located in Oklahoma. It is situated next to an existing hospital and parking garage. The building houses offices, examination rooms and labs for the expanding OU Children's Hospital. It is the largest free standing clinical office in the state and provides much needed medical services to the children of Oklahoma and their families. The building is twelve stories tall for a total of 170 feet and is approximately 320,000 psf.

The structure of the building is reinforced concrete. The building uses a flat slab system supported by columns and exterior beams. Drop panels are used at the column locations to provide extra shear and moment capacity to the slab. The columns are supported on drilled piers that transfer the loads to bedrock underneath the building. The building also uses shear walls and moment frames to resist the lateral forces.

This building provides several unique challenges that a typical office building would not otherwise have. These include a parking garage located on the first floor, a future helicopter pad positioned on the roof, and impact loads on lower levels for vehicle collisions with the building. These design parameters will increase the difficulty of future design assignments as all load cases must be analyzed.

The redesign of the structural system uses composite steel wide flanges and girders with composite decking. The roof is designed as k-series joists spanning between wide flange girders. The lateral system is comprised of concentric braced frames located in the existing shear wall locations. Additional moment frames where incorporated to increase the stiffness of the system. These frames are located along the eastern façade wall. Due to the architectural layout of the windows and floor plan, these frames could not be designed as braced frames.

With buildings becoming more energy efficient, a green roof breadth was conducted to study the vegetation and materials that are involved with a typical assembly. Hardy, succulent plants called sedums where chosen as the vegetation. These plants are typically used on green roofs since they tolerate droughts and can survive in a wide variety of climates. The materials where selected based on their durability, performance, and energy efficiency.

A cost and schedule analysis was also conducted to determine the cost difference and schedule impacts between a cast-in-place reinforced concrete system and a composite steel system. From the analysis, it was concluded that the steel system was more cost effective and less time consuming than the original concrete system.

Acknowledgements

I would like to thank the building owner, University Hospital Trust, for the use of their building. I would also like to thank Miles Associates and Zahl-Ford for providing me with the building drawings and specifications. I would like to thank the AE faculty for their dedication and guidance. Finally I would like to thank my friends and family for their continued support.

Introduction

Building Description

Oklahoma University Children's Medical Office Building is located on 1200 N. Children's Avenue Oklahoma City, Oklahoma between Stanton L. Young Blvd and N.E. 13th Street. Figure 1 shows the building's location and orientation on the site, highlighted in red. The building is twelve stories above grade and is approximately 170 feet tall. Miles Associates, Inc. designed the building for the University Hospitals Trust to provide additional medical offices for the expanding Oklahoma University Children's Hospital next door. The building is the first free-standing, multi-specialty physicians' office building in the state that will meet the needs of the children of Oklahoma as well as their families.

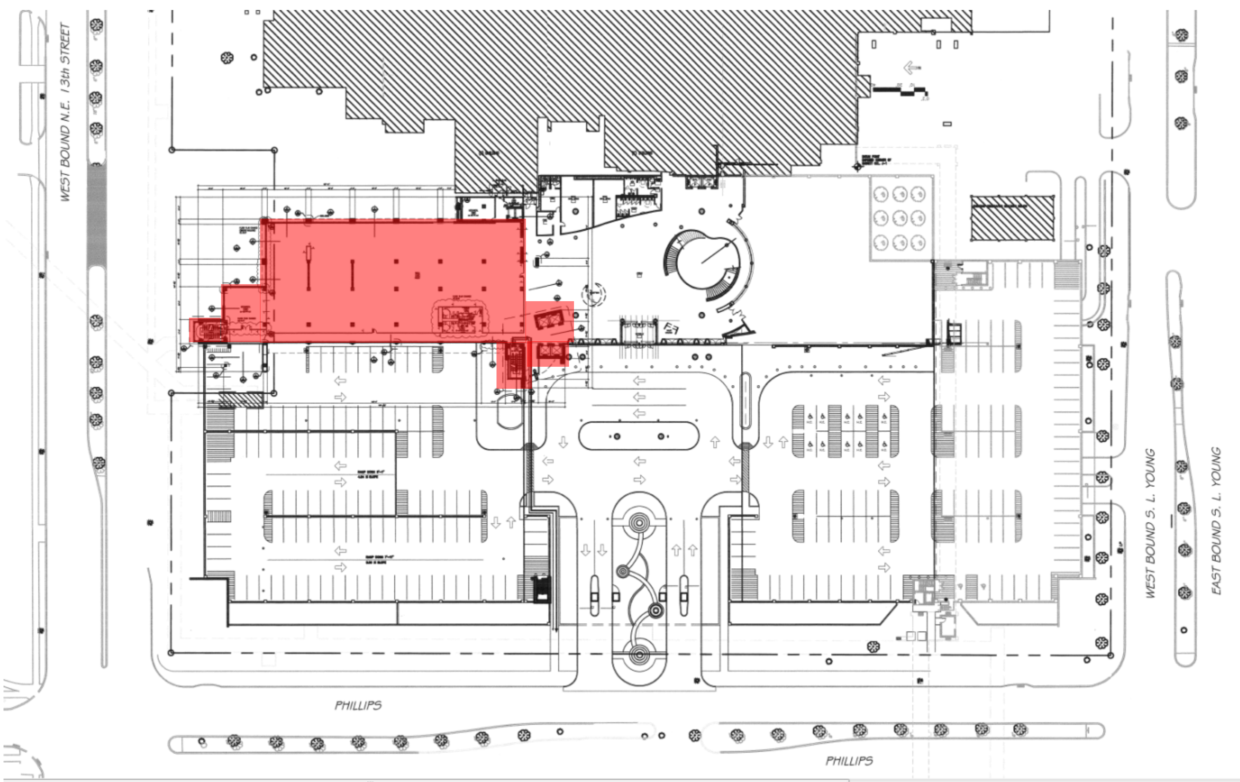


Figure 1. Building's location on the site.

The typical floor plan includes various necessities to the Oklahoma University Children's Medical Office Building that are not typical for other office buildings, such as exam rooms, x-ray rooms, and labs. However, there are similar rooms to the typical office building which include offices, storage rooms, waiting areas, and conference rooms. One challenge that this building possesses is a parking garage occupies the first floor of the building. This challenge will be especially difficult structurally as the basement is to be occupied by offices, exam rooms, and work areas. The typical floor is set up so that an elevator lobby and stairwell is located at the Southwest corner of the building as shown in red in Figure 2. The lobby leads to a waiting and reception area on the western face of the building and eventually to a stairwell at the northwest corner, shown in yellow. The waiting areas branch out to four rows of corridors with offices and exam rooms on each side, shown in blue. Finally shown in green, the corridors lead to another corridor on the eastern side of the building that runs parallel to the waiting areas. A conference room, shown in purple, is located at the northwest corner of the building. Each floor also contains its own mechanical rooms, shown in brown. This layout allows the building to be easily navigable for new guest and emergency situations. The floor layouts do not differ widely as most have the layout as described above. The only changes are the room types and sizes.

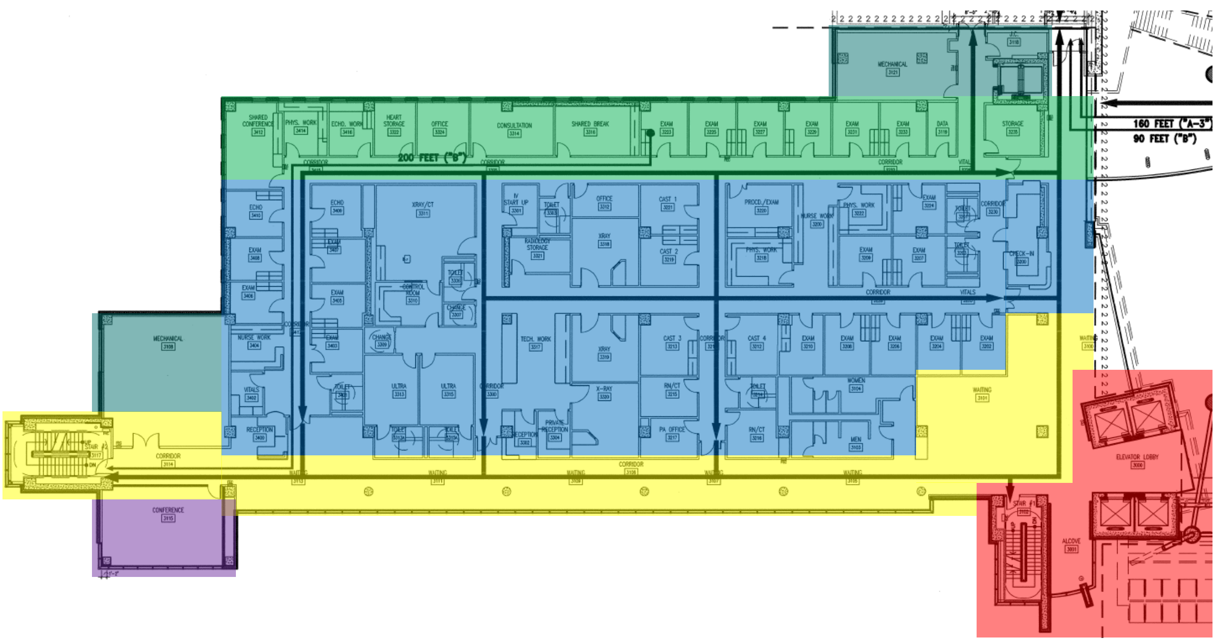


Figure 2. Floor plan showing typical rooms and layout.

Structural Framing System

In order to design a safe, functional building, the designers must review the codes to determine the appropriate loading conditions and standards to design the building by. Once the loads are determined, the designer must then understand how to transfer these loads into the ground. Next, the designer can analyze the structure to develop the appropriate sizes for the foundation, columns, beams, and slabs. An analysis for the lateral loads must be completed and the most efficient lateral system must be chosen. Finally the connections and reinforcement must be detailed.

Codes

Since the building was in the design phase in 2006, most of the newer updates of the codes were not released yet. The structural designers instead used the 2003 International Building Code, ASCE 7-02, and the ACI 318-02 codes. The International Building Code describes the live load cases and the general practices a designer should use while designing a building, but does not detail proper procedures for a structural analysis. ASCE 7-02 is used to determine the proper procedures for wind and seismic design. ASCE 7-02 also has factored load cases for dead and live loads as well as snow loads. Since the building is constructed from reinforced concrete, ACI 318-02 provides the proper procedures for designing concrete structures.

Loading

The primary gravity resisting system of the building is the columns, beams, and slabs. This system resists loading that are separated into three categories which are live load, dead load, and snow load. All three of these categories are loads that result from the force of gravity acting on the structure. Live loads are loads that are produced by the use and occupancy of the building. These loads include people and furniture. The loading can vary depending on the occupancy of the building and the room type. The Oklahoma University Children's Medical Office Building is designed as an office occupancy with interior partitions. The International Building Code requires that the minimum loading for an office building is 50 psi for the live loads, however; the designed loading is based on an 80 psf corridor loading to allow flexibility in the floor plan layout. The code standard for the corridor live load is then added to the 20 psf allowance for interior partitions for a total of 100 psf. The stairs and exits are also designed at 100 psf due to a higher occupancy for emergencies, while the mechanical rooms and electric rooms are designed at 125 psf. The roof also sees a live load for maintenance which is a minimum of 20 psf.

Dead loads consist of the weight of all the construction materials that are incorporated into the building. These are much easier to design for as they are known or can be easily approximated. Dead loads include the weight of the structure itself and the weight of mechanical equipment and lights. Typical dead loads are about 2 psf for ceilings and 10 psf for the duct systems, just to name a few.

The third load category is snow loads. This type of loading is caused by snow lying on the roof of the structure. Based on ASCE 7-02, the snow load calculations will be determined based on the criteria of a flat roof since the slope of the roof is less than 5°. ASCE 7-02 suggests that the ground snow load used in the calculations is to be 10 psf for Oklahoma City. In addition to the snow lying on the roof, a drift load must also be incorporated into the load. The snow drift load is the result of wind causing the snow to build up around obstructions on the roof, which adds additional loads to these areas. These obstructions include the helicopter pad, the parapet, stairwells, and elevator mechanical rooms.

OU Children's Medical Office Building has several unique loading cases which include ambulance load, vehicle impact load, and a load for a helicopter pad. One of the design parameters for the building is to have an ambulance bay, which presents another unique loading case. This loading is specified by AASHTO. In addition to the live loads, the building has a vehicle impact load. This load is located 18" above the finish floor and is a 6 kip unfactored load. Due to the proximity of a parking garage, a vehicle impact load must be applied to ensure the stability of the building in the event a column is struck by a vehicle. Another design requirement of the building is for the future installation of a helicopter pad on the roof. This loading is determined by the helicopter pad manufacture.

Foundations

The foundations receive the loading from the columns and must transfer it onto stable ground. A foundation plan is shown in Figure 3. The foundations are comprised of concrete drilled piers underneath the columns, shown in blue, and spread footings under the shear walls in the southwestern corner, shown in red. Areas shown in green have a spread footing underneath the column with a drilled pier under the spread footing.

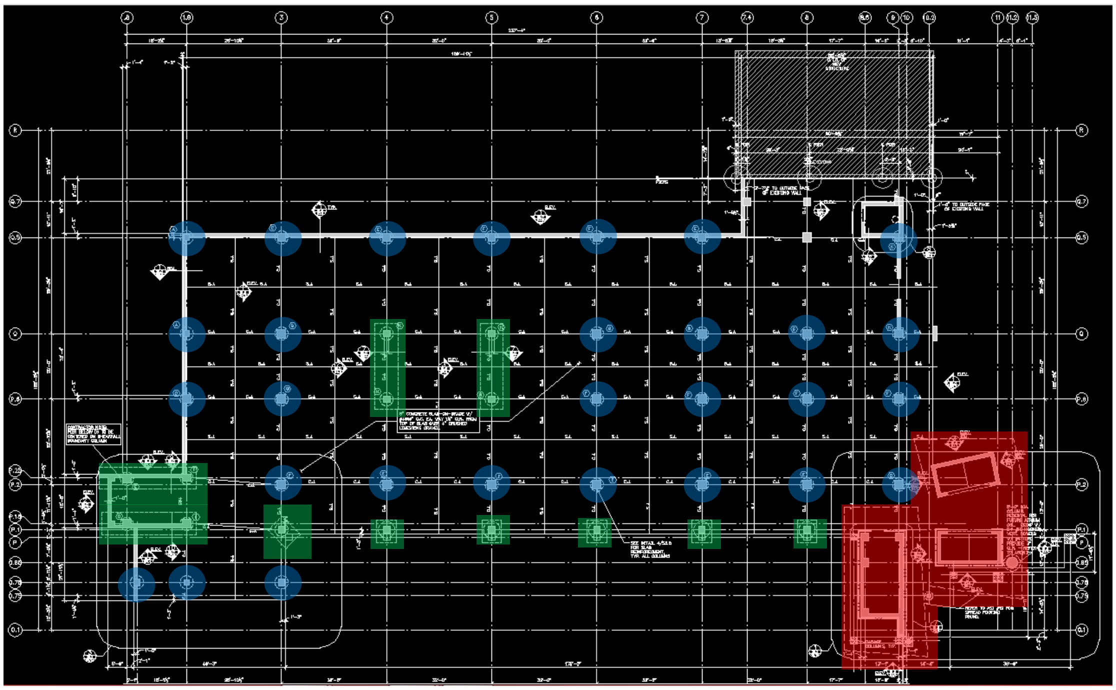


Figure 3. Foundation Plan highlighting the footings and drilled piers.

The drilled piers are used to transfer the loading from the columns down into the bedrock. From the geotechnical report, the pier bearing elevation must be below 1195 feet in order to achieve the maximum bearing capacity. The bottom of pier elevation exceeds the 1195 feet with most at 1190 feet. The lowest elevation is at 1167 feet. The shaft size ranges from 30" in diameter to 72" in diameter. The bearing capacity ranges depending on the pier depth and diameter with the minimum at 679 kips and the maximum at 4307 kips. The reinforcement depends on the diameter. The smallest pier (30" in diameter) uses 8 #6 bars while the largest pier (72" in diameter) uses 21 #9 bars. The ties are typically #5 bars but #3 bars are used for the #6 vertical bars. The spacing for the ties is different depending on the pier and vertical bars. The smallest spacing is 10" on center and largest spacing is 18" on center. Figure 4 shows a typical detail of a drilled pier.

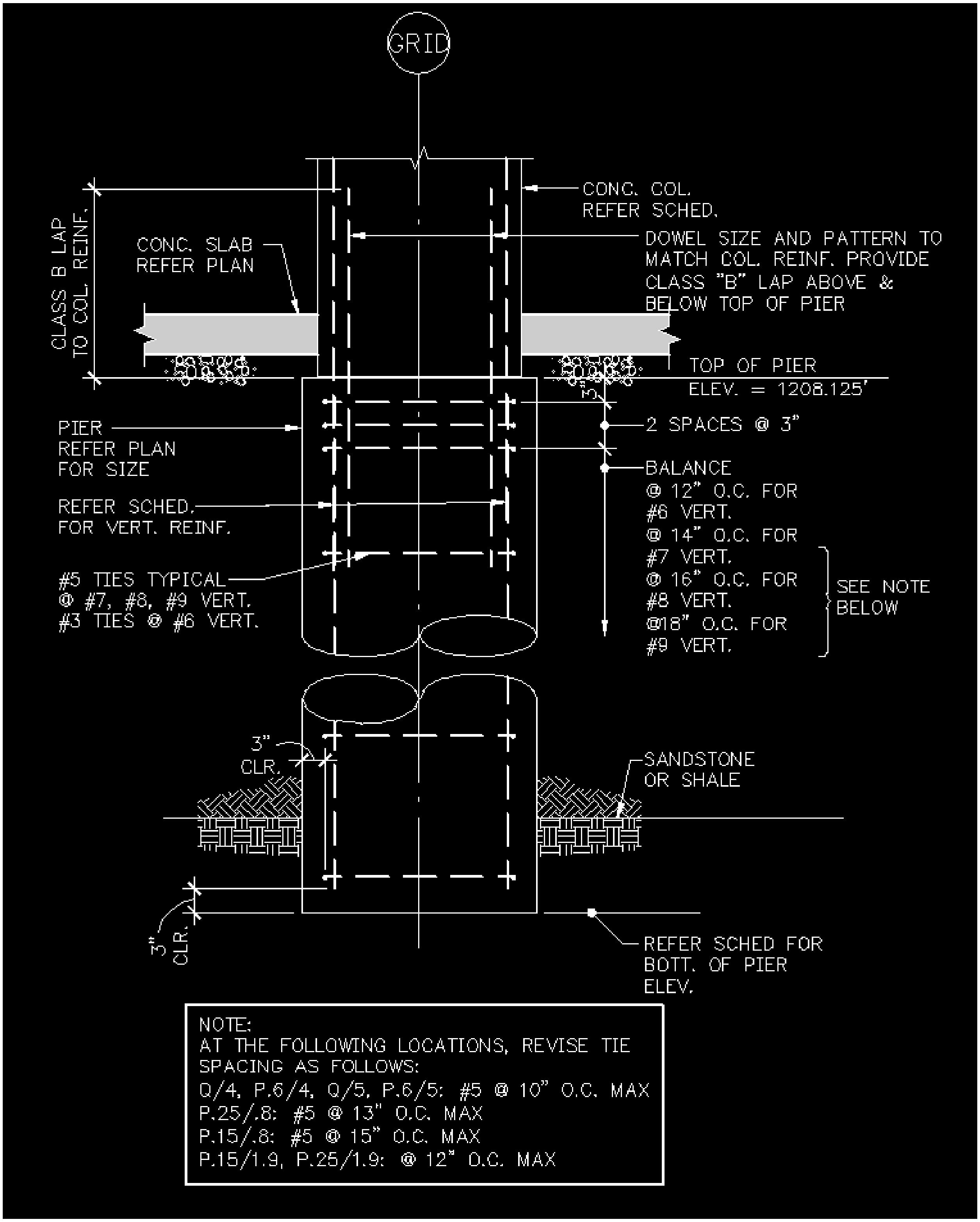


Figure 4. Typical Pier Detail

The spread footings are designed to transfer the load from a column or wall over a larger area so the soil can resist the loading without significant settling. The footings are made of 4'-6" deep cast-in-place reinforced concrete. 21 #11 bars are used on both the top and bottom to resist the tensile forces created by the column. Since the footings are relatively short in length, 90° hooks are required on each end in order to get a full development length. #5 stirrups are used at 24" on center to resist the shear forces. Figure 5 shows a typical footing detail.

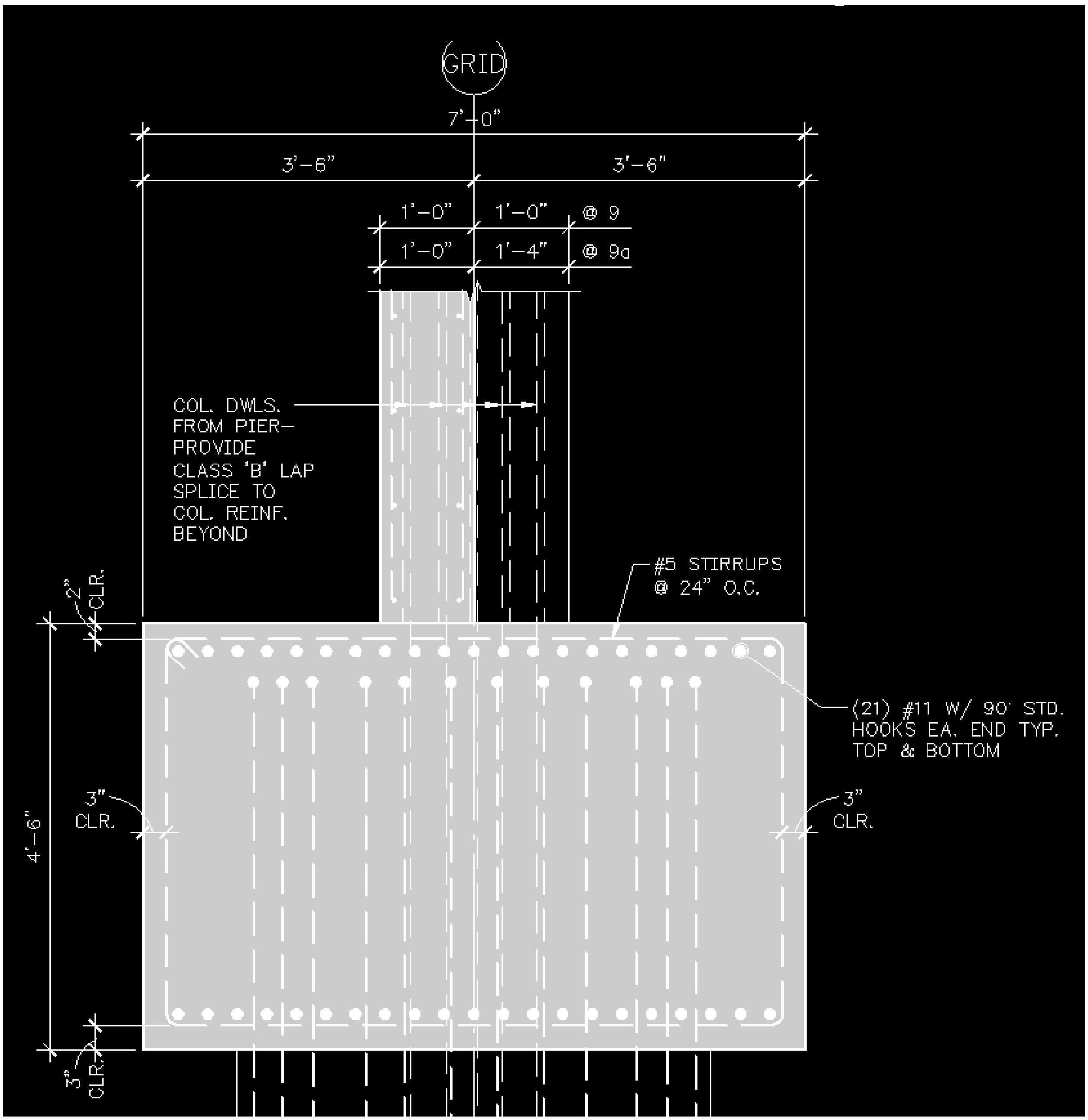


Figure 5. Typical Footing Detail

Typical Bay

A typical bay consists of four columns situated in a square as shown in Figure 6. A flat slab spans between the columns with drop panels. A flat slab refers to a slab that is supported by columns and drop panels and not by beams. The reinforced slab is divided into the column strips, which span between the columns, and the middle strips, which are at the interior of the bay. The bay also consists of drop panels located below the slab at the four columns. The purpose of the drop panel is to provide extra thickness to control the negative moment created by the load case and to resist shear. Without the drop panels, the slab would have to be thicker in order to resist the distributed load case. As a result, the drop panels use less material and therefore save money on construction costs.

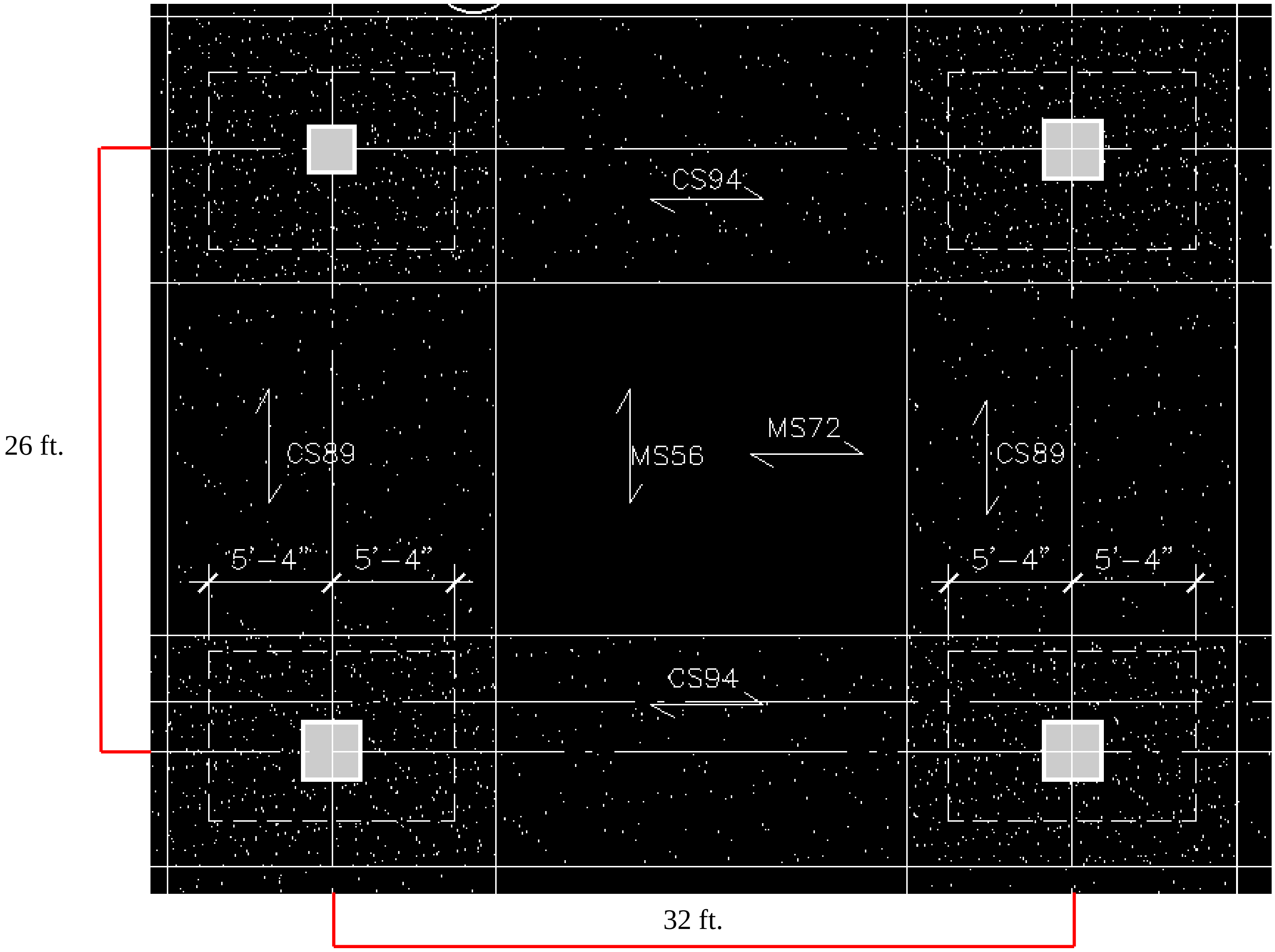


Figure 6. Typical Bay Plan
The strips designated with a C are column strips and those designated with an M are middle strips.

Columns

The columns are generally designed to handle the gravity loads of the structure. The columns for this particular building vary in size from 20" x 20" to 30" x 30" with most in the range of 22" x 22" and 28" x 28". As the building increases in height, the columns decrease slightly in size due to the decrease in the loading. Typically the columns are square with the exception of the columns facing the western exterior curtain wall which are circular. The typical concrete strength for the columns is 7000 psi. The reinforcement varies depending on the column size and orientation but most use #6 bars. Most columns require #3 ties at 12" on center. The reinforcement is arranged around the perimeter of the column with a typical 2" offset from the face of the column. This is called clear cover and it is to ensure that enough concrete surrounds the bar to allow the maximum bonding strength between the steel and concrete. ACI 318-02 states that the minimum cover for a column is 1 ½"; however, since the parking garage columns are exposed to the weather, the code states that the minimum cover is now 2". The designers chose to use the 2" cover for the rest of the building for constructability. Refer to Figure 7 for a typical column section. The vertical bars are spliced together with a class B lap splice from the ACI 318-02 code, which refers to a splice where a percentage of the development length of a bar overlaps another bar. This overlap allows for continuous reinforcement the total height of the column. Due to the helicopter pad on the roof, six of the columns are raised above the roof level and are larger in size on account of the larger load.

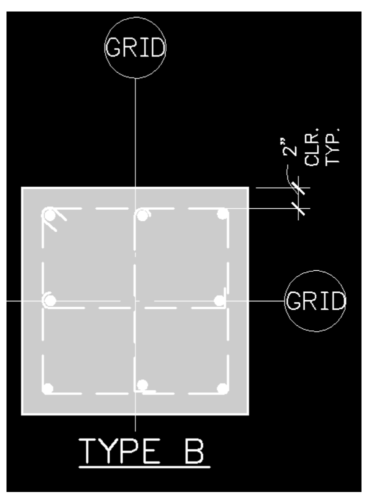


Figure 7. Typical Column Section

Beams

The majority of the beams are located at the exterior of the building. The beams are mainly rectangular in shape and have a range of sizes from 16" x 60" to 36" x 24". Most beams are designed with a concrete strength of 5000 psi. All beams have top reinforcement as well as bottom reinforcement to resist the positive and negative moments resulting from the distributed loading. However, a portion of the beams also contain middle bars. The typical bar size is #9 but #7 is also used. #4 and #3 stirrups are used for the shear reinforcement with a clear cover of 1 ½". Refer to Figure 8 for a typical beam section.

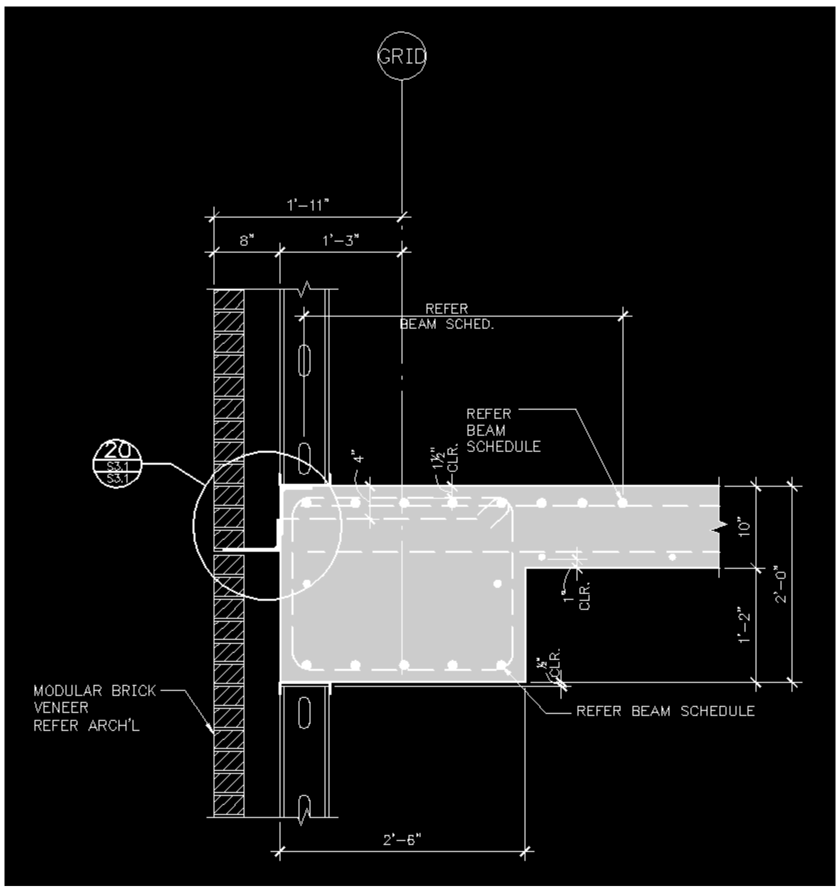


Figure 8. Typical Beam Section

Slabs

In order for the slabs to resist the dead and live loading, they must be reinforced. They are divided into column strips and middle strips, as shown in Figure 6, with different reinforcing in each. The column strip is located at locations between the columns. Since the column strips are significantly thinner than beams, a drop panel is used to carry the extra moment that the slab cannot. The slab thicknesses for the column strip ranges from 10 inches to 12 inches. The 10 inch slab is used in the second through twelfth floors and the roof. The 12 inch slab is used for the parking garage located on the first floor. These thicknesses do not include the drop panels. The reinforcement is typically placed on the top at column locations and on the bottom at mid span. Typically # 6 bars are used at 6" on center. One the other hand, the middle strips are designed differently. The middle strips have reinforcement spanning in two directions. The slab thickness for the middle strips is similar to the column strips in that they also range from 10 inches, for all floors not including the first floor, to 12 inches, for the first floor. The placement reinforcement is also similar to column strips in that the top reinforcement is located at the supports, while the bottom reinforcement is located at mid span. Typically #6 bars are used at 12" on center. All structural slabs have a concrete strength of 5000 psi and a typical clear cover of 1".

Lateral system

Since the building is a reinforced concrete structure, all of the connections between the columns are considered rigid. This means that the connection between the column and the beam has the ability to transfer a lateral load from the diaphragm to the column and into the ground. However, if the lateral loads are great enough, a separate system must be used to transfer the loads. This is called the Lateral Force Resisting System and it involves the use of shear walls, bracing, or moment frames to transfer the loads. The primary Lateral Force Resisting System for this building relies on the shear walls located in the elevator shafts, stairwells and some interior walls as shown in red in Figure 9. Since a shear wall must be continuous from the roof to the foundation, they are typically placed in the elevator shafts or stairwells as shown in Figure 10. The shear walls are typically one foot thick with #4 vertical and horizontal bars at 12" on center. They typically span between two columns, shown in Figure 11, allowing the reinforcement of the shear wall to tie into the column, making the system stiffer.

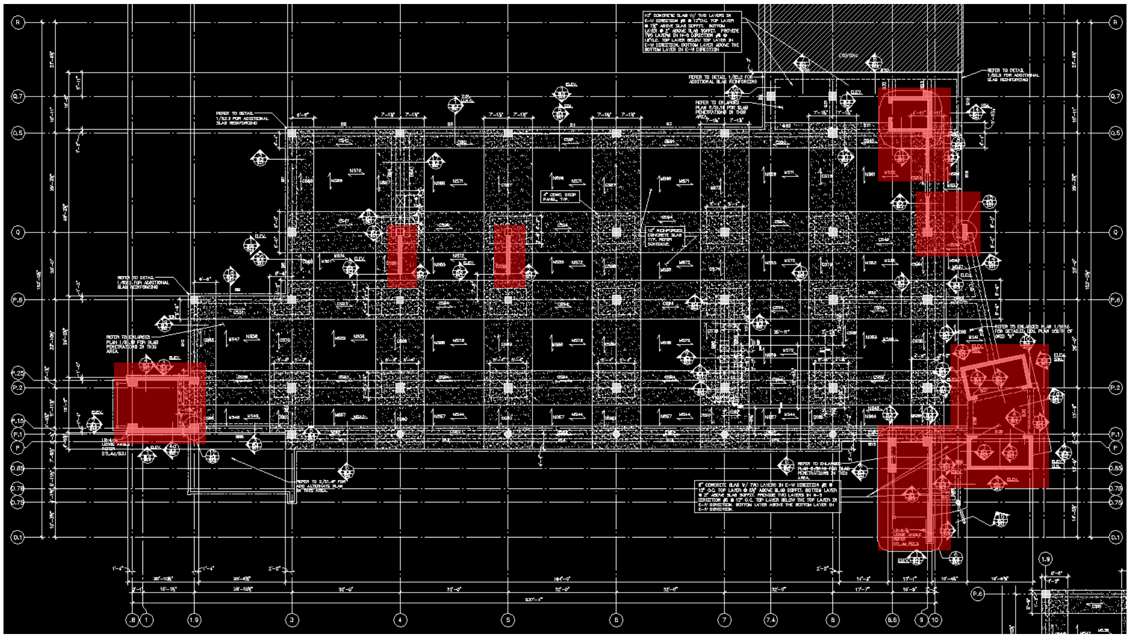


Figure 9. Floor plan highlighting shear wall locations.

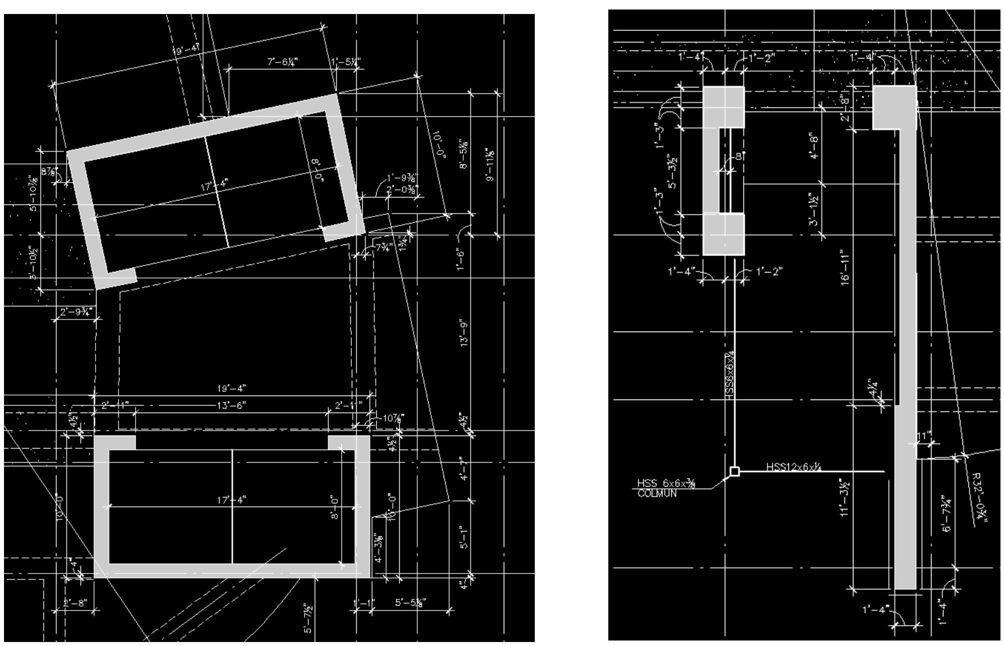


Figure 10. Left – Shear wall located at elevator shafts. Right – Shear wall located at stairwells.

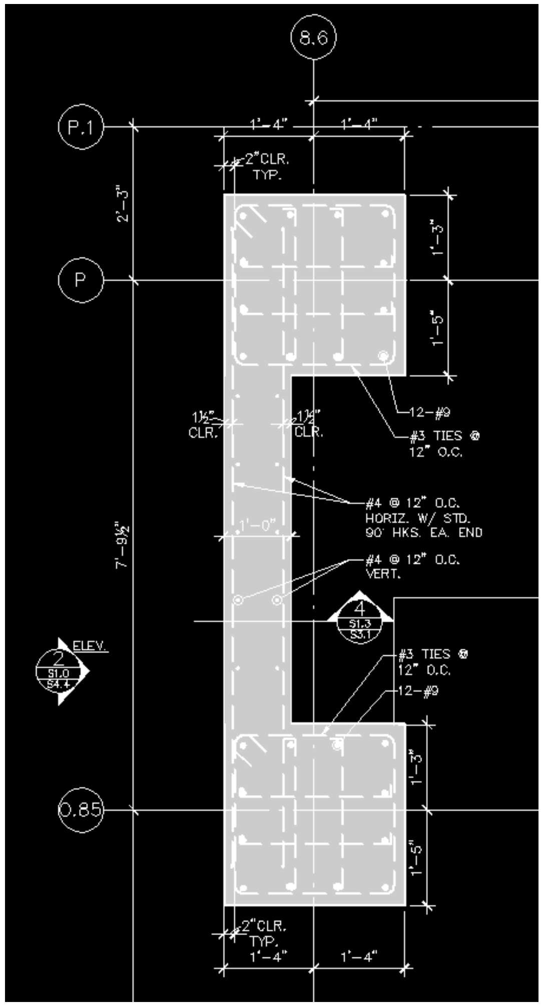


Figure 11. Typical Shear Wall Section

Reinforcement

The placement of reinforcement is key to a sturdy concrete building. Since concrete becomes brittle in tension, steel reinforcement is needed to carry the extra tensile forces. The bars are placed at high tensile stress locations in order to prevent the concrete from cracking. In the case of a beam or slab, the reinforcement is placed at the top towards the supports. This is where a negative moment occurs and therefore where higher tensile stresses occur. Reinforcement is located at the bottom towards the mid span of the beam or slab to provide extra tension capacity where a positive moment occurs. The location of the reinforcement for the beams is shown in Figure 8 while the slab reinforcement is shown in Figure 6. Columns, on the other hand, are reinforced around the perimeter as shown in Figure 7. Columns are reinforced this way to provide tension capacity during lateral loads such as wind or seismic. The reinforcement in columns must be continuous. When are bar is not long enough to span a certain length, it is spliced together to form a continuous bond. These splices can either be mechanical splices or non-mechanical splices such as the class "B" splice mentioned in the column. For non-mechanical splices, the bars must overlap each other a distance relative to the development length to achieve the desired load transfer. Stirrups are used to prevent shear and torsion. Stirrups are smaller bars that encase the horizontal or vertical reinforcement as shown in Figure 8 for beams and Figure 7 for columns. The spacing is critical to prevent failure under torsional conditions. The reinforcement must also have a certain length embedded in the concrete to create a strong bond between the concrete and the steel. This length is called development length. If development length cannot be achieved with a shorter reinforcement span, then special methods must be used, such as bending the bar to form a hook, to achieve the desired length.

Proposed Structural Depth

Since the Oklahoma University Children's Medical Office Building is constructed using a two way concrete system, the construction process is extremely long to allow the concrete to cure. This adds additional costs to the project, making the job more expensive. Concrete is a labor intensive material meaning that the installation requires many skilled labors to construct the forms, set the reinforcement, and leveling the concrete. The high amount of labor adds to the overall project costs, making concrete a relatively expensive material. The proposed thesis will be a redesign of the building structure using steel. The gravity system will consist of a composite steel system with composite decking for the floors. The roof will consist of k series joists with wide flange girders and roof deck. The lateral system will consist of steel braced frames located at existing shear wall locations. Since concentric braced frames are not quite as stiff as shear walls, additional braced frames or moment fames will have to be included in the design. A redesign using steel instead of concrete should reduce construction time resulting in lower costs to the owner. Steel in comparison with concrete is not as labor intensive. Steel does not require the level of skilled labor as concrete does to install unless complicated field welds are used. As part of the proposal, bolted or factory welded connections will be used where ever possible. These connections will be used to speed up the construction process and reduce the costs as field welds become expensive and time consuming due to the skilled labor and precision of the weld.

Proposed Construction Breadth

Due to the proposed change of material, the first proposed breadth is a detailed cost analysis and schedule impacts of the proposed steel system. Since the material will be changed from concrete to steel, the costs for labor should decrease reducing the overall project costs. The construction time will be decreased due to the material change. The time it takes for the concrete to cure dramatically increases construction times. With using steel as the structural material, these times can be greatly reduced saving money.

Proposed Green Roof Breadth

The second breadth will be the addition of an extensive green roof. With the addition of the green roof, this will potentially reduce the heat island effect of the building. The green roof also has the potential to clean the air by filtering pollutants such as carbon dioxide. This will potentially reduce the heating and cooling cost of the building. An extensive green roof will be used instead of an intensive green roof due to the lower initial costs, lower maintenance costs, and lower roof loads. Research will be conducted to use local plants to reduce the amount of maintenance. The plants that are typically used for an extensive green roof are hardy perennials that can withstand wind and extreme temperature fluctuations. Sedums are typically used because they are drought resistant and require little maintenance. Other components of the green roof must also be researched to provide a stable area for growing the plants without causing damage to the building such as water leaks. These areas include the growing medium, a filter membrane, a drainage layer, a root barrier, and a waterproofing membrane. The down side of the green roof addition will be higher initial costs and additional loads which will have to be accounted in the load calculations.

Structural Redesign

Codes and References

The structural redesign will conform to the latest codes and standards available. The codes used are as follows:

- International Building Code 2009
- ASCE 7-10
- AISC Steel Construction Manual 14th Edition
- AISC Steel Design Guide 11

Additional references:

- Vulcraft Steel Deck Catalog
- Steel Joist Institute Joist Catalog
- AISC Design Examples Version 14.1
- RS Means Cost Construction Data 2014

Gravity

The current system for the building is a reinforced concrete two way slab system with drop panels located at interior column locations. This system will be replaced with a composite steel system with composite decking for each of the floors. A typical floor plan is shown in Figure 12. The roof will consist of roof deck, K-series joists and wide flange girders. To make this system economical, an unshored condition will be used in the design of the metal deck, beams and girders. The new building design will be analyzed using RAM software with spot checks done by hand calculations. The typical bay that was spot checked is shown in Figure 12 in red for the floor plan and in Figure 13 in green for the roof plan.

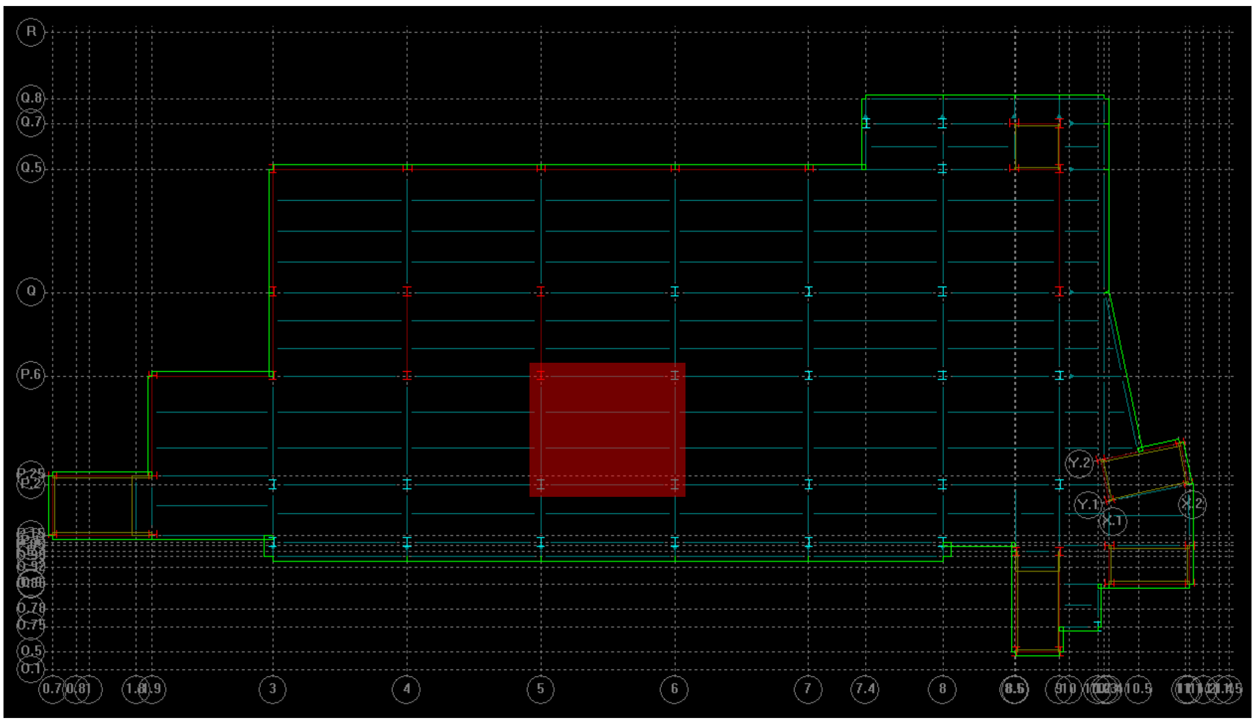


Figure 12. Typical Floor Plan

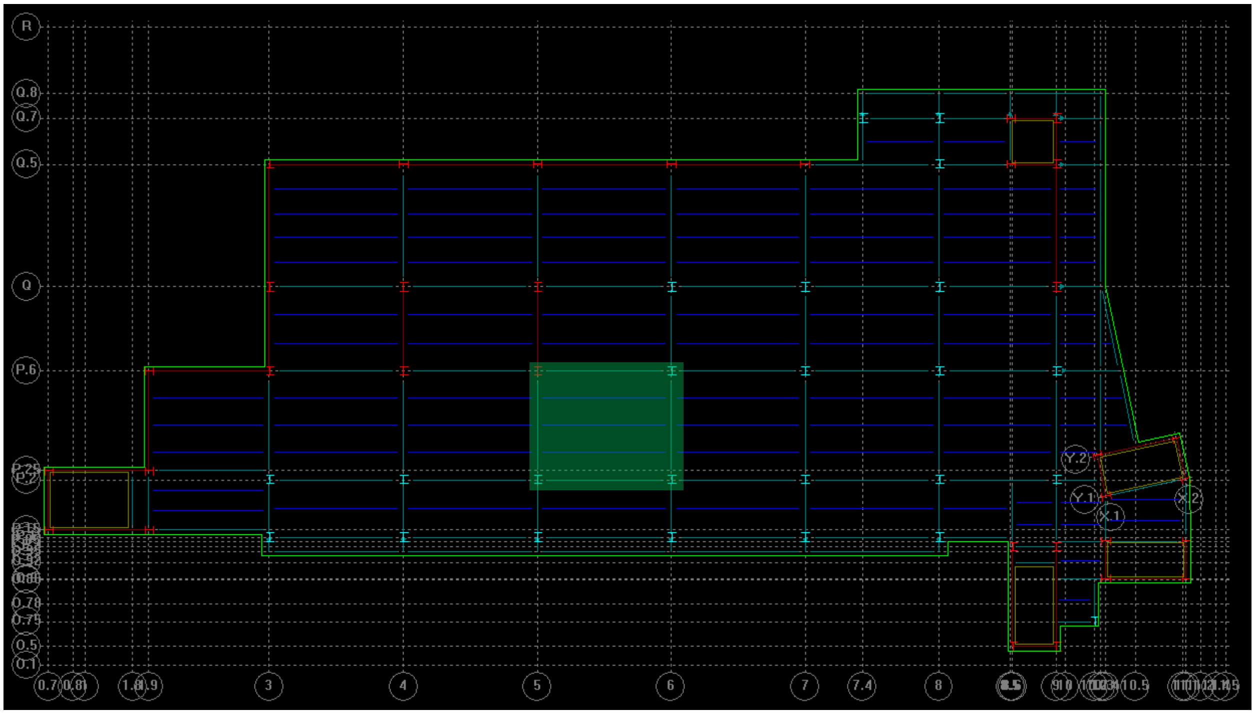


Figure 13. Roof Plan

Load Combinations

The basic load combinations where determined from ASCE 7-10. It can be concluded that load combination 2 will control for gravity members. Lateral members will be either controlled by load combination 4 for wind or load combination 5 for seismic.

1. $1.4D$
2. $1.2D + 1.6L + 0.5(Lr \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(Lr \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$
4. $1.2D + 1.0W + L + 0.5(Lr \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + L + 0.2S$
6. $0.9D + 1.0W$
7. $0.9D + 1.0E$

Gravity Loads

The gravity loads where determined using the IBC 2009 building code and ASCE 7-10 code. A summary of the loads is shown in Table 1. The live load of 80 psf for corridors was chosen over the office live load of 70 psf to provide flexibility to the floor plan. Loads for specific materials were determined from AISC Steel Construction Manual Table 17-13. A breakdown of the green roof loads will be explained in further detail during the Green Roof Breadth section. This value is an estimate. The determination of the wall loads and miscellaneous loads are located in Appendix A.

Description	Load
Live Load	80 psf
Roof Live Load	20 psf
Snow Load	10 psf
Superimposed	15 psf
Carpet with Pad	2 psf
Rigid Insulation	2 psf
Green Roof	30 psf
Floor Deck	41 psf
Roof Deck	3 psf
Beam Allowance	5 psf
Joist	12 plf
Girder Allowance	2 psf
Ambulance Bay	60 psf
Helicopter Pad	8.33 kips
Exterior Brick Wall	660 plf
Exterior Glass Wall	220 plf

Table 1. Gravity Load Summary

Lateral Loads

The lateral loads where determined using procedures outlined in ASCE 7-10. The wind loads where determined from the Directional Procedure in ASCE 7-10. The wind loads determined in the RAM model are higher and therefore more conservative than the loads calculated by hand. The loads in the North-South direction are summarized in Table 2. The East-West direction loads are summarized in Table 3. The hand calculations are shown in Appendix B.

Height	qz	G	Cp Windward	Cp Leeward	Pressure (psf)	Story Force (kips)	Story Shear (kips)	Overturning Moment (k-ft)
165.78	20.13	0.97	0.80	-0.30	21.36	59.08	59.08	9794.28
153.78	19.70	0.97	0.80	-0.30	21.03	33.79	92.87	14281.55
141.78	19.25	0.97	0.80	-0.30	20.68	33.23	126.10	17878.46
129.78	18.77	0.97	0.80	-0.30	20.31	32.63	158.73	20599.98
117.78	18.25	0.97	0.80	-0.30	19.91	31.99	190.72	22463.00
105.78	17.70	0.97	0.80	-0.30	19.48	31.30	222.02	23485.28
93.78	17.10	0.97	0.80	-0.30	19.02	30.55	252.57	23686.01
81.78	16.45	0.97	0.80	-0.30	18.51	29.74	282.31	23087.31
69.78	15.72	0.97	0.80	-0.30	17.95	30.13	312.44	21802.06
56.67	14.81	0.97	0.80	-0.30	17.25	31.98	344.42	19518.28
42.00	13.59	0.97	0.80	-0.30	16.31	31.25	375.67	15778.14
28.00	12.11	0.97	0.80	-0.30	15.16	28.30	403.97	11311.16
14.00	10.13	0.97	0.80	-0.30	13.63	26.03	430.00	6020.00
				Base Shear (kips)		430.00		
						Total Overturning Moment (k-ft)		229705.52

Table 2. Wind Loads in the North-South direction

Height	qz	G	Cp Windward	Cp Leeward	Pressure (psf)	Story Force (kips)	Story Shear (kips)	Overturning Moment (k-ft)
165.78	20.13	0.86	0.80	-0.50	22.45	122.65	59.08	9794.28
153.78	19.70	0.86	0.80	-0.50	22.15	72.81	195.46	30057.84
141.78	19.25	0.86	0.80	-0.50	21.84	71.79	267.25	37890.71
129.78	18.77	0.86	0.80	-0.50	21.51	70.70	337.95	43859.15
117.78	18.25	0.86	0.80	-0.50	21.16	69.54	407.49	47994.17
105.78	17.70	0.86	0.80	-0.50	20.79	68.29	475.78	50328.01
93.78	17.10	0.86	0.80	-0.50	20.37	66.94	542.72	50896.28
81.78	16.45	0.86	0.80	-0.50	19.92	65.45	608.17	49736.14
69.78	15.72	0.86	0.80	-0.50	19.42	66.70	674.87	47092.43
56.67	14.81	0.86	0.80	-0.50	18.80	71.34	746.21	42287.72
42.00	13.59	0.86	0.80	-0.50	17.97	70.44	816.65	34299.30
28.00	12.11	0.86	0.80	-0.50	16.94	64.76	881.41	24679.48
14.00	10.13	0.86	0.80	-0.50	15.59	60.64	942.05	13188.70
					Base Shear (kips)	942.05		
						Total Overturning Moment (k-ft)		482104.21

Table 3. Wind Loads in the East-West Direction

The seismic loads where determined by the Equivalent Lateral Force Procedure outlined in ASCE 7-10. The seismic design criterion for Oklahoma University Children's Medical Office Building was determined from the USGS Seismic Design Maps application. The seismic loads are summarized in Table 4 for both the X and Y directions. The hand calculations can be found in Appendix C.

hx (ft)	Wx (kips)	$Wxhx^k$	Cvx	Fx (kips)	Vx (kips)	Overturning Moment (k-ft)
166	144	359028	0.135	60.2	60.2	9993.2
154	154	342316	0.129	57.5	117.7	18125.8
142	154	315642	0.119	53.1	170.8	24253.6
130	154	288968	0.109	48.6	219.4	28522
118	154	262294	0.099	44.2	263.6	31104.8
106	154	235620	0.089	39.7	303.3	32149.8
94	154	208946	0.078	34.8	338.1	31781.4
82	154	182272	0.068	30.3	368.4	30208.8
70	154	155598	0.058	25.9	349.3	24451
56	154	124479	0.047	21	415.3	23256.8
42	154	93359	0.035	15.6	430.9	18097.8
28	154	62239	0.023	10.3	441.2	12353.6
14	154	31119	0.012	5.4	446.6	6252.4
		Base Shear (kips)		446.6		
				Total Overturning Moment (k-ft)		290551

Table 4. Seismic Loads for North-South and East-West Directions

As shown in the tables, the wind in the Y direction is the controlling load case for the lateral system.

RAM Model

The proposed building solution will be analyzed using RAM software. Several modeling assumptions must be made in order to create the model. These are:

- Each floor diaphragm is considered to be rigid
- The bottoms of columns will be considered as pinned connections

The model shall be designed conforming to the 2009 International Building Code, ASCE 7-10 code, and AISC 360-10 LRFD steel code. Figure 14 and Figure 15 shows a typical floor plan and an isometric view of the RAM model, respectively.

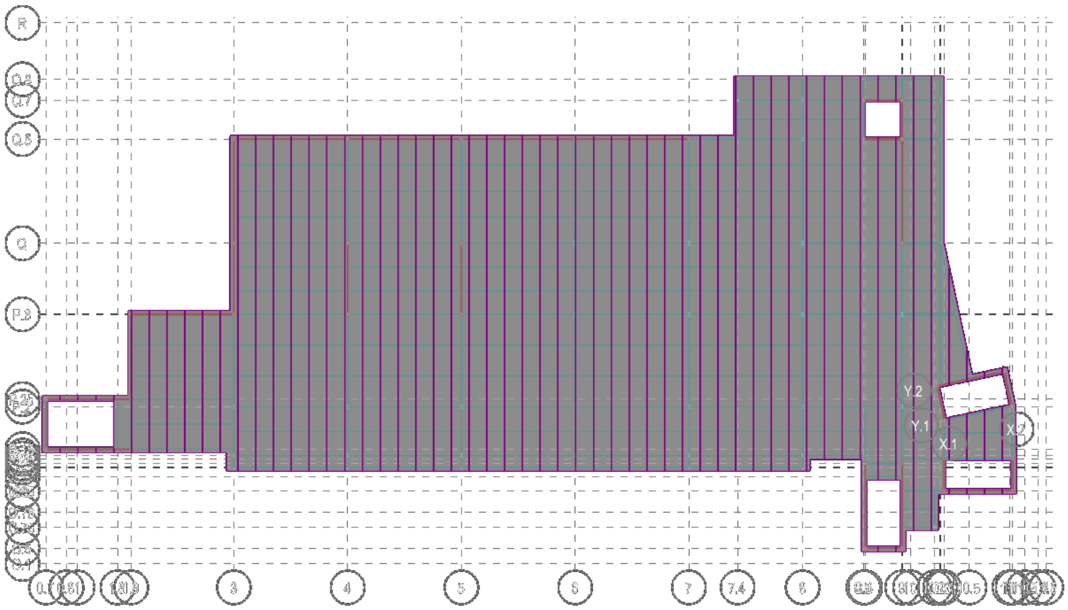


Figure 14. Typical Floor Plan of RAM Model

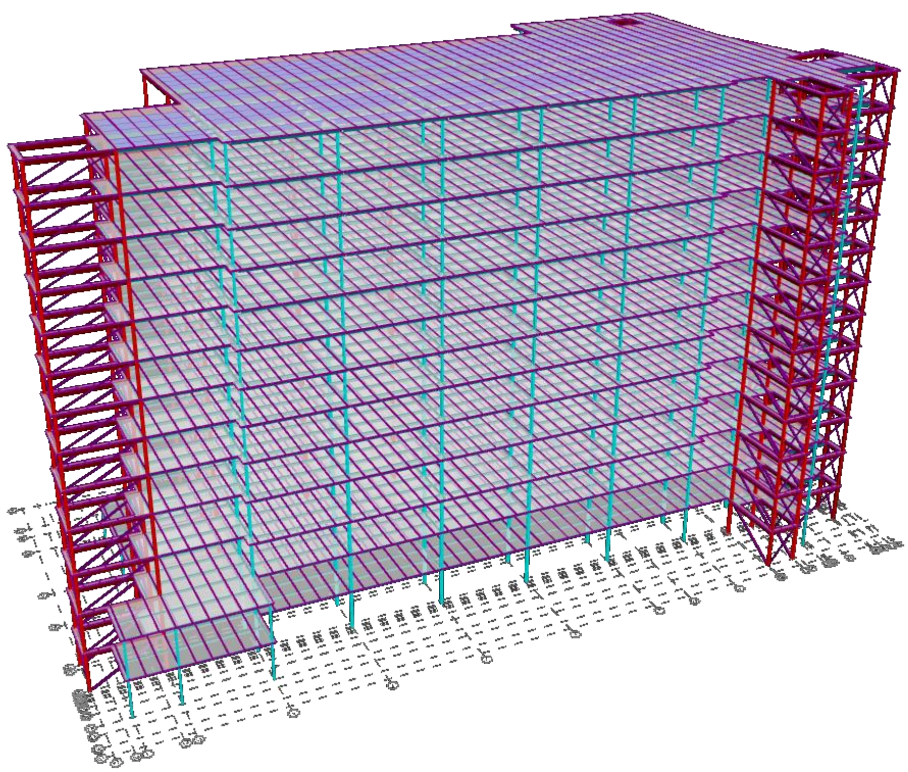


Figure 15. Isometric View of RAM Model

Typical Bay

This section explains the breakdown of the design of the typical bay. A plan view of the typical bay is shown in Figure 16.

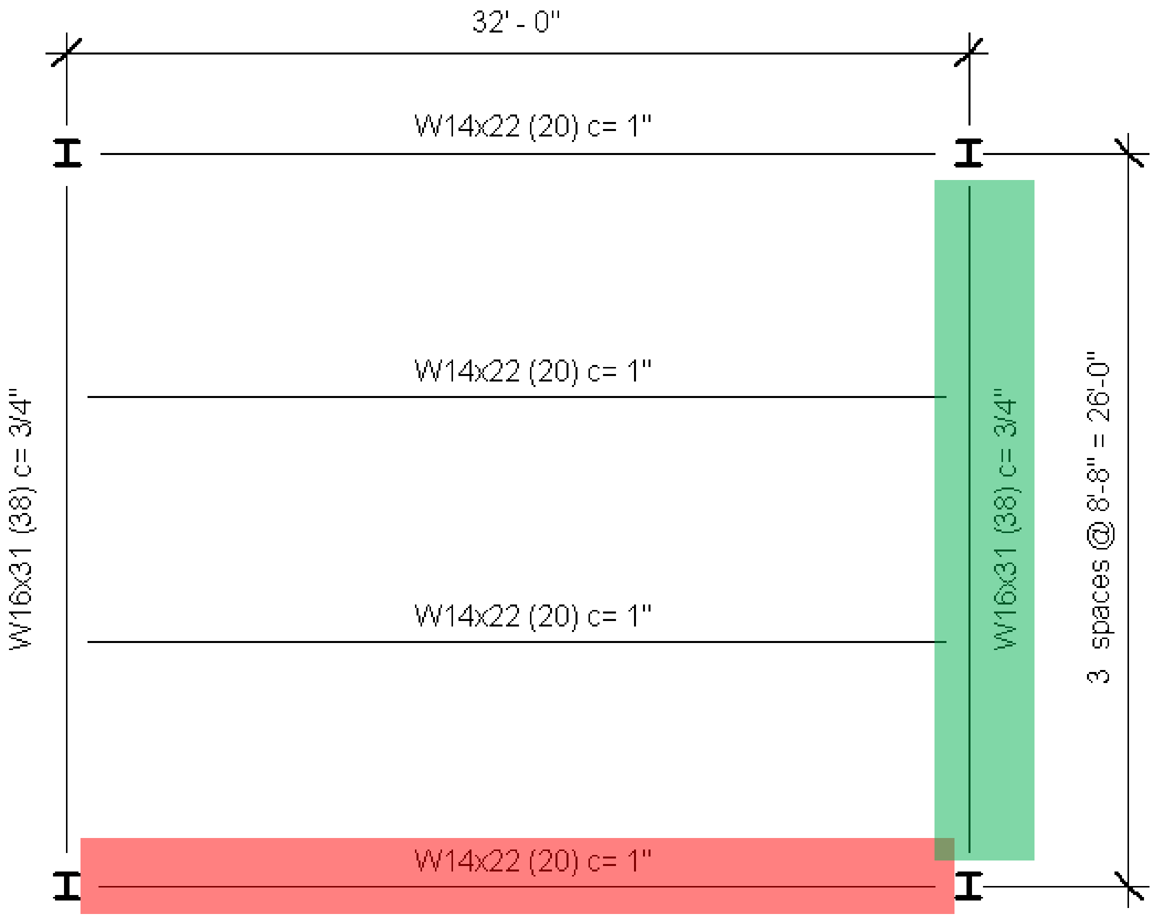


Figure 16. Typical Bay Plan

The decking will be composite decking with a 3 ¼" lightweight concrete. The topping was chosen to conform to a two hour fire rating without the deck having to be fireproofed. The deck will be designed using the Vulcraft Deck Catalog. Based on the applied loading and a 3 span condition, the deck will be 1.5 VLR 19 gauge. The unshored span for the deck controls the beam spacing for the bay. Hand calculations for the determination of the deck are located in Appendix D. The deck does not need fireproofing; however, the deck in the parking garage should be encased in at least 2" of concrete to protect it from the weather.

The beams will be designed based on composite action with the concrete. In order to analyze the composite section, several design assumptions must be made:

- F'c = 4000 psi
- The deck is perpendicular with the beam
- 1 weak stud per rib
- ¾" diameter per stud
- 1 stud per foot

The unshored construction, wet concrete deflection, and live load deflection was checked to determine the final beam design. The final design will be a W14x22 with 20 studs per beam with a 1" chamber. The full hand calculations for the beam are located in Appendix D. The beam is shown in Figure 16 in red. The beams will be sprayed with fireproofing based on a two hour fire rating for offices. The beams in the parking garage will need to be encased in a least 2" of concrete.

The girders will also be designed based on composite action with the concrete. The girder will be designed based on similar assumptions:

- F'c =4000 psi
- Y = 5"
- The deck is parallel with the girder
- 1 weak stud per rib
- ¾" diameter per stud
- 1 stud per foot

The girder will also be checked based on unshored strength and live load deflections. The final girder design is a W16x31 with 38 studs and a ¾" chamber. The full calculations for the girder are located in Appendix D. The girder is shown in green in Figure 16. The girders will be sprayed with two hour fireproofing. Similar to the beams, the girder will also need to have a concrete cover of 2" for weather protection.

The roof will be designed using roofing deck with k-series bar joists spanning between wide flange girders. The roof deck will designed based on the Vulcraft Deck Catalog for roof. The deck will be 1.5 B 22 gauge roofing deck. The joists for the typical bay will be 24K9 joists as shown in red in Figure 17. The joists where designed from the LRFD economy tables from the joist catalog. The girders will be a W18x40 based on table 3-2 from AISC Steel Construction Manual. The girders where checked for unbraced length since they are not continuously braced by the roof deck. The girder is shown in the typical roof plan in green in Figure 17. Hand calculations for the typical roof bay are found in Appendix E. The roof deck, joists, and girders will be sprayed with a two hour fire rating.

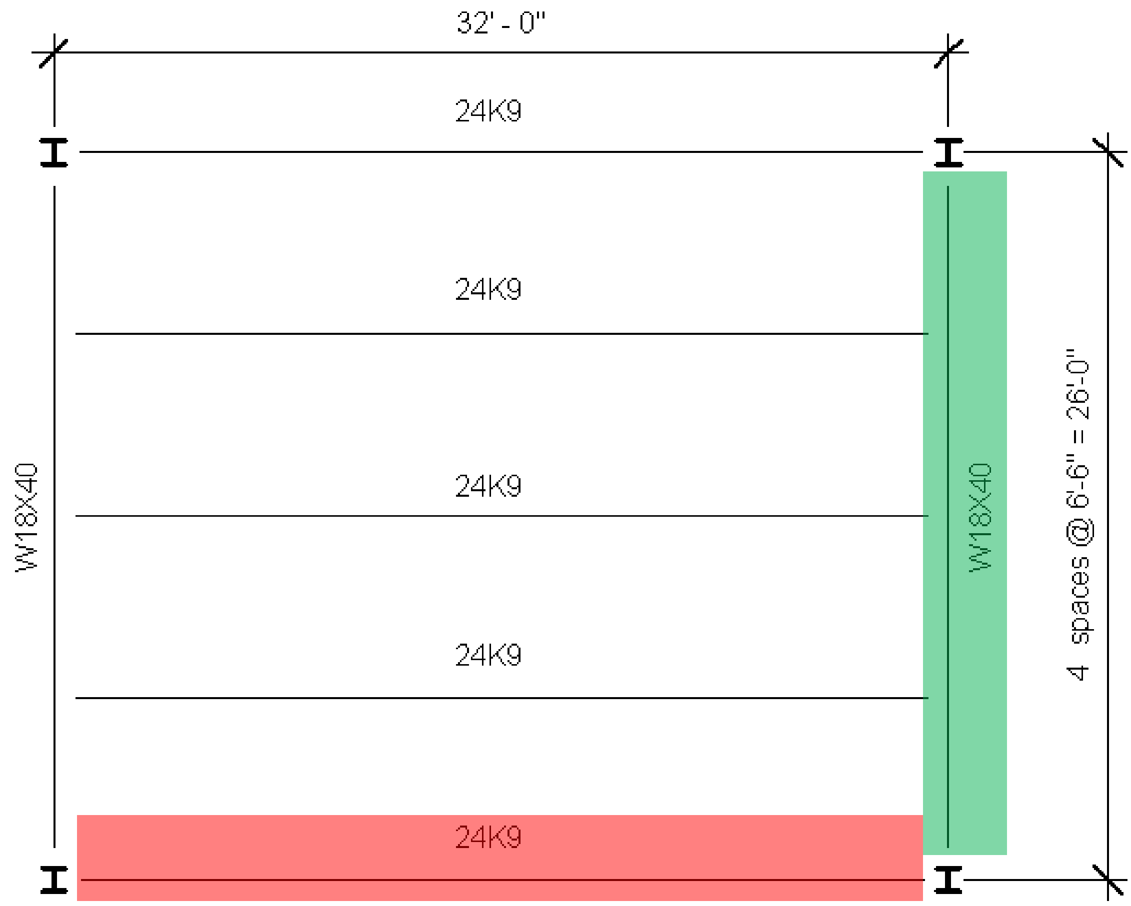


Figure 17. Typical Roof Bay

The columns will be spliced at every other floor beginning with the second floor and ending with the twelfth floor. The columns for the roof level will not be spliced due to the odd number of floors. The columns will be encased in concrete for the bottom two floors with a minimum cover of 3" below the first floor due to the permanent exposure with the earth. The columns below the first floor will have a cover of at least 2" to protect the steel from the weather. A column hand check at the base of the building is shown in Appendix F.

Vibrations

Since the building lost mass due to the reduction of material, vibrations must be considered. The vibration analysis will be conducted using AISC Design Guide 11 for human activity. The vibrations analysis for the typical bay passes the criteria based in AISC Design Guide 11. The hand calculations are located in Appendix G for the vibration analysis.

Impact on Foundations

Since the building decreased in weight, the foundations can be reduced in size. This analysis however, is outside the scope of this thesis but will be looked at conceptually. The foundations will have about 40% less load and therefore could be redesigned to reduce some costs on the building.

Lateral System

The current lateral system comprises of shear walls located in the stairwells and elevator shafts with two shear walls located in the center of the floor plan. These shear walls are replaced with braced frames located at the original shear wall locations. The braced frames will be concentric diagonal braces composed of square HSS tubes. The braces are shown in Figure 18 in red. The critical brace frame sizes where checked based on AISC Steel Manual and AISC Design Examples. The columns where determined to be a W12x106 at the base. The beam is a W18x40 and the brace is an HSS 8x8x5/16. The critical frame is located in orange in Figure 18. The hand calculations for the brace are located in Appendix H. Since the braced frames are not as stiff as the shear walls, additional moment frames are located along the East wall. Unfortunately these frames could not be braced frames do to impacts from the architecture. The critical moment frame was also checked using the AISC Steel Manual and the AISC Design Examples. The critical moment frame is shown in blue in Figure 18. The hand calculation for the moment frames are located in Appendix I.

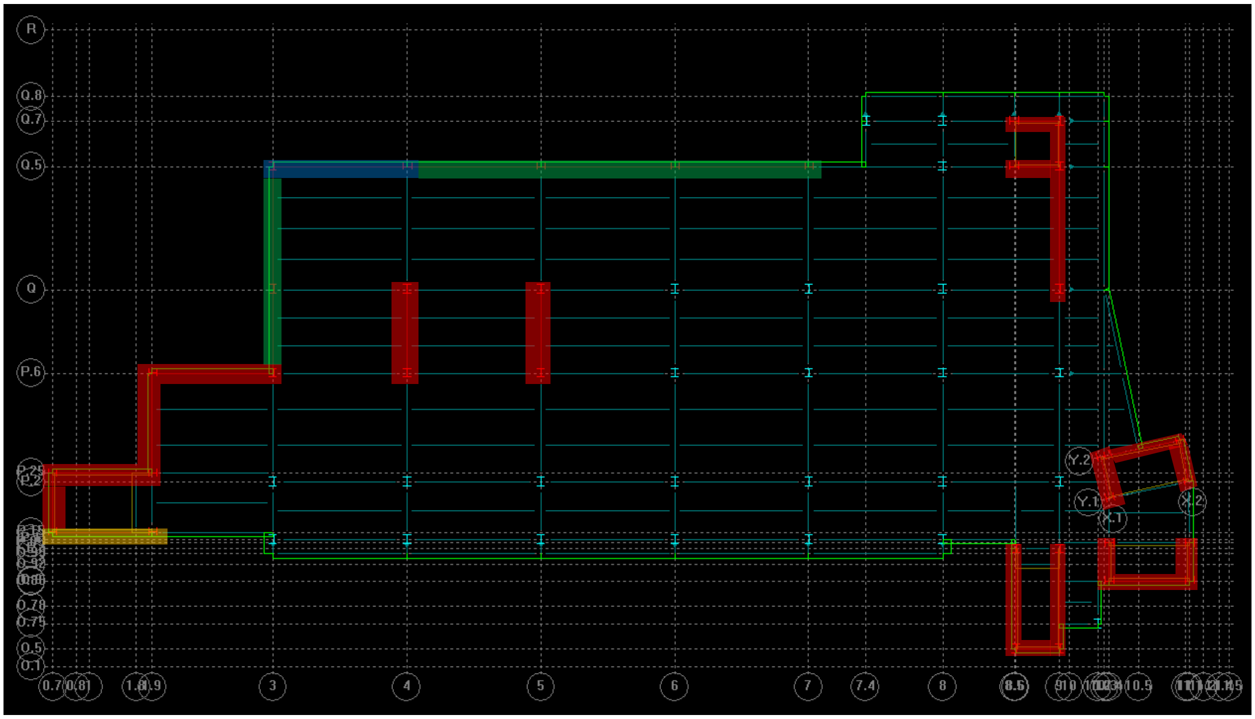


Figure 18. Lateral System Layout

Story Drift

The story drift determined from the RAM model is 4.75 inches. The existing building drift is 4.77 inches calculated from an ETABS model. The allowable drift is calculated from the IBC 2009 allowable equation of h/400. Based on this equation the redesign of the lateral system is under this allowable drift of 4.98 inches at the roof.

Structural Depth Conclusion

From the structural redesign, steel can greatly reduce the amount of weight and material used to construct a building, drastically changing how the building preforms under certain loading patterns. The proposed braced frames located in the shear wall locations did not provide enough stiffness to control the building under the determined lateral forces; thus, additional moment frames had to be included in order to reduce the drift under the code allowable.

Green Roof Breadth

Green roofs are one of the new developments for providing an energy efficient building while having the ability to reduce carbon emissions in heavy urbanized areas. The two types of green roofs are intensive and extensive. An intensive green roof is uses a larger growing medium to provide a larger assortment of plants that can range from flowers to trees and can provide tenants with a garden environment. These roofs are typically more expensive, require more maintenance, and have higher loads associated with them. An extensive green roof has a shallower growing medium in which only small plants and grasses can grow. These are typically designed to have little human intervention to sustain the roof. This means that extensive green roofs have a lower maintenance cost, but are typically designed not as a roof garden but as a cost effective way to reduce the energy costs of the building. These roofs also have a lower initial cost and have lower loads. The green roof that will be used as part of this study will be an extensive green roof due to the lower costs and impact on the structure. Building occupants will also have limited access to the roof to justify the need for an intensive roof garden. A study is conducted to determine the appropriate plants and materials to utilize for a green roof that is site specific for the Oklahoma University Children's Medical Office Building.

Plant Types

In order to determine the appropriate plants that can survive in Oklahoma's environment, the hardiness zone for Oklahoma needs to be determined. As shown in Figure 19 from the USDA website, the hardiness zone for Oklahoma City is zone 7a and 7b. The hardiness zone allows gardeners and growers to determine which plants will most likely thrive in a specific environment.

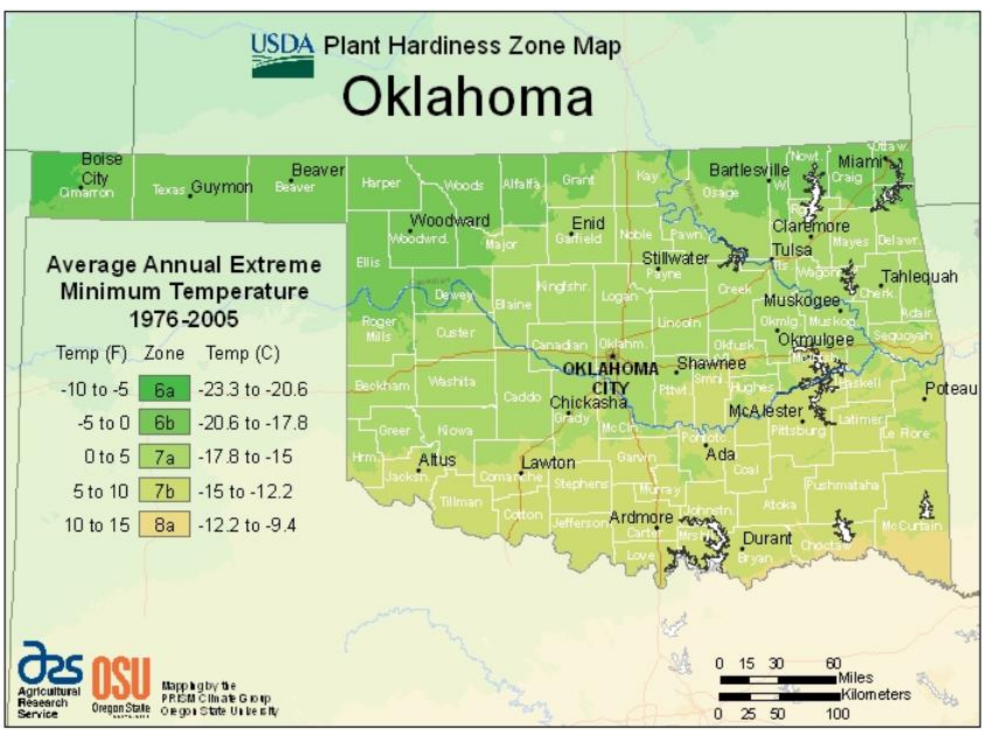


Figure 19. Hardiness Zone Map

(Image obtained from http://planthardiness.ars.usda.gov/PHZMWeb/)

Due to the selection of the green roof being an extensive green roof, the selection of plants becomes very limited because of the shallow growing medium. Sedum plants are typically used for an extensive green roof because of their low maintenance and ability to resist long droughts. These plants can grow in soil as shallow as four inches. Five plants have been selected for the green roof, which include: sedum oreganum (Figure 20), sedum sexangulare (Figure 21), sedum floriferum (Figure 22), sedum spurium (Figure 23), and phedimus takesimensis (Figure 24).

Figure 20. Sedum Oreganum

(Image obtained from http://www.greatcity.org/)

Figure 21. Sedum Sexangulare

(Image obtained from http://www.greenroofplants.com/catalog/plant-catalog/viewplant/?plantid=717)

Figure 22. Sedum Floriferum

(Image obtained from http://macgardens.org/?m=201306)

Figure 23. Sedum Spurium

(Image obtained from http://www.greenroofplants.com/catalog/plant-catalog/viewplant/?plantid=733)

Figure 24. Phedimus Takesimensis

(Image obtained from http://www.greenroofplants.com/catalog/plant-catalog/viewplant/?order_code=PHGC)

All of these plants can grow in Oklahoma's hardiness zone and can tolerate droughts, which reduces the need for an irrigation system. These plants are typically small with the average height being five inches. As shown in the figures above, these plants come in a range of colors including green, yellow and red.

Materials

A typical green roof is comprised of the vegetation, growing media, a filter fabric, a drainage panel, a root barrier, and a water proof membrane. Rigid insulation and vapor barrier can also be included above the roof deck. A typical section for a green roof is shown in Figure 25.

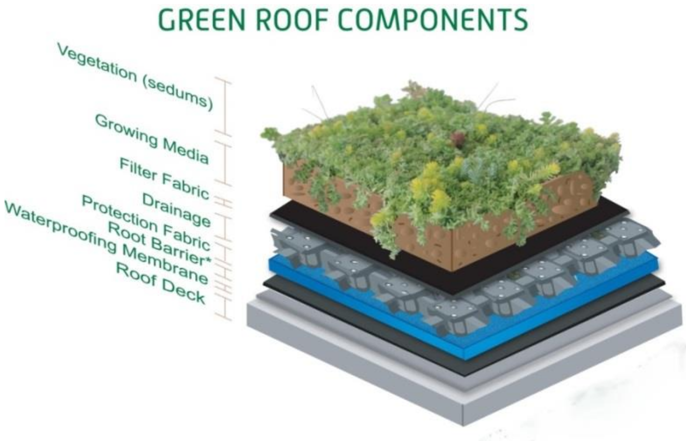


Figure 25. Green Roof Cross Section

(Image obtained from http://www.vegetalid.us/green-roof-systems/green-roof-101/what-is-a-green-roof)

For an extensive green roof, the growing medium is typically four inches. Rooflite extensive mcl growing medium will be used for this green roof. This growing medium provides the correct nutrients needed by the sedum plants selected while providing enough porosity for water and air. A filter fabric is used to filter sediments from the growing medium before it enters the drainage panel. The filter fabric that will be used is FF35 from Green Roof Solutions. This fabric was selected due to its high tear strength and high puncture strength while allowing water to pass through. A drainage panel is used to collect water and distribute that water throughout the entire green roof system while allowing excess water to flow into the roof drain. The drainage panel that was used is GRS 32 from Green Roof Solutions. This drainage panel has rounded edges on the bottom of the panel to prevent damage to the roof. Since the drainage panel has a built in roof protection system, no protection fabric is needed to protect the roofing assembly. A snapping system is incorporated into the design which allows for fast and easy installation. A root barrier is essential to prevent damage to the roofing assembly from root penetration. The RB20 root barrier selected is also from Green Roof Solutions and was selected based the high puncture and tear resistance. A lower grade root barrier can be selected for this roof since the plants are typically small. A water proof membrane beneath the root barrier ensures that no water will penetrate the roof causing leaks, damage, and mold. The Kemper System Kemperol 2K-PUR was selected because it is idea for areas such as roof gardens. The membrane is root resistant which provides extra protection from the plant roots. Rigid insulation was also chosen for this roofing assembly to further increase the energy efficiency of the building. Styrofoam Brand Highload 60 Insulation was selected from DOW Building Solutions due to its performance under the higher loads for the green roof. 2" thick insulation will be used which provides an R-value of 10. Finally a vapor barrier is required to prevent the passage of moisture to the ceiling. A Roof Aqua Guard BREA vapor barrier will be used. The material specification sheets can be found in Appendix J.

Impact on Structure

Since a green roof has much higher loads due to the additional materials and water storage capacity, they greatly impact the structure of the building. Breakdowns of the weights of the materials associated with the green roof assembly are summarized in Table 5 shown below.

Material	Weight
Vegetation	2 psf
Growing Media	17 psf
Filter Fabric	0.024 psf
Drainage Panel (Including Water)	2 psf
Root Barrier	0.05 psf
Water Proof Membrane	0.05 psf
Total	22 psf

Table 5. Weights of Green Roof Materials

The total weight for the green roof assembly is 22 psf which is less than the 30 psf assumed for the green roof dead load.

Costs

In addition to the increased dead loads on the structural system, a green roof also adds an increase in initial costs for the building. RS Means reports that a typical built up roof costs about $3.04 per square foot verses the green roof which costs $9.37 per square foot. A cost analysis for the green roof is shown in Table 6. The additional cost for the green roof will be about $412,000.00 more than a typical built up roof.

Green Roof							
	Unit	Quantity	Waste Factor	Unit Price	Labor	Equipment	Total
Vegetation	S.F.	22705.50	1.00	2.50	0.33	0.00	64256.57
Growing Medium	S.F.	22705.50	1.00	0.25	0.53	0.41	27019.55
Filter Fabric	S.F.	22705.50	1.00	0.26	3.88	0.51	105580.58
Drainage Panel	S.F.	22705.50	1.00	2.70	0.67	0.00	76517.54
Root Barrier	S.F.	22705.50	1.00	0.70	0.77	0.00	33377.09
Water Proof Membrane	S.F.	22705.50	1.00	0.26	3.88	0.51	105580.58
						Total:	$412,331.88

Table 6. Cost Analysis of Green Roof

Conclusion

The choice of an extensive green roof is ideal for this type of building since the roof reduces energy costs, has lower maintenance costs, and has the lowest impact on the structure. Even though the green roof costs significantly more than a built-up roof initially, the reduced energy costs will offset the initial costs within a few years. An extensive green roof requires little maintenance to sustain itself. The choice of plants require little upkeep and irrigation. Also, extensive green roofs have the lightest loads requiring the least impact on the structural layout.

References

- Emory Knoll Farms. "Green Roof Plants." 2014. <http://www.greenroofplants.com/>.
- Green Roof Solutions. "Green Roof Solutions." 2013. <http://www.greenroofsolutions.com/>.
- United States. Environmental Protection Agency. "Reducing Urban Heat Island: Compendium of Strategies." <http://www.epa.gov/heatislands/resources/pdf/GreenRoofsCompendium.pdf>.

Cost Analysis and Schedule Analysis Breadth

Since the building material changed from cast in place concrete to structural steel, a cost analysis and schedule analysis must be conducted in order to determine if the new system will be cost effective.

Cost Analysis

To determine if the new system is more cost effective, a cost breakdown of each structural component was established. The cost data was taken from RS Means Building Construction Data 2014. RS Means divides the cost of a specific element by the cost of the material, labor, and equipment to determine the approximate cost of an element. The cost breakdown for the original concrete structure can be seen in Table 7. The cost for the proposed steel system is shown in Table 8.

Existing Building Conditions							
Total Cost of Concrete							
	Unit	Quantity	Waste Factor	Unit Price	Labor	Equipment	Total
Beams	C.Y.	1843.33	1.05	110.00	28.00	8.65	280462.66
Columns 6000	C.Y.	1591.00	1.05	113.00	18.00	5.55	226240.20
Columns 7000	C.Y.	19.56	1.05	116.00	18.00	5.55	2843.05
Slabs	C.Y.	9298.33	1.05	110.00	15.75	4.85	1265502.71
Walls	C.Y.	1524.14	1.05	110.00	23.00	7.05	221838.58
Drop Panels	C.Y.	204.00	1.05	110.00	15.75	4.85	27764.40
Total Cost of Formwork							
	Unit	Quantity	Waste Factor	Unit Price	Labor	Equipment	Total
Beams	SFCA	98379.48	1.10	2.47	8.10	0.00	1064170.84
Columns	SFCA	69762.00	1.10	2.30	6.95	0.00	661343.76
Slabs with Drop Panels	S.F.	296828.00	1.10	4.20	4.77	0.00	2787214.92
Walls	SFCA	79457.19	1.10	2.91	7.65	0.00	862189.97
Total Cost of Reinforcement							
	Unit	Quantity	Waste Factor	Unit Price	Labor	Equipment	Total
Beams	Lb.	264258.00	1.05	0.50	0.30	0.00	218012.85
Columns	Lb.	230888.20	1.05	0.50	0.35	0.00	202027.18
Slabs	Lb.	1333333.00	1.05	0.50	0.28	0.00	1073333.07
Walls	Lb.	218499.00	1.05	0.50	0.20	0.00	158411.78

Total	$9,051,355.94

Table 7. Concrete Cost Analysis

Proposed Building Solution							
Total Cost of Steel Beams							
	Unit	Quantity	Waste Factor	Unit Price	Labor	Equipment	Total
Beams simple	ton	491.40	1.00	2750.00	455.00	131.00	1639310.40
Beams Moment	ton	158.80	1.00	3175.00	360.00	196.00	592482.80
Total Cost of Steel Columns							
	Unit	Quantity	Waste Factor	Unit Price	Labor	Equipment	Total
Columns	ton	345.30	1.00	2805.50	455.00	131.00	1171084.95
Total Cost of Steel Braces							
	Unit	Quantity	Waste Factor	Unit Price	Labor	Equipment	Total
Braces	ton	74.50	1.00	2750.00	455.00	131.00	248532.00
Total Cost of Steel Deck							
	Unit	Quantity	Waste Factor	Unit Price	Labor	Equipment	Total
Floor Deck	S.F.	274122.00	1.00	2.13	0.43	0.04	712717.20
Roof Deck	S.F.	22705.50	1.00	1.53	0.34	0.03	43140.45
Total Cost of Concrete Topping							
	Unit	Quantity	Waste Factor	Unit Price	Labor	Equipment	Total
Topping 4000	C.Y.	2746.90	1.05	104.00	18.00	5.55	364650.98
Reinforcement	C.S.F.	2741.22	1.00	17.20	26.00	0.00	118420.70
Total Cost of Steel Joists							
	Unit	Quantity	Waste Factor	Unit Price	Labor	Equipment	Total
	L.F.	2246.49	1.00	10.35	1.77	0.79	29002.19
Total Cost of Fire Proofing							
	Unit	Quantity	Waste Factor	Unit Price	Labor	Equipment	Total
Beams	S.F.	36194.20	1.00	0.60	0.81	0.12	55377.13
Columns	S.F.	19197.00	1.00	0.66	1.03	0.15	35322.48
Braces	S.F.	3809.30	1.00	0.60	0.81	0.12	5828.23
Roof Deck	S.F.	22705.50	1.00	0.86	0.93	0.14	43821.62

Joists	S.F.	1123.25	1.00	0.60	0.84	0.12	1752.27
Total Cost of Shear Connectors							
	Unit	Quantity	Waste Factor	Unit Price	Labor	Equipment	Total
Shear connectors	Ea.	33365.00	1.00	0.53	0.87	0.49	63059.85

Total	$5,124,503.23

Table 8. Steel Cost Analysis

The concrete structure costs about $9,050,000.00 which is extremely close to the cost of $9,500,000.00 obtained from the building developer. There is a difference in cost because the foundations and slab on grade where not included in the cost analysis. These two items were not redesigned and therefore not included in the cost analysis. The cost of the steel totaled at about $5,100,000.00, a drop of about $4,000,000.00 from the original design.

Schedule Analysis

Since steel erection times are much shorter than the placement of cast-in-place concrete, a schedule analysis was conducted to determine the estimated time of both systems. To develop the schedule, daily output data obtained from RS Means was used to determine to how long each element will take to construct. The schedule was developed using Microsoft Project. The start date for the construction was assumed to be February 7, 2007 since no official construction date was given. The cast-in-place concrete system took approximately 710 days to complete, assuming three crews worked eight hours a day, five days a week. The full schedule for the concrete system can be found in Appendix K. The steel system only took 189 days, assuming one crew worked eight hours a day, five days a week. The schedule for the steel system is located in Appendix L. The cast-in-place concrete system took about four times longer than the proposed steel system. This extreme difference in time reflects the cost difference between the two systems. Hand calculations to determine the amount of days each component took is located in Appendix M.

Conclusion

Based on the cost data above, the new steel system is more cost effective than the original concrete system. This reduction in cost is impacted by the amount of time it takes to erect both structures. As shown in the schedule analysis, the steel system requires about a quarter of the amount of time the concrete system takes. Therefore, the proposed steel system is a cost effective alternative compared to the cast-in-place system.

Conclusion

Oklahoma University Children's Medical Office Building was designed using a cast-in-place reinforced concrete structural system. This system is a two-way flat plate system that has drop panels located at interior columns and perimeter beams located along the exterior of the building. The lateral system is comprised of cast-in-place reinforced concrete shear walls that are located in the stairwells along the northern face of the building as well as the southwestern corner. Shear walls are also located in elevator shafts in the southeast corner and southwest corner. Two shear walls are located in the interior of the floor plan.

The structural redesign utilizes composite steel beams and girders with composite floor decking. The roof is comprised of K series joists with wide flange girders. The lateral system uses concentrically braced frames located at the shear wall locations to resist the lateral loads. Since the brace frames are not as stiff as the shear walls, additional moment frames had to be designed to carry the additional lateral loads. These frames are located along the eastern wall.

In addition to the structural depth, two breadths where conducted. The first breadth is a green roof breadth that studied an extensive green roof. Several different plants where researched that provide some color to the roof while requiring low maintenance and good drought tolerance. The typical green roof assembly was also researched to provide the best materials that will provide the lowest cost and weight on the structure.

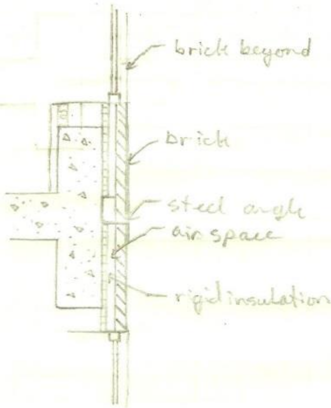
The second breadth topic is a cost and schedule analysis. The original system has a total cost of about $9,050,000.00 while the steel system has a cost of about $5,100,000.00. The amount of time that the concrete system took to construct is 710 days. The steel system only took a quarter of the time at 189 days. The steel system proved to be the most cost effective while reducing construction times.

Appendix A

appendix A — A-1

Exterior Wall

Brick Wall



Dead Load

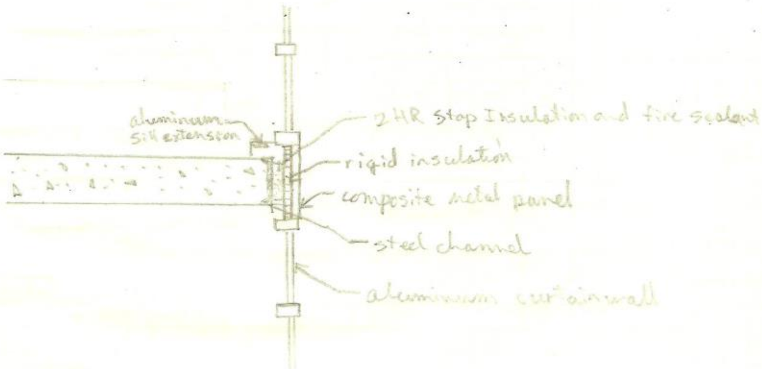
Materials
brick - 42 psf
rigid insulation (2" Thk.) - 3 psf

Total Dead Load - 45 psf · 14 ft = 630 plf (for floors 1-3)
- 45 psf · 14.67 ft = 660 plf (for floor 4)
- 45 psf · 13.115 ft = 590.2 plf (for floor 5)
- 45 psf · 12 ft = 540 plf for floor 6 - roof)

Load transfer
- brick → steel angle → structure

A-2

aluminum and Glass wall



Materials
aluminum -4 psf
rigid insulation (2" thk) -3 psf
glass (2 pane @ 1/4" thk ea.) -8 psf

Total dead load -15 psf · 14 ft = 210 plf (for floors 1-3)
-15 psf · 14.67 ft = 220 plf (for floor 4)
-15 psf · 13.115 ft = 197 plf (for floor 5)
-15 psf · 12 ft = 180 plf (for floors 6-roof)

Load Transfer
- composite metal panel → steel channel → structure

Non-typical Loads

- vehicle impact load - 6 kips at 18" above finish floor
  - 6 kips is approximately the weight of a large vehicle
- helicopter pad load - 60 psf on roof between grid 6 and 7 and G and P.2
  - weight includes the weight of the helicopter and the weight of the pad.
- ambulance load - 480 plf of load lane - designed as HS15 from AASHTO - located at the first floor at joists

Appendix B

Appendix B B-1

<u>Lateral Loads</u>

Wind (East-West Direction)

Directional Procedure

Risk category II
$V = 90$ mph
$K_d = 0.85$
Exposure category B

Topographic Factor

$K_{zt} = 1.0$ (no nearby hill, ridge, or escarpment)

Determine Fundamental Frequency

$$T_a = C_t h_n^x = 0.02 \cdot (176)^{0.75} = 0.966$$

$C_t = 0.02$
$h_n = 176'$
$x = 0.75$

$T_a = \frac{1}{n_a}$ $\quad 0.966 = \frac{1}{n_a}$ $\quad n_a = 1.04$ ∴ considered rigid

Gust Effect Factor

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

$$I_{\bar{z}} = c(33/\bar{z})^{1/6} = 0.30(33/99.6)^{1/6} = 0.25$$

$c = 0.30$
$\bar{z} = 0.6 \cdot 166 = 99.6$
$g_Q = 3.4$
$g_v = 3.4$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B + h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1 + 0.63\left(\frac{282 + 166}{462.45}\right)^{0.63}}} = 0.786$$

$B = 282$
$h = 166$

$$L_{\bar{z}} = \ell(\bar{z}/33)^{\bar{\epsilon}} = 320 \cdot (99.6/33)^{1/3} = 462.45$$

$\ell = 320$
$\bar{z} = 99.6$
$\bar{\epsilon} = 1/3$

B-2

$$G = 0.925\left(\frac{(1+1.7\cdot 3.4\cdot 0.25\cdot 0.786)}{1+1.7\cdot 3.4\cdot 0.25}\right) = 0.808$$

Building is Enclosed

$GC_{pi} = \pm 0.18$

Find K_z

For $z < 15\text{ft}$, $K_z = 2.01(15/z_g)^{2/\alpha}$

For $15\text{ft} < z < z_g$, $K_z = 2.01(z/z_g)^{2/\alpha}$

$z_g = 1200$
$\alpha = 7.0$

z	K_z
0'	0.575
7'	0.575
14'	0.575
21'	0.633
28'	0.687
35'	0.732
42'	0.771
49'	0.806
56'	0.837
63'	0.866
70'	0.892
76'	0.914
82'	0.934
88'	0.953
94'	0.971
100'	0.988
106'	1.000
112'	1.021
118'	1.036
124'	1.051
130'	1.065
136'	1.079
142'	1.092
148'	1.105
154'	1.118
160'	1.130
166'	1.142

Find q_z

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

B-3

z	K_z	q_z (psf)
0'	0.575	$0.00256 \cdot 0.575 \cdot 1.0 \cdot 0.85 \cdot 90^2 \cdot 1.0 = 10.13$
7'	0.575	10.13
14'	0.575	10.13
21'	0.633	11.16
28'	0.687	12.11
35'	0.732	12.90
42'	0.771	13.59
49'	0.806	14.21
56'	0.837	14.75
63'	0.866	15.26
70'	0.892	15.72
76'	0.914	16.11
82'	0.934	16.46
88'	0.953	16.80
94'	0.971	17.11
100'	0.988	17.41
106'	1.000	17.63
112'	1.021	18.00
118'	1.036	18.26
124'	1.051	18.52
130'	1.065	18.77
136'	1.079	19.02
142'	1.092	19.23
148'	1.105	19.48
154'	1.118	19.71
160'	1.130	19.92
166'	1.142	20.13

$p = qG_iC_p - q_i(GC_{pi})$

$q = q_z$ @ z

$G_p = 0.808$

C_p - windward wall - 0.80

C_p - Leeward wall - 0.50
$L/B = 152/282 = 0.539$

C_p - side wall - -0.70

C_p, roof
$h/L = 166/152 = 1.09 > 1.0$

from 0' to 83' → $C_p = -1.3$
→ area reduction - $83 \cdot 152 = 12616 > 1000 \therefore 0.8$
from 83' to 152' → $C_p = -0.7$

B-4

$q_i = q_h = 20.13$

$(GC_{pi}) = \pm 0.18$

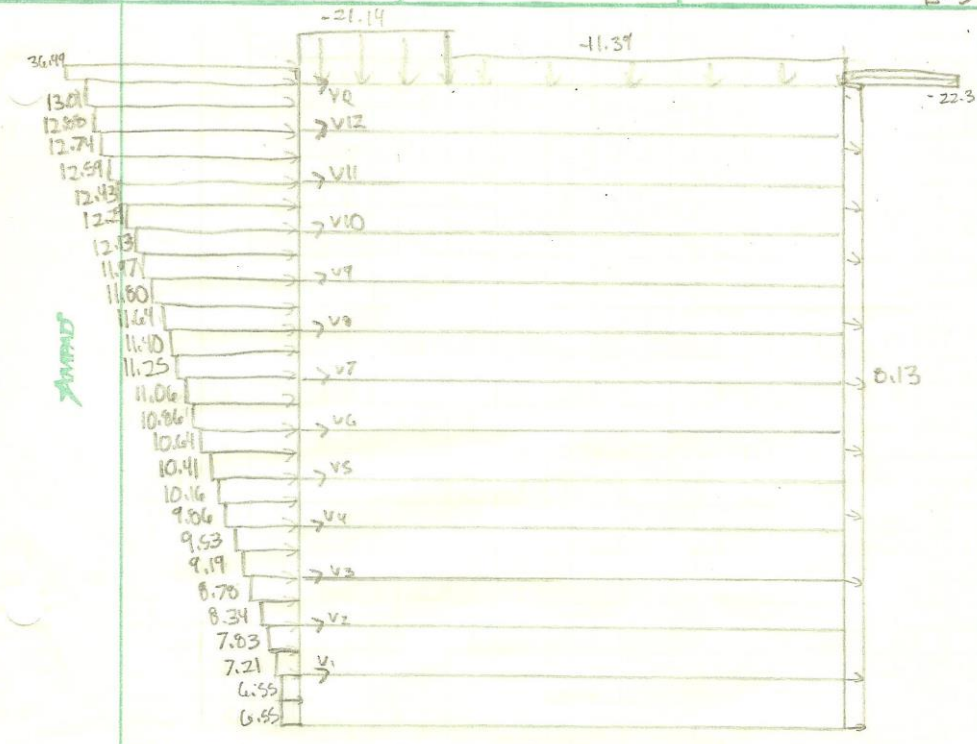
Parapets

$P_p = q_p G C_{pn}$

$q_p = 20.27 \cdot 1.8 = 36.49$ windward
$q_p = 20.27 \cdot -1.1 = -22.3$ leeward

Location	z	q	q GCP	qi(GCPi)	Net Pressure +GCpi	Net Pressure −GCPi
	0'	10.13	6.55	± 3.62	10.17	2.93
	7'	10.13	6.55	± 3.62	10.17	2.93
	14'	10.13	6.55	± 3.62	10.17	2.93
	21'	11.16	7.21	± 3.62	10.83	3.59
	28'	12.11	7.83	± 3.62	11.45	4.21
	35'	12.90	8.34	± 3.62	11.96	4.72
	42'	13.59	8.78	± 3.62	12.40	5.16
	49'	14.21	9.19	± 3.62	12.81	5.57
	56'	14.75	9.53	± 3.62	13.15	5.91
	63'	15.26	9.86	± 3.62	13.48	6.24
	70'	15.72	10.16	± 3.62	13.78	6.54
windward	76'	16.11	10.41	± 3.62	14.03	6.79
	82'	16.46	10.64	± 3.62	14.26	7.02
	88'	16.80	10.86	± 3.62	14.48	7.24
	94'	17.11	11.06	± 3.62	14.68	7.44
	100'	17.41	11.25	± 3.62	14.87	7.63
	106'	17.63	11.40	± 3.62	15.02	7.78
	112'	18.00	11.64	± 3.62	15.26	8.02
	118'	18.26	11.80	± 3.62	15.42	8.18
	124'	18.52	11.97	± 3.62	15.59	8.36
	130'	18.77	12.13	± 3.62	15.75	8.51
	136'	19.02	12.29	± 3.62	15.91	8.67
	142'	19.23	12.43	± 3.62	16.05	8.81
	148'	19.48	12.59	± 3.62	16.21	8.97
	154'	19.71	12.74	± 3.62	16.36	9.12
	160'	19.92	12.88	± 3.62	16.50	9.26
	166'	20.13	13.01	± 3.62	16.63	9.39
leeward	166'	20.13	8.13	± 3.62	11.75	4.51
side wall	166'	20.13	-11.39	± 3.62	-7.77	-15.01
Roof (0' to 83')	166'	20.13	-21.14	± 3.62	-17.52	-24.76
Roof (83' to 152')	166'	20.13	-11.39	± 3.62	-7.77	-15.01

B-5



$V = B \cdot (p_{wind} + p_{lee})(\text{trib area})$

$V_1 = 282 \cdot (6.55+8.13)(7) + 282 \cdot (7.21+8.13) \cdot (7) = 59.3\text{k}$

$V_2 = 282 \cdot (7.83+8.13)(7) + 282 \cdot (8.34+8.13) \cdot (7) = 64\text{k}$

$V_3 = 282 \cdot (8.78+8.13)(7) + 282 \cdot (9.19+8.13) \cdot (7) = 67.6\text{k}$

$V_4 = 282 \cdot (9.53+8.13)(7) + 282 \cdot (9.86+8.13) \cdot (7) = 70.4\text{k}$

$V_5 = 282 \cdot (10.16+8.13)(7) + 282 \cdot (10.41+8.13)(6) = 67.5\text{k}$

$V_6 = 282 \cdot (10.64+8.13)(6) + 282(10.86+8.13)(6) = 63.9\text{k}$

$V_7 = 282 \cdot (11.06+8.13)(6) + 282(11.25+8.13)(6) = 65.3\text{k}$

$V_8 = 282 \cdot (11.40+8.13)(6) + 282(11.64+8.13)(6) = 66.5\text{k}$

$V_9 = 282 \cdot (11.80+8.13)(6) + 282(11.97+8.13)(6) = 67.7\text{k}$

B-6

$V_{10} = 282 \cdot (12.13 + 8.13)(6) + 282(12.29 + 8.13)(6) = 68.8k$

$V_{11} = 282 \cdot (12.43 + 8.13)(6) + 282(12.59 + 8.13)(6) = 69.8k$

$V_{12} = 282 \cdot (12.74 + 8.13)(6) + 282(12.88 + 8.13)(6) = 70.9k$

$V_R = 282 \cdot (13.01 + 8.13)(6) + 282(36.49 + 8.13)(6) = 111.3k$

$V_{hs} = 59.3 + 64 + 67.6 + 70.4 + 62.3 + 63.9 + 65.3 + 66.5 + 67.7 + 68.8 + 69.8 + 70.9 + 111.3 = 907.8k$

AMPAD

B-7

Wind (North - South Direction)

Analytical Procedure

Risk Category - II
$V = 90$ mph
$K_d = 0.85$
Exposure Category B

Topographic factor

$K_{zt} = 1.0$ (no nearby hill, ridge, or escarpment)

Determine Fundamental Frequency

$T_a = C_t h_n^x = 0.02 \cdot (176)^{0.75} = 0.966$

$C_t = 0.02$
$h_n = 176$
$x = 0.75$

$T_a = \frac{1}{n_a}$ $\quad 0.996 = \frac{1}{n_a}$ $\quad n_a = 1.04$ ∴ considered rigid

Gust Effect Factor

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

$I_{\bar{z}} = c(33/\bar{z})^{1/6} = 0.30(33/99.6)^{1/6} = 0.25$

$c = 0.30$
$\bar{z} = 0.6 \cdot 166 = 99.6$
$g_Q = 3.4$
$g_v = 3.4$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1 + 0.63\left(\frac{152+166}{462.45}\right)^{0.63}}} = 0.817$$

$B = 152$
$h = 166$
$L_{\bar{z}} = \ell(\bar{z}/33)^{\bar{\epsilon}} = 320 \cdot (99.6/33)^{1/3} = 462.45$

$\ell = 320$
$\bar{z} = 99.6$
$\bar{\epsilon} = 1/3$

B-6

$G = 0.925\left(\frac{(1+1.7\cdot3.4\cdot0.25\cdot0.917)}{1+1.7\cdot3.4\cdot0.25}\right) = 0.825$

Building is Enclosed

$GC_{pi} = \pm 0.18$

Find K_z

for $z < 15ft$, $K_z = 2.01(15/z_g)^{2/\alpha}$ $z_g = 1200$

For $15ft < z < z_g$, $K_z = 2.01(z/z_g)^{2/\alpha}$ $\alpha = 7.0$

z	K_z
0'	0.575
7'	0.575
14'	0.575
21'	0.633
28'	0.687
35'	0.732
42'	0.771
49'	0.806
56'	0.837
63'	0.866
70'	0.892
76'	0.914
82'	0.934
88'	0.953
94'	0.971
100'	0.988
106'	1.000
112'	1.021
118'	1.036
124'	1.051
130'	1.065
136'	1.079
142'	1.092
148'	1.105
154'	1.118
160'	1.130
166'	1.147

Find q_z

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

B-9

z	kz	qz (psf)
0'	0.575	0.00256 · 0.575 · 1.0 · 0.85 · 90² · 1.0 = 10.13
7'	0.575	10.13
14'	0.575	10.13
21'	0.633	11.16
28'	0.687	12.11
35'	0.732	12.90
42'	0.771	13.59
49'	0.806	14.21
56'	0.837	14.75
63'	0.866	15.26
70'	0.892	15.72
76'	0.914	16.11
82'	0.934	16.46
88'	0.953	16.80
94'	0.971	17.11
100'	0.988	17.41
106'	1.000	17.63
112'	1.021	18.00
118'	1.036	18.26
124'	1.051	18.52
130'	1.065	18.77
136'	1.079	19.02
142'	1.092	19.23
148'	1.105	19.48
154'	1.118	19.71
160'	1.130	19.92
166'	1.142	20.13

$p = qG_iC_p - q_i(GC_{pi})$

$q = q_z$ @ z

$G_p = 0.825$

C_p - windward wall - 0.80

C_p - leeward wall = -0.328
$L/B = 282/152 = 1.86$

C_p - side wall - -0.70

C_p; roof
$h/L = 166/282 = 0.589$

. from 0' to 83' → C_p = -0.972
from 83' to 166' → C_p = -0.864
from 166' to 282 → C_p = -0.536

B-10

$q_i = q_h = 20.13$

$(G C_{pi}) = \pm 0.18$

Parapets

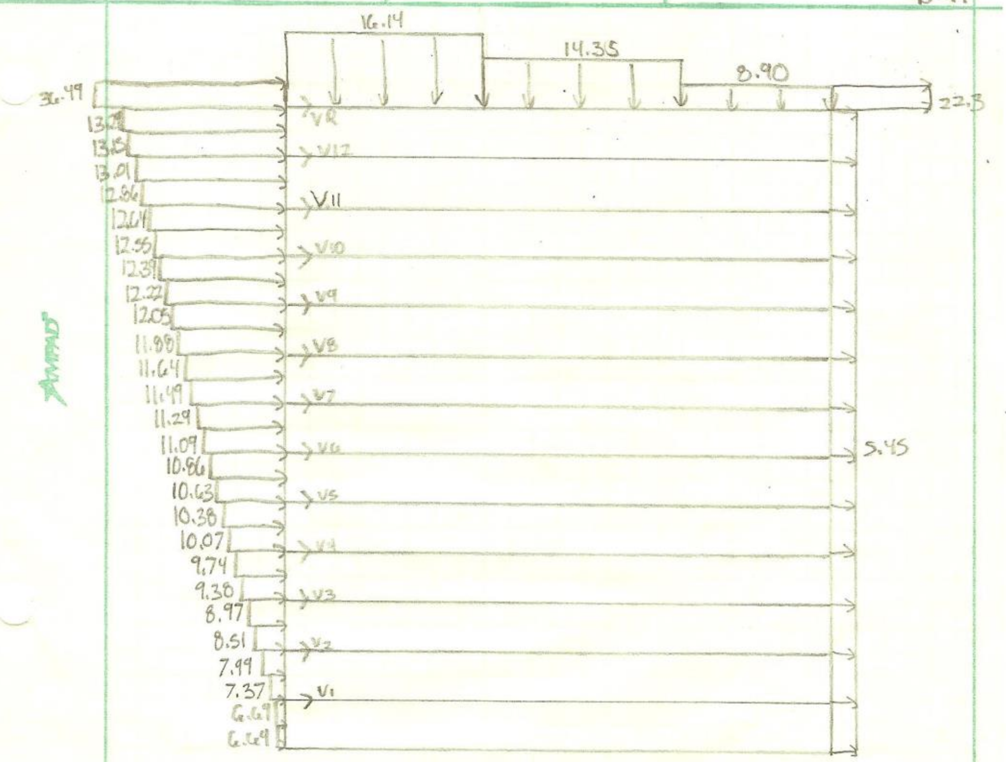
$P_p = q_p \, G C_{pn}$

$q_p = 20.27 \cdot 1.8 = 36.49$ windward

$q_p = 20.27 \cdot -1.1 = -22.3$ leeward

Location	z	q	qGCp	$q_i(GC_{pi})$	Net Pressure +GCpi	Net Pressure -GCpi
windward	0'	10.13	6.69	±3.62	10.31	3.07
	7'	10.13	6.69	±3.62	10.31	3.07
	14'	10.13	6.69	±3.62	10.31	3.07
	21'	11.16	7.37	±3.62	10.99	3.75
	28'	12.11	7.99	±3.62	11.61	4.37
	35'	12.90	8.51	±3.62	12.13	4.89
	42'	13.59	8.97	±3.62	12.59	5.35
	49'	14.21	9.38	±3.62	13.00	5.76
	56'	14.75	9.74	±3.62	13.36	6.12
	63'	15.26	10.07	±3.62	13.69	6.45
	70'	15.72	10.38	±3.62	14.00	6.76
	76'	16.11	10.63	±3.62	14.25	7.01
	82'	16.46	10.86	±3.62	14.48	7.24
	88'	16.80	11.09	±3.62	14.71	7.47
	94'	17.11	11.29	±3.62	14.91	7.67
	100'	17.41	11.49	±3.62	15.11	7.87
	106'	17.63	11.64	±3.62	15.26	8.02
	112'	18.00	11.88	±3.62	15.50	8.26
	118'	18.26	12.05	±3.62	15.67	8.43
	124'	18.52	12.22	±3.62	15.84	8.60
	130'	18.77	12.39	±3.62	16.01	8.77
	136'	19.02	12.55	±3.62	16.17	8.93
	142'	19.23	12.69	±3.62	16.31	9.07
	148'	19.48	12.86	±3.62	16.48	9.24
	154'	19.71	13.01	±3.62	16.63	9.39
	160'	19.92	13.15	±3.62	16.77	9.53
	166'	20.13	13.29	±3.62	16.91	9.67
leeward	166'	20.13	-5.45	±3.62	9.07	1.83
side wall	166'	20.13	-11.63	±3.62	-8.01	-15.25
Roof (0'-83')	166'	20.13	-16.14	±3.62	-12.52	-19.76
Roof (83'-166')	166'	20.13	-14.35	±3.62	-10.73	-17.97
Roof (166'-282')	166'	20.13	-8.90	±3.62	-5.28	-12.52

B-11



$V = B\cdot(p_{wind} + p_{lee})(\text{trib area})$

$V_1 = 152\cdot(6.69+5.45)(7) + 152\cdot(7.37+5.45)(7) = 26.6k$

$V_2 = 152\cdot(7.99+5.45)(7) + 152\cdot(8.51+5.45)(7) = 29.2k$

$V_3 = 152\cdot(8.97+5.45)(7) + 152\cdot(9.38+5.45)(7) = 31.1k$

$V_4 = 152\cdot(9.74+5.45)(7) + 152\cdot(10.07+5.45)(7) = 32.7k$

$V_5 = 152\cdot(10.38+5.45)(7) + 152\cdot(10.63+5.45)(6) = 31.5k$

$V_6 = 152\cdot(10.86+5.45)(6) + 152\cdot(11.09+5.45)(6) = 30.0k$

$V_7 = 152\cdot(11.29+5.45)(6) + 152\cdot(11.49+5.45)(6) = 30.7k$

$V_8 = 152\cdot(11.64+5.45)(6) + 152\cdot(11.88+5.45)(6) = 31.4k$

$V_9 = 152\cdot(12.05+5.45)(6) + 152\cdot(12.22+5.45)(6) = 32.1k$

B-12

$V_{10} = 152 \cdot (12.39 + 5.45)(6) + 152 \cdot (12.55 + 5.45)(6) = 32.7k$

$V_{11} = 152 \cdot (12.64 + 5.45)(6) + 152 \cdot (12.86 + 5.45)(6) = 33.2k$

$V_{12} = 152 \cdot (13.01 + 5.45)(6) + 152 \cdot (13.15 + 5.45)(6) = 33.8k$

$V_R = 152 \cdot (13.29 + 5.45)(6) + 152 \cdot (36.49 + 5.45)(6) = 55.3k$

$V_{ns} = 24.6 + 29.2 + 31.1 + 32.7 + 29.1 + 30.0 + 30.7 + 31.4 + 32.1 + 32.7 + 33.2 + 33.8 + 55.3 = 427.9k$

Appendix C

Appendix C

C-1

Seismic Loads

Site Class C - From Geotechnical Report
-soft rock conditions

S_s - 0.406g

S_1 - 0.089g

S_{MS} - 0.490g

S_{M1} - 0.152g

S_{DS} - 0.327g

S_{D1} - 0.101g

Seismic Design Category

$0.167g < S_{DS} < 0.33g \Rightarrow$ Category B
$0.067g < S_{D1} < 0.133g \Rightarrow$ Category B
Use B

Use Equivalent Lateral Force Procedure

Ordinary Concentric Braced Frames - $\underbrace{R=3.25,\ \Omega_0=2,\ C_d=3.25}$ → controls

Ordinary Moment Frames - $R=3.5$, $\Omega_0=3$, $C_d=3$

* Note - Seismic details must conform to ASCE 7-10 14.1

$I_e=1.0$

$T_a = C_t h_n^x$

$C_t=0.02$
$x=0.75$
$h_n=166\text{ ft}$

$T_a = 0.02 \cdot 166^{0.75} = 0.925$ secs, $T = 1.569$ secs

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{0.327}{\left(\frac{3.25}{1.0}\right)} = 0.101$$

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \frac{0.101}{1.569\left(\frac{3.25}{1.0}\right)} = 0.02$$

$C_s = 0.044 S_{DS} I = 0.044 \cdot 0.327 \cdot 1.0 = 0.0144 \geq 0.01$

$C_s = 0.02$

C-2

Total building Weight = 22300 kips

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

$$k = \frac{1.569 - 0.5}{2.5 - 0.5}(2-1) + 1 = 1.53$$

$V = C_s W = 22300 \cdot 0.02 = 446$ kips

Level	h_x	W_x	$W_x h_x^k$	C_{vx}	F_x	V_x
Roof	166	144	359028	0.135	60.2	60.2
12	154	154	342316	0.129	57.5	117.7
11	142	154	315642	0.119	53.1	170.8
10	130	154	288968	0.109	48.6	219.4
9	118	154	262294	0.099	44.2	263.6
8	106	154	235620	0.089	39.7	303.3
7	94	154	208946	0.078	34.8	338.1
6	82	154	182272	0.068	30.3	368.4
5	70	154	155598	0.058	25.9	394.3
4	56	154	124479	0.047	21.0	415.3
3	42	154	93359	0.035	15.6	430.9
2	28	154	62239	0.023	10.3	441.2
1	14	154	31119	0.012	5.4	446.6

Appendix D

Appendix D D-1

Alternate System: Composite Steel (Floor)

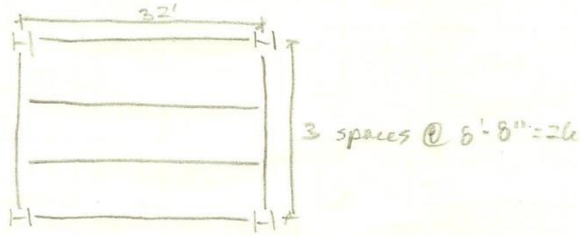
Loading

Live Load

Office = 50 + 20 = 70 psf
Corridor = 80 psf → used for building flexibility

Dead Load

Carpet with pad - 2 psf
Superimposed Dead Load - 15 psf



Determine Deck size (Composite w/ LW concrete)

- use 3 1/4" LW concrete for 2 Hr fire rating
- use unshored construction for more economical design

Try 1.5 VLR19

- 3 span construction - 9'-9" > 8'-8" ok
  Superimposed Load - 97 psf
  Superimposed Live Load for 9'-0" span - 197 psf
  197 > 97 ∴ OK
- use 1.5 VLR 19 gauge with 3 1/4" LW topping

Determine Beam Size

Loads

Live Load - 80 psf (reducable)

D-2

$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) \geq 0.5L_o$

$= 80\left(0.25 + \frac{15}{\sqrt{\underbrace{17.333 \cdot 32}_{555 \geq 400 \checkmark}}}\right) = 80 \cdot 0.887 = 70.96$ psf

Dead Load

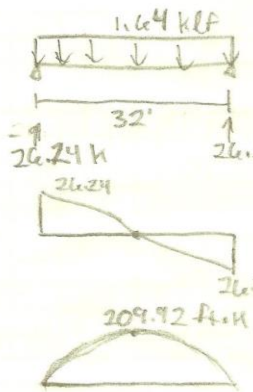
Deck - 41 psf
Carpet with pad - 2 psf
Superimposed dead Load - 15 psf
beam self weight allowance - 5 psf

Total - 63 psf

$w_u = 1.2(63) + 1.6(70.96) = 189.14$ psf

$w_u = \frac{189.14 \cdot 8.67}{1000} = 1.64$ klf

Shear and Moment Diagrams



Determine ϕM_n and ΣQ_n

Use table 3-19 from AISC 14th ed.
- assume $f'c = 4000$ psi
- assume $a \approx 1$
- assume deck is perpendicular
- assume 1 weak stud per rib

D-3

- assume 3/4" stud
- assume 1 stud per foot

If $a \approx 1$, $y = 4.75 - 1/2 = 4.25"$ Try 4.5"

most economical → W14x22 $\Sigma Q_n = 157 = \frac{157}{17.1} = 9.18 \Rightarrow 10 \times 2 = 20$ studs/beam
$\phi M_n = 218$ ft·k $= 904$ #

W12x26 $\Sigma Q_n = 116 = \frac{116}{17.1} = 6.78 \Rightarrow 7 \times 2 = 14$ studs/beam
$\phi M_n = 210$ ft·k $= 972$ #

W12x22 $\Sigma Q_n = 196 = \frac{196}{17.1} = 11.46 \Rightarrow 12 \times 2 = 24$ studs/beam
$\phi M_n = 214$ ft·k $= 944$ #

W16x26 $\Sigma Q_n = 96 = \frac{96}{17.1} = 5.61 \Rightarrow 6 \times 2 = 12$ studs/beam
$\phi M_n = 241$ ft·k $= 952$ #

$a = \frac{\Sigma Q_n}{0.85 F'_c b_{eff}}$

$b_{eff} = \min \left| \begin{array}{l} \frac{32 \cdot 12}{8} = 48 \\ \frac{8.67 \cdot 12}{2} = 52.02 \end{array} \right. + \min \left| \begin{array}{l} \frac{32 \cdot 12}{8} = 48 \\ \frac{8.67 \cdot 12}{2} = 52.02 \end{array} \right.$

$b_{eff} = 48 + 48 = 96$ in

$a = \frac{157}{0.85 \cdot 4 \cdot 96} = 0.481 \therefore y_{actual} = 4.75 - \frac{0.481}{2} = 4.51$

use $y_2 = 5.0"$
$\Sigma Q_n = 157 \Rightarrow 20$ studs/beam
$\phi M_n = 223$

Check Unshored Strength

W14x22, $\phi M_n = 218$ ft·k

$w_u = 1.4(41)(8.67) + 1.4(22) = 0.528$ klf

$w_u = 1.2[(41)(8.67) + 22] + 1.6(20)(8.67) = 0.730$ klf

$M_u = \frac{0.730 \cdot 32^2}{8} = 93.44$ ft·k < 218 ft·k ⇒ OK for unshored construction

Check wet concrete deflection

$w_{wc} = 41(8.67) + 22 = 0.377$ klf

D-4

$\Delta_{WC} = \frac{5(0.377)(32)^4(1728)}{384(29000)(199)} = 1.54$ in

$\Delta_{WC\,max} = \frac{32 \cdot 12}{240} = 1.6$ in > 1.54 in ∴ Ok but should chamber
0.8(1.54) = 1" chamber

Check Live Load Deflection

$w_{LL} = 70.96 \cdot 8.67 = 0.615$ klf

I_{LB} @ $y = 5.0$ and $\Sigma Q_n = 157$

$I_{LB} = 496$

$\Delta_{LL} = \frac{5(0.615)(32)^4(1728)}{384(29000)(496)} = 1.01$ in

$\Delta_{LL\,max} = \frac{L}{360} = \frac{32 \cdot 12}{360} = 1.07$ in > 1.01 in ∴ Ok but should chamber 0.8(1.01) = 1"

Use W14x22 with 20 studs per beam with a 1" chamber

Determine Girder Size

Load

Live Load - 80 psf (reduceable)

$L = L_0\left(0.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right) \geq 0.5 L_0$

$= 80\left(0.25 + \frac{15}{\sqrt{\underbrace{26 \cdot 32 \cdot 2}_{1664 > 400}}}\right) = 80 \cdot 0.618 = 49.44$ psf

Dead Load

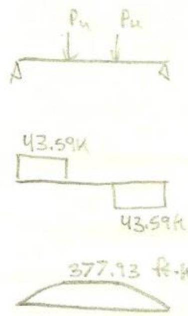
Deck - 41 psf
Carpet with pad - 2 psf
beam self weight allowance - 5 psf
Girder self weight allowance - 2 psf
Superimposed dead load - 15 psf

$w_u = 1.2(65) + 1.6(49.44) = 157.1$

$P_u = 157.1(8.67)(32) = 43.59$ k

D-5

Shear and Moment Diagrams



Determine ϕM_n and ΣQ_n
Use Table 3-19

- assume f'c = 4000 psi
- assume y = 5"
- assume deck is parallel
- assume 1 weak stud per rib
- assume 3/4" stud
- assume 1 stud per foot

$y = 5.0''$

W16x31 $\Sigma Q_n = 335 = \frac{335}{18.3} = 19 \times 2 = 38$ studs per girder
$\phi M_n = 397$ k·ft = 1186 #

W16x40 $\Sigma Q_n = \frac{148}{18.3} = 8.1 \Rightarrow 9 \cdot 2 = 18$ studs per girder
$\phi M_n = 391$ k·ft = 1220 #

W14x38 $\Sigma Q_n = \frac{299}{18.3} = 16.3 \Rightarrow 17 \cdot 2 = 34$ studs per girder
$\phi M_n = 404$ k·ft = 1328 #

Check unshored strength

$P_u = [1.2[(41 \cdot 8.67) + 31] + 1.6(20 \cdot 8.67)]32 = 23.72k$

$M_u = 23.72k \cdot 8.67 = 205.7$ ft·k < 397 k·ft Ok

Check Live Load Deflection

$\Delta_{LL} = \frac{13.72 \cdot 26^3 \cdot 1728}{28 \cdot 29000 \cdot 1020} = 0.503$ in

Δ_{LL} max $= \frac{L}{360} = \frac{26 \cdot 12}{360} = 0.867$ in > 0.503 in ∴ Ok

$\boxed{\text{Use W16x31 with 38 studs}}$

Appendix E

Appendix E

E-1

alternate system : steel Joists (Roof)

Loading

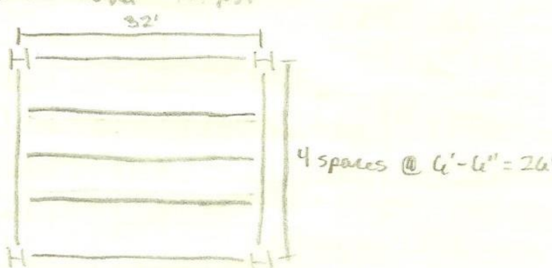
Live Load

Roof Live Load - 20 psf (unreducable)

Dead Loads

- Green Roof - assuming 30 psf (Materials to be determined as part of breadth)
- Metal Deck - 3 psf
- superimposed Dead Load - 15 psf
- rigid insulation - 2 psf

Snow Load - 10 psf



- Total load - 70 = w_{TL}

Use 1.5 B deck

- max construction span ⇒ B22 - 6'-11"
- max load → B22 = 74 > 70 psf
- max load for $\frac{\Delta}{240}$ → B22 = $76 \cdot \frac{240}{180}$ = 101.3 psf > 70 psf

Use 1.5 B 22 guage decking

Determine Joist Size

$w_{utl} = [1.2(50) + 1.6(20)]\, 6.5\text{ ft} = 598$ plf + 1.2 · joist wt

$w_{tl} = (50+20)(6.5) = 455$ plf + joist wt

E-2

Try 24K9: $w_{ult} = 717\ plf < 598 + 1.2 \cdot 12 = 612.4 \therefore OK$

w for $L/360 = 344\ plf$

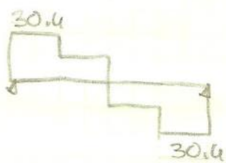
w for $L/240 = 344\ plf \cdot 1.5 = 516 > 455 + 12 = 467\ plf \therefore OK$.

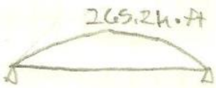
Use 24K9 joists @ 6'-6"

Determine Girder Size

$P_u = (598\ plf + 40\ plf)\left(\frac{32+32}{2}\right)/1000 = 20.4$

Shear and Moment Diagrams





Use Z_x tables from AISC 14th ed.

$W18 \times 40 - 294\ k \cdot ft > 265.2k \cdot ft$

$\Delta LL = \frac{P_{LL} L^3}{28 E I_x} = \frac{6.66 \cdot 26^3}{28 \cdot 29000 \cdot 612} = 0.000236\ in < \frac{26 \cdot 12}{360} = 0.8667 \therefore OK$

Check unbraced length

$L_b = 6.5\ ft < L_p = 6.67 \therefore OK$

Use W18 x 40 for girders

Appendix F

Appendix F

F-2

Gravity Column Check

Check Column P.6, 6

Area: 23 ft · 32 ft = 736 ft²

Loading

Dead load

Deck - 41 psf
Carpet with pad - 2 psf
Superimposed dead Load - 15 psf
beam self weight allowance - 5 psf
girder self weight allowance - 2 psf
column self weight allowance - 1 psf
Helicopter pad - 60 psf

Live Load

$= 80\left(0.25 + \frac{15}{\sqrt{\underbrace{46 \cdot 64}_{2944 > 400}}}\right) = 80 \cdot 0.526 = 42.08$

Snow Load

10 psf

Roof Live Load

20 psf

Dead Load Roof

- Green Roof - 30 psf
- Metal Deck - 3 psf
- superimposed Dead Load - 15 psf
- rigid insulation - 2 psf

Roof

$-1.2 \cdot 50 + 1.6 \cdot 20 = 92$ psf

Floor

$-1.2 \cdot 66 + 1.6 \cdot 42.08 = 146.5$ psf

$P_u = 736 \cdot 92 + 736 \cdot 146.5 \cdot 12 = 1361.6$ Kips
$KL = 14$

W14x132 – $\phi P_n = 1510$ Kips > 1361.6 kips $\therefore$ OK

Appendix G

Appendix G

G-1

Vibration Calculations

- Due to people walking
  - $\frac{a_p}{g} = \frac{P_o \exp(-0.35 f_n)}{\beta W} \leq \frac{a_o}{g}$
  - office occupancy with full height partitions
  - $P_o = 65$ lbs
  - $\beta = 0.05$
  - $\frac{a_o}{g} = 0.5\%$
  - $f_n = 0.18\sqrt{\frac{g}{\Delta_j + \Delta_g}}$
    - $g = 386.4\ in/sec^2$
    - $\Delta_j = \frac{5 w_j L_j^4}{384 E_s I_j}$
      - beam properties
        - $A = 6.49\ in^2$
        - $I_x = 199\ in^4$
        - $d = 13.7\ in$

$0.4L_j = 0.4 \cdot 32 \cdot 12 = 153.6\ in > 104\ in$ ∴ full width is effective

$E_c = 33 w_c^{1.5} \sqrt{f'_c}$

- $w_c = 110$

$E_c = 33 \cdot 110^{1.5} \sqrt{4000} = 2{,}407{,}870$

$n = E_s / 1.35 E_c = \frac{29{,}000{,}000}{1.35 \cdot 2{,}407{,}870} = 8.92$

$\bar{y} = \frac{6.49(1.5 + 13.7/2) - (104/8.92)(3.25)(3.25/2)}{6.49 + (104/8.92)(3.25)}$

$= -0.164$ in above top of form deck

$I_j = 199 + 6.49(1.5 + 13.7/2 + 0.164)^2 + (104/8.92)(3.25)^3/12 + (104/8.92)(3.25)(-0.164 + 3.25/2)^2$

$= 783.68\ in^4$

$w_j = 8.67(11 + 4 + 41 + 22/8.67) = 507.5$ plf

G-2

$$\Delta_j = \frac{5 w_j L_j^4}{384 E_s I_j} = \frac{5 \cdot 507.5 \cdot 32^4 \cdot 1728}{384 \cdot 29000000 \cdot 783.68} = 0.527 \text{ in}$$

$$\Delta_g = \frac{5 w_g L_g^4}{384 E_s I_g}$$

girder properties

$A = 9.13 \text{ in}^2$
$I_x = 375 \text{ in}^4$
$d = 15.9 \text{ in}$

$0.4 L_g = 0.4 \cdot 26 \cdot 12 = 124.8 \text{ in} < L_j = 32 \cdot 12 = 384 \text{ in}$

$$\bar{y} = \frac{9.13(0.75 + 15.9/2) - (125/8.92)(4)(4/2)}{9.13 + (125/8.92)(4)}$$

$= -0.501 \text{ in}$ above top of form deck

$$I_g = 375 + 9.13(0.75 + 15.9/2 + 0.501)^2 + (125/8.92)(4)^3/12 + (125/8.92)(4)(-0.501 + 4/2)^2$$

$= 1348.6 \text{ in}^4$

$w_g = L_j(w_j/s) + \text{self weight}$

$= 32(507.5/8.67) + 31 = 1904.13 \text{ plf}$

$$\Delta_g = \frac{5 \cdot 1904.13 \cdot 26^4 \cdot 1728}{384 \cdot 29000000 \cdot 1348.6} = 0.5 \text{ in}$$

$$f_n = 0.18\sqrt{\frac{386.4}{0.527 + 0.5}} = 3.49 \text{ Hz}$$

$$W = \frac{\Delta_j}{\Delta_j + \Delta_g} W_j + \frac{\Delta_g}{\Delta_j + \Delta_g} W_g$$

$W_j = 1.5(w_j/S)(B_j L_j)$

$B_j = C_j (D_s/D_j)^{1/4} L_j$

$C_j = 2.0$

$D_s = 12 d_e^3/(12n) = 12 \cdot (4)^3/(12 \cdot 8.92) = 7.17 \text{ in}^4/\text{ft}$

$D_j = I_j/S = 783.68/8.67 = 90.39 \text{ in}^4/\text{ft}$

$B_j = 2.0(7.17/90.39)^{1/4} \cdot 32 = 33.96 \text{ ft} < 78 \text{ ft}$

G-3

$W_j = 1.5(507.5/8.67)(33.96 \cdot 32) = 95{,}417\text{ lbs} = 95.4\text{ Kips}$

$W_g = (w_g/L_j)B_gL_g$

$B_g = C_g(D_j/D_g)^{1/4}L_g$

$D_g = I_g/L_j = 1348.6/32 = 42.14\text{ in}^4/\text{ft}$

$C_g = 1.8$

$B_g = 1.8(90.39/42.14)^{1/4} \cdot 26 = 56.64\text{ ft}$

$W_g = (1904.13/32)56.64 \cdot 26 = 87628 = 87.6\text{ Kips}$

$W = \frac{0.527}{0.527+0.5} \cdot 95.4k + \frac{0.5}{0.527+0.5} \cdot 87.6k = 48.95 + 42.65 = 91.6\text{ Kips}$

$\frac{a_p}{g} = \frac{65\,\exp(-0.35 \cdot 3.49)}{0.05 \cdot 91.6 \cdot 1000} = 0.0042 < 0.005\ \text{Ok}$

Appendix H

Appendix H H-1

Brace Frame Check

column: W12x106

$I_x = 933\ in^4$

Controlling Load Case

$1.2D + 0.5L + 1.0W$

Loads

$P_u = 923.14\ k$

$M_{ux\,bottom} = 0.0$

$M_{ux\,top} = 9.14\ k\text{-}ft$

$M_r = B_1 M_{nt} + B_2 M_{lt}$

$$B_1 = \frac{C_m}{1 - \frac{\alpha P_r}{P_{el}}} \geq 1$$

$C_m = 0.6 - 0.4(0/9.14)$

$C_m = 0.6$

$\alpha = 1.0$

$$P_{el} = \frac{\pi^2 EI}{(KL)^2} = \frac{\pi^2 (0.8)(29000)(933)}{(1.0 \cdot 14 \cdot 12)^2} = 7569.2\ k$$

$P_r = 923.14\ k$

$$B_1 = \frac{0.6}{1 - \frac{1.0 \cdot 923.14}{7569.2}} \geq 1$$

$= 0.683 \geq 1 \therefore$ use 1.0

$$B_2 = \frac{1}{1 - \frac{\alpha P_{story}}{P_{e\,story}}} \geq 1$$

$\alpha = 1.0$

$P_{story} = 39{,}600\ k$

$P_{e\,story} = R_m \frac{HL}{\Delta H}$

H-2

$R_m = 1 - 0.15 \frac{P_{mf}}{P_{story}}$

$P_{mf} = 1527$

$R_m = 1 - 0.15 \cdot \frac{1527}{39{,}800} = 0.994$

$H = 1.0W$
$W = 390.58$

$H = 390.58$
$\Delta H = 0.0872 \text{ in}$

$P_{estory} = 0.994 \cdot \left(\frac{390.58 \cdot 14 \cdot 12}{0.0872} \right) = 747{,}979$

$B_2 = \frac{1}{1 - \left(\frac{1 \cdot 39{,}800}{747979} \right)} = 1.06 \geq 1$

$M_r = 1.0(0.0) + 1.06 \cdot 9.14$
$= 9.69 \text{ k}\cdot\text{ft}$

$P_r = P_{nt} + B_2 P_{lt}$
$= 923.14 + 1.06(0)$
$= 923.14$

$P_c = 1130 \text{ k}$
$M_{cx} = 615 \text{ k-ft}$

$\frac{P_r}{P_c} = \frac{923.14}{1130} = 0.817 \geq 0.2$

$\frac{P_r}{P_c} + \left(\frac{8}{9}\right)\left(\frac{M_{rx}}{M_{cx}}\right) \leq 1.0$

$\frac{923.14}{1130} + \left(\frac{8}{9}\right)\left(\frac{9.69}{615}\right) = 0.831 \leq 1.0 \therefore$ ok RAM output = 0.829

Beam: W18x40

$M_{ux} = 84.59 \text{ k-ft}$ (From RAM output)

$\phi M_p = 294 \text{ k-ft} > 84.59 \text{ k-ft} \therefore$ ok

H-3

Check unbraced length

$L_b = 20.10$ ft
$\phi M_n = 103$ k-ft for unbraced Length

103 k-ft > 84.59 k-ft

Brace : HSS 8x8x5/16

KL = 27.0 ft

Controlling Load Combination
0.9D - 1.0W

P_u = 79.96 kips (From RAM output)

ϕP_n = 167 kips > 79.96 kips ∴ OK

Appendix I

Appendix I

Moment Frame Check

column : W12x136

$I_x = 1240\ in^4$

Controlling Load Case

$1.467D + 0.5L + 1.0Q_e$

Loads

$P_u = 1174\ k$

$M_{u\,bottom} = 0.00\ k\text{-}ft$

$M_{ux\,top} = -43.12\ k\text{-}ft$

$M_r = B_1 M_{nt} + B_2 M_{lt}$

$$B_1 = \frac{C_m}{1 - \frac{\alpha P_r}{P_{el}}} \geq 1$$

$$\begin{aligned} C_m &= 0.6 - 0.4\,(M_{bottom}/M_{top}) \\ &= 0.6 - 0.4\,(0/-43.99) \\ &= 0.6 - 0.4(0) \\ &= 0.6 \end{aligned}$$

$\alpha = 1.0$

$$P_{el} = \frac{\pi^2 EI}{(K_1 L)^2} = \frac{\pi^2 (0.8)(29000)(1240)}{(1.0 \cdot 14 \cdot 12)^2} = 10{,}060\ \text{Kips}$$

$P_r = 1174\ k$

$$B_1 = \frac{0.6}{1 - \frac{1.0 \cdot 1174}{10{,}060}} \geq 1$$

$= 0.679 \geq 1.0 \therefore$ use 1.0

$$B_2 = \frac{1}{1 - \frac{\alpha P_{story}}{P_{estory}}} \geq 1$$

$\alpha = 1.0$

$P_{story} = 39{,}800\ k$

$P_{estory} = R_m \frac{HL}{\Delta H}$

$R_m = 1 - 0.15 \frac{P_{mf}}{P_{story}}$

I-2

$P_{mf} = 5400\ k$

$R_m = 1 - 0.15 \cdot \frac{5400}{39{,}800} = 0.979$

$H = 1.0\ Q_E$

$Q_E = 358.32$

$H = 358.32$

$\Delta H = 0.1360\ in$

$P_{estory} = 0.979 \cdot \left(\frac{358.32 \cdot 14 \cdot 12}{0.1360}\right) = 433335\ k$

$B_2 = \frac{1}{1 - \left(\frac{1 \cdot 39{,}800}{433335}\right)} \geq 1$

$= 1.10$

$M_r = 1.0 \cdot (0.0) + 1.10 \cdot 43.12$

$= 47.43\ kip\text{-}ft$

$P_r = P_{nt} + B_2 P_{lt}$

$= 1174 + 1.10 \cdot 0.0$

$= 1174$

$P_c = 1460\ k$

$M_{cx} = 803\ k\text{-}ft$

$\frac{P_r}{P_c} = \frac{1174}{1460} = 0.804 \geq 0.2$

$\frac{P_r}{P_c} + \left(\frac{8}{9}\right)\left(\frac{M_{rx}}{M_{cx}}\right) \leq 1.0$

$\frac{1174}{1460} + \left(\frac{8}{9}\right)\left(\frac{47.43}{803}\right) = 0.857 \leq 1.0 \therefore OK$ RAM output $= 0.840$

Beam: W24x55

$B_2 = 1.10$ below $B_2 = 1.10$ above

$M_{dead} = 88.30\ k\text{-}ft$

$M_{live} = 37.37\ k\text{-}ft$

$M_{eq} = 27.61\ k\text{-}ft$

I-3

Controlling Load Combination

$1.467D + 0.5L + 1.0Q_E$

B_2 $M_{lt} = 1.10 \cdot 27.61 = 30.37$ k-ft

$M_u = 1.467 \cdot 88.30 + 0.5 \cdot 37.37 + 1.0 \cdot 30.37 = 176.59$ k-ft

$C_b = 2.53$

$\phi M_n = 503$ k-ft

ϕM_n for $L_b = 10.0$ ft $= 386$ k-ft

$\phi M_n = 386$ k-ft $\cdot 2.53 = 976.58$ k-ft ≤ 503 k-ft

$\phi M_n = 503$ k-ft > 176.59 k-ft $\therefore$ Ok

Appendix J

rooflite®
Certified Green Roof Media

Specifications

Because rooftop gardens are living systems, Skyland can only guarantee their products to meet all of the standards of the FLL and/or ASTM at the time of delivery. Therefore any claim of potential non-compliance must be at this time. All warranty claims made subsequent to the delivery of the product will not be honored. Additionally, nutrients of newly blended products may temporarily exceed upper limits. Consistent nutrient values will typically be achieved soon after installation.*

All Density Measurements reflect typical ranges for the respective rooflite products. For more detailed information please refer to our region specific analysis or inquire about latest test results.

If Air-filled Porosity is measured instead of being determined according the FLL Green Roofing Guidelines reference value may be below 10.

The details contained in these specifications correspond with Skyland USA's technical knowledge at the time of publication. Skyland USA, LLC reserves the right to update and adjust performance specifications from time to time in accordance with new insight and to modify the named properties of the product.

Supplier:
Please find your regional supplier at www.rooflitesoil.com or call 1.877.268.0017

**Forschungsgesellschaft Landschaftsentwicklung Landschaftsbau e.V. (FLL) Landscape Development and Landscaping Research Society, 2008*

rooflite® extensive mcl

A very light growing medium for extensive green roof systems with a separate drainage course or a synthetic drainage layer. rooflite® extensive mcl is a precisely balanced blend of light weight mineral aggregates like HydRocks® or pumice and premium organic components like USCC STA approved compost complying with the following requirements:

Particle Size Distribution			
Proportion of silting components < 0.063 mm		Mass %	≤ 10
Proportion of particles < 0.25 mm	60 mesh	Mass %	5 - 20
Proportion of particles < 1.00 mm	18 mesh	Mass %	10 - 40
Proportion of particles < 2.00 mm	10 mesh	Mass %	30 - 50
Proportion of particles < 3.20 mm	1/8 inch	Mass %	40 - 70
Proportion of particles < 6.30 mm	1/4 inch	Mass %	65 - 95
Proportion of particles < 9.50 mm	3/8 inch	Mass %	80 - 100
Proportion of particles < 12.50 mm	1/2 inch	Mass %	100
Density Measurements			
Bulk Density (dry weight basis)		g/cm^3	0.55 - 0.80
Bulk Density (dry weight basis)		lb/ft^3	35 - 50
Bulk Density (at max. water-holding capacity)		g/cm^3	1.05 - 1.15
Bulk Density (at max. water-holding capacity)		lb/ft^3	66 – 72
Water/Air Measurements			
Total Pore Volume		Vol. %	≥ 60
Maximum water-holding capacity		Vol. %	35 – 65
Air-filled porosity at max water-holding capacity		Vol. %	≥ 10
Water permeability (saturated hydraulic conductivity)		cm/sec	0.001 – 0.12
Water permeability (saturated hydraulic conductivity)		in/min	0.024 – 2.83
pH and Salt Content			
pH (in $CaCl_2$)			6.0 - 8.5
Soluble salts (water, 1:10, m:v)		g (KCl)/L	< 3.5
Organic Measurements			
Organic matter content		g/L	25 - 60
Nutrients			
Phosphorus, P205 (CAL)		mg/L	≤ 200
Potassium, K2O (CAL)		mg/L	≤ 700
Magnesium, Mg (CaCl2)		mg/L	≤ 200
Nitrate + Ammonium (CaCl2)		mg/L	≤ 80

All values are based on compacted materials according to laboratory standards and testing methods defined by the Forschungsgesellschaft Landschaftsentwicklung Landschaftsbau e.V. (FLL) Landscape Development and Landscaping Research Society, Guidelines for the Planning Construction and Maintenance of Green-Roofing, Green Roofing Guideline, 2008

www.rooflitesoil.com V3/14

rooflite®

Certified Green Roof Media

Lightweight Green Roofs

For huge building structures with wide construction spans roof loads are often very limited. Typical green roof systems for these kinds of buildings consist of a synthetic drainage in combination with a rather shallow layer of lightweight growth media and Sedum blankets. These systems work best if all components are fine-tuned for optimized performance. As for the growth media, it is important to consider that Sedum blankets already contain a fair amount of organic matter and particle Fines, and that the vegetation provides instant coverage. Depending on the region, irrigation may be necessary, particularly if the system design specifies less than four inches of growing media.

rooflite® extensive mcl

An extra-light growing media designed for extensive green roof systems in multilayer construction. Typical systems are a combination of a synthetic drainage layer with a high performance media layer. rooflite extensive mcl is the ideal substrate for the placement of vegetation blankets or pre-vegetated mats.

www.rooflitesoil.com V3/14

greenroofsolutions

4336 Regency Drive
Glenview, IL 60025

866.675.9963
847.297.7936
www.greenroofsolutions.com

Product Data

Non-recycled poly-propylene staple fiber, needle punched non-woven geotextile

Roll sizes:
6.25' x 200' (1200 sq ft)
6.25' x 360' (2250 sq ft)
12.5' x 360' (4500 sq ft)

Made in USA

Filter Fabric

Filter Fabric is designed to separate the growing media from the drainage system on vegetative green roofs. Polypropylene fibers are needled to filter fabric for a stable network that retains dimensional stability relative to one another. The fabric is resistant to degradation from UV exposure, as well as the biological and chemical environments found in soil.

	FF35	Unit	Test Method
Grab Tensile Strength	90	lbs	ASTM D-4632
Elongation	50	%	ASTM D-4632
Trapezoid Tear	40	lbs	ASTM D-4533
CBR Puncture	265	lbs	ASTM D-6241
UV Stability	70	% at 500 hrs	ASTM D-4355
Permitivity	2.1	sec	ASTM D-4491
Water Flow Rate	150	gpm/sq ft	ASTM D-4491
A.O.S.	70	US Sieve #	ASTM D-4751
A.O.S.	0.212	mm	ASTM D-4751
Weight	0.024	lbs/sq ft	ASTM D-5261
Thickness	0.05	in	ASTM D-5199

greenroofsolutions

4336 Regency Drive
Glenview, IL 60025

866.675.9963
847.297.7936
www.greenroofsolutions.com

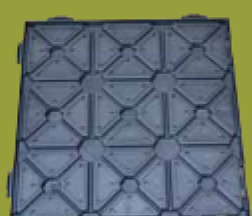
Product Data

Made of 100% Recycled HDPP

Compatible with various roofing membranes

Patent Pending

Made in USA



GRS 32 Drainage Panel

The GRS 32 is designed to be a water retention and drainage component suitable for intensive and extensive green roof systems.

Water flows through the entire panel via reservoir cups and channels to assure uniform distribution.

Rounded edges on bottom of panels prevent roof damage. Panels snap to lock together for fast, easy installation.

Technical Data

Size	24" x 24" x 1 1/4"
Weight	
Dry	0.6 lbs/sf
Including Water	1.93 lbs/sf
Water Capacity	0.165 gal/sf
Material	100% recycled HDPE (color black - shade may
Working Temp	-40°F to 212°F

greenroofsolutions

4336 Regency Drive
Glenview, IL 60025

866.675.9963
847.297.7936
www.greenroofsolutions.com

Product Data

Available in two thicknesses:
RB20: 20 mil
RB30: 30 mil

Roll Sizes:
RB20 & RB30
10.16' x 50' (508 sq ft)
10.16' x 75' (762 sq ft)

Made in USA

Root Barrier RB20 & RB30

-Protects roofing assembly and building from root penetration

-Flexible, and conforms to a variety of surfaces

-Single (4" wide) or double-sided (2" wide) butyl tape available for a waterproof seal

-Always overlap by 12" at seams

Technical Data

	Unit	RB20	RB30	Test Method
Tensile Strength at Break 1"	lbs	75	114	ASTM D-6693
Elongation at Break	%	800	800	ASTM D-6693
Tear Resistance	lbf	11	16	ASTM D-1004
Hydrostatic Resistance	psi	100	170	ASTM D-751
Puncture Resistance	lbf	30	45	ASTM D-4833
Volatile Loss	%	<1	<1	ASTM D-1203
Dimensional Stability	%	<2	<2	ASTM D-1204
Perm Rating	U.S. Perms	0.041	0.031	ASTM E-96

Kemperol 2K-PUR is a solvent-free, fleece-reinforced and liquid-applied waterproofing system based on polyurethane resin. The odor-free product can be used universally on roof decks, roof gardens, plazas, balconies, terraces and also for indoor areas such as bath rooms, catering kitchens and plant rooms.

It is the ideal waterproofing system for sensitive areas such as nurseries, hospitals, schools or senior citizens' homes. It is not only a solvent-free product, but 80% of the polyalcohols (resins) are obtained from renewable raw materials. Kemperol 2K-PUR is root and rot resistant according to FLL testing and offers a proven performance for 25 years.

Typical Physical Properties

PROPERTY	VALUE
Color	yellow-gray
Physical State	cures to solid
Thickness (165 fleece)	70 mils
VOC Content	6 g/l
Tensile Strength @ Break	120 lb/in
Elongation	50 %
Tear Resistance	5.0 lbs
Puncture Resistance	150 lbf
Dimensional Stability	0.1 %
Water Vapor Transmission	0.04 perms
Water Absorption	2.2 %
Impact Resistance	Shore A:85
Usage Time*	30 minutes
Water resistant after*	2 hours
Solid to walk on after*	48 hours
Can be driven on after*	48 hours
Apply surfacing/coating after*	16-48 hours
Apply overburden after*	2 days
Completely hardened after*	3 days
Crack Spanning	2 mm/0.08 inch
Short-term temperature resistance	250 °C/ 482 °F

PRODUCT INFORMATION . COMMERCIAL . US/CANADA

STYROFOAM™ HIGHLOAD 40, 60 AND 100 EXTRUDED POLYSTYRENE INSULATION

1. PRODUCT NAME

STYROFOAM™ HIGHLOAD Extruded Polystyrene Foam Insulation

2. MANUFACTURER

The Dow Chemical Company
Dow Building Solutions
200 Larkin
Midland, MI 48674
1-866-583-BLUE (2583)
Fax 1-989-832-1465

Dow Chemical Canada ULC
Dow Building Solutions
450 – 1st St. SW, Suite 2100
Calgary, AB T2P 5H1
1-866-583-BLUE (2583) (English)
1-800-363-6210 (French)

www.dowbuildingsolutions.com

3. PRODUCT DESCRIPTION

STYROFOAM™ HIGHLOAD Extruded Polystyrene Foam Insulation is a closed-cell foam insulation. Available in compressive strengths of 40, 60 and 100 psi (275, 415 and 690 kPa), STYROFOAM™ HIGHLOAD insulation features exceptional moisture resistance and R-value* retention. All three STYROFOAM™ HIGHLOAD insulation products resist compressive creep and fatigue, delivering long-term compressive strength. Like all STYROFOAM™ insulation products, STYROFOAM™ HIGHLOAD 40, 60 and 100 are durable, versatile and reusable – making them a preferred choice for a variety of high-load applications.

BASIC USE

STYROFOAM™ HIGHLOAD insulation is designed for use in low-temperature (freezer floor) applications, highways, airport runways, bridge abutments, parking decks, utility lines, ice rinks and plaza decks. It is the responsibility of the designer to select the proper STYROFOAM™ HIGHLOAD insulation product based on the dead and live loads expected in the application.

4. TECHNICAL DATA

APPLICABLE STANDARDS

STYROFOAM™ HIGHLOAD 40, 60 and 100 insulation meets ASTM C578 – Standard Specification for Rigid Cellular Polystyrene Thermal Insulation. Applicable ASTM standards include:

- C518 – Standard Test Method for Steady-State Thermal Transmission Properties by Means of the Heat Flow Meter Apparatus
- C177 – Standard Test Method for Steady-State Heat Flux Measurements and Thermal Transmission Properties by Means of the Guarded-Hot-Plate Apparatus
- D1621 – Standard Test Method for Compressive Properties of Rigid Cellular Plastics
- D2842 – Standard Test Method for Water Absorption of Rigid Cellular Plastics
- E96 – Standard Test Methods for Water Vapor Transmission of Materials
- C272 - Standard Test Method for Water Absorption of Core Materials for Structural Sandwich Constructions
- D696 – Standard Test Method for Coefficient of Linear Thermal Expansion of Plastics Between -30°C and 30°C With a Vitreous Silica Dilatometer
- C203 – Standard Test Methods for Breaking Load and Flexural Properties of Block-Type Thermal Insulation Cellular Plastics
- D4716 – Standard Test Method for Determining the (In-plane) Flow Rate per Unit Width and Hydraulic Transmissivity of a Geosynthetic Using a Constant Head
- CAN/ULC S701 Type 4

TABLE 1: U.S. VALUES AND TYPICAL PHYSICAL PROPERTIES OF STYROFOAM™ HIGHLOAD 40, 60 AND 100 INSULATION

PROPERTY AND TEST METHOD	HIGHLOAD 40	HIGHLOAD 60	HIGHLOAD 100
Thermal Resistance, per inch, ASTM C518, C177, @ 75°F mean temp., ft²•h•°F/Btu, R-value, min.	5.0	5.0	5.0
Compressive Strength(1), ASTM D1621, psi, min.	40	60	100
Water Absorption, ASTM C272, % by volume, max. (24hr water immersion)	0.3	0.3	0.3
Water Vapor Permeance(2), ASTM E96, perms	1.0 (57.2 ng/Pa.s.m²)	0.8 (45.8 ng/Pa.s.m²)	0.8 (45.8 ng/Pa.s.m²)
Maximum Use Temperature, °F	165	165	165
Coefficient of Linear Thermal Expansion, ASTM D696, in/in•°F	3.5×10^{-5}	3.5×10^{-5}	3.5×10^{-5}
Flexural Strength, ASTM C203, psi, min.	60	75	100
Complies with ASTM C578, Type	VI	VII	V

(1) Vertical compressive strength is measured at 5 percent deformation or at yield, whichever occurs first. Since STYROFOAM insulations are visco-elastic materials, adequate design safety factors should be used to prevent long-term creep. For static loads, 3:1 is suggested. For dynamic loads, call 1-866-583-BLUE (2583) for safety factor recommendation.
(2) Water vapor permeance varies with product type and thickness. Values are based on the desiccant method and they apply to insulation 1" or greater in thickness.

* R means resistance to heat flow. The higher the R-value or RSI, the greater the insulating power.

PRODUCT INFORMATION . COMMERCIAL . US/CANADA

TABLE 2: CANADA VALUES AND TYPICAL PHYSICAL PROPERTIES OF STYROFOAM™ HIGHLOAD 40, 60 AND 100 INSULATION

PROPERTY AND TEST METHOD	HIGHLOAD 40	HIGHLOAD 60	HIGHLOAD 100
Thermal Resistance, per inch (25 mm), ASTM C518, C177, @ 75°F (24°C) mean temp., ft^2•h•°F/Btu (m^2•°C/W), R-value (RSI), min.	5.0 (.88)	5.0 (.88)	5.0 (.88)
Compressive Strength[1], ASTM D1621, psi (kPa), min.	40 (275)	60 (415)	100 (690)
Water Absorption, ASTM D2842, % by volume, max. (96hr water immersion)	0.7	0.7	0.7
Water Vapour Permeance[2], ASTM E96, perms (ng/Pa•s•m^2)	1.0 (57.2 ng/Pa.s.m^2)	0.8 (45.8 ng/Pa.s.m^2)	0.8 (45.8 ng/Pa.s.m^2)
Maximum Use Temperature, °F (°C)	165 (74)	165 (74)	165 (74)
Coefficient of Linear Thermal Expansion, ASTM D696, in/in•°F (mm/m•°C)	3.5×10^{-5} (6.3×10^{-2})	3.5×10^{-5} (6.3×10^{-2})	3.5×10^{-5} (6.3×10^{-2})
Flexural Strength, ASTM C203, psi (kPa), min.	70 (480)	85 (585)	85 (585)
Compressive Modulus (typical), ASTM D1621, psi (kPa)	1,400 (9,650)	2,200 (15,170)	3,700 (25,510)
Complies with CAN/ULC S701, Type	4	4	4

(1) Vertical compressive strength is measured at 5 percent deformation or at yield, whichever occurs first. Since STYROFOAM insulations are visco-elastic materials, adequate design safety factors should be used to prevent long-term creep. For static loads, 3:1 is suggested. For dynamic loads, call 1-866-583-BLUE (2583) for safety factor recommendation.

(2) Water vapour permeance varies with product type and thickness. Values are based on the desiccant method and they apply to insulation 1" (25 mm) or greater in thickness.

CODE COMPLIANCES

STYROFOAM™ HIGHLOAD 40, 60 and 100 insulation complies with the following codes:

- International Residential Code (IRC) and International Building Code (IBC); see ICC-ES ESR 2142 (excluding STYROFOAM™ HIGHLOAD 100)
- California Std. Reg. #CA T-064
- Underwriters Laboratories, see Classification Certificate D369
- Underwriters Laboratories Verified to ESR 2142
- CCMC - EVALUATION 04888-L

Contact your Dow sales representative or local authorities for state/provincial and local building code requirements and related acceptances.

TYPICAL PHYSICAL PROPERTIES

STYROFOAM™ HIGHLOAD 40, 60 and 100 insulation products exhibit the typical physical properties indicated in Tables 1 and 2 when tested as represented.

ENVIRONMENTAL DATA

STYROFOAM™ Brand HIGHLOAD 40, 60, 100 Insulation is hydrochlorofluorocarbon (HCFC) free with zero ozone-depletion potential.

STYROFOAM™ Brand HIGHLOAD 40, 60, 100 Insulation is reusable in many applications.

PRODUCT INFORMATION . COMMERCIAL . US/CANADA

FIRE INFORMATION
STYROFOAM™ HIGHLOAD 40, 60 and 100 insulation is combustible; protect from high heat sources. Local building codes may require a protective or thermal barrier. For more information, consult MSDS, call Dow at 1-866-583-BLUE (2583) or contact your local building inspector.

5. INSTALLATION

STYROFOAM™ HIGHLOAD 40, 60 and 100 insulation boards are easy to handle and install. They can be cut with a utility knife or any sharp blade. Contact a local Dow representative or access the literature library at www.dowbuildingsolutions.com for more specific instructions.

6. AVAILABILITY

STYROFOAM™ HIGHLOAD 40, 60 and 100 insulation products are distributed through an extensive network. For more information, call:
1-800-232-2436 (English)
1-800-565-1255 (French)

7. WARRANTY

In the United States, a 50-year thermal limited warranty is available on STYROFOAM™ Insulation products 1.5 inches and greater. For thickness less than 1.5 inches, other warranties may apply. Warranties are available as described at www.dbswarranties.com

8. MAINTENANCE

Not applicable.

9. TECHNICAL SERVICES

Dow can provide technical information to help address questions when using STYROFOAM™ HIGHLOAD 40, 60 and 100 insulation products. Technical personnel are available to assist with any insulation project. For technical assistance call:
1-866-583-BLUE (2583) (English)
1-800-363-6210 (French)

10. FILING SYSTEMS

- www.dowbuildingsolutions.com


ROOFAQUAGUARD BREA
High Performance Building Products
INSTALLATION INSTRUCTIONS:
All material must be installed in compliance with applicable codes and ordinances
Marketing, Sales & Technical Support:
Nemco Industries Inc.
4655 Caulfeild Dr
West Vancouver, BC V7W 1E9
Tel: 1 888 840 3435
nemco@telus.net
Manufactured for:
NORDIC WATERPROOFING INC.
PL 10
04361 Tuusula, Finland
LIMITED 35 YEAR WARRANTY
MIAMI-DADE COUNTY APPROVED
NATIONAL ROOFING CONTRACTORS ASSOCIATION MEMBER
ROOFAQUAGUARD BREA
High Performance Building Products
A Truly Breathable, Microporous Underlayment
RoofAquaGuard BREA
A very strong, multilayer, breathable, mechanically fastened Synthetic Roofing Underlayment for use on pitched roofs
Constructed around a "Microporous" Film that combines a Weather Barrier with excellent Breathability properties and additional reinforcement mesh
Superior Non-Abrasive Surface and Excellent Walkability Characteristics
Allows moisture to evaporate without letting in water or wind
One of the best Breathable Synthetic Underlayment Products on the market using proven European Technology
Suitable for ventilated or non-ventilated cold or warm roof applications
Hydrophobically Treated
Microporous film is engineered with special polymer chemistry that uses less filler than conventional Microporous films and is thus truly breathable
UV stable up to 4 months without final roof cover
Excellent stability at High Temperatures
PPE membranes are totally recyclable
Limited 35 Year Warranty
www.roofaquaguard.com
HIGH-TECH PROTECTION AGAINST ELEMENTS

BREA

ROOFAQUAGUARD
High Performance Building Products

LIMITED 35 YEAR WARRANTY

ROOFAQUAGUARD LIMITED WARRANTY

Nordic Waterproofing Inc. ("Manufacturer") warrants to the buyer ("Buyer") of the Manufacturer's synthetic roofing underlayment product ("Product") that the Product will not prematurely deteriorate to the point of failure because of weathering for a period of 35 years from the sales invoice date ("Warranty Period") only if installed in a workmanlike manner strictly in accordance with and for the purpose stated in Manufacturer's installation instructions.

This warranty does not cover leaks or damage caused by any penetrations (including penetrations by fasteners or objects placed on the roof), or by animals, vandalism, fire, abusive conditions, inadequate or faulty structural design, structural defects, building alterations, natural forces such as tornados, hurricanes, earthquakes, other acts of nature, or any other cause beyond Manufacturer's control. This warranty covers the Product material only. Flashings, adhesives and other accessories used in the roofing system are not covered by this warranty. This warranty does not cover any costs or expenses associated with testing, repair, removal, or replacement of the Product.

To make a warranty claim, Buyer must, within 15 days from the date that the claimed defect in the Product was discovered: (1) Give Manufacturer written notice of the defect including, without limitation, detailed descriptions of how the Product was used, the defect and how and when the defect was discovered; (2) Deliver to Manufacturer field samples or digital pictures of the Product legibly showing the Product production codes and digital pictures showing the defect. To be effective, any such notice, samples and pictures must be sent by registered or certified mail to Manufacturer addressed as follows:

Nordic Waterproofing Inc.
c/o Nemco Industries Inc.
4655 Caulfeild Dr
West Vancouver, BC V7W 1E9
Canada

Manufacturer shall, at its sole option, and as Buyer's sole remedy, either, repair, refund the purchase price of, or provide replacement for, that portion of the Product which has been proven to be defective in a manner covered by this warranty. The value of these remedies shall be determined solely by the Manufacturer based upon the current prices for the Product prorated by the number of remaining months of the unexpired Warranty Period. The maximum pro-rated value for repair or refund shall not exceed the original Product purchase price. Any such replacement or refund shall constitute the limit of Manufacturer's liability or obligation for any defective Product. Buyer shall pay all handling and transport costs in connection with any warranty claim.

This warranty shall become void if any person not expressly authorized by Manufacturer performs any repairs or alterations within the Warranty Period or if Buyer fails to give notice of defect within the time and in the manner described above.

The warranty set forth herein is Manufacturer's sole and exclusive warranty for the Product. This warranty shall be the Buyer's exclusive remedy for any use of the Product against Manufacturer or its distributors or sales representatives. No representative of the manufacturer has authority to give any warranties except as stated herein. THERE ARE NO WARRANTIES EITHER EXPRESSED OR IMPLIED, EITHER AT COMMON LAW OR BY STATUTE INCLUDING WITHOUT LIMITATION THE IMPLIED WARRANTIES UNDER THE SALE OF GOODS ACT [RSBC 1996] C410, AND THE IMPLIED WARRANTIES OF MERCHANTABILITY AND FITNESS FOR A PARTICULAR PURPOSE, WHICH MODIFY THE WARRANTY CONTAINED IN THIS DOCUMENT. MANUFACTURER SHALL NOT BE LIABLE FOR ANY INCIDENTAL, CONSEQUENTIAL OR OTHER DAMAGES ARISING FROM THE USE OF THE PRODUCT, INCLUDING BUT NOT LIMITED TO, LOSS OF PROFITS, DAMAGES TO THE STRUCTURE OR ITS CONTENTS, PERSONAL INJURY OR PUNITIVE DAMAGES, BASED ON ANY THEORY OF LAW WHATSOEVER.

No modification of this warranty or waiver of its terms shall be binding on either party unless approved in writing by an authorized representative of the party. This warranty may not be modified by any course of dealing or performance, trade usage or failure to enforce any term of this warranty. Buyer may not assign or permit any other transfer of this warranty without Manufacturer's written consent. If any of the terms contained herein are unenforceable, such term shall be limited only to the extent necessary to make it enforceable, and all other terms shall remain in full force and effect. No action for breach of this warranty may be commenced more than one year after the accrual of the cause of action. This warranty is the complete, final and exclusive agreement of the parties with respect to the quality or performance of the Product and any and all warranties for the Product. All legal aspects of this warranty including, without limitation, its interpretation, the rights and duties of the parties under this warranty and conflict of laws shall be governed by and construed in accordance with the laws of British Columbia, Canada, and the parties submit and attorn to the jurisdiction of the courts of the province of British Columbia.

ROOFAQUAGUARD
High Performance Building Products

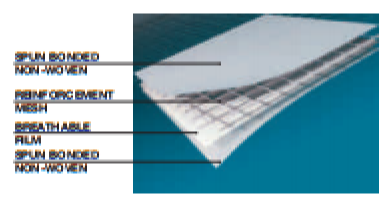
ROOFAQUAGUARD BREA (BREATHABLE ROOFING UNDERLAYMENT SPECIFICATION DATA)

CHECKED CHARACTERISTICS	NORM (RELAVANT STANDARD)	VALUE	UNIT OF MEASURE	CONVERTED UNIT OF MEASURE
Product Code	KU0044	Roll Size	1.5m x 50m	59" x 164' (60 x 164)
No of Layers		Meltblown/fleece/grid combination	4	4
Weight Mass Per Unit Area	EN 1849-2(TAS 104-95)	175 ± 5%	175 g/m² (13kg/roll)	28.7 lb/roll
Thickness	EN 1849-2(TAS 104-95)	0.8 ± 5%	0.75 mm	30 mil (0.03")
Tensile Strength (longitudinal)	EN 12311-1 (AC 38)	MD: 310±15% CD:340±15%	460N/5cm	52 lbf/in
Tensile Strength (transverse)	EN 12311-1 (AC 38)	MD: 310±15% CD:340±15%	400N/5cm	45 lbf/in
Elongation at Break set strength longitudinal			20%	20%
Nail Tearing Strength (longitudinal)	EN 12310-1	MD: 190±15% CD:200±15%	350 N	39 lbf
Nail Tearing Strength (transverse)	EN 12310-1		375 N	42 lbf
UV Resistance	EN 13589-1 (AC 38)	4 months	4 months	4 months
Water Vapour Transmission Property	EN 13859-1 (AC 38)	WDD=810 ± 15% 23°C 50-93% RH	1043g/m²-24h	*see chart below
Water Vapour resistance	BS 3177		0.18MNs/g	
Water Column			>3000mm H_2O	118.1 in
Resistance to Temperature	EN 13859-1 (AC 38)		-40°C to 80°C	-40°F to 176°F
Approvals				ICC-ESR #3017
Approvals				Miami Dade NOA #09-0915.10
Approvals				FL #13064

*chart

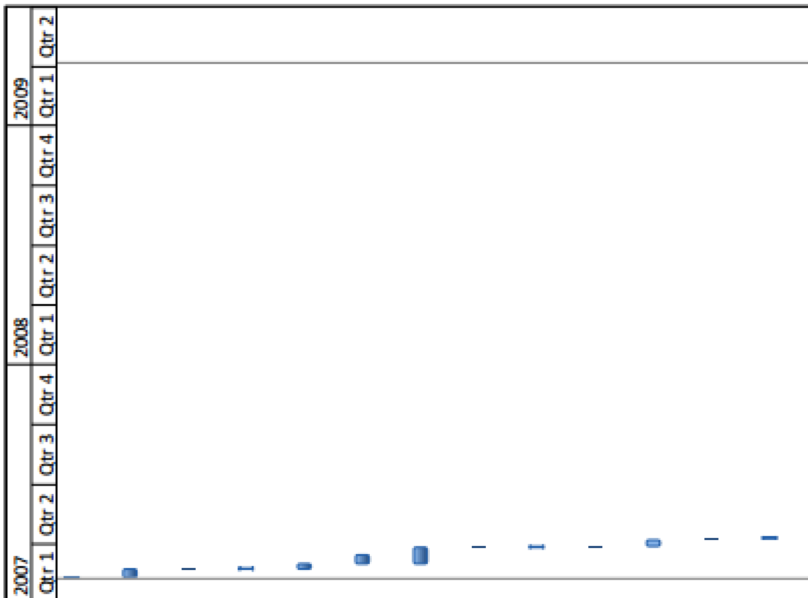
TEST RESULT SUMMARY	METRIC UNITS		IMPERIAL UNITS	
Water Vapor Transmission	43	g/hr.m²	62	grns/hr.ft²
	1034	g/day.m²	1479	grns/day.ft²
Water Vapor Permeance	8333	ng/Pa.s.m²	146	perms
Water Vapor Permeability	4	ng/Pa.s.m	9530	Perm inch

4-LAYER CONSTRUCTION

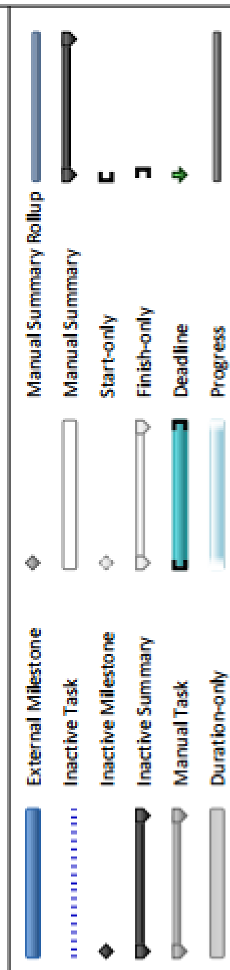


Appendix K

ID	Task Mode	Task Name	Duration	Start	Finish
1		Column Reinforcement (First Floor)	1.3 days	Wed 2/7/07	Thu 2/8/07
2		Column Formwork (First Floor)	9 days	Thu 2/8/07	Wed 2/21/07
3		Column Concrete Placement (First Floor)	0.3 days	Wed 2/21/07	Wed 2/21/07
4		Beam Reinforcement (First Floor)	1.25 days	Wed 2/21/07	Thu 2/22/07
5		Elevated Slab Reinforcement (First Floor)	5.9 days	Wed 2/21/07	Thu 3/1/07
6		Beams Formwork (First Floor)	9.5 days	Thu 3/1/07	Wed 3/14/07
7		Elevated Slab Formwork (First Floor)	17 days	Thu 3/1/07	Mon 3/26/07
8		Beam Concrete Placement (First Floor)	0.5 days	Mon 3/26/07	Mon 3/26/07
9		Elevated Slab Concrete Placement (First Floor)	1.5 days	Mon 3/26/07	Tue 3/27/07
10		Shear Wall Reinforcement (First Floor)	0.7 days	Wed 3/28/07	Wed 3/28/07
11		Shear Wall Formwork (First Floor)	7.3 days	Wed 3/28/07	Fri 4/6/07
12		Shear Wall Concrete Placement (First Floor)	0.4 days	Mon 4/9/07	Mon 4/9/07
13		Column Reinforcement (Second Floor)	1.3 days	Mon 4/9/07	Tue 4/10/07

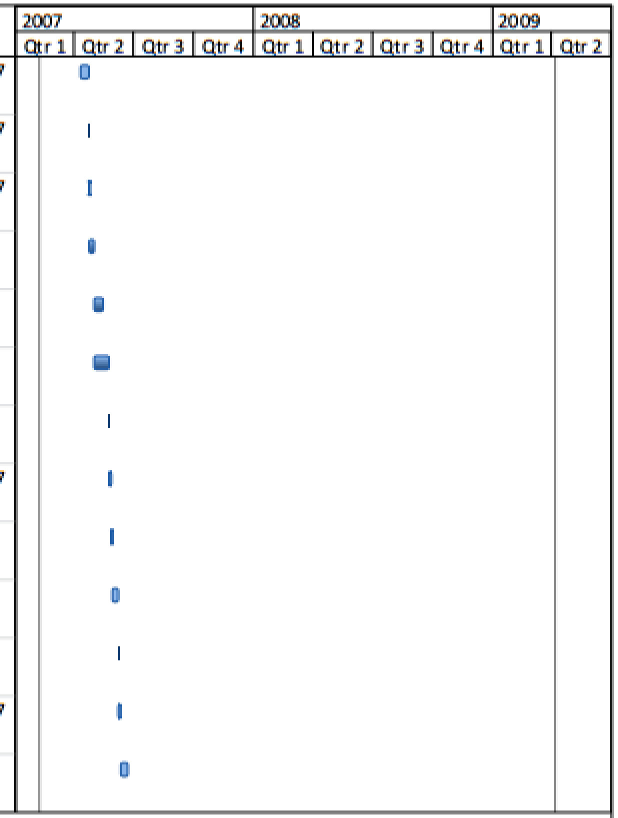


Project: Concrete Schedule
Date: Tue 4/8/14



Page 1

ID		Task Mode	Task Name	Duration	Start	Finish
14			Column Formwork (Second Floor)	9 days	Tue 4/10/07	Mon 4/23/07
15			Column Concrete Placement (Second Floor)	0.3 days	Mon 4/23/07	Mon 4/23/07
16			Beam Reinforcement (Second Floor)	1.25 days	Tue 4/24/07	Wed 4/25/07
17			Elevated Slab Reinforcement (Second Floor)	5.9 days	Tue 4/24/07	Tue 5/1/07
18			Beams Formwork (Second Floor)	9.5 days	Tue 5/1/07	Tue 5/15/07
19			Elevated Slab Formwork (Second Floor)	17 days	Tue 5/1/07	Thu 5/24/07
20			Beam Concrete Placement (Second Floor)	0.5 days	Thu 5/24/07	Fri 5/25/07
21			Elevated Slab Concrete Placement (Second Floor)	1.5 days	Thu 5/24/07	Mon 5/28/07
22			Shear Wall Reinforcement (Second Floor)	0.7 days	Mon 5/28/07	Tue 5/29/07
23			Shear Wall Formwork (Second Floor)	7.3 days	Tue 5/29/07	Thu 6/7/07
24			Shear Wall Concrete Placement (Second Floor)	0.4 days	Thu 6/7/07	Thu 6/7/07
25			Column Reinforcement (Third Floor)	1.3 days	Thu 6/7/07	Mon 6/11/07
26			Column Formwork (Third Floor)	9 days	Mon 6/11/07	Fri 6/22/07



Project: Concrete Schedule
Date: Tue 4/8/14

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

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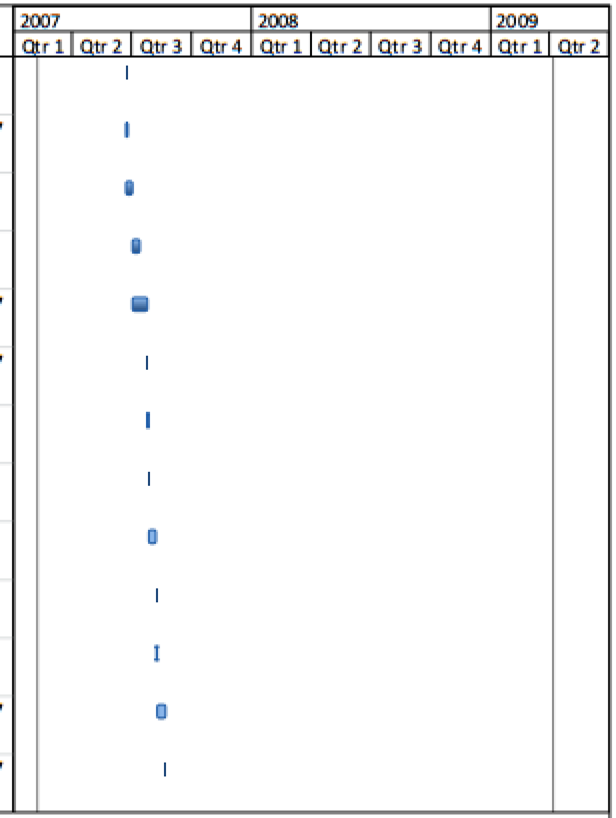
ID		Task Mode	Task Name	Duration	Start	Finish
27			Column Concrete Placement (Third Floor)	0.3 days	Fri 6/22/07	Fri 6/22/07
28			Beam Reinforcement (Third Floor)	1.25 days	Fri 6/22/07	Mon 6/25/07
29			Elevated Slab Reinforcement (Third Floor)	5.9 days	Fri 6/22/07	Mon 7/2/07
30			Beams Formwork (Third Floor)	9.5 days	Mon 7/2/07	Fri 7/13/07
31			Elevated Slab Formwork (Third Floor)	17 days	Mon 7/2/07	Wed 7/25/07
32			Beam Concrete Placement (Third Floor)	0.5 days	Wed 7/25/07	Wed 7/25/07
33			Elevated Slab Concrete Placement (Third Floor)	1.5 days	Wed 7/25/07	Thu 7/26/07
34			Shear Wall Reinforcement (Third Floor)	0.7 days	Thu 7/26/07	Fri 7/27/07
35			Shear Wall Formwork (Third Floor)	7.3 days	Fri 7/27/07	Tue 8/7/07
36			Shear Wall Concrete Placement (Third Floor)	0.4 days	Tue 8/7/07	Wed 8/8/07
37			Column Reinforcement (Fourth Floor)	1.3 days	Wed 8/8/07	Thu 8/9/07
38			Column Formwork (Fourth Floor)	9 days	Thu 8/9/07	Wed 8/22/07
39			Column Concrete Placement (Fourth Floor)	0.3 days	Wed 8/22/07	Wed 8/22/07



Project: Concrete Schedule
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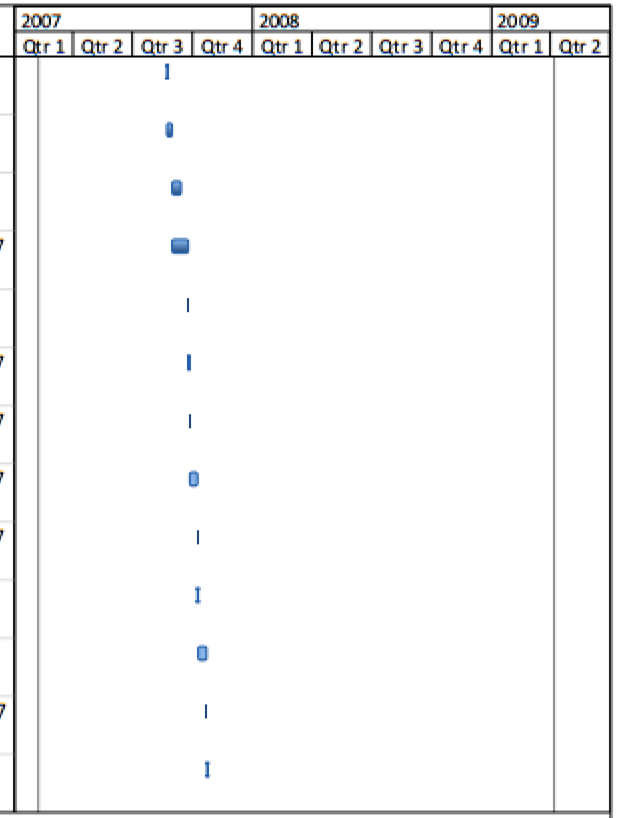
ID		Task Mode	Task Name	Duration	Start	Finish
40			Beam Reinforcement (Fourth Floor)	1.25 days	Wed 8/22/07	Fri 8/24/07
41			Elevated Slab Reinforcement (Fourth Floor)	5.9 days	Wed 8/22/07	Thu 8/30/07
42			Beams Formwork (Fourth Floor)	9.5 days	Thu 8/30/07	Thu 9/13/07
43			Elevated Slab Formwork (Fourth Floor)	17 days	Thu 8/30/07	Mon 9/24/07
44			Beam Concrete Placement (Fourth Floor)	0.5 days	Mon 9/24/07	Tue 9/25/07
45			Elevated Slab Concrete Placement (Fourth Floor)	1.5 days	Mon 9/24/07	Wed 9/26/07
46			Shear Wall Reinforcement (Fourth Floor)	0.7 days	Wed 9/26/07	Wed 9/26/07
47			Shear Wall Formwork (Fourth Floor)	7.3 days	Wed 9/26/07	Mon 10/8/07
48			Shear Wall Concrete Placement (Fourth Floor)	0.4 days	Mon 10/8/07	Mon 10/8/07
49			Column Reinforcement (Fifth Floor)	1.3 days	Mon 10/8/07	Tue 10/9/07
50			Column Formwork (Fifth Floor)	9 days	Tue 10/9/07	Mon 10/22/07
51			Column Concrete Placement (Fifth Floor)	0.3 days	Mon 10/22/07	Tue 10/23/07
52			Beam Reinforcement (Fifth Floor)	1.25 days	Tue 10/23/07	Wed 10/24/07



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

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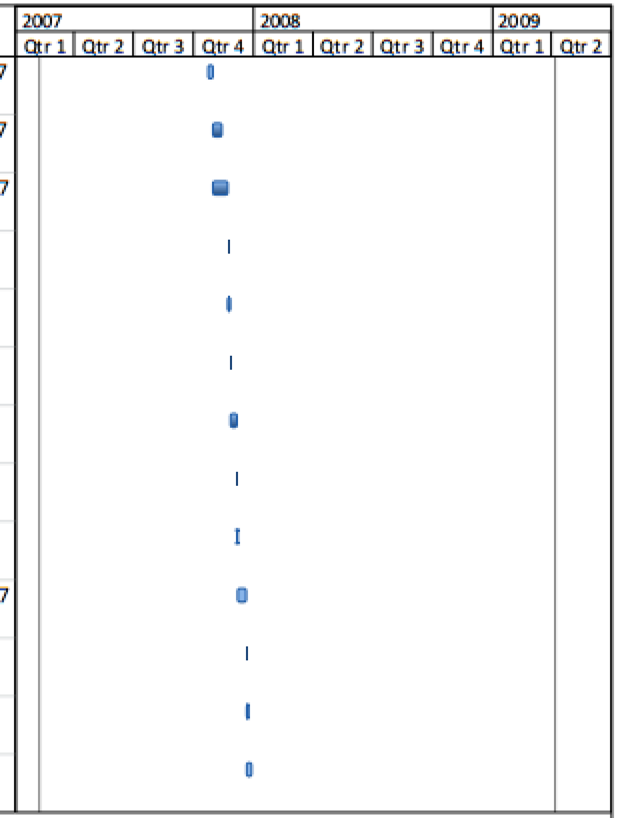
ID	Task Mode	Task Name	Duration	Start	Finish
53		Elevated Slab Reinforcement (Fifth Floor)	5.4 days	Tue 10/23/07	Tue 10/30/07
54		Beams Formwork (Fifth Floor)	9.5 days	Tue 10/30/07	Tue 11/13/07
55		Elevated Slab Formwork (Fifth Floor)	17 days	Tue 10/30/07	Thu 11/22/07
56		Beam Concrete Placement (Fifth Floor)	0.5 days	Thu 11/22/07	Fri 11/23/07
57		Elevated Slab Concrete Placement (Fifth Floor)	1.5 days	Thu 11/22/07	Mon 11/26/07
58		Shear Wall Reinforcement (Fifth Floor)	0.7 days	Mon 11/26/07	Mon 11/26/07
59		Shear Wall Formwork (Fifth Floor)	7.3 days	Mon 11/26/07	Thu 12/6/07
60		Shear Wall Concrete Placement (Fifth Floor)	0.4 days	Thu 12/6/07	Thu 12/6/07
61		Column Reinforcement (Sixth Floor)	1.3 days	Thu 12/6/07	Fri 12/7/07
62		Column Formwork (Sixth Floor)	9 days	Fri 12/7/07	Thu 12/20/07
63		Column Concrete Placement (Sixth Floor)	0.3 days	Thu 12/20/07	Fri 12/21/07
64		Beam Reinforcement (Sixth Floor)	1.25 days	Fri 12/21/07	Mon 12/24/07
65		Elevated Slab Reinforcement (Sixth Floor)	5.9 days	Fri 12/21/07	Fri 12/28/07



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

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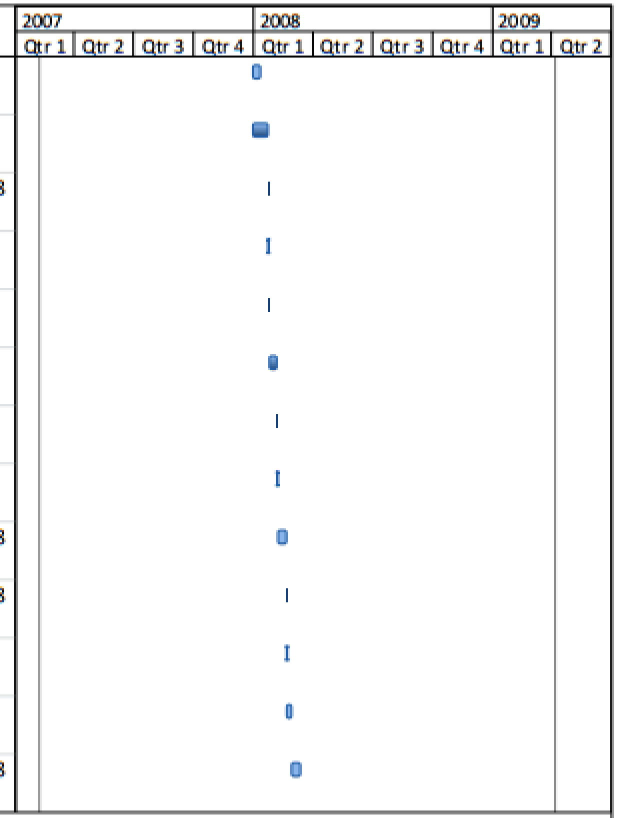
ID		Task Mode	Task Name	Duration	Start	Finish
66			Beams Formwork (Sixth Floor)	9.5 days	Mon 12/31/07	Fri 1/11/08
67			Elevated Slab Formwork (Sixth Floor)	17 days	Mon 12/31/07	Tue 1/22/08
68			Beam Concrete Placement (Sixth Floor)	0.5 days	Wed 1/23/08	Wed 1/23/08
69			Elevated Slab Concrete Placement (Sixth Floor)	1.5 days	Wed 1/23/08	Thu 1/24/08
70			Shear Wall Reinforcement (Sixth Floor)	0.7 days	Thu 1/24/08	Fri 1/25/08
71			Shear Wall Formwork (Sixth Floor)	7.3 days	Fri 1/25/08	Tue 2/5/08
72			Shear Wall Concrete Placement (Sixth Floor)	0.4 days	Tue 2/5/08	Tue 2/5/08
73			Column Reinforcement (Seventh Floor)	1.3 days	Tue 2/5/08	Thu 2/7/08
74			Column Formwork (Seventh Floor)	9 days	Thu 2/7/08	Wed 2/20/08
75			Column Concrete Placement (Seventh Floor)	0.3 days	Wed 2/20/08	Wed 2/20/08
76			Beam Reinforcement (Seventh Floor)	1.25 days	Wed 2/20/08	Thu 2/21/08
77			Elevated Slab Reinforcement (Seventh Floor)	5.9 days	Wed 2/20/08	Thu 2/28/08
78			Beams Formwork (Seventh Floor)	9.5 days	Thu 2/28/08	Wed 3/12/08



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ID		Task Mode	Task Name	Duration	Start	Finish
79			Elevated Slab Formwork (Seventh Floor)	17 days	Thu 2/28/08	Mon 3/24/08
80			Beam Concrete Placement (Seventh Floor)	0.5 days	Mon 3/24/08	Mon 3/24/08
81			Elevated Slab Concrete Placement (Seventh Floor)	1.5 days	Mon 3/24/08	Tue 3/25/08
82			Shear Wall Reinforcement (Seventh Floor)	0.7 days	Tue 3/25/08	Wed 3/26/08
83			Shear Wall Formwork (Seventh Floor)	7.3 days	Wed 3/26/08	Fri 4/4/08
84			Shear Wall Concrete Placement (Seventh Floor)	0.4 days	Fri 4/4/08	Mon 4/7/08
85			Column Reinforcement (Eighth Floor)	1.3 days	Mon 4/7/08	Tue 4/8/08
86			Column Formwork (Eighth Floor)	9 days	Tue 4/8/08	Mon 4/21/08
87			Column Concrete Placement (Eighth Floor)	0.3 days	Mon 4/21/08	Mon 4/21/08
88			Beam Reinforcement (Eighth Floor)	1.25 days	Mon 4/21/08	Tue 4/22/08
89			Elevated Slab Reinforcement (Eighth Floor)	5.9 days	Mon 4/21/08	Tue 4/29/08
90			Beams Formwork (Eighth Floor)	9.5 days	Tue 4/29/08	Mon 5/12/08
91			Elevated Slab Formwork (Eighth Floor)	17 days	Tue 4/29/08	Thu 5/22/08

2007: Qtr 1 | Qtr 2 | Qtr 3 | Qtr 4 — 2008: Qtr 1 | Qtr 2 | Qtr 3 | Qtr 4 — 2009: Qtr 1 | Qtr 2

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

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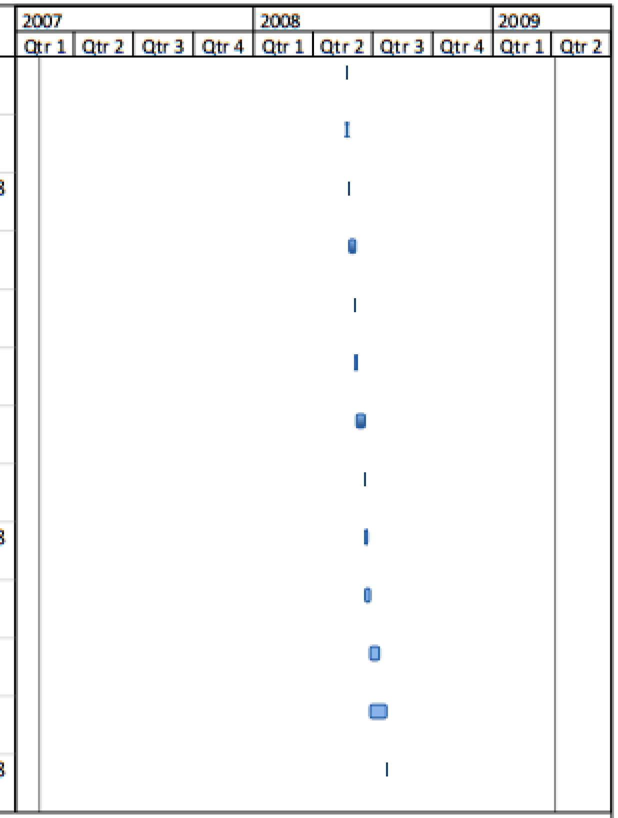
ID		Task Mode	Task Name	Duration	Start	Finish
92			Beam Concrete Placement (Eighth Floor)	0.5 days	Thu 5/22/08	Thu 5/22/08
93			Elevated Slab Concrete Placement (Eighth Floor)	1.5 days	Thu 5/22/08	Fri 5/23/08
94			Shear Wall Reinforcement (Eighth Floor)	0.7 days	Mon 5/26/08	Mon 5/26/08
95			Shear Wall Formwork (Eighth Floor)	7.3 days	Mon 5/26/08	Wed 6/4/08
96			Shear Wall Concrete Placement (Eighth Floor)	0.4 days	Thu 6/5/08	Thu 6/5/08
97			Column Reinforcement (Ninth Floor)	1.3 days	Thu 6/5/08	Fri 6/6/08
98			Column Formwork (Ninth Floor)	9 days	Fri 6/6/08	Thu 6/19/08
99			Column Concrete Placement (Ninth Floor)	0.3 days	Thu 6/19/08	Thu 6/19/08
100			Beam Reinforcement (Ninth Floor)	1.25 days	Fri 6/20/08	Mon 6/23/08
101			Elevated Slab Reinforcement (Ninth Floor)	5.9 days	Fri 6/20/08	Fri 6/27/08
102			Beams Formwork (Ninth Floor)	9.5 days	Fri 6/27/08	Fri 7/11/08
103			Elevated Slab Formwork (Ninth Floor)	17 days	Fri 6/27/08	Tue 7/22/08
104			Beam Concrete Placement (Ninth Floor)	0.5 days	Tue 7/22/08	Wed 7/23/08



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

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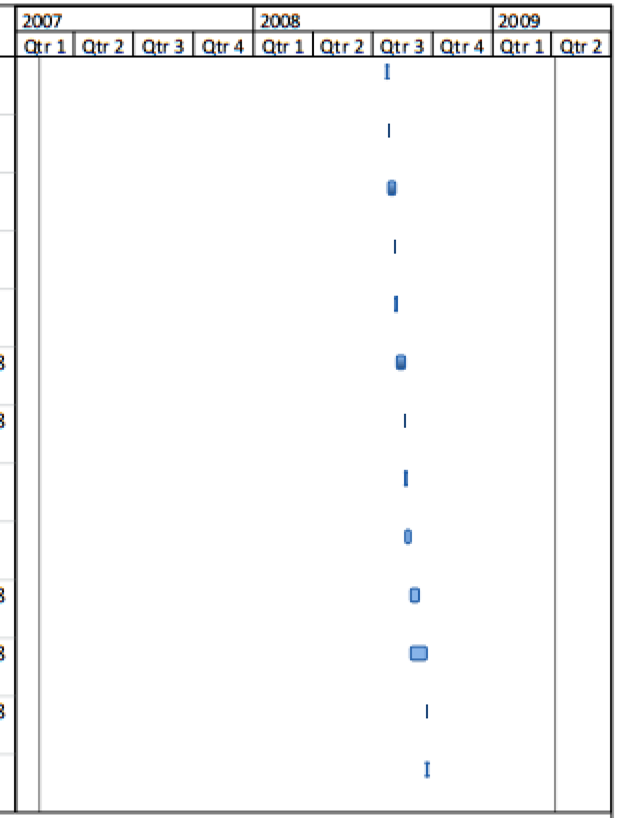
ID		Task Mode	Task Name	Duration	Start	Finish
105			Elevated Slab Concrete Placement (Ninth Floor)	1.5 days	Tue 7/22/08	Thu 7/24/08
106			Shear Wall Reinforcement (Ninth Floor)	0.7 days	Thu 7/24/08	Fri 7/25/08
107			Shear Wall Formwork (Ninth Floor)	7.3 days	Fri 7/25/08	Tue 8/5/08
108			Shear Wall Concrete Placement (Ninth Floor)	0.4 days	Tue 8/5/08	Tue 8/5/08
109			Column Reinforcement (Tenth Floor)	1.3 days	Tue 8/5/08	Thu 8/7/08
110			Column Formwork (Tenth Floor)	9 days	Thu 8/7/08	Wed 8/20/08
111			Column Concrete Placement (Tenth Floor)	0.3 days	Wed 8/20/08	Wed 8/20/08
112			Beam Reinforcement (Tenth Floor)	1.25 days	Wed 8/20/08	Thu 8/21/08
113			Elevated Slab Reinforcement (Tenth Floor)	5.9 days	Wed 8/20/08	Thu 8/28/08
114			Beams Formwork (Tenth Floor)	9.5 days	Thu 8/28/08	Wed 9/10/08
115			Elevated Slab Formwork (Tenth Floor)	17 days	Thu 8/28/08	Mon 9/22/08
116			Beam Concrete Placement (Tenth Floor)	0.5 days	Mon 9/22/08	Mon 9/22/08
117			Elevated Slab Concrete Placement (Tenth Floor)	1.5 days	Mon 9/22/08	Tue 9/23/08



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Task	External Milestone	Manual Summary Rollup
Split	Inactive Task	Manual Summary
Milestone	Inactive Milestone	Start-only
Summary	Inactive Summary	Finish-only
Project Summary	Manual Task	Deadline
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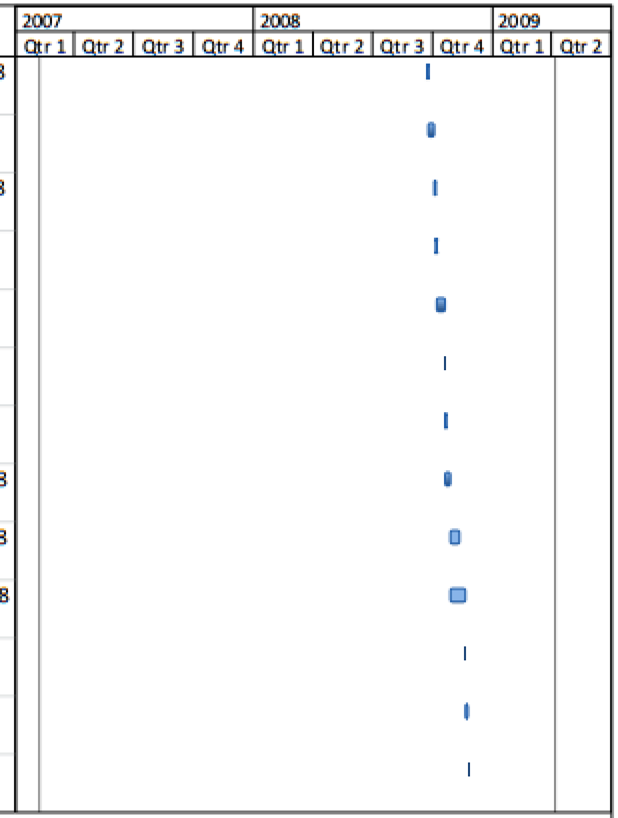
ID	Task Mode	Task Name	Duration	Start	Finish
118		Shear Wall Reinforcement (Tenth Floor)	0.7 days	Tue 9/23/08	Wed 9/24/08
119		Shear Wall Formwork (Tenth Floor)	7.3 days	Wed 9/24/08	Fri 10/3/08
120		Shear Wall Concrete Placement (Tenth Floor)	0.4 days	Fri 10/3/08	Mon 10/6/08
121		Column Reinforcement (Eleventh Floor)	1.3 days	Mon 10/6/08	Tue 10/7/08
122		Column Formwork (Eleventh Floor)	9 days	Tue 10/7/08	Mon 10/20/08
123		Column Concrete Placement (Eleventh Floor)	0.3 days	Mon 10/20/08	Mon 10/20/08
124		Beam Reinforcement (Eleventh Floor)	1.25 days	Mon 10/20/08	Wed 10/22/08
125		Elevated Slab Reinforcement (Eleventh Floor)	5.9 days	Mon 10/20/08	Tue 10/28/08
126		Beams Formwork (Eleventh Floor)	9.5 days	Tue 10/28/08	Tue 11/11/08
127		Elevated Slab Formwork (Eleventh Floor)	17 days	Tue 10/28/08	Thu 11/20/08
128		Beam Concrete Placement (Eleventh Floor)	0.5 days	Thu 11/20/08	Fri 11/21/08
129		Elevated Slab Concrete Placement (Eleventh Floor)	1.5 days	Thu 11/20/08	Mon 11/24/08
130		Shear Wall Reinforcement (Eleventh Floor)	0.7 days	Mon 11/24/08	Mon 11/24/08



Project: Concrete Schedule
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Task | Split | Milestone | Summary | Project Summary | External Tasks
External Milestone | Inactive Task | Inactive Milestone | Inactive Summary | Manual Task | Duration-only
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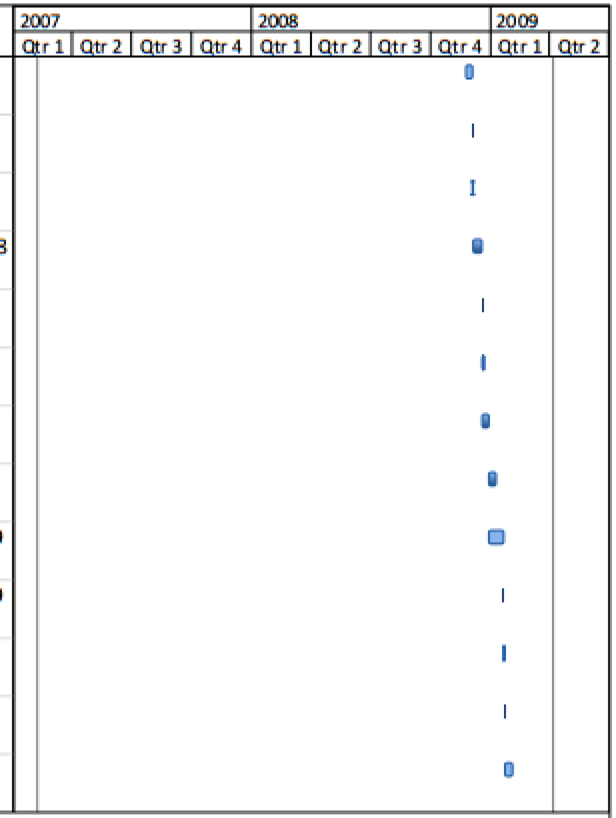
ID	Task Mode	Task Name	Duration	Start	Finish
131		Shear Wall Formwork (Eleventh Floor)	7.3 days	Mon 11/24/08	Thu 12/4/08
132		Shear Wall Concrete Placement (Eleventh Floor)	0.4 days	Thu 12/4/08	Thu 12/4/08
133		Column Reinforcement (Twelfth Floor)	1.3 days	Thu 12/4/08	Fri 12/5/08
134		Column Formwork (Twelfth Floor)	9 days	Fri 12/5/08	Thu 12/18/08
135		Column Concrete Placement (Twelfth Floor)	0.3 days	Thu 12/18/08	Fri 12/19/08
136		Beam Reinforcement (Twelfth Floor)	1.25 days	Fri 12/19/08	Mon 12/22/08
137		Elevated Slab Reinforcement (Twelfth Floor)	5.9 days	Fri 12/19/08	Mon 12/29/08
138		Beams Formwork (Twelfth Floor)	9.5 days	Mon 12/29/08	Fri 1/9/09
139		Elevated Slab Formwork (Twelfth Floor)	17 days	Mon 12/29/08	Wed 1/21/09
140		Beam Concrete Placement (Twelfth Floor)	0.5 days	Wed 1/21/09	Wed 1/21/09
141		Elevated Slab Concrete Placement (Twelfth Floor)	1.5 days	Wed 1/21/09	Thu 1/22/09
142		Shear Wall Reinforcement (Twelfth Floor)	0.7 days	Thu 1/22/09	Fri 1/23/09
143		Shear Wall Formwork (Twelfth Floor)	7.3 days	Fri 1/23/09	Tue 2/3/09



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Task	External Milestone	Manual Summary Rollup
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Summary	Inactive Summary	Finish-only
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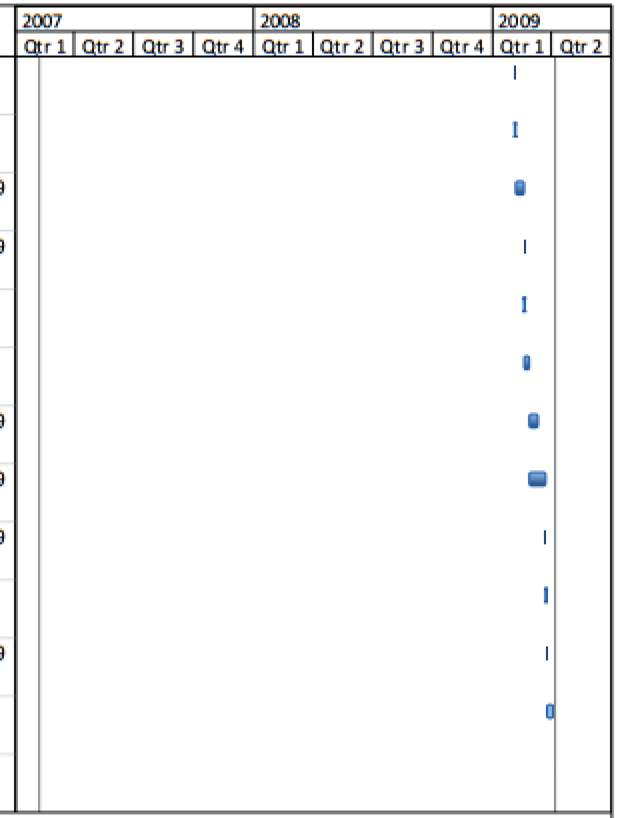
ID	Task Mode	Task Name	Duration	Start	Finish
144		Shear Wall Concrete Placement (Twelfth Floor)	0.4 days	Tue 2/3/09	Tue 2/3/09
145		Column Reinforcement (Thirteenth Floor)	1.3 days	Wed 2/4/09	Thu 2/5/09
146		Column Formwork (Thirteenth Floor)	9 days	Thu 2/5/09	Wed 2/18/09
147		Column Concrete Placement (Thirteenth Floor)	0.3 days	Wed 2/18/09	Wed 2/18/09
148		Beam Reinforcement (Thirteenth Floor)	1.25 days	Wed 2/18/09	Thu 2/19/09
149		Elevated Slab Reinforcement (Thirteenth Floor)	5.9 days	Wed 2/18/09	Thu 2/26/09
150		Beams Formwork (Thirteenth Floor)	9.5 days	Thu 2/26/09	Wed 3/11/09
151		Elevated Slab Formwork (Thirteenth Floor)	17 days	Thu 2/26/09	Mon 3/23/09
152		Beam Concrete Placement (Thirteenth Floor)	0.5 days	Mon 3/23/09	Mon 3/23/09
153		Elevated Slab Concrete Placement (Thirteenth Floor)	1.5 days	Mon 3/23/09	Tue 3/24/09
154		Shear Wall Reinforcement (Thirteenth Floor)	0.7 days	Wed 3/25/09	Wed 3/25/09
155		Shear Wall Formwork (Thirteenth Floor)	7.3 days	Wed 3/25/09	Fri 4/3/09
156		Shear Wall Concrete Placement (Thirteenth Floor)	0.4 days	Mon 4/6/09	Mon 4/6/09



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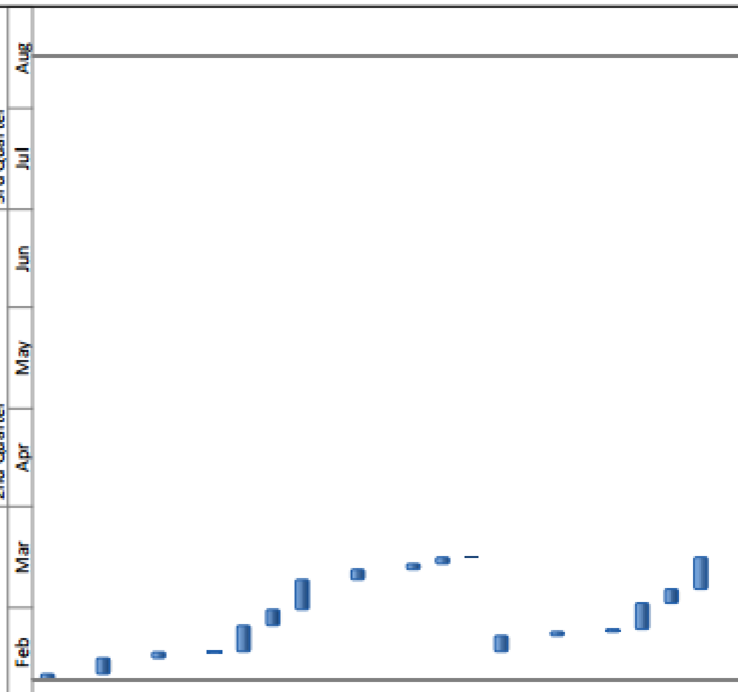
Appendix L

ID	Task Mode	Task Name	Duration	Start
1		Steel Columns (First-Second Floor)	1.87 days	Wed 2/7/07
2		Steel Beams (Shear Connections) (First Floor)	2.66 days	Thu 2/8/07
3		Steel Beams (Moment Connections) (First Floor)	1.57 days	Tue 2/13/07
4		Braces (First Floor)	0.4 days	Thu 2/15/07
5		Floor Deck (First Floor)	6 days	Thu 2/15/07
6		Shear Studs (First Floor)	2.67 days	Fri 2/23/07
7		Concrete Reinforcement (First Floor)	7.4 days	Wed 2/28/07
8		Concrete Topping (First Floor)	1.4 days	Fri 3/9/07
9		Column Fireproofing	1.3 days	Mon 3/12/07
10		Beam Fireproofing	1.9 days	Wed 3/14/07
11		Brace Fireproofing	0.2 days	Fri 3/16/07
12		Steel Beams (Shear Connections) (Second Floor)	2.66 days	Thu 2/15/07
13		Beams (Moment Connections) (Second Floor)	1.57 days	Tue 2/20/07
14		Braces (Second Floor)	0.4 days	Wed 2/21/07
15		Floor Deck (Second Floor)	6 days	Thu 2/22/07
16		Shear Studs (Second Floor)	2.67 days	Fri 3/2/07
17		Concrete Reinforcement (Second Floor)	7.4 days	Tue 3/6/07



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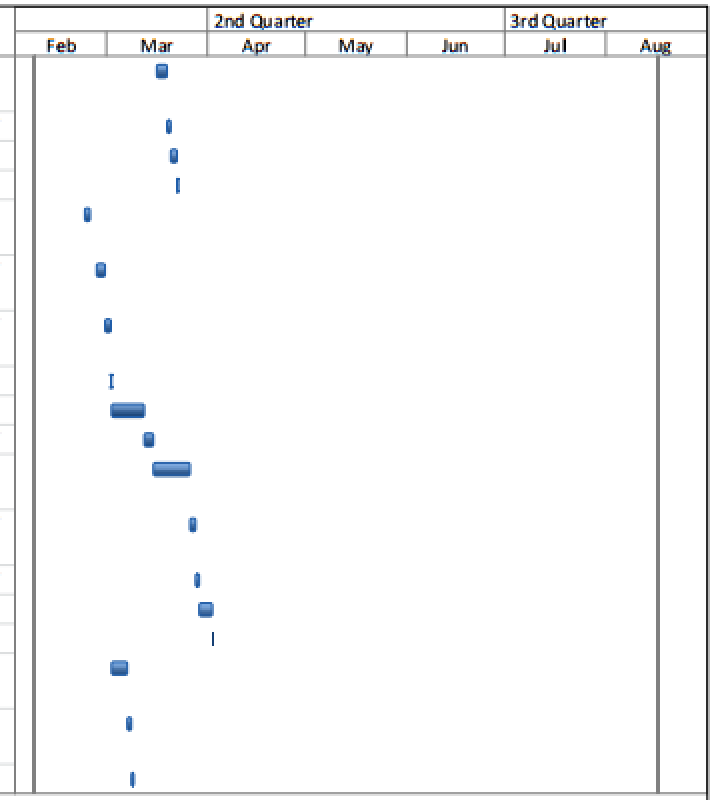
ID		Task Mode	Task Name	Duration	Start
18			Concrete Topping (Second Floor)	1.4 days	Fri 3/16/07
19			Column Fireproofing	1.3 days	Mon 3/19/07
20			Beam Fireproofing	1.9 days	Tue 3/20/07
21			Brace Fireproofing	0.2 days	Thu 3/22/07
22			Steel Columns (Third-Fourth Floor)	1.87 days	Thu 2/22/07
23			Steel Beams (Shear Connections) (Third Floor)	2.66 days	Mon 2/26/07
24			Steel Beams (Moment Connections) (Third Floor)	1.57 days	Wed 2/28/07
25			Braces (Third Floor)	0.4 days	Fri 3/2/07
26			Floor Deck (Third Floor)	6 days	Fri 3/2/07
27			Shear Studs (Third Floor)	2.67 days	Mon 3/12/07
28			Concrete Reinforcement (Third Floor)	7.4 days	Thu 3/15/07
29			Concrete Topping (Third Floor)	1.4 days	Mon 3/26/07
30			Column Fireproofing	1.3 days	Wed 3/28/07
31			Beam Fireproofing	1.9 days	Thu 3/29/07
32			Brace Fireproofing	0.2 days	Mon 4/2/07
33			Steel Beams (Shear Connections) (Fourth Floor)	2.66 days	Fri 3/2/07
34			Steel Beams (Moment Connections) (Fourth Floor)	1.57 days	Wed 3/7/07
35			Braces (Fourth Floor)	0.4 days	Thu 3/8/07



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

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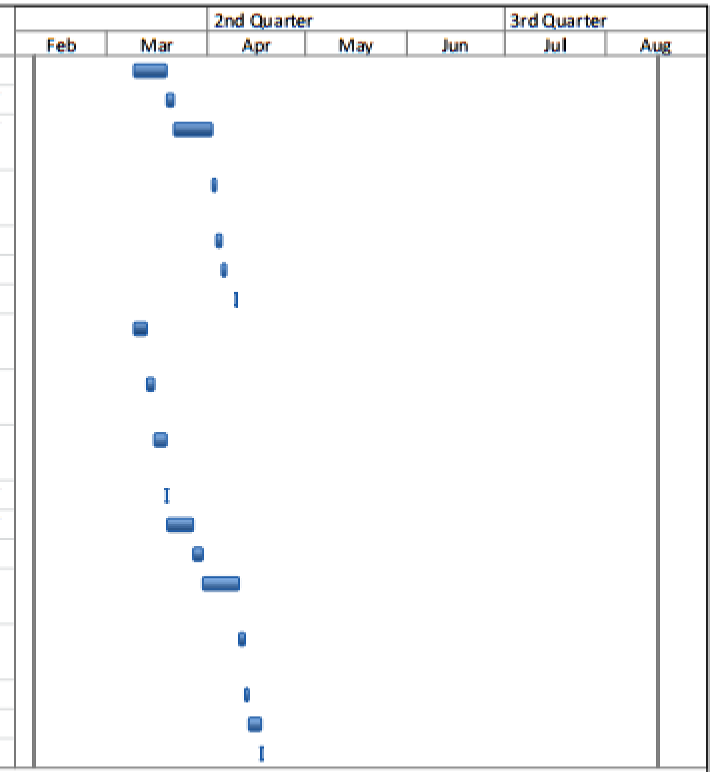
ID		Task Mode	Task Name	Duration	Start
36			Floor Deck (Fourth Floor)	6 days	Fri 3/9/07
37			Shear Studs (Fourth Floor)	2.67 days	Mon 3/19/07
38			Concrete Reinforcement (Fourth Floor)	7.4 days	Wed 3/21/07
39			Concrete Topping (Fourth Floor)	1.4 days	Mon 4/2/07
40			Column Fireproofing	1.3 days	Tue 4/3/07
41			Beam Fireproofing	1.9 days	Thu 4/5/07
42			Brace Fireproofing	0.2 days	Mon 4/9/07
43			Steel Columns (Fifth-Sixth Floor)	1.87 days	Fri 3/9/07
44			Steel Beams (Shear Connections) (Fifth Floor)	2.66 days	Tue 3/13/07
45			Steel Beams (Moment Connections) (Fifth Floor)	1.57 days	Thu 3/15/07
46			Braces (Fifth Floor)	0.4 days	Mon 3/19/07
47			Floor Deck (Fifth Floor)	6 days	Mon 3/19/07
48			Shear Studs (Fifth Floor)	2.67 days	Tue 3/27/07
49			Concrete Reinforcement (Fifth Floor)	7.4 days	Fri 3/30/07
50			Concrete Topping (Fifth Floor)	1.4 days	Tue 4/10/07
51			Column Fireproofing	1.3 days	Thu 4/12/07
52			Beam Fireproofing	1.9 days	Fri 4/13/07
53			Brace Fireproofing	0.2 days	Tue 4/17/07



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

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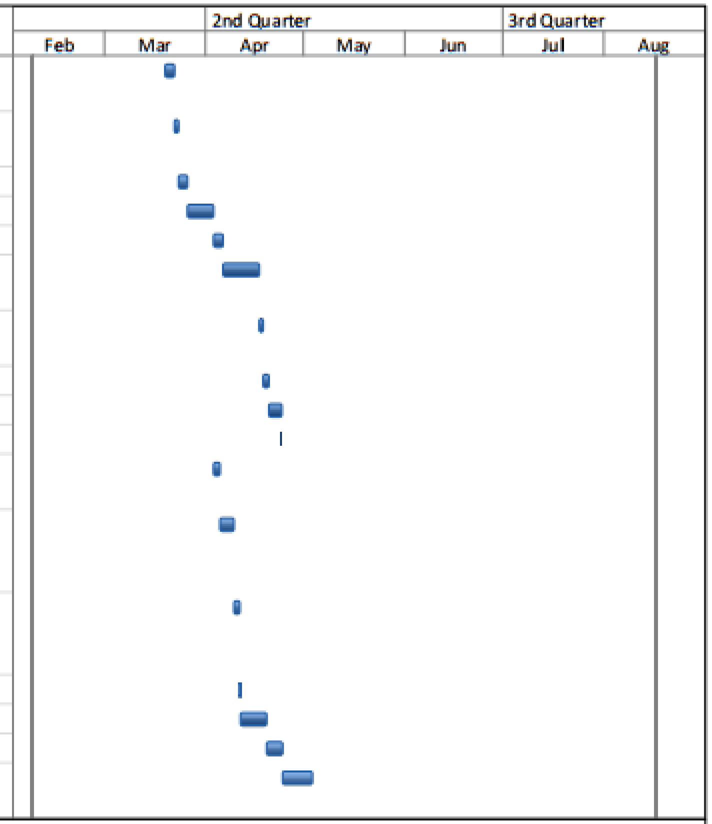
ID		Task Mode	Task Name	Duration	Start
54			Steel Beams (Shear Connections) (Sixth Floor)	2.66 days	Mon 3/19/07
55			Steel Beams (Moment Connections) (Sixth Floor)	1.57 days	Thu 3/22/07
56			Braces (Sixth Floor)	0.4 days	Fri 3/23/07
57			Floor Deck (Sixth Floor)	6 days	Mon 3/26/07
58			Shear Studs (Sixth Floor)	2.67 days	Tue 4/3/07
59			Concrete Reinforcement (Sixth Floor)	7.4 days	Fri 4/6/07
60			Concrete Topping (Sixth Floor)	1.4 days	Tue 4/17/07
61			Column Fireproofing	1.3 days	Wed 4/18/07
62			Beam Fireproofing	1.9 days	Fri 4/20/07
63			Brace Fireproofing	0.2 days	Tue 4/24/07
64			Steel Columns (Seventh-Eighth Floor)	1.87 days	Tue 4/3/07
65			Steel Beams (Shear Connections) (Seventh Floor)	2.66 days	Thu 4/5/07
66			Steel Beams (Moment Connections) (Seventh Floor)	1.57 days	Mon 4/9/07
67			Braces (Seventh Floor)	0.4 days	Wed 4/11/07
68			Floor Deck (Seventh Floor)	6 days	Wed 4/11/07
69			Shear Studs (Seventh Floor)	2.67 days	Thu 4/19/07
70			Concrete Reinforcement (Seventh Floor)	7.4 days	Tue 4/24/07



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

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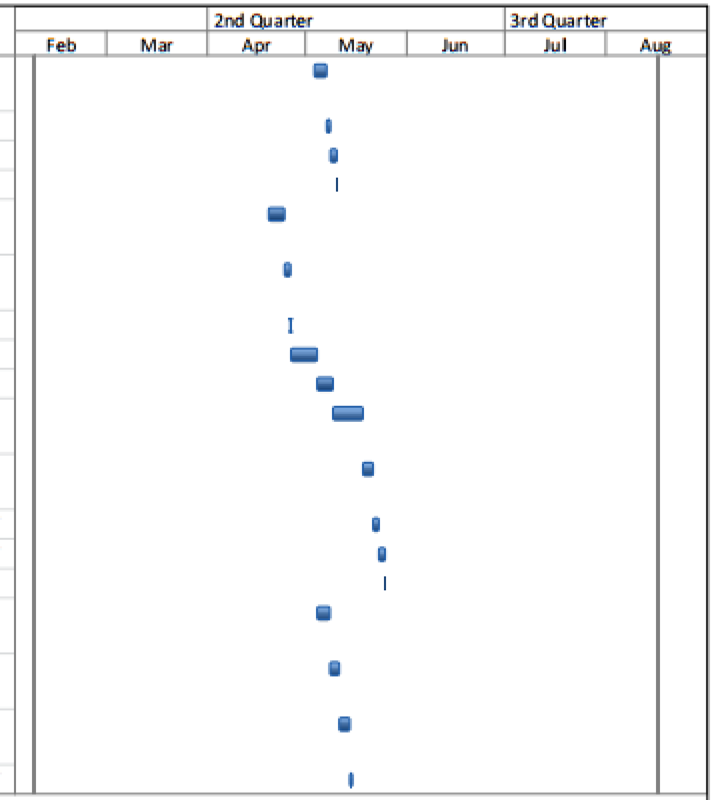
ID	Task Mode	Task Name	Duration	Start
71		Concrete Topping (Seventh Floor)	1.4 days	Thu 5/3/07
72		Column Fireproofing	1.3 days	Mon 5/7/07
73		Beam Fireproofing	1.9 days	Tue 5/8/07
74		Brace Fireproofing	0.2 days	Thu 5/10/07
75		Steel Beams (Eighth Connections) (Sixth Floor)	2.66 days	Thu 4/19/07
76		Steel Beams (Moment Connections) (Eighth Floor)	1.57 days	Tue 4/24/07
77		Braces (Eighth Floor)	0.4 days	Thu 4/26/07
78		Floor Deck (Eighth Floor)	6 days	Thu 4/26/07
79		Shear Studs (Eighth Floor)	2.67 days	Fri 5/4/07
80		Concrete Reinforcement (Eighth Floor)	7.4 days	Wed 5/9/07
81		Concrete Topping (Eighth Floor)	1.4 days	Fri 5/18/07
82		Column Fireproofing	1.3 days	Mon 5/21/07
83		Beam Fireproofing	1.9 days	Wed 5/23/07
84		Brace Fireproofing	0.2 days	Fri 5/25/07
85		Steel Columns (Ninth-Tenth Floor)	1.87 days	Fri 5/4/07
86		Steel Beams (Shear Connections) (Nineth Floor)	2.66 days	Tue 5/8/07
87		Steel Beams (Moment Connections) (Ninth Floor)	1.57 days	Fri 5/11/07
88		Braces (Ninth Floor)	0.4 days	Mon 5/14/07



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

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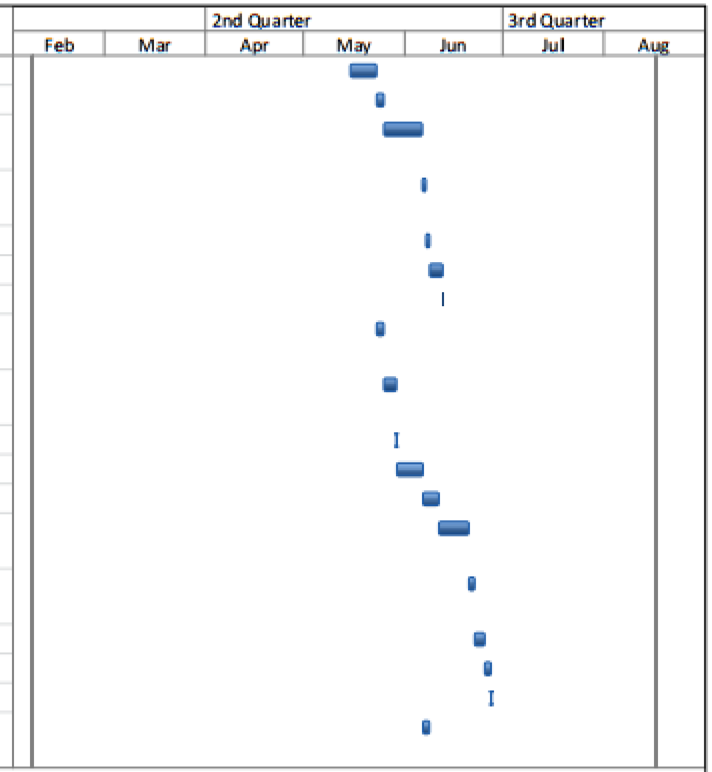
ID		Task Mode	Task Name	Duration	Start
89			Floor Deck (Ninth Floor)	6 days	Tue 5/15/07
90			Shear Studs (Ninth Floor)	2.67 days	Wed 5/23/07
91			Concrete Reinforcement (Ninth Floor)	7.4 days	Fri 5/25/07
92			Concrete Topping (Ninth Floor)	1.4 days	Wed 6/6/07
93			Column Fireproofing	1.3 days	Thu 6/7/07
94			Beam Fireproofing	1.9 days	Fri 6/8/07
95			Brace Fireproofing	0.2 days	Tue 6/12/07
96			Steel Beams (Shear Connections) (Tenth Floor)	2.66 days	Wed 5/23/07
97			Steel Beams (Moment Connections) (Tenth Floor)	1.57 days	Fri 5/25/07
98			Braces (Tenth Floor)	0.4 days	Tue 5/29/07
99			Floor Deck (Tenth Floor)	6 days	Tue 5/29/07
100			Shear Studs (Tenth Floor)	2.67 days	Wed 6/6/07
101			Concrete Reinforcement (Tenth Floor)	7.4 days	Mon 6/11/07
102			Concrete Topping (Tenth Floor)	1.4 days	Wed 6/20/07
103			Column Fireproofing	1.3 days	Fri 6/22/07
104			Beam Fireproofing	1.9 days	Mon 6/25/07
105			Brace Fireproofing	0.2 days	Wed 6/27/07
106			Steel Columns (Eleventh-Twelfth Floor)	1.87 days	Wed 6/6/07



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

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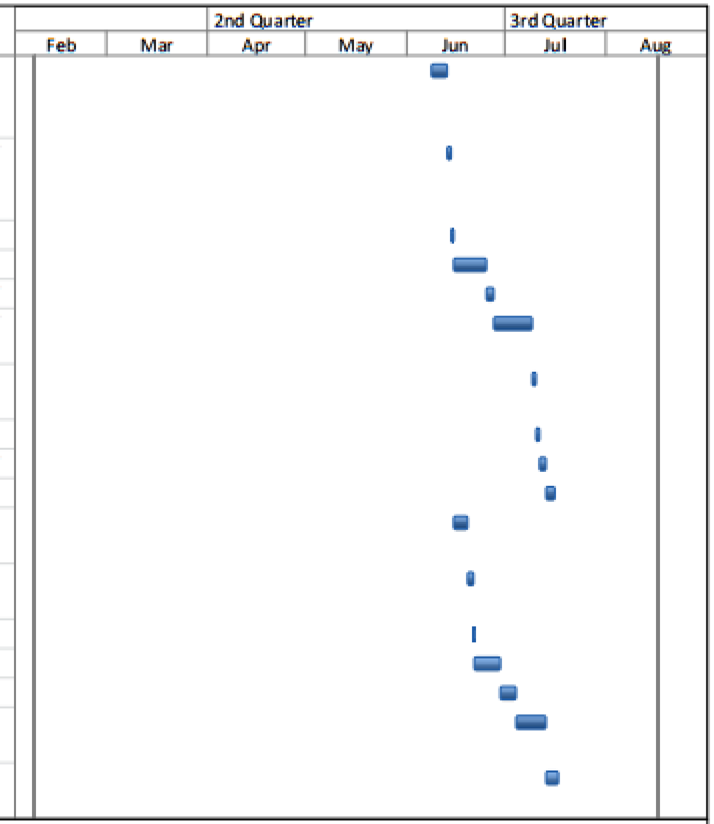
ID		Task Mode	Task Name	Duration	Start
107			Steel Beams (Shear Connections) (Eleventh Floor)	2.66 days	Fri 6/8/07
108			Steel Beams (Moment Connections) (Eleventh Floor)	1.57 days	Wed 6/13/07
109			Braces (Eleventh Floor)	0.4 days	Thu 6/14/07
110			Floor Deck (Eleventh Floor)	6 days	Fri 6/15/07
111			Shear Studs (Eleventh Floor)	2.67 days	Mon 6/25/07
112			Concrete Reinforcement (Eleventh Floor)	7.4 days	Wed 6/27/07
113			Concrete Topping (Eleventh Floor)	1.4 days	Mon 7/9/07
114			Column Fireproofing	1.3 days	Tue 7/10/07
115			Beam Fireproofing	1.9 days	Wed 7/11/07
116			Brace Fireproofing	0.2 days	Fri 7/13/07
117			Steel Beams (Shear Connections) (Twelfth Floor)	2.66 days	Fri 6/15/07
118			Steel Beams (Moment Connections) (Twelfth Floor)	1.57 days	Tue 6/19/07
119			Braces (Twelfth Floor)	0.4 days	Thu 6/21/07
120			Floor Deck (Twelfth Floor)	6 days	Thu 6/21/07
121			Shear Studs (Twelfth Floor)	2.67 days	Fri 6/29/07
122			Concrete Reinforcement (Twelfth Floor)	7.4 days	Wed 7/4/07
123			Concrete Topping (Twelfth Floor)	1.4 days	Fri 7/13/07



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Task
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Inactive Summary
Manual Task
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Manual Summary Rollup
Manual Summary
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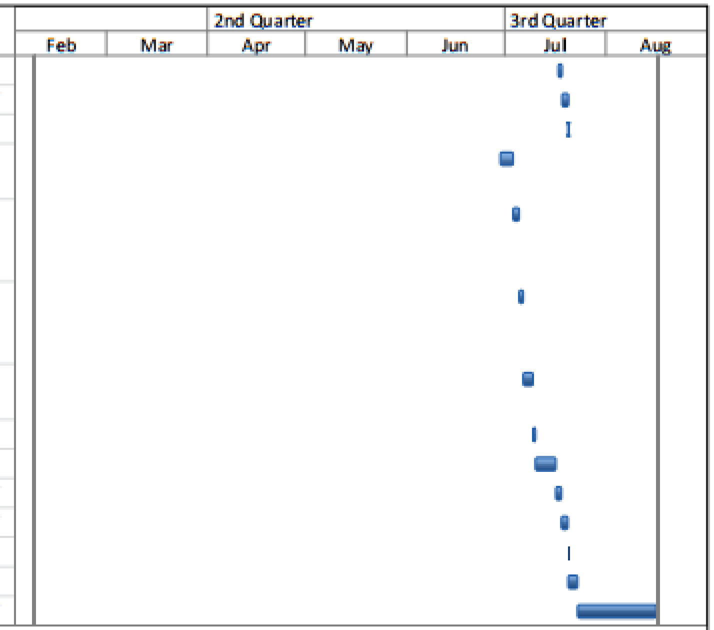
ID		Task Mode	Task Name	Duration	Start
124			Column Fireproofing	1.3 days	Tue 7/17/07
125			Beam Fireproofing	1.9 days	Wed 7/18/07
126			Brace Fireproofing	0.2 days	Fri 7/20/07
127			Steel Columns (Thirteenth Floor)	1.87 days	Fri 6/29/07
128			Steel Beams (Shear Connections) (Thirteenth Floor)	1.66 days	Tue 7/3/07
129			Steel Beams (Moment Connections) (Thirteenth Floor)	1.57 days	Thu 7/5/07
130			Steel Joists (Thirteenth Floor)	1 day	Fri 7/6/07
131			Braces (Thirteenth Floor)	0.4 days	Mon 7/9/07
132			Roof Deck (Thirteenth Floor)	4.6 days	Tue 7/10/07
133			Column Fireproofing	1.3 days	Mon 7/16/07
134			Beam Fireproofing	1.9 days	Wed 7/18/07
135			Brace Fireproofing	0.2 days	Fri 7/20/07
136			Joist Fireproofing	0.75 days	Fri 7/20/07
137			Roof Deck Fireproofing	18.2 days	Mon 7/23/07



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Task	External Milestone	Manual Summary Rollup
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Project Summary	Manual Task	Deadline
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Appendix M

Appendix M

M-2

Steel

Columns

$\frac{26.6}{14.20} = 1.87$ days per every other floor

Beams (Simple Connections)

$\frac{37.8}{14.20} = 2.66$ days per floor

Beams (Moment Connections)

$\frac{12.22}{7.8} = 1.57$ days per floor

Braces

$\frac{5.73}{14.2} = 0.4$ days per floor

Floor Deck

$\frac{22844}{3800} = 6.0$ days per floor

Shear Studs

$\frac{2566.5}{960} = 2.67$ days per floor

Concrete Reinforcement

$\frac{228.4}{31} = 7.4$ days per floor

Concrete Topping

$\frac{228.9}{140} = 1.4$ days per floor

Steel Joists

$\frac{2246.5}{2200} = 1$ day per roof

Roof Deck

$\frac{22705.5}{4900} = 4.6$ days per roof

M-2

Fire proofing Columns

$\frac{1476.7}{1100} = 1.3$ days per floor

Fire proofing Beams

$\frac{2784.2}{1500} = 1.9$ days per floor

Fireproofing Braces

$\frac{293}{1500} = 0.2$ days per floor

Fire proofing Joists

$\frac{1123.25}{1500} = 0.75$ days per roof

Fire proofing Deck

$\frac{22705.5}{1250} = 18.2$ days per roof

Concrete (assuming three crews)

Reinforcement - Columns

$\frac{17760.6}{4600 \cdot 3} = 1.3$ days per floor

Formwork - Columns

$\frac{5366.3}{200 \cdot 3} = 9$ days per floor

Concrete Placement - Columns

$\frac{123.9}{140 \cdot 3} = 0.3$ days per floor

Reinforcement - Beams

$\frac{20327.5}{5400 \cdot 3} = 1.25$ days per floor

M-3

Form work - Beams

$\frac{7567.65}{265 \cdot 3} = 9.5$ days per floor

Concrete - Placement Beams

$\frac{141.8}{90 \cdot 3} = 0.5$ days per floor

Reinforcement - Elevated Slabs

$\frac{102564}{5800 \cdot 3} = 5.9$ days per floor

Formwork - Elevated Slabs

$\frac{228329}{449 \cdot 3} = 17$ days per floor

Concrete Placement - Elevated Slabs

$\frac{731}{160 \cdot 3} = 1.5$ days per floor

Reinforcement - Walls

$\frac{16807.6}{8000 \cdot 3} = 0.7$ days per floor

Formwork - Walls

$\frac{6112.1}{280 \cdot 3} = 7.3$ days per floor

Concrete Placement - Walls

$\frac{117.2}{110 \cdot 3} = 0.4$ days per floor