TECHNICAL REPORT I

Gen*NY*Sis Center for Excellence in Cancer Genomics

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Structural Option

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The Gen*NY*Sis Center for Excellence is the new signature building of University at Albany's East Campus. The conditions of the site prior to construction included the old Sterling Winthrop Facility just off the Columbia Turnpike in East Greenbush, NY. A four-story steel framed laboratory, the Cancer Research Center falls on 117,400 square feet of space with about 26,000 square feet per floor. The Ground Floor is mostly below grade and houses laboratory space, an animal facility, mechanical rooms, and a loading dock. Just above on the First Floor, there is more laboratory space, offices, public space and a seminar room. The remaining Second and Third floors accommodate additional offices and laboratories.

The structural system is comprised of conventional framing with composite decking and composite steel beams at the floor levels and the roof. Column placement along exterior walls and on both sides of a ten-foot wide corridor allows for minimized foot-traffic vibration from the corridor to adjacent lab spaces and maximizes vertical space in the corridor. This column grid creates bays sizes of 21-feet by 27-feet. Upon exploration, structural steel was selected over reinforced concrete.

The following is a technical report on the structural steel system through detailed descriptions of the foundation, floor, column and lateral systems.

Foundation

The geotechnical report indicates that the allowable bearing capacity is 4000 psf. Typical column footings are 9-feet square 25-inches deep calling for 11#9 reinforcing bars each way on bottom. Typical continuous wall footings are 1-foot deep by 2-feet wide calling for 3#5 continuous bars and 1#5 bar at 12-inches on center, transverse. The 20-inch thick basement walls retain 20-feet of soil (see diagram for reinforcement). Typical slab-on-grade is 5-inch thick with steel fiber reinforcement. The mechanical room slabs are 6-inch thick with steel fiber reinforcement. All steel fibers in slab-on-grade are at 30 pounds/cy. Weights for cast-in-place concrete, footings, foundation walls and piers, and slabs on metal deck are 4000 psi, 3000 psi, 4000 psi, and 3500 psi, respectively.

Floor Framing

Typical floor framing includes 2-inch, 20-gauge, galvanized composite metal deck with 4½-inches normal weight concrete (total slab thickness of 6½-inches) with 6x6-W2.9xW2.9 wire welded fabric. Normal weight concrete was chosen over lightweight for vibration control. The structural steel used has a weight of 8 psf of floor area. Typical floor beams are W16x31 spaced 7-feet apart and 20 shear connectors. Filler beam across the 10-foot corridor are W10x12 spaced 7-feet apart. Girders along the interior column lines and along the exterior walls are W18x35 with 32 shear connectors. Camber will not be accounted for due to relatively short spans. Atypical framing is located in the lobby and offices along the North wall. Transfer girders are required in the lobby and mechanical rooms along the North wall to maintain column-free areas. Offices along the North wall are cantilevered over columns along the First Floor terrace.

Roof

To satisfy the extra HVAC loading on the roof, a concrete slab is set on the metal deck framing that is supported by steel beams. The 6½-inch slab is on 2-inch, 20-gauge, galvanized composite metal deck with 4½-inches of normal weight concrete reinforced with 6x6xW2.9xW2.9 wire welded fabric. Roof framing supports a screen wall set back from the face of the building, extending 15 to 20-feet above the roof slab. Typical roof framing filler beams are W16x31 spaced 7-feet apart with 20 shear connectors. Deeper beams will be required at bearing points of the penthouse posts. Filler beams spanning the corridor bay will be W10x12 spaced 7-feet apart with no shear connectors. Girders along the interior column lines and along the exterior walls will be W18x40 with 32 shear connectors. The structural steel used in the Main Roof framing is 10 psf of roof area. Penthouses on the roof have cross-braced steel-frames supporting steel joists and 1½", 22-gauge, galvanized, wide-rib (type B) roof deck. The structural steel used in the Penthouse Roof framing is 5 psf of penthouse area.

Columns

Typical columns are W12x72 members at the lower tier and W12x53 members at the top tier. Using W12 columns as a minimum size simplifies fabrication of connections of beams framing into the columns, and allows the OSHA-required four anchor bolts to fit within the flanges at the base, minimizing base plate and pier sizes. A column splice with a bolted web and welded flanges is required 4-feet above the Second Floor for all columns. Perimeter columns will bear on piers 1-foot below the First Floor elevation of 195.0'. Interior columns will bear on footings 1-foot below the Ground Floor elevation of 175.0'.

Lateral Force Resisting System

Steel braced frames will resist wind and seismic lateral loads. An expansion joint at the intersection of the two building wings will isolate the two sections from each other. The expansion joint will require a row of columns along each side of the joint, with the building structures separated by a distance sufficient to provide seismic isolation—approximately 6 to 8-inches. Each building section has braced frames across the ends, and two bays of bracing along the length of each exterior wall. Bracing diagonals are tube-shaped steel members in non-moment-resisting eccentrically braced frames. The building is designed for wind loading drift criteria of H/400, including second order effects.

[1]All information extracted from "Structural Schematic Design Narrative" provided by Einhorn Yaffee Prescott Architecture & Engineering, P.C.

Materials[1] Gen*NY*Sis Tech Report I

Concrete	
Cast-in-place (general)	f'c = 4000 psi
Footings	f'c = 3000 psi
Foundation Walls and Piers	f'c = 4000 psi
Slabs on Metal Deck	f'c = 3500 psi
Steel	
Reinforcing Steel	ASTM A 615, Grade 60
Welded Wire Fabric	ASTM A 185
Structural Steel	ASTM 992, SQ
Steel Tubes	ASTM A 500, Grade B
Steel Plates and Angles	ASTM A36
Anchor Bolts	ASTM A 307
High Strength Bolts	ASTM A 325
Vapor Retarder	10 mil Polyethylene

List of Codes and Code Requirements[1] — Gen*NY*Sis Tech Report I

- ~ Building Code of New York State (The New York State Building Code is an replica of the IBC with amendments).
- ~ Minimum Design Loads for Buildings and Other Structures (ASCE 7)
- ~ Building Code Requirements for Reinforced Concrete (ACI 318)
- ~ Specifications for Structural Concrete for Buildings (ACE 301)
- ~ Specifications for Structural Steel Buildings (AISC)
- ~ Seismic Provisions for Structural Steel Buildings (AISC)
- ~ Code of Standard Practice for Steel Buildings and Bridges (AISC)
- ~ Structural Welding Code—Steel (AWS D1.1)

Gravity Loads[1] — Gen*NY*Sis Tech Report I

Dead Loads	
Construction Dead Load	
Concrete	150 pcf
Steel	490 pcf
Construction Dead Load	
Partitions	20 psf
M.E.P.	10 psf
Finishes	5 psf
Windows and Framing	20 psf
Roof	20 psf
Live Loads	
Laboratories	60 psf 70 psf for office/lab flexibility
Offices	70 psf
Lobbies	100 psf
First Floor Corridor	100 psf
Corridors above First Floor	80 psf
Stairs and Exits	100 psf
Seminar Room	100 psf
Catwalks	40 psf
Balcony/Terrace	100 psf
Mechanical Rooms	Weight of equipment

The Gen*NY*Sis Center for Excellence in Cancer Genomics has four floors. Grade level starts at an elevation of 176 feet on the northwest corner of the building and extends to 194 feet on the southeast corner of the building. To ease calculations and ensure the strength of the building, I took to assuming the entire building was above ground. The L-shape of the building I took into review as two rectangles and took the more conservative values for design.

For a layout of the structural system, please see Appendix B. The calculations can be found in Appendix C and D.

Wind

The Analytical Procedure was used from ASCE 7-05 Section 6 to calculate the wind loads. The mean roof height was determined based on the average roof heights without the mechanical equipment screens on the Penthouse.

Basic Wind Speed	V	90 mph
Importance Factor	I_w	1.0
Exposure Category		B
Roof Angle	Θ	1.07°
Height and Exposure Coefficient	λ	1.16
Building Period Coefficient	C_t	0.03
Effective Height Coefficient	x	0.75
Maximum Height Above Base	h_n	87 ft
Coefficient for Upper Limit on Period	C_u	1.70
Fundamental Period	T	1.454
Topographic Factor	K_{zt}	see Figure 7
Building Natural Frequency	n_1	0.69 Hz
Mean Hourly Wind Speed Factor	$\bar{b}$	0.45
Height Above Ground Level	z	87 ft
3-Sec Gust Speed Power Law Exponent	α	0.25
Basic Wind Speed	V	90 mph
Mean Hourly Speed	V_z	75.69
Integral Length Scale Factor	ℓ	320 ft
Integral Length Scale Power Law Exponent	$\bar{\epsilon}$	0.333
Resonant Response Factor	R	see Figure 8
Gust-Effect Factor	G_f	see Figure 9
Internal Pressure Coefficient	GC_{pi}	+/- 0.18
Windward External Pressure Coefficient	C_p	0.8
Leeward External Pressure Coefficient	C_p	-0.2/-0.5
Wind Pressure	p	see Figure 2

determined from seismic calcs in ASCE 7-05 Section 12.8.2

< 1 Hz therefore structure is flexible

Figure 1

Wind Pressure [p]

	Elevation	Height Above Base (ft)	q_z	N-S Windward (psf)	N-S Leeward (psf)	E-W Windward (psf)	E-W Leeward (psf)
Penthouse	263'-0"	87.00	20.0	17.63	-12.37	10.35	-12.32
Roof	242'-0"	66.00	18.2	16.37	-12.37	9.10	-12.32
3rd	226'-0"	50.00	17.0	15.33	-12.37	8.26	-12.32
2nd	210'-0"	34.00	14.5	13.77	-12.37	6.52	-12.32
1st	194'-0"	18.00	12.1	12.09	-12.37	4.84	-12.32

Figure 2

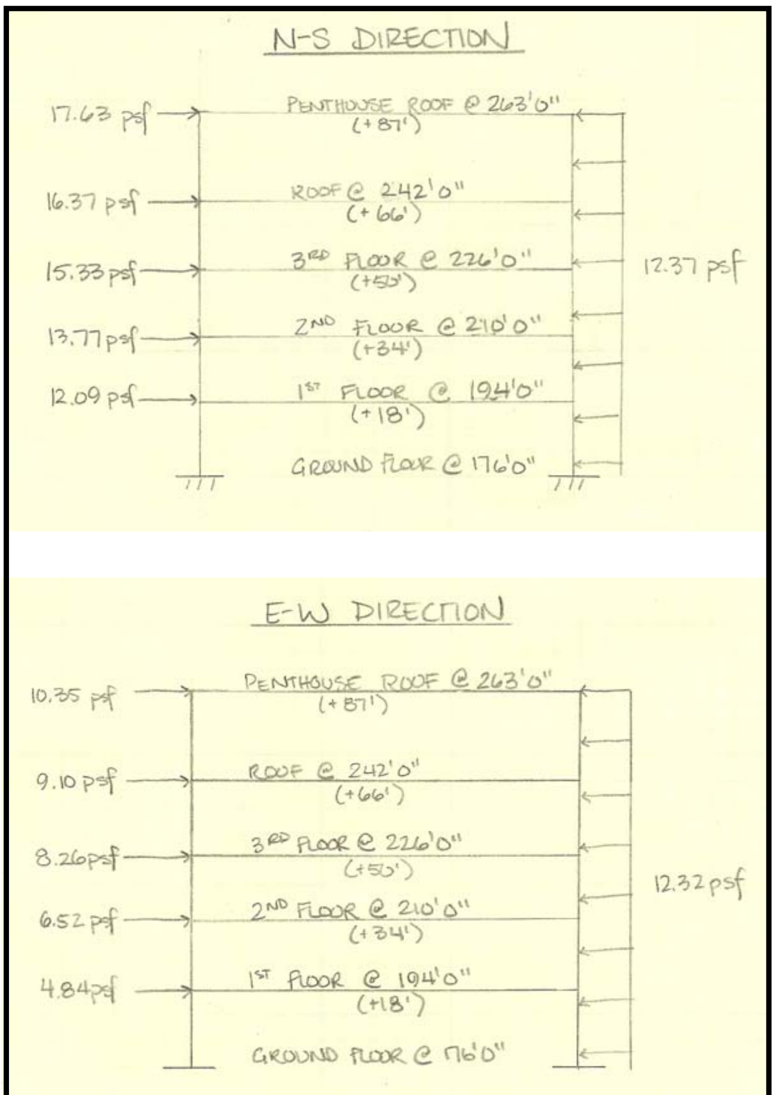


Figure 3

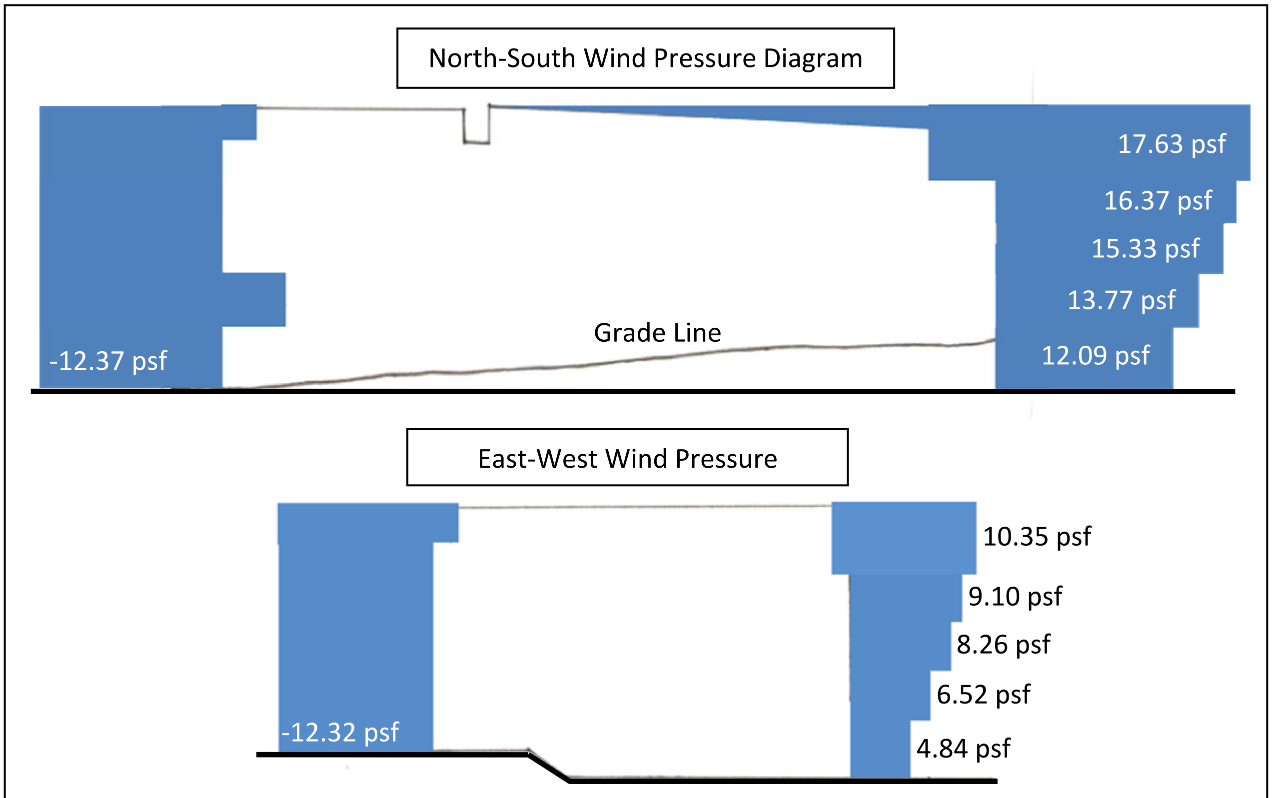


Figure 4

Base Shear and Overturning Moment

	Vert. Dist. To Base (ft)	Load (kips) N-S	Load (kips) E-W	Shear (kips) N-S	Shear (kips) E-W	Moment (ft-kips) N-S	Moment (ft-kips) E-W
Penthouse	87.0	69.3	71.4	0.0	0.0	6029	6212
Roof	66.0	50.6	51.4	69.3	71.4	3340	3392
3rd	50.0	48.8	49.4	119.9	122.8	2440	2470
2nd	34.0	46.0	45.2	168.7	172.2	1564	1537
1st	18.0	48.4	46.3	214.7	217.4	871	833
BASE	0.0	263.1	263.7	263.1	263.7	14244	14444

Figure 5

Figure 1 includes all values dependent on location, size, and overall plan. Figure 2 displays the wind pressures at each floor level. Figures 3 and 4 show the progression of wind pressure along the exterior of both the North-South and East-West elevations. The base shear, seen calculated in Figure 5, is 264^k on both the North-South and East-West elevations with an overturning moment of 14,244 ft-kips and 14,444 ft-kips in the Norht-South and East-West Elevation, respectively.

Seismic

The Equivalent Lateral Force Procedure was used from ASCE 7-05 Section 11 and 12 to calculate the seismic loads.

0.2-Sec Spectral Response Acceleration	S_s	0.220
1.0-Sec Spectral Response Acceleration	S_1	0.076
Site Class		C
Short-Period Site Coefficient	F_a	1.2
Long-Period Site Coefficient	F_v	1.7
MCE on Short Period	S_{MS}	0.264
MCE on Long Period	S_{M1}	0.129
Design Spectral Response Acceleration on Short Period	S_{DS}	0.176
Design Spectral Response Acceleration on Long Period	S_{D1}	0.086
Seismic Design Category		B
Importance Factor	I	1.0
Response Modification Coefficient	R	7.0
Building Period Coefficient	C_t	0.03
Effective Height Coefficient	x	0.75
Maximum Height Above Base	h_n	87 ft
Coefficient for Upper Limit on Period	C_u	1.70
Approximate Fundamental Period	T_a	0.855 sec
Fundamental Period	T	1.454 sec
Seismic Response Coefficient	C_s	0.0251 sec
Distribution Exponent	k	1.480
Building Weight	W	see Figure 10

Figure 6

Base Shear and Overturning Moment

	h_x (ft)	w_x (kips)	$h_x^k w_x$ (ft-kips)	C_{vx}	$F_x = C_{vx}V$ (kips)	V (kips)	$M = F_x h_x$ (ft-kips)
Penthouse	87.0	564	418569.5	0.164	45.8	0	3984
Roof	66.0	1921	947223.3	0.372	103.6	46	6840
3rd	50.0	1947	636563.2	0.250	69.6	149	3482
2nd	34.0	1913	353430.3	0.139	38.7	219	1315
1st	18.0	2644	190574.5	0.075	20.9	258	375
Ground	0.0	1738	0	0.000	0.0	279	0
Total		10727	2546361	1.000	278.6	279	15997

Figure 7

Figure 6 includes all values dependent on location, size, and overall plan. The base shear, seen calculated in Figure 7, is 279^k with an overturning moment of 15,997 ft-kips.

Lateral Load Conclusions

Wind load calculations determined the following:

$V = 264^k$ M = 14,444 ft-kips

Seismic load calculations determined the following:

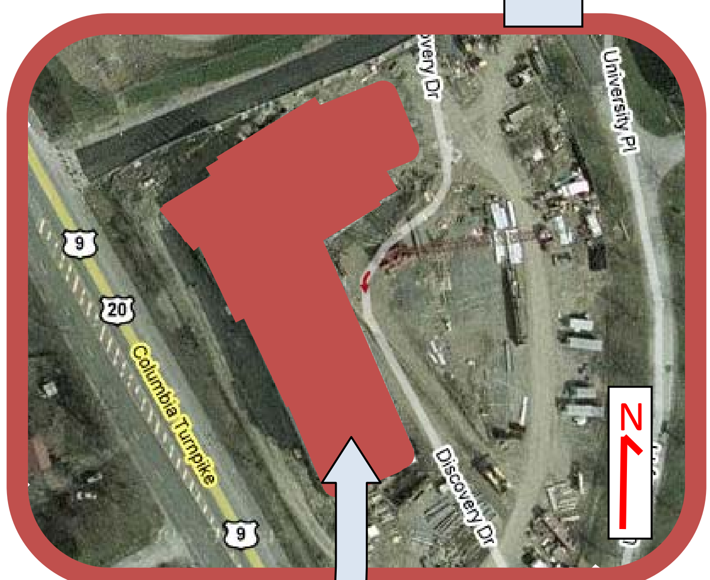
$V = 279^k$ M = 15,997 ft-kips

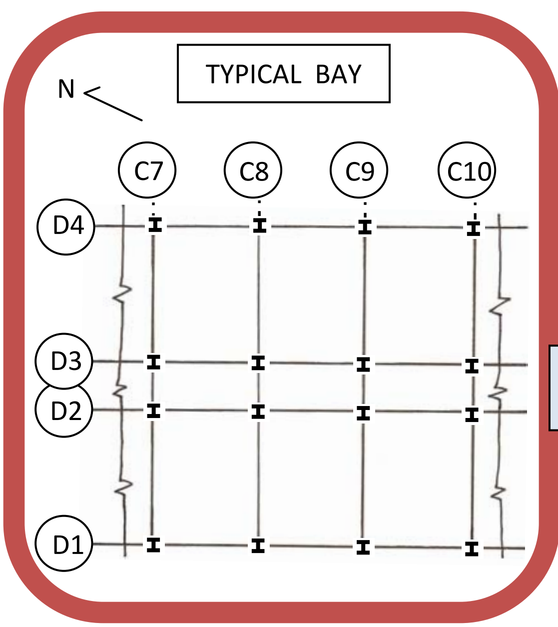
Therefore the seismic design load controls with my calculations. This is significantly lower than what the original designer calculated. One of the reasons for this is that the roof snow load (p_f) that the designer used was 50 psf while these calculations used a value of 30.8 psf. Furthermore, some of the loads for these calculations were taken from the designers (mechanical live load, terrace live load). Also, the lateral braces were not considered since there was not one on the typical bay that was analyzed. The stiffness of the braces effects the design values of the shear moment which could affect the governing loads (wind or seismic).

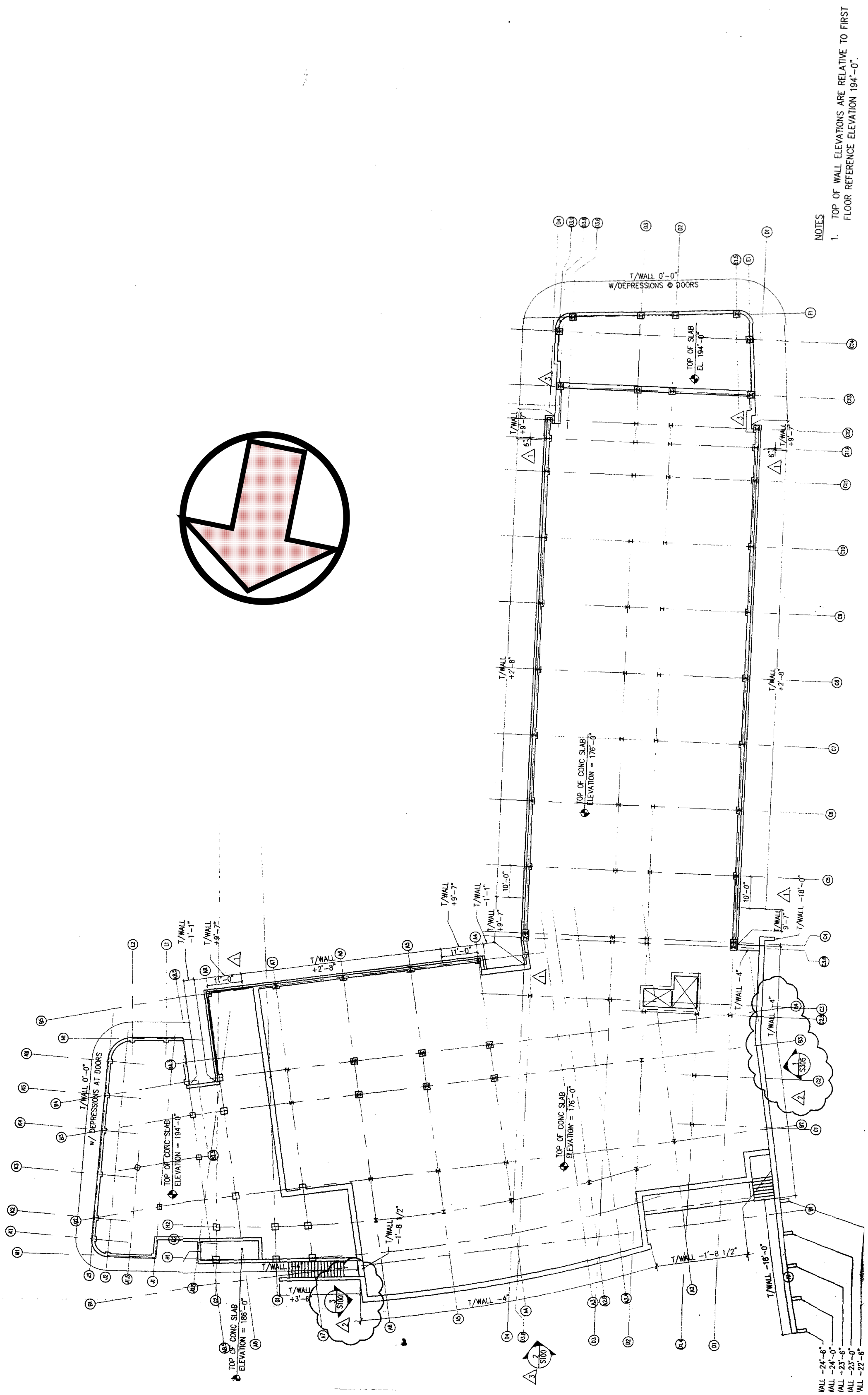
Conclusions

The structural system of Gen*NY*Sis Center for Excellence in Cancer Genomics has been analyzed in this first technical report. It consists of a composite slab-on-deck system with reinforced concrete columns, which sit on top of square piers. The lateral loads are resisted by eccentrically braced steel frames along the exterior walls. The envelope and façade are held up by the columns and girders of the structural frame. Through the spot checks (found in the Appendix H), I found that the members were oversized based on my low wind, seismic, and snow load calculations. All of the values were found a simple, preliminary analysis using applicable and most recent codes.


Sudbury
North Bay
Québec
Trois-Rivières
Sorel
Victoriaville
St-Georges
Drummondville
Repentigny
Ottawa
Terrebonne
Montréal
Sherbrooke
Châteauguay
St-Jean-Sur-Richelieu
Cornwall
Maine
Bangor
Augusta
Orillia
Peterborough
Barrie
Kingston
Vermont
Markham
Oshawa
Belleville
Portland
Bay City
Mississauga
Toronto
New Hampshire
Saginaw
Cambridge
Rome
Flint
Sarnia
Hamilton
St Catharines
Rochester
Utica
Manchester
Sterling Heights
London
New York
Albany
Buffalo
Livonia
Detroit
Massachusetts
Erie
Springfield
Providence
Toledo
Cleveland
Connecticut
Lorain
Akron
Scranton
Danbury
Bridgeport
Youngstown
Lima
Mansfield
Pennsylvania
Allentown
New York
Canton
Pittsburgh
Altoona

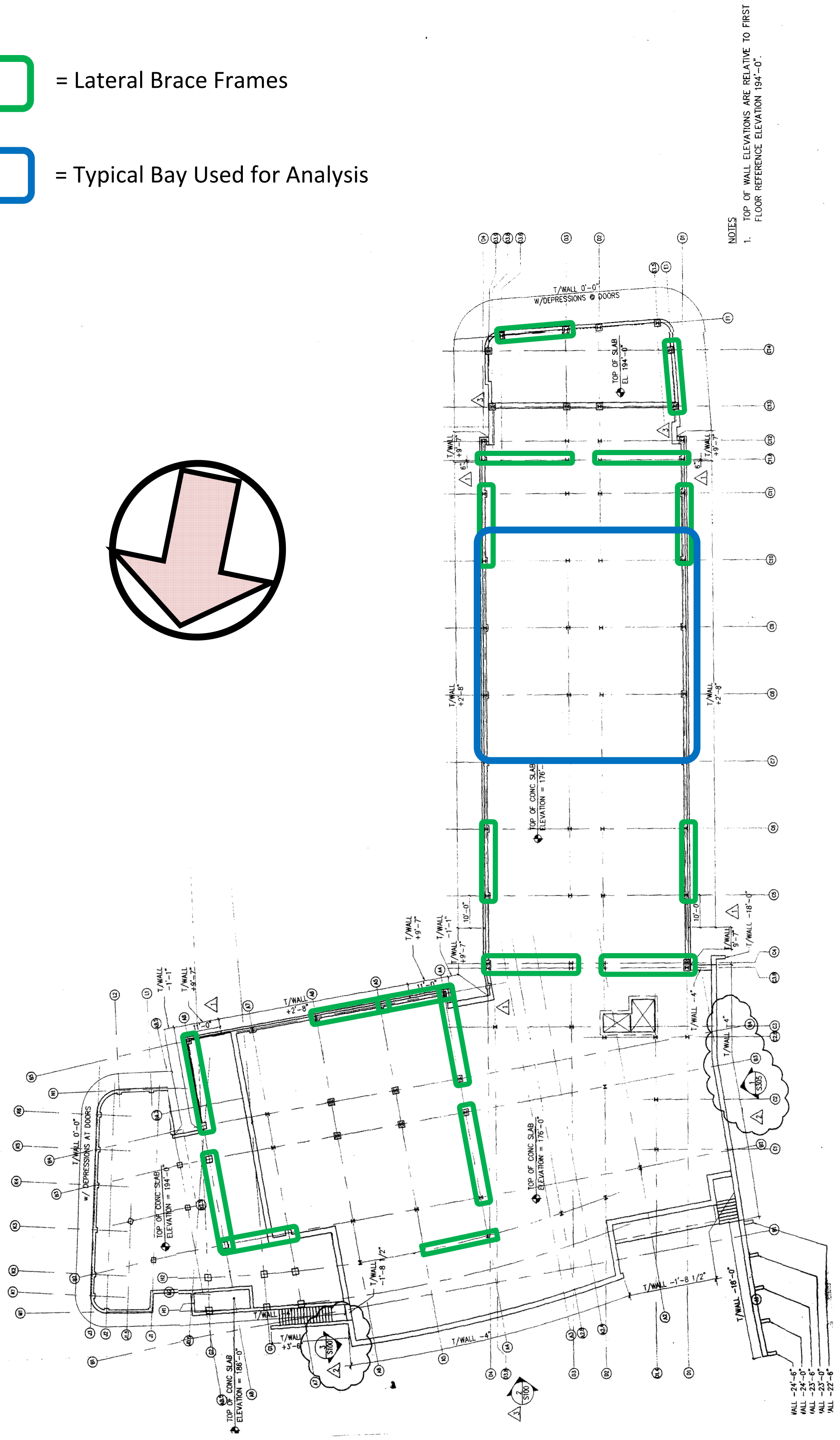

University Pl
Columbia Turnpike
Discovery Dr
9
20
N


TYPICAL BAY
N
C7
C8
C9
C10
D4
D3
D2
D1


NOTES
1. TOP OF WALL ELEVATIONS ARE RELATIVE TO FIRST FLOOR REFERENCE ELEVATION 194'-0".
TOP OF CONC SLAB ELEVATION = 176'-0"
TOP OF CONC SLAB ELEVATION = 194'-0"
TOP OF CONC SLAB ELEVATION = 186'-0"
T/WALL 0'-0" W/DEPRESSIONS @ DOORS

= Lateral Brace Frames

= Typical Bay Used for Analysis



Picture 1: Typical Structural Column on Pier

Picture 2: View of Structural Frame

Picture 3: Interior View of 1^{st} Floor

Picture 4: Typical Lateral Brace Connection

Picture 5: Typical Column-Girder Connection

Picture 6: Penthouse Mechanical Screen

GEN* NY*SIS CENTER FOR GENOMICS - WIND LOADS 1/4

ASCE 7-05 SECTION 6

ROOF ANGLE = $\tan^{-1}\left(\frac{5\text{ ft}}{268\text{ ft}}\right)$

BUILDING NATURAL FREQUENCY $[n_1]$:

$[C_t C_u h_n^x]^{-1}$

$[(0.03)(1.7)(87.09)^{0.75}]^{-1}$

$n_1 = 0.69$ Hz [SEE ALSO SEISMIC CALCS]

< 1.0 Hz ∴ FLEXIBLE

MEAN HOURLY SPEED $[\bar{V}_z]$:

$\bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right)$ TABLE 6-2: $\bar{b}, \bar{\alpha}$

$0.45\left(\frac{87}{33}\right)^{1/4}(90)\left(\frac{88}{60}\right)$

$\bar{V}_z = 75.7$ mph

INTEGRAL LENGTH SCALE OF TURBULENCE $[L_{\bar{z}}]$:

$\ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}}$

$320\text{ ft}\left(\frac{87\text{ ft}}{33}\right)^{1/3}$

$L_{\bar{z}} = 442.1$ ft

REDUCED FREQUENCY $[N_1]$:

$\frac{n_1 L_{\bar{z}}}{\bar{V}_z}$

$\frac{(0.69)(442.1)}{75.7}$

$N_1 = 4.03$ Hz

$\eta_h = \frac{4.6 n_1 h}{\bar{V}_z} = \frac{(4.6)(0.69)(50)}{75.69} = 2.097$

$R_h = \frac{1}{\eta_h} - \frac{1}{2\eta_h^2}\left(1 - e^{-2\eta_h}\right) = \frac{1}{2.097} - \frac{1}{2(2.097)^2}\left(1 - e^{-2(2.097)}\right)$

$= 0.365$

$\eta_B = \frac{4.6 n_1 E B}{\bar{V}_z} = \frac{(4.6)(0.69)(1.3)B}{75.69}$ 4 DIFFERENT VALUES 35, 110, 50, 150

GEN*NY*SIS CENTER FOR GENOMICS - WIND LOADS 2/4

VELOCITY PRESSURE EXPOSURE COEFFICIENTS [K_h]

80	0.93
87	K_h
90	0.96

$$\frac{K_h - 0.93}{87-80} = \frac{0.96-0.93}{90-80}$$

$K_h = 0.951$

COMPONENTS & CLADDING COEFFICIENT [GC_p]

$a_{N\text{-}S} = (0.1)(50) = 5 \leftarrow$

$(0.4)(50) = 20$

$a_{E\text{-}W} = (0.1)(35) = 3.5 \leftarrow$

$(0.4)(50) = 20$

WIND PRESSURES

$qGC_p - q_i(GC_{pi})$

q_z = certain height

$q_h = q(87)$

		q: 0.877, 0.855
N-S, WW, $+GC_{pi}$	(20)(0.877)(0.8) − (20)(0.18) =	10.43, 10.08
	(18.2)(0.877)(0.8) − (20)(0.18) =	9.17, 8.85
	(17)(0.877)(0.8) − (20)(0.18) =	8.32, 8.08
	(14.5)(0.877)(0.8) − (20)(0.18) =	6.57, 6.32
	(12.1)(0.877)(0.8) − (20)(0.18) =	4.89, 4.68
	(11.9)(0.877)(0.8) − (20)(0.18) =	4.75, 4.54
N-S, WW, $-GC_{pi}$	(20)(0.877)(0.8) − (20)(−0.18) =	17.63, 17.28
	(18.2)(0.877)(0.8) − (20)(−0.18) =	16.37, 16.05
	(17)(0.877)(0.8) − (20)(−0.18) =	15.53, 15.23
	(14.5)(0.877)(0.8) − (20)(−0.18) =	13.77, 13.52
	(12.1)(0.877)(0.8) − (20)(−0.18) =	12.09, 11.88
	(11.9)(0.877)(0.8) − (20)(−0.18) =	11.95, 11.74
N-S, LW, $+GC_{pi}$	(20)(0.877)(−0.5) − (20)(0.18) =	−12.37, −12.15
N-S, LW, $-GC_{pi}$	(20)(0.877)(−0.5) − (20)(−0.18) =	−5.17, −4.95
N-S, SW, $+GC_{pi}$	(20)(0.877)(−0.7) − (20)(0.18) =	−15.88, −15.57
N-S, SW, $-GC_{pi}$	(20)(0.877)(−0.7) − (20)(−0.18) =	−8.68, −8.37
		q: 0.872, 0.846
E-W, WW, $+GC_{pi}$	(20)(0.872)(0.8) − (20)(0.18) =	10.35, 9.92
	(18.2)(0.872)(0.8) − (20)(0.18) =	9.10, 8.70
	(17)(0.872)(0.8) − (20)(0.18) =	8.26, 7.89
	(14.5)(0.872)(0.8) − (20)(0.18) =	6.52, 6.20
	(12.1)(0.872)(0.8) − (20)(0.18) =	4.84, 4.58
	(11.9)(0.872)(0.8) − (20)(0.18) =	4.70, 4.44
E-W, WW, $-GC_{pi}$	(20)(0.872)(0.8) − (20)(−0.18) =	17.55, 17.12
	(18.2)(0.872)(0.8) − (20)(−0.18) =	16.30, 15.90
	(17)(0.872)(0.8) − (20)(−0.18) =	15.46, 15.09
	(14.5)(0.872)(0.8) − (20)(−0.18) =	13.72, 13.40
	(12.1)(0.872)(0.8) − (20)(−0.18) =	12.04, 11.78
	(11.9)(0.872)(0.8) − (20)(−0.18) =	11.90, 11.64
E-W, LW, $+GC_{pi}$	(20)(0.872)(−0.5) − (20)(0.18) =	−12.32, −12.05
E-W, LW, $-GC_{pi}$	(20)(0.872)(−0.5) − (20)(−0.18) =	−5.12, −4.85
E-W, SW, $+GC_{pi}$	(20)(0.872)(−0.7) − (20)(0.18) =	−15.81, −15.43
$-GC_{pi}$	(−0.18) =	−8.61, −8.23

GEN*NY*SIS CENTER FOR GENOMICS – WIND LOADS 3/4

PARAPET COEFFICIENTS [P_p]

$q_p GC_{pn}$

$(20.0)(+1.5)$, $(20.0)(-1.0)$

30 , -20

TOTAL WIND PRESSURES

N-S 110 ft 35 ft

- PENTHOUSE
  (17.63 psf + 12.37 psf)(21 ft)(110 ft) = 69.3^k 22.1^k
- ROOF
  (16.37 psf + 12.37 psf)(16 ft)(110 ft) = 50.6^k 16.1^k
- 3RD
  (15.33 psf + 12.37 psf)(16 ft)(110 ft) = 48.8^k 15.5^k
- 2ND
  (13.77 psf + 12.37 psf)(16 ft)(110 ft) = 46.0^k 14.6^k
- 1ST
  (12.09 psf + 12.37 psf)(18 ft)(110 ft) = 48.4^k 15.4^k

263.1^k 83.7^k

E-W

150 ft 50 ft

- PENTHOUSE
  (10.35 psf + 12.32 psf)(21 ft) B = 71.4^k 23.8^k
- ROOF
  (9.10 psf + 12.32 psf)(16 ft) B = 51.4^k 17.1^k
- 3RD
  (8.26 psf + 12.32 psf)(16 ft) B = 49.4^k 16.5^k
- 2ND
  (6.52 psf + 12.32 psf)(16 ft) B = 45.2^k 15.1^k
- 1ST
  (4.84 psf + 12.32 psf)(18 ft) B = 46.3^k 15.4^k

263.7^k 87.9^k

Topographic Factor [K_{zt}]

N-S	Elevation	Height Above Base (ft)	Height Above Ground (ft)	K_1 (0.75* 5/40)	K_2 (1- (12 /(1.4*40)))	K_3 (e^ - 0.0625z)	K_{zt}
Penthouse	263'-0"	87.00	78.00	0.09375	0.786	7.64E-03	1.00
Roof	242'-0"	66.00	54.00	0.09375	0.786	3.42E-02	1.01
3rd	226'-0"	50.00	38.00	0.09375	0.786	9.30E-02	1.01
2nd	210'-0"	34.00	22.00	0.09375	0.786	2.53E-01	1.04
1st	194'-0"	18.00	6.00	0.09375	0.786	6.87E-01	1.10
Grade	188'-0"	12.00	0.00	0.09375	0.786	1.00E+00	1.15
Ground	176'-0"	0.00	-12.00	0.09375	0.786	2.12E+00	1.34

E-W	Elevation	Height Above Base (ft)	Height Above Ground (ft)	K_1 (0.75* 4/20)	K_2 (1- (25 /(4*20)))	K_3 (e^ - 0.125z)	K_{zt}
Penthouse	263'-0"	87.00	78.00	0.15	0.6875	8.82E-01	1.19
Roof	242'-0"	66.00	54.00	0.15	0.6875	8.82E-01	1.19
3rd	226'-0"	50.00	38.00	0.15	0.6875	8.81E-01	1.19
2nd	210'-0"	34.00	22.00	0.15	0.6875	8.78E-01	1.19
1st	194'-0"	18.00	6.00	0.15	0.6875	8.71E-01	1.19
Grade	188'-0"	12.00	0.00	0.15	0.6875	8.66E-01	1.19
Ground	176'-0"	0.00	-12.00	0.15	0.6875	8.46E-01	1.18

Figure 7

Resonant Response Factor [R]

	N-S (long direction)	N-S (short direction)	E-W (long direction)	E-W (short direction)
N_1	4.02	4.02	4.02	4.02
R_n	0.0582	0.0582	0.0582	0.0582
h	50	50	50	50
η_h	2.09	2.09	2.09	2.09
R_h	0.366	0.366	0.366	0.366
B	35	110	150	50
η_B	1.90	5.98	8.16	2.72
R_B	0.390	0.153	0.115	0.300
L	150	50	110	35
η_L	21.01	7.00	15.41	4.90
R_L	0.046	0.133	0.063	0.183
β	1.0	1.0	1.0	1.0
R	0.068	0.044	0.037	0.063

Figure 8

Gust-Effect Factor [G_f]

	N-S (long direction)	N-S (short direction)	E-W (long direction)	E-W (short direction)
g_Q	3.4	3.4	3.4	3.4
g_v	3.4	3.4	3.4	3.4
g_R	4.10	4.10	4.10	4.10
Q	0.904	0.866	0.851	0.896
I_z	0.255	0.255	0.255	0.255
G_f	0.877	0.855	0.846	0.872

Figure 9

GEN*NY*SIS CENTER FOR GENOMICS - SEISMIC CALCS 1/2

ASCE 7-05 SECTION 11 & 12

SPECTRAL RESPONSE ACCELERATION, SHORT-PERIOD [S_{MS}]
ASCE 7-05 TABLE 11.4-1, FIGURE 22-1

$F_a S_s$

$(1.2)(22\%)$

$S_{MS} = 0.264$

SPECTRAL RESPONSE ACCELERATION, 1-SEC PERIOD [S_{M1}]
ASCE 7-05 TABLE 11.4-2, FIGURE 22-2

$F_v S_1$

$(1.7)(7.6\%)$

$S_{M1} = 0.129$

DESIGN SPECTRAL ACCELERATION, SHORT-PERIOD [S_{DS}]

$2/3\ S_{MS}$

$(2/3)(0.264)$

$S_{DS} = 0.176$ ⟶ ASCE 7-05 TABLE 11.6-1 : SEISMIC CATEGORY B

DESIGN SPECTRAL ACCELERATION, 1-SEC PERIOD [S_{D1}]

$2/3\ S_{M1}$

$(2/3)(0.129)$

$S_{D1} = 0.086$ ⟶ ASCE 7-05 TABLE 11.6-2 : SEISMIC CATEGORY B

SEISMIC RESPONSE COEFFICIENT [C_s]

$S_{DS}/(R/I)$

ASCE 7-05 TABLE 12.2-1 ⟶ ECCENTRICALLY BRACED STEEL FRAMES (B.2)

$R = 7.0$

$\Omega_0 = 2.0$

$C_d = 4.0$

$(0.176)/(7.0/1.0)$

$C_s = 0.0251 > 0.01$ ✓

APPROXIMATE FUNDAMENTAL PERIOD [T_a]
ASCE 7-05 TABLE 12.8-2 ⟶ ECCENTRICALLY BRACED STEEL FRAMES

$C_t = 0.03$

$x = 0.75$

$C_t h_n^x$

$(0.03)(87\ ft)^{0.75}$

$T_a = 0.855$ sec

GEN*NY*SIS CENTER FOR GENOMICS - SEISMIC CALCS 2/2

FUNDAMENTAL PERIOD [T]

$C_u T_a$

ASCE 7-05 TABLE 12.8-1 → $S_{D1} \le 0.1$, $C_u = 1.7$

(1.7)(0.855)

$T = 1.454$ sec

SEISMIC RESPONSE COEFFICIENT [C_s]

$T \le T_L$ → FIGURE 22-15: $T_L = 6$ sec

$(S_{D1} T_L) / T(R/I)$

$(0.086)(6) / (1.454)(7/1)$

$C_s = 0.0507 > 0.01$ ✓

$C_s =$ 0.0251 ← CONTROLS
0.0507

WEIGHTS

ACTUAL WEIGHT OF "HEAVY" MECHANICAL EQUIPMENT [W_M]

FROM PROJECT SPECS

5700 + 31616 + 1637 + 432 + 3155 lbs

$W_M = 43$ kips

SEISMIC BASE SHEAR [V]:

$C_s W$

$(0.0251)(11100^k)$

$V = 278.6^k$

	h_x	$T = (1.7)(0.03)(h_x)^{0.75}$	$C_s = \frac{S_{D1} T_L}{T(R/I)}$, $\frac{S_{D1}}{R/I}$	
PENTHOUSE	87'	1.453	0.0507	0.0251 CONTROLS FOR ALL
ROOF	66'	1.181	0.0624	
3RD	50'	0.959	0.0769	
2ND	34'	0.718	0.1027	
1ST	18'	0.446	0.1654	
GROUND	0'	0		

DISTRIBUTION EXPONENT [k]:

T	k
0.5	1
1.454	k
2.5	2

$\frac{k-1}{1.454-0.5} = \frac{2-1}{2.5-0.5}$

$k = 1.48$

Square Footage Breakdown by Floor

Ground Floor	4300 Mechanical, 18300 Floor	22600 SF
1st Floor	4300 Terrace, 18200 Floor	22500 SF
2nd Floor	900 Mechanical, 21600 Floor	22500 SF
3rd Floor	200 Terrace, 2300 Mechanical, 20000 Floor	22500 SF
Roof		22500 SF
Penthouse		4700 SF
		117400 SF

Figure 10

GEN*NY*SIS CENTER FOR GENOMICS - SNOW LOAD 1/1

GROUND SNOW LOAD [P_g]

ASCE 7-05 FIGURE 7-1 $\longrightarrow$ $P_g = 40$ psf

ROOF SLOPE [θ]

$\tan^{-1}(\text{RISE}/\text{RUN})$

$\tan^{-1}(5/268)$

$\theta = 1.07° < 5°$

FLAT ROOF SNOW LOAD [P_f]

$0.7 C_e I C_t P_g$

→ ASCE 7-05 TABLE 7-3

→ ASCE 7-05 TABLE 7-4

→ ASCE 7-05 TABLE 7-2

$0.7(1.0)(1.1)(1.0)(40\text{ psf})$

30.8 psf > 20I ✓

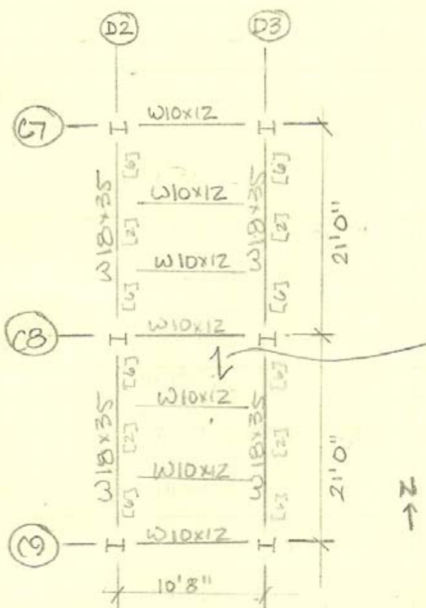
Figure 11

Beam Analysis

GEN*NY*SIS CENTER FOR GENOMICS – FRAME ANALYSIS 1/11

TYPICAL INTERIOR BAY

ANALYSIS OF TYPICAL INTERIOR BAY ON 1ST FLOOR



PLAN

$f'_c = 3500$ psi
$f_y = 50{,}000$ psi

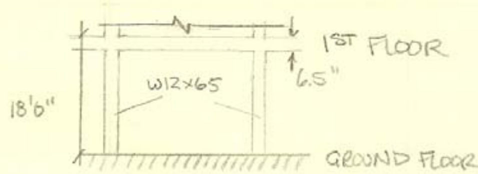
$A_T = (10.67')(7') = 75\ ft^2$

4-1/2" N.W.C.
2" 20 GAGE COMPOSITE METAL DECK
#4 @ 16" o.c.
6x6-W2.9xW2.9 w.w.f.

DEAD LOAD [DL]:
20 psf (PARTITIONS) + 10 psf (MEP) + 5 psf (FINISHES)
35 psf

SLAB SW = 6.5/12 ' x 150 pcf = 81.25 psf

120 psf



ELEVATION

LIVE LOAD [LL]:
70 psf (LAB)
100 psf (CORRIDOR)

100 psf

* $K_{LL} A_T = (4)(75) = 300\ ft^2 < 400$
∴ NO LL REDUCTION

BEAM CHECK - LRFD

FACTORED LOAD [W_U]:
[1.2 DL + 1.6 LL] x SPAN
[1.2 (120 psf) + 1.6 (100 psf)] x 7'

$W_U = 2128$ plf

DESIGN MOMENT [M_U]:
$w_u \ell^2 / 8$
$(2.128\ klf)(10.67')^2 / 8$

$M_U = 30.3$ k-ft

GEN*NY*SIS CENTER FOR GENOMICS – FRAME ANALYSIS 2/11

EFFECTIVE FLANGE WIDTH [b_{eff}]:

SPACING | SPAN/4 | MIN

$(7'+7')(12''/_{1})/2 = 84''$

$(21')(12''/_{1})/4 = 63''$

$b_{eff} = 63''$

CONCRETE IN COMPRESSION [V_c']:

$0.85 f'c \cdot b_{eff} \cdot t$

$(0.85)(3500\text{ psi})(63'')(6.5'')$

$V_c' = 1218^k$

STEEL IN TENSION [V_s']:

$A_s \cdot F_y$

$(3.54\text{ in}^2)(50\text{ ksi})$

$V_s' = 177^k$

$V_s' < V_c'$ ∴ STEEL CONTROLS

PNA IS AT OR ABOVE TOP OF FLANGE

STUD STRENGTHS [ΣQ_n]:

STEEL MANUAL 13TH EDITION TABLE 3-19 → $\Sigma Q_n = 177^k$ → $\phi M_n = 47.3$ k-ft

3-21 → $Q_n = 17.2^k$/stud

12 STUDS

DEPTH OF CONCRETE [a]:

$\Sigma Q_n / 0.85 f'c\, b_{eff}$

$177^k / (0.85)(3.5\text{ ksi})(63'')$

$a = 0.94''$

MOMENT ARM [$Y2$]:

$t - a/2$

$6.5'' - 0.94''/2$

$Y2 = 6.03''$

DESIGN MOMENT [ϕM_n]:

STEEL MANUAL 13TH EDITION TABLE 3-19 → $\phi M_n = 145$ k·ft

145 k·ft > 47.3 k·ft ✓

47.3 k·ft > 30.3 k·ft

GEN*NY*SIS CENTER FOR GENOMICS - FRAME ANALYSIS 3/11

LIMITING DEFLECTION [Δ]:

$\ell/360$

$10'8''/360$

$\Delta = 0.36''$

LOWER BOUND MOMENT OF INERTIA [I_{LB}]:

STEEL MANUAL 13TH EDITION TABLE 3-20 $\rightarrow I_{LB} = 265 \text{ in}^4$

CONSTRUCTION LOAD [w_c]:

$(t_s)(W_c)$

$[(4.5''/12)(150 \text{ pcf})] \times 7'$

$w_c = 420 \text{ plf}$

CALCULATED DEFLECTION [Δ_{CONST}]

$5w\ell^4/384EI$

$5(0.42 \text{ klf})(128'')^4 / 384(29000 \text{ ksi})(265 \text{ in}^4)$

$\Delta_{CONST} = 0.19''$

$\Delta_{CONST} < \Delta_{LIMITING}$ ✓

BUT CANNOT HOLD TOTAL LOAD

LIVE LOAD [w_L]:

$1.6(100 \text{ psf}) \times 7'$

$w_L = 1.12 \text{ klf}$

CALCULATED DEFECTION [Δ_{LL}]:

$5w\ell^4/384EI$

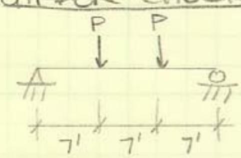
$5(1.12 \text{ klf})(128'')^4 / 384(29000 \text{ ksi})(265 \text{ in}^4)$

$\Delta_{LL} = 0.51''$

Girder Analysis

GEN*NY*SIS CENTER FOR GENOMICS - FRAME ANALYSIS 4/11

GIRDER CHECK - LRFD



CONCENTRATED LOAD [P]:

$w_u l / 2$

$(2.128 \text{ klf})(10.67')/2$

$P = 12^k$

MAXIMUM MOMENT [M_{max}]:

Pa

$(12^k)(7')$

$M_{max} = 84^k$

W18x35

STEEL MANUAL 13TH EDITION TABLE 3-19 → $\phi M_n = 249$ k-ft

$I = 510 \text{ in}^4$

$A_s = 10.3 \text{ in}^2$

CALCULATED DEFLECTION [Δ]:

$Pl^3 / 28EI$

$(12^k)(21 \times 12'')^3 / 28(29000 \text{ ksi})(510 \text{ in}^4)$

$\Delta = 0.46''$

LIMITING DEFLECTION [Δ_{LIMIT}]:

$l/360$

$21 \times 12/360$

$\Delta_{LIMIT} = 0.70''$

$\Delta < \Delta_{LIMIT}$ ✓

STUD STRENGTH [ΣQ_n]:

T=C ∴ PNA IS IN TOP OF FLANGE

STEEL MANUAL 13TH EDITION TABLE 3-19 → $\Sigma Q_n = 515^k$

TABLE 3-21 → $Q_n = 21.25$ k/stud

24 STUDS

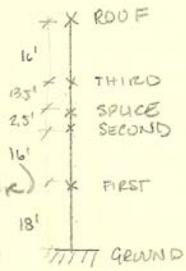
Column Analysis

GEN*NY*SIS CENTER FOR GENOMICS – FRAME ANALYSIS 5/11

COLUMN CHECK – LRFD

COLUMN @ D2-C8 [INTERIOR GRAVITY COLUMN]

W12x65
66'0" ABOVE GROUND FLOOR SLAB
SPLICE @ 36'6" ABOVE SLAB



TRIBUTARY AREA [A_T]:
$(\tfrac{1}{2}\text{SPAN}_L + \tfrac{1}{2}\text{SPAN}_R)(\tfrac{1}{2}\text{SPACING}_L + \tfrac{1}{2}\text{SPACING}_R)$
(10'6" + 10'6")(13'6" + 5'4")

$A_T = 396\ ft^2 \approx 400\ ft^2$

AREA OF INFLUENCE [A_I]:
$(\text{DIST TO COL}_N + \text{DIST TO COL}_S)(\text{DIST TO COL}_E + \text{DIST TO COL}_W)$
(21'0" + 21'0")(27'0 + 10'8")

$A_I = 1582\ ft^2 \approx 1590\ ft^2$

- ROOF COLUMN

DEAD LOAD [DL]:
85 psf (SEE SEISMIC CALCS)
43 kips (MECHANICAL EQUIPMENT)

LIVE LOAD [LL]:
20 psf (ASCE 7-05)
21 psf (ROOF SNOW LOAD)

WIND LOAD [WL]:
21 psf (ROOF SNOW LOAD)

AXIAL LOAD [P_U]:
$(1.2DL + 1.6LL + 0.5WL)A_T$
$((1.2)(85\ psf) + (1.6)(20 + 21\ psf) + 0.5(21\ psf))\ 400\ ft^2 + 1.2\ (43000\ lbs)$

$P_U = 122.8^k$

NO ROOF LIVE LOAD REDUCTION

MAX ALLOWABLE LOAD [ϕP_n]
STEEL MANUAL 13TH EDITION TABLE 4-1
KL = 16.0 ft

$\phi P_n = 639^k$

$\phi P_n > P_U$ ✓

GEN*NY*SIS CENTER FOR GENOMICS - FRAME ANALYSIS 6/11

- 3RD FLOOR COLUMN

DEAD LOAD [DL]:
85 psf (SEISMIC CALCS)

LIVE LOAD [LL]:
100 psf (ASCE 7-05)

LIVE LOAD REDUCTION [L]:
$L_o\left(0.25 + 15/\sqrt{A_T}\right)$
$L_o\left(0.25 + 15/\sqrt{1590}\right)$

$0.626L_o > 0.5L_o$ ✓

AXIAL LOAD [P_U]:
$(1.2DL + 1.6LL)A_T$
$[(1.2)(85\text{ psf}) + (1.6)(0.626)(100\text{psf})](400\text{ ft}^2)$

$P_U = 80.9^k + 122.8^k$
$= 203.7^k$

MAX ALLOWABLE LOAD [ϕP_n]:
STEEL MANUAL 13TH EDITION TABLE 4-1
KL = 13.5 ft

$\phi P_n = 685^k > 203.7^k = P_U$ ✓

- 2ND FLOOR COLUMN

DEAD LOAD [DL]:
85 psf (SEISMIC CALCS)

LIVE LOAD [LL]:
100 psf (ASCE 7-05)

LIVE LOAD REDUCTION [L]:
$0.626L_o$ (SAME AS ABOVE)

AXIAL LOAD [P_U]:
80.9^k (SAME AS ABOVE)
$P_U = 284.6^k$

MAX ALLOWABLE LOAD [ϕP_n]:
AISC 13TH EDITION TABLE 4-1
KL = 16.0 ft

$\phi P_n = 639^k$

$\phi P_n > P_U$ ✓

GEN*NY*SIS CENTER FOR GENOMICS- FRAME ANALYSIS 7/11

- 1^{ST} FLOOR COLUMN

DEAD LOAD [DL]:
85 psf (SEISMIC CALCS)

LIVE LOAD [LL]:
100 psf (ASCE 7-05)

LIVE LOAD REDUCTION [L]
$0.626 L_0$ (SAME AS ABOVE)

AXIAL LOAD [P_U]:
80.9 (SAME AS ABOVE)
$P_U = 365.5^k$

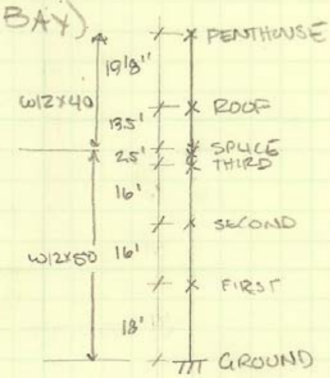
MAX ALLOWABLE LOAD [ϕP_n]:
AISC 13TH EDITION TABLE 4-1
KL = 18.0 ft

$\phi P_n = 615^k$

$\phi P_n > P_U$ ✓

* NO PENTHOUSE OR TERRACE LOADS ON THIS TYPICAL BAY AND DESIGNER USED DIFFERENT SNOW LOAD

COLUMN CHECK - LRFD (NOT ON TYP. BAY)



COLUMN @ D2-C13
W12 x 40 / W12x50
85'8" ABOVE GROUND FLOOR SLAB
SPLICE @ 52'6" ABOVE SLAB

TRIBUTARY AREA [A_T]:
$(\frac{1}{2}SPAN_L + \frac{1}{2}SPAN_R)(\frac{1}{2}SPACING_L + \frac{1}{2}SPACING_R)$
(13'6" + 5'4")(5'4" + 8'9")

$A_T = 266\ ft^2$

AREA OF INFLUENCE [A_I]:
$(DIST\ TO\ COL_N + DIST\ TO\ COL_S)(DIST\ TO\ COL_E + DIST\ TO\ COL_W)$
(10'8" + 17'6")(27'0" + 10'8")

$A_I = 1062\ ft^2$

GEN*NY*SIS CENTER FOR GENOMICS - FRAME ANALYSIS 8/11

- PENTHOUSE COLUMN
  DEAD LOAD [DL]:
  30 psf (DESIGNER LOAD - S400)

  LIVE LOAD [LL]:
  50 psf (DESIGNER LOAD - S400)

  AXIAL LOAD [P_U]:
  $(1.2DL + 1.6LL)A_T$
  $[(1.2)(30psf) + (1.6)(50psf)](266 ft^2)$

  $P_U = 30.9^k$

  MAX ALLOWABLE LOAD [ϕP_n]:
  AISC 13TH EDITION TABLE 4-1, W12x40
  KL = 19.67 ft

  $\phi P_n = 196^k$

  $\phi P_n > P_U$ ✓

- ROOF COLUMN
  DEAD LOAD [DL]:
  85 psf (DESIGNER LOAD - S400)
  43 kips (MECHANICAL EQUIP)

  LIVE LOAD [LL]:
  200 psf (DESIGNER LOAD - S400)

  OTHER LOADS:
  50 psf (DESIGNER LOAD - S400) SNOW LOAD
  30 psf (DESIGNER LOAD - S400) COMPONENT & CLADDING

  NO ROOF LIVE LOAD REDUCTION

  AXIAL LOAD
  $1.2DL + 1.6LL + 0.5(L_r, S)$
  $[(1.2)(85 psf) + (1.6)(200 psf) + (0.5)(50psf)](266 ft^2) + (1.2)(43000 lb)$

  $P_U = 170.5^k + 30.9^k$
  $= 201.4^k$

  MAX ALLOWABLE LOAD [ϕP_n]:
  AISC 13TH EDITION TABLE 4-1, W12x40
  KL = 13.5 ft

  $\phi P_n = 304^k$

  $\phi P_n > P_U$ ✓

GEN*NY*SIS CENTER FOR GENOMICS – FRAME ANALYSIS 9/11

• 3RD FLOOR COLUMN

DEAD LOAD [DL]:

85 psf (DESIGNER LOAD – S400)

LIVE LOAD [LL]:

80 psf (DESIGNER LOAD – S400)

LIVE LOAD REDUCTION [L]:

$L_0 (0.25 + 15/\sqrt{A_T})$

$L_0 (0.25 + 15/\sqrt{1062})$

$0.710 L_0 > 0.5 L_0$ ✓

AXIAL LOAD $[P_U]$:

$(1.2DL + 1.6LL) A_T$

$[(1.2)(85\text{ psf}) + (1.6)(0.710)(80\text{ psf})](266\text{ ft}^2)$

$P_U = 51.3^k + 201.4^k$
$= 252.7^k$

MAX ALLOWABLE LOAD $[\phi P_n]$:

AISC 13TH EDITION TABLE 4-1, W12×50

KL = 16.0 ft

$\phi P_n = 326^k$

$\phi P_n > P_U$ ✓

• 2ND FLOOR COLUMN

ALL LOADS SAME AS ABOVE

AXIAL LOAD $[P_U]$:

51.3^k (SAME AS ABOVE)

252.7^k

$P_U = 304.0^k$

MAX ALLOWABLE LOAD $[\phi P_n]$

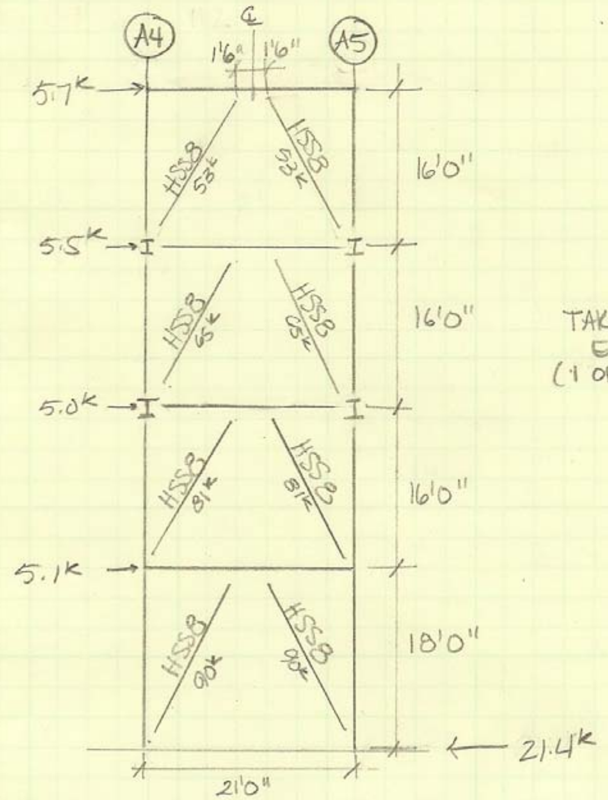
SAME AS ABOVE

$\phi P_n = 326^k$

$\phi P_n > P_U$ ✓

GEN*NY*SIS CENTER FOR GENOMICS – FRAME ANALYSIS 10/1

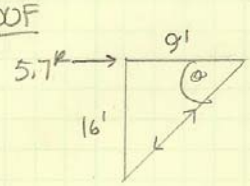
LATERAL BRACE CHECK



TAKES ON 1/3
E-W WIND LOAD
(1 OF 3 E-W BRACES)

LATERAL BRACE ON GRID H2

ROOF

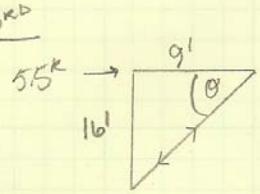


$\theta = 60.6°$

$\cos(60.6) = \frac{5.7}{P}$

$P = 11.6^K$

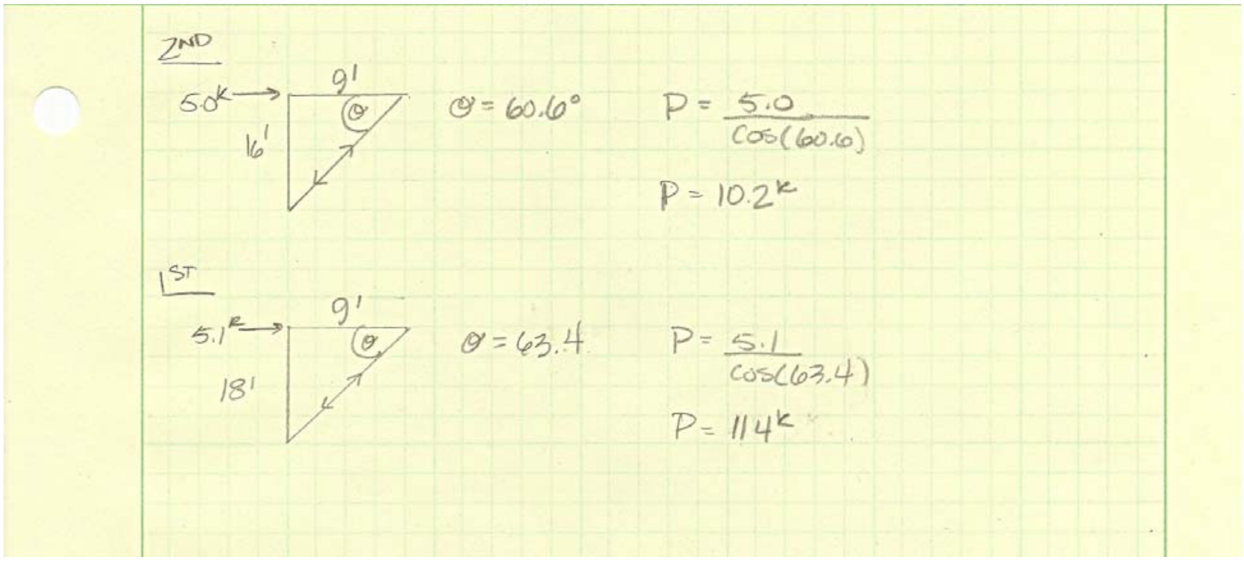
3RD



$\theta = 60.6$

$P = \frac{5.5}{\cos(60.6)}$

$P = 11.2^K$



The loads used for this analysis are significantly lower than the engineer's numbers and will be investigated more in a later technical report.