Weill Cornell Medical Research Building
413 E. 69^{th} Street
New York, NY

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Structural Option

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Technical Report 1

9/23/11

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Executive Summary

The following Technical Report analyzes the existing conditions of the Weill Cornell Medical Research Building located on E. 69^{th} Street in New York City. Plans were provided by Severud Associates. Architectural images courtesy of Ennead Architects (formerly Polshek Partnership Architects, LLP). An analysis of various loading conditions as well as spot checks was carried out using the newest applicable codes and standards.

The building consists mostly of concrete with two way slabs and concrete beams and columns. There is some structural steel at the top for mechanical and window washing equipment. The lateral system consists of Ordinary Reinforced Concrete Shear Walls as well as a few large columns.

ASCE 7-05 was used to determine loads on the building. Snow, Seismic, Wind, and Gravity loads were analyzed. Analysis of snow load resulted in the value of 17.3 psf which is less than the roof live load of 30 psf. Seismic analysis resulting in a base shear value of 979.6 kips and an overturning moment of 191,420 k-ft. Wind load were calculated to produce a base shear of 2,548 kips and overturning moment of 205,488 k-ft in the North-South direction and a base shear of 856 kips and overturning moment of 21,949 k-ft in the East-West direction. This means that wind loads control in the North-South direction, but seismic loads are greater in the East-West direction.

Spot checks of typical members under gravity loads showed that typical beams are adequate for strength and spacing while typical columns appear to be overdesigned. This could be due to a few errors in load calculation or perhaps the assumption that the column is only loaded axially.

Introduction

The Weill Cornell Medical Research Building is the newest addition to the campus of the Weill Cornell Medical College on the upper east side of Manhattan. Located at 413 East 69th Street in New York City, the Medical Research Building is adjacent to other Weill Cornell buildings. The Weill Greenberg Center on its northeast side is an educational facility designed by the same architects as the Medical Research Building. Olin Hall to the east, and the Lasdon House to the north are residential buildings that house undergraduate and graduate students of the medical college. 69th Street slopes down to the east across the site of the Medical Research Building and the utilities run under it. The Conn. Edison power vaults are also located under 69th Street and the sidewalk in front of the building.

The $650 million Medical Research Building is approximately 455,000 square feet with three stories below grade and eighteen, plus a penthouse and an interstitial floor, above grade. The total height of the building above grade is 294'-6." Floors 4-16 are dedicated to laboratory space. The first basement level, as well as the interstitial floor between floors 16 and 17, and the 17th and 18th floors are designated as mechanical floors. The bottom two levels of the basement contain the MRB's animal facility. Service and freight elevators and vertical circulation are located on the west side of the building next to the loading docks on the 69th Street side. Passenger elevators and vertical circulation are nearer the center of the building where the two story lobby atrium welcomes people into this hub of scientific exploration.

In the back of the building, off of the second floor, there is a terrace that bridges the gap between the rear façade of the MRB and the Lasdon House. A grand staircase leads from the lobby on the ground floor up to the enclosed lounge on the second floor that opens onto the terrace. There are two entryways from the Lasdon House to the terrace so anyone living in that building and working in the Medical Research Building would have easy access. The terrace also wraps around the side of the Lasdon House and connects to a stairway leading down to the sidewalk on 70th street.

The building is defined visually by the undulating glass sunshade curtain wall across the front of the building. This curtain wall is attached to the floor slabs that are cantilevered out approximately 12’-8” from the exterior row of columns to meet it. The curtain wall itself has two layers. The outer layer features the glass sunshade wall with aluminum mullions. That is tied to the inner layer of insulated glass (also with aluminum mullions) by aluminum. The inner layer is anchored to the slab either directly through the mullion or with a steel outrigger.

Structural Systems

Foundation System

The foundation system consists of spread footings bearing on undisturbed bedrock with strap beams as necessary around the perimeter. This undisturbed bedrock is required to support 40 tons per square foot. According to the geotechnical report, there are two types of bedrock encountered on the site. One which supports 40 tsf and the other 60 tsf, but it is recommended by Langan Engineering and Environmental Services that the footings be designed to rest on 40 tsf bedrock. The slab on grade is a 6” concrete slab resting on a 3” mud slab on 24” of crushed stone. The perimeter concrete walls of the basement are 20” thick with strip footings. Below, Figure 1 is an image of the foundation plan.

The geotechnical report also states that the water table is approximately 50 feet above the foundation level. This poses the problem of seepage through the rock and also uplift on the foundation. A few different design solutions are presented in the report. The resolution of this problem comes in the form of 4-50 ton rock anchors located at the bottom of Stairwell B on the East side of the building to resist the uplift.

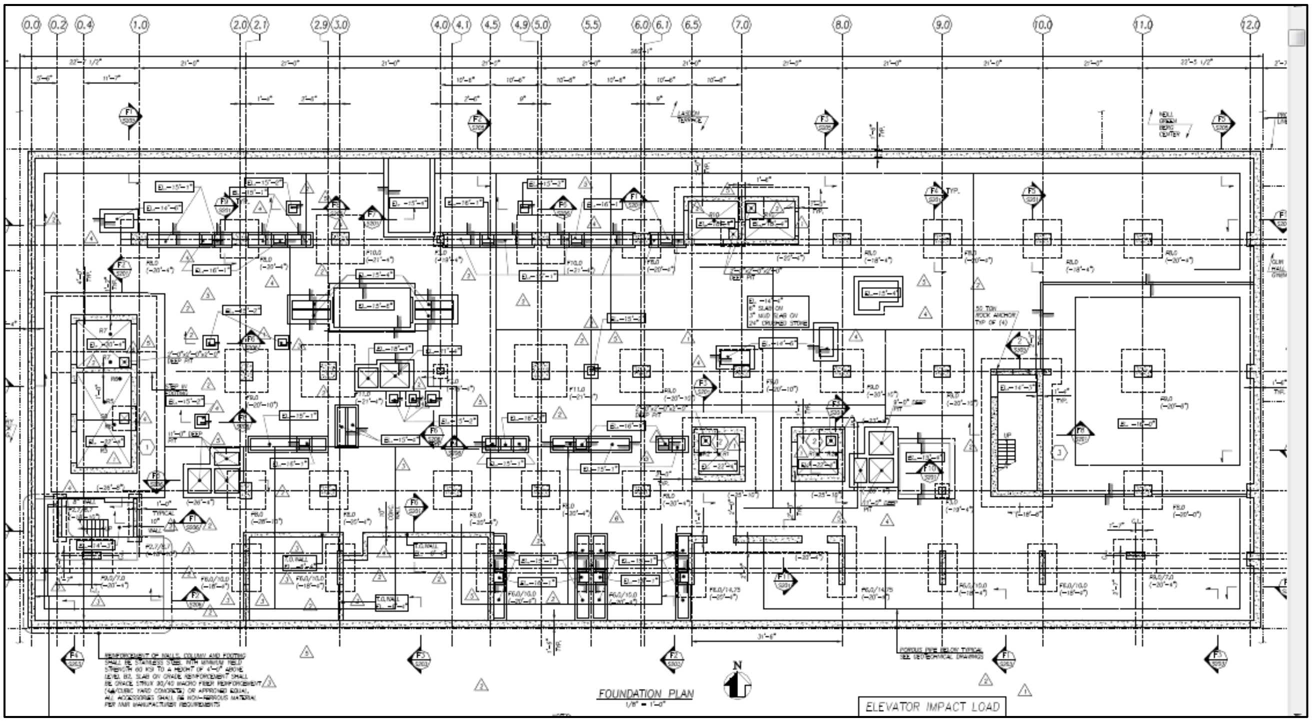

Figure 1: Basement Level 3 – Foundation Plan

Floor System

The floor system in the Medical Research Building is 2 way flat plate concrete slabs. These slabs vary in depth from floor to floor (see Figure 2 below). The bottom reinforcement is typically #5 bars at 12." Top reinforcement and additional bottom reinforcement varies as needed throughout the building. The slabs are especially thick in this building because much of the design was constrained by strict vibration requirements of the medical and research equipment in the building. Laboratory floors were designed to limit vibration velocities to 2000 micro-inches per second. Walking paces were assumed to be moderate (75 footfalls per minute) in the labs and corridors and fast (100 footfalls per minute) only in public areas such as the lobby. There are also vertical HSS members every other floor through the middle of the building where the laboratories are located. These members serve no structural load bearing purpose, they are simply meant to tie each floor to another floor to further limit vibrations by forcing any impact to vibrate two floors instead of just one.

Floor	Slab Depth (in)
B3	6
B2	12.5
B1	12.5
1	11
2	12
3	12.5
4	12.5
5	12.5
6	12.5
7	12.5
8	12.5
9	12.5
10	12.5
11	12.5
12	12.5
13	12.5
14	12.5
15	12.5
16	12.5
Interstitial	10.5
17	10.5
18	12.5
19	10.5

Figure 2: Slab Depth per Floor

Lateral System

Lateral loads, such as seismic and wind loads, are primarily resisted by 12”-14” concrete shear walls located around the stairwells and elevator cores. A couple of these shear walls step in at the second floor. Extra precautions were taken to make sure that the lateral moment still has a viable path to travel through that step in. Severud, the structural engineers for the project, desired to transfer lateral loads toward the perimeter of the building. In the front of the building there are massive 12/14 x 72 inch columns from which the slabs cantilever out and the glass sunshade curtain wall is hung. These columns also take some of the lateral loads. See the sketch in Appendix E for the location of lateral system elements on a typical floor.

Beams and Columns

There is a very wide variety of beam and column sizes in this building. There are almost forty different sizes of columns with dimensions ranging from 12" to 84," with the most typical column being 24 x 36, and approximately fifty five different sizes of beams ranging from 8 x24 to 84 x 48. Except on the laboratory floors, which are quite uniform, the column sizes tend to change from floor to floor. Extra precaution was taken during design and reinforcement was provided to ensure the continuity of the load path through these column transfers.

Columns are located on the specified grid of 4 major rows in the East-West direction for the majority of the floors—except the first floor and below grade, which have a fifth row in the back of the building. Bay sizes are 27'-7," 25'-0," and 16'-3" in the North-South direction and the typical bay in the East-West direction is 21'-0" with end spans approximately 22'-6." Beams, however, are only placed where they are needed. They are rarely in the same place from floor to floor and each floor has a different number of beams. The fourth floor has the fewest with 6, and the second floor has the most with 33. Below in Figure 3 is a typical framing plan for the 5^{th}-15^{th} floors.

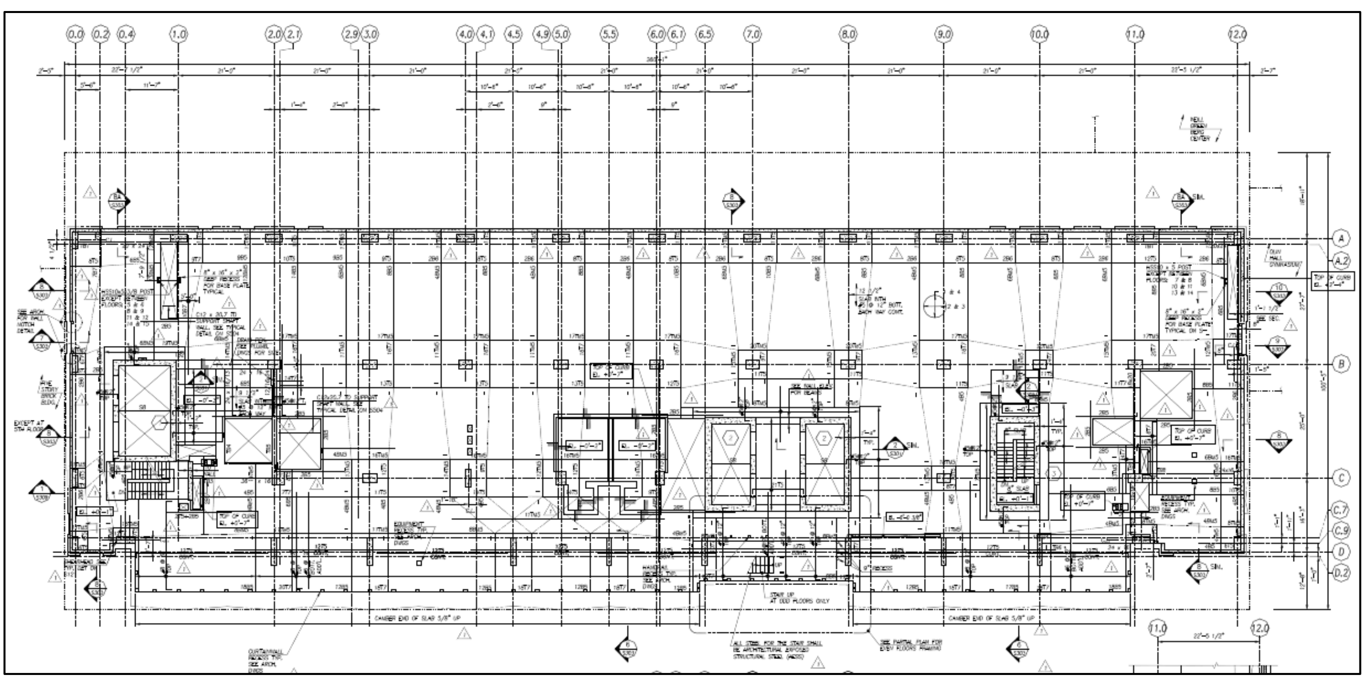

Figure 3: Typical Framing Plan – 5^{th}-15^{th} Floors

Design Codes and Standards

The Weill Cornell Medical Research Building was designed according to the 1968 New York City Building Code based on the UBC. In 2008 New York City updated their building code, which is now based on the IBC. For this report, the new 2008 code for analysis and design is being used; which references ASCE 7-02, ACI 318-02, etc. For relevance, ASCE 7-05, ACI 318-08, and the AISC Steel Construction Manual 14th ed. will be referenced in this report. The design for the Medical Research Building was submitted in 2008 and the project team decided to file under the old code. The MRB is located in New York City's zoning district R8, the use group is 3 (college), the construction class is I-C, and the occupancy group is D-2.

Structural Materials

The Medical Research Building is a predominantly concrete structure. The f'_c of the concrete varies throughout. See the table below in Figure 4 for the strength of concrete per floor.

On the roof and penthouse levels, there are structural steel members that frame platforms for mechanical equipment (cooling towers on the roof level), and also the window washing platform on the penthouse level. This penthouse level platform provides the means from which the window washing apparatus are hung and operated.

Steel members include W14s as horizontal framing members and HSS 10x8x5/8 for the perimeter. Columns, some of which extend down to the 19th floor (on the west side of the building) and some which continue to the 18th floor (on the east side) are HSS 8x8x3/8. The cooling tower platform consists of horizontal members ranging from W8s – W18s and HSS 8x8s as the columns. Figures 5 and 6 show the window washing platform and 19th floor framing plans.

Floor	f'_c Beams and Slabs(psi)	f'c Columns (psi)
B3	4000	8000
B2	5950	8000
B1	5950	8000
1	5950	8000
2	5950	8000
3	5950	8000
4	5950	8000
5	5950	8000
6	5000	5950
7	5000	5950
8	4000	5000
9	4000	5000
10	4000	4000
11	4000	4000
12	4000	4000
13	4000	4000
14	4000	4000
15	4000	4000
16	4000	4000
Interstitial	4000	4000
17	4000	4000
18	4000	4000
19	4000	4000

Figure 4: Concrete Strength per floor

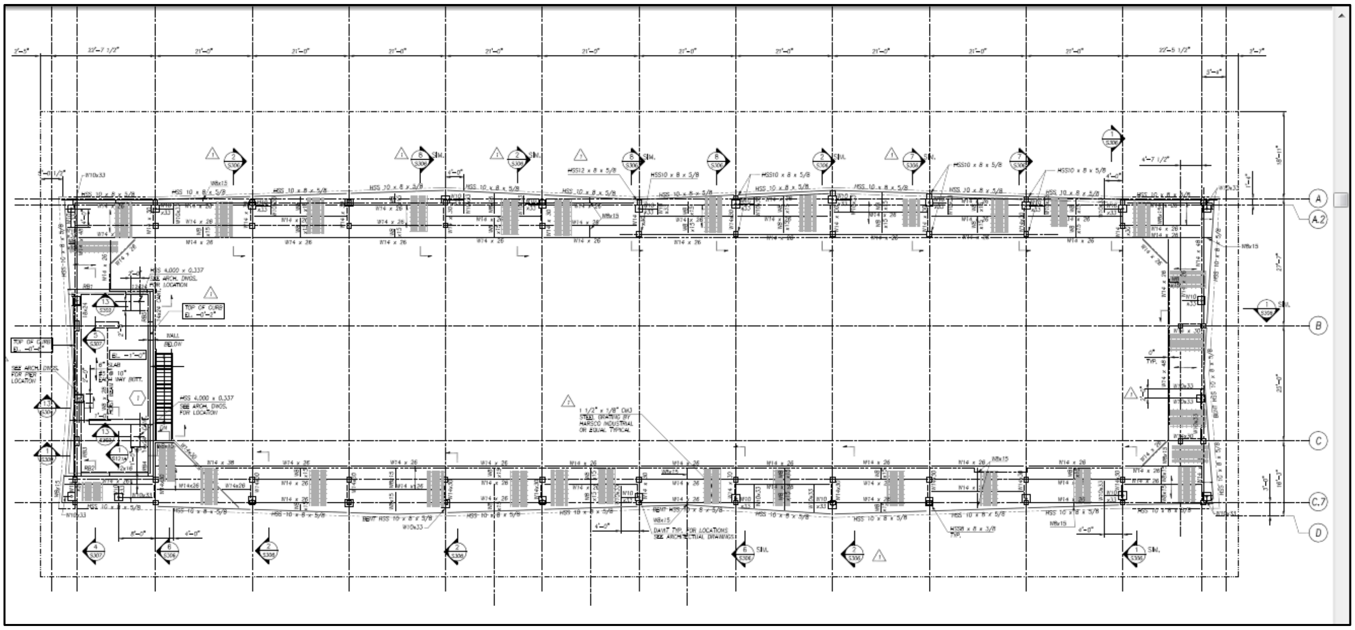

Figure 5: Window Washing Platform Framing Plan

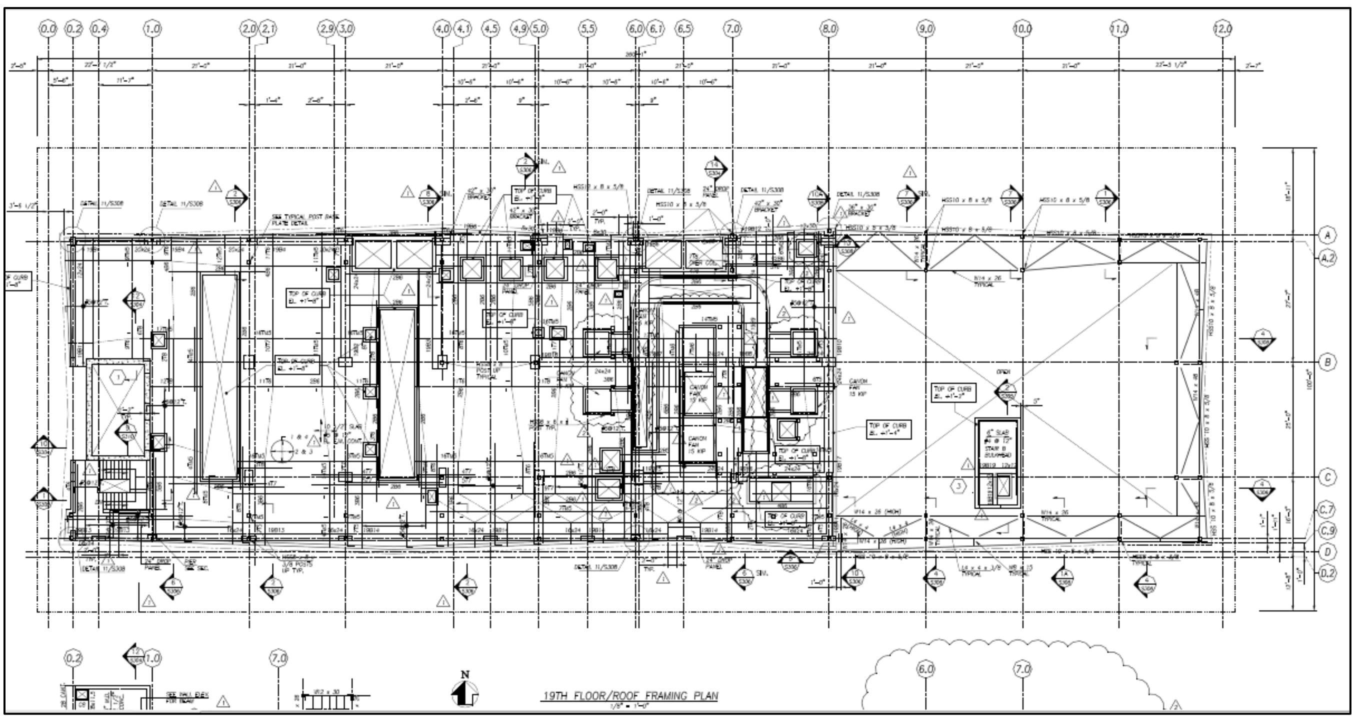

Figure 6: 19th Floor/Roof Framing Plan

Building Loads

Dead and Live Loads

There are a number of different occupancies within this building. The lower floors feature more business and office-like occupancies while the labs and mechanical rooms present more unique circumstances. The table below in Figure 7 shows some typical loads seen throughout the building.

LOADING SCHEDULE (PSF)

LEVEL	SLAB	CEILING AND MECH.	PART'N.	MISC. DL.	LIVE LOAD	TOTAL LOAD	REMARKS
VIVARIUM	160	20	60	5	60	305	–
VIVARIUM MEZZ.	VARIES	10	–	15	50	VARIES	OR EQUIP.
B1	VARIES	30	10	15	150	VARIES	OR EQUIP.
LOADING DOCK	150	10	60	5	400	625	+4" TOPPING SLAB
SIDEWALK	150	10	–	50	600	810	–
LOBBY	140	10	–	25	100	275	–
AUDITORIUM	140	10	12	15	100	277	–
LABORATORY	160	10	12	5	60	247	–
OFFICES	160	10	12	5	50	237	–
MECHANICAL	160	30	12	5	150	357	OR EQUIP.
CORRIDOR	VARIES	10	12	5	100	VARIES	–
INTERSTITIAL	130	30	–	5	50	195	–
DATA CENTER	150	10	12	15	300	487	–
ROOF	130	30	–	15	30	205	OR EQUIP.
STORAGE	VARIES	10	12	5	150	VARIES	–

FACADE LOADS:
BLOCK AND BRICK 95 PSF
DOUBLE GLASS CURTAIN WALL 46 PSF

Figure 7: Loading Schedule

Snow Load

The snow load was calculated using ASCE 7-05 section 7.3. The actual roof surfaces are either steel grating on the structural steel members or the concrete slab of the 18th or 19th floors, so the roof was assumed to be flat for the calculation of snow load. From Figure 7-1 it was determined that the ground snow load in New York City is 25 psf. Following the procedure, the roof snow load was calculated to be 17.3 psf. According to the loading schedules in the plans, the roof live load is 30 psf, which would therefore control in design.

Wind Load

ASCE 7-05 was used to calculate wind pressures and story forces transferred to the Main Wind Force Resisting System (MWFRS) for both the East-West and North-South direction.

The basic wind speed was determined to be 110 mph in New York City from Figure 6-1C. The plans list the exposure category as B, and the occupancy category was determined to be III because it is an educational research lab and part of Weill Cornell Medical College.

The structure was assumed to be rigid, which meant the gust effect factor, G=.85. An excel spreadsheet was created to carry out the calculations of wind pressure and force for each story on the windward and leeward sides (Figures 8 and 9). Another excel spreadsheet was created to calculate the total base shear and overturning moment (Figure 11). Wind pressure diagrams were drawn to show how pressure is distributed in each direction (Figure 10).

Floor	Elev	z	K_z	q_z	Windward (psf)	Windward (plf)	Windward (k)	Leeward (psf)	Leeward (plf)	Leeward (k)
1	5.08	0.00	0.57	17.26	18.712	1309.871	9.824	-14.158	-991.06	-7.433
2	20.08	15.00	0.57	17.26	18.712	1309.871	18.884	-14.158	-991.06	-14.288
3	33.92	28.83	0.66	19.98	20.566	1439.587	19.914	-14.158	-991.06	-13.710
4	47.75	42.67	0.76	23.01	22.624	1583.715	21.908	-14.158	-991.06	-13.710
5	61.58	56.50	0.81	24.53	23.654	1655.779	22.905	-14.158	-991.06	-13.710
6	75.42	70.33	0.89	26.95	25.301	1771.082	24.500	-14.158	-991.06	-13.710
7	89.25	84.17	0.93	28.16	26.125	1828.733	25.297	-14.158	-991.06	-13.710
8	103.08	98.00	0.96	29.07	26.742	1871.971	25.896	-14.158	-991.06	-13.710
9	116.92	111.83	0.99	29.98	27.360	1915.210	26.494	-14.158	-991.06	-13.710
10	130.75	125.67	1.04	31.49	28.390	1987.274	27.491	-14.158	-991.06	-13.710
11	144.58	139.50	1.09	33.00	29.419	2059.338	28.488	-14.158	-991.06	-13.710
12	158.42	153.33	1.09	33.00	29.419	2059.338	28.488	-14.158	-991.06	-13.710
13	172.25	167.17	1.13	34.22	30.243	2116.989	29.285	-14.158	-991.06	-13.710
14	186.08	181.00	1.17	35.43	31.066	2174.641	30.083	-14.158	-991.06	-13.710
15	199.92	194.83	1.17	35.43	31.066	2174.641	30.083	-14.158	-991.06	-13.710
16	213.75	208.67	1.20	36.33	31.684	2217.879	32.252	-14.158	-991.06	-14.412
Interstitial	229.00	223.92	1.20	36.33	31.684	2217.879	28.001	-14.158	-991.06	-12.512
17	239.00	233.92	1.20	36.33	31.684	2217.879	34.377	-14.158	-991.06	-15.361
18	260.00	254.92	1.28	38.76	33.331	2333.182	44.914	-14.158	-991.06	-19.078
19	277.50	272.42	1.28	38.76	33.331	2333.182	40.247	-14.158	-991.06	-17.096
Penthouse	294.50	289.42	1.28	38.76	33.331	2333.182	19.832	-14.158	-991.06	-8.424

Figure 8: Wind Load Excel Sheet – East-West Direction

Floor	Elev	z	K_z	q_z	Windward (psf)	Windward (plf)	Windward (k)	Leeward (psf)	Leeward (plf)	Leeward (k)
1	5.08	0.00	0.57	17.26	18.712	4771.674	35.788	-23.448	-1641.37	-12.310
2	20.08	15.00	0.57	17.26	18.712	4771.674	68.792	-23.448	-1641.37	-23.663
3	33.92	28.83	0.66	19.98	20.566	5244.209	72.545	-23.448	-1641.37	-22.706
4	47.75	42.67	0.76	23.01	22.624	5769.247	79.808	-23.448	-1641.37	-22.706
5	61.58	56.50	0.81	24.53	23.654	6031.766	83.439	-23.448	-1641.37	-22.706
6	75.42	70.33	0.89	26.95	25.301	6451.797	89.250	-23.448	-1641.37	-22.706
7	89.25	84.17	0.93	28.16	26.125	6661.813	92.155	-23.448	-1641.37	-22.706
8	103.08	98.00	0.96	29.07	26.742	6819.324	94.334	-23.448	-1641.37	-22.706
9	116.92	111.83	0.99	29.98	27.360	6976.836	96.513	-23.448	-1641.37	-22.706
10	130.75	125.67	1.04	31.49	28.390	7239.355	100.144	-23.448	-1641.37	-22.706
11	144.58	139.50	1.09	33.00	29.419	7501.874	103.776	-23.448	-1641.37	-22.706
12	158.42	153.33	1.09	33.00	29.419	7501.874	103.776	-23.448	-1641.37	-22.706
13	172.25	167.17	1.13	34.22	30.243	7711.890	106.681	-23.448	-1641.37	-22.706
14	186.08	181.00	1.17	35.43	31.066	7921.905	109.586	-23.448	-1641.37	-22.706
15	199.92	194.83	1.17	35.43	31.066	7921.905	109.586	-23.448	-1641.37	-22.706
16	213.75	208.67	1.20	36.33	31.684	8079.417	117.488	-23.448	-1641.37	-23.868
Interstitial	229.00	223.92	1.20	36.33	31.684	8079.417	102.003	-23.448	-1641.37	-20.722
17	239.00	233.92	1.20	36.33	31.684	8079.417	125.231	-23.448	-1641.37	-25.441
18	260.00	254.92	1.28	38.76	33.331	8499.448	163.614	-23.448	-1641.37	-31.596
19	277.50	272.42	1.28	38.76	33.331	8499.448	146.615	-23.448	-1641.37	-28.314
Penthouse	294.50	289.42	1.28	38.76	33.331	8499.448	72.245	-23.448	-1641.37	-13.952

Figure 9: Wind Load Excel Sheet – North-South Direction

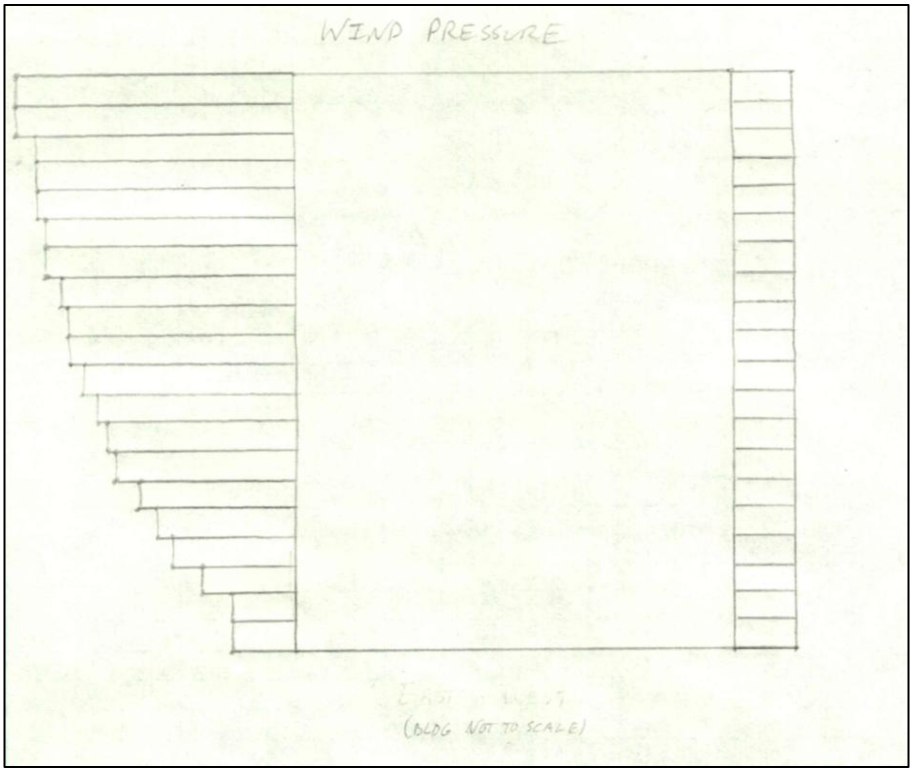

Figure 10: Wind Pressure Diagram

Floor	Force (k)	Height (ft)	Moment (k-ft)
1	17.257	0.00	0.00
2	33.172	15.91	527.92
3	33.624	16.37	550.32
4	35.618	18.36	653.97
5	36.615	19.36	708.77
6	38.210	20.95	800.59
7	39.007	21.75	848.41
8	39.605	22.35	885.11
9	40.203	22.95	922.52
10	41.200	23.94	986.47
11	42.197	24.94	1052.41
12	42.197	24.94	1052.41
13	42.995	25.74	1106.58
14	43.792	26.54	1162.03
15	43.792	26.54	1162.03
16	46.663	29.41	1372.20
Interstitial	40.513	23.26	942.16
17	49.739	32.48	1615.59
18	63.992	46.73	2990.63
19	57.343	40.09	2298.67
Penthouse	28.256	11.00	310.79
Total	855.990		21949.58

Floor	Force (k)	Height (ft)	Moment (k-ft)
1	48.098	0.00	0.00
2	92.455	44.36	4101.00
3	95.250	47.15	4491.31
4	102.513	54.42	5578.34
5	106.145	58.05	6161.42
6	111.955	63.86	7149.21
7	114.861	66.76	7668.42
8	117.040	68.94	8068.91
9	119.218	71.12	8478.90
10	122.850	74.75	9183.30
11	126.481	78.38	9914.09
12	126.481	78.38	9914.09
13	129.387	81.29	10517.70
14	132.292	84.19	11138.20
15	132.292	84.19	11138.20
16	141.356	93.26	13182.70
Interstitial	122.725	74.63	9158.60
17	150.672	102.57	15455.09
18	195.211	147.11	28718.00
19	174.929	126.83	22186.47
Penthouse	86.197	38.10	3284.03
Total	2548.409		205487.98

Figure 11: Wind Load Base Shear and Overturning Moment – East-West Direction (to the left), and North-South (to the right)

Seismic Load

For the seismic load evaluation of the Medical Research Building, the Equivalent Lateral Force Method as outlined in ASCE 7-05 was employed. The Site Class was determined to be A from Table 20.3-1 because the building sits on hard rock. An occupancy category of III resulted in an importance factor of 1.25 from Table 11.5-1. The Seismic Design Category based on short period response yielded Category B (Table 11.6-1), while the SDC based on 1 second period response yielded Category A (Table 11.6-2). To be conservative, Category B (the more severe category) was chosen. The Seismic Response Modification Factor, R, was labeled 4 on the drawings, which corresponds to the lateral resisting system of Ordinary Reinforced Concrete Shear Walls in Table 12.2-1.

The remainder of the procedure was followed resulting in a seismic base shear of approximately 980 kips. A spreadsheet developed in AE 597A was used to calculate the forces and moment at each floor as well as the overall overturning moment, calculated as 191,420 kip-ft.

Gravity Load Spot Checks

Typical Beam

A typical beam on the 15th floor, beam TB2, located between columns B1 and B2 and spanning 21'-0" was checked for strength and spacing. The beam was 24x36 and had 7-#7 bars on the bottom and 7-#6 bars on the top. The beam was found to pass for strength as well as minimum spacing and minimum width. The flexural capacity of the beam was determined to be approximately 15% above the maximum moment from the load.

Typical Column

A typical column on the 15th floor, column B4, 13'-10" tall was assessed for strength assuming that the column, under gravity loads, is subjected to pure axial compression. The column was 36x24 and contained 16-#7 bars. An excel spreadsheet was created to determine the axial load on the column. The beam was found to pass, as the determined compressive capacity of the column was over 3 times the calculated axial load on the column. This discrepancy is significant and could have resulted either from an error in load calculations or perhaps the column isn't only subjected to pure axial load. It should also be mentioned that the assumption was made that fs = fy. If this is not the case under the axial load, then that would affect the result of the axial capacity of the column, but due to the area of concrete being much greater than the area of steel, this difference isn't significant enough to correct the difference between axial capacity and calculated load.

Conclusion

A close examination of the Weill Cornell Medical Research Building revealed a beautiful building filled with complexity. This examination of the existing conditions, which included an evaluation of the structural systems as well as calculations of gravity and lateral loads and spot checks of typical members, resulted in the conclusion that the existing design is intricate and adequate.

Calculations of snow loads revealed that the roof live load was the controlling load case. The determination of seismic base shear as 979.6 kips is almost exactly the 980 kip base shear listed on the drawings under design conditions. Of note regarding the wind load calculations is that the building was designed with the old New York City Building Code, which called for a design wind speed of 98 mph, whereas the new building code, referencing ASCE 7, called for a design wind speed of 110 mph. Also, from the results tabulated a comparison can be made of base shear and overturning moment from seismic loads versus wind loads. The outcome of this comparison is significant because in the North-South direction wind loads control, but in the East-West direction the seismic loads cause a greater base shear and moment. This greatly influences the design of the lateral system.

Spot checks of gravity members revealed them to be adequate for strength and spacing, but it is still to be determined if they are subjected to lateral loads. Something to also be aware of is the importance of limiting vibrations. This will be a factor in analyzing floor systems in Technical Report 2 and will also be an important design consideration.

Appendix A:
Snow Load

Jonathan Coan | AE Senior Thesis | Snow Load | 1

ASCE 7-05 CH 7
Assume: Flat Roof

$p_f = .7 C_e C_t I p_g$ Fig. 7-1: $p_g = 25$ psf

Table 7.2 – $C_e = .9$
Category B, Fully exposed

Table 7.3 – $C_t = 1.0$

Table
Table 7.4 – $I = 1.1$
Category III

$p_f = .7(.9)(1.0)(1.1)(25) = $ **17.3 psf**

Appendix B:
Seismic Load

Jonathan Coan | AE Senior Thesis | Seismic Loads | 1

ASCE 7-05
Equivalent Lateral Force method

Fig 22-1: $S_S = 35\%g$ for NYC
Fig 22-2: $S_1 = 6\%g$ for NYC

Table 20.3-1: Site Class A (Hard Rock)

Table 11.4-1: $F_a = .8 \Rightarrow S_{MS} = F_a S_S = .8(.35) = .28$
Table 11.4-2: $F_v = .8 \Rightarrow S_{M1} = F_v S_1 = .8(.06) = .048$

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(.28) = .187$
$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(.048) = .032$

Table 1-1: Occupancy Category III
Table 11.5-1: $I = 1.25$

Table 11.6-1: SDC = B $\Rightarrow$ use SDC = B
Table 11.6-2: SDC = A

Table 12.2-1: $R = 4$

Table 12.8-2: $C_t = .02$, $x = .75$

$T = C_t h_n^x = .02(294.5)^{.75} = 1.42\text{ sec}$

Fig 22-15: $T_L = 6\text{ sec} \Rightarrow T < T_L$

$C_S = \frac{S_{DS}}{R/I} = \frac{.187}{\frac{4}{1.25}} = .058 < \frac{S_{D1}}{(R/I)T} = \frac{.032}{\frac{4}{1.25}(1.42)} = .007$

$C_S = .01$

calculated separately ↓

$V = C_S W = .01(97,960) = 979.6\text{ kips}$ Base Shear

$F_x = C_{vx} V \qquad C_{vx} = \frac{w_x h_x^k}{\sum_{i=1} w_i h_i^k}$

(by interpolation) $k = 1.46$

See spreadsheet on pg for the rest of the calculations

Level	Height (ft)	Weight (k)	$w*h^k$	C_{vx}	F_i (k)	V_i (k)	M (k-ft)
Penthouse	294.50	318.29	1281420	0.0106	10.42	10.42	3069.51
19	277.50	1669.14	6161082	0.0512	50.11	60.54	13906.32
18	260.00	4997.25	16772289	0.1393	136.42	196.96	35469.73
17	239.00	5402.93	16035778	0.1331	130.43	327.39	31173.11
Interstitial	229.00	3547.31	9891438	0.0821	80.45	407.84	18424.14
16	213.75	4091.69	10317278	0.0857	83.92	491.76	17937.56
15	199.92	4091.69	9357110	0.0777	76.11	567.87	15215.39
14	186.08	4091.69	8427041	0.0700	68.54	636.41	12754.83
13	172.25	4091.69	7528261	0.0625	61.23	697.65	10547.42
12	158.42	4091.69	6662105	0.0553	54.19	751.84	8584.29
11	144.58	4091.69	5830084	0.0484	47.42	799.26	6856.23
10	130.75	4091.69	5033929	0.0418	40.94	840.20	5353.54
9	116.92	4091.69	4275646	0.0355	34.78	874.98	4066.03
8	103.08	4091.69	3557602	0.0295	28.94	903.92	2982.90
7	89.25	4091.69	2882639	0.0239	23.45	927.36	2092.62
6	75.42	4091.69	2254263	0.0187	18.34	945.70	1382.81
5	61.58	4091.69	1676944	0.0139	13.64	959.34	839.99
4	47.75	4214.07	1191249	0.0099	9.69	969.03	462.67
3	33.92	4598.03	788815.4	0.0065	6.42	975.44	217.61
2	20.08	6402.62	511090.6	0.0042	4.16	979.60	83.49
				Base Shear:	979.60	Total Mom:	191420.19

Appendix C:
Wind Load

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ASCE 7-05

Basic Wind Speed (Fig 6-1C): 110mph
Exposure Category (From Plans): B
Occupancy Category (Table 1-1): III

K_z = See spreadsheet
$K_{zt} = 1.0$
$K_d = .85$ (Table 6-4)
$V = 110$ mph
$I = 1.15$ (Table 6-1)

$q_z = .00256\, K_z K_{zt} K_d V^2 I$
See spreadsheet for calculations

Assume: Rigid Structure $\Rightarrow G = .85$
Figure 6-5: $GC_{pi} = \pm .18$
Figure 6-6: $C_p = .8$ (windward)
$C_p = -.5$ (leeward) N-S, $-.218$ E-W (from interpolation ($L/B = 3.64$))
$p = q\, GC_p - q_h (GC_{pi})$

See spreadsheet for calculations

Appendix D: Spot Checks

Typical Beam

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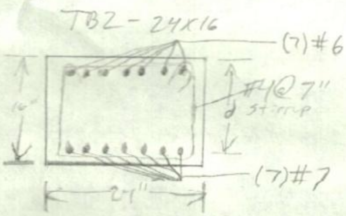
Typical Beam

TB2 - 24X16



Assume Trib Area extends halfway to each adjacent column in the N-S direction and along the span in the E-W direction

Spans Col. B1 + B2 (21'-0")

Let $d = 16 - 1.5 - \frac{4}{8} - \frac{1}{2}\left(\frac{7}{8}\right) = 13.56''$ (1.5 = cc, 4/8 = stirrup)

Loads

Superimposed Dead Load: 27 psf
Ceiling + Mech: 12 psf
Partitions: 10 psf
Misc.: 5 psf

Self-weight: $\frac{16(24)}{144}(150) = 400\,plf$

$w_D = 400 + (12+10+5)\left(\frac{22'7''}{2} + \frac{25'}{2}\right) = 1.04\,klf$

Live Load

$w_L = 150\,psf\left(\frac{22'-7''}{2} + \frac{25'}{2}\right) = 3.57\,klf$

$w_u = 1.2w_D + 1.6w_L = 6.96\,klf$

Assume: Fixed ends

$M_u = \frac{w_u l^2}{12} = \frac{6.96(21)^2}{12} = 255.8\,k\text{-}ft$

Beam

$A_s = 7(.6) + 7(.44) = 7.28\,in^2$

$f'_c = 4000\,psi \Rightarrow \beta_1 = .85$

$A_{s,min} = \frac{3\sqrt{f'_c}}{f_y} b_w d \geq \frac{200 b_w d}{f_y}$

$A_{s,min} = \frac{3\sqrt{4000}}{60000}(24)(13.56) \geq \frac{200(24)(13.56)}{60000}$

$A_{s,min} = 1.03\,in^2 \geq 1.08\,in^2$

$A_{s,min} = 1.08\,in^2 < 7.28\,in^2$ ok

Min Spacing = max { $d_b = \frac{7}{8} = .875''$; 1" ; 4/3 Agg size = 1.33" (assume 1") }

$b_{min} = 2(1.5) + 2\left(\frac{4}{8}\right) + 7\left(\frac{7}{8}\right) + 6(1.33) = 18.1'' < b_w = 24''$ ok

Typical Beam (cont'd)

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Beam Strength

Assume $f_s > f_y$

$$a = \frac{A_s f_y}{.85 f'_c b} = \frac{7.28(60)}{.85(4)(24)} = 5.35\text{ in}$$

$$c = \frac{a}{\beta_1} = \frac{5.35}{.85} = 6.29\text{ in}$$

$$\varepsilon_s = \frac{\varepsilon_u}{c}(d-c) = \frac{.003}{6.29}(13.56-6.29) = .00347 > \varepsilon_y = .002 \text{ ok}$$

$$\varepsilon_t = .00347 \Rightarrow \phi = .65 + .25\frac{\varepsilon_t - \varepsilon_y}{.005 - \varepsilon_y} = .65 + .25\frac{.00347 - .002}{.005 - .002}$$

$$\phi = .74$$

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right) = .74(7.28)(60)\left(13.56 - \frac{5.35}{2}\right)$$

$$\phi M_n = \boxed{293.2\text{ k-ft}}$$

$$\phi M_n = 293.2\text{ k-ft} > M_u = 255.8\text{ k-ft} \quad \text{ok}$$

Beam Passes

Typical Column

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Typical Column

Column 13-7.0 on the 15th Floor
36x24
(16)#7

Loads on Column

	Dead (psf)	Live (psf)
19th Floor:	105	30
18th Floor:	57	150
17th Floor:	97	150
Interstitial:	47	150
16th Floor:	27	60

Loads calculated in spreadsheet on next page

Axial Load on Column = 941.9 kips

Compressive Strength: Assume: $\phi = .9$

$A_{st} = 16(.6) = 9.6 \text{ in}^2$

$f'_c = 4000 \text{ psi}$

Assume: $f_y = 60000 \text{ psi}$

$\phi P_o = .9(.85) f'_c (bh - A_{st}) + A_{st} f_y$

$\phi P_o = .9(.85)(4)(36(24) - 9.6) + 9.6(60)$

$\phi P_o = 3190 \text{ kips}$

$\phi P_o = 3190 k > P_u = 941.9 k$ ok

Appendix E:
Lateral Force Resisting Elements