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STRUCTURAL OPTION

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AE 481 W

TECHNICAL REPORT THREE
LATERAL SYSTEM ANALYSIS AND CONFIRMATION DESIGN

INTRODUCTION

This third technical report presents a detailed analysis of the current lateral force resisting system found within Memorial Sloan-Kettering Cancer 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. MSK's infrastructure is made up of braced steel framing supported by a concrete foundation. It is laterally supported by four identical systems composed of diagonal bracing and shear walls. This report goes into great depth analyzing this system against the seismic and wind lateral forces developed from ASCE 7.

The first section of this report provides a structural description of Memorial Sloan-Kettering, focusing mainly on its lateral system. This discussion goes into detail describing the locations, connections, member sizes, and footings for each of the four systems. The next section of this report focuses of lateral load development seen on the building. ASCE 7 was used to calculate the seismic and wind forces for each floor, along with all relevant load cases. From these calculations, it was determined that the seismic forces controlled the lateral loading in both directions.

To help analyze the lateral loading on MSK, a model of the building was created in the ETABS computer program. This program helped calculate the building's total drift, displacement, base shear, and overturning moments. It was also used to verify the accuracy of the seismic and wind loads calculated from ASCE 7. Hand calculations were then performed to ensure the values outputted from ETABS was accurate and reasonable. Shear strength, torsion, and lateral members were also checked through hand calculations to determine whether or not the system is adequately designed.

The report was able to conclude that the lateral force resisting system in MSK is adequately designed to resist the lateral forces seen in northern New Jersey. After verifying and enhancing the seismic and wind load values from Tech 1, it was obvious that seismic forces would control the lateral loading of the building. Because of this, all hand calculations performed to check the lateral system strength used those seismic forces. From those calculations, along with the ETABS model, it was established that Memorial Sloan Kettering is sufficiently designed to resist its lateral forces.

BUILDING DESCRIPTION

Memorial Sloan-Kettering Cancer Center is a four-story health-care facility located in the scenic region of Somerset County, New Jersey. Each story spans 14 feet from floor to floor, and with a two foot parapet located on the roof, the building's total height is approximately 58 feet. When MSK opens its doors in the summer of 2006, it will serve as one of the premiere cancer treatment centers in nation, accommodating offices, exam rooms, chemotherapy bays, radiotherapy treatment, a laboratory, and pharmacy.

The infrastructure of MSK is made up of steel braced-framing supported by a concrete foundation. The structure below grade consists of foundation walls and piers made exclusively of reinforced concrete. The structural steel skeleton in MSK begins at the first floor level and continues for the remainder of the building. Because each steel column sits directly on top of a concrete pier, the typical bay size remains at 30'x 30' throughout the first floor. However, beginning on the second floor, a number of columns near the south end of the building are removed in order to create more of an open floor plan. This causes some bays to span 30' x 45' in the upper level floors. A number of bays are also reduced in size near the exterior walls of the building due to Memorial Sloan Kettering's curved exterior façade.

The steel columns vary in size throughout the building according to their location and purpose. These columns remain constant in size between the first floor and fourth floor. A typical interior column ranges between W12 x 87 and W12 x 96.These steel columns connect into the concrete piers below through ASTM A572, Grade 50 steel base plates. A typical base plate used for these connections is 18"x 18" and 1-½" thick. These plates are kept in place by four ¾" A449 anchor bolts embedded 2' into the concrete column.

The first floor of Memorial Sloan-Kettering is constructed as a one-way concrete slab system that is structurally supported by the foundation walls and concrete columns below. The 6" slab lies on top of concrete beams spanning in the E – W direction and concrete girders spanning in the N – S direction. The second, third, and fourth floors of MSK make use of steel beams and girders to support their floor systems. The second floor is the last floor to maintain the typical bays sizes and because of this, has the most consistent steel sizes throughout the floor. A typical interior beam is a W16 x 26 while a typical interior girder is a W24 x 96. For smaller bays near the exterior walls, beam sizes fall to a W12 x 16. The exterior girder sizes do not show much consistency, ranging anywhere from a W18 x 35 to a W30 x 108. The third floor and fourth floor both have the same layout. Where the interior spans continue from the second floor, the structural design is maintained with W16 x 26 beams connecting into W24 x 96 girders. For those spans which become 30' x 45', beam sizes are W24 x 62 and girders use W30 x 90. The roof of the building follows the same framing as the fourth floor, only with slighting smaller beams and girders that support the 30' x 45' bays.

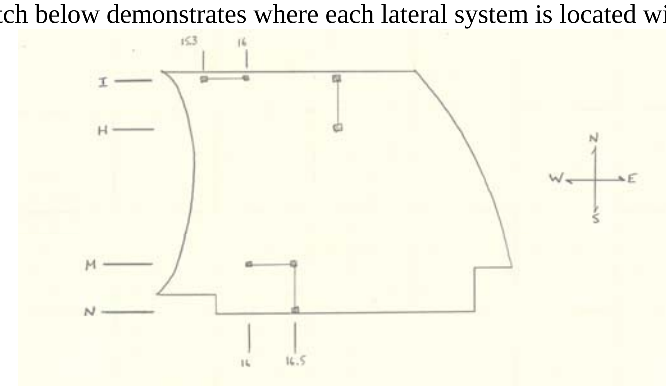
LATERAL SYSTEM DESCRIPTION

The lateral force resisting system of Memorial Sloan Kettering is made up of a combination of shear walls and steel cross-bracing. The four shear walls are located below grade and are all positioned near the exterior walls, typically around stairwells or elevator shafts. This positioning allows for a lateral system which does not protrude into the interior office space of the building. At grade level, these shear walls connect into steel columns through the base plates described earlier. These columns span the remaining four floors to the roof and frame the lateral bracing steel members. Two lateral systems span in the N – S direction and two span in the E – W direction.

The first lateral system oriented in the N – S direction is located on 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" thick base plate. These two column 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 8x8x1/2 members span diagonally through the steel frames and are braced at midspan by a ¾" gusset plate. See diagram X-3 in Appendix D to view this connection. The bracing between the 2nd and 3rd floors also consists of two diagonal HSS 8x8x1/2 members. These braces gradually become smaller, with two HSS 7x7x1/2 steel members between the third and fourth floors. The system culminates with two HSS 6x6x1/2 members between the fourth floor and the roof. Refer to figure X-2 in Appendix D to view the entire system.

The second lateral system oriented in the N – S direction is located on the southwest end of MSK, between column lines M and L. This lateral system is slightly smaller with two HSS 7x7x1/2 diagonal members spanning between the first and second floor, supported beneath by a 12" thick shear wall. The remaining three floors reduce the diagonal member size to two HSS 6x6x1/2's spanning between floors.

The two lateral systems running in the E – W direction follow the same framing as the two systems described above. 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.



Shear Wall Locations

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 (See diagram X-1 in Appendix D). 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 25' 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.

LATERAL LOAD DEVELOPMENT

Seismic Loads

As noted in Technical Report 1, the wind and seismic loads acting on Memorial Sloan-Kettering have been calculated from the methods provided by ASCE 7-02. Each variable used in those methods has been verified to ensure accuracy in the end results. The seismic forces were found using the Equivalent Lateral Force Method, which is outlined in Chapter 9. Like Tech 1, this report develops lateral forces acting in both the N – S and E – W directions. These results however, show higher seismic load values then those developed in the first report (Refer to Appendix C). This is partially due to the gravity loads being refined to replicate the actual loads found in MSK. The most notable load change would be the curtain wall weight being increased to 55 psf to reflect the integrated weight of brick façade and glass windows. Another reason for the increased seismic loads has to do with the incorrect assumptions and variable values used in the Tech 1, described below.

A number of assumptions used in Tech 1 were proven to be inaccurate and have since been modified. The projected floor areas for the top three stories were overestimated by approximately 2000 sq. ft. which resulted in heavier floor weights and larger seismic loads. Also, the Occupancy Importance Factor design parameter was underestimated to be 1.15, when healthcare facilities use a 1.5 value, creating lower seismic forces. These two mistakes appear to have offset each other in Tech 1 because when using the correct variables, the building's base shear increases by only 14 kips. The updated seismic loads are shown below.

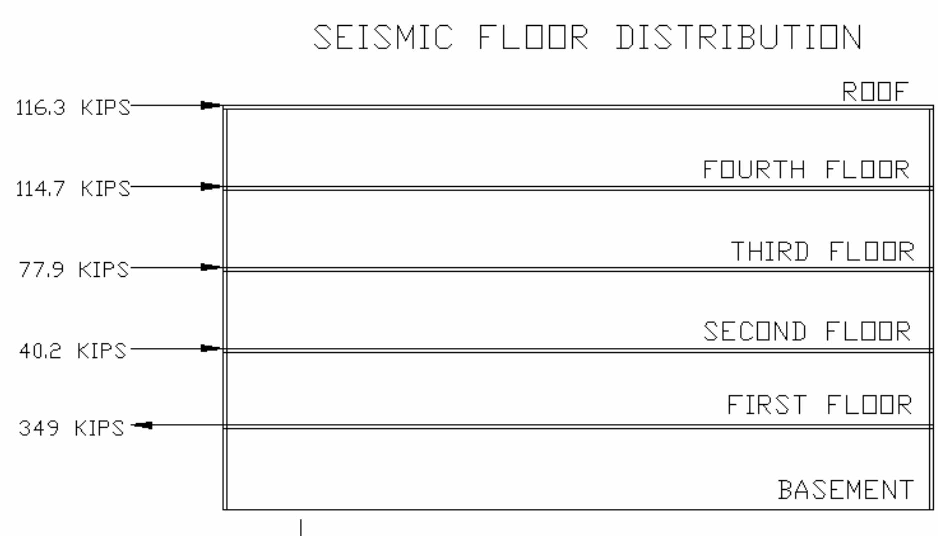
<u>Seismic: Equivalent Lateral Force Method (ASCE 7-02)</u>

Exponent $k_{N\text{-}S}$: 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

Exponent $k_{N\text{-}S}$: 0.954711

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	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



Wind Loads

Wind: Method 2 (ASCE 7-02)

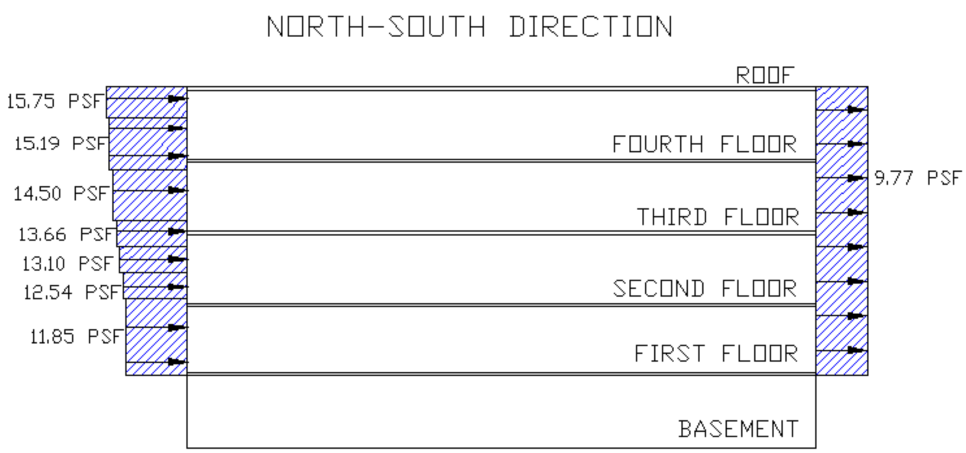
The wind loads developed for this report were found using the Analytical Approach (Method 2), located in Chapter 6 of the design code. Although Memorial Sloan-Kettering is less then 60 feet tall, the Analytical Method was chosen over the Simplified Procedure due to the building's curved exterior walls, which constitute an irregularly shaped building. The results found from the Analytical Approach are almost identical to Tech Report 1's results due to the fact that no variables were required to be changed. Calculations can be found in Appendix B.

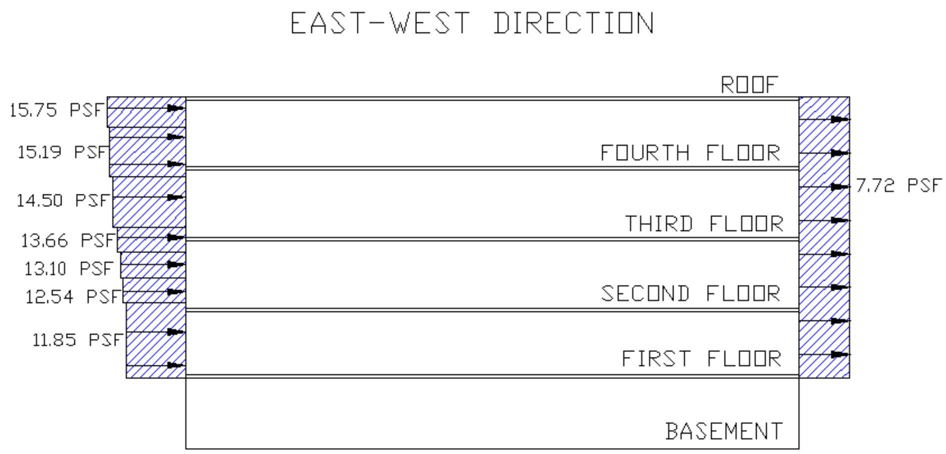
Velocity Pressure Envelope

	Windward		Leeward		Max psf	
Z(ft)	N-S	E-W	N-S	E-W	N-S	E-W
0-15	11.85	11.85	-9.77	-7.72	21.62	19.57
15-20	12.54	12.54	-9.77	-7.72	22.32	20.27
20-25	13.10	13.10	-9.77	-7.72	22.88	20.83
25-30	13.66	13.66	-9.77	-7.72	23.43	21.38
30-40	14.50	14.50	-9.77	-7.72	24.27	22.22
40-50	15.19	15.19	-9.77	-7.72	24.97	22.92
50-60	15.75	15.75	-9.77	-7.72	25.52	23.47
58	15.64	15.64	-9.77	-7.72	25.41	23.36

Shear Summary

(kips)	N-S	E-W
Shear @ Roof	34	20.86
Shear @4	67.6	40.5
Shear @3	64.47	38.43
Shear @2	59.97	35.5
Shear @1	0	0
Base Shear	226.04	135.301
Overturning Moment	7660.30	4601

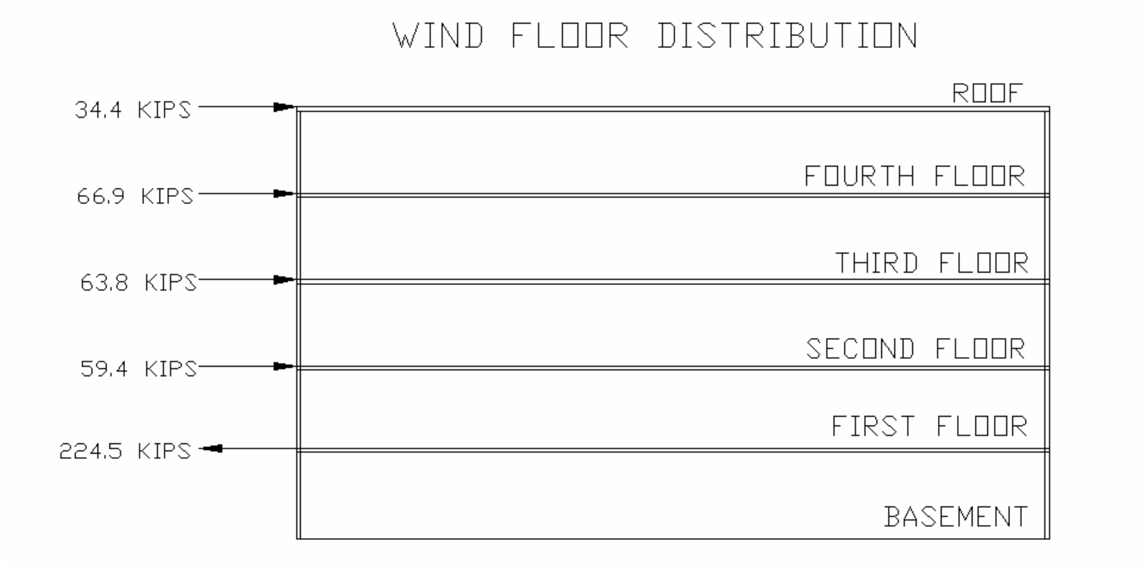




Design Wind Load Cases 1 and 3 were also taken into consideration in order to determine which loading case controlled the design. Load Case 1 looks at 100% of the wind pressure acting on the walls perpendicular to each axis of a building, considering each pressure separately. Load Case III considers 75% of wind pressures acting simultaneously on both axises of a building. From these calculations, it was determined that Case 1 produces the maximum wind loads on Memorial Sloan Kettering.

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	7.25	58.25	34.42	20.76	0	0	2004.94	1209.40
4	14.5	43.50	66.93	40.27	34.42	20.76	2911.33	1751.76
3	14.5	29.00	63.84	38.24	101.35	61.03	1851.23	1109.03
2	14.5	14.50	59.37	35.31	165.18	99.28	860.87	512.05
1	7.50	0.00	0.00	0.00	224.55	134.59	7628.38	4582.24

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	N-S	E-W	N-S	E-W
Roof	7.25	58.25	25.81	15.57	30.15	25.81	15.57	30.15	0	0	1756.10	1756.10
4	14.5	43.50	50.20	30.20	58.58	50.20	30.20	58.58	30.15	30.15	2548.29	2548.29
3	14.5	29.00	47.88	28.68	55.81	47.88	28.68	55.81	88.73	88.73	1618.51	1618.51
2	14.5	14.50	44.53	26.49	51.81	44.53	26.49	51.81	144.54	144.54	751.23	751.23
1	7.50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	196.35	196.35	6674.13	6674.13



CONTROLLING LATERAL FORCE

After analyzing both the wind and seismic forces acting on Memorial Sloan Kettering, it is apparent that the seismic forces control the lateral loads in both directions. When looking at both base shears created by these lateral loads, seismic generates about 349 kips compared to 225 kips due to wind. Three of the four floor loads are also controlled by the seismic values. The roof experiences approximately 116 kips of seismic load compared to just 34 kips from wind. The fourth and third floors also experience high seismic loads of 114 kips and 78 kips, respectively. Only the second floor of MSK has a higher wind (59.4 k) then seismic (40.2 k) load, and that is only for the N – S direction. The second floor's E – W direction only experiences 35 k of wind force and is thus controlled by seismic as well.

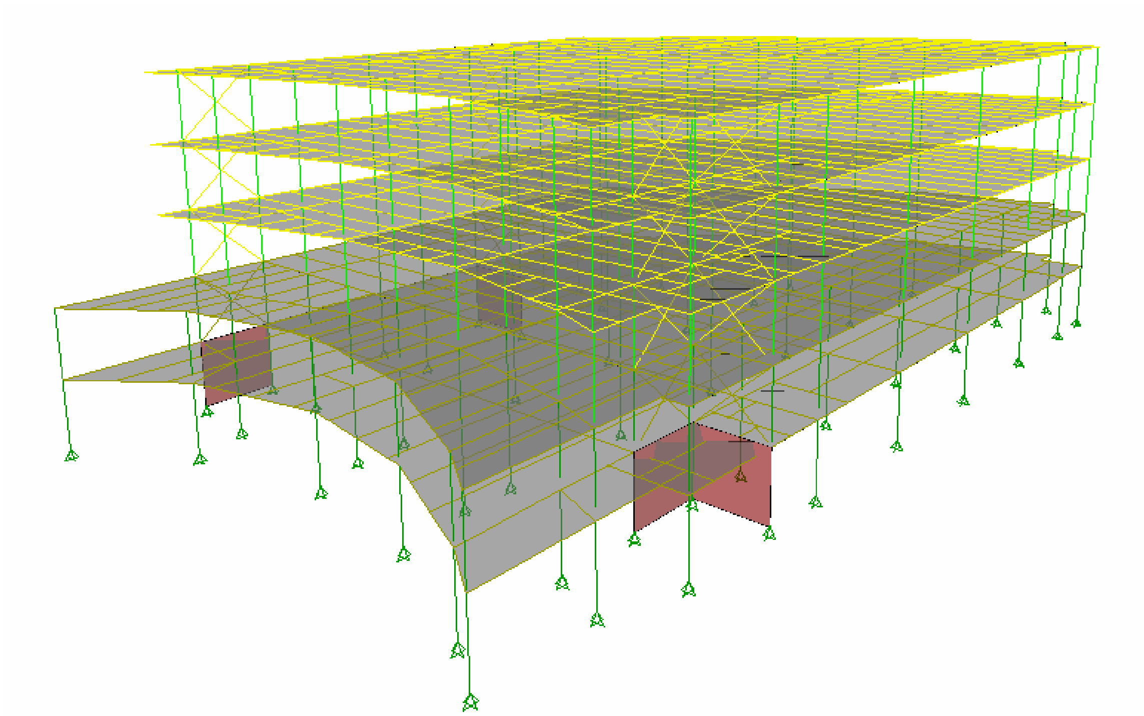
LATERAL LOAD DISTRIBUTION

Due to the fact that Memorial Sloan-Kettering has two identical lateral systems running in both the N – S and E – W directions, one is able to distribute the laterals loads acting on the building equally between the two systems. All four lateral systems are identical and because of this, the same stiffness is assumed. This distribution between systems can be observed though the hand calculations introduced later in the report.

LATERAL LOAD ANALYSIS

For this technical report, a model of Memorial Sloan-Kettering was created in ETABS to assist in the analysis of the lateral loads and how they affect each lateral member in the building. The ETABS program has the ability to both analyze and design buildings based on the parameters set in the program. For this report, the actual infrastructure was put into the program in order to analyze the lateral system under ASCE7 wind conditions and IBC 2000 seismic conditions.

Because there are no fixed moment connections within MSK, all structural steel members for this model were designed as pinned connections along with the lateral bracing members. The midpoint of these cross braces were also modeled as a pin connection due to the gusset plate at that point. Each concrete floor system was assigned rigid diaphragm characteristics in order to transfer the loads into the lateral system. Furthermore, all dead loads acting on MSK were distributed accordingly to ensure the building's correct weight was being used for all seismic calculations. Superimposed dead loads were assigned to all floors along with an additional mechanical dead load added to the roof. Distributed line loads of 55 psf were assigned to all exterior members to mimic the weight applied from the brick façade.



ETABS can also provide verification for all the wind and seismic calculations performed earlier in the report. By arriving at similar values between the hand calculations and computer model values, one can conclude that those values are correct.

Through hand calculations I was able to determine that the base shear obtained from seismic loads was about 349 kips. After analyzing my model in ETABS, the computer program came up with a base shear value of 329 kips. Although the two values are off by about 6%, I can conclude that these numbers are reasonable in relation to each other. Another set of values that was able to be verified through ETABS were the wind loads acting on each floor. Each floor wind load computed through ETABS was within 4% of the value calculated by hand. The seismic loads acting on each floor were also checked against the computer to demonstrate the accuracy of ETABS.

STORY DRIFT

After analyzing Memorial Sloan Kettering, a maximum drift was found to be 2.98 inches due to seismic forces in the N-S direction. After going into ASCE 7, a maximum building drift was found to be .015h for non-masonry structures designed to accommodate story drifts (Table 9.5.2.8). Therefore, the maximum building drift for MSK's roof would be approximately 10.4", which is well above the actual 2.98" drift. Tables calculating the controlling seismic drifts can be viewed below.

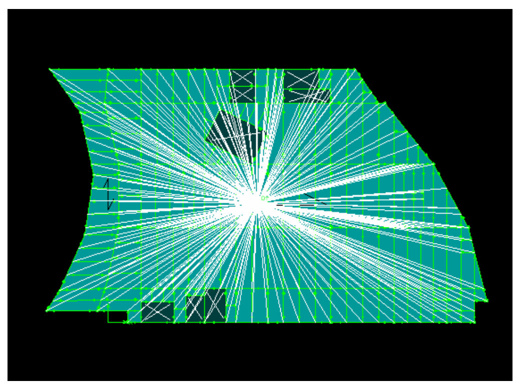
Story	Item	Load	Point	X	Y	Z	DriftX	DriftY	1/DRIFT	Story Height	Story Drift
ROOF	Max Drift X	EQUAKEY	459	2006	0	840	0.001847		541.42	168	0.310
ROOF	Max Drift Y	EQUAKEY	41	2154	123	840		0.005237	190.95	168	0.880
4TH FLOOR	Max Drift X	EQUAKEY	610	442.6	0	672	0.001761		567.86	168	0.296
4TH FLOOR	Max Drift Y	EQUAKEY	41	2154	123	672		0.00513	194.93	168	0.862
3RD FLOOR	Max Drift X	EQUAKEY	610	442.6	0	504	0.001425		701.75	168	0.239
3RD FLOOR	Max Drift Y	EQUAKEY	41	2154	123	504		0.004303	232.40	168	0.723
2ND FLOOR	Max Drift X	EQUAKEY	610	442.6	0	336	0.000862		1160.09	168	0.145
2ND FLOOR	Max Drift Y	EQUAKEY	41	2154	123	336		0.002744	364.43	168	0.461
1ST FLOOR	Max Drift X	EQUAKEY	36	2085	0	168	0.000113		8849.56	168	0.019
1ST FLOOR	Max Drift Y	EQUAKEY	41	2154	123	168		0.000351	2849.00	168	0.059

Total Drift X:	1.009	inches
Total Drift Y:	2.985	inches

Story	Item	Load	Point	X	Y	Z	DriftX	DriftY	1/DRIFT	Story Height	Story Drift
ROOF	Max Drift X	EQUAKEX	713	120.84	1440	840	0.00236		423.7288	168	0.396
ROOF	Max Drift Y	EQUAKEX	707	-77.638	1440	840		0.000561	1782.531	168	0.094
4TH FLOOR	Max Drift X	EQUAKEX	610	442.6	0	672	0.002355		424.6285	168	0.396
4TH FLOOR	Max Drift Y	EQUAKEX	41	2154	123	672		0.000512	1953.125	168	0.086
3RD FLOOR	Max Drift X	EQUAKEX	515	997	1440	504	0.002014		496.5243	168	0.338
3RD FLOOR	Max Drift Y	EQUAKEX	56	-64	64	504		0.000391	2557.545	168	0.066
2ND FLOOR	Max Drift X	EQUAKEX	610	442.6	0	336	0.001352		739.645	168	0.227
2ND FLOOR	Max Drift Y	EQUAKEX	41	2154	123	336		0.000325	3076.923	168	0.055
1ST FLOOR	Max Drift X	EQUAKEX	36	2085	0	168	0.000152		6578.947	168	0.026
1ST FLOOR	Max Drift Y	EQUAKEX	41	2154	123	168		0.000035	28571.43	168	0.006

Total Drift X:	1.383	inches
Total Drift Y:	0.306	inches

FLOOR DISPLACEMENT



The building's overall displacement was also found at each floor's center of rigidity (shown on the right). It was determined that the maximum displacement for all floors of Memorial Sloan-Kettering occur with seismic loading. The maximum displacement in the entire building occurs on the roof with the center of rigidity moving approximately 1.7". A rule of thumb dealing with total building displacement is that infrastructure should not deflect more then L/400. The roof displacement meets that criterion, with a total displacement of L/409. When analyzing just wind loads, the total roof displacement is approximately 1.15", which occurs during Case III loading when wind applies force on the north and west sides of the building. That produces a total displacement of L/605, well above the required value. A table showing the maximum displacements on all floors is shown below.

		N - S	E-W	Force
Max displacements:	Roof	1.3704	1.7045	Earthquake
	4th floor	0.9739	1.1994	Earthquake
	3rd floor	0.5789	0.7125	Earthquake
	2nd floor	0.2422	0.2809	Earthquake
	First floor	0.0253	0.032	Earthquake

OVERTURNING

Overturning is another factor that needed to be addressed when analyzing the building's lateral system. An overturning moment of 14,075 foot-kips is produced during seismic loading in the N – S direction. In order to ensure that Memorial Sloan-Kettering's foundation can withstand that value, hand calculations were performed on the lateral system between column lines 16 and 16.5. The reason behind choosing this lateral system is that its columns have the smallest tributary areas out of all the lateral systems and thus have the smallest axial forces to counteract the moment.

After performing this calculation it was determined that the axial forces alone would not be sufficient enough to counteract the overturning moment and that the foundation beneath the shear wall would need to be considered as well. As previously mentioned in this technical report, the footing around this shear wall is approximately 8' wide by 30' long by 4' deep. The shear size of this footing is enough to counteract the overturning moment and ensure building stability. These hand calculations can be seen in Appendix E.

LATERAL MEMBER PERFORMANCE

When analyzing the lateral member performance, the seismic loads seen in the N – S direction were chosen to be applied at each story level. The representative lateral system between column lines 15.3 and 16 was chosen to check the lateral bracing. This particular system is the smaller of the two systems in the N – S direction and provides more conservative values, assuring that all systems in Memorial Sloan-Kettering are adequately designed.

After analysis, it seems that this system was properly designed. Although the top two braced-frames appear to be over-designed, this could be due to the desire to have repetitive members in the design. Each braced frame is controlled in compression, with the diagonal bracing between floors two and three coming close to the design yield. I believe this design is adequate though because the effective length chosen was slightly larger then the actual length, and would result in a lower compressive strength. The calculations used to check these lateral members can be found in Appendix E.

SHEAR WALL STRENGTH

The shear wall strength was checked through the hand calculations shown in Appendix E. The controlling seismic loads create a total base shear of 349 kips in each direction, which place approximately 174.5 kips on each shear wall. The concrete details describe a typical shear wall of have #5 bars spaced 12" on center for each face of the wall. This reinforcement placement creates a Vs strength of 520 kips. Also, Vc on the 12" thick, 14' long shear wall calculates to approximately 255 kips. Therefore $\Phi V_n = 581$ kips, which is more then strong enough to take the 175 kips it experiences from the seismic loading.

TORSION

Torsion is experienced in Memorial Sloan-Kettering due to the eccentric placement of the lateral system. These shear walls are positioned more towards the west side of the building. Because no shear walls reside in the south-east corner of MSK, a torsional moment develops around the building's center of mass.

Hand calculations were performed to determine how much torsion was applied to each lateral system on any given lateral load. The representative load chosen was wind loading on the second floor in the N-S direction. The assumption was also made that since all four shear walls are identical in size, they all share the same stiffness. Through these calculations, it was determined that this particular force places 18 kips of eccentric shear on the two shear walls running in the E – W direction and approximately 8 kips on the shear walls in the N – S direction. In conclusion, the calculations demonstrated that torsion forces must be considered when analyzing Memorial Sloan-Kettering.

SUMMARY

This technical report provided an in depth analysis of the lateral system in Memorial Sloan-Kettering. Using a combination of hand calculations and ETABS results, it can be concluded that the current system adequately resists all seismic and wind loads acting on the building. This technical report also provides a solid basis for further investigation into looking at other lateral systems, such as only shear walls or moment-frame connections. The main consideration at this time would be the placement of each shear wall. Because the lateral system is concentrated toward the west side of MSK, torsion considerations and drift issues arise. It would be interesting to see how moving one of those systems to the right would affect the overall system and whether or not it could be more effective. In conclusion, this analysis provided a much better understanding of the current lateral system of Memorial Sloan-Kettering.

APPENDIX A: Load Calculations

LOAD CALCULATIONS

DEAD LOADS

1st Floor

$(6''/12'')(150\ PCF) = 75\ PSF$ SLAB

$= 15\ PSF$ SUPER IMPOSED

$(18''/12)(24''/12)(150)(1/10') = 45\ PSF \rightarrow$ BEAM $\rightarrow$ (450 PLF @ 10' O.C)

$(24/12)(30/12)(150)(1/30') = 50\ PSF \rightarrow$ GIRDER $\rightarrow$ (750 PLF @ 30' O.C; FOR BOTH DIRECTIONS)

185 PSF

2ND FLOOR – 4TH FLOOR

$(4.5''/12'')(150\ PCF) = 56.25\ PSF$ SLAB

$= 2\ PSF$ METAL DECK

$= 15\ PSF$ STEEL FRAMING

$= 15\ PSF$ SUPERIMPOSED

88.25 PSF

ROOF

$(3.5/12)(150) = 43.8\ PSF$ CONCRETE

2 PSF METAL DECK

15 PSF STEEL FRAMING

20 PSF MECHANICAL

80.75 PSF

LIVE LOADS

ALL FLOORS $\rightarrow$ 100 PSF $\rightarrow$ TABLE 4-1 ASCE 7-02

SNOW LOADS

GROUND SNOW LOAD $P_g = 30$ PSF $\rightarrow$ TABLE

FLAT ROOF SNOW LOADS

$P_f = 0.7 C_e C_t I P_g \quad \therefore \quad C_E = 0.9$ TABLE 7-2

$C_T = 1.0$ TABLE 7-3

$I = 1.2$ TABLE 7-4

$P_f = (.7)(0.9)(1.0)(1.2)(30)$

$P_f = 22.68\ psf$

APPENDIX B: Wind Check

(1)

- WIND ANALYSIS

- IBC 2000 refers to ASCE 7 for WIND LOAD CALCULATIONS
  - → TRY USING METHOD TWO - ANALYTICAL PROCEDURE

ASSUMPTIONS:

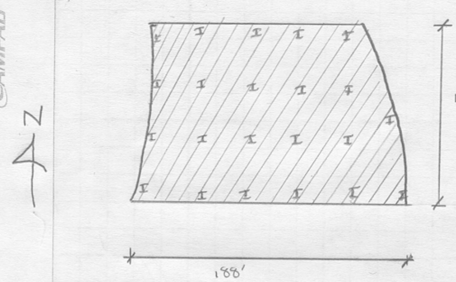
- RIGID STRUCTURE
- HEIGHT = 58' < 60' ∴ QUALIFIES AS LOW-RISE BUILDING
- REGULAR SHAPED BUILDING

$B = 126'$

$h = 58'$

$L = 188'$

→ TYPICAL FLOOR PLAN SIZE



EXPOSURE CATAGORY = C

$V = 90$ MPH

$K_d = 0.85$ (MAIN WIND FORCE RESISTING COLUMN)

$I = 1.15$ where $V = 85-100$ MPH

$K_z =$ TABLE 6.3 for EXPOSURE C

$K_{zt} = 1.0$

$G = 0.858821$

Enclosure CLASSIFICATION = ENCLOSED

$C_p =$ ∴ LOW RISE BLDG. FIG. 6-10

$q_h =$ → computed in TABLE

FIND GUST EFFECT FACTOR

$$G = 0.925\left(\frac{(1+1.7 g_q I_{\bar{z}} Q)}{1+1.7 g_v I_{\bar{z}}}\right) \qquad I_{\bar{z}} = (.2)\left(\frac{33}{34.8}\right)^{1/6} = 0.1982 \qquad L_{\bar{z}} = 500\left(\frac{34.8}{33}\right)^{.2} = 505.339$$

$$\therefore\ G = 0.858821 \qquad Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = 0.866$$

→ CALCS

∴ MAIN WIND FORCE RESISTING Systems (PERTAINING TO LOW-RISE BUILDING)

$$p = q_h(GC_{pf} - GC_{pi})$$

FIND q_h

0-15 ft: $0.00256 \cdot (.85)(1)(0.85)(90^2)(1.15) = 17.229$ lbs/ft²

15-20 ft: $0.00256 \cdot (.9)(1)(0.85)(90^2)(1.15) = 18.2425$ lb/ft²

20-25 ft: $0.00256 \cdot (.94)(1)(0.85)(90^2)(1.15) = 19.053$ lbs/ft²

25-30 ft: $0.00256 \cdot (.98)(1)(0.85)(90^2)(1.15) = 19.864$ lbs/ft²

30-40 ft: $0.00256 \cdot (1.04)(1)(0.85)(90^2)(1.15) = 21.0802$ lbs/ft²

40-50 ft: $0.00256 \cdot (1.09)(1)(0.85)(90^2)(1.15) = 22.09$ lbs/ft²

50-60 ft: $0.00256 \cdot (1.13)(1)(0.85)(90^2)(1.15) = 22.9045$ lbs/ft²

INTERPOLATION REQUIRED FOR 58' → = 22.742 lb/ft²

FIND WIND PRESSURES

$p = qGC_p - q_i(GC_{p_i})$

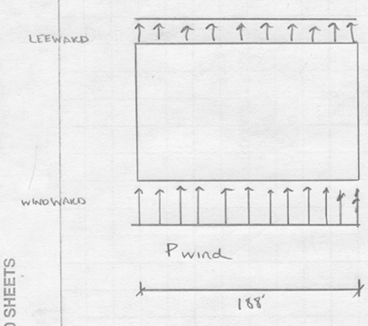
NOTE: $GC_p = 0.687$ ∴ $GC_{p_i} = -.429$ N-S
$= -.345$ E-W

∴ FIRST FIND P in N-S DIRECTION

0'-15' = (17.229)(.687) − (22.742)(−.429) = 21.60 lbs/ft²
15'-20' = (18.2425)(.687) + (9.756) = 22.30 lbs/ft²
20'-25' = (19.0533)(.687) + (9.756) = 22.86 lbs/ft²
25'-30' = (19.8641)(.687) + (9.756) = 23.41 lbs/ft²
30'-40' = (21.0802)(.687) + (9.756) = 24.25 lbs/ft²
40'-50' = (22.0937)(.687) + (9.756) = 24.95 lbs/ft²
50'-60' = (22.9045)(.687) + (9.756) = 25.50 lbs/ft²
@ 58' ⟶ = 25.39 lbs/ft²

→ P in E-W DIRECTION

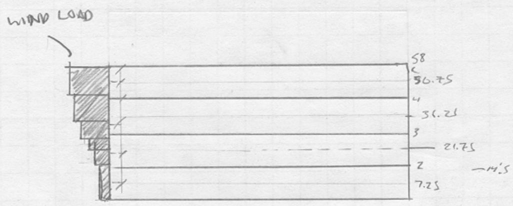
0'-15' → (17.229)(.687) − (22.7433)(− 0.345) = 19.68 lbs/ft²
15'-20' → (18.2425)(.687) + (7.846) = 20.38 lbs/ft²
20'-25' → (19.0533)(.687) + (7.846) = 20.93 lbs/ft²
25'-30' → (19.8641)(.687) + (7.846) = 21.49 lbs/ft²
30'-40' → (21.0802)(.687) + (7.846) = 22.37 lbs/ft²
40'-50' → (22.0937)(.687) + (7.846) = 23.0233 lbs/ft²
50'-60' → (22.9045)(.687) + (7.846) = 23.58 lbs/ft²
@ 58' ⟶ = 23.469 lbs/ft²



Typical Floor HT = 14.5'

$\frac{}{2}$ = 7.25

→ NORTH - SOUTH DIRECTION



FLOOR 2

$= (7.75')(21.60\ lbs/ft^2) + (5')(22.30\ lbs/ft^2) + (1.75')(22.86\ lbs/ft^2) = 319\ lbs/ft \cdot 188' = 59.97\ K$

FLOOR 3

$= (3.25')(22.86) + (5')(23.41\ lbs/ft^2) + (6.25')(24.25\ lbs/ft^2) = 342.9\ lbs/ft = 64.47\ K$

FLOOR 4

$= (3.75')(24.25\ lbs/ft^2) + (10')(24.95\ lbs/ft^2) + .75'(25.50) = 359.56\ lbs/ft = 67.60\ K$

ROOF

$= (7.25')(25.50\ lbs/ft^2) = 180.875\ lbs/ft \times 188' = 34\ K$

TOTAL BASE SHEAR = 225.91 K

OVERTURNING MOMENTS

ROOF = $(34^K)(58') = 1,972\ 'K$

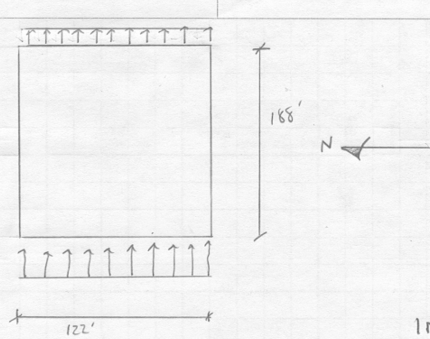
4^{TH} FLOOR = $(67.60)(43.5) = 2,940\ 'K$

3^{RD} FLOOR = $(64.47)(29) = 1,869\ 'K$

2^{ND} FLOOR = $(59.97)(14.5) = 870\ 'K$

OVERTURNING MOMENT = 7,651 'K

(4)



IN E-W DIRECTION

SAME WIND LOAD DISTRIBUTION, JUST DIFFERENT LOADS

CALCS

ROOF: $(7.25')(23.58') = 171\ \text{lbs/ft} \cdot 122' = 20.86$ kips

4^{TH} FLOOR: $(3.75')(22.33\ \text{lbs/ft}^2) + (10')(23.02\ \text{lbs/ft}) + (.75)(23.58\ \text{lbs/ft}) = 331.62\ \text{lbs/ft}^2 \cdot 122' = 40.5$ kips

3^{rd} FLOOR: $(3.25')(20.93\ \text{lbs/ft}^2) + (5')(21.48) + (6.25')(22.33\ \text{lbs/ft}) = 315.035\ \text{lbs/ft}^2 \cdot 122' = 38.434$ kips

2^{nd} FLOOR: $(7.75')(19.68\ \text{lbs/ft}) + (5')(20.38\ \text{lbs/ft}) + (1.75')(20.93\ \text{lbs/ft}) = 291.05\ \text{lbs/ft} \cdot 122' = 35.507$ kips

TOTAL BASE SHEAR = 135.301 K

OVERTURNING MOMENTS

ROOF = $(20.86)(58') = 1209.88$ 'K

4^{TH} FLOOR = $(43.5')(40.5k) = 1761.75$ 'K

3^{RD} FLOOR = $(29')(38.43k) = 1114.5$ 'K

2^{ND} FLOOR = $(14.5')(35.507k) = 514.75$ 'K

OVERTURNING MOMENT = 4601 'K

Wind Load Analysis

Building Properties	
B (ft)	122
L (ft)	186
h (ft)	58.00
K_{zt}	1
K_d	0.85
V(mph)	90
Importance	III
I_w	1.15
Exposure	C
α	9.5
z_g	900
z_{min}	15
c	0.2
$\in$	0.2
ℓ	500
$\hat{b}$	0.650
$\hat{\alpha}$	0.153846
$\bar{a}$	0.105
$\bar{b}$	1

Fundamental Period	
Struct. Type	Eccentric Steel
C_t	0.03
x	0.75
T	0.63051
Natural f	1.586017
Rigidity	Rigid

Rigid	
$g_Q=g_v$	3.4
$\bar{z}$	34.8
I_z	0.198237
L_z	505.3393
Q	0.867508
G	0.859559

Flexible	
g_R	4.30
R_n	0.034
N_1	9.27
η_h	4.89
η_B	0.084
η_L	52.52
R_h	0.184
R_B	0.946
R_L	0.019
V_z	86.50
β	0.05
R	0.25
G_f	0.8877

Windward	
Cp	0.8

Leeward	Ratio	Cp
N-S	0.656	-0.50
E-W	1.525	-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
58	1.122	22.7423

Pressures			
Windward	N-S	P_z	0.688
	E-W	P_z	0.688
Leeward	N-S	P_h	-0.430
	E-W	P_h	-0.340

Velocity Pressure Envelope						
	Windward		Leeward		Max psf	
Z(ft)	N-S	E-W	N-S	E-W	N-S	E-W
0-15	11.85	11.85	-9.77	-7.72	21.62	19.57
15-20	12.54	12.54	-9.77	-7.72	22.32	20.27
20-25	13.10	13.10	-9.77	-7.72	22.88	20.83
25-30	13.66	13.66	-9.77	-7.72	23.43	21.38
30-40	14.50	14.50	-9.77	-7.72	24.27	22.22
40-50	15.19	15.19	-9.77	-7.72	24.97	22.92
50-60	15.75	15.75	-9.77	-7.72	25.52	23.47
58	15.64	15.64	-9.77	-7.72	25.41	23.36

Shear Summary		
(kips)	N-S	E-W
Shear @ Roof	34	20.86
Shear @4	67.6	40.5
Shear @3	64.47	38.43
Shear @2	59.97	35.5
Shear @1	0	0
Base Shear	226.04	135.301
Overturning Moment	7660.30	4601

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	7.25	58.25	34.42	20.76	0	0	2004.94	1209.40
4	14.5	43.50	66.93	40.27	34.42	20.76	2911.33	1751.76
3	14.5	29.00	63.84	38.24	101.35	61.03	1851.23	1109.03
2	14.5	14.50	59.37	35.31	165.18	99.28	860.87	512.05
1	7.50	0.00	0.00	0.00	224.55	134.59	7628.38	4582.24

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	N-S	E-W	N-S	E-W
Roof	7.25	58.25	25.81	15.57	30.15	25.81	15.57	30.15	0	0	1756.10	1756.10
4	14.5	43.50	50.20	30.20	58.58	50.20	30.20	58.58	30.15	30.15	2548.29	2548.29
3	14.5	29.00	47.88	28.68	55.81	47.88	28.68	55.81	88.73	88.73	1618.51	1618.51
2	14.5	14.50	44.53	26.49	51.81	44.53	26.49	51.81	144.54	144.54	751.23	751.23
1	7.50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	196.35	196.35	6674.13	6674.13

APPENDIX C: Seismic Check

SEISMIC LOAD DESIGN

Design Parameters	
# of stories	4
h_s	14 ft
h_n	56 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_{EW}	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}	0.41		T_{EW}	0.41
but not greater than:			but not greater than:		
	$C_{smax, N-S}$	0.044		$C_{smax, E-W}$	0.044
and	$C_{s\,min}$	0.0172		$C_{s\,min}$	0.0172
Therefore, ($C_{S,N-S}$) used is:		0.044	Therefore, ($C_{S,N-S}$) used is:		0.044

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 ft2
Total weight of roof:	1621.6 kips
Total weight 2nd-4th floor:	2106.4 kips
Total Weight 1st Floor:	
Total Building Weight:	7940.7 kips
Seismic Shear, V_{N-S}:	**349.1 kips**
Seismic Shear, V_{EW}:	**349.1 kips**

Vertical Distribution of Seismic Forces

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

Exponent k_{N-S}: 0.954711

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	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 Load Design

General

\# of stories → 4 floors
story height → 14 ft
building height → 56 ft
} Architectural Cover Sheet

First Floor 26,500
18,500

Design Parameters

Seismic Use Group → III → (Table 1-1 gives healthcare facility Category IV)
Occ. Importance Factor → 1.5 (Table 9.1.4)
S_s → .39 Figure 9.4.1.1 (a) S_{ms} → 0.39
S_1 → .09 Figure 9.4.1.1 (b) S_{m1} → 0.09
F_a → 1.0 Table 9.4.1.2.4a S_{DS} → $(2/3)(.39) = 0.26$
F_v → 1.0 Table 9.4.1.2.4b S_{D1} → $(2/3)(.09) = 0.06$
Rigid Component
Seismic Design Category → C 9.4.1.2
Site Class → B

Equivalent Lateral Force Procedure

North - South

R = 5 Steel concentrically braced frames (Table 9.5.2.2)
C_s = 0.078 → Eqn 9.5.5.2.1-1
C_T = 0.02 Table 9.5.5.3.2
X = .75 (All other structures) Table 9.5.5.3.2
T = 0.42
$C_{s\,max}$ = 0.043 (Eqn 9.5.5.2.1-3)
$C_{s\,min}$ = 0.0172 (Eqn 9.5.5.2.1-4)
$C_{s\,used}$ = 0.043

East - West

R = 5 Steel concentrically braced frames
C_s = 0.078
C_T = 0.02
X = .75
T = 0.42
$C_{s\,max}$ = 0.043
$C_{s\,min}$ = 0.0172
$C_{s\,used}$ = 0.043

Calculation Variables

Building Width: 126 ft
Building Height: 188 ft
Floor Area: 23,688 ft²

Allowable Story Drift, Δ_a (Table 9.5.2.8)

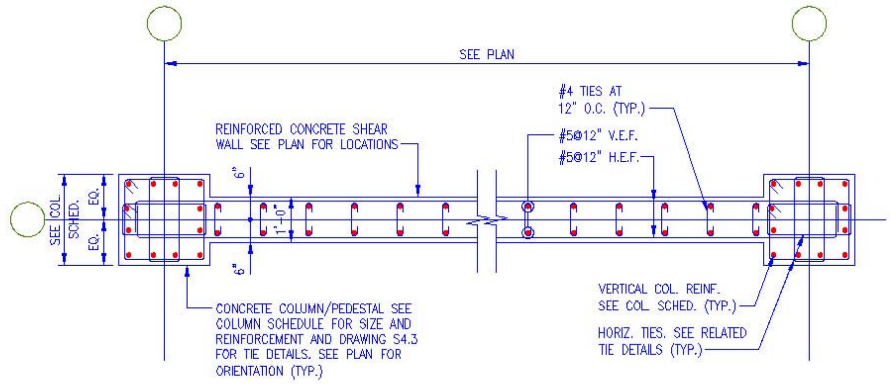
– Structures other than masonry shear wall → Seismic Use Group III
Four stories or less

$\Delta_a \leq 0.015 h_{sx}$

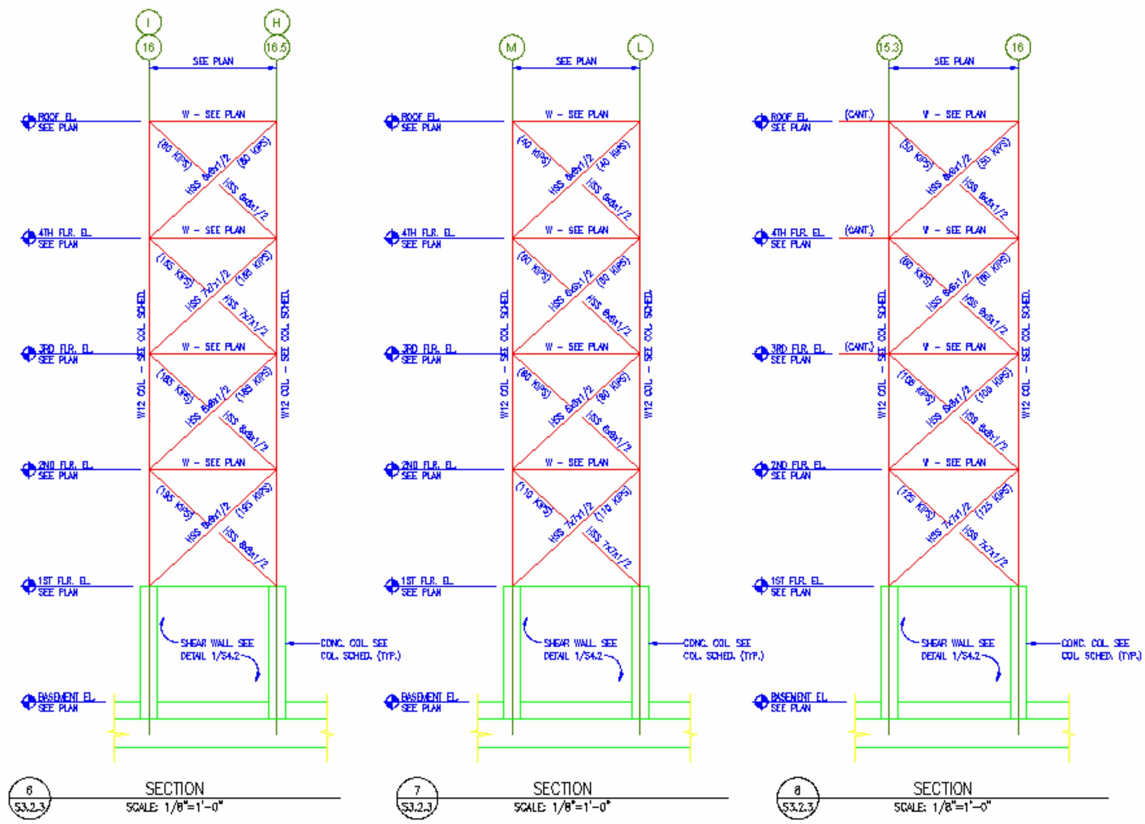
(h_{sx} is story height below level x)

$\therefore \Delta_a = 0.015(58 \cdot 12)$
$= 10.44''$

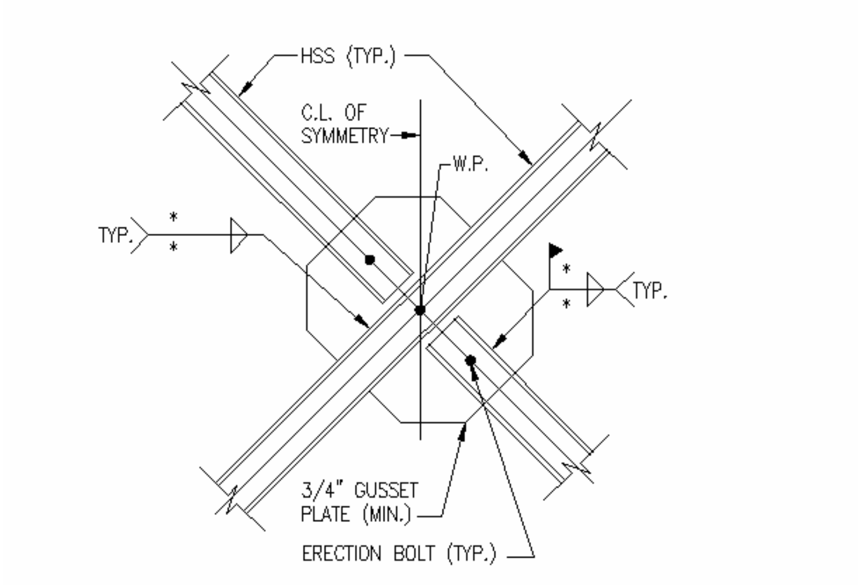
APPENDIX D:



(X-1) SHEAR WALL SECTION



(X-2) LATERAL SYSTEM SECTION



(X-3) BRACING CONNECTIONS

APPENDIX E: Hand Calculations

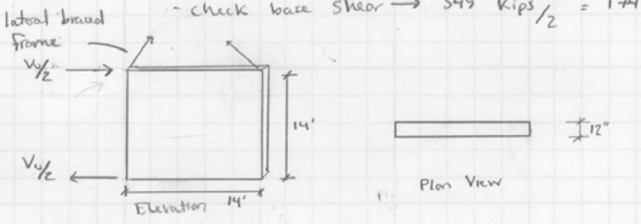
Shear Strength

Shear Strength Check

Shear Wall Check

- check overturning moment − 14075 ft·kips
- check base shear → 349 kips/2 = 174.5^k

* Two shear walls in each direction



t = 12"
b = 168"
h = 168"

$V_u = 349^k$

$V_u/2 = 174.5^k$

$V_n = V_u/\phi = 174.5^k/.75 = 232.6^k$

* because all shear walls are identical, assume same stiffness for each

* because shear wall spans in between two axial columns, assume shear wall only takes shear and flexure

$\therefore V_c = 2\sqrt{f_c'}\, b_w \cdot d$

$= 2\sqrt{4000} \cdot (168")(12") = 255$ k

$\frac{\phi V_c}{2} = 82.9^k$ ∴ Reinforcement Required

→ Try #5 bars @ 12" o.c. for each face

$A_v = .75\sqrt{4000} \cdot \frac{(12)(12)}{50} = 0.11$

but must be greater than $(50 \cdot 12 \cdot 12)/60{,}000 = 0.12$ → use

∴ Use #5 bars @ 12" on each face

$V_s = \frac{(2 \cdot .31)(60\,000)(168)}{12} = 520^k$

d bending in strong axis ∴ d = 168"

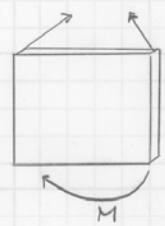
$\phi V_u = (.75)(520 + 255) = 581^k > 174.5^k$

∴ Shear Wall Strength adequate

Overturning

Check Overturning Moment

$\rightarrow$ 14,075 ft·kips (from seismic loading)



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

$14{,}075/2 = 7037.5\ ^{k \cdot ft}$

$d = 14'$ (length of shear wall)

FIND LOADS ACTING ON COLUMNS

$A_T = 368\ ft^2$

$DL \approx 80$ psf
$LL = 100$ psf
$S_{DL} = 20$ psf

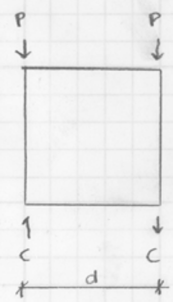
Interior Column $K_{LL} = 4$

$A_T = 5(368) = 1840$

$(4)(1840) = 7360$

$LL_R = .25 \quad \frac{15}{\sqrt{7360}} = .4$

$LL = 1840(100)(.40) = 73.6^k$



COUPLE ACTING ON SHEAR WALL

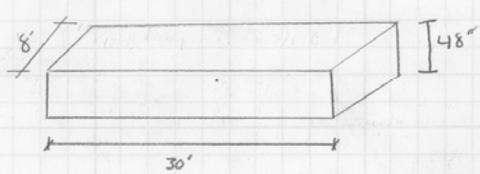
$C = M_u/d = 502^k$

$DL = (140.75)(368) + 3(85.25)(368)$
$+ .185(368) = 210.7$

$1.2(210.7) + 1.6(73.6) = 370.6^k$

$370.6^k < 502^k$

NEED TO CONSIDER WEIGHT OF FOUNDATION AS WELL



- FOUNDATION UNDER REPRESENTATIVE SHEAR WALL IS $30' \times 8' \times 4'$

$w = (48/12)(150)(8')(30') = 144^k$

$144 + 370 = 514^k > 502^k$

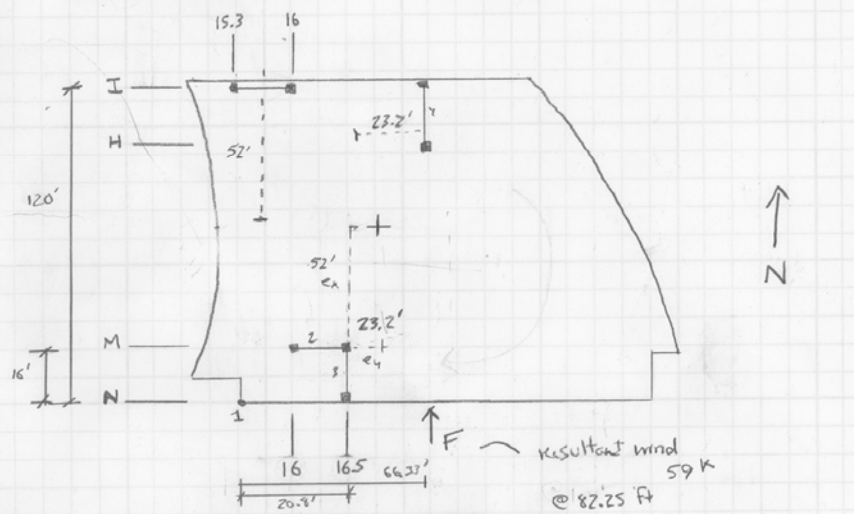
$\therefore$ OVERTURNING MOMENT IS PREVENTED

Torsion

Torsion Check

→ Each shear wall is 12" thick by 14' long

SHEAR WALL LOCATIONS



→ Assume rigidity of each shear wall is the same

- From location 1 → because rigidity is the same → assume it equals 1

– In N-S direction

$$\frac{(R)(16') + (R)(120')}{2R} = 68'$$

centroid @ (44', 68')

In E-W direction

$$\frac{(R)(20.8) + (R)(66.33)}{2R} = 44'$$

Torsion

→ Place representative wind load on 2nd Floor (N-S direction)

∴ Resultant wind load = 59.4^k

$M_T = P_y \cdot e_x + P_x \cdot e_y$

$M_T = (59^k)(82.25 - 44') = 2256.75\ 'F$

Assume same stiffness

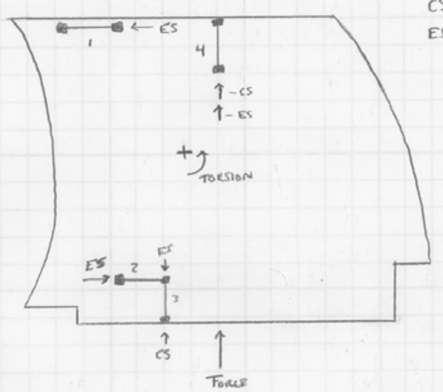
$J = 52^2 + 52^2 + 23^2 + 23^2 = 6466$

$F_2 = F_1 = \frac{(52)}{6466}(2256.75) = 18^k$ eccentric torsion shear

$F_3 = F_4 = \frac{23.2}{6466}(2256.75) =$ 8.09 eccentric torsional shear

Torsion Check Continued

From 2ND Floor Wind Load



CS = concentric shear
ES = eccentric shear

Wall 1 → Experiences 18^k of Eccentric Torsional Shear

Wall 2 → Experiences 18^k of Eccentric Torsional Shear

Wall 3 → ES and CS counteract each other ∴ consider only CS for worst case senario ∴ Concentric Shear = 29.5^k

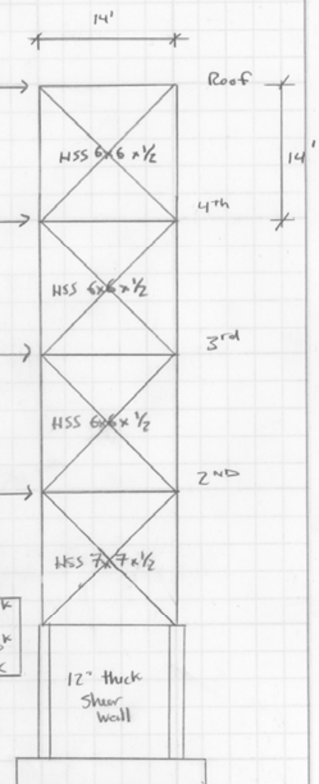
Wall 4 ∴ ES and CS act in Same direction
∴ $8.09^k + 29.5^k = 37.6^k$

∴ Torsion Exists in MSK

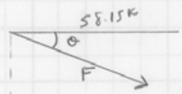
Lateral Member Check

LATERAL MEMBER CHECK

→ Two shear walls in each direction (identical)
∴ each force is $V/2$



<u>Roof</u> ∴ $V = 116.3/2 = 58.15^k$



Effective length = 17.5'

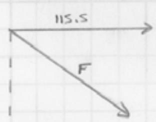
$\tan^{-1}(14'/14') = \theta = 45°$

$F = \dfrac{58.15^k}{\cos 45} = 82.2\text{ K}$ TENSION
82.2^k COMPRESSION

TENSION → HSS 6x6 x 1/2 = $403^k > 82.2^k$ ∴ OK

TABLE 4-6 COMP → HSS 6 x 6 x 1/2 = $217^k > 82^k$ ∴ OK

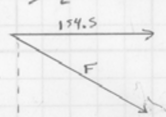
<u>4TH Floor</u>
∴ $V = 116.3 + 114.7 = 231/2 = 115.5$



$F = \dfrac{115.5}{\cos(45)} = 163.3^k$ TENSION
163.3^k COMPRESSION

TEN: HSS 6x6 x 1/2 = $403^k > 218.42^k$
COM: HSS 6x6x 1/2 = $217^k > 163.5^k$ ok

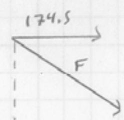
<u>3rd Floor</u>
$V = 308.9/2 = 154.45$



$F = \dfrac{154.5}{\cos(45)} = 218.42^k$ TENSION
218.42^k COMPRESSION

TEN HSS 6x6 x 1/2 = $403^k > 218.42^k$ ∴ OK
COMP: HSS 6x6 x 1/2 = $217^k < 218.42^k$ ∴ SMALL

<u>2ND Floor</u>
$V = 349^k/2 = 174.5$



$F = 174.5 / \cos 45 = 246.7^k$ TENSION
$= 246.7^k$ COMPRESSION

TEN: HSS 7"x7"x 1/2" = 480 k > 246.7^k OK
COMP: HSS 7"x7"x 1/2" = $303^k > 246.7^k$ OK

After checking the lateral force members, it seems that they system is properly designed. Although the two top braces appear to be overdesigned, the fact that the engineers may want repetitive members explains the member choice. Each braced-frame is controlled in compressive, with the braced frame between floors two and three coming close to design yield. I believe this design is ok though because the effective length chosen is slightly larger than the actual length which will result in a larger actual compressive strength.