MEMORIAL SLOAN-KETTERING CANCER CENTER

SOMERSET COUNTY, NEW JERSEY

JEFFREY SUTTERLIN

STRUCTURAL OPTION

SPRING 2006 SENIOR THESIS

DEPARTMENT OF ARCHITECTURAL ENGINEERING

Somerset County, New Jersey

Project Team

- •Owner: Sloan-Kettering Institute
- •Architect: Ewing Cole
- •Structural: Ewing Cole
- •General Contractor: Barr & Barr Builders
- •MEP: AKF Engineers

Project Overview

- •Four Stories Tall
- •85,000 square feet
- •Overall Cost: $41 Million
- •Construction taking place from September 2004 to June 2006
- •Project Delivery Method: Gross Maximum Price

Architectural

- •Exterior articulates soft undulating curves with large glass windows
- •Façade maintains a natural face, composed of brick and stone
- •Interior comprised of dynamic hallways filled with natural light and breathtaking views of the surrounding wilderness

Structural

- •Braced steel framing above grade with a concrete foundation
- •Masonry and glass curtain walls supported by structural framing
- •Floors: 4 ½" concrete slab on galvanized composite metal decking
- •Typical Bay Size: 30' x 30'

Electrical/ Lighting

- •15 kV power distribution to 15kV / 600A switch board
- •Uses 480/277V system
- •1000kW/1250kVA emergency generator
- •Interior uses incandescent, fluorescent, and day light

Mechanical

- •Interior air-handling has VAV system on every floor
- •Building perimeter uses continuous slot linear diffusers
- •Two 250-ton chillers located on building's south side
- •Air handling units located in basement and on roof

Jeff Sutterlin

Structural Option

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EXECUTIVE SUMMARY

This final report is a culmination of the year-long thesis work undertaken on Memorial Sloan-Kettering Cancer Research Center. Located in Somerset County, New Jersey, this four story health-care facility will open its doors in the summer of 2006 to serve as one of the premiere cancer treatment centers in the nation. A combination of steel, concrete, and masonry, MSK's layout includes a plethora of exam rooms, offices, and chemotherapy bays to compliment a Laboratory, Pharmacy, and radiotherapy treatment area. Furthermore, an 80,000+ square foot addition is still in its design stage and will later be constructed to the north side of the building, doubling the facility's size.

For this thesis, a study was performed to determine whether Memorial Sloan Kettering's Outpatient Addition would be both structurally and economically feasible if it were built vertically on top of the existing structure instead of to adjacent to it. The objective of this study was to design a structural system that effectively resisted both the gravity and lateral loads it experienced. To do so, the existing structure needed to be reanalyzed under the increased loads it was now experiencing. At four stories, Memorial Sloan Kettering was controlled exclusively by seismic loading. Conversely, once the infrastructure rose to 126 feet, wind loads significantly increased and generated the largest lateral forces. In respect to axial loading, the existing structure now has the weight of an additional five stories acting on it.

To help determine whether MSK displayed acceptable performance criteria under the necessary loading conditions, RAM Structural Program was used to analyze the infrastructure. The lateral loads developed in this analysis came from procedures outlined in ASCE 7-02. In addition, the redesign of this infrastructure utilized a building drift limitation of H/480 to ensure serviceability issues were addressed. In order to meet this criteria, a number of plausible lateral systems were investigated and the most efficient design was incorporated into the structure. The foundation of Memorial Sloan-Kettering was also examined and with the exception of a few increased footing sizes, everything remained structurally sound.

On top of designing an effective structural system, two breath studies were conducted to determine the practicability of a vertical expansion. A construction management study carried out both a cost analysis and time schedule of the proposed addition and compared those results with the initial plan. This comparison illustrated that a vertical expansion would be the more expensive option. The second study examined the building's mechanical system and how it would supply the five additional stories. A layout was created of the mechanical room in the basement, showing locations of all required equipment. Also, the 5th floor of Memorial Sloan Kettering was deemed a mechanical floor and now accommodates five air-handling units. To supply these units with outdoor air, louvers were designed to allow airflow into the area. Finally, an acoustic study was performed to determine whether additional soundproofing was needed between the mechanical room and those floors surrounding it. From the study, it was concluded that noise would not be a problem.

BUILDING SUMMARY

PROJECT INFORMATION

Memorial Sloan-Kettering Cancer Center is owned and operated by Sloan-Kettering Institute, a highly-respected organization dedicated to improving the understanding and treatment of cancer. To ensure that Memorial Sloan-Kettering Cancer Center maintained it's traditional of high standards, Sloan-Kettering Institute brought on a number of high profile firms to create this facility. Ewing Cole was put in charge of both the structural and architectural design of MSK. Barr & Barr Builders is responsible for the construction management services of the project. All environmental and geotechnical engineering fell into the hands of Langan Engineering & Environmental Services. AKF Engineers is the MEP firm for this project.

Primary Project Team

Owner: Sloan-Kettering Institute
www.ski.mskcc.org

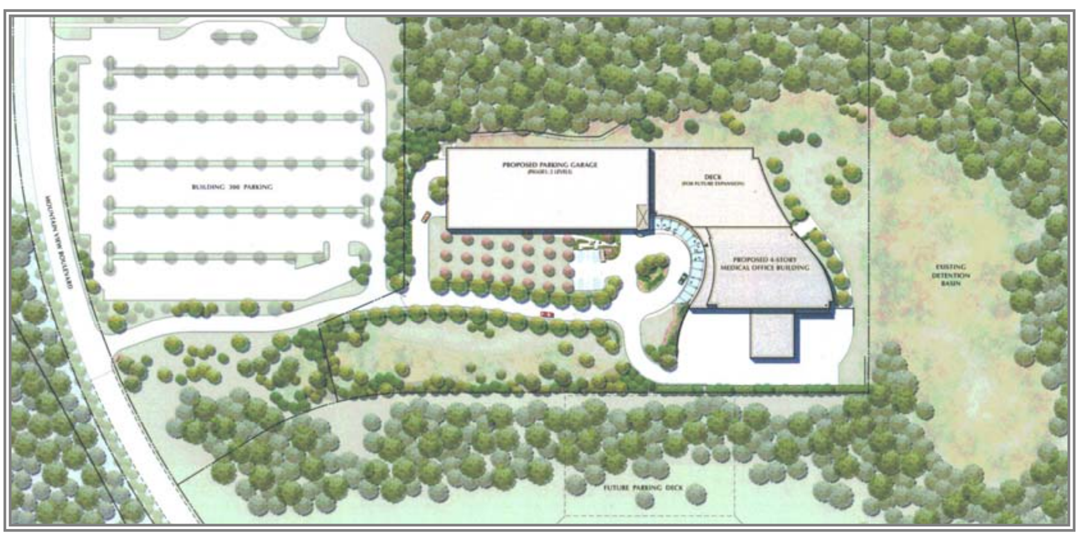
Structural Engineer: Ewing Cole
www.ewingcole.com

Architect of Record: Ewing Cole
www.ewingcole.com

MEP Engineer: AKF Engineers
www.akf-engineers.com

General Contractor: Barr & Barr Builders
www.barrandbarr.com

Memorial Sloan-Kettering's design has been divided into in two phases. The first phase is currently under construction will be opening its doors in the summer of 2006. This four story, 85,000 square foot facility will accommodate offices, exam rooms, chemotherapy bays, a laboratory, a pharmacy, and ambulatory surgery areas. The second phase of MSK is still in its design stage and will most likely break ground around 2009. The 80,000+ square foot addition is also four stories and will be built on the north side of Phase One. This extension will be home to physician practices, an education/prevention center, and a diagnostic imaging area. When fully completed, Memorial Sloan-Kettering will be one of the most equipped cancer treatment centers in the nation.



Images courtesy of Ewing Cole

Site Map

SITE LOCATION

Memorial Sloan Kettering is located at 400 Mountain View in Bernards Township, New Jersey. The site is bounded to the north by wooded wetlands and to the south, east, and west by future development. Approximately 35 miles from New York City, MSK provides a serene, relaxing atmosphere without secluding itself from the rest of the world.

SOIL SITE CONDITIONS

A geotechnical engineering study was conducted by Langan Engineering and Environmental Services to assess subsurface conditions and develop geotechnical recommendations for Memorial Sloan Kettering. Based on their study, it was determined that MSK can be supported by shallow foundations bearing directly on basalt bedrock or on decomposed rock. The footings bearing on the bedrock could have an allowable bearing capacity of 10 tsf while the footings on decomposed rock may only be designed for an allowable bearing capacity of 5 tsf.

From the subsurface conditions encountered, it is expected that the southern portion of the Phase One building will be founded on rock. The northern portion of Phase 2, however, will be founded on decompressed rock and thus will require compacted fill for its basement. The footings within the transition area between Phase 1 and 2 can be conservatively designed to rest on decompressed rock although bedrock may be provided. Furthermore, it was determined that Memorial Sloan-Kettering rests on Site Class "B" according to the 2000 IBC, New Jersey Edition.

ARCHITECTURE

Following Memorial Sloan-Kettering's tradition of "patient-oriented" cancer care, this facility is designed to create a serene environment for all of its patients. The actual building is strategically located on the north end of the 25-acre wooded lot to maximize patients' interaction with nature as they approach the building. The exterior of MSK articulates soft undulating curves with large windows. The façade's natural face, comprised of brick and stone, accents the calming views of the mountain surrounding it.

The interior of Memorial Sloan-Kettering creates a warm and reassuring, yet sophisticated, experience for its patients. The exterior curves of the building transform the interior by creating dynamic hallways filled with natural light and breathtaking views of the wilderness around it. Soft tones and textures, natural materials such as wood and stone, a large fish tank, and many other interior elements are fine-tuned to focus on the patient. From an architectural standpoint, Memorial Sloan-Kettering creates a soothing and relaxing atmosphere for patients and personnel alike.

Memorial Sloan-Kettering Model

BUILDING ENVELOPE

The building envelope of Memorial Sloan-Kettering consists of curtain walls supported by its structural steel framing. The overall façade of MSK is primarily brick with vertical strips of stone panel offset between windows to give the building its natural look. Behind the 3-5/8" brick face, these curtain walls are made up of mortar barrier mesh, 1 ½" thick rigid insulation, vapor barrier, and 8" grout-filled CMU's.

Building Envelope

Along with the brick face, Memorial Sloan-Kettering's front façade is full of large glass windows. These window frames span anywhere from 3 to 18 feet wide and have a typical of about 8' on the front elevation and 5'-4" on the remaining three faces. The windows are made up of 1" insulated glass with aluminum framing.

The front entrance of MSK is entirely glass, providing an open, welcoming feel to the building. Similarly, the northwest and southwest corners of the building are incased entirely in spandrel glass for the three highest floors, framing the brick facade inside and creating a symmetrical look. Above the front entrance of Memorial Sloan Kettering is a cantilevered canopy, providing shelter to entering patients and divides the glass façade from the brick. The canopy is made up of steel beams and ties directly into the structural framing of the building.

MECHANICAL

The mechanical and boiler rooms in Memorial Sloan-Kettering Cancer Center are both located in the basement. There are three large air handling units in the basement along with chiller water pumps, two boilers, hot water pumps, and a number of air separators. There are also three Rooftop Air Handling Units and coils located on top of the roof, each having a 3-phase, 480 volt energy supply.

The interior portion of MSK has a variable air volume system (VAV) for its HVAC system on every floor above grade. The perimeter areas of the building use continuous slot linear diffusers as its mechanical system. There are two 250 ton chillers located on the south side of the building which provide for all floors, including the roof. There is also room for two additional 250 ton chillers for Phase 2 of MSK.

ELECTRICAL/LIGHTING

The electrical system of Memorial Sloan-Kettering is a 15 kV service to a 15 kV/ 600A switch meter. This then proceeds to the new substation located in the main electrical room, coming in via three #4/0 15 kV aluminum cables and one #4/0 copper insulated 600V ground wire in a 5" conduit. This steps down to a 2500 kVa transformer, rated 12.47 kV. Emergency power is provided by a 1000 kw/1250kVa generator.

Memorial Sloan Kettering primarily uses downlighting to illuminate its interior. The offices and exam rooms are lit exclusively by 1' x 4' pendant fluorescent lighting. The laboratory/pharmacy and surgical areas, located on the third and fourth floors, also make use of fluorescent lighting. All of the corridors throughout the building are accented with wall washers. Daylight also permeated the perimeter of the building through the large glass windows on all sides.

Interior Lobby Space

Fire Protection

Memorial Sloan Kettering follows IBC fire-protection requirements. The floors and structural framing are constructed with a two-hour fire rating, while the roof maintains the required one-hour fire rating.

The interior of MSK also exhibits required fire protection. Two-hour rated fire walls are built throughout each floor to minimize the spread of a possible fire. All floors and elevator shafts are also equipped with automatic wet sprinkler systems. Emergency lighting, standard smoke detectors, pull stations, and exit signs are all found throughout the building to assure that all codes are met.

Transportation

The building is equipped with two passenger elevators, along with one larger service elevator. The two stairwells in MSK are located on opposite ends of the building. A third stairwell welcomes patients in the lobby and climbs up to the second floor.

STRUCTURAL SYSTEM

STRUCTURAL SYSTEM DESCRIPTION

INTRODUCTION

Memorial Sloan-Kettering utilizes a combination of steel and concrete to create its efficient structural system. Below grade, MSK consists of shear walls and piers made exclusively of reinforced concrete. The infrastructure switches over to steel at the first floor level and continues for the remainder of the building. W12 columns support the gravity loads while braced frames, spanning diagonally between floors, resist the lateral forces that act on MSK.

Because each steel column sits directly on top of a concrete pier, the typical bay size remains at 30'x 30' through much of the building. The only alteration begins on the second floor, where a number of columns on the south end of the building are removed, creating a more open floor plan. This architectural layout creates bays sizes of 30'x 45'on the building's south side for floors two through four. Furthermore, a number of bays are also reduced in size near the exterior walls of the building due to the Memorial Sloan Kettering's curved exterior façade.

Photos courtesy of Ewing Cole

EXISTING FLOOR SYSTEM

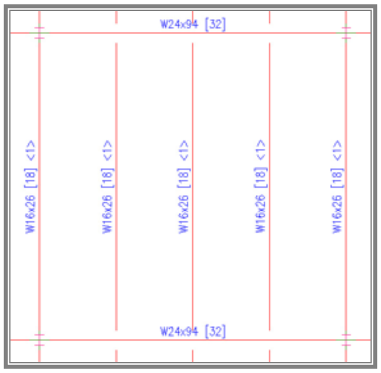
1st floor

MSK's first floor is constructed as a one-way concrete slab system. The 6" floor slab is supported by concrete beams spanning in the east-west direction and concrete girders spanning in the north-south direction. The concrete beams have a typical tributary width of 10' and span 30' between girders. The girders, in turn, span 30' from pier to pier. All beams are identical with an 18" x 24" dimension, and are reinforced by four #8 top bars and four #7 bottom bars. A typical girder's dimensions are 24" wide and 30" deep with top reinforcement of eight #9 bars and bottom reinforcement of six #8 bars.

First Floor Slab

2nd Floor through Roof

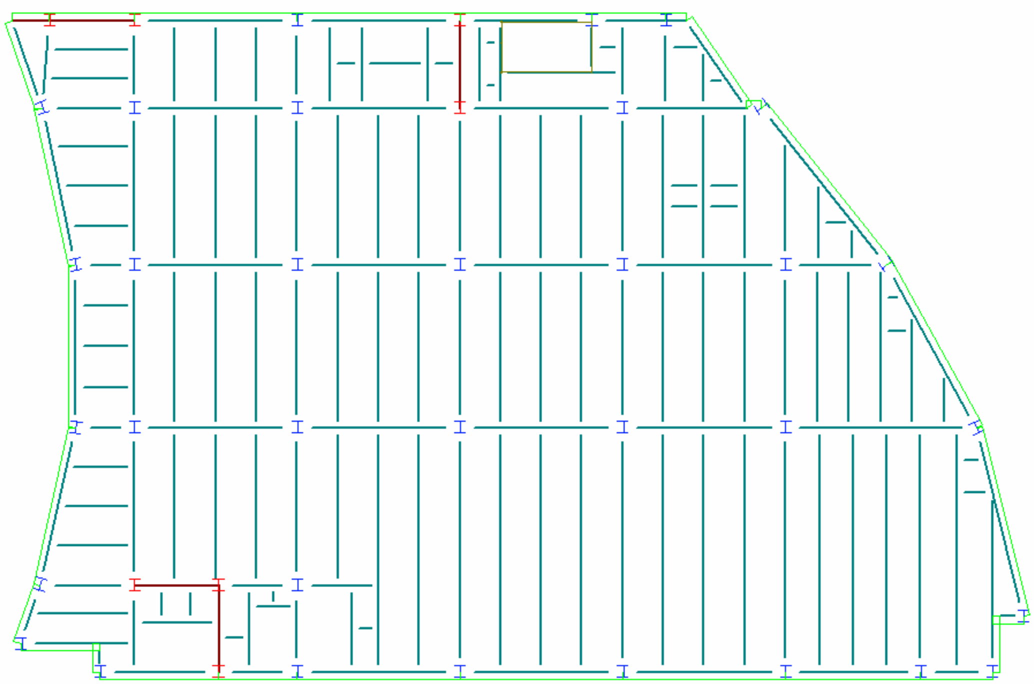
The existing floor system for Floors 2 through 4 in Memorial Sloan-Kettering is a composite concrete slab on metal decking. This system consists of a 4 ½" normal-weight concrete slab poured onto 2" 20-gauge galvanized metal decking. The slab is reinforced with 6x6-W2.9 x W2.9 welded wire fabric. The metal floor deck spans in the east – west direction and is continuous over a minimum of two or more spans. This decking ties into the wide flange steel beams through equally spaced ¾" diameter by 4" long headed shear studs welded into the center of the flange.



Typical Bay

This floor system is supported by steel beams and girders that span from column to column. Because the second floor maintains the typical 30' x 30' bay size, its framing members remain consistent. A typical interior beam size is W16x26 while a typical interior girder size is W24x96. For the smaller bays adjacent to the exterior walls, beam sizes decrease to W12x16.

As previously mentioned, the third and fourth floor layouts eliminate columns on the south side in order to create a more open space. Where the interior spans remain constant from the second floor, structural member sizes are maintained with W16x26 beams connecting into W24x96 girders. For the spans which become 30' x 45', beam and girder sizes increase to W24x62 and W30x90, respectively.



3rd Floor Structural Framing

GRAVITY COLUMNS

With the exception of those columns framing the building's lateral braces, all columns in Memorial Sloan-Kettering are designed as gravity columns. These steel columns vary in size throughout MSK depending on their location and purpose. A typical interior column has a tributary area of 900 square feet ranges between W12x87 and W12x96. Columns near the exterior walls are typically smaller, ranging between W12x 45 and W12x72.

These steel columns connect into the concrete piers below through ASTM A572, Grade 50 steel base plates. The base plates used for these connections are dimensioned at 18"x 18" and are typically 1 ½" thick. The plates are secured in place by four ¾" A449 anchor bolts embedded 2' into the concrete below.

LATERAL SYSTEM

The lateral force resisting system of Memorial Sloan Kettering is made up of a vertical combination of shear walls and steel cross-bracing. The four shear walls are located below grade and are all positioned near the exterior, typically around stairwells or elevator shafts. This positioning creates a lateral system that does not protrude into the interior office space of the building. At grade level, these shear walls connect into steel columns though the base plates described earlier. These columns span the remaining four floors to the roof and frame the lateral bracing. Two lateral systems span in the north-south direction and two span in the east-west direction.

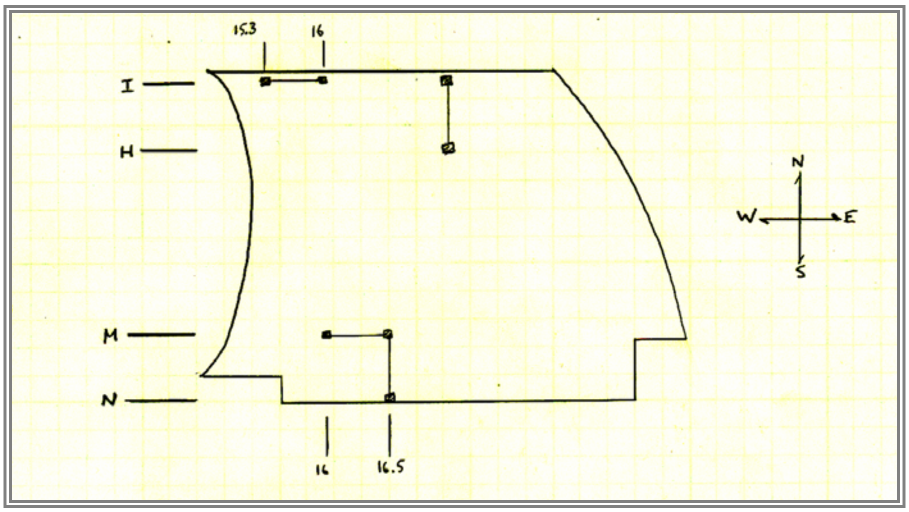
Lateral Cross-Bracing

The first lateral system oriented in the north-south direction is located on the north side of MSK, between column lines H and I. This system is comprised of a 12" thick shear wall spanning between the first floor and foundation. Once above grade, this wall connects into two W12x79 columns through a 1 ½" thick base plate. These two columns sizes remain the same throughout the four floors above grade; however, the diagonal bracing between them does not. Between the 1st and 2nd floor, two HSS 8x8x½ members span diagonally through the steel frames and are braced at midspan by a ¾" gusset plate. The bracing between the 2nd and 3rd floors also consists of two diagonal HSS 8x8x ½ members. These braces gradually become smaller, with two HSS 7x7x½ steel members between the third and fourth floors. The system culminates with two HSS 6x6x ½ members between the fourth floor and the roof.

The second lateral system oriented in the north-south direction in located on the southwest end of MSK, between column lines M and L. This lateral system is slightly smaller with two HSS 7x7x ½ diagonal members spanning between the first and second

floor and supported beneath by a 12" thick shear wall. The remaining three floors reduce the diagonal member size to two HSS 6x6x ½'s spanning between floors.

The two lateral systems running in the E – W direction follow the same framing as the two systems previously described. The larger system is located in the S-W corner of MSK, between column lines 16 and 16.5. The slightly smaller system is located against the northern wall of the building, between column lines 15.3 and 16. The sketch below demonstrates where each lateral system is located within the building.



Lateral System Locations

SHEAR WALLS

As previously mentioned, shear walls are located on the north and south sides of Memorial Sloan-Kettering surrounding the basement's concrete stairwells and framing into supporting columns. These 12" thick shear walls span in both the N-S and E-W directions and are approximately 14' long. Two of these walls span in the N-S direction and two span in the E-W direction. Each shear wall is reinforced vertically with #5 bars at 12" on center for both faces of the wall. These two faces are tied together with #4 ties spaced 12" on center. Similarly, the horizontal reinforcement on each wall face is made up of #5 bars at 12" on center. The columns supporting these shear walls have sixteen #9 bars of vertical reinforcement, about twice as much as that found in a typical column.

The lateral system is tied into concrete footings beneath each shear wall that have a minimum depth of 4 feet below the basement floor. The footings around each shear wall also extend at least 4 feet beyond the face of wall to create a plan dimension of 8' wide by 30' long. These massive footings are created to be large enough to counteract the overturning moments produced by the wind and seismic forces acting on the building.

FOUNDATION

The lateral system of Memorial Sloan-Kettering is supported by a shallow foundation that sits directly on top of basalt bedrock. Reinforced concrete piers, spaced in 30' x 30' bays, support the steel structural system above. Spanning between the piers at the basement level are concrete grade beams which provide support for the basement slab. Furthermore, each pier rests on a 6' x 6' footing, typically four feet thick, that disperses the axial loads uniformly.

LATERAL LOAD DEVELOPMENT

The wind and seismic loads acting on the existing structure of Memorial Sloan-Kettering have been calculated from the methods provided by ASCE 7-02. Seismic forces were found using the Equivalent Lateral Force Method, outlined in Chapter 9 of the code. Because MSK is a healthcare facility, a number of design parameters are required to be increased in order to reach an adequate safety factor. For instance, due to being a healthcare facility, Memorial Sloan-Kettering uses Seismic Use Group III and has an Occupancy Importance Factor of 1.5. Because of these factors, seismic loading produces relatively large forces acting on the building.

Due to the irregular shape of MSK, the wind forces acting on the existing structure were found using the Analytical Method, provided in Chapter 6 of the code. Once again, a healthcare facility warrants a higher Importance Factor and Design Category. Because the existing infrastructure of Memorial Sloan-Kettering is four stories tall, its natural frequency value is above 1.0 and is therefore deemed a rigid structure.

CONTROLLING LATERAL FORCE

After analyzing both lateral forces on the existing infrastructure of Memorial Sloan-Kettering, it becomes apparent that the seismic forces control the lateral loads in both directions. When comparing the both shears created by the lateral loads, seismic generates 349 kips whereas 225 kips due to wind. The building's drift is also controlled in both directions by seismic loads. The center of rigidity is displaced approximately 1.37 inches in the north-south direction and 1.70 inches in the east-west direction.

Exponent k_{N-S} : 0.954711

North - South Direction							
Level, x	w_x	h_x	$w_x h_x^k$	C_{VX}	F_x	V_x	M_x
	kips	feet			kips	kips	ft-kips
Roof	1622	56	75,676	0.333	116.3		6510.3
4	2106	42	74,691	0.329	114.7	116.3	4819.2
3	2106	28	50,717	0.223	77.9	231.0	2181.5
2	2106	14	26,167	0.115	40.2	308.9	562.8
1						349.1	
Σ	7941		227251.4	1	349.1		14073.8

Seismic: Equivalent Lateral Force Method (ASCE 7-02)

PROBLEM STATMENT

PROBLEM STATEMENT

After an extensive analysis of Memorial Sloan-Kettering's structural system, it was apparent that the existing design of this building was expertly performed. Both hand calculations and software analysis arrived at the same conclusion that the structure of MSK was sufficiently designed. Even from an economic point of view, Memorial Sloan-Kettering left little room for improvement. In the north Jersey area, steel is less expensive to build in than concrete. The only reason the entire building wasn't erected in steel was due to vibration and noise issues from the adjacent underground parking. Also consider that concrete is a heavier material which will directly result in larger seismic forces and footing sizes. As a final point, the original floor system was contrasted to four alternate systems to determine its efficiency. After analysis and comparison, the composite slab on deck proved to be the most effective floor system of the group.

After further deliberation, it was resolved that instead of changing the existing structural system of MSK, it would be better to redesign the Outpatient Addition. As noted earlier, a four-story Outpatient Addition is currently in its design stage and plans on breaking ground around 2009. The current site plan calls for the addition to be erected on the north side of the MSK, extending the signature curved façade that it possesses an additional 120 feet. It is assumed that this lateral addition to the building is due to the amount of open space provided on the site. There are, however, a few drawbacks that arise when building the addition adjacent to the north face of the existing structure.

The main drawback deals with constructing the addition's foundation. Because Phase 2 of Memorial Sloan-Kettering is basically a reproduction of Phase 1, it would be fair to assume that the addition would need the same foundation to support its four stories. By doing so, the site would need to be excavated to provide that footprint. In addition, the footings for Phase 2 would need to be enlarged due to the findings of the geotechnical engineers. From their geotechnical report, it states that the Outpatient Addition would sit on decomposed rock and would need an over-excavation up to feet 10 or more just to reach the required bedrock. If the addition were to be built vertically above the existing structure rather then to its north side, these drawbacks would be eliminated all together. These issues summarize why an investigational thesis was preformed to conclude whether a vertical addition would prove to be a more beneficial design.

The objective of this thesis was to investigate the structural design of both the existing infrastructure and Outpatient Addition as if the original design called for the addition's four stories to be erected on top of the MSK. Without changing the existing structural floor plan, both the gravity and lateral system were analyzed and redesigned to maintain structural integrity from the loads generated from the increased height and weight. In order to keep the Rooftop Air-Handling Units where they are, the future 5th floor of Memorial Sloan Kettering acted as a mechanical floor and had five additional stories erected above it. In all, Memorial Sloan Kettering will stand nine stories tall.

SOLUTION OVERVIEW

STRUCTURAL REDESIGN

By building this addition vertically above the existing floors, the height of Memorial Sloan- Kettering more than doubled from 58 feet to 126 feet. Because of this, the first step in this structural redesign was to recalculate both the gravity and lateral loads acting on the building. Dealing first with the gravity loads, there were a number of factors which required the columns be re-evaluated in the existing structural system. These columns, intended to only support four floors, were dramatically under-designed to resist the weight that the addition provided. Because of that, those columns had to be redesigned to support the axial forces acting on them. While the size of each column increased, the sizing was required to remain a W12. After completing the redesign of those columns, it was then necessary to redesign the concrete piers located below the first floor. Once again, their dimensions of 24”x24” were maintained to ensure that there was no interference with the floor plan. When dealing with the gravity loads, only the columns were considered during this redesign. Because the floor loads and building materials are remained constant, there is not need to redesign the floor systems in MSK.

The next step to this redesign was to look at the structure’s lateral system. After both the seismic and wind forces were recalculated, it was confirmed that the seismic and wind loads had significantly increased in both the north-south and east-west directions. Because of the noticeable increase, it was determined that the lateral system required additional braced frames to resist the forces and torsion acting on the building. Furthermore, the member components of each lateral system needed to be resized to ensure that an acceptable story drift of H/480 was maintained while resisting the lateral forces.

Finally, the foundation of Memorial Sloan Kettering was analyzed. Because more lateral systems were added to resist the updated lateral forces, the number of shear walls in each direction increased as well. Each shear wall needed to be analyzed to determine whether or not it was able to withstand both the base shear and overturning moment that acted on it. The footings supporting each shear wall were also resized in order to help resist excessive overturning moments. From the calculations performed, appropriate shear wall sizes and footing dimensions were allocated to maintain structural integrity.

To help assist in the analysis of the lateral loads and how each influenced the lateral frames, a three-dimensional model of Memorial Sloan-Kettering was created in RAM Structural System. RAM has the ability to both analyze and design a building based on the loads and parameters assigned in the program. Not only did RAM produce constructive output data relating directing to the structural design of this addition, but it also provided a way to double check all hand calculations.

CONSTRUCTION MANAGEMENT STUDY

After the structural analysis of Memorial Sloan Kettering was complete, it was necessary to look at the addition from an economic point of view in order to determine how efficient the redesign was. Because the Outpatient Addition is still in its design stage, there are no tangible costs that would allow for straight comparison. Because of this, an assumption was made that if the addition were built adjacent to the existing infrastructure, its structural components would cost roughly the same due to their similarities. From that assumption, structural costs were taken from the Financial Status Report provided by Barr & Barr Builders, Inc.. To find the cost of only the addition, it was necessary to add up the price of the entire infrastructure and subtract out the existing structure's cost. Doing so would find the price of the additional five floors and take into account the cost of resized members on the lower floors. For this study, the structural steel and concrete for both additions were analyzed and compared. Also, a schedule was created to determine how long the redesign would take to construct. That time frame was also compared to that of the original schedule.

MECHANICAL AND ACOUSTIC STUDY

When dealing with a multiple story addition such as the one proposed onto MSK, structural integrity is not the only technical issue that arises. A proper mechanical system must also be established for those stories, and an issue that arises within this subject is where to put that equipment. For the initial four stories, a large mechanical room exists in the basement along with three additional air handing units on the roof. Fortunately, the basement's mechanical room layout left open room for most of the equipment required for the addition. The only mechanical units still needing placement were the air-handlers for addition.

Due to the amount of room needed for those air-handing units, the logical option was to leave the existing units where they were. Because of this, the 5th floor of Memorial Sloan-Kettering has been converted into a mechanical floor for both the addition and existing building. The three air-handlers will remain where they are, and two additional units will be added on the level to supply the 6th and 7th Floors. The remaining 8th and 9th floors will receive air from units positioned on the addition's roof. In order to supply the required outdoor air to those units on the 5th floor, louvers were sized and positioned on all four exterior walls to allow air to flow freely through that level.

One consideration that this new building configuration brought up was whether there would be noise issues between the mechanical room and floors surrounding it. The 4th Floor is home to both examination rooms and surgical areas. The 6th floor will be home to practicing offices. To identify whether or not further acoustical measures were necessary, an acoustic study was performed to determine the noise levels experienced in both an operating room and private office.

STRUCTURAL REDESIGN

STRUCTURAL REDESIGN

INTRODUCTION

With the redesign of Memorial Sloan Kettering's Outpatient Addition, both the existing structure and vertical expansion needed to be accessed structurally for gravity and lateral loads. Adding five stories onto Phase One significantly increased the axial forces acting on the existing columns. Furthermore, the addition's height and weight increase directly amplified the wind and seismic loads, respectively. These increased loads required additional braced systems to counteract the building's drift. MSK's foundation components were also analyzed and enhanced to withstand the loads acting on them. Concrete piers supporting gravity columns were increased in compressive strength while shear walls and their footings were resized to withstand the base shear and overturning moments acting on them. When completed, the redesigned structure was structurally sound under its new loading.

GRAVITY SYSTEM

GRAVITY DESIGN CRITERIA

The following codes were used in the structural redesign of Memorial Sloan-Kettering:

National Code: *International Building Code 2000*

Design Codes:

- *American Society of Civil Engineers* (ASCE 7-02)
- *American Institute of Steel Construction* (AISC – 3rd Edition)
- *ASTM Standards – Properties of Building Materials*

Existing Gravity Loads:

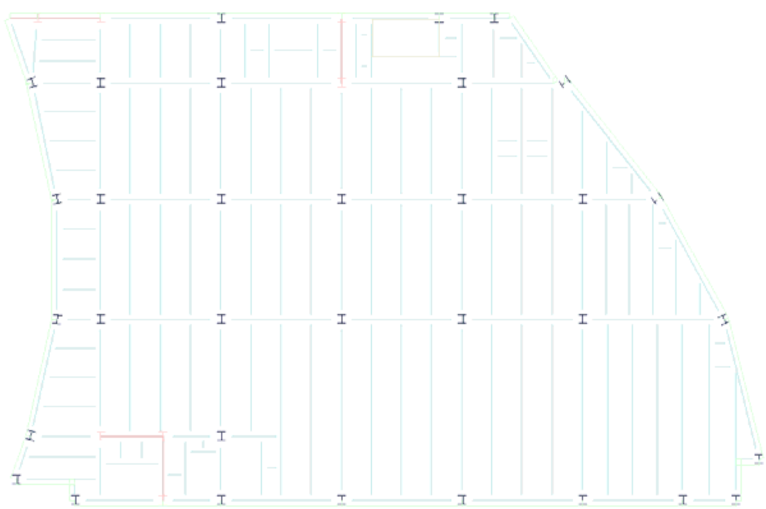
Gravity Loads			
Floor: 2nd - 9th		***Floor:*** 5th	
Dead		Dead	
56 psf	slab on deck	56 psf	slab on deck
2 psf	metal deck	2 psf	metal deck
12 psf	steel framing	12 psf	steel framing
15 psf	superimposed	65 psf	mechanical
85 psf		**135 psf**	
Floor: Roof		Live - **100 psf**	
Dead		*(Table 4-1) ASCE7-02*	
46 psf	slab on deck		
2 psf	metal deck		
12 psf	steel framing		
65 psf	mechanical		
125 psf		Snow - **23 psf**	
****** See Appendix A for Load Calculations**			

The current loading found on the floors of Memorial Sloan-Kettering are listed to the left.

The live load value of 100 psf was taken from Table 4-1 found in ASCE7-02. The same live load value was used in the initial design of MSK.

GRAVITY COLUMN REDESIGN

The first task in the redesign of Memorial Sloan-Kettering was to resize the gravity columns to support the increased axial loads. After adding five stories to the existing structure, it was obvious that its columns were well under-designed to withstand the weight from the floors above. The tributary area was determined for each column along with the gravity loads each of those floors received. Live Load Reduction Factors were also assigned to maintain realistic and economic column sizes. A spreadsheet was created to layout the axial loads cumulated on the 2nd floor columns for different locations and can be referenced in Appendix B. Once preliminary column sizes were established, a RAM model was created with appropriate loadings. The columns were then designed using that program and after comparison, both analyses produced very similar sizes. This confirmed that the RAM model was working properly. Shown below is a floor plan highlighting the gravity columns. A chart comparing the column sizes reached by hand calculation against the sizes developed by RAM can be viewed on the next page.

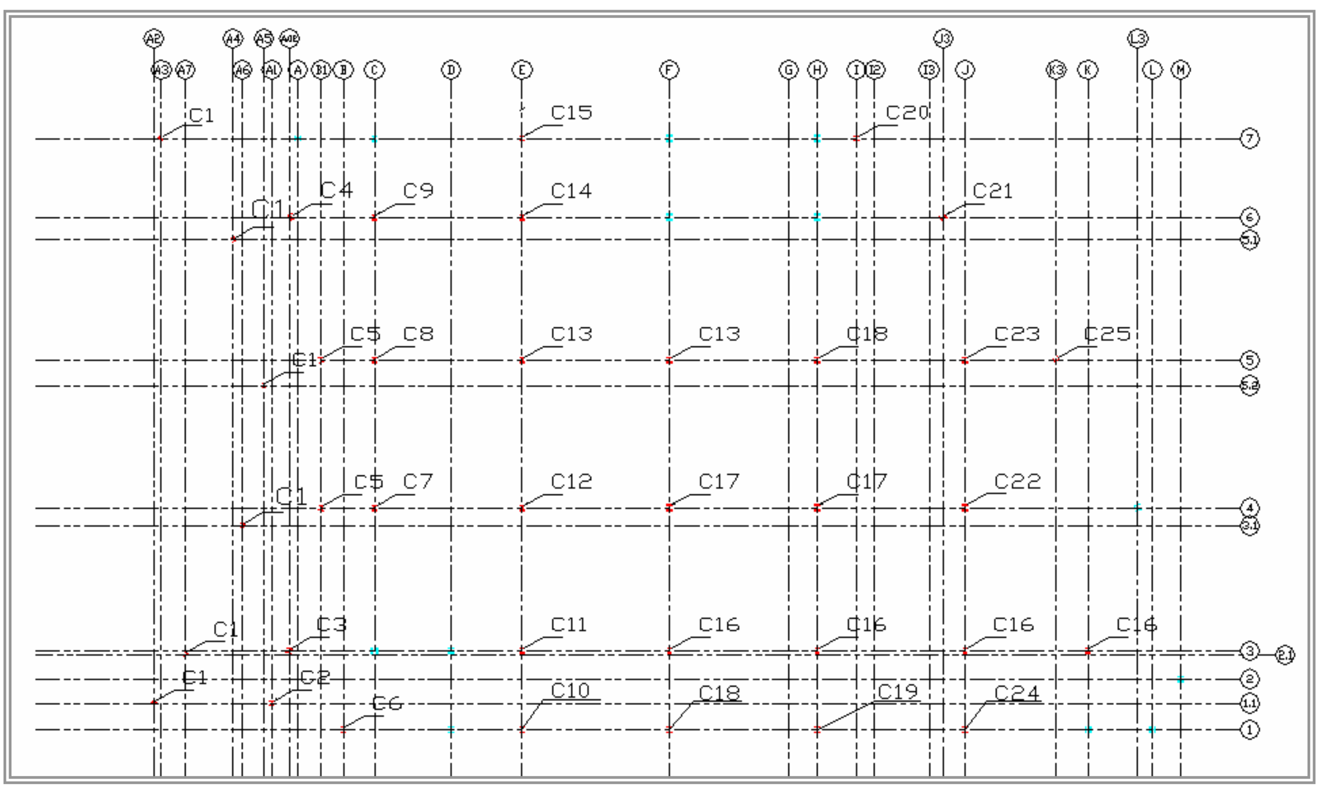


Gravity Column Locations

When choosing appropriate column sizes, there were some design conditions that needed to be met to ensure that the architectural building layout would not be compromised. The first criterion was that the new column sizes were required to remain W12's, even if another size was slightly more economical. By doing so, the column schedule remained clear and simple and there would be no question to whether the new sizes would impede on wall thicknesses. Another criterion that was administered for this task was that columns with the same location would take on the same column sizes. For instance, if two interior columns shared similar tributary areas, both would be sized the same. This action would create repetitiveness, and further simplify both the column schedule and Memorial Sloan Kettering's steel order.

Gravity Column Redesign Comparison				
Hand Calcuations vs. RAM Design				
	Column Location	Hand Calculations		RAM
		Force (kips)	Size	Size Given
Design A	Typical Interior Column	1455.74	W12x152	W12x152
Design B	Atypical Interior Column (South Side)	1778.96	W12x190	W12x190
Design C	Typical Exterior Column (South Side)	1326.75	W12x136	W12x136
Design D	Typical Exterior Column (North Side)	712.36	W12x72	W12x79
Design E	Atypical Interior Column (North Side)	1120.48	W12x120	W12x120

Below is the finalized gravity column layout for Memorial Sloan-Kettering Cancer Center. The column schedule referencing this image can be found on the following page. To briefly explain where these column lines are in relation to the building's structural layout, Column Line 7 represents the north exterior wall of MSK (Design D). Column Line 1 represents the building's south wall (Design C), and Column line 4 denotes the columns that support the 45'x30' bays (Design B). The members on Column Line 3 are removed after the second floor to create a more open layout.



Column Layout

	C1	C2	C3	C4	C5	C6	C7	C8	C9	C10	C11
Roof – 9th Floor		W12X40	W12X40	W12X40	W12X40	W12X40	W12X40	W12X53	W12X50	W12X40	W12X40
9th Floor – 8th Floor		W12X40	W12X40	W12X40	W12X40	W12X40	W12X40	W12X53	W12X50	W12X40	W12X40
8th Floor – 7th Floor		W12X40	W12X45	W12X50	W12X50	W12X40	W12X65	W12X65	W12X65	W12X53	W12X40
7th Floor – 6th Floor		W12X40	W12X45	W12X50	W12X50	W12X40	W12X65	W12X65	W12X65	W12X53	W12X40
6th Floor – 5th Floor		W12X40	W12X45	W12X50	W12X50	W12X40	W12X65	W12X65	W12X65	W12X53	W12X40
5th Floor – 4th Floor		W12X40	W12X53	W12X53	W12X58	W12X40	W12X87	W12X96	W12X79	W12X58	W12X53
4th Floor – 3rd Floor		W12X40	W12X53	W12X53	W12X58	W12X40	W12X87	W12X96	W12X79	W12X58	W12X53
3rd Floor – 2nd Floor		W12X45	W12X65	W12X65	W12X72	W12X53	W12X120	W12X120	W12X96	W12X65	W12X65
2nd Floor – Base	W10X33	W12X45	W12X65	W12X65	W12X72	W12X53	W12X120	W12X120	W12X96	W12X65	W12X65
BASEPLATE											
COLUMN MARK	C1	C2	C3	C4	C5	C6	C7	C8	C9	C10	C11

MARK ORDER: BY COORDINATES

COLUMN SCHEDULE
SCALE: 1/8" = 1'

	C12	C13	C14	C15	C16	C17	C18	C19	C20	C21	C22	C23	C24	C25
Roof – 9th Floor	W12X58	W12X58	W12X53	W12X40		W12X65	W12X50	W12X50	W12X40	W12X50	W12X65	W12X40	W12X50	W12X40
9th Floor – 8th Floor	W12X58	W12X58	W12X53	W12X40		W12X65	W12X50	W12X50	W12X40	W12X50	W12X65	W12X40	W12X50	W12X40
8th Floor – 7th Floor	W12X96	W12X87	W12X65	W12X58		W12X120	W12X87	W12X96	W12X40	W12X65	W12X120	W12X72	W12X87	W12X65
7th Floor – 6th Floor	W12X96	W12X87	W12X65	W12X58		W12X120	W12X87	W12X96	W12X40	W12X65	W12X120	W12X72	W12X87	W12X65
6th Floor – 5th Floor	W12X96	W12X87	W12X65	W12X58		W12X120	W12X87	W12X96	W12X40	W12X65	W12X120	W12X72	W12X87	W12X65
5th Floor – 4th Floor	W12X136	W12X120	W12X87	W12X72		W12X152	W12X120	W12X136	W12X50	W12X87	W12X170	W12X96	W12X120	W12X87
4th Floor – 3rd Floor	W12X136	W12X120	W12X87	W12X72		W12X152	W12X120	W12X136	W12X50	W12X87	W12X170	W12X96	W12X120	W12X87
3rd Floor – 2nd Floor	W12X152	W12X152	W12X120	W12X79		W12X190	W12X152	W12X152	W12X65	W12X106	W12X210	W12X120	W12X136	W12X106
2nd Floor – Base	W12X152	W12X152	W12X120	W12X79	W12X40	W12X190	W12X152	W12X152	W12X65	W12X106	W12X210	W12X120	W12X136	W12X106
	C12	C13	C14	C15	C16	C17	C18	C19	C20	C21	C22	C23	C24	C25

LATERAL SYSTEM

LATERAL DESIGN CRITERIA

The lateral loads acting on Memorial Sloan-Kettering were recalculated to account for the increased height and weight the Outpatient Addition would provide. Wind loads were found using the Analytical Approach outlined in Chapter 6 of ASCE 7-02. The five additional stories pushed MSK way beyond 60 feet in height, making this method the most precise. Also, its curved façade constitutes an irregularly shaped building, which provides yet another reason for using the Analytical Approach. Below are the results of this method, breaking down the load into story forces. Full calculations and design parameters can be found in Appendix A.

Wind Analysis (Analytical Approach)								
CASE 1			Story Force		Cumulative Shear		Overturning Moment	
Level	Trib. Height (ft)	Total Height (ft)	N-S	E-W	N-S	E-W	N-S	E-W
Roof	*7.00*	126.00	41.97	25.77	0	0	5287.96	3246.79
9	*14.00*	112.00	82.29	50.44	41.97	25.77	9216.09	5649.25
8	*14.00*	98.00	81.05	49.62	124.25	76.21	7942.85	4862.49
7	*14.00*	84.00	79.40	48.52	205.30	125.83	6669.61	4075.73
6	*14.00*	70.00	77.67	47.37	284.70	174.35	5436.78	3315.84
5	*14.00*	56.00	75.58	45.98	362.37	221.71	4232.43	2574.88
4	*14.00*	42.00	73.13	44.35	437.95	267.69	3071.56	1862.84
3	*14.00*	28.00	69.97	42.25	511.08	312.05	1959.19	1183.04
2	*14.00*	14.00	65.52	39.29	581.06	354.30	917.25	550.07
1	7.00	0.00	0.00	0.00	**646.57**	**393.59**	**44733.72**	**27320.92**

Wind Analysis (Analytical Approach)													
CASE 3 (75% simultaneous directions)			NW-SE direction			NE-SW Direction			Cumulative Shear		Overturning Moment	
Level	Trib. Height (ft)	Total Height (ft)	N-S	E-W	Total	N-S	E-W	Total	NW-SE	NE-SW	NW-SE	NE-SW
Roof	*7.00*	126.00	31.48	19.33	**36.94**	31.48	19.33	**36.94**	0	0	4653.88	4653.88
9	*14.00*	112.00	61.71	37.83	**72.39**	61.71	37.83	**72.39**	36.94	36.94	8107.30	8107.30
8	*14.00*	98.00	60.79	37.21	**71.27**	60.79	37.21	**71.27**	109.32	109.32	6984.78	6984.78
7	*14.00*	84.00	59.55	36.39	**69.79**	59.55	36.39	**69.79**	180.60	180.60	5862.26	5862.26
6	*14.00*	70.00	58.25	35.53	**68.23**	58.25	35.53	**68.23**	250.38	250.38	4776.11	4776.11
5	*14.00*	56.00	56.68	34.49	**66.35**	56.68	34.49	**66.35**	318.61	318.61	3715.60	3715.60
4	*14.00*	42.00	54.85	33.26	**64.15**	54.85	33.26	**64.15**	384.96	384.96	2694.23	2694.23
3	*14.00*	28.00	52.48	31.69	**61.30**	52.48	31.69	**61.30**	449.11	449.11	1716.50	1716.50
2	*14.00*	14.00	49.14	29.47	**57.30**	49.14	29.47	**57.30**	510.42	510.42	802.16	802.16
1	*7.00*	0.00	0.00	0.00	**0.00**	0.00	0.00	**0.00**	**567.71**	**567.71**	**39312.82**	**39312.82**

Wind Load Cases 1 and 3

After comparing the results of the Analytical Approach, it was determined that with wind, Load Case 1 generated the strongest lateral forces on Memorial Sloan Kettering at 647 kips. This proved to be a substantial increase in wind lateral forces, seeing that this same load case only created 226 kips on MSK when it was four stories. By doubling the height of the building, the wind loads acting on more than doubled as well. Furthermore, the increase in height lowered the building's natural frequency to the extent to where it is now a flexible structure. This amplified force proved to be a formidable challenge to resist during the lateral redesign of this addition. Although only Load Cases 1 and 3 were calculated by hand, the RAM model took into account all four load cases when performing its analysis.

The seismic forces were found using the Equivalent Lateral Force Method, outlined in Chapter 9 of ASCE7-02. Much like the wind loads, these seismic forces were projected to increase due to the extra weight created by those five additional stories. Because Memorial Sloan Kettering is a healthcare facility, many of its safety parameters are larger then a typical building's. Falling under the category of a healthcare facility automatically denotes a Seismic Use Group III building and an Importance Factor of 1.5. These provisions increase the seismic design loads, demanding a more rigid lateral system for the structure. This also explains why this structure was made out of steel and not the heavier concrete. Below are the results of the Equivalent Lateral Force Method. Full calculations and a list of the seismic design parameters can be referenced in Appendix A.

Vertical Distribution of Seismic Forces

North - South Direction

Level, x	w_x	h_x	$w_x h_x^k$	C_{VX}	F_X	V_x	M_x
	kips	feet			kips	kips	ft-kips
Roof	1622	126	375,948	0.171	75.5		9512.4
9	2106	112	427,675	0.194	85.9	75.5	9618.8
8	2106	98	367,968	0.167	73.9	161.4	7241.5
7	2106	84	309,331	0.141	62.1	235.3	5217.9
6	2106	70	251,918	0.114	50.6	297.4	3541.2
5	2106	56	195,943	0.089	39.3	348.0	2203.5
4	2106	42	141,723	0.064	28.5	387.3	1195.3
3	2106	28	89,773	0.041	18.0	415.8	504.8
2	2106	14	41,131	0.019	8.3	433.8	115.6
1						442.1	
Σ	7941		2,201,410	1.000	442.1		39151.0
	Exponent k_{N-S} :	1.126078					

Vertical Distribution of Seismic Forces

East - West Direction

Level, x	w_x	h_x	$w_x h_x^k$	C_{VX}	F_X	V_x	M_x
	kips	feet			kips	kips	ft-kips
Roof	1622	126	375,948	0.171	75.5		9512.4
9	2106	112	427,675	0.194	85.9	75.5	9618.8
8	2106	98	367,968	0.167	73.9	161.4	7241.5
7	2106	84	309,331	0.141	62.1	235.3	5217.9
6	2106	70	251,918	0.114	50.6	297.4	3541.2
5	2106	56	195,943	0.089	39.3	348.0	2203.5
4	2106	42	141,723	0.064	28.5	387.3	1195.3
3	2106	28	89,773	0.041	18.0	415.8	504.8
2	2106	14	41,131	0.019	8.3	433.8	115.6
1						442.1	
Σ	7941		2,201,410	1.000	442.1		39151.0
	Exponent k_{E-W} :	1.126078					

By performing the Equivalent Lateral Force Method, it was confirmed that the seismic loads acting on Memorial Sloan Kettering had in fact increased to 442 kips in both the north-south and east-west directions. This increase in the seismic design forces is primarily due to the structure's weight increase rather then its height increase. With the exception of the approximate fundamental period, all the parameters used in this analysis method are affected either by the site location or weight. That helps clarify why the seismic loads did not increase to the extent that the wind loads did.

CONTROLLING LATERAL FORCE

After analyzing both the wind and seismic forces acting on Memorial Sloan-Kettering, it was discovered that wind now created the largest overall lateral loads on the structure. This is a change from the existing structure, where seismic controlled the lateral design in both directions. However after erecting an additional 68 feet onto the structure, the wind loads had increased by 190%. Wind provided the controlling lateral load in the north-south direction with 647 kips compared to 442 kips generated by seismic. Seismic, however, still controlled in the east-west direction over wind, with 442 kips and 393 kips, respectfully. This is due to the fact that Memorial Sloan Kettering is only 66% as wide in the east-west direction as it is in the north-south direction, creating a smaller tributary area. Both wind and seismic also proved to be controlling factors for drift in their dominating directions.

LATERAL REDESIGN

INTRODUCTION

The preliminary lateral force resisting system of Memorial Sloan Kettering was made up of braced frames positioned around the building's elevator shafts and stairwells. By doing so, minimal interference was created with both the interior layout and architectural façade. X bracing was chosen due to the high stiffness it provides in a relatively small area. The new lateral system makes use of this configuration to maintain its stiffness.

A braced frame is an effective way of resisting lateral loads on a building because the produced lateral shear forces are resisted by the diagonal members spanning between bays. By adding cross bracing into a framed bay, the system is basically transformed into a vertical truss. This action eliminates the majority of the bending from the columns. A high stiffness is achieved with braced frames because the story shear is now being absorbed axially by the braced instead of with through bending moments with the columns. These braces take the axial forces and transfer them into the framing members through axial loads, eliminating bending moment deformation. Because of the efficiency of this system in regards to MSK, braced frames will remain the primary force resisting system during the lateral system redesign.

LIMITING FACTORS AND DESIGN GOALS

In order to determine how effective the proposed experimental braced frame systems would in Memorial Sloan-Kettering, a number of limiting factors had to be considered. These aspects had a direct influenced on the location and type of each of the braced frames. The limiting facts are as follows:

Limiting Factors:

- Calculated wind loads control lateral design in the north-south direction. See Appendix A.
- Calculated seismic loads control lateral design in the east-west direction. See Appendix A.
- Lateral system shall be positioned to minimize interference with the architectural layout. This includes façade windows, doors, and hallways.
- Use concentrically braced frames whenever possible in order to maximize that frame's stiffness.

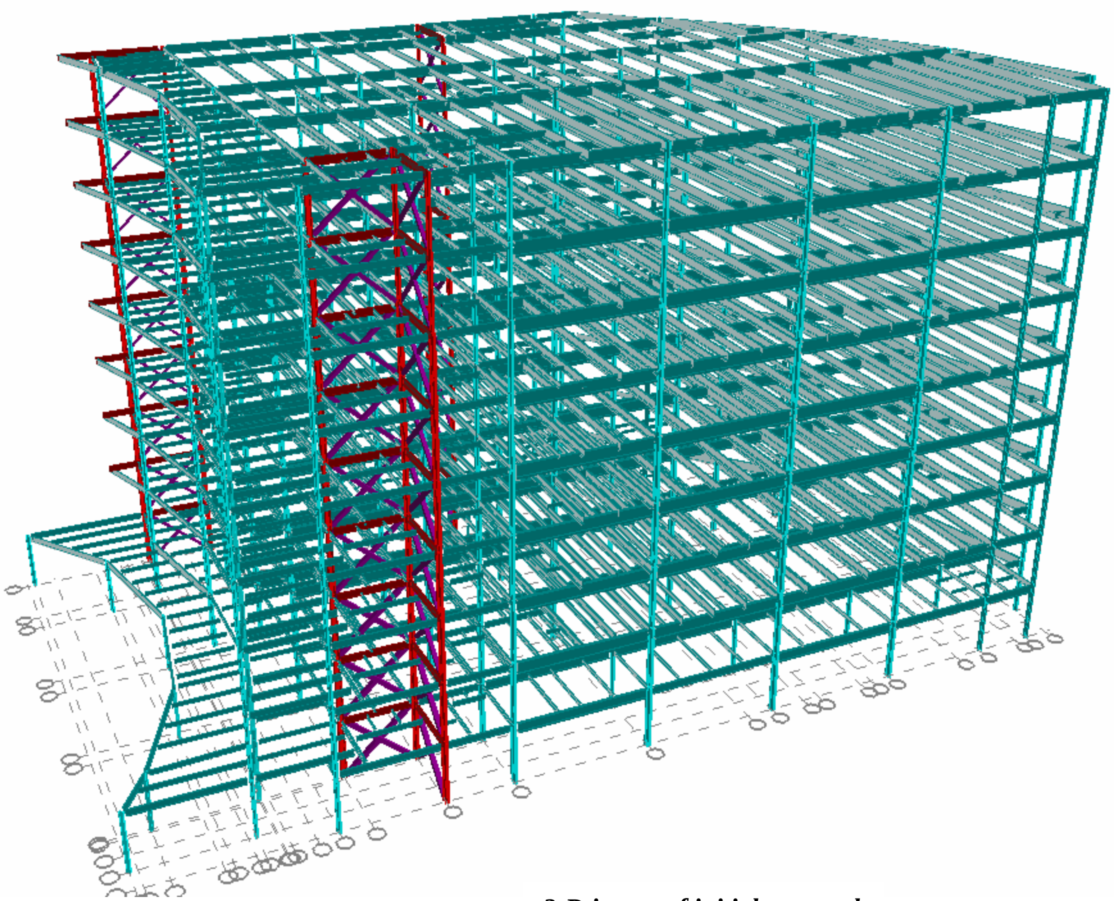
Furthermore, a list of design goals was implemented to ensure that the lateral system was designed under the same conditions of the initial system. The design goals are as follows:

Design Goals

- Design an efficient lateral system while keeping braces, columns, and beam sizes as light as possible
- Maintain W12 column sizes throughout design
- Reduce drift to L/480 design criteria in both directions under all load cases
- Minimize impact on interior spaces, floor plan layouts, and the exterior façade
- Create lateral column splices on the same levels as the gravity column splices
- When possible, keep connections as "pinned" to avoid excessive material and labor intensive installations
- Avoid altering beam sizes between floors to maintain repetitive floor framing

PROPOSED DESIGN 1

The initial lateral redesign of Memorial Sloan-Kettering simply involved extending each of the braced frames to the addition's roof. With this proposal, only the sizes of the columns and braces would need to be altered in order to determine whether or not this system would work. In addition, this attempt would comply with the proposed limiting factors. There would be no need to alter the current structural layout and each system would still make use of concentrically braced frames.



3-D image of initial proposal (red signifies lateral bracing)

After constructing Memorial Sloan Kettering in RAM and inputting all the necessary load parameters, this system was analyzed and showed the expected; that initial sizes of the columns and braces were under designed. Once the braced frames were resized to withstand the axial loads placed on them, it was obvious that this proposed design would not be the most effective. 2nd Floor columns ranged in size from W12x170 all the way up to W12x336. It also became apparent from this analysis that west side of MSK was significantly stiffer then its east side. This was due to the fact that because an open floor plan was developed when the infrastructure was four stories tall, the building only incorporated braced frames into its east half. However, once an additional five stories were erected onto MSK, the braced frame designed to resist the right half's forces was no longer adequate. To add to everything, further problems surfaced once building drift was investigated. In order to clarify further drift discussion, below is a list of wind loads RAM takes into account during its analysis.

RAM Load Cases			
Notation	**Lateral Load**	**Description**	**Load Case**
W_1	Wind	X Direction	Case 1
W_2	Wind	Y Direction	Case 1
W_3	Wind	X + Eccentricity	Case 2
W_4	Wind	X - Eccentricity	Case 2
W_5	Wind	Y + Eccentricity	Case 2
W_6	Wind	Y - Eccentricity	Case 2
W_7	Wind	X + Y Directions	Case 3
W_8	Wind	X - Y Directions	Case 3
W_9	Wind	Clockwise Moment	Case 4
W_{10}	Wind	Counterclockwise	Case 4
O_1	Seismic	East-West Direction	

When building drift was investigated, it was clear that additional lateral resisting frames would be required throughout Memorial Sloan Kettering. At the center of rigidity, the building drifts 7.11" in the east-west direction due to seismic (0_1) and 5.91" in the north-south direction due to wind in the y-direction (W_2). Worse yet is the fact that MSK drifts 14.71" at its east exterior wall due to W_2 in the north-south direction. This deflection is attributed to lack of stiffness on the east side of the building.

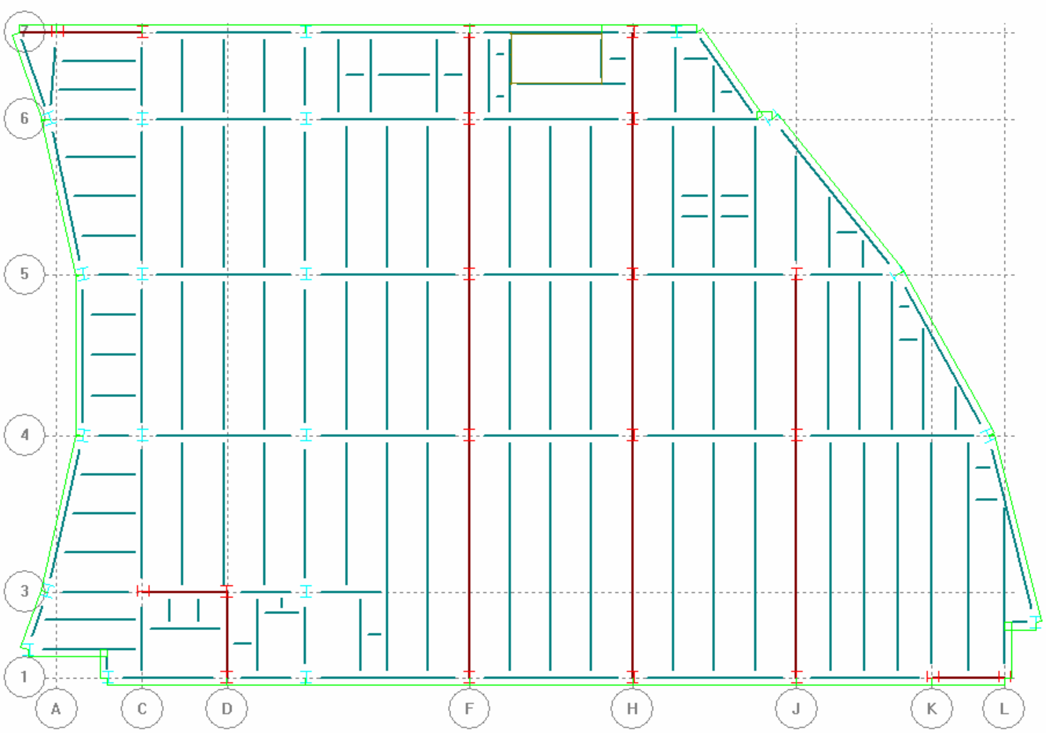
From the initial lateral system design of MSK, it is obvious that a number of additional steps must be taken in order to deem this building structurally sound. For one, both the braces and columns are excessively large for a nine story building. This is due to the fact that there are only two braced frames resisting lateral loads in each direction. To counteract this problem, more lateral bracing will have to be added throughout the building.

Another large problem with this initial design is the lack of stiffness provided from the east side of Memorial Sloan Kettering. As previously noted, this problem is attributed to the actuality that only the west half of the building accommodates braced frames. This uneven distribution is a direct result from the open floor layout desired in the upper stories of the existing structure. Unfortunately, now that the lateral force has more than doubled in the north-south direction, braced frames are required. Because one of the design goals for this redesign is to minimize the impact on interior spaces and floor plans, it will be challenging to find a way to brace that side.

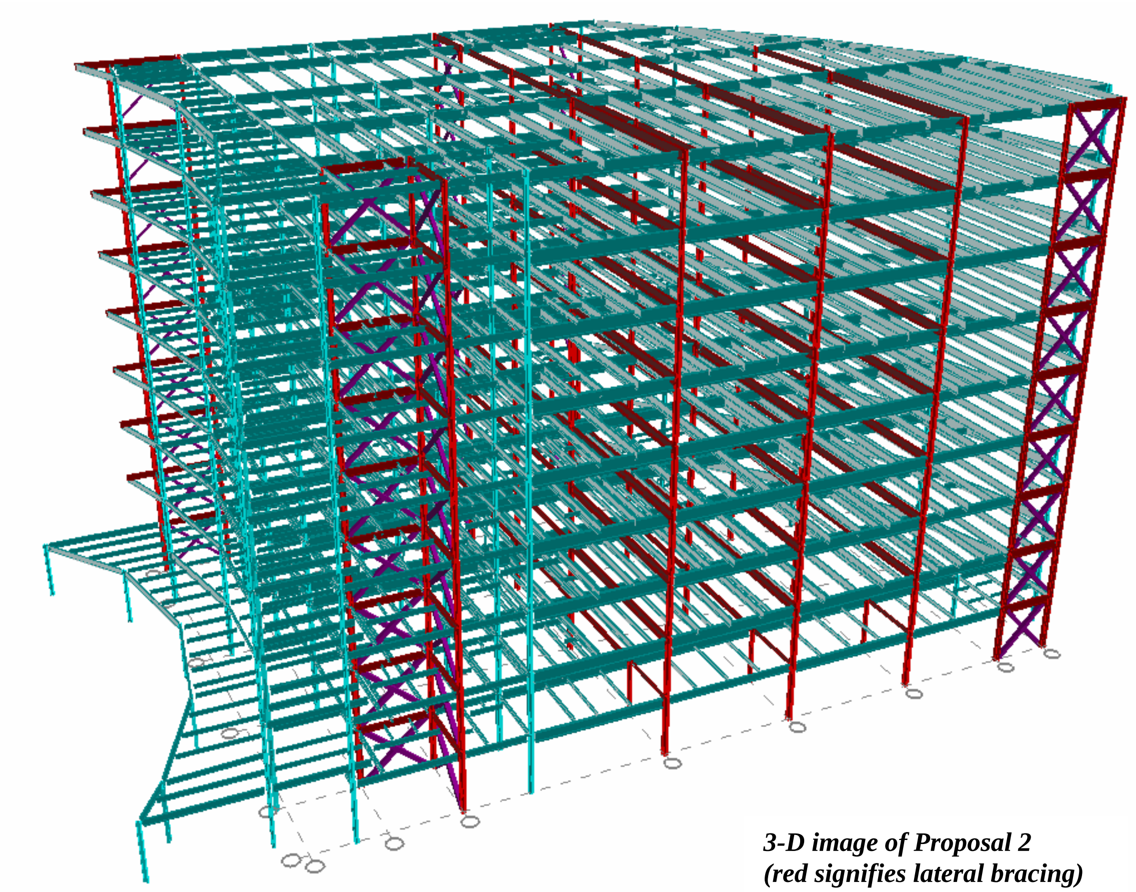
PROPOSED DESIGN 2

This second proposal addresses all the problems found from the first analysis and offers a more refined design. One problem the first design faced was excessively large braces and columns. This issue was addressed by adding a braced frame in each direction. In the north-south direction, a diagonal braced frame was positioned perpendicular to the north wall between column lines 7 and 6. This location is optimal for a braced frame because it is located near a stairwell, minimizing interior space interference, and is as far east as possible under the existing floor plan. In the east-west direction, a diagonal braced frame was placed along the south wall between column lines K and L. This location was chosen because any other bay on that exterior wall would cause interference with window placement.

The next issue addressed with this proposal was the lack of stiffness encountered with the building's east side. This problem was already partially attended to with the addition of the north-south braced frame. However, to further stiffen this area, moment frames were added between column lines 6 through 1 on column lines F, H, and J. Although one of the design goals implemented dealt with keeping connections pinned if possible, this exception had to be made. The reason being that because of the open floor plan, there was no location to place a braced frame. Because of this, and the need to add stiffness to the area, moment frames were the next best alternative. They maintain an open layout while resisting the lateral force acting on the infrastructure.



Structural Floor Plan of Proposal 2
(red signifies lateral system)



3-D image of Proposal 2
(red signifies lateral bracing)

After analyzing this lateral system in RAM, it became apparent that while the additional lateral frames helped reduce the amount of shear force on each frame, Memorial Sloan-Kettering would still need additional bracing to meet its design goals. The combination of cross-bracing and moment frames was able to reduce the column sizes down to more suitable sizes. 2^{nd} floor columns ranged in size between W12x136 through W12x210. The drawback to this format, however, was that the moment frames also needed to be adjusted in size. Previously sized as gravity frames, adding moment connections demanded that larger beams and columns be incorporated in the design. Beams W24x55 and W16x26 were resized to become W24x107 and W24x68, respectively. 2^{nd} floor columns within these moment frames also reached sizes of W12x210. So although the additional moment frames reduced the column sizes in the existing braced frames, it counteracted those reductions though increasing their own column sizes.

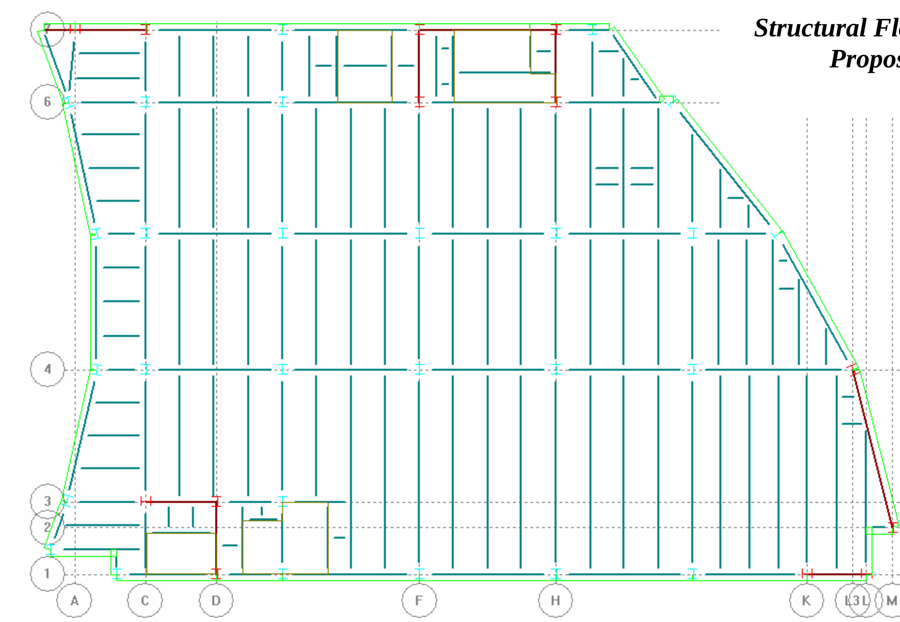
When drift was examined for this proposal, it was still obvious that additional resisting systems were required to subdue the building's displacement below the drift limit. At the center of rigidity, MSK still drifted 5.82" in the east-west direction due to seismic and 4.78" in the north-south because of W_2. Although each of these directions saw a loss in

drift, it still was nowhere near enough to be considered acceptable. Once again, the east exterior wall displayed the largest displacements with over 7.8" of drift in the north-south direction. The additional stiffness provided by the moment frames did reduce this drift in half, however it simply wasn't enough.

Proposal 2 was a step in the right direction, however more extreme measures are needed before this lateral system design can be finalized. Adding braced frames in both directions were effective in lowering all column sizes into an acceptable range. They also helped reduce drift. Adding moment frames on the east side of Memorial Sloan Kettering reduced the drift on the east exterior wall by a half. The downside of this design is that those bays dramatically increased member sizes in order to resist those loads. As a whole, Proposal 2 was effective in the fact that it displayed what concepts brought about a more efficient design. The final step was to utilize of those concepts to design a satisfactory lateral system. The right ideas were implemented with this proposal, they just need to be further exploited to generate an efficient design.

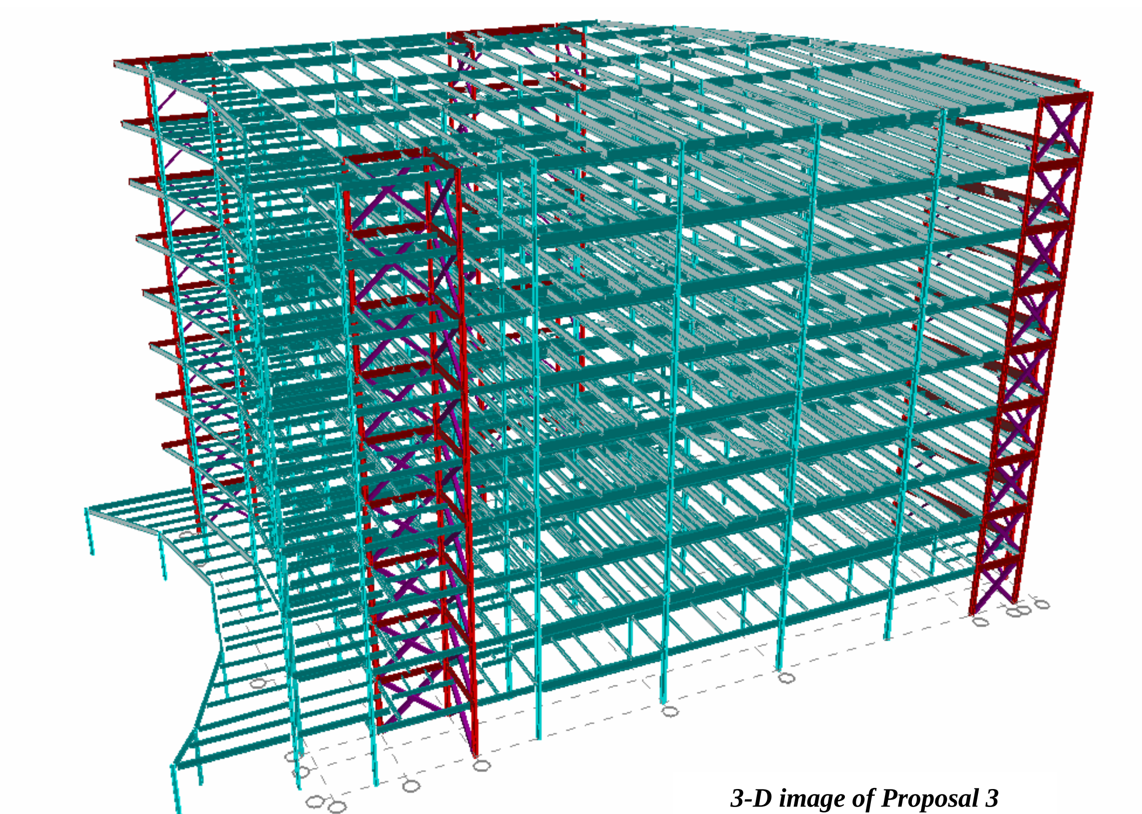
PROPOSED DESIGN 3

This third proposal is based off of the concepts implemented in Proposal 2, only to a larger degree. Braced frames were added in Proposal 2 which greatly helped regulate column sizes and drift. Therefore, additional braced frames will be added into the lateral system of Memorial Sloan Kettering. Moment frames were also introduced in order to provide stiffness in the east wing of the building. Unfortunately, this action provided just as many inconveniences as it did usefulness. Column sizes needed to be enlarged and once the moment frames were designed, they provided very little stiffness compared to the braced frames. There is no doubt that a lateral system must be provided on that east side, but moment frames are not the answer to this problem. Below is the structural floor plan for Proposal 3.



Structural Floor Plan of Proposal 3

Proposal 3 has been designed entirely with braced frames given that they provide the stiffness necessary to significantly reduce building drift. One difference, evident from the image on the previous page, is that all moment frames integrated into Proposal 2 have been removed. This is due to the fact that these frames provided as many drawbacks as advantages in the system. In addition to the braced frames from Proposal 2, one braced frame was incorporated in each direction for this design. In the east-west direction, a braced frame was added between column lines F and H. Chevron bracing was used for this bay instead of X-bracing due to the larger span it possessed. The north-south brace was positioned between column lines 2 and 4 along the east wall. The reason for doing so was that there was simply no other feasible place to brace. The southeast corner of MSK is very spacious with bays spanning 45' from column to column. Placing a braced frame into one of these bays would dissect the floor and require a new layout. Because the proposed braced frame would affect the east wall's façade, an eccentric knee brace was developed to minimize interference with window positioning. This brace allows the façade to continue in normal fashion while providing enough bracing to significantly reduce drift in that area. A 3-D image of the design is provided below.



3-D image of Proposal 3
(red signifies lateral bracing)

After running this model through RAM frame, it was established that this particular proposal would in fact provide an effective lateral system for Memorial Sloan Kettering. Because there are now four braced frames in both the north-south and east-west directions, 2^{nd} floor columns sizes range between W12x136 and W12x210. Brace sizes ranged from HSS 6x6x½ to HSS 12x12x½ and can be referenced in Appendix B. In addition, drift was significantly reduced in both directions. At the center of rigidity, Memorial Sloan-Kettering only drifts 2.66" in the north-south direction, due to W_2, and 2.77" in the east-west direction, due to seismic. Furthermore, the frame positioned on the east exterior wall was successful in reducing drift down to 2.38" well below the design limit of H/480. Below is a chart showing displacement at the center of rigidity by each load case.

RAM Load Cases				Displacement at COR	
Notation	Lateral Load	Description	Load Case	X (inches)	Y (inches)
W_1	Wind	X Direction	Case 1	1.642	0.184
W_2	Wind	Y Direction	Case 1	0.2994	2.659
W_3	Wind	X + Eccentricity	Case 2	1.398	0.169
W_4	Wind	X - Eccentricity	Case 2	1.4756	0.154
W_5	Wind	Y + Eccentricity	Case 2	0.3629	2.306
W_6	Wind	Y - Eccentricity	Case 2	0.1611	2.347
W_7	Wind	X + Y Directions	Case 3	1.4561	2.13
W_8	Wind	X - Y Directions	Case 3	1.007	-1.856
W_9	Wind	Clockwise Moment	Case 4	1.169	1.887
W_{10}	Wind	Counterclockwise	Case 4	1.378	1.845
O_1	Seismic	East-West Direction		2.77	0.32

Comparing this final design with the limiting factors and design goals, it appears that all criteria were essentially met. The final lateral system locations do not impede at all with the existing floor plan layout. Although minimal interference results from systems located along exterior walls, those braces were configured to allow a normal façade layout. To go along with the last comment, concentrically braced frames were used except when they interfered with the existing architectural design. W12 columns were maintained for all frames, and moment connections were avoided in the final design. Finally, drift was reduced below the design criteria of H/480, or 3.15 inches.

FOUNDATION REDESIGN

The final task of the structural redesign for Memorial Sloan Kettering had to do with analyzing and resizing the foundation members. Because five stories have been added onto the existing structure, both the concrete piers and shear walls have additional forces acting on them. In addition, the footings beneath these components were proven to be under sized.

Concrete Piers

The concrete piers supporting each of the steel gravity columns were the first to be looked at for this foundation analysis. As described previously in this report, these columns are 24" by 24" in dimension and spaced 30' apart. They have originally been sized to support the weight of the four stories above them, but with the addition of five stories, they needed to be resized.

Before starting calculations, there were a few assumptions made to simplify the design. First, the columns were required to remain at 24"x 24" in dimension. To do so, the compression strength of the concrete was increased from 4 ksi to 5 ksi. The second assumption made for this design was that these columns only resisted axial loads from the structure above and moments only from the tributary area surrounding it. The axial loads acting on each column were calculated in excel and can be found in Appendix B. To find the value of this bending moment, a worst case scenario was developed having live load throughout the bay on the left and no live load on the bay to the right From this alternative bay loading, it was determined that the worst case fixed end moment acting on the column was 468 ft-kips. It was also assumed that the concrete pier took 100% of the moment. Reference Appendix B for these calculations.

After obtaining the axial and moment values, it was possible to use the Design Aid Interaction Diagram to estimate the needed steel reinforcement. This only provided an approximate amount of concrete since the columns now used 5 ksi concrete and the design aid used 4 ksi. Once steel values were found and an appropriate bar configuration was developed, the section was checked by determining ΦPn_{max} for the column. In addition, the CRSI Handbook was referenced as one additional check to confirm the columns were not under designed.

The chart on the following page lays out the final configurations for the concrete piers. The pier designs were attempted to stay as similar as possible in order to simplify the construction process.

Concrete Pier Reinforcement Design							
Hand Calculations							
	Column Location	Applied Loads		RAM	Steel	Checks	
		Axial	Moment	Rq'd Amt.	Configuation	Pnmax	Adequate?
Design A	Typical Interior Column	1664.54	468	13.82	(12) #10 bars	1714	YES
Design B	Atypical Interior Column (South Side)	1987.76	468	20.32	(16) # 11 bars	1996	YES
Design C	Typical Exterior Column (South Side)	1420.23	468	11.52	(12) #10 bars	1714	YES
Design D	Typical Exterior Column (North Side)	805.96	468	11.52	(12) #10 bars	1714	YES
Design E	Atypical Interior Column (North Side)	1280.56	468	11.52	(12) #10 bars	1714	YES

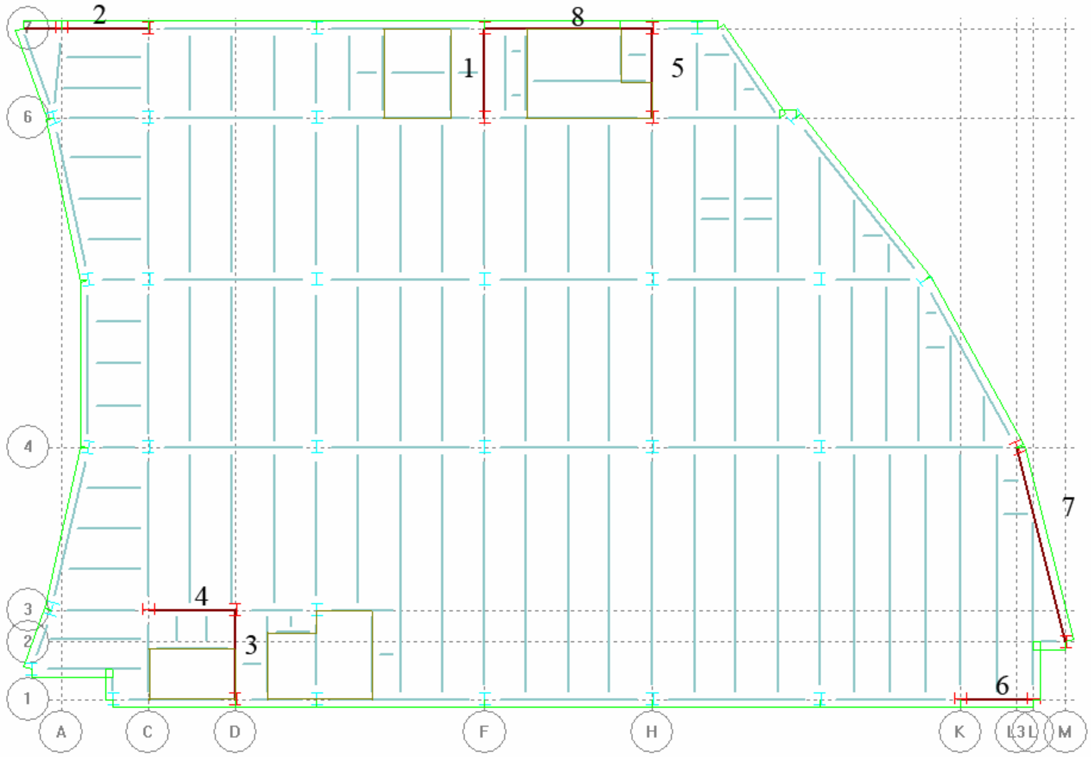
Once the concrete piers were redesigned to withstand the axial forces acting on them, their footings were inspected to see whether they had to be increased in dimension. The geotechnical report stated that the existing structure of Memorial Sloan Kettering was on basalt bedrock and had an allowable bearing capacity of 20 kips per square foot. This bearing capacity was the controlling factor for this analysis. Below is a chart comparing the new required footing dimensions to the old dimensions.

Allowable Bearing Stress for Fo **20 ksf**

Resizing of Pier Footings					
from Hand Calcuations					
	Column Location	Footing Loads		New Footing	Old Footing
		Force (kips)	Sq. Ft.	Redesigned Size	Previous Size
Design A	Typical Interior Column	1664.54	83.22679	9' x 10'	6' x 6'
Design B	Atypical Interior Column (South Side)	1987.76	99.38802	10' x 10'	6' x 6'
Design C	Typical Exterior Column (South Side)	1420.23	71.01127	8' x 10'	6' x 6'
Design D	Typical Exterior Column (North Side)	805.96	40.29798	7' x 7'	6' x 6'
Design E	Atypical Interior Column (North Side)	1280.56	64.0278	8' x 8'	6' x 6'

Shear Walls

The final task in analyzing Memorial Sloan-Kettering's foundation was to determine whether or not more reinforcement needed to be added to the shear walls. The lateral forces acting on this infrastructure had dramatically increased due to the Outpatient Addition. Wind loads had increased from 226 to 647 kips and seismic increased to 442 kips. There are, however, more shear walls in each direction due to the redesigned system, all with similar stiffnesses. Because of this, the lateral loads should distribute somewhat evenly between the shear walls in each direction. The following page details the locations of each shear wall along with the maximum shear force and overturning moment it experiences. From those results, the shear wall with the highest forces will be analyzed to determine whether or not the shear wall design should be adjusted. A diagram providing the forces experienced from each load case can be found in Appendix B.



Shear Wall Locations

Shear Walls			
North - South Direction			
Location	**relative stiffness**	**Max shear force**	**Overturning Moment**
1	0.330	213.62	14751
3	0.316	204.49	14120
5	0.187	120.84	8344
7	0.168	108.87	7518
	1.000	647.82	44733
East - West Direction			
Location	**relative stiffness**	**Max shear force**	**Overturning Moment**
2	0.225	98.2	8816
4	0.313	136.4	12246
6	0.235	102.61	9212
8	0.227	98.88	8877
	1.000	436.09	39151

Once the controlling shear forces and overturning moments were found for each wall, it was necessary to check whether or not the current reinforcement configuration would work. The existing layout calls for #5 bars spaced 12" on center for both faces of the wall. Frame 4 was chosen to analyze because it had a relatively high amount of shear on its shorter wall. From analysis, it was determined that this configuration could resist up to 604 kips of shear force, far more then that acting on the wall (see Appendix B for calculations). Because there are no shear walls in Memorial Sloan Kettering that see anywhere near 600 kips of shear force, it can be assumed that this reinforcement layout is adequate for all of the shear walls.

Overturning moments were then investigated to determine whether the footings beneath each shear wall would need to be increased in size. Once again, Frame 4 was chosen due to the large amount of moment on its relatively short shear wall. The 12,245 foot-kips created 875 kip couple acting vertically on the wall. To try to counteract this couple, the cumulative axial force acting on the shear wall was 683 kips. Because the couple is only partially resisted from this weight, it was necessary to look at the weight of the footing. The current footing dimensions under this shear wall was 8' x 30' x 48", adding 144 kips of resistance to the couple. Unfortunately, this additional weight does not counteract the couple, and the footing needed to be resized. After increasing the dimensions to 12'x 30'x 48", the couple was sufficiently resisted. This calculations are referenced Appendix B.

When looking at the other walls to determine whether they would have the same problem, it was determined that they would in fact be able to resist their overturning moments. Frames 1, 8, and 5 are all supported by the same MAT foundation, whose weight alone is almost enough to resist the couples acting on those walls. Walls 2, 3, and 6 all have the additional weight of the building façade to counteract against their couples. Wall 7 is significantly longer then any of the other shear walls, and that length reduces the size of the couple acting on the wall.

BREATH STUDIES

CONSTRUCTION MANAGEMENT STUDIES

INTRODUCTION

Just because Memorial Sloan-Kettering has been redesigned to support the Outpatient Addition does not imply that this alternative is a logical choice. In order to determine how efficient the structure actually is, it must be analyzed from both a cost and time perspective. Even though MSK has been designed to withstand the gravity and lateral loads acting on its structure, if the building is unreasonably expensive or impractical to erect, then it simply cannot be considered as an option. This construction management study was performed with the goal of determining how expensive the structure of this addition would be compared to if it were built on the north side, as planned. In addition, a structural schedule was created to establish the time it would take to erect the five additional floors. This can be referenced in Appendix C. From these two variables, a much better conclusion was developed to whether or not this alternative design was feasible.

STRUCTURAL COST ANALYSIS

The first step in this study was to analyze Memorial Sloan Kettering's addition from a cost perspective. This task, however, proved to be more complex then initially anticipated. This was due to the fact that when designing the addition's structure system, it was also necessary to redesign the existing four stories beneath it. Those lower stories experienced a large increase in load acting on them and needed to be bulked up in member sizes. Because this action would not be necessary if the addition were placed on the north side of the existing structure, it was decided that this variable should be included in the overall addition price.

Another setback in performing this cost analysis was that there were no prices to compare the findings to. This addition is still in its design phase and because of that, there aren't any figures addressing its overall cost. All of these adaptations and setbacks made it necessary to create assumptions addressing these concerns. The assumptions made for this cost analysis are as follows:

1) The "structural cost" for this analysis will include structural steel and concrete. This includes materials, placement, labor, and formwork. See the following pages and Appendix C for a more detailed summary.

2) The total cost of this Outpatient Addition will include both the structural cost of the five additional stories <u>AND</u> the increased cost created by increasing member sizes on the first four floors.

3) Because the Outpatient Addition is almost identical to the existing structure, it is assumed that if built adjacent to the first four floors, it would cost virtually the same amount as the existing structure did. This allows for a tangible cost comparison rather then a hypothetical one.

COST ANALYSIS RESULTS

To determine the cost of only the Outpatient Addition, it was necessary to find the cost of the entire nine stories and then subtract out the existing values of the first four. That way, the value remaining would include the five additional stories and any extra cost brought about by the increased member sizes. By referencing a Financial Status Report provided by BARR & BARR BUILDERS, cost values were established for the four existing floors of Memorial Sloan Kettering. These values are shown in the chart below:

Phase One Price	
Structural Components	**Price**
Structural Steel	$1,839,199
Concrete on metal decking	$375,000
Total	**$2,214,199**

This chart takes a number of components into consideration for both of those groupings. For instance, the structural steel above includes: gravity columns, gravity beams, frame columns, frame beams, frame braces, shear studs, metal decking. Likewise, the concrete on metal decking includes: concrete slab, welded wire fabric, concrete slab edge formwork. In order to compare costs efficiently, take-offs of all these components were required.

To help accomplish this task, RAM Structural System was used to obtain take-offs for the steel members and shear studs. Metal decking quantities were determined simply by finding the floor area of each floor. The concrete component values were also conceived in a similar way, only with minor alterations. A 7% increase was added to the amount of concrete required due to spillage and shrinkage. Likewise, a 10% increase was calculated into to amount of welded wire fabric needed to account for overlapping. The required formwork for the slab edges was found using the perimeter length for each floor.

Once the take-offs were finished for all of Memorial Sloan Kettering, the only task left to do was find the overall cost. The 2006 R.S. Means was used for this process to calculate all cost values. For each price estimate, the material, labor, and equipment were all taken into account. An overhead and profit adjustment was also added into the price since these values were being compared to contract values. The following page provides a chart summarizing the structural component costs. Also, a full cost breakdown of each component by floor can be referenced in Appendix C.

Total Addition Price (Structural Steel)	
Structural Components	**Price**
Gravity Columns	$338,482
Frame Columns	$206,616
Frame Beams	$96,858
Frame Braces	$171,988
Gravity Beams	$1,905,107
Shear Studs	$46,259
Metal Decking	$1,195,026
Total	$3,960,335
Phase One Cost	$1,839,199
Addition Cost	**$2,121,136**

Total Addition Price (Structural Concrete)	
Stuctural Components	**Price**
Slab Edge Formwork	$112,680.00
Welded Wire Fabric	$58,784.83
Concrete Slab	$302,474.65
Total	**$473,939.47**

Total Addition Price	
Stuctural Components	**Price**
Structural Steel	$2,121,136
Structural Concrete	$473,939
Phase 2 Total	**$2,595,076**
Phase1 Total	**$2,214,199**
Difference	**$380,877**

From the results of this cost analysis, it has been determined that Phase Two would be more expensive to erect vertically above the existing building then if it were being build adjacent to MSK. After a further look at the breakdown of each component, these values make a lot sense. Comparing the structural steel values, Phase 2 would cost approximately $282,000 more by building the addition vertically. This is due to the fact that a vertical addition requires an additional five stories of structural steel compared to the four needed if it were built next to the building. Also, this cost includes the additional material needed by resizing the existing four stories.

When comparing the concrete values, Phase 2 costs approximately $100,000 dollars more by being built vertically. Once again, this has to do with the fact that an additional story would need to be created in order to get the addition's allotted amount of space. In terms of floor by floor cost however, the prices would be almost exact if the Outpatient Addition only required four additional floors.

ADDITION SCHEDULE

The other consideration from a construction management point of view would be the difference in schedule time between the two options. From a financial standpoint, time is money, and the more quickly the addition can be completed and put into use, the more useful it will be. Once again a number of assumptions had to be made to complete this comparison. Only the structural components of each option would be considered, and since Phase 2 is still being designed, the schedule time for Phase 1 would be used for comparison.

To determine the schedule time for Phase 2, both R.S. Means and Microsoft Project were used. R.S. Means provided a daily output value to determine how many units of a certain item could be constructed in a day. The takeoff numbers for each material were divided

by the daily output values which in turn determined the number of days required for construction. Below is a table showing the time breakdown for the erection of the existing structural system.

PHASE ONE	
Concrete	
Slab on Metal Deck (2nd Floor)	8 days
Slab on Metal Deck (3rd Floor)	8 days
Slab on Metal Deck (4th Floor)	8 days
Slab on Metal Deck (Roof)	8 days
	32 days
Structural Steel	
Steel Erection	45 days
Install Metal Deck	15 days
	60 days

Following the procedure explained on the previous page, time schedules were developed for each component of the structural system for Phase Two. In order to create an authentic time frame, labor crews were doubled for concrete installations in order to make working schedules more realistic. Crews erecting the steel structure remained the same. Below is a chart summarizing the time frames required for erecting the Outpatient Addition. An entire schedule breaking down each task can be referenced in Appendix C.

ADDITION	
Concrete	
Placing Slab Reinforcement	
6th Floor	4 days
7th Floor	4 days
8th Floor	4 days
9th Floor	4 days
Roof	4 days
	20 days
Placing Slab Edge	
6th Floor	4 days
7th Floor	4 days
8th Floor	4 days
9th Floor	4 days
Roof	4 days
	20 days
Pouring Slab on Metal Deck	
6th Floor	4 days
7th Floor	4 days
8th Floor	4 days
9th Floor	4 days
Roof	4 days
	20 days
Concrete Total	**60 days**
Structural Steel	
Steel Column Erection	
6th - 8th Floor	6 days
9th - Roof	4 days
	10 days
Steel Floor Frame Erection	
6th Floor	4 days
7th Floor	4 days
8th Floor	4 days
9th Floor	4 days
Roof	4 days
	20 days
Install Metal Deck	
6th Floor	5 days
7th Floor	5 days
8th Floor	5 days
9th Floor	5 days
Roof	5 days
	25 days
Install Shear Studs	
6th Floor	3 days
7th Floor	3 days
8th Floor	3 days
9th Floor	3 days
Roof	3 days
	15 days
Structural Steel Total	**70 days**

When the existing schedule and estimated addition schedule were compared, it was once again obvious that the addition took more time to erect. There are a number of reasons to justify the increased length. As noted in the cost analysis section, this addition possesses an additional story that needs to be erected. This explains the increase in schedule time for both the steel and the concrete. Another justification for the increase in time is that there are now more braced frames throughout the building. This difference will require additional labor hours to erect the braces into place. The final justification in the noticeable time difference is that it is more time consuming to place steel and concrete floor elevations increase. All these reasons directly result in an increase in time.

CONSTRUCTION MANAGEMENT CONCLUSION

After performing both a cost analysis and time schedule for the vertical addition, it was determined that this option was not as efficient as the original position from a cost perspective. The analysis concluded that due to an additional floor and increased member sizes, both the steel and concrete prices would increase by building vertically. Overall, erecting the Outpatient Addition on top of the existing structure would cost approximately 17% more then if it were kept where it was originally proposed to be built.

Comparing both schedules on a time perspective also displayed negative aspects for the vertical expansion of this addition. By adding those stories, the scheduled time of erection for the structural system alone increased by over 40%. This does not even consider the amount of downtime Memorial Sloan Kettering would experience from this construction as well.

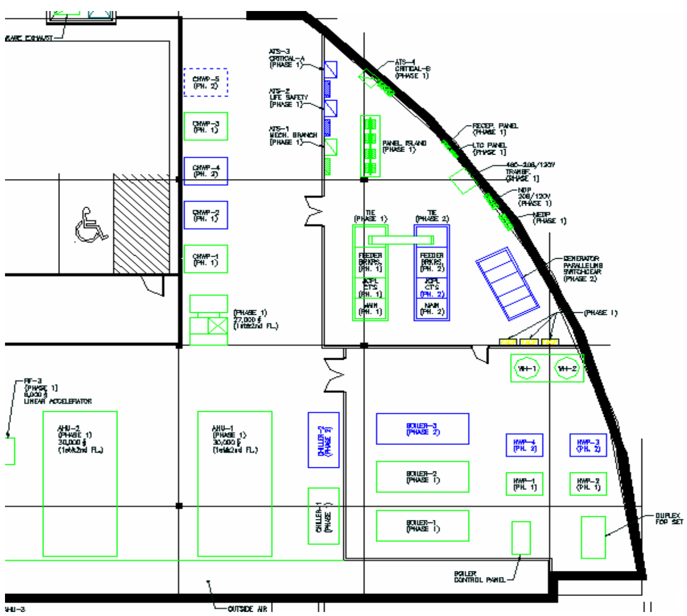
In conclusion, this construction management study proved that changing the site plan for the Outpatient Addition would prove to be an expensive choice, from both a cost and time perspective. An additional $381,000 would have to be spent on the structural system. Furthermore, it would require an extra seven weeks to construct. Memorial Sloan-Kettering would have to close for at least some of this process, creating another negative feature this proposal would create. Simply from the results of this breath study, it would be suggested that Memorial Sloan-Kettering continue with the original design of placing the addition to the north side of the existing structure.

MECHANICAL & ACOUSTIC STUDY

Erecting an addition on top of an existing, operational facility requires more then just a structural redesign. Every system within that building needs to be resized or repositioned in order to support that new area. This study focuses on the MEP system within Memorial Sloan-Kettering and more specifically the Air-Handling Units located on the roof of the existing structure. To avoid disrupting air flow in MSK and having to reposition a large amount of equipment, the 5th floor of the addition will maintain the Air-Handling Units and become a mechanical floor A layout will be formed to position all of the additional required mechanical equipment. Also, an acoustics study will be performed between the mechanical floor and adjacent floors in order to determine whether or not additional soundproofing will be required. In all, this study hopes to prove whether or not this addition is feasible from a mechanical perspective.

MECHANICAL STUDY

Now that the structural design of the Outpatient Addition is complete, its necessary to look at how that space will be provided with the essential mechanical equipment. The current mechanical room for the existing infrastructure is located in the basement. Three additional air-handling units are also located on the roof and provide air circulation for the 3rd and 4th floors. When laying out the mechanical floor plan for the existing structure, additional room was left for MEP equipment supplying Phase 2. This situation worked out perfectly for erecting the addition vertically because now the new equipment was able to be placed in the basement and only the air-handling units needed to be positioned elsewhere in the building. Below is the mechanical layout provided for both Phase 1 and Phase 2 of Memorial Sloan-Kettering. Phase 2 equipment is shown in dark blue.



Mechanical Layout

Instead of moving the three air-handling units to a different location, it proved to be simpler to leave them alone and instead make the 5th floor a mechanical floor. After all, each unit is approximately 27' x 10' in dimension and weighs almost 7 kips. Two of the units provide air to the 4th floor, which acts as a surgical floor, while the other circulates the 3rd floor. In addition to the three existing systems, two more air-handling units were placed on this floor to supply the 6th and 7th floors. The 8th and 9th floors would have air supplied to them by units on the addition's roof.

In order to get outdoor air to the equipment on the mechanical floor, louvers needed to be installed on each exterior wall. To determine a proper dimension for each louver, it was necessary to find the required amount of fresh air needed for each unit. ASHRAE Standard 62.1 outlines proper ventilation for acceptable indoor air quality and proved to be the right place to look. Table E-1, shown below, gives outdoor air requirements for ventilation of healthcare facilities.

TABLE E-1*

Outdoor Air Requirements for Ventilation of Health Care Facilities (Hospitals, Nursing and Convalescent Homes)

Application	Estimated Maximum** Occupancy P/1000 ft^2 or 100 m^2	Outdoor Air Requirements cfm/ person	L/s · person	cfm/ft^2	$L/s \cdot m^2$	Comments
Patient rooms	10	25	13			Special requirements or codes and pressure relationships may determine minimum ventilation rates and filter efficiency. Procedures generating contaminants may require higher rates.
Medical procedure	20	15	8			
Operating rooms	20	30	15			
Recovery and ICU	20	15	8			
Autopsy rooms	20			0.50	2.50	Air shall not be recirculated into other spaces.
Physical therapy	20	15	8			

* Table E-1 prescribes supply rates of acceptable outdoor air required for acceptable indoor air quality. These values have been chosen to dilute human bioeffluents and other contaminants with an adequate margin of safety and to account for health variations among people and varied activity levels.

** Net occupiable space.

From the chart above, the 4th floor fell under "operating rooms" application while the 3rd, 6th, and 7th floors were all "medical procedure" areas. Manipulating those values gave the required amount of cubic feet per minute necessary for the entire floor. From that, it was necessary to find the average wind velocity acting in that area. For this piece of data, a RETScreen Energy Model, shown in Appendix C, was referenced for the New York City area. It was found that an average wind velocity would be somewhere around 4.9 mph, which converts to around 431.2 feet per minute. The calculations on the following page show how a louver size was determined.

Louver Calculations

Air Handling Units on 5th Floor		
Unit	**Handles**	**Dimension**
RAHU-1	3rd Floor	12' x 27'
RAHU-2	Ambulatory Surgery	12' x 27'
RAHU-3	Ambulatory Surgery	12' x 27'
RAHU-4	6th Floor	12' x 27'
RAHU-5	7th Floor	12' x 27'

TABLE E-1

ASHRAE Standard 62.1 (Ventilation for Acceptable Indoor Air Quality)		
Application	**Max Occupancy Density**	**Outdoor Air Requirement**
	#/1000 ft^2	**cmf/person**
Medical Procedure	20	15
Operating Rooms	20	30

Each Floor Area is Approximately **20,000 square feet**
Five Air Handling Units located on the 5th Floor (See Above)
- 3rd, 6th, and 7th Floors - Medical Procedure Floors
- 4th Floor - Operating Room (2 units)

Required CFM Calculations

Medical Procedure Floors

= (20 people/1000 ft^2)(20,000 ft^2) = 400 people
= (400 people)(15 cfm/person) = 6000 cfm
= (6000 cfm per floor)(3 floors) = **18,000 cfm**

Operating Room Floor

= (20 people/1000 ft^2)(20,000 ft^2) = 400 people
= (400 people)(30 cfm/person) = **12000 cfm**

Total Required cfm = 30,000 cfm

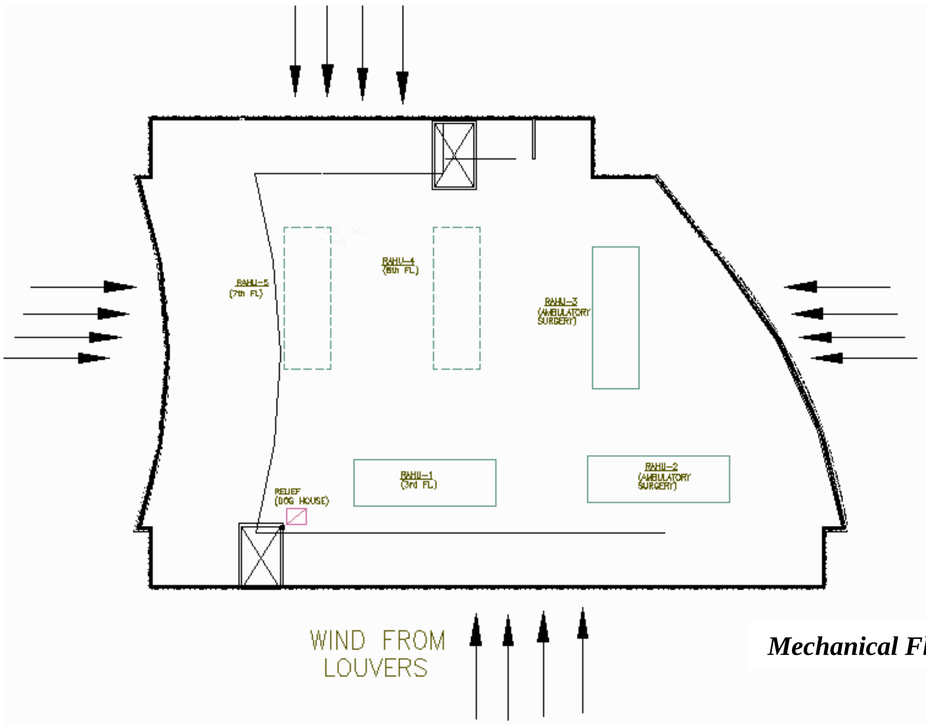
Convert Values to Area of Louver needed (ft^2)

Wind Velocity = 4.9 mph ----> convert to ft/min = **431.2 ft/min**
cfm/(ft/min) = ft^2 -----> gives area
= (30,000 cfm)/(431.2 ft/min) = **69.57 ft^2**
- Multiply Area by 1.43, assume that louver only provides 70% free area

(1.43)*(69.57 ft^2) = **100 ft^2** per wall

- Also take into account louver size needed for maintenance/ repair
- Increase louver size to 15' x 10' , therefore **150 ft^2** per wall

From these calculations, it was determined that the minimum louver size that would provide adequate air flow into the mechanical room would be 100 square feet on each wall. However, in order to make sure that an air-handling unit would be able to be repaired, each louver size was increased to 15' wide by 10' high. The reason for putting a louver of each wall is so air would flow into the space no matter which direction it's blowing. Also, this design would not allow excessive internal pressure to build up on the floor. Below is a layout of the mechanical floor. The arrows represent where wind can enter/exit from the louvers. The dashed air handling units represent those units that will supply floors on the addition.



Mechanical Floor Plan

ACOUSTIC STUDY

Once it was decided that the 5th floor of Memorial Sloan-Kettering was to become a mechanical floor, the question arose to whether or not acoustic issues would arise on the 6th and 4th floors. In terms of acoustics, different rooms have different acceptable noise levels. For a building like a healthcare facility, all floors should remain quiet enough to allow conversation while at the same time upholding privacy. Therefore, these floors should have a relatively low range of noise criteria. Noise criteria (NC) ranges provide acceptable background noise levels in order to achieve satisfactory sound isolation. The goal for this study was to determine whether these NC ranges were upheld even with the additional noise of the air-handling units.

The first task of this study was to determine the sound absorption coefficients provided from the building elements surrounding both the source and receiver areas. This helped determine how much noise would be absorbed and how much continued to the receiver areas. The mechanical room has a concrete floor and ceiling, which provide very little sound absorption. The louvers in this room, however, act as an open space and do not reflect any sound. Similarly, the materials in the office and operating room are all good sound absorbers. Once all these variables were taken into consideration and the source noise level was reduced, it was possible to determine what transmission loss value was necessary for the partition separating the source and the receiver. This transmission loss measures how much sound energy is reduced in transmission through materials. If that partition was adequate in reducing the sound into the required noise range, then no additional acoustical measures would need to be taken. Below are the calculations performed for both the operating room and private offices.

Required Transmition Loss for 4th Floor Operating Rooms

Sound Absorption coeffecents for source and reciever rooms								
Frequency	Mechanical Room				O.R.			Source
Hz	Walls (α)	Floor (α)	Ceiling (α)	Louver (α)	Walls (α)	Floor (α)	Ceiling (α)	Lw
125	0.36	0.01	0.01	1.00	0.55	0.02	0.76	89
250	0.44	0.01	0.01	1.00	0.14	0.03	0.93	88
500	0.31	0.02	0.02	1.00	0.08	0.03	0.83	89
1000	0.29	0.02	0.02	1.00	0.04	0.03	0.99	86
2000	0.39	0.02	0.02	1.00	0.05	0.03	0.99	82
4000	0.25	0.02	0.02	1.00	0.11	0.02	0.94	77

Frequency											
Hz	α sab (avg)	Sα	RTs	α sab (avg)	Sα	RTr	RC-25 Lp	Source Lp	NR	TL	Adj TL
125	0.0817	362.56	394.81	0.4614	77.52	143.94	40	69.04	29	24.1	29.13
250	0.0954	423.52	468.20	0.3282	55.14	82.08	35	67.30	32	29.8	34.83
500	0.0813	360.70	392.61	0.2738	45.99	63.33	30	69.06	39	37.7	42.72
1000	0.0778	345.46	374.62	0.3002	50.43	72.06	25	66.26	41	39.4	44.36
2000	0.0950	421.66	465.93	0.3046	51.18	73.60	20	61.32	41	39.3	44.32
4000	0.0710	314.98	339.04	0.3148	52.89	77.19	15	57.70	43	40.5	45.50

Source				Receiver			
A (walls)	A (floor)	A (ceiling)	A (louver)	A (walls)	A (floor)	A (ceiling)	A (partition)
762	1812	1812	52	75	46.5	46.5	46.5

Mechanical Room
Floor: Concrete
Ceiling: Concrete
Walls: Coarse Concrete Block

Operating Room
Floor: Linoleum
Ceiling: 3/4" thick acoustical board
Walls: Gypsum board

Transmission Loss from Partition			
(4.5" Reinforced Concrete Slab)			
Frequency	TL (dB)	Rq'd TL	Addition TL needed?
125 Hz	48	29.13	**NO**
250 Hz	42	34.83	**NO**
500 Hz	45	42.72	**NO**
1000 Hz	56	44.36	**NO**
2000 Hz	57	44.32	**NO**
4000 Hz	66	45.50	**NO**

Required Transmition Loss for 6th Floor Private Offices

Sound Absorption coeffiecents for source and reciever rooms								
Frequency	Mechanical Room				Private Offices			Source
Hz	Walls (α)	Floor (α)	Ceiling (α)	Louver (α)	Walls (α)	Floor (α)	Ceiling (α)	Lw
125	0.36	0.01	0.01	1.00	0.55	0.02	0.76	89
250	0.44	0.01	0.01	1.00	0.14	0.06	0.93	88
500	0.31	0.02	0.02	1.00	0.08	0.14	0.83	89
1000	0.29	0.02	0.02	1.00	0.04	0.37	0.99	86
2000	0.39	0.02	0.02	1.00	0.05	0.60	0.99	82
4000	0.25	0.02	0.02	1.00	0.11	0.65	0.94	77

Frequency											
Hz	α sab (avg)	Sα	RTs	α sab (avg)	Sα	RTr	RC-30 Lp	Source Lp	NR	TL	Adj TL
125	0.0817	362.56	394.81	0.4762	57.434	109.66	**45**	69.04	24	18.1	23.08
250	0.0954	423.52	468.20	0.3037	36.622	52.59	**40**	67.30	27	24.5	29.53
500	0.0813	360.70	392.61	0.2667	32.166	43.87	**35**	69.06	34	32.1	37.08
1000	0.0778	345.46	374.62	0.3351	40.408	60.77	**30**	66.26	36	32.9	37.87
2000	0.0950	421.66	465.93	0.3935	47.452	78.23	**25**	61.32	36	31.8	36.82
4000	0.0710	314.98	339.04	0.4258	51.352	89.43	**20**	57.70	38	32.6	37.62

Source				Receiver			
A (walls)	A (floor)	A (ceiling)	A (louver)	A (walls)	A (floor)	A (ceiling)	A (partition)
762	1812	1812	52	65	27.8	27.8	27.8

Mechanical Room
Floor: Concrete
Ceiling: Concrete
Walls: Coarse Concrete Block

Operating Room
Floor: Heavy Carpet
Ceiling: 3/4" thick acoustical board
Walls: Gypsum board

Transmission Loss from Partition			
(4.5" Reinforced Concrete Slab)			
Frequency	TL (dB)	Rq'd TL	Addition TL needed?
125 Hz	48	23.08	**NO**
250 Hz	42	29.53	**NO**
500 Hz	45	37.08	**NO**
1000 Hz	56	37.87	**NO**
2000 Hz	57	36.82	**NO**
4000 Hz	66	37.62	**NO**

From the calculations provided, it was concluded that although the mechanical room would provide additional noise, it was not necessary to provide addition sound absorption in either area. The private office passed acoustic inspection with plenty of decibels to spare under all frequencies. This has to do with the amount of sound absorption throughout the space and the fact that each office only has a small partition area between them and the mechanical room.

The operating room also fell within an adequate noise criteria, however it was a lot closer to being deemed unsatisfactory. This is because a lower noise criteria of 25 was chosen due to need to effectively communicate while in surgery. At a frequency of 500 hertz, the transmission loss was separated by only 2 decibels from its required value. Still, all frequencies passed and as a result, this acoustic study has shown that there was no need to provide additional soundproofing between the mechanical room and adjacent floors.

CONCLUSION

CONCLUSION

For this thesis, a study was performed in order to determine whether or not Memorial Sloan Kettering's Outpatient Addition would be both structurally and economically feasible if it were built vertically above the existing structure. The objective of this study was to design an efficient structural system that effectively resisted both the gravity and lateral loads it experienced. To do so, the existing structure needed to be reanalyzed under the increased loads it now experienced. At four stories, Memorial Sloan Kettering was controlled exclusively by seismic loading. Conversely, once the infrastructure rose to 126 feet, wind loads significantly increased and generated lateral forces exceeding 640 kips. In respect to axial loading, the existing structure now experiences the weight of an additional five stories.

RAM Structural Program was used to analyze the structure and help determine whether MSK displayed acceptable performance criteria under the necessary loading conditions. The lateral loads developed in this analysis came from procedures outlined in ASCE 7-02. In addition, the redesign of this infrastructure utilized a building drift limitation of H/480 to ensure serviceability issues were addressed. In order to meet this criteria, a number of plausible lateral system were investigated. The final design makes use of four braced frames in each direction, positioned to diminish drift throughout the entire structure. The foundation of Memorial Sloan-Kettering was also examined due to the increased loads on the structure. It was determined that while the lateral system remained efficient, the increased axial loads on the building required an increase in footing sizes.

In addition to designing an effective structural system, two breath studies were conducted to determine the practicability of a vertical expansion. A construction management study carried out both a cost analysis and time schedule of the proposed addition and compared those results with the initial plan. This comparison concluded that a vertical expansion would cost roughly 17% more and take 41% longer to build then if it were built in its original location. The second study examined the building's mechanical system and how it would supply the five additional stories. A layout was created of the mechanical room in the basement, showing locations of all required equipment. In addition, the 5th floor of Memorial Sloan Kettering was deemed a mechanical floor and now houses five air-handling units. To supply these units with outdoor air, louvers were designed to allow airflow through the floor. Finally, an acoustic study was performed to determine whether additional soundproofing was needed between the mechanical room and those floors above and below it. From the study, it was determined that noise would not be a problem.

After analyzing this building structurally, mechanically, and financially, it has been determined that yes, it is possible to design the Outpatient Addition this way. However, given the circumstances Memorial Sloan Kettering is currently under, I see no need to recommend this design over the existing one. Perhaps if MSK were in an urban atmosphere where space was an issue, this redesign would be more sensible. However, placed on its own 25 acre lot, there is no need to build vertically. Doing so requires additional lateral systems, larger members, and wider footings. More importantly this design breaks up the lateral internal flow of the original design. Offices originally projected to be next to each other are now five stories apart. Especially in a health-care facility specializing in one field, it is far more efficient to have sectors working together as a team rather then sectioned off between floors. Therefore, because of these reasons and others previously determined in this report, I recommend that the original design for the Outpatient Center be used for Memorial Sloan-Kettering.

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APPENDIX A

LOAD CALCULATIONS

`

WIND LOAD ANALYSIS

Building Properties	
B (ft)	125
L (ft)	188
h (ft)	126.00
K_{zt}	1
K_d	0.85
V (mph)	90
Importance	III
I_w	1.15
Exposure	C
≅	9.5
z_g	900
z_{min}	15
c	0.2
∈	0.2
l	500
ƀ	0.650
≅	0.153846
a	0.105
b	1

Fundamental Period	
Struct. Type	Steel
C_t	0.028
x	0.8
T	1.341079
Natural *f*	0.745668
Rigidity	Flex

Rigid	
g_Q=g_v	3.4
ž	75.6
$I_{ž}$	0.174192
$L_{ž}$	590.1625
Q	0.855094
G	0.857753

Flexible	
g_R	4.12
R_n	0.054
N_1	4.51
≌$_h$	4.43
≌$_B$	0.035
≌$_L$	22.15
R_h	0.200
R_B	0.977
R_L	0.044
$V_{ž}$	97.47
≅	0.05
R	0.34
G_f	0.9017

Windward	
Cp	0.8

Leeward	Ratio	Cp
N-S	0.665	-0.50
E-W	1.504	-0.40

Pressure Coefficients		
Internal		
Enc. Type	Enclosed	
Internal (GC_{pi})	0.18	+/-

K_z and q_z		
Z(ft)	K_z	q_z
0-15	0.85	17.2290
20	0.90	18.2425
25	0.94	19.0533
30	0.98	19.8641
40	1.04	21.0802
50	1.09	22.0937
60	1.13	22.9045
70	1.17	23.7152
80	1.21	24.5260
90	1.24	25.1341
100	1.26	25.5395
120	1.31	26.5530
140	1.36	27.5664
126	1.325	26.8570

Pressures			
Windward	N-S	P_z	0.721
	E-W	P_z	0.721
Leeward	N-S	P_h	-0.451
	E-W	P_h	-0.360

Velocity Pressure Envelope						
	Windward		Leeward		Max psf	
Z(ft)	N-S	E-W	N-S	E-W	N-S	E-W
0-15	12.43	12.43	-12.11	-9.67	24.54	22.10
15-20	13.16	13.16	-12.11	-9.67	25.27	22.83
20-25	13.74	13.74	-12.11	-9.67	25.85	23.41
25-30	14.33	14.33	-12.11	-9.67	26.44	24.00
30-40	15.21	15.21	-12.11	-9.67	27.32	24.87
40-50	15.94	15.94	-12.11	-9.67	28.05	25.61
50-60	16.52	16.52	-12.11	-9.67	28.63	26.19
60-70	17.11	17.11	-12.11	-9.67	29.22	26.78
70-80	17.69	17.69	-12.11	-9.67	29.80	27.36
80-90	18.13	18.13	-12.11	-9.67	30.24	27.80
90-100	18.42	18.42	-12.11	-9.67	30.53	28.09
100-120	19.15	19.15	-12.11	-9.67	31.26	28.82
120-140	19.89	19.89	-12.11	-9.67	31.99	29.55
126	19.37	19.37	-12.11	-9.67	31.48	29.04

Wind Analysis (Analytical Approach)									
CASE 1			Story Force		Cumulative Shear		Overturning Moment	
Level	Trib. Height (ft)	Total Height (ft)	N-S	E-W	N-S	E-W	N-S	E-W
Roof	*7.00*	126.00	41.97	25.77	0	0	5287.96	3246.79
9	*14.00*	112.00	82.29	50.44	41.97	25.77	9216.09	5649.25
8	*14.00*	98.00	81.05	49.62	124.25	76.21	7942.85	4862.49
7	*14.00*	84.00	79.40	48.52	205.30	125.83	6669.61	4075.73
6	*14.00*	70.00	77.67	47.37	284.70	174.35	5436.78	3315.84
5	*14.00*	56.00	75.58	45.98	362.37	221.71	4232.43	2574.88
4	*14.00*	42.00	73.13	44.35	437.95	267.69	3071.56	1862.84
3	*14.00*	28.00	69.97	42.25	511.08	312.05	1959.19	1183.04
2	*14.00*	14.00	65.52	39.29	581.06	354.30	917.25	550.07
1	7.00	0.00	0.00	0.00	**646.57**	**393.59**	**44733.72**	**27320.92**

Wind Analysis (Analytical Approach)													
CASE 3 (75% simultaneous directions)			NW-SE direction			NE-SW Direction			Cumulative Shear		Overturning Moment	
Level	Trib. Height (ft)	Total Height (ft)	N-S	E-W	Total	N-S	E-W	Total	NW-SE	NE-SW	NW-SE	NE-SW
Roof	*7.00*	126.00	31.48	19.33	**36.94**	31.48	19.33	**36.94**	0	0	4653.88	4653.88
9	*14.00*	112.00	61.71	37.83	**72.39**	61.71	37.83	**72.39**	36.94	36.94	8107.30	8107.30
8	*14.00*	98.00	60.79	37.21	**71.27**	60.79	37.21	**71.27**	109.32	109.32	6984.78	6984.78
7	*14.00*	84.00	59.55	36.39	**69.79**	59.55	36.39	**69.79**	180.60	180.60	5862.26	5862.26
6	*14.00*	70.00	58.25	35.53	**68.23**	58.25	35.53	**68.23**	250.38	250.38	4776.11	4776.11
5	*14.00*	56.00	56.68	34.49	**66.35**	56.68	34.49	**66.35**	318.61	318.61	3715.60	3715.60
4	*14.00*	42.00	54.85	33.26	**64.15**	54.85	33.26	**64.15**	384.96	384.96	2694.23	2694.23
3	*14.00*	28.00	52.48	31.69	**61.30**	52.48	31.69	**61.30**	449.11	449.11	1716.50	1716.50
2	*14.00*	14.00	49.14	29.47	**57.30**	49.14	29.47	**57.30**	510.42	510.42	802.16	802.16
1	*7.00*	0.00	0.00	0.00	**0.00**	0.00	0.00	**0.00**	**567.71**	**567.71**	**39312.82**	**39312.82**

SEISMIC LOAD DESIGN

Design Parameters	
# of stories	9
h_s	14 **ft**
h_n	126 **ft**
Seismic Use Group	**III**
Occ. Importance Factor	**1.5**
S_S	0.39 **g-s**
S_1	0.09 **g-s**
F_a	1.00
F_v	1.00
S_{MS}	0.39 **g-s**
S_{M1}	0.09 **g-s**
S_{DS}	0.26 **g-s**
S_{D1}	0.06 **g-s**
Seismic Design Cat.	C

Assumptions:

1) Assumed stiff soil
2)not specifically detailed for seismic resistance
3) Ordiinary Steel Concentrically braced
4) NO partition LL accounted for
5) Site Class B

Equivalent Lateral Force Procedure					
N-S Direction			E-W Direction		
	R_{N-S}	5		R_{E-W}	5
	$C_{S,N-S}$	0.078		$C_{S,E-W}$	0.078
	$C_{T,N-S}$	0.02		$C_{T,E-W}$	0.02
	X	0.75		X	0.75
	T_{N-S}	0.75		T_{E-W}	0.75
but not greater than:			but not greater than:		
	$C_{smax,\ N-S}$	0.024		$C_{smax,\ E-W}$	0.024
and	$C_{s\ min}$	0.0172		$C_{s\ min}$	0.0172
Therefore, ($C_{S,\ N-S}$) used is:		0.024	Therefore, ($C_{S,\ N-S}$) used is:		0.024

Loading Characteristics

Roof:		Slab Floors:	
DL (psf)		DL (psf)	
3.5" Concrete Slab	43.8	Concrete Slab	56.3
Metal Deck Roof	2	Metal Deck	2
Structural Framing	10	Structural Framing	10
Superimposed Dead Loads	15	Superimposed Dead Loads	15
Total:	70.8	Total:	83.3

Perimeter Wall:	
DL (psf)	
	55

Calculation Variables	
Building Width:	125 **ft**
Building Length:	188 **ft**
First Floor Area:	26,500 **ft^2**
2nd-4th Floor Areas:	19500 **ft^2**
Total weight of roof:	1621.6 **kips**
Total weight 2nd-9th floor:	2106.4 **kips**
Total Weight 1st Floor:	
Total Building Weight:	18472.6 **kips**
Seismic Shear, V_{N-S} :	**442.1 kips**
Seismic Shear, V_{E-W} :	**442.1 kips**

Vertical Distribution of Seismic Forces

North - South Direction

Level, x	w_x kips	h_x feet	$w_x h_x^k$	C_{VX}	F_X kips	V_x kips	M_x ft-kips
Roof	1622	126	375,948	0.171	75.5		9512.4
9	2106	112	427,675	0.194	85.9	75.5	9618.8
8	2106	98	367,968	0.167	73.9	161.4	7241.5
7	2106	84	309,331	0.141	62.1	235.3	5217.9
6	2106	70	251,918	0.114	50.6	297.4	3541.2
5	2106	56	195,943	0.089	39.3	348.0	2203.5
4	2106	42	141,723	0.064	28.5	387.3	1195.3
3	2106	28	89,773	0.041	18.0	415.8	504.8
2	2106	14	41,131	0.019	8.3	433.8	115.6
1						442.1	
Σ	7941		2,201,410	1.000	442.1		39151.0

Exponent k_{N-S} : 1.126078

Vertical Distribution of Seismic Forces

East - West Direction

Level, x	w_x kips	h_x feet	$w_x h_x^k$	C_{VX}	F_X kips	V_x kips	M_x ft-kips
Roof	1622	126	375,948	0.171	75.5		9512.4
9	2106	112	427,675	0.194	85.9	75.5	9618.8
8	2106	98	367,968	0.167	73.9	161.4	7241.5
7	2106	84	309,331	0.141	62.1	235.3	5217.9
6	2106	70	251,918	0.114	50.6	297.4	3541.2
5	2106	56	195,943	0.089	39.3	348.0	2203.5
4	2106	42	141,723	0.064	28.5	387.3	1195.3
3	2106	28	89,773	0.041	18.0	415.8	504.8
2	2106	14	41,131	0.019	8.3	433.8	115.6
1						442.1	
Σ	7941		2,201,410	1.000	442.1		39151.0

Exponent k_{E-W} : 1.126078

APPENDIX B

HAND AND EXCEL DESIGN CALCULATIONS

APPENDIX B

COLUMN DESIGN

TABLE 4-2

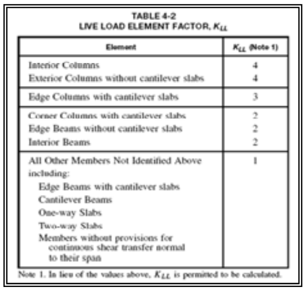


TABLE 4-2
LIVE LOAD ELEMENT FACTOR, K_{LL}

Element	K_{LL} (Note 1)
Interior Columns Exterior Columns without cantilever slabs	4 4
Edge Columns with cantilever slabs	3
Corner Columns with cantilever slabs Edge Beams without cantilever slabs Interior Beams	2 2 2
All Other Members Not Identified Above including: Edge Beams with cantilever slabs Cantilever Beams One-way Slabs Two-way Slabs Members without provisions for continuous shear transfer normal to their span	1

Note 1. In lieu of the values above, K_{LL} is permitted to be calculated.

Assumptions:

1) S.D.L. = 15 lbs
2) Roof Snow Load = 23 psf
- added in after Roof LL accessed

**** Air Handling Units add approximately 50 psf to DL

Roof Live Load Reduction	
R_1 =	0.6
R_2 =	1

Perimeter Wall DL			
Span	Weight	Story Ht.	Total DL
ft	psf	ft	kips
30	75	14	31.5

Gravity Column Design A
Typical Interior Gravity Loads

Column	Floor Above	Span	Area	A_T	K_{LL}	A_I	L	DL	LL	LLR	Tot. LL	Tot. DL	Factored	Unfactored	
		ft x ft	sq. ft	sq. ft		sq. ft		psf	psf	psf	kips	kips	kips	kips	
9	Roof	30 x 30	900	900	4	3600	0.600	125	40	47	42.3	112.5	202.68	154.80	------->> Air Handling Units
8	9	30 x 30	900	1800	4	7200	0.427	85	100	42.68	38.41	76.5	355.94	269.71	
7	8	30 x 30	900	2700	4	10800	0.400	85	100	40	36	76.5	505.34	382.21	
6	7	30 x 30	900	3600	4	14400	0.400	85	100	40	36	76.5	654.74	494.71	
5	6	30 x 30	900	4500	4	18000	0.400	135	100	40	36	121.5	858.14	652.21	------->> "Mechanical Floor"
4	5	30 x 30	900	5400	4	21600	0.400	85	100	40	36	76.5	1007.54	764.71	
3	4	30 x 30	900	6300	4	25200	0.400	85	100	40	36	76.5	1156.94	877.21	
2	3	30 x 30	900	7200	4	28800	0.400	85	100	40	36	76.5	1306.34	989.71	
1	2	30 x 30	900	8100	4	32400	0.400	85	100	40	36	76.5	1455.74	1102.21	------->> Last Steel Column
Conc. Pier	1	30 x 30	900	9000	4	36000	0.400	140	100	40	36	126	1664.54	1264.21	------->> Concrete Pier

Gravity Column Design B
Atypical Interior Gravity Loads (South Side)

Column	Floor Above	Span*	Area	A_T	K_{LL}	A_I	L	DL	LL	LLR	Tot. LL	Tot. DL	Factored	Unfactored	
		ft x ft	sq. ft	sq. ft		sq. ft		psf	psf	psf	kips	kips	kips	kips	
9	Roof	30 x 45	1125	1125	4	4500	0.600	125	40	47	52.875	140.625	253.35	193.50	------->> Air Handling Units
8	9	30 x 45	1125	2250	4	9000	0.408	85	100	40.81	45.91	95.625	441.56	335.04	
7	8	30 x 45	1125	3375	4	13500	0.400	85	100	40	45	95.625	628.31	475.66	
6	7	30 x 45	1125	4500	4	18000	0.400	85	100	40	45	95.625	815.06	616.29	
5	6	30 x 45	1125	5625	4	22500	0.400	135	100	40	45	151.875	1069.31	813.16	------->> "Mechanical Floor"
4	5	30 x 45	1125	6750	4	27000	0.400	85	100	40	45	95.625	1256.06	953.79	
3	4	30 x 45	1125	7875	4	31500	0.400	85	100	40	45	95.625	1442.81	1094.41	
2	3	30 x 45	1125	9000	4	36000	0.400	85	100	40	45	95.625	1629.56	1235.04	
1	2	30 x 30	900	9900	4	39600	0.400	85	100	40	36	76.5	1778.96	1347.54	------->> Last Steel Column
Conc. Pier	1	30 x 30	900	10800	4	43200	0.400	140	100	40	36	126	1987.76	1509.54	------->> Concrete Pier

* Atypical Span constitutes 30' x 30' bay adjacent to 30' x 45' bay

Gravity Column Design C
Typical Exterior Gravity Loads (South Side)

Column	Floor Above	Span	Area	A_T	K_{LL}	A_I	L	DL	LL	LLR	Tot. LL	Tot. DL	Factored	Unfactored	
		ft x ft	sq. ft	sq. ft		sq. ft		psf	psf	psf	kips	kips	kips	kips	
9	Roof	30 x 45	675	675	4	2700	0.600	125	40	47.00	31.73	84.38	152.01	116.10	------->> Air Handling Units
8	9	30 x 45	675	1350	4	5400	0.454	85	100	45.41	30.65	88.88	307.71	235.63	
7	8	30 x 45	675	2025	4	8100	0.417	85	100	41.67	28.13	88.88	459.36	352.63	
6	7	30 x 45	675	2700	4	10800	0.400	85	100	40.00	27.00	88.88	609.21	468.50	
5	6	30 x 45	675	3375	4	13500	0.400	135	100	40.00	27.00	122.63	799.56	618.13	------->> "Mechanical Floor"
4	5	30 x 45	675	4050	4	16200	0.400	85	100	40.00	27.00	88.88	949.41	734.00	
3	4	30 x 45	675	4725	4	18900	0.400	85	100	40.00	27.00	88.88	1099.26	849.88	
2	3	30 x 45	675	5400	4	21600	0.400	85	100	40.00	27.00	88.88	1249.11	965.75	
1	2	16 x 30	240	5640	4	22560	0.400	85	100	40.00	9.60	51.90	1326.75	1027.25	------->> Last Steel Column
Conc. Pier	1	16 x 30	240	5880	4	23520	0.400	140	100	40	9.6	65.1	1420.23	1101.95	------->> Concrete Pier

Gravity Column Design D
Typical Exterior Gravity Loads (North Side)

Column	Floor Above	Span	Area	A_T	K_{LL}	A_I	L	DL	LL	LLR	Tot. LL	Tot. DL	Factored	Unfactored	
		ft x ft	sq. ft	sq. ft		sq. ft		psf	psf	psf	kips	kips	kips	kips	
9	Roof	30 x 16	240	240	4	960	0.600	125	40	47.00	11.28	30.00	54.05	41.28	------->> Air Handling Units
8	9	30 x 16	240	480	4	1920	0.592	85	100	59.23	14.22	51.90	139.07	107.40	
7	8	30 x16	240	720	4	2880	0.530	85	100	52.95	12.71	51.90	221.69	172.00	
6	7	30 x16	240	960	4	3840	0.492	85	100	49.21	11.81	51.90	302.86	235.71	
5	6	30 x16	240	1200	4	4800	0.467	135	100	46.65	11.20	63.90	397.46	310.81	------->> "Mechanical Floor"
4	5	30 x16	240	1440	4	5760	0.448	85	100	44.76	10.74	51.90	476.92	373.45	
3	4	30 x16	240	1680	4	6720	0.433	85	100	43.30	10.39	51.90	555.83	435.74	
2	3	30 x16	240	1920	4	7680	0.421	85	100	42.12	10.11	51.90	634.28	497.75	
1	2	30 x16	240	2160	4	8640	0.411	85	100	41.14	9.87	51.90	712.36	559.53	------->> Last Steel Column
Conc. Pier	1	30 x16	240	2400	4	9600	0.403	140	100	40.30931	9.67	65.1	805.96	634.30	------->> Concrete Pier

APPENDIX B

Gravity Column Design E															
Atypical Interior Gravity Loads (North Side)															
Column	Floor Above	Span	Area	A_T	K_{LL}	A_I	L	DL	LL	LLR	Tot. LL	Tot. DL	Factored	Unfactored	
		ft x ft	sq. ft	sq. ft		sq. ft		psf	psf	psf	kips	kips	kips	kips	
9	Roof	30 x 23	690	690	4	2760	0.600	125	40	47.00	32.43	86.25	155.39	118.68	------->> Air Handling Units
8	9	30 x 23	690	1380	4	5520	0.452	85	100	45.19	31.18	58.65	275.66	208.51	
7	8	30 x 23	690	2070	4	8280	0.415	85	100	41.48	28.62	58.65	391.84	295.78	
6	7	30 x 23	690	2760	4	11040	0.400	85	100	40.00	27.60	58.65	506.38	382.03	
5	6	30 x 23	690	3450	4	13800	0.400	135	100	40.00	27.60	93.15	662.32	502.78	------->> "Mechanical Floor"
4	5	30 x 23	690	4140	4	16560	0.400	85	100	40.00	27.60	58.65	776.86	589.03	
3	4	30 x 23	690	4830	4	19320	0.400	85	100	40.00	27.60	58.65	891.40	675.28	
2	3	30 x 23	690	5520	4	22080	0.400	85	100	40.00	27.60	58.65	1005.94	761.53	
1	2	30 x 23	690	6210	4	24840	0.400	85	100	40.00	27.60	58.65	1120.48	847.78	------->> Last Steel Column
Conc. Pier	1	30 x 23	690	6900	4	27600	0.400	140	100	40.00	27.60	96.60	1280.56	971.98	------->> Concrete Pier

Gravity Column Redesign Comparison				
Hand Calcuations vs. RAM Design				
	Column Location	Hand Calculations		RAM
		Force (kips)	Size	Size Given
Design A	Typical Interior Column	1455.74	W12x152	W12x152
Design B	Atypical Interior Column (South Side)	1778.96	W12x190	W12x190
Design C	Typical Exterior Column (South Side)	1326.75	W12x136	W12x136
Design D	Typical Exterior Column (North Side)	712.36	W12x72	W12x79
Design E	Atypical Interior Column (North Side)	1120.48	W12x120	W12x120

Concrete Pier Reinforcement Design							
Hand Calculations							
	Column Location	Applied Loads		RAM	Steel	Checks	
		Axial	Moment	Rq'd Amt.	Configuation	Pnmax	Adequate?
Design A	Typical Interior Column	1664.54	468	13.82	(12) #10 bars	1714	YES
Design B	Atypical Interior Column (South Side)	1987.76	468	20.32	(16) # 11 bars	1996	YES
Design C	Typical Exterior Column (South Side)	1420.23	468	11.52	(12) #10 bars	1714	YES
Design D	Typical Exterior Column (North Side)	805.96	468	11.52	(12) #10 bars	1714	YES
Design E	Atypical Interior Column (North Side)	1280.56	468	11.52	(12) #10 bars	1714	YES

Allowable Bearing Stress for Fo **20 ksf**

Resizing of Pier Footings					
from Hand Calcuations					
	Column Location	Footing Loads		New Footing	Old Footing
		Force (kips)	Sq. Ft.	Redesigned Size	Previous Size
Design A	Typical Interior Column	1664.54	83.22679	9' x 10'	6' x 6'
Design B	Atypical Interior Column (South Side)	1987.76	99.38802	10' x 10'	6' x 6'
Design C	Typical Exterior Column (South Side)	1420.23	71.01127	8' x 10'	6' x 6'
Design D	Typical Exterior Column (North Side)	805.96	40.29798	7' x 7'	6' x 6'
Design E	Atypical Interior Column (North Side)	1280.56	64.0278	8' x 8'	6' x 6'

RAM Gravity Column Redesign Results (2nd Floor Column)

Row	Column	Given	Designed	Notes
H	RA	W10x33	W10x33	Entrance Canopy
H	15.3	W12x79	W12x136	Lateral System
H	16	W12x79	W12x136	Lateral System
H	17	W12x65	W12x79	
H	18	W12x96	W12x210	Lateral System
H	18.8	W12x45	W12x190	Lateral System
H	19.3	W12x45	W12x65	

Row	Column	Given	Designed	Notes
I	RB	W12x53	W12x65	
I	16	W12x72	W12x96	
I	17	W12x72	W12x120	
I	18	W12x120	W12x210	Lateral System
I	19	W12x65	W12x190	Lateral System
I	RC	W12x72	W12x106	

Row	Column	Given	Designed	Notes
J	RB	W12x53	W12x72	
J	16	W12x87	W12x120	
J	17	W12x96	W12x152	
J	18	W12x96	W12x152	
J	19	W12x87	W12x152	
J	20	W12x72	W12x120	
J	RC	W12x72	W12x106	

Row	Column	Given	Designed	Notes
K	RB	W12x53	W12x72	
K	16	W12x87	W12x120	
K	17	W12x106	W12x152	
K	18	W12x96	W12x190	
K	19	W12x96	W12x190	
K	20	W12x106	W12x210	
K	RC	W12x106	W12x170	Lateral System

Row	Column	Given	Designed	Notes
L	RB	W12x53	W12x65	
L	16	W12x96	W12x190	Lateral System
L	16.5	W12x79	W12x210	Lateral System
L	17	W12x40	W12x65	
L	18	W12x40	W12x40	One Floor
L	19	W12x40	W12x40	One Floor
L	20	W12x40	W12x40	One Floor
L	21	W12x40	W12x40	One Floor

Row	Column	Given	Designed	Notes
M	15.8	W12x45	W12x53	
M	16.5	W12x79	W12x210	Lateral System
M	17	W12x65	W12x65	
M	18	W12x87	W12x152	
M	19	W12x87	W12x152	
M	20	W12x72	W12x136	
M	21	W12x65	W12x136	
M	21.3	W12x45	W12x136	

Row	Column	Given	Designed	Notes
L3	RC	W12x45	W12x106	Lateral System
L7	RA	W10x33	W10x33	
L7	RB	W12x45	W12x45	

Row	Column	Given	Designed	Notes
RA	R7	W10x33	W10x33	Entrance Canopy
RA	R9	W10x33	W10x33	Entrance Canopy
RA	R11	W10x33	W10x33	Entrance Canopy
RA	R13	W10x33	W10x33	Entrance Canopy

Shear Wall Loads

North-South Direction

Lateral Frame # 1

W_1	16.82	kips
W_2	213.62	kips
W_3	14.96	kips
W_4	14.47	kips
W_5	186.3	kips
W_6	187.53	kips
W_7	172.83	kips
W_8	147.6	kips
W_9	151.88	kips
W_{10}	150.57	kips

Lateral Frame # 3

W_1	1.99	kips
W_2	204.49	kips
W_3	6.5	kips
W_4	-3.01	kips
W_5	159.24	kips
W_6	184.61	kips
W_7	148.86	kips
W_8	-145.87	kips
W_9	143.22	kips
W_{10}	117.28	kips

Lateral Frame # 5

W_1	-15.07	kips
W_2	120.84	kips
W_3	-15.21	kips
W_4	-11.17	kips
W_5	102.35	kips
W_6	91.62	kips
W_7	71.82	kips
W_8	-94.44	kips
W_9	57.35	kips
W_{10}	68.34	kips

Lateral Frame # 7

W_1	-3.87	kips
W_2	108.87	kips
W_3	-6.37	kips
W_4	-0.41	kips
W_5	90.05	kips
W_6	74.23	kips
W_7	-67.5	kips
W_8	-73.31	kips
W_9	50.96	kips
W_{10}	67.16	kips

East-West Direction

Lateral Frame # 2

W_1	81.32	kips
W_2	-12.47	kips
W_3	75.06	kips
W_4	67.25	kips
W_5	-21.39	kips
W_6	-0.43	kips
W_7	51.64	kips
W_8	70.34	kips
W_9	55.86	kips
W_{10}	34.51	kips
O_1	98.2	kips

Lateral Frame # 4

W_1	113.99	kips
W_2	9.63	kips
W_3	95.85	kips
W_4	103.63	kips
W_5	18.8	kips
W_6	-1.95	kips
W_7	92.71	kips
W_8	78.27	kips
W_9	70.52	kips
W_{10}	91.72	kips
O_1	136.4	kips

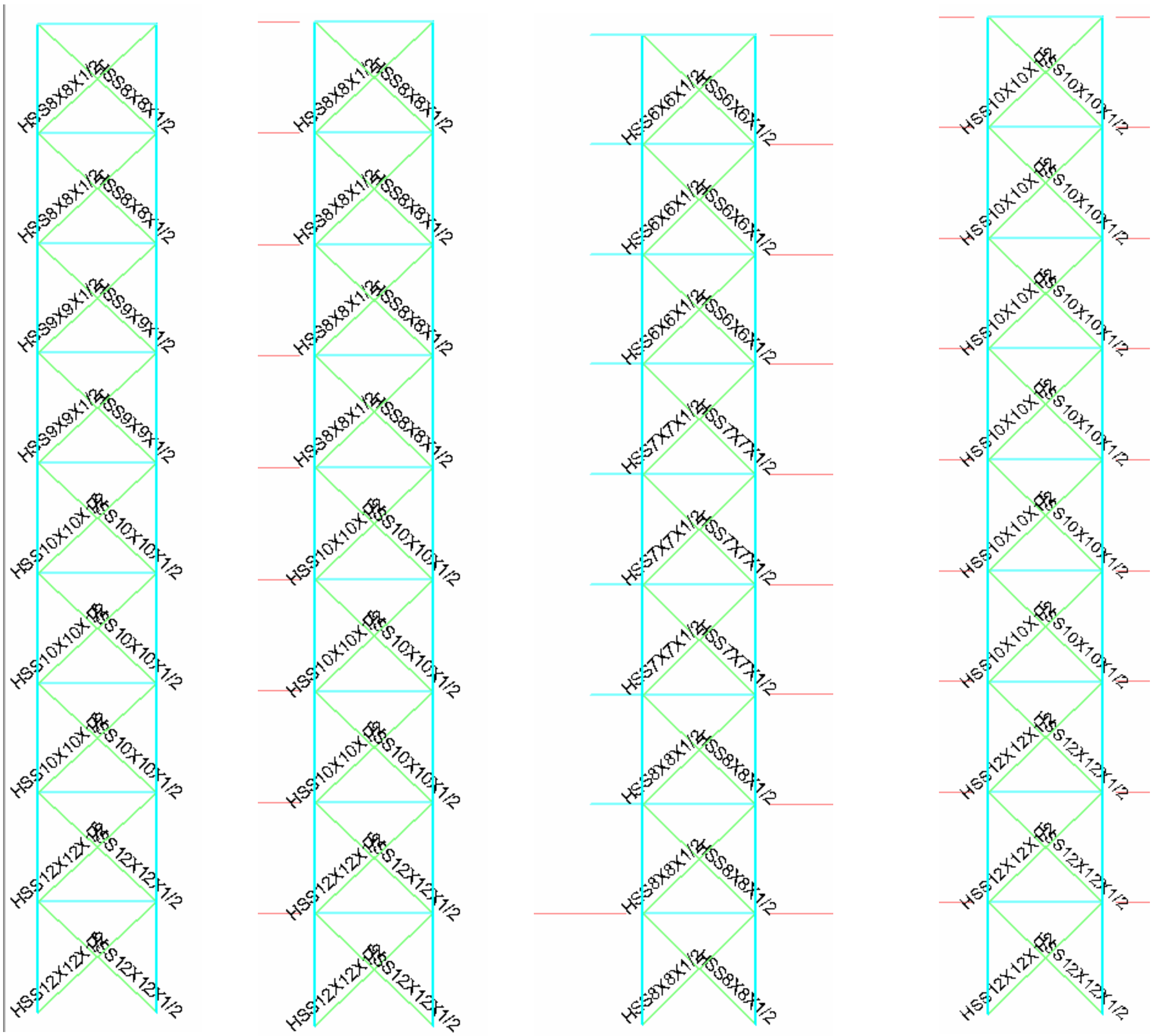
Lateral Frame # 6

W_1	87.62	kips
W_2	27.24	kips
W_3	72.76	kips
W_4	80.58	kips
W_5	34.32	kips
W_6	13.36	kips
W_7	86.15	kips
W_8	45.28	kips
W_9	64.71	kips
W_{10}	86.06	kips
O_1	102.61	kips

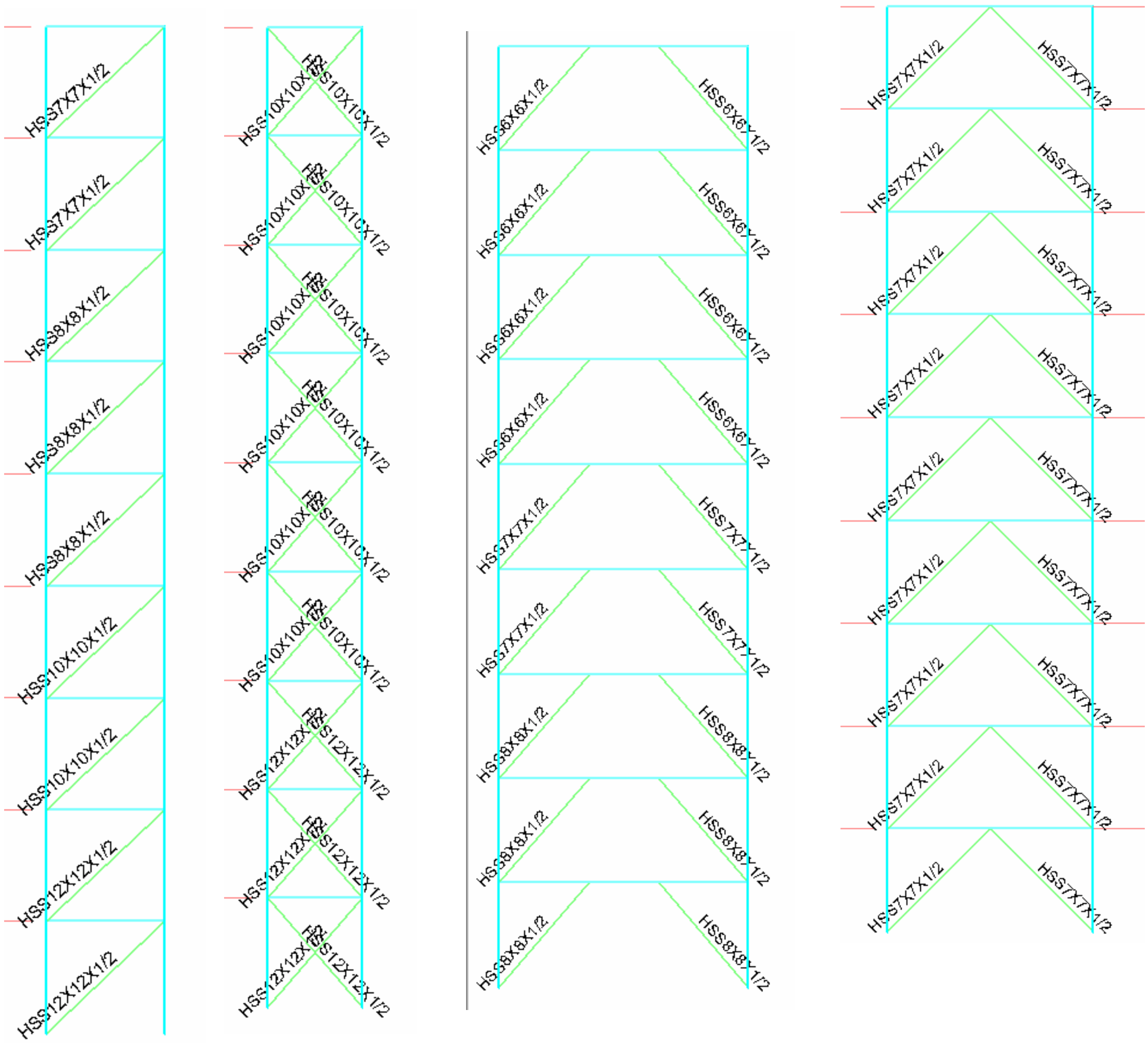
Lateral Frame # 8

W_1	75.34	kips
W_2	-1.61	kips
W_3	69.1	kips
W_4	62.74	kips
W_5	-9.89	kips
W_6	7.06	kips
W_7	55.29	kips
W_8	57.71	kips
W_9	57.04	kips
W_{10}	39.72	kips
O_1	98.88	kips

APPENDIX B



Braced Frame Sections



Braced Frame Sections (Continued)

Shear Wall Check – Base Shear

E-W → FRAME # 4

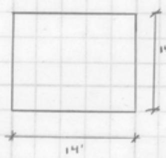
1

SHEAR WALL DESIGN

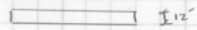
-Check BASE SHEAR → Look @ Frame 4 → Worst IN E-W Direction

→ $V_u = 136.40^k$

ELEVATION



PLAN VIEW



$t = 12"$
$b = (12)(14) = 168"$
$h = (12)(14) = 168"$

$$V_n = \frac{136.40^k}{.75} = 181.86^k$$

– BECAUSE SHEAR WALL SPANS BETWEEN TWO AXIAL COLUMNS, ASSUME SHEAR WALL ONLY TAKES SHEAR AND FLEXURE

$$V_c = 2\sqrt{f'c} \cdot b_w \cdot d = 2\sqrt{5000} \cdot (168)(12") = 285\ K$$

$$\frac{\phi V_c}{2} = \frac{(.65)(285)}{2} = 92^k \therefore \text{REINFORCEMENT REQUIRED}$$

→ Try EXISTING REINFORCEMENT

→ Try #5 bars @ 12" o.c. for EACH FACE

$$A_v = .75\sqrt{5000} \cdot \frac{(12)(12)}{50} = .152$$

MUST BE GREATER THAN $(50 \cdot 12 \cdot 12)/_{60,000} = .12$

→ USE #5 bars @ 12" ON EACH FACE

d bending in STRONG AXIS ∴ d = 168"

$$V_s = \frac{(2 \cdot .31)(60{,}000)(168)}{12} = 520.8^K$$

$$\phi V_n = (.75)(520.8 + 285) = 604^k > 181.86^k$$

∴ Shear WALL DESIGN ADEQUATE FOR BASE SHEAR

Shear Wall Check – Overturning Moment

E-W → FRAME #4

- CHECK FRAME 4 FOR OVERTURNING MOMENT

→ from seismic loading = 12,245 ft kips on Frame 4

$t = 12''$
$b = 168''$
$h = 168''$

FIND LOADS ACTING ON COLUMNS

$A_T = 400\ ft^2$

DL = 80 psf
LL = 100 psf
S_{DL} = 20 psf

INTERIOR COLUMN $K_{LL} = 4$

$A_T = (10)(400) = 4000$

$(4)(4000) = 16000$

$LLR = .25\left(\frac{15}{\sqrt{16000}}\right) = .4$

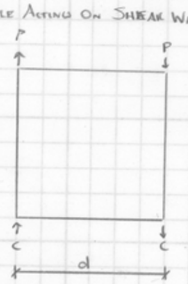
$LL = (4000)(100)(.4)\ \ 160^K$

$DL = (140.75)(400) + 9(83.25)(400)$

$= 356^K$

$1.2(356\ K) + 1.6(160\ K) = \underline{683.2^K}$ → NEED TO ALSO CONSIDER WEIGHT OF FOUNDATION

→ COUPLE ACTING ON SHEAR WALL



$Couple = M_u/d = \frac{12,245}{14} = 874.64^K$

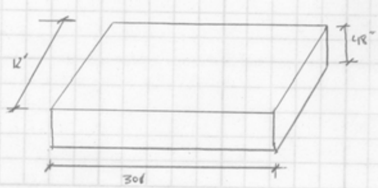
→ FOUNDATION UNDER REPRESENTATIVE SHEAR WALL is 30' x 8' x 4'

$w = (4')(150)(8')(30') = 144^K$

$144^K + 683.2^K = 827.2^K < 874.64^K$

→ larger footing required

→ Try a 30' x 12' x 4' FOUNDATION



$w = (4')(150\ pcf)(12')(30') = 216^K$

$216^K + 683.2^K = 899.2^K$

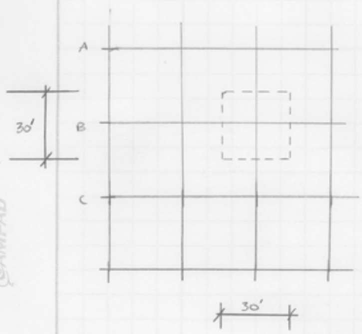
$899.2^K > 874.64^K$

$\therefore$ OK

Concrete Pier Design Moment

Concrete Pier Moments - INTERIOR Columns

→ 1st Floor Bays → 30' x 30'



$A_T = 30' \times 30' = 900\ ft^2$

$A_I = 4(900) = 3600\ ft^2$

k_{LL}

$LL_{reduction} = lo\left(.25 + \frac{15}{\sqrt{3600}}\right) = .5$

Loads

DEAD LOAD

- Floor – 6" slab and 20 psf S.D.L
- GIRDERS – 24" x 30" → span from col. to col.

DEAD LOAD

$-(6/12")(150\ pcf) + (18/12)(24/12)(150\ pcf)(1/10)$

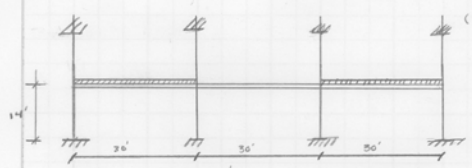
$+(24/12)(30/12)(150\ pcf)(1/30) + 20\ psf.$

$= 165\ psf \cdot (30') = 4.95\ k/ft$

LIVE LOAD

100 psf

$(100\ psf)(.5)(30) = 1.5\ k/ft$



→ All bays 30' x 30'

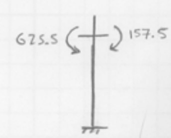
- WORST CASE SENARIO: ALL LL ON ONE SIDE, NONE ON THE OTHER
  - CAUSES MOMENT
- FIXED ENDS MOMENT – CONCRETE POURED MONOLITHICALLY
  - $\frac{w\ell^2}{12}$

→ FULLY LOADED BAY → $1.2(4.95) + 1.6(1.5) = 8.34\ k/ft$

$\frac{(8.34)(30)^2}{12} = 625.5\ ft\text{-}kips$

→ Dead LOAD ONLY → $1.4(1.5) = 2.1\ k/ft$

$\frac{(2.1)(30)^2}{12} = 157.5$



$\Delta FEM = 625.5 - 157.5$

$\boxed{\Delta FEM = 468\ ft\text{-}kips}$

- Assum 100% of moment goes into column due to no moment above

Concrete Pier Reinforcement Design

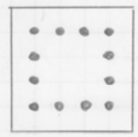
1

→ Typical Interior Column
w/ 30'x30' bay on all Floors

$P_u = 1665$ k
$M_u = 468$ ft-kips

→ 24"x24" concrete pier
- Square ties
→ $f'_c = 5$ ksi
– 1 ½" concrete cover

Try 12 bar configuration



→ USE Design Aid interaction diagram to estimate steel reinforcement

$\gamma = 24 - 2(1.5+.5+.5) = 19"$ $\quad \gamma = 19/24 = .79 \rightarrow$ use .8 $\quad A_g = 24 \times 24 = 576 \text{ in}^2$

$$R_n = \frac{M_u}{(\phi \cdot f_c \cdot A_g \cdot h)} = \frac{(468)(12)}{(.65 \cdot 5 \cdot 576 \cdot 24)} = 0.125$$

→ Look @ CRSI HANDBOOK @ 0% F_y
(12) #10's most efficient $\quad \phi M_n = 6204 > 5616 \therefore$ OK
$\phi P_n = 1547 > 1540 \therefore$ OK

$$K_n = \frac{P_u}{\phi f_c \cdot A_g} = \frac{1665}{(.65 \cdot 5 \cdot 576)} = 0.88$$

→ Go INTO INTERACTION DIAGRAM → $\rho = 0.024$ $\quad A_{ST} = (576)(.024) = 13.82"$
→ Use (12) #10 bars
$A_{ST} = 15.24 \text{ in}^2$

Check

$$\phi P_{n_{max}} = 0.80 \cdot \phi\left[.85 \cdot f_c \cdot (A_g - A_{ST}) + f_y \cdot A_{ST}\right]$$

$$= 0.80 \cdot .65\left[.85 \cdot 5 \cdot (576 - 15.24) + 60 \cdot 15.24\right] = 1714^k > 1665^k \therefore \text{OK}$$

→ Typical INTERIOR COLUMN
W/ 30'x45' bays on Floors 3 through Roof

$P_u = 1980$ K
$M_u = 468$ ft-kips

$$R_n = \frac{M_u}{(\phi \cdot f'_c \cdot A_g \cdot h)} = \frac{(468 \cdot 12)}{(.65 \cdot 5 \cdot 576 \cdot 24)} = 0.125$$

→ Go INTO INTERACTION DIAGRAM → $\rho = 0.034$

$$k_n = \frac{P_u}{(\phi \cdot f_c \cdot A_g)} = \frac{1980}{(.65 \cdot 5 \cdot 576)} = 1.05$$

$A_{ST} = (576)(0.034) = 19.58$
→ Use (16) #10 bars
$A_{ST} = 20.32"$

Check

$$\phi P_{n_{max}} = 0.80 \cdot \phi\left[.85 \cdot f_c \cdot (A_g - A_{ST}) + f_y \cdot A_{ST}\right]$$

$$= 0.80 \cdot .65\left[.85 \cdot 5 \cdot (576 - 20.32) + 60(20.32)\right] = 1862^k > 1830^k \therefore \text{OK}$$

→ Look @ CRSI Handbook @ 0% f_y
(16) #11's more efficient $\quad \phi P_n = 2150^k$

→ Use (16) #11 bars $\quad A_{ST} = 24.96"$

$$= .8 \cdot .65\left[.85 \cdot 5 \cdot (576 - 24.96) + 60(24.96)\right] = 1996^k > 1980^k \therefore \text{OK}$$

2

→ EXTERIOR COLUMN

→ SOUTH SIDE → (WORSE SENARIO THAN NORTH SIDE)

$P_u = 1426$

$M_u = 625.5$

$R_n = \frac{(625.5 \cdot 12)}{(.65 \cdot 5 \cdot 576 \cdot 24)} = 0.16$

$\rho = 0.02$

$K_n = \frac{1420}{(.65 \cdot 5 \cdot 576)} = .75$

→ $A_{ST} = (.02)(576) = 11.52$

→ Try (12) #10 bars

$A_{ST} = 15.24''$

Check

$\emptyset P_{n_{max}} = 0.80 \cdot .65\left[.85 \cdot 5 \cdot (576 - 15.24) + 60(15.24)\right] = 1714^k > 1426^k$

→ Look @ CRSI Handbook

$$\boxed{\begin{array}{l} \emptyset M_n = 6204 > 5616\ k \\ \emptyset P_n = 1547 > 1426\ k \end{array}}$$

→ Use (12) #10 bars

SQUARE TIED COLUMNS 22" × 22"

Short columns; no sidesway
Bars symmetrical in 4 faces

f'_c = 5,000 psi ϕM in inch-kips f_y = 60,000 psi ϕP in kips

BARS	RHO	Max Cap		0% fy		25% fy		50% fy		100% fy		.1f'c Ag		Zero Axial Load
		φM	φP	φM	φP	φM	φP	φM	φP	φM	φP	φM	φP	φM
4-#10	1.05	2931	1310	4111	1120	4855	943	5243	805	5604	596	3978	242	2575
4-#11	1.29	2998	1347	4326	1132	5088	951	5506	807	5943	589	4330	242	3087
4-#14	1.86	3172	1433	4780	1178	5637	984	6157	827	6811	580	5214	242	4298
4-#18	3.31	3566	1651	5868	1300	6963	1071	7738	879	8922	561	7363	242	7208
8-# 8	1.31	2885	1349	4035	1158	4785	972	5172	826	5520	606	4340	242	3180
8-# 9	1.65	2969	1402	4255	1189	5051	995	5488	839	5942	603	4866	242	3947
8-#10	2.10	3075	1469	4532	1229	5389	1024	5890	857	6478	600	5478	242	4911
8-#11	2.58	3166	1542	4831	1265	5729	1049	6282	871	6991	591	6044	242	5869
8-#14	3.72	3412	1714	5496	1370	6545	1124	7258	916	8297	582	7456	242	8157
8-#18	6.61	4011	2151	7097	1636	8522	1316	9628	1033	11474	556	10757	242	12884
12-#10	3.15	3333	1628	5080	1338	6050	1112	6687	917	7545	596	6797	242	7035
12-#11	3.87	3471	1736	5486	1399	6521	1156	7236	943	8269	584	7667	242	8224
12-#14	5.58	3866	1995	6423	1561	7665	1278	8607	1019	10104	570	9691	242	11123
16-#10	4.20	3574	1786	5660	1448	6760	1193	7537	970	8676	599	8115	242	8943
16-#11	5.16	3773	1931	6180	1532	7369	1255	8253	1006	9622	586	9191	242	10556
20-#10	5.25	3819	1945	6208	1564	7489	1274	8396	1030	9834	602	9420	242	10798

SQUARE TIED COLUMNS 24" × 24"

BARS	RHO	Max Cap φM	Max Cap φP	0% fy φM	0% fy φP	25% fy φM	25% fy φP	50% fy φM	50% fy φP	100% fy φM	100% fy φP	.1f'c Ag φM	.1f'c Ag φP	Zero Axial Load φM
4-#11	1.08	3825	1566	5367	1338	6348	1126	6866	961	7359	711	5247	288	3446
4-#14	1.56	4028	1652	5883	1384	6969	1159	7601	980	8338	702	6244	288	4812
4-#18	2.78	4494	1870	7123	1504	8476	1245	9395	1031	10731	662	8681	288	8112
8-# 8	1.10	3692	1568	5028	1366	5999	1150	6483	981	6878	730	5248	288	3540
8-# 9	1.39	3790	1621	5277	1397	6299	1172	6838	995	7350	727	5853	288	4403
8-#10	1.76	3912	1688	5589	1437	6678	1202	7289	1012	7951	724	6596	288	5489
8-#11	2.17	4019	1760	5933	1473	7067	1226	7736	1026	8534	715	7255	288	6580
8-#14	3.13	4303	1933	6687	1577	7989	1302	8837	1072	10006	706	8883	288	9180
8-#18	5.56	5000	2370	8508	1844	10234	1494	11525	1189	13606	684	12700	288	15057
12-#10	2.65	4206	1847	6204	1547	7420	1291	8182	1074	9148	721	8035	288	7994
12-#11	3.25	4368	1955	6670	1607	7957	1335	8808	1100	9970	710	9032	288	9407
12-#14	4.69	4797	2214	7732	1770	9251	1458	10357	1177	12042	698	11498	288	12717
16-#10	3.53	4479	2005	6854	1658	8216	1373	9136	1128	10416	726	9554	288	10118
16-#11	4.33	4691	2150	7450	1742	8912	1435	9952	1165	11492	715	10800	288	11978
16-#14	6.25	5270	2495	8837	1963	10604	1600	11978	1269	14200	702	13851	288	16273
20-#10	4.41	4734	2164	7461	1773	9034	1455	10100	1189	11700	724	11036	288	12313
20-#11	5.42	5021	2345	8186	1884	9892	1535	11107	1239	13049	719	12534	288	14458

(1) "0% f_y" indicates zero tension in bars on the tension side, "50% f_y" indicates 50% f_y stress in bars on the tension side, and "100% f_y" indicates 100% f_y stress (i.e., balance point) in bars on the tension side.

CONCRETE REINFORCING STEEL INSTITUTE

SQUARE TIED COLUMNS 26" × 26"

Short columns; no sidesway
Bars symmetrical in 4 faces

f'_c = 5,000 psi ϕM in inch-kips f_y = 60,000 psi ϕP in kips

BARS	RHO	Max Cap		0% fy		25% fy		50% fy		100% fy		.1f'c Ag		Zero Axial Load
		φM	φP	φM	φP	φM	φP	φM	φP	φM	φP	φM	φP	φM
4-#14	1.33	5023	1890	7140	1608	8496	1350	9256	1146	10078	835	7397	338	5325
4-#18	2.37	5561	2108	8533	1727	10184	1435	11262	1197	12752	814	10120	338	9015
8-# 9	1.18	4748	1859	6449	1624	7739	1366	8398	1164	8972	862	6949	338	4857
8-#10	1.50	4887	1926	6798	1664	8161	1395	8898	1182	9638	859	7802	338	6065
8-#11	1.85	5011	1998	7188	1699	8598	1419	9400	1194	10290	850	8596	338	7289
8-#14	2.66	5335	2171	8031	1803	9627	1495	10627	1241	11929	842	10445	338	10200
8-#18	4.73	6135	2608	10074	2070	12140	1688	13632	1360	15952	821	14792	338	17166
12-#10	2.25	5220	2085	7479	1775	8983	1486	9888	1244	10963	858	9397	338	8861
12-#11	2.77	5405	2193	8007	1834	9587	1530	10590	1270	11884	847	10521	338	10614
12-#14	3.99	5892	2452	9194	1998	11031	1653	12317	1348	14193	836	13292	338	14362
12-#18	7.10	7121	3107	12076	2413	14558	1965	16545	1547	19852	805	19485	338	23230
16-#10	3.01	5524	2243	8200	1886	9865	1569	10945	1299	12369	864	11103	338	11317
16-#11	3.69	5766	2388	8874	1970	10648	1631	11861	1336	13575	853	12522	338	13415
16-#14	5.33	6393	2733	10424	2192	12538	1797	14122	1443	16594	843	15955	338	18476
20-#10	3.76	5811	2402	8866	2001	10772	1652	12014	1361	13792	863	12789	338	13850
20-#11	4.62	6108	2583	9682	2111	11738	1731	13145	1412	15286	850	14483	338	16306

SQUARE TIED COLUMNS 28" × 28"

BARS	RHO	Max Cap φM	Max Cap φP	0% fy φM	0% fy φP	25% fy φM	25% fy φP	50% fy φM	50% fy φP	100% fy φM	100% fy φP	.1f'c Ag φM	.1f'c Ag φP	Zero Axial Load φM
4-#14	1.15	6169	2147	8561	1852	10232	1558	11137	1328	12048	979	8681	392	5837
4-#18	2.04	6781	2365	10109	1970	12103	1642	13358	1377	15004	957	11693	392	9916
8-# 9	1.02	5856	2116	7785	1870	9388	1576	10185	1347	10824	1008	8174	392	5311
8-#10	1.30	6012	2183	8170	1910	9852	1605	10735	1365	11555	1005	9113	392	6640
8-#11	1.59	6154	2255	8607	1944	10339	1629	11291	1377	12276	996	10047	392	7996
8-#14	2.30	6519	2428	9540	2048	11475	1704	12644	1424	14081	988	12152	392	11218
8-#18	4.08	7385	2865	11806	2315	14257	1898	15968	1543	18528	969	17041	392	18973
12-#10	1.94	6383	2342	8918	2021	10754	1696	11821	1428	13009	1005	10883	392	9725
12-#11	2.39	6594	2450	9507	2080	11426	1740	12600	1454	14028	994	12144	392	11679
12-#14	3.44	7140	2709	10821	2244	13021	1864	14505	1533	16574	985	15194	392	16057
12-#18	6.12	8522	3364	14018	2660	16927	2178	19184	1734	22834	957	22257	392	25955
16-#10	2.59	6721	2500	9710	2133	11724	1780	12982	1484	14553	1012	12790	392	12539
16-#11	3.18	6994	2645	10461	2217	12593	1842	13998	1521	15888	1002	14374	392	14878
16-#14	4.59	7694	2990	12176	2440	14681	2010	16493	1629	19219	994	18185	392	20499
20-#10	3.24	7040	2659	10434	2247	12719	1864	14155	1547	16116	1012	14663	392	15291
20-#11	3.98	7374	2840	11341	2358	13793	1944	15411	1598	17770	1000	16582	392	18211
20-#14	5.74	8235	3271	13436	2644	16385	2156	18502	1739	21894	990	21153	392	24793
24-#11	4.78	7801	3035	12274	2489	14958	2053	16848	1665	19681	1008	18748	392	21276

APPENDIX C

BREATH CALCULATIONS

Cost Analysis

Gravity Column Design Take-Off							
I-Section Size	Linear Footage (in feet)	Bare Cost				Total Incl O&P	Actual Total
		Material	Labor	Equipment	Total		
W 10x33	84.0	34.50	2.02	1.32	37.84	42.50	3,570.00
W 12x40	644.0	42.00	2.08	1.36	45.44	51.00	32,844.00
W 12x45	70.0	47.00	2.11	1.38	50.49	56.50	3,955.00
W 12x50	322.0	52.50	2.11	1.38	55.99	62.50	20,125.00
W 12x53	210.0	55.50	2.11	1.38	58.99	66.00	13,860.00
W 12x58	210.0	61.50	2.11	1.38	64.99	72.50	15,225.00
W 12x65	476.0	68.00	2.16	1.41	71.57	80.00	38,080.00
W 12x72	126.0	75.00	2.16	1.41	78.57	87.75	11,056.50
W 12x79	56.0	82.68	2.21	1.45	86.34	96.50	5,404.00
W 12x87	336.0	91.00	2.21	1.45	94.66	105.00	35,280.00
W 12x96	154.0	100.00	2.21	1.45	103.66	116.00	17,864.00
W 12x106	56.0	110.77	2.27	1.49	114.53	128.00	7,168.00
W 12x120	378.0	125.17	2.27	1.49	128.93	144.00	54,432.00
W 12x136	84.0	142.00	2.33	1.52	145.85	163.00	13,692.00
W 12x152	224.0	159.09	2.33	1.52	162.94	182.00	40,768.00
W 12x170	28.0	177.91	2.39	1.56	181.86	203.50	5,698.00
W 12x190	56.0	199.00	2.39	1.56	202.95	224.00	12,544.00
W 12x210	28.0	219.50	2.39	1.56	223.45	247.00	6,916.00
Total	3542.0						**$338,481.50**

Frame Columns Design Take-Off							
I-Section Size	Linear Footage (in feet)	Bare Cost				Total Incl O&P	Actual Total
		Material	Labor	Equipment	Total		
W 12x50	364.0	52.50	2.11	1.38	55.99	62.50	22,750.00
W 12x58	28.0	61.50	2.11	1.38	64.99	72.50	2,030.00
W 12x72	14.0	75.00	2.16	1.41	78.57	87.75	1,228.50
W 12x79	140.0	82.68	2.21	1.45	86.34	96.50	13,510.00
W 12x87	266.0	91.00	2.21	1.45	94.66	105.00	27,930.00
W 12x96	126.0	100.00	2.21	1.45	103.66	116.00	14,616.00
W 12x106	112.0	110.77	2.27	1.49	114.53	128.00	14,336.00
W 12x120	56.0	125.17	2.27	1.49	128.93	144.00	8,064.00
W 12x136	224.0	142.00	2.33	1.52	145.85	163.00	36,512.00
W 12x152	84.0	159.09	2.33	1.52	162.94	182.00	15,288.00
W 12x170	70.0	177.91	2.39	1.56	181.86	203.50	14,245.00
W 12x190	84.0	199.00	2.39	1.56	202.95	224.00	18,816.00
W 12x210	70.0	219.44	2.45	1.61	223.5	247.00	17,290.00
Total	1638.0						**$206,615.50**

Frame Beams Design Take-Off							
I-Section Size	Linear Footage (in feet)	Bare Cost				Total Incl O&P	Actual Total
		Material	Labor	Equipment	Total		
W 18x35	1007.1	36.50	3.28	1.58	41.36	47.50	47,837.25
W 24x55	148.5	57.50	2.84	1.37	61.71	69.50	10,320.75
W 30x99	322.5	103.00	2.63	1.26	106.89	120.00	38,700.00
Total	1478.1						**$96,858.00**

APPENDIX C

Frame Braces Design Take-Off

Tube Size	Linear Footage (in feet)	Bare Cost				Total Incl O&P	Actual Total
		Material	Labor	Equipment	Total		
HSS 6x6x.5	201.9	21.42	2.58	1.74	27.00	32.00	6,460.80
HSS 7x7x.5	689.9	29.03	2.63	1.76	35.89	42.50	29,320.75
HSS 8x8x.5	558.9	36.64	2.71	1.78	44.78	51.25	28,643.63
HSS 9x9x.5	85	50.35	2.78	1.8	54.93	64.00	5,440.00
HSS 10x10x.5	778.9	64.06	2.84	1.84	68.74	77.00	59,975.30
HSS 12x12x.5	453.2	77.77	2.89	1.87	82.53	93.00	42,147.60
Total	2767.8						**$171,988.08**

Total Structure Gravity Beam Takeoff

I-Section Size	Linear Footage (in feet)	Bare Cost				Total Incl O&P	Actual Total
		Material	Labor	Equipment	Total		
W 8x10	46.3	10.45	3.63	2.38	16.46	20.50	949.15
W 8x18	25.7	18.85	3.63	2.38	24.86	31.00	795.77
W 10x12	787.6	12.55	3.63	2.38	18.56	22.50	17,720.78
W 10x22	96.0	23.00	3.63	2.38	29.01	34.50	3,312.00
W 12x16	3365.2	16.50	2.48	1.62	20.60	24.00	80,763.84
W 12x26	460.7	27.00	2.48	1.62	31.10	36.00	16,584.12
W 12x58	207.5	60.75	3.26	2.17	66.18	73.50	15,254.19
W 14x22	1461.5	23.00	2.2	1.44	26.64	32.00	46,769.28
W 16x26	10984.2	27.00	2.18	1.43	30.61	35.50	389,940.52
W 16x31	115.3	32.50	2.42	1.59	36.51	41.50	4,785.78
W 16x36	1242.0	37.50	2.52	1.68	41.70	50.00	62,100.00
W 16x67	58.0	70.00	2.83	1.58	74.41	84.00	4,872.00
W 18x35	2047.2	36.50	3.28	1.58	41.36	47.50	97,242.00
W 18x40	685.3	42.00	3.28	1.58	46.86	53.50	36,665.69
W 21x50	119.4	52.50	2.82	1.46	56.78	64.50	7,699.37
W 24x55	2405.0	57.50	2.84	1.37	61.71	70.00	168,346.50
W 24x62	3239.7	65.00	2.84	1.37	69.21	78.00	252,694.26
W 24x68	645.0	71.00	2.84	1.37	75.21	84.50	54,502.50
W 24x76	495.0	79.50	2.84	1.37	83.71	94.00	46,530.00
W 24x84	1050.0	88.00	2.92	1.40	92.32	103.00	108,150.00
W 24x94	1140.0	98.00	2.92	1.40	102.32	115.00	131,100.00
W 24x103	60.0	107.00	2.92	1.40	111.32	127.00	7,620.00
W 27x94	420.2	98.00	2.65	1.27	101.92	114.00	47,899.38
W 30x90	908.6	94.00	2.63	1.26	97.89	112.50	102,213.00
W 30x99	240.0	103.00	2.63	1.26	106.89	120.00	28,794.00
W 30x108	333.1	113.00	2.63	1.32	116.95	130.00	43,305.60
W 30x118	911.3	123.00	2.72	1.31	127.03	141.00	128,497.53
Total	33549.7						**1,905,107.25**

Total Structure Shear Studs

Level	Shear Studs	# of studs (To the nearest 100)	Bare Cost				Total Incl O&P	Actual Total
			Material	Labor	Equipment	Total		
Roof	3/4 dia. - 3" long	2238	0.41	0.68	0.28	1.37	2.000	4,476.00
9th Floor	3/4 dia. - 4" long	2511	0.50	0.70	0.29	1.49	2.080	5,222.88
8th Floor	3/4 dia. - 4" long	2511	0.50	0.70	0.29	1.49	2.080	5,222.88
7th Floor	3/4 dia. - 4" long	2511	0.50	0.70	0.29	1.49	2.080	5,222.88
6th Floor	3/4 dia. - 4" long	2511	0.50	0.70	0.29	1.49	2.080	5,222.88
5th Floor	3/4 dia. - 4" long	2511	0.50	0.70	0.29	1.49	2.080	5,222.88
4th Floor	3/4 dia. - 4" long	2511	0.50	0.70	0.29	1.49	2.080	5,222.88
3rd Floor	3/4 dia. - 4" long	2511	0.50	0.70	0.29	1.49	2.080	5,222.88
2nd Floor	3/4 dia. - 4" long	2511	0.50	0.70	0.29	1.49	2.080	5,222.88
	Total	22326						**$46,259.04**

Total Structure Metal Decking							
Level	Area (square feet)	Bare Cost				Total Incl O&P	Actual Total
		Material	Labor	Equipment	Total		
Roof	19433	5.11	0.91	0.06	6.08	6.76	131,367.08
9th Floor	19433	5.11	0.91	0.06	6.08	6.76	131,367.08
8th Floor	19433	5.11	0.91	0.06	6.08	6.76	131,367.08
7th Floor	19433	5.11	0.91	0.06	6.08	6.76	131,367.08
6th Floor	19433	5.11	0.91	0.06	6.08	6.76	131,367.08
5th Floor	19433	5.11	0.91	0.06	6.08	6.76	131,367.08
4th Floor	19433	5.11	0.91	0.06	6.08	6.76	131,367.08
3rd Floor	19433	5.11	0.91	0.06	6.08	6.76	131,367.08
2nd Floor	21315	5.11	0.91	0.06	6.08	6.76	144,089.40
Total	176779						**$1,195,026.04**

Addition Welded Wire Fabric (6x6-W2.9 x W2.9)							
Level	Area ** (square feet)	Bare Cost				Total Incl O&P	Actual Total
		Material	Labor	Equipment	Total		
Roof	21,376	17.15	22.00	0	39.15	55.00	11,756.97
9th Floor	21,376	17.15	22.00	0	39.15	55.00	11,756.97
8th Floor	21,376	17.15	22.00	0	39.15	55.00	11,756.97
7th Floor	21,376	17.15	22.00	0	39.15	55.00	11,756.97
6th Floor	21,376	17.15	22.00	0	39.15	55.00	11,756.97
Total	106,882						**$58,784.83**

**** 10% Adjustment Added for Overlapping**

Addition Slab on Metal Deck							
Level	Area ** (square feet)	Bare Cost				Total Incl O&P	Actual Total
		Material	Labor	Equipment	Total		
Roof	21,376	1.32	0.67	0.27	2.26	2.83	60,494.93
9th Floor	21,376	1.32	0.67	0.27	2.26	2.83	60,494.93
8th Floor	21,376	1.32	0.67	0.27	2.26	2.83	60,494.93
7th Floor	21,376	1.32	0.67	0.27	2.26	2.83	60,494.93
6th Floor	21,376	1.32	0.67	0.27	2.26	2.83	60,494.93
Total	106,882						**$302,474.65**

**** 7% Adjustment Added for Spillage and Shrinkage**

Addition Concrete Slab Edge Formwork							
Level	Perimeter (linear feet)	Bare Cost				Total Incl O&P	Actual Total
		Material	Labor	Equipment	Total		
Roof	626	14.60	11.90	0	26.50	36.00	22,536.00
9th Floor	626	14.60	11.90	0	26.50	36.00	22,536.00
8th Floor	626	14.60	11.90	0	26.50	36.00	22,536.00
7th Floor	626	14.60	11.90	0	26.50	36.00	22,536.00
6th Floor	626	14.60	11.90	0	26.50	36.00	22,536.00
Total	3,130						**$112,680.00**

Time Schedule

ADDITION	
Concrete	
Placing Slab Reinforcement	
6th Floor	4 days
7th Floor	4 days
8th Floor	4 days
9th Floor	4 days
Roof	4 days
	20 days
Placing Slab Edge	
6th Floor	4 days
7th Floor	4 days
8th Floor	4 days
9th Floor	4 days
Roof	4 days
	20 days
Pouring Slab on Metal Deck	
6th Floor	4 days
7th Floor	4 days
8th Floor	4 days
9th Floor	4 days
Roof	4 days
	20 days
Concrete Total	**60 days**
Structural Steel	
Steel Column Erection	
6th - 8th Floor	6 days
9th - Roof	4 days
	10 days
Steel Floor Frame Erection	
6th Floor	4 days
7th Floor	4 days
8th Floor	4 days
9th Floor	4 days
Roof	4 days
	20 days
Install Metal Deck	
6th Floor	5 days
7th Floor	5 days
8th Floor	5 days
9th Floor	5 days
Roof	5 days
	25 days
Install Shear Studs	
6th Floor	3 days
7th Floor	3 days
8th Floor	3 days
9th Floor	3 days
Roof	3 days
	15 days
Structural Steel Total	**70 days**

PHASE ONE	
Concrete	
Slab on Metal Deck (2nd Floor)	8 days
Slab on Metal Deck (3rd Floor)	8 days
Slab on Metal Deck (4th Floor)	8 days
Slab on Metal Deck (Roof)	8 days
	32 days
Structural Steel	
Steel Erection	45 days
Install Metal Deck	15 days
	60 days

Gravity Column Design Take-Off				
I-Section Size	Linear Footage (in feet)	Labor Hours		
		(L.F per day)	Crew	Days
W 10x33	84.0	1032	E-2	0.081
W 12x40	644.0	1032	E-2	0.624
W 12x45	70.0	1032	E-2	0.068
W 12x50	322.0	1032	E-2	0.312
W 12x53	210.0	1032	E-2	0.203
W 12x58	210.0	1032	E-2	0.203
W 12x65	476.0	984	E-2	0.484
W 12x72	126.0	984	E-2	0.128
W 12x79	56.0	984	E-2	0.057
W 12x87	336.0	984	E-2	0.341
W 12x96	154.0	984	E-2	0.157
W 12x106	56.0	960	E-2	0.058
W 12x120	378.0	960	E-2	0.394
W 12x136	84.0	960	E-2	0.088
W 12x152	224.0	912	E-2	0.246
W 12x170	28.0	912	E-2	0.031
W 12x190	56.0	912	E-2	0.061
W 12x210	28.0	912	E-2	0.031
Total	3542.0			**3.567**

Frame Column Design Take-Off				
I-Section Size	Linear Footage (in feet)	Labor Hours		
		(L.F per day)	Crew	Days
W 12x50	364.0	1032	E-2	0.353
W 12x58	28.0	1032	E-2	0.027
W 12x72	14.0	984	E-2	0.014
W 12x79	140.0	984	E-2	0.142
W 12x87	266.0	984	E-2	0.270
W 12x96	126.0	984	E-2	0.128
W 12x106	112.0	960	E-2	0.117
W 12x120	56.0	960	E-2	0.058
W 12x136	224.0	960	E-2	0.233
W 12x152	84.0	912	E-2	0.092
W 12x170	70.0	912	E-2	0.077
W 12x190	84.0	912	E-2	0.092
W 12x210	70.0	912	E-2	0.077
Total	1638.0			**1.681**

Frame Braces Design Take-Off				
Tube Size	Linear Footage (in feet)	Labor Hours		
		(L.F per day)	Crew	Days
HSS 6x6x.5	201.9	648	E-2	0.312
HSS 7x7x.5	689.9	624	E-2	1.106
HSS 8x8x.5	558.9	600	E-2	0.932
HSS 9x9x.5	85	576	E-2	0.148
HSS 10x10x.5	778.9	576	E-2	1.352
HSS 12x12x.5	453.2	552	E-2	0.821
Total	2767.8			**4.670**

Total Time	9.917
Total Schedule Time Alloted for Column/Brace Erection	**10 days**

Typical Floor Gravity Beam Takeoff				
I-Section Size	Linear Footage (in feet)	Labor Hours		
		(L.F. per day)	Crew	Days
W 10x12	93.8	600	E-2	0.156
W 10x22	12.0	600	E-2	0.020
W 12x16	310.5	880	E-2	0.353
W 12x26	65.8	880	E-2	0.075
W 12x58	29.7	750	E-2	0.040
W 14x22	148.0	990	E-2	0.149
W 16x26	1170.5	1000	E-2	1.170
W 16x36	148.0	900	E-2	0.164
W 18x35	215.9	960	E-2	0.225
W 18x40	73.5	960	E-2	0.077
W 24x55	225.0	1110	E-2	0.203
W 24x62	430.6	1110	E-2	0.388
W 24x68	75.0	1110	E-2	0.068
W 24x76	45.0	1110	E-2	0.041
W 24x84	150.0	1080	E-2	0.139
W 24x94	120.0	1080	E-2	0.111
W 27x84	25.0	1190	E-2	0.021
W 30x90	120.0	1200	E-2	0.100
W 30x99	34.3	1200	E-2	0.029
W 30x108	37.0	1200	E-2	0.031
W 30x118	125.2	1176	E-2	0.106
Total	3654.6			**3.665**

Roof Gravity Beam Takeoff				
I-Section Size	Linear Footage (in feet)	Labor Hours		
		(L.F. per day)	Crew	Days
W 10x12	82.2	600	E-2	0.137
W 12x14	163.3	600	E-2	0.272
W 12x16	160.6	880	E-2	0.182
W 14x22	140.1	990	E-2	0.142
W 16x26	1408.1	1000	E-2	1.408
W 18x35	254.0	1000	E-2	0.254
W 21x50	89.4	960	E-2	0.093
W 24x55	745.6	1110	E-2	0.672
W 24x62	115.0	1110	E-2	0.104
W 24x76	45.0	1110	E-2	0.041
W 24x84	90.0	1080	E-2	0.083
W 24x94	120.0	1200	E-2	0.100
W 27x84	90.0	1190	E-2	0.076
W 30x90	34.2	1200	E-2	0.029
W 30x108	37.0	1200	E-2	0.031
W 30x118	35.2	1176	E-2	0.030
Total	3609.5			**3.652**

Typical Floor Metal Decking				
Level	Area (square feet)	Labor Hours		
		(S.F. per day)	Crew	Days
Roof	19433	3900	E-4	4.98
9th Floor	19433	3900	E-4	4.98
8th Floor	19433	3900	E-4	4.98
7th Floor	19433	3900	E-4	4.98
6th Floor	19433	3900	E-4	4.98
Total	97165			**24.91**

Addition's Shear Studs				
Levels	# of studs	Labor Hours		
		(Studs per day)	Crew	Days
Roof	2238	1030	E-10	2.173
9th Floor	2511	1030	E-10	2.438
8th Floor	2511	1030	E-10	2.438
7th Floor	2511	1030	E-10	2.438
6th Floor	2511	1030	E-10	2.438
Total	12282			**11.924**

Welded Wire Fabric (6x6-W2.9 x W2.9)				
Level	Area	Labor Hours		
	(square feet)	(S.F. per day)	Crew	Days
Roof	19433	5800	4 Rodm	3.35
9th Floor	19433	5800	4 Rodm	3.35
8th Floor	19433	5800	4 Rodm	3.35
7th Floor	19433	5800	4 Rodm	3.35
6th Floor	19433	5800	4 Rodm	3.35
Total	97165			**16.75**

Slab on Metal Decking				
Level	Area	Labor Hours		
	(square feet)	(S.F. per day)	Crew	Days
Roof	19433	5226	(2) C-8	3.72
9th Floor	19433	5226	(2) C-8	3.72
8th Floor	19433	5226	(2) C-8	3.72
7th Floor	19433	5226	(2) C-8	3.72
6th Floor	19433	5226	(2) C-8	3.72
Total	97165			**18.59**

Slab Edge Formwork				
Level	Length	Labor Hours		
	(linear feet)	(L.F. per day)	Crew	Days
Roof	626	180	(2) C-1	3.48
9th Floor	626	180	(2) C-1	3.48
8th Floor	626	180	(2) C-1	3.48
7th Floor	626	180	(2) C-1	3.48
6th Floor	626	180	(2) C-1	3.48
Total	3130			**17.39**

Louver Calculations

Air Handling Units on 5th Floor		
Unit	Handles	Dimension
RAHU-1	3rd Floor	12' x 27'
RAHU-2	Ambulatory Surgery	12' x 27'
RAHU-3	Ambulatory Surgery	12' x 27'
RAHU-4	6th Floor	12' x 27'
RAHU-5	7th Floor	12' x 27'

TABLE E-1

ASHRAE Standard 62.1 (Ventilation for Acceptable Indoor Air Quality)		
Application	Max Occupancy Density	Outdoor Air Requirement
	#/1000 ft^2	cmf/person
Medical Procedure	20	15
Operating Rooms	20	30

Each Floor Area is Approximately **20,000 square feet**
Five Air Handling Units located on the 5th Floor (See Above)
- 3rd, 6th, and 7th Floors - Medical Procedure Floors
- 4th Floor - Operating Room (2 units)

Required CFM Calculations

Medical Procedure Floors

= (20 people/1000 ft^2)(20,000 ft^2) = 400 people
= (400 people)(15 cfm/person) = 6000 cfm
= (6000 cfm per floor)(3 floors) = **18,000 cfm**

Operating Room Floor

= (20 people/1000 ft^2)(20,000 ft^2) = 400 people
= (400 people)(30 cfm/person) = **12000 cfm**

Total Required cfm = 30,000 cfm

Convert Values to Area of Louver needed (ft^2)

Wind Velocity = 4.9 mph ----> convert to ft/min = **431.2 ft/min**
cfm/(ft/min) = ft^2 -----> gives area
= (30,000 cfm)/(431.2 ft/min) = **69.57 ft^2**
- Multiply Area by 1.43, assume that louver only provides 70% free area
(1.43)*(69.57 ft^2) = **100 ft^2** per wall

- Also take into account louver size needed for maintanance/ repair
- Increase louver size to 15' x 10' , therefore **150 ft^2** per wall

RETScreen® Energy Model - Wind Energy Project

Units: Imperial

Site Conditions		Estimate	Notes/Range
Project name		MSK	
Project location		North Jersey	
Wind data source		Wind speed	
Nearest location		New York City, NY	
Annual wind power density	W/m²	362	
Height of wind power density	ft	50.0	
Annual average wind speed	mph	5.4	
Height of wind measurement	ft	60.0	3.0 to 100.0 m
Wind shear exponent	-	0.16	0.10 to 0.40
Wind speed	mph	4.9	
Average atmospheric pressure	psi	101.6	60.0 to 103.0 kPa
Annual average temperature	°F	12	-20 to 30 °C

Required Transmition Loss for 6th Floor Private Offices

Sound Absorption coeffiecents for source and reciever rooms								
Frequency	**Mechanical Room**				**Private Offices**			**Source**
Hz	Walls (α)	Floor (α)	Ceiling (α)	Louver (α)	Walls (α)	Floor (α)	Ceiling (α)	Lw
125	0.36	0.01	0.01	1.00	0.55	0.02	0.76	89
250	0.44	0.01	0.01	1.00	0.14	0.06	0.93	88
500	0.31	0.02	0.02	1.00	0.08	0.14	0.83	89
1000	0.29	0.02	0.02	1.00	0.04	0.37	0.99	86
2000	0.39	0.02	0.02	1.00	0.05	0.60	0.99	82
4000	0.25	0.02	0.02	1.00	0.11	0.65	0.94	77

Frequency											
Hz	**α sab (avg)**	**Sα**	**RTs**	**α sab (avg)**	**Sα**	**RTr**	**RC-30 Lp**	**Source Lp**	**NR**	**TL**	**Adj TL**
125	0.0817	362.56	394.81	0.4762	57.434	109.66	**45**	69.04	24	18.1	23.08
250	0.0954	423.52	468.20	0.3037	36.622	52.59	**40**	67.30	27	24.5	29.53
500	0.0813	360.70	392.61	0.2667	32.166	43.87	**35**	69.06	34	32.1	37.08
1000	0.0778	345.46	374.62	0.3351	40.408	60.77	**30**	66.26	36	32.9	37.87
2000	0.0950	421.66	465.93	0.3935	47.452	78.23	**25**	61.32	36	31.8	36.82
4000	0.0710	314.98	339.04	0.4258	51.352	89.43	**20**	57.70	38	32.6	37.62

Source				**Receiver**			
A (walls)	A (floor)	A (ceiling)	A (louver)	A (walls)	A (floor)	A (ceiling)	A (partition)
762	1812	1812	52	65	27.8	27.8	27.8

Mechanical Room
Floor: Concrete
Ceiling: Concrete
Walls: Coarse Concrete Block

Operating Room
Floor: Heavy Carpet
Ceiling: 3/4" thick acoustical board
Walls: Gypsum board

Transmission Loss from Partition			
(4.5" Reinforced Concrete Slab)			
Frequency	TL (dB)	Rq'd TL	Addition TL needed?
125 Hz	48	23.08	**NO**
250 Hz	42	29.53	**NO**
500 Hz	45	37.08	**NO**
1000 Hz	56	37.87	**NO**
2000 Hz	57	36.82	**NO**
4000 Hz	66	37.62	**NO**

APPENDIX C

Required Transmition Loss for 4th Floor Operating Rooms

Sound Absorption coeffiecents for source and reciever rooms								
Frequency	Mechanical Room				O.R.			Source
Hz	Walls (α)	Floor (α)	Ceiling (α)	Louver (α)	Walls (α)	Floor (α)	Ceiling (α)	Lw
125	0.36	0.01	0.01	1.00	0.55	0.02	0.76	89
250	0.44	0.01	0.01	1.00	0.14	0.03	0.93	88
500	0.31	0.02	0.02	1.00	0.08	0.03	0.83	89
1000	0.29	0.02	0.02	1.00	0.04	0.03	0.99	86
2000	0.39	0.02	0.02	1.00	0.05	0.03	0.99	82
4000	0.25	0.02	0.02	1.00	0.11	0.02	0.94	77

Frequency											
Hz	α sab (avg)	Sα	RTs	α sab (avg)	Sα	RTr	RC-25 Lp	Source Lp	NR	TL	Adj TL
125	0.0817	362.56	394.81	0.4614	77.52	143.94	**40**	69.04	29	24.1	29.13
250	0.0954	423.52	468.20	0.3282	55.14	82.08	**35**	67.30	32	29.8	34.83
500	0.0813	360.70	392.61	0.2738	45.99	63.33	**30**	69.06	39	37.7	42.72
1000	0.0778	345.46	374.62	0.3002	50.43	72.06	**25**	66.26	41	39.4	44.36
2000	0.0950	421.66	465.93	0.3046	51.18	73.60	**20**	61.32	41	39.3	44.32
4000	0.0710	314.98	339.04	0.3148	52.89	77.19	**15**	57.70	43	40.5	45.50

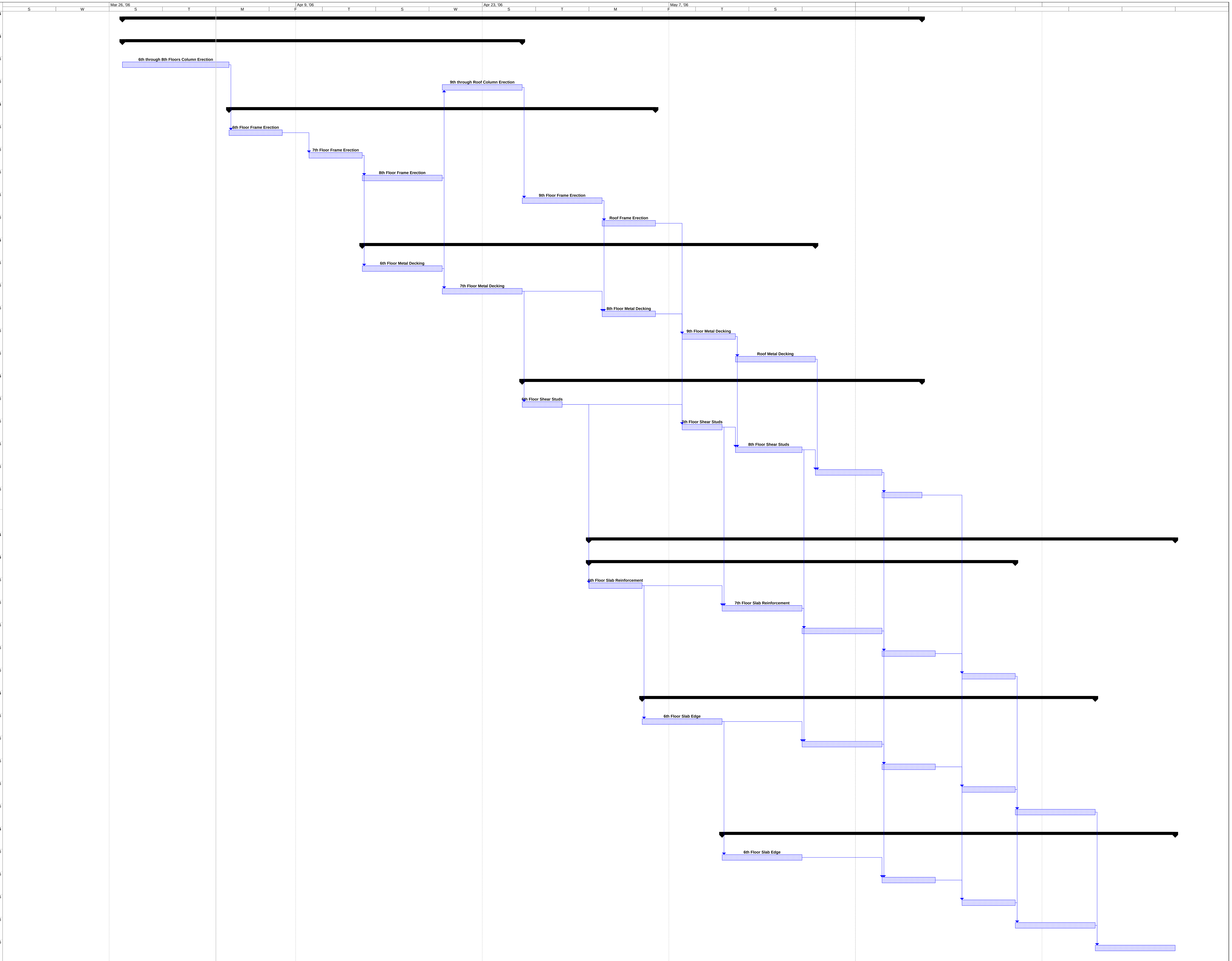
Source				Receiver			
A (walls)	A (floor)	A (ceiling)	A (louver)	A (walls)	A (floor)	A (ceiling)	A (partition)
762	1812	1812	52	75	46.5	46.5	46.5

Mechanical Room
Floor: Concrete
Ceiling: Concrete
Walls: Coarse Concrete Block

Operating Room
Floor: Linoleum
Ceiling: 3/4" thick acoustical board
Walls: Gypsum board

Transmission Loss from Partition			
(4.5" Reinforced Concrete Slab)			
Frequency	TL (dB)	Rq'd TL	Addition TL needed?
125 Hz	48	29.13	**NO**
250 Hz	42	34.83	**NO**
500 Hz	45	42.72	**NO**
1000 Hz	56	44.36	**NO**
2000 Hz	57	44.32	**NO**
4000 Hz	66	45.50	**NO**

ID		Task Name	Duration	Start	Finish
1		**Stuctural Steel**	**44 days**	**Mon 3/27/06**	**Thu 5/25/06**
2		**Steel Column Erection**	**22 days**	**Mon 3/27/06**	**Tue 4/25/06**
3		6th through 8th Floors Column Erection	6 days	Mon 3/27/06	Mon 4/3/06
4		9th through Roof Column Erection	4 days	Thu 4/20/06	Tue 4/25/06
5		**Steel Floor Frame Erection**	**24 days**	**Tue 4/4/06**	**Fri 5/5/06**
6		6th Floor Frame Erection	4 days	Tue 4/4/06	Fri 4/7/06
7		7th Floor Frame Erection	4 days	Mon 4/10/06	Thu 4/13/06
8		8th Floor Frame Erection	4 days	Fri 4/14/06	Wed 4/19/06
9		9th Floor Frame Erection	4 days	Wed 4/26/06	Mon 5/1/06
10		Roof Frame Erection	4 days	Tue 5/2/06	Fri 5/5/06
11		**Metal Deck Installation**	**24 days**	**Fri 4/14/06**	**Wed 5/17/06**
12		6th Floor Metal Decking	4 days	Fri 4/14/06	Wed 4/19/06
13		7th Floor Metal Decking	4 days	Thu 4/20/06	Tue 4/25/06
14		8th Floor Metal Decking	4 days	Tue 5/2/06	Fri 5/5/06
15		9th Floor Metal Decking	4 days	Mon 5/8/06	Thu 5/11/06
16		Roof Metal Decking	4 days	Fri 5/12/06	Wed 5/17/06
17		**Shear Stud Installation**	**22 days**	**Wed 4/26/06**	**Thu 5/25/06**
18		6th Floor Shear Studs	3 days	Wed 4/26/06	Fri 4/28/06
19		7th Floor Shear Studs	3 days	Mon 5/8/06	Wed 5/10/06
20		8th Floor Shear Studs	3 days	Fri 5/12/06	Tue 5/16/06
21		9th Floor Shear Studs	3 days	Thu 5/18/06	Mon 5/22/06
22		Roof Shear Studs	3 days	Tue 5/23/06	Thu 5/25/06
23					
24		**Concrete**	**32 days**	**Mon 5/1/06**	**Tue 6/13/06**
25		**Placing Slab Reinforcement**	**24 days**	**Mon 5/1/06**	**Thu 6/1/06**
26		6th Floor Slab Reinforcement	4 days	Mon 5/1/06	Thu 5/4/06
27		7th Floor Slab Reinforcement	4 days	Thu 5/11/06	Tue 5/16/06
28		8th Floor Slab Reinforcement	4 days	Wed 5/17/06	Mon 5/22/06
29		9th Floor Slab Reinforcement	4 days	Tue 5/23/06	Fri 5/26/06
30		Roof Slab Reinforcement	4 days	Mon 5/29/06	Thu 6/1/06
31		**Placing Slab Edge**	**24 days**	**Fri 5/5/06**	**Wed 6/7/06**
32		6th Floor Slab Edge	4 days	Fri 5/5/06	Wed 5/10/06
33		7th Floor Slab Edge	4 days	Wed 5/17/06	Mon 5/22/06
34		8th Floor Slab Edge	4 days	Tue 5/23/06	Fri 5/26/06
35		9th Floor Slab Edge	4 days	Mon 5/29/06	Thu 6/1/06
36		Roof Slab Edge	4 days	Fri 6/2/06	Wed 6/7/06
37		**Pouring Slab on Metal Deck**	**24 days**	**Thu 5/11/06**	**Tue 6/13/06**
38		6th Floor Slab Edge	4 days	Thu 5/11/06	Tue 5/16/06
39		7th Floor Slab Edge	4 days	Tue 5/23/06	Fri 5/26/06
40		8th Floor Slab Edge	4 days	Mon 5/29/06	Thu 6/1/06
41		9th Floor Slab Edge	4 days	Fri 6/2/06	Wed 6/7/06
42		Roof Slab Edge	4 days	Thu 6/8/06	Tue 6/13/06



Project: Addition Schedule
Date: Mon 4/3/06