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:

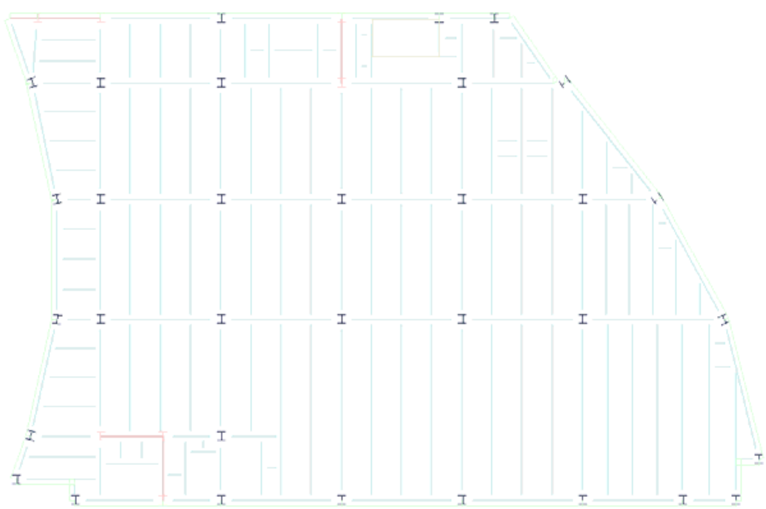
Gravity Loads			
Floor: 2nd - 9th		***Floor:*** 5th	
Dead		Dead	
56 psf	slab on deck	56 psf	slab on deck
2 psf	metal deck	2 psf	metal deck
12 psf	steel framing	12 psf	steel framing
15 psf	superimposed	65 psf	mechanical
85 psf		**135 psf**	
Floor: Roof		Live - **100 psf**	
Dead		*(Table 4-1) ASCE7-02*	
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.

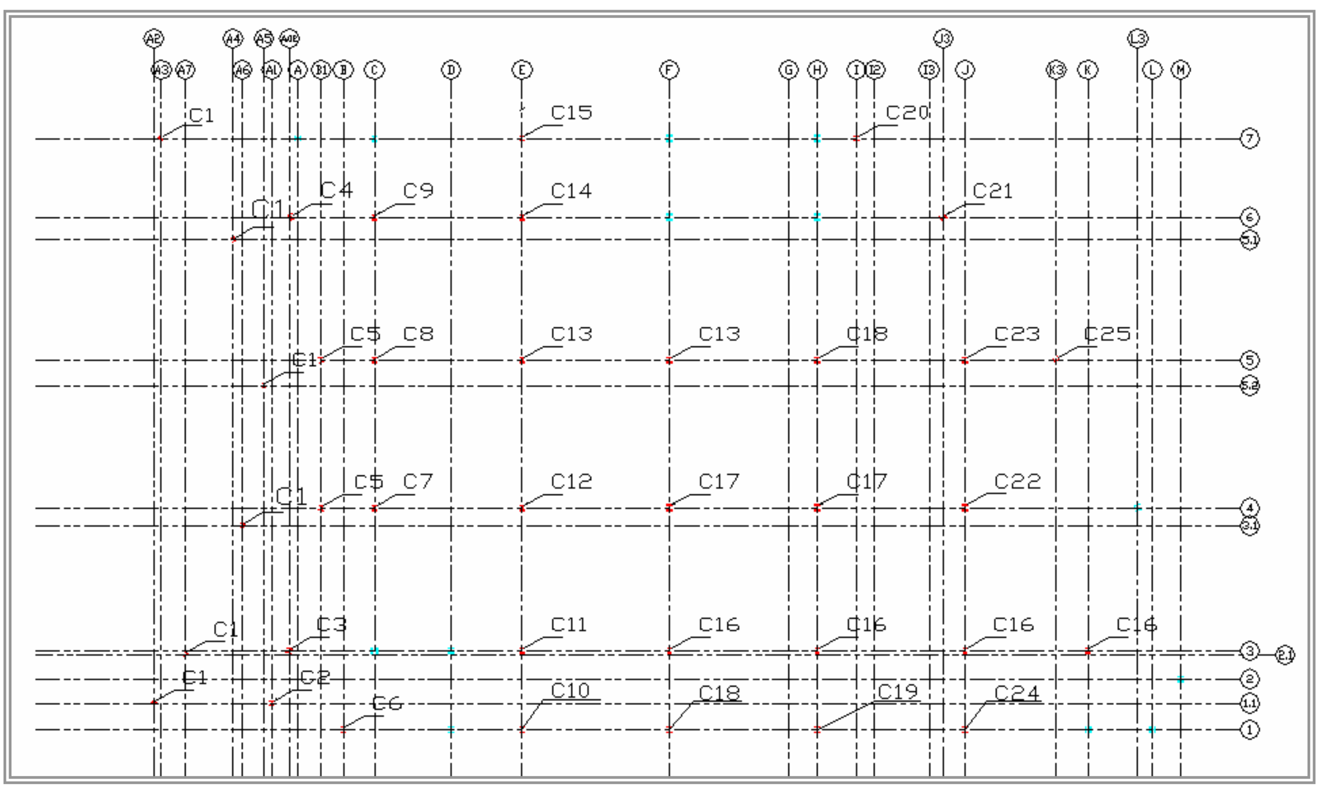


Gravity Column Locations

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.

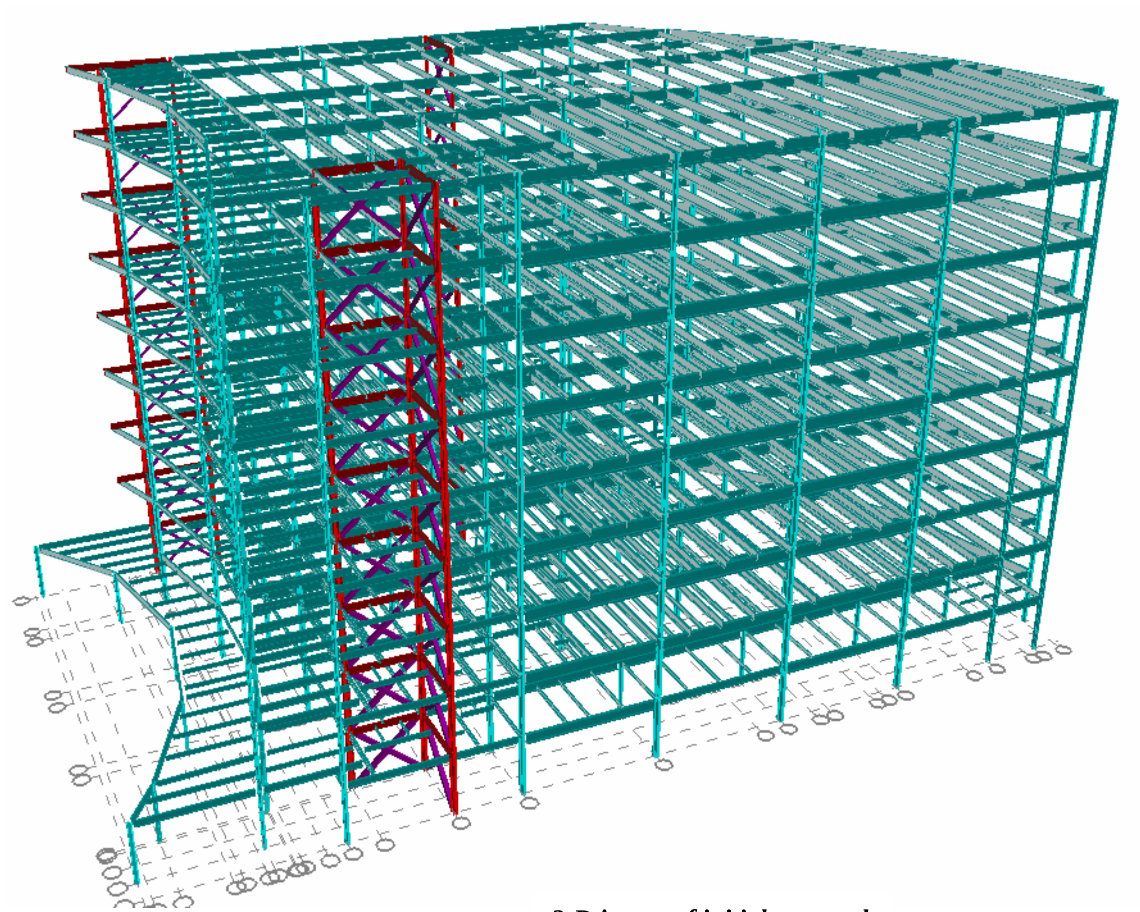
Furthermore, a list of design goals was implemented to ensure that the lateral system was designed under the same conditions of the initial system. The design goals are as follows:

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.

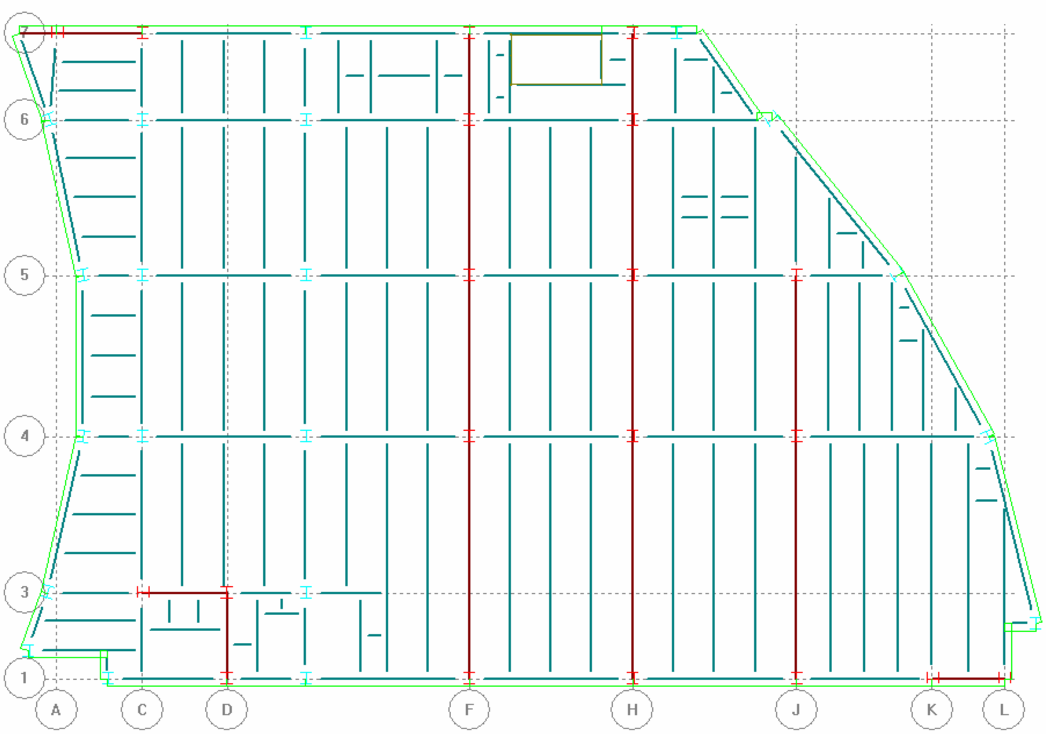
From the initial lateral system design of MSK, it is obvious that a number of additional steps must be taken in order to deem this building structurally sound. For one, both the braces and columns are excessively large for a nine story building. This is due to the fact that there are only two braced 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.

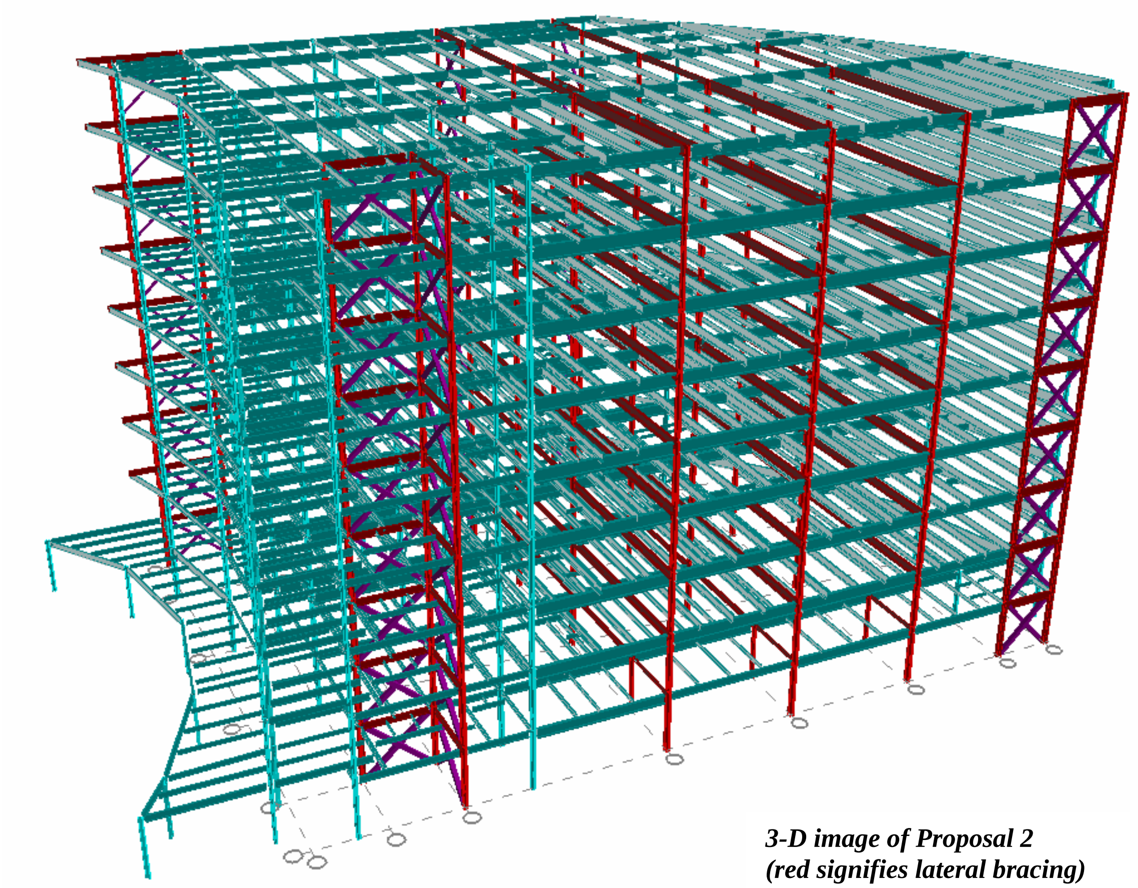
PROPOSED DESIGN 2

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.

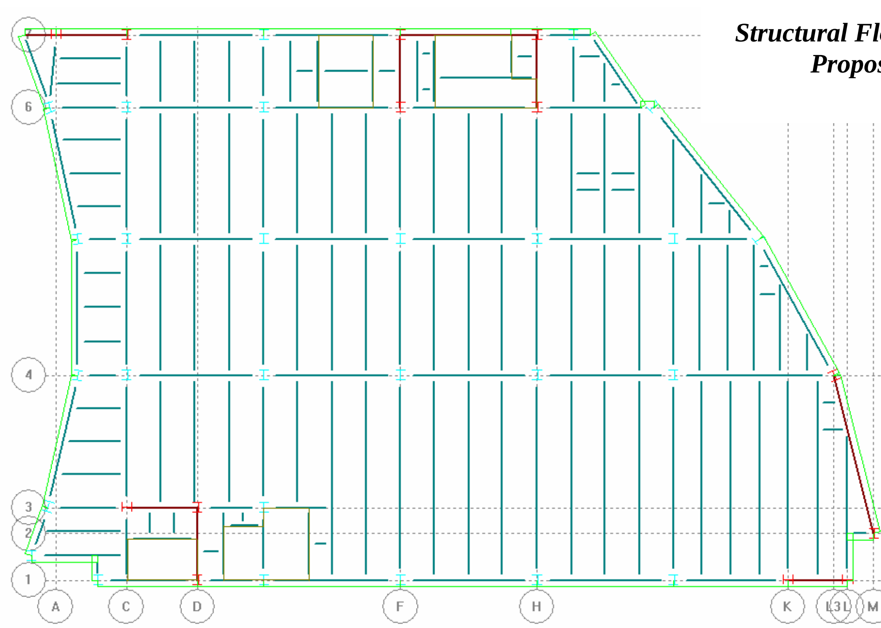
When drift was examined for this proposal, it was still obvious that additional resisting systems were required to subdue the building's displacement below the drift limit. At the center of rigidity, MSK still drifted 5.82" in the east-west direction due to seismic and 4.78" in the north-south because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

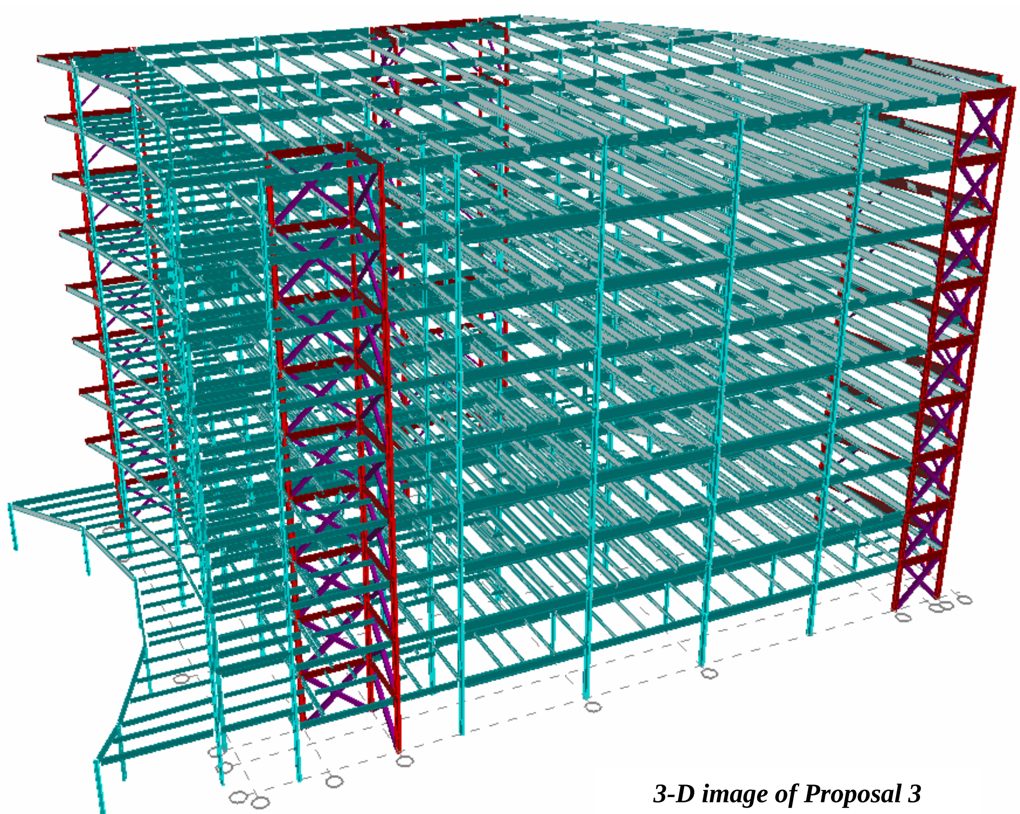
PROPOSED DESIGN 3

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, 2nd 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

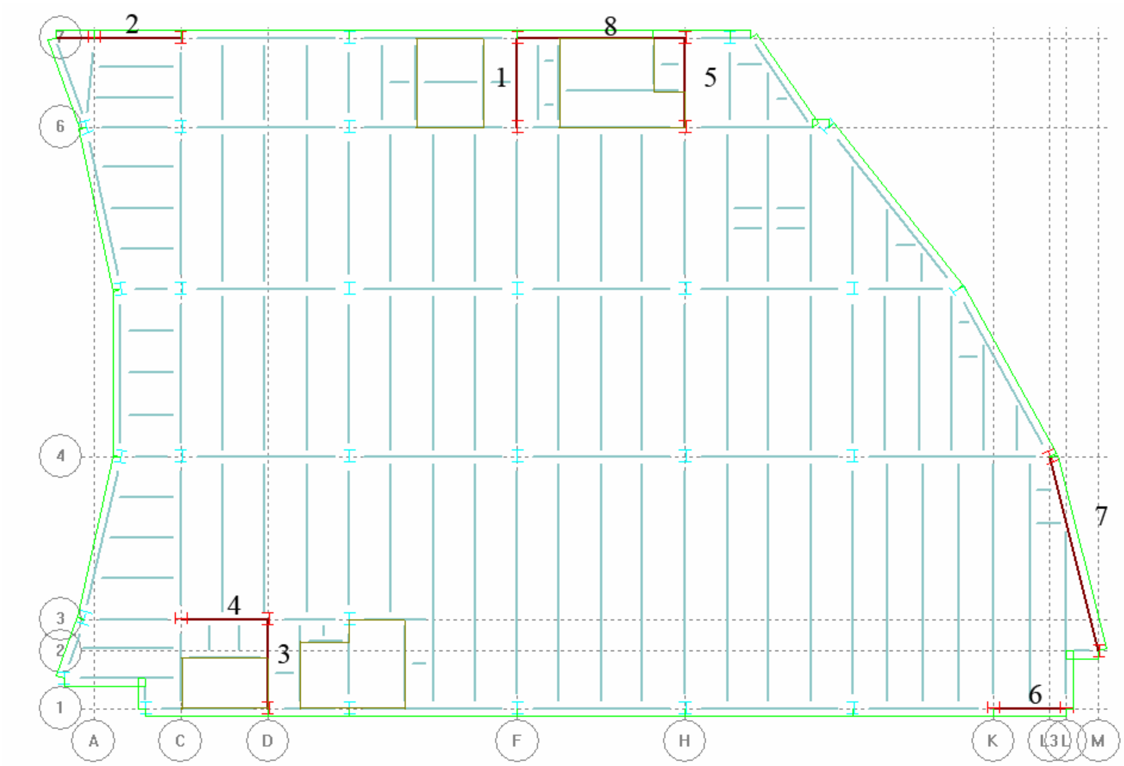
Once the concrete piers were redesigned to withstand the axial forces acting on them, their footings were inspected to see whether they had to be increased in dimension. The geotechnical report stated that the existing structure of Memorial Sloan Kettering was on basalt bedrock and had an allowable bearing 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.