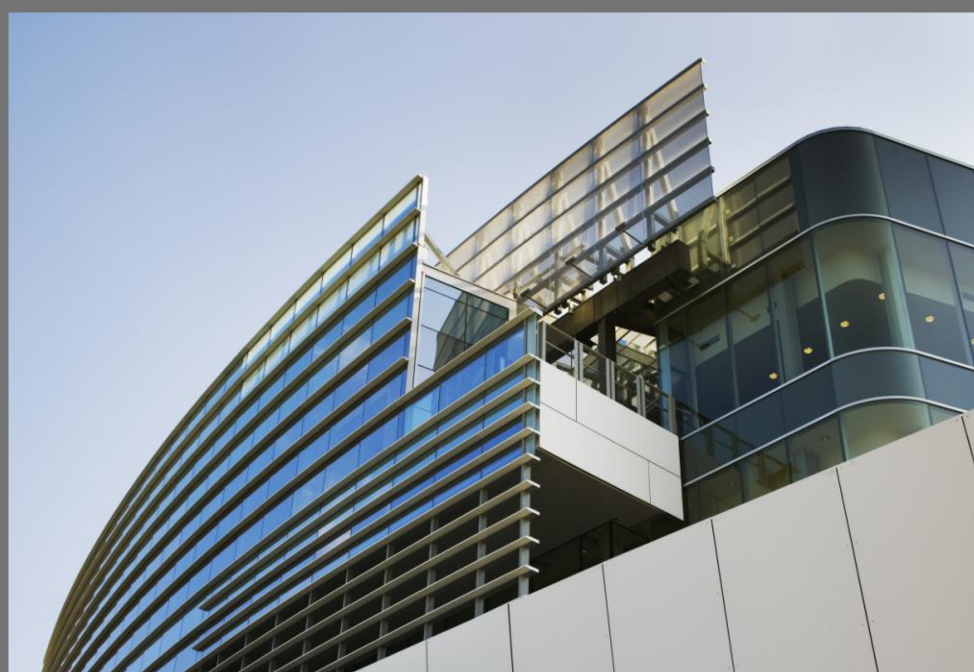
**TECHNICAL REPORT III**

# Gen*NY*Sis Center for Excellence in Cancer Genomics

Rensselaer, NY

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Structural Option

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This account details the lateral analysis of the Gen*NY*Sis Center for Excellence in Cancer Genomics. Using RAM Structural system and hand calculations, wind and seismic forces are applied to the lateral force resisting system, which consists of steel eccentrically, chevron braced frames. Using the analysis from the hand calculations and basic computer analysis (in Technical Report I), a more in-depth computer analysis has been compared and evaluated.

The outcome validates the original belief that the seismic forces govern in this case. Computer based shear values were smaller than the conservative hand calculated values. However, using the computer generated loads, strength and serviceability checks were completed to verify the sizes of the lateral braced frame members. Spot checks determined that some of the lateral bracing may have been oversized and therefore are a possibility for resizing. However, the building was able to meet drifting codes based on wind and seismic forces.

Gen*NY*Sis Center for Excellence in Cancer Genomics is University at Albany owned, state-funded medical research lab. Standing four stories tall with the first floor partially below grade, the Center for Genomics sits atop a hill with a beautiful outlook over Rensselaer, NY and the Hudson River. The Research Center houses research laboratories, offices, an animal facility, a seminar room, mechanical rooms and a loading dock.

As the signature building of University at Albany's East Campus Technology Park, the Research Center is a model for the co-location of academia, industry, and government. To signify its technological presence, a glass curtain wall and exposed frames promote a fresh, new look for the campus.

A main design goal was to maximize vertical space for utilities in the corridor and in the laboratories. Another concern was the minimization of vibration from foot-traffic in the corridor through the center of the building so a 100 psf live load was predominantly used for designing. The use of composite steel with concrete slab on deck forms the 117,400 square feet plan with a typical bay size of 21 feet by 27 feet. The lateral system is a series of braces frames spaced throughout the plan of the building.

This report examines the distribution of the lateral loads through the building frame of Gen*NY*Sis Center for Genomics. The ASCE 7-05 Code is used for load case input into RAM Structural System, in addition to hand calculations. Spot checks were performed on typical lateral brace members to confirm that computer strength design checks were appropriate.

**Figure 1:** ***Gen*NY*Sis Center for Excellence in Cancer Genomics overlooks***

Foundation

Bearing on fill and the indigenous soils was selected to simplify the excavation techniques. With this option, rock encountered above the desired footing elevation must be over-excavated 18-inches, and a fill "cushion" placed beneath the footings. The allowable bearing capacity of this foundation system is 4000 psf. Typical 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.

Floor Framing

The floor system of the Center for Genomics is composed of a composite steel system with a typical bay of 21 feet by 27 feet. It includes 2.0 inch, 20-gage composite decking with a 4.5" normal weight concrete slab, and ¾" diameter, 4" long studs. A 2 hour-rated construction is provided for all columns and beams supporting all floors. Typical floor beams (displayed in teal to the right) are W16x31 spaced 7-feet apart and 20 shear connectors. Filler beams 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 is not be accounted for due to relatively short spans.

**Figure 2:** ***$2^{nd}$ Floor Plan***

Lateral Force Resisting System

Steel braced frames (displayed in red above) 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 typically HSS8x8x5/16 in non-moment-resisting eccentrically braced frames.

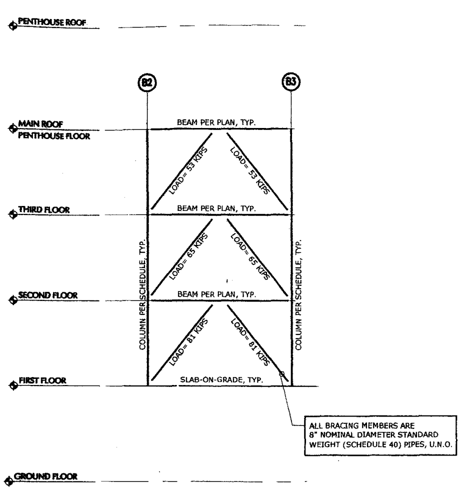



**Figure 3:** ***Typical Lateral Brace Frame***

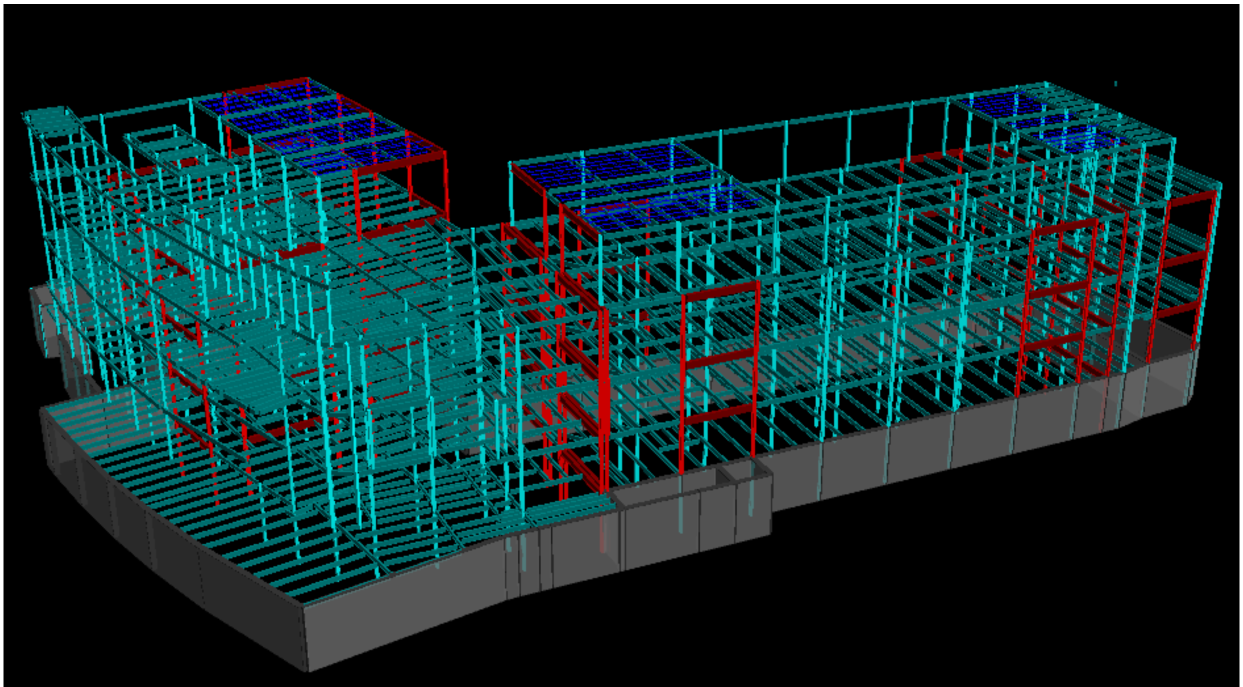


**Figure 4:** ***RAM 3-D View of Structural System with Lateral Bracing Highlighted*** **(Northwest corner)**

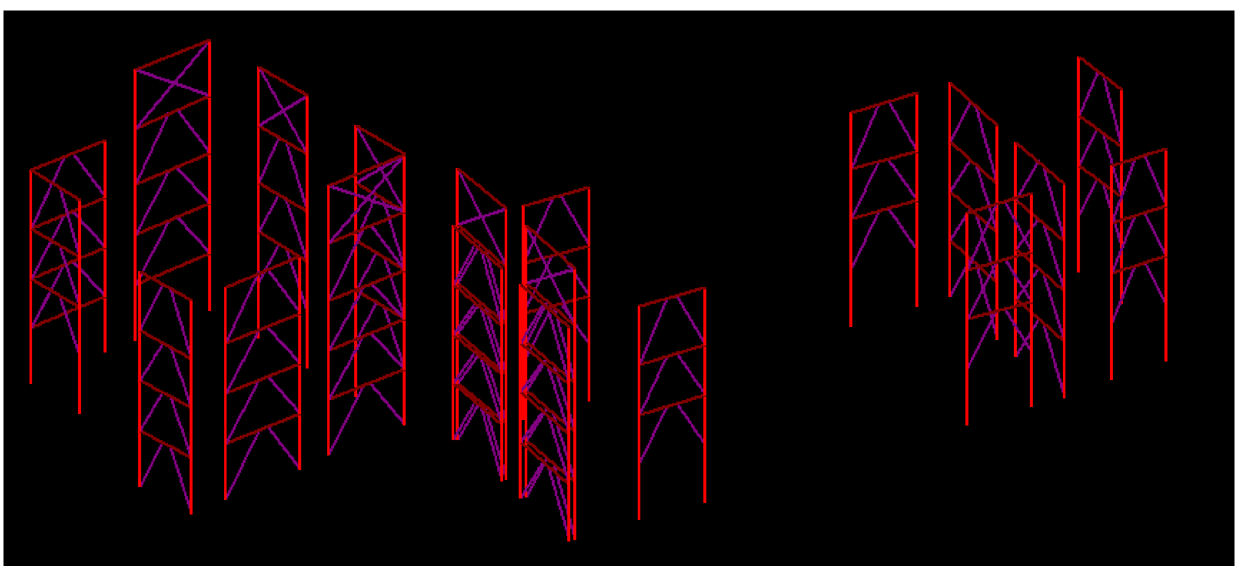


**Figure 5:** ***RAM 3-D View of Lateral Braces***

Codes and References

- ~ The Building Code of New York State (a replica of the IBC with amendments)
- ~ Minimum Design Loads for Buildings and Other Structures (ASCE 7-05)
- ~ The Building Code Requirements for Reinforced Concrete (ACI 318-05)
- ~ Specifications for Structural Steel Buildings (AISC $13^{th}$ Edition)
- ~ Seismic Provisions for Structural Steel Buildings (AISC $13^{th}$ Edition)
- ~ Code of Standard Practice for Steel Buildings and Bridges (AISC $13^{th}$ Edition)

LRFD Load Cases

1.4(Dead)

1.2(Dead) + 1.6(Live) + 0.5(Roof Live)

1.2(Dead) + 1.6(Roof Live) + 1.0(Live) or 0.8(Wind)

1.2(Dead) + 1.6(Wind) + 1.0(Live) + 0.5(Roof Live)

1.2(Dead) + 1.0(Seismic) + 1.0(Live)

0.9(Dead) + 1.6(Wind)

0.9(Dead) + 1.0(Seismic)

*Snow Loads were not considered in this analysis

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 |

## Wind Criteria

The Analytical Procedure was used from ASCE 7-05 Section 6.5 to calculate the wind loads. Below is a list of the major assumptions made for determining the building's wind loads.

Basic Wind Speed……………………………..90 mph
Exposure Category_________________B
Importance Factor_________________1.15
Building Category__________________II
Internal Pressure Coefficient_________+/- 0.18



## Seismic Criteria

The Equivalent Lateral Force Procedure was used from ASCE 7-05 Section 11 and 12 to calculate the seismic loads. Below is a list of the major assumptions made for determining the building's seismic loads.

Seismic Use Group________________II
Importance Factor ________________1.0
Mapped Spectral Response Accelerations
$S_S$__________________________0.220
$S_1$__________________________0.076
Site Class_________________________C
Site Class Factors
$F_a$__________________________1.2
$F_v$__________________________1.7
$S_{MS}$______________________________0.264g
$S_{M1}$______________________________0.129g
$S_{DS}$______________________________0.159g
$S_{D1}$______________________________0.073g
Seismic Design Category ____________B
Response Modification Factor (R) ____7.0
Nonmoment-Resisting Eccentrically Braced Frames
Seismic Period Coefficient ($C_t$)_______0.03
Seismic Response Coefficient ($C_s$) ____0.0251 sec
Period Coefficient (x)______________0.75

Distribution of lateral forces is based on frame relative stiffness. The composite deck and slab are treated as a rigid diaphragm, which designates load to each vertical lateral frame based upon this specified stiffness. The working point was set as (0, 0) at the intersection of A1-B1.

Hand Calculations

A 1.0 kips unit load was applied to each story level. The inverse of the deflection of caused by the unit load is then recorded as the relative stiffness. This is done at each of the five floors above the base of the structure. Using the stiffness of each frame (see Appendix F for Lateral Frame Number Labels), story shear can be determined for each frame. A simple calculation of the center of rigidity was calculated for the determination of wind story shears, and the center of mass for the seismic story shears. It was affirmed that the seismic forces govern for the lateral bracing, so overturning moment and torsion were further investigated.

The equation used to determine torsion was:

$$\frac{V.e.K_{SN}.C_N}{\Sigma(K_{SN}C_N^2)}$$

where V=story shear, e=eccentricity, $K_{SN}$=relative stiffness, and $C_N$=distance from frame to center of rigidity.

In calculating the center of rigidity and the center of mass, all slab openings and varying floor levels are ignored. There is a small change for the different floor layouts, but it was not accounted for in the computer model.

| | Frame Number | 2 | 3 | 6 | 7 | 13 | 14 | 15 | 16 | 19 | Sum |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Frame Stiffness | Penthouse | | 388 | 231 | | | | | | | 619 |
| | Roof | 303 | 239 | 152 | 294 | 200 | 200 | 200 | 200 | 138 | 1926 |
| | 3rd | 416 | 398 | 219 | 453 | 299 | 299 | 299 | 299 | 211 | 2893 |
| | 2nd | 485 | 500 | 255 | 500 | 357 | 357 | 357 | 357 | 261 | 3429 |

**Figure 6:** ***Frame Stiffness in East-West Frames***

| | Frame Number | 1 | 4 | 5 | 8 | 9 | 10 | 11 | 12 | 17 | 18 | 20 | Sum |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Frame Stiffness | Penthouse | | 71 | 231 | | | | 53 | 72 | | | | 427 |
| | Roof | 312 | 27 | 153 | 526 | 51 | 51 | 41 | 51 | 23 | 27 | 17 | 1279 |
| | 3rd | 425 | 27 | 219 | 952 | 55 | 55 | 46 | 53 | 20 | 21 | 16 | 1889 |
| | 2nd | 485 | 21 | 255 | 813 | 60 | 61 | 52 | 57 | 21 | 22 | 11 | 1858 |

**Figure 7:** ***Frame Stiffness in North-South Frames***

| Story Shear (kips) | Eccentricity, x (ft) | Eccentricity, y (ft) |
|---|---|---|
| 0.0 | 13.76 | 8.14 |
| 45.8 | 15.45 | 8.12 |
| 149.4 | 15.45 | 8.12 |
| 219.0 | 15.62 | 9.97 |

| | Frame Number | 2 | 3 | 6 | 7 | 13 | 14 | 15 | 16 | 19 | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Relative Stiffness | Penthouse | 0 | 0.63 | 0.37 | 0 | 0 | 0 | 0 | 0 | 0 | 1 |
| | Roof | 0.157 | 0.12 | 0.08 | 0.15 | 0.1 | 0.1 | 0.1 | 0.1 | 0.07 | 1 |
| | 3rd | 0.144 | 0.14 | 0.08 | 0.16 | 0.1 | 0.1 | 0.1 | 0.1 | 0.07 | 1 |
| | 2nd | 0.141 | 0.15 | 0.07 | 0.15 | 0.1 | 0.1 | 0.1 | 0.1 | 0.08 | 1 |
| | | 0.443 | 1.03 | 0.6 | 0.46 | 0.31 | 0.31 | 0.31 | 0.31 | 0.22 | |
| Distance to Center of Rigidity (ft) | | 131 | 110 | 63 | 93 | 107 | 7 | 129 | 73 | 165 | SUM |
| Torsional Force, x | Penthouse | 0 | 0.08 | 0.14 | 0 | 0 | 0 | 0 | 0 | 0 | 0.2 |
| | Roof | 1.931 | 0.77 | 1.48 | 2.52 | 2.29 | 34.3 | 1.9 | 3.35 | 1.4 | 49.9 |
| | 3rd | 5.757 | 2.78 | 4.62 | 8.45 | 7.43 | 111 | 6.16 | 10.9 | 4.64 | 162.0 |
| | 2nd | 8.393 | 4.36 | 6.73 | 11.7 | 11.1 | 166 | 9.2 | 16.3 | 7.17 | 241.1 |
| Torsional Force, y | Penthouse | 0 | 0.61 | 1.11 | 0 | 0 | 0 | 0 | 0 | 0 | 1.7 |
| | Roof | 1.015 | 0.14 | 0.26 | 0.51 | 0.34 | 0.34 | 0.34 | 0.34 | 0.24 | 3.5 |
| | 3rd | 3.026 | 0.15 | 0.25 | 0.52 | 0.34 | 0.34 | 0.34 | 0.34 | 0.24 | 5.6 |
| | 2nd | 5.357 | 0.2 | 0.31 | 0.6 | 0.43 | 0.43 | 0.43 | 0.43 | 0.31 | 8.5 |

**Figure 8:** ***Torsional Forces in North-South Frames***

| Story Shear (kips) | Eccentricity, x (ft) | Eccentricity, y (ft) |
|---|---|---|
| 0.0 | 13.76 | 8.14 |
| 45.8 | 15.45 | 8.12 |
| 149.4 | 15.45 | 8.12 |
| 219.0 | 15.62 | 9.97 |

| | Frame Number | 1 | 4 | 5 | 8 | 9 | 10 | 11 | 12 | 17 | 18 | 20 | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Relative Stiffness | Pent-house | 0 | 0.17 | 0.54 | 0 | 0 | 0 | 0.12 | 0.17 | 0 | 0 | 0 | 1 |
| | Roof | 0.24 | 0.02 | 0.12 | 0.41 | 0.04 | 0.04 | 0.03 | 0.04 | 0.02 | 0.02 | 0.01 | 1 |
| | 3rd | 0.22 | 0.01 | 0.12 | 0.5 | 0.03 | 0.03 | 0.02 | 0.03 | 0.01 | 0.01 | 0.01 | 1 |
| | 2nd | 0.26 | 0.01 | 0.14 | 0.44 | 0.03 | 0.03 | 0.03 | 0.03 | 0.01 | 0.01 | 0.01 | 1 |
| | | | | | | | | | | | | | |
| Distance to Center of Rigidity (ft) | | 121 | 45 | 77 | 119 | 38 | 67 | 36 | 66 | 125 | 137 | 161 | SUM |
| Torsional Force, x | Pent-house | 0 | 0.24 | 0.11 | 0 | 0 | 0 | 0.24 | 0.13 | 0 | 0 | 0 | 0.7 |
| | Roof | 1.98 | 1.57 | 1.19 | 1.81 | 7.34 | 4.2 | 3.14 | 1.58 | 2.54 | 2.72 | 1.94 | 30 |
| | 3rd | 5.96 | 3.47 | 3.78 | 7.24 | 17.5 | 10 | 7.78 | 3.63 | 4.87 | 4.67 | 4.04 | 723 |
| | 2nd | 10.2 | 4.07 | 6.62 | 9.32 | 28.8 | 16.7 | 13.3 | 5.88 | 7.71 | 7.38 | 4.18 | 114 |
| Torsional Force, y | Pent-house | 0 | 0.02 | 0.05 | 0 | 0 | 0 | 0.01 | 0.02 | 0 | 0 | 0 | 0.1 |
| | Roof | 1.04 | 0.09 | 0.51 | 1.76 | 0.17 | 0.17 | 0.14 | 0.17 | 0.08 | 0.09 | 0.06 | 4.3 |
| | 3rd | 3.13 | 0.2 | 1.61 | 7.02 | 0.41 | 0.41 | 0.34 | 0.39 | 0.15 | 0.15 | 0.12 | 14 |
| | 2nd | 6.54 | 0.28 | 3.44 | 11 | 0.81 | 0.82 | 0.7 | 0.77 | 0.28 | 0.3 | 0.15 | 25 |

**Figure 9:** ***Torsional Forces in East-West Frames***

RAM Structural System Model and Comparisons

RAM Structural System was used for a more detailed lateral analysis. Because the difference of each floor frame could be taken into account more precisely, the centers of rigidity and the centers of mass were more accurate. However, slab openings were still not included in the calculation. Also, the starting point for the RAM calculations was slightly different than the one used by hand.

| | Hand Calculations | | | | RAM Calculations | | | |
|---|---|---|---|---|---|---|---|---|
| Story | Center of Mass, x | Center of Mass, y | Center of Rigidity, x | Center of Rigidity, y | Center of Mass, x | Center of Mass, y | Center of Rigidity, x | Center of Rigidity, y |
| Penthouse | 26.80 | 129.60 | 70.36 | 111.84 | 31.57 | 133.90 | 78.31 | 126.02 |
| Main Roof | 102.90 | 85.20 | 111.54 | 89.16 | 111.89 | 76.36 | 96.72 | 86.37 |
| 3rd | 82.70 | 77.00 | 111.54 | 89.16 | 112.90 | 75.82 | 106.33 | 90.74 |
| 2nd | 82.70 | 77.00 | 111.54 | 89.16 | 105.47 | 88.64 | 114.56 | 92.58 |
| 1st | 76.20 | 87.50 | 62.56 | 82.62 | 92.85 | 96.32 | 123.82 | 90.38 |

**Figure 10:** ***Comparison of Hand Calculations and RAM Output***

Serviceability Check

The IBC was applied for the deflection criteria so as to compare drift by story and by overall drift. Wind drift was evaluated against $\Delta_{WIND}$= H/400. Seismic drift was evaluated against $\Delta_{SEISMIC}$= $0.025h_{SX}$.

| Floor | Story Height (ft) | Allowable Story Drift H/400 (in) | Story Drift (in) | Height Above Base (ft) | Allowable Drift H/400 (in) | Total Drift (in) |
|---|---|---|---|---|---|---|
| Penthouse | 18.42 | 0.55 | 0.48 | 87 | 2.61 | 1.72 |
| Roof | 18.58 | 0.56 | 0.34 | 68.58 | 2.06 | 1.24 |
| 3rd | 16.00 | 0.48 | 0.33 | 50 | 1.50 | 0.90 |
| 2nd | 16.00 | 0.48 | 0.32 | 34 | 1.02 | 0.57 |
| 1st | 18.00 | 0.54 | 0.32 | 18 | 0.54 | 0.25 |

**Figure 11:** ***Computed Wind Drift to Code Limit***

| Floor | Story Height (ft) | Allowable Drift $0.025h_{sx}$ (in) | Story Drift (in) | Height Above Base (ft) | Allowable Drift $0.025h_{sx}$ (in) | Total Drift (in) |
|---|---|---|---|---|---|---|
| Penthouse | 18.42 | 0.46 | 0.43 | 87.00 | 2.18 | 1.73 |
| Roof | 18.58 | 0.46 | 0.36 | 68.58 | 1.71 | 1.30 |
| 3rd | 16.00 | 0.40 | 0.36 | 50.00 | 1.25 | 0.94 |
| 2nd | 16.00 | 0.40 | 0.33 | 34.00 | 0.85 | 0.58 |
| 1st | 18.00 | 0.45 | 0.25 | 18.00 | 0.45 | 0.25 |

**Figure 12:** ***Computed Seismic Drift to Code Limit***

As seen here, overall drift is up to code and the all individual floors are acceptable as well. Because it seems as though some of the deflection is considerably lower than code, the braces and beams may be able to be decreased in size which could affect the overall pricing of the lateral system. Another possibility would be to decrease the length of the diagonal bracing by bringing the braced length of the top beams of each section closer to the exterior nodes of that beam.

Overturning Check

The overturning moments were evaluated to see if the wind forces would overpower the 4000 psf bearing capacity of the foundation system. By inspection, some of the foundations don't seem to be strong enough and must be re-evaluated.

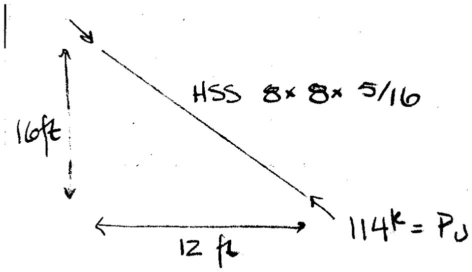
| Floor | Height Above Base (ft) | Wind Shear N/S (kips) | Calculated Moment (ft-kips) | Overturning Moment (ft-kips) | Wind Shear E/W (kips) | Calculated Moment (ft-kips) | Overturning Moment (ft-kips) |
|---|---|---|---|---|---|---|---|
| Penthouse | 87.00 | 69.37 | 6029 | 6035 | 71.43 | 6212 | 6214 |
| Roof | 68.58 | 32.62 | 3340 | 2237 | 60.04 | 3392 | 4118 |
| 3rd | 50.00 | 6.16 | 2440 | 308 | 32.36 | 2470 | 1618 |
| 2nd | 34.00 | 40.87 | 1564 | 1390 | 210.09 | 1537 | 7143 |
| 1st | 18.00 | 175.97 | 871 | 3167 | 272.68 | 833 | 4908 |

To validate the output given by RAM Structural System, some lateral members were strength-checked using the governing loads of those members.

GEN*NY*SIS CENTER FOR GENOMICS - TECH 3 SPOT CHECK 1/3

LATERAL BRACING LOAD CHECK

BRACE @ A8 (B2, B3)




$L = 20\text{ ft}$
$F_y = 46\text{ ksi}$
$A_g = 8.76\text{ in}^2$
$r = 3.13\text{ in}$

$$\frac{kL_x}{r} = \frac{(1.0)(20\text{ ft})}{3.13\text{ in}} = 76.7 < 200 \checkmark$$

$$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29000\text{ ksi}}{46\text{ ksi}}} = 118.3$$

$76.7 < 118.3 \therefore$ INELASTIC [AISC 13TH ED. EQ.3-2]

$$F_e = \frac{\pi^2 E}{\left(\frac{kL}{r}\right)^2} = \frac{\pi^2(29000\text{ ksi})}{(76.7)^2} = 48.7\text{ ksi}$$

$$F_{cr} = \left[0.658^{(F_y/F_e)}\right]F_y = \left[0.658^{(46\text{ ksi}/48.7\text{ ksi})}\right](46\text{ ksi}) = 31.0\text{ ksi}$$

$$\phi P_n = 0.85 A_g F_{cr} = 0.85(8.76\text{ in}^2)(31.0\text{ ksi}) = 231^k$$

$$\phi P_n > P_u \checkmark$$

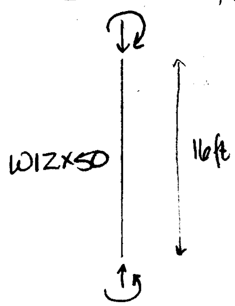
TABLE 4-4 [AISC 13TH ED.]

$$\phi P_n = 244^k \checkmark$$

$$\frac{P_u}{\phi P_n} = \frac{114^k}{231^k} = 0.49 < 1.0 \checkmark$$

GEN*NY*SIS CENTER FOR GENOMICS – TECH 3 SPOT CHECK 2/3

COL @ A8-B3, B2




$P_{u\,TOP} = 150^k$
$V_{u\,TOP} = 46^k$
$M_{u\,TOP} = 32\ ft \cdot k$

$P_{u\,BOT} = 150^k$
$V_{u\,BOT} = 46^k$
$M_{u\,BOT} = 54\ ft \cdot k$

$F_y = 50\ ksi$
$A_g = 14.6\ in^2$
$r_y = 1.96\ in$
$r_x = 5.17\ in$
$d = 12.2\ in$
$t_w = 0.370\ in$

$$\frac{KL_x}{r} = \frac{(1.0)(16\ ft)}{5.17\ in} = 37.1 < 200$$

$$\frac{KL_y}{r} = \frac{(1.0)(16\ ft)}{1.96\ in} = 98.0 < 200 \longleftarrow \text{CONTROLS}$$

$$\frac{KL}{r\pi}\sqrt{\frac{F_y}{E}} = \frac{98}{\pi}\sqrt{\frac{50\ ksi}{29000\ ksi}} = 1.3 < 1.5$$

$$\phi P_n = 0.9 F_{cr} A_g = 0.9\left(0.658\right)^{(1.3)^2}(50\ ksi)(14.6\ in^2) = 324^k$$

$\phi P_n > P_u$ ✓

$$\phi V_n = 0.9 F_y d t_w = 0.9(50\ ksi)(12.2\ in)(0.37\ in) = 203^k$$

$\phi V_n > V_u$ ✓

$B_{1x} = 1.0$

$B_{1y} = 1.0$

TABLE 4-1 [AISC 13TH ED.]

$L_p = 6.92\ ft < 16\ ft$
$L_r = 23.9\ ft > 16\ ft$

$C_b = 1.0$

TABLE 3-2 [AISC. 13TH ED.]

$\phi M_{px} = 270\ ft \cdot kips$
$\phi M_{rx} = 169\ ft \cdot kips$

$$\phi M_n = C_b\left[\phi M_p - (\phi M_p - \phi M_r)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] = 1.0\left[270 - (270 - 169)\left(\frac{16 - 6.92}{23.9 - 6.92}\right)\right]$$

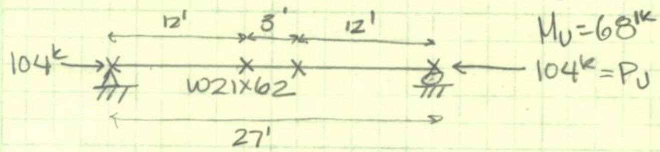
$$= 90\ ft \cdot kips$$

$\phi M_n > M_u$ ✓

GEN*NY*SIS CENTER FOR GENOMICS – TECH 3 SPOT CHECK 3/3

$\frac{P_u}{\phi P_n} = \frac{150^k}{324^k} = 0.43 > 0.2 \therefore$ [AISC 13TH ED.] EQ. H1-1A

$\frac{P_u}{\phi P_n} + \frac{8}{9}\left(\frac{M_u}{\phi M_n}\right) = 0.43 + \frac{8}{9}\left(\frac{54}{90}\right) = 0.96 < 1.0$ ✓




$V_u = 46^k$

$M_u = 68^{1k}$

$F_y = 50$ ksi

$A_g = 18.3 \text{ in}^2$

$d = 21$ in

$t_w = 0.40$ in

$r_x = 8.54$ in

$r_y = 1.77$ in

$\frac{kL_y}{r} = \frac{(1.0)(12 \text{ ft})}{1.77 \text{ in}} = 81.4 < 200$ ← CONTROLS

$\frac{kL_x}{r} = \frac{(1.0)(27 \text{ ft})}{8.54 \text{ in}} = 38.0 < 200$

$\phi V_n = \phi\, 0.6 F_y d t_w = (1.0)(0.6)(50 \text{ ksi})(21 \text{ in})(0.4 \text{ in}) = 252^k$

$\phi V_n > V_u$ ✓

$\frac{kL}{r\pi}\sqrt{\frac{F_y}{E}} = \frac{81.4}{\pi}\sqrt{\frac{50 \text{ ksi}}{29000 \text{ ksi}}} = 1.1 < 1.5$

$\phi P_n = \phi F_{cr} A_g = 0.85\left(0.658^{(1.1)^2}\right)(50 \text{ ksi})(18.3 \text{ in}^2) = 469^k$

$\phi P_n > P_u$ ✓

$L_b = 12$ ft

TABLE 6-1 [AISC 13TH ED.]

$p = 1.99 \times 10^{3}$ 1/kips

$pP_u = (1.99 \times 10^{-3})(104^k) = 0.21 < 1.0$ ✓ ∴ EQ. H1-1A

$b_x = 2.02 \times 10^{-3}$ 1/kip-ft

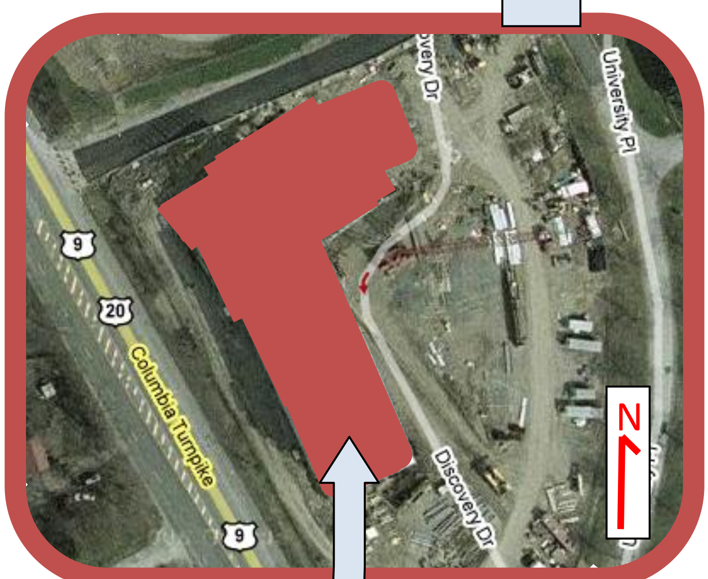
$0.21 + (2.02 \times 10^{-3})(68) = 0.35 < 1.0$ ✓

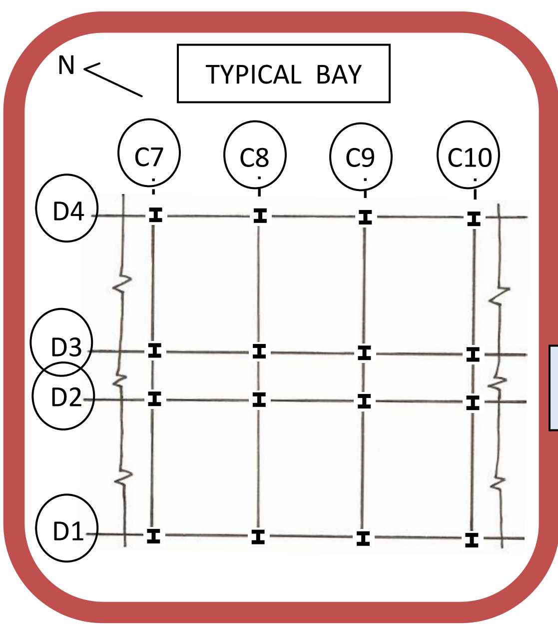
This report is an evaluation of hand calculations against computer modeling to further understand the lateral system for the Gen*NY*Sis Center for Genomics. For the most part, the calculations were consistent with each other, except in the case of story shears which most likely arise from different assumptions between the computer model and the original assumptions in Technical Report I. In other words, attention to detail and precise modeling can provide more comparable calculations.

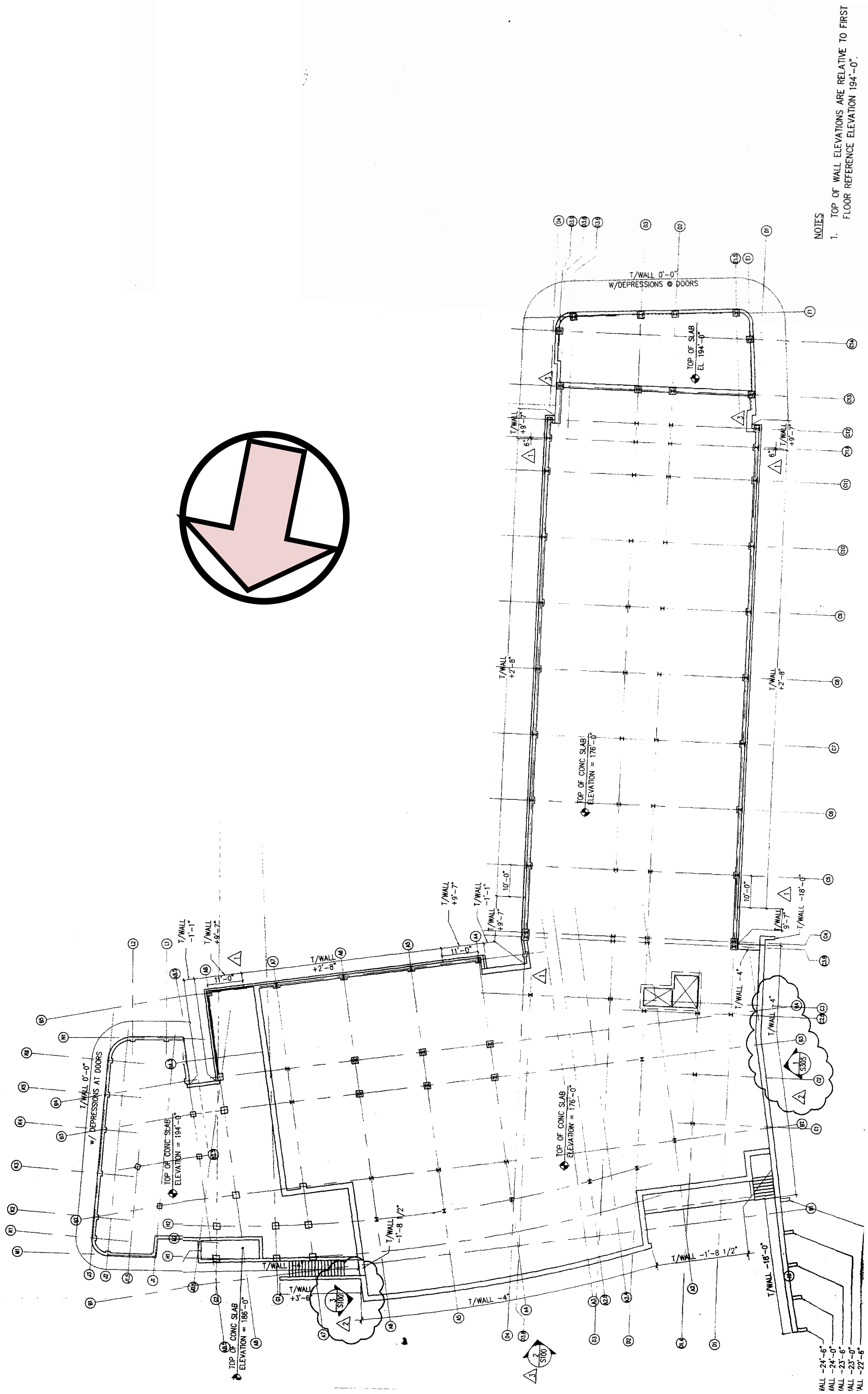
Problems arise with the disagreement of the relative stiffness which is partially due to the disconnected diaphragm at the Penthouse level. It is also due to the disagreement of story shears between the hand calculations and the model calculations. The set point of the model turned out to be different than what was used in design which caused a difference in basic length and distance calculations. However, the calculations for drift are completely within the limits as is the strength in the lateral brace members for the loads designed by hand.

The confusion between model and hand calculations of shear and distances will be re-evaluated as part of my thesis proposal. The brace configurations provide a lot of chance for disagreement and will also be looked into as part of the proposal.


Sudbury
North Bay
Québec
Trois-Rivières
Sorel
Victoriaville
St-Georges
Repentigny
Drummondville
Ottawa
Terrebonne
Montréal
Sherbrooke
Châteauguay
St-Jean-Sur-Richelieu
416
Cornwall
401
Maine
Bangor
Augusta
Orillia
Peterborough
Kingston
Vermont
Barrie
Markham
Belleville
Oshawa
295
Portland
Bay City
Mississauga
Toronto
81
New Hampshire
Saginaw
Cambridge
Rome
Flint
Sarnia
Hamilton
St Catharines
Rochester
Utica
Manchester
Sterling Heights
London
New York
Albany
Livonia
401
Buffalo
390
Detroit
Erie
86
88
Massachusetts
Springfield
Providence
275
Toledo
Cleveland
87
Connecticut
Lorain
Akron
79
Scranton
84
Danbury
Bridgeport
Youngstown
80
476
Mansfield
Pennsylvania
New York
495
Lima
Canton
Pittsburgh
Altoona
Allentown
287

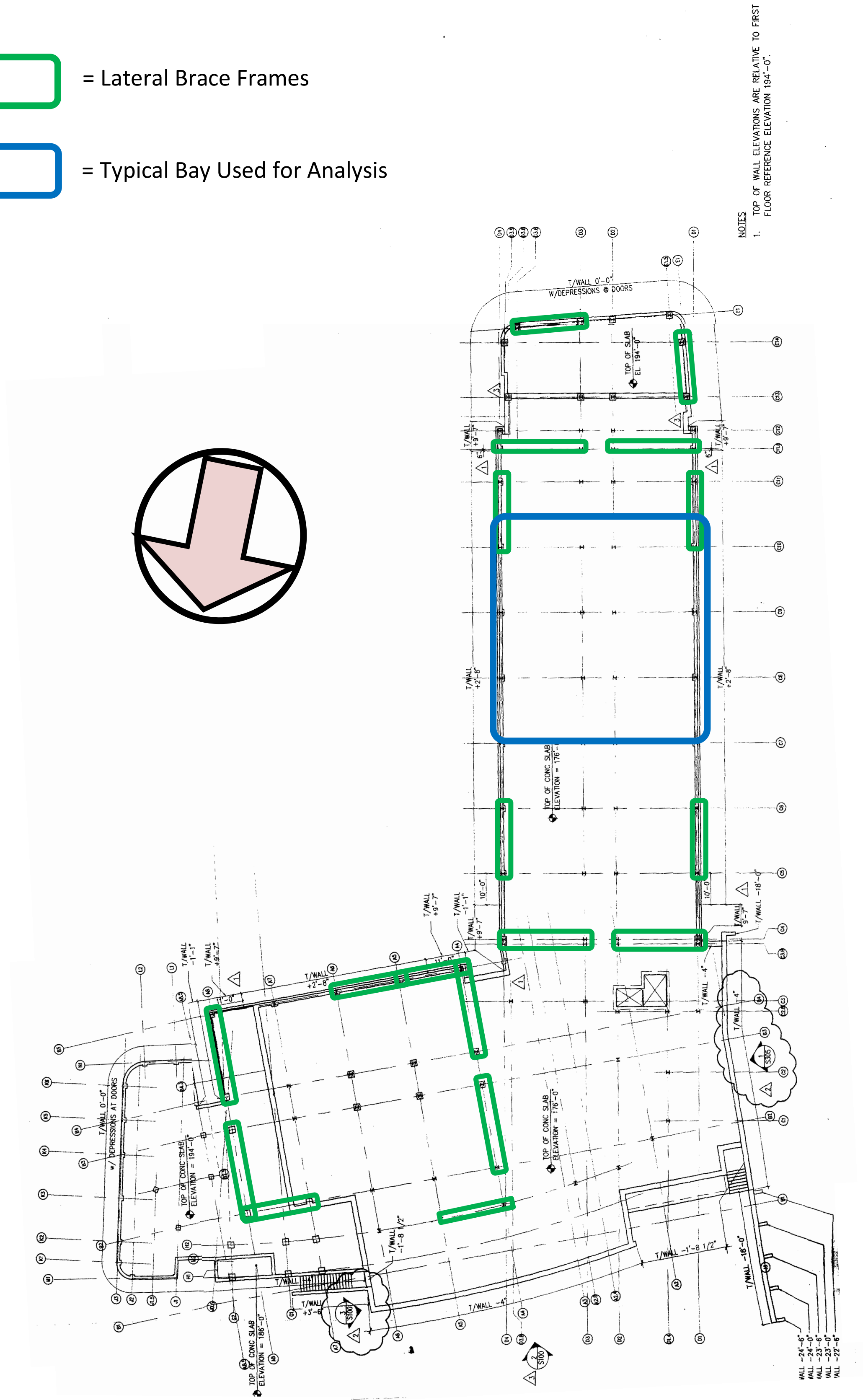

Discovery Dr
University Pl
9
20
Columbia Turnpike
9
N


N
TYPICAL BAY
C7
C8
C9
C10
D4
D3
D2
D1


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

= Lateral Brace Frames

= Typical Bay Used for Analysis



Picture 1: Typical Structural Column on Pier

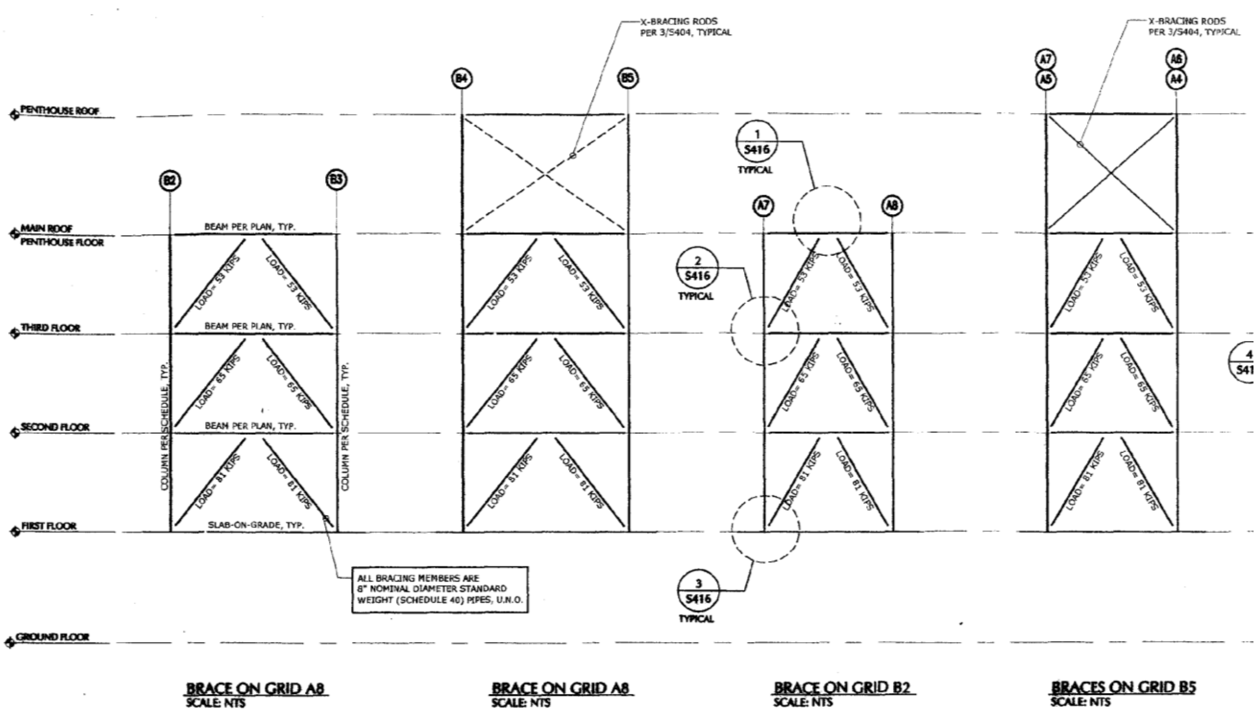
Picture 2: View of Structural Frame

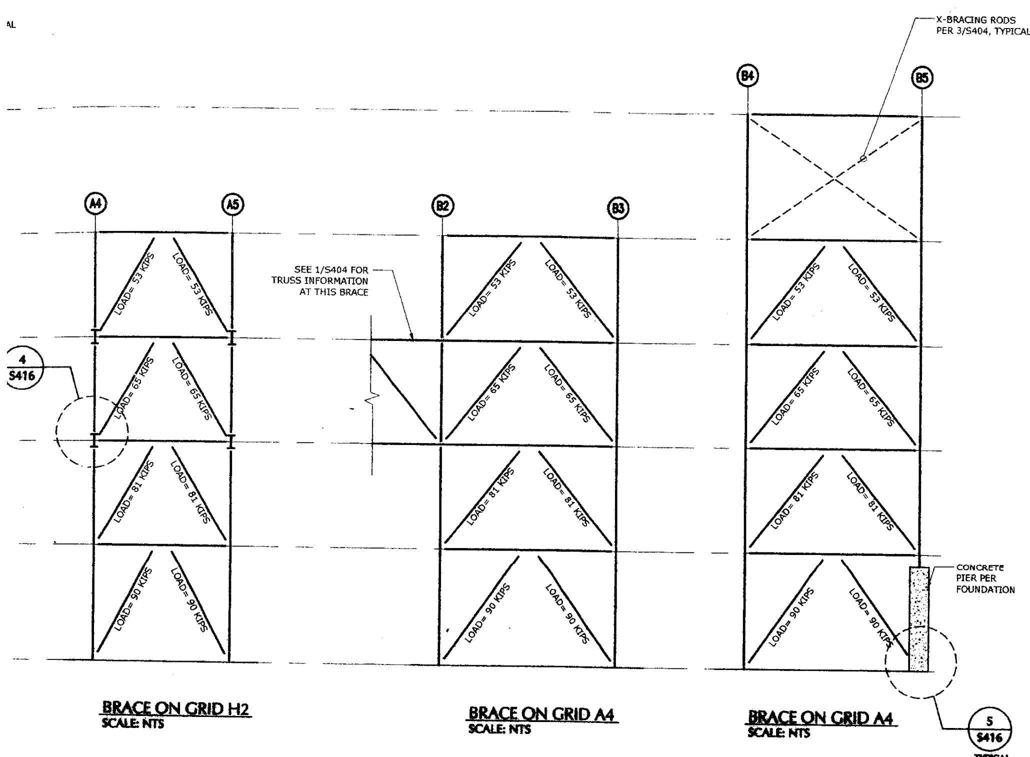
Picture 3: Interior View of 1st Floor

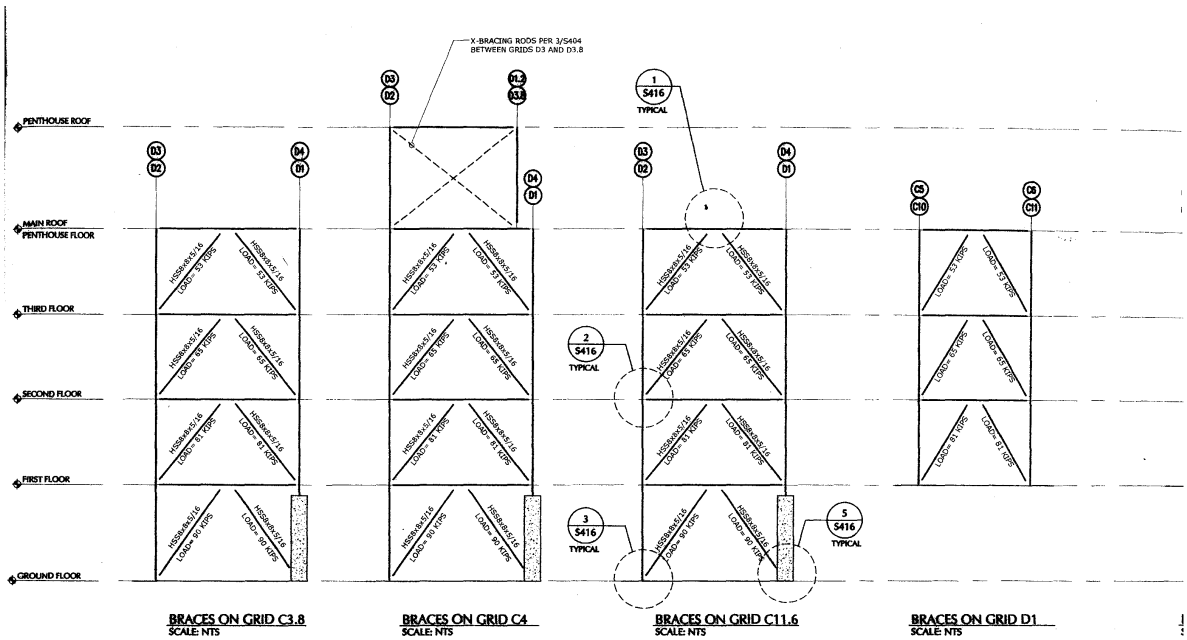
Picture 4: Typical Lateral Brace Connection

Picture 5: Typical Column-Girder Connection

Picture 6: Penthouse Mechanical Screen

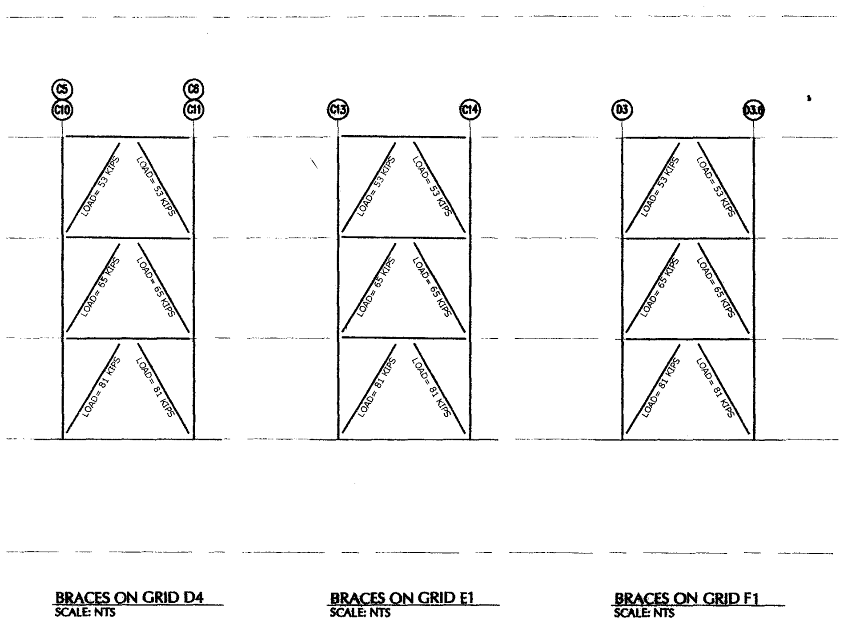

PENTHOUSE ROOF
MAIN ROOF
PENTHOUSE FLOOR
THIRD FLOOR
SECOND FLOOR
FIRST FLOOR
GROUND FLOOR
X-BRACING RODS PER 3/S404, TYPICAL
BEAM PER PLAN, TYP.
SLAB-ON-GRADE, TYP.
COLUMN PER SCHEDULE, TYP.
LOAD= 53 KIPS
LOAD= 65 KIPS
LOAD= 81 KIPS
ALL BRACING MEMBERS ARE 8" NOMINAL DIAMETER STANDARD WEIGHT (SCHEDULE 40) PIPES, U.N.O.
1 S416 TYPICAL
2 S416 TYPICAL
3 S416 TYPICAL
BRACE ON GRID A8
SCALE: NTS
BRACE ON GRID A8
SCALE: NTS
BRACE ON GRID B2
SCALE: NTS
BRACES ON GRID B5
SCALE: NTS

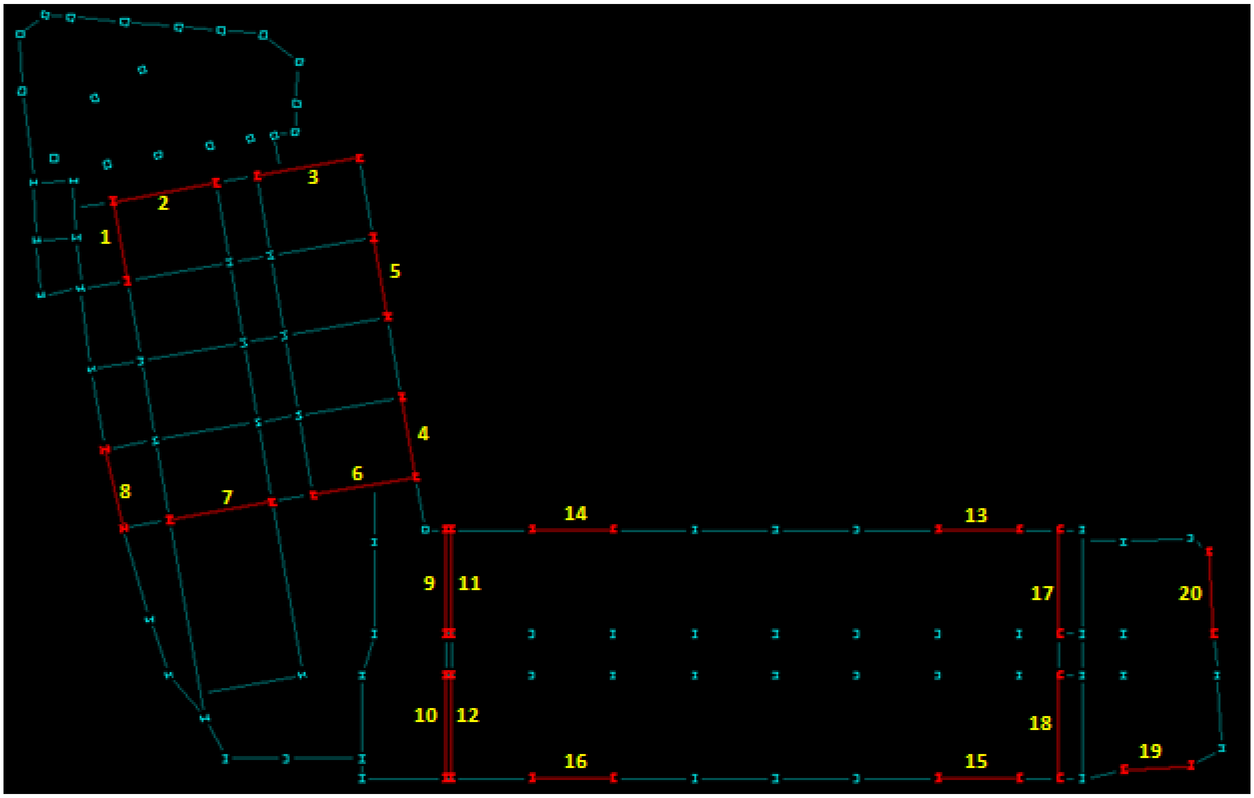

X-BRACING RODS PER 3/S404, TYPICAL
SEE 1/S404 FOR TRUSS INFORMATION AT THIS BRACE
4 S416
LOAD= 53 KIPS
LOAD= 65 KIPS
LOAD= 81 KIPS
LOAD= 90 KIPS
CONCRETE PIER PER FOUNDATION
5 S416 TYPICAL
BRACE ON GRID H2
SCALE: NTS
BRACE ON GRID A4
SCALE: NTS
BRACE ON GRID A4
SCALE: NTS




1 S403 SCHEMATIC CHEVRON BRACE DETAILS
SCALE: 3/32" = 1'-0"±

NOTES:
1. STRUCTURAL REFERENCE ELEVATION 0'-0" = T/GROUND FLOOR SLAB = 176'.
2. SEE DRAWINGS S401 AND S402 FOR COLUMN SCHEDULE/INFORMATION.
3. SEE FRAMING PLANS FOR BEAM INFORMATION.
4. ALL BRACE LOAD MAGNITUDES SHOWN ARE TENSION AND COMPRESSION LOADS.
5. SEE DRAWING S209, S210 AND 3/S404 FOR X-BRACING LOCATIONS AND REQUIREMENTS.

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| | | |
|---|---|---|
| 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

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 |

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 |

| | | |
|---|---|---|
| 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 |

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 |

GEN*NY*SIS CENTER FOR GENOMICS - TECH 3 CENTER OF MASS 1/1

PART I

$(57' \times 9') + (69' \times 20') + (1/2)(46' \times 14') + (46' \times 7')$

2537 ft²

PART II

$(104' \times 64') + (13' \times 87')$

7787 ft²

PART III

$(1/2)(20' \times 104') + (1/2)(88' \times 18') + (76' \times 92')$

8824 ft²

PART IV

$(117' \times 52')$

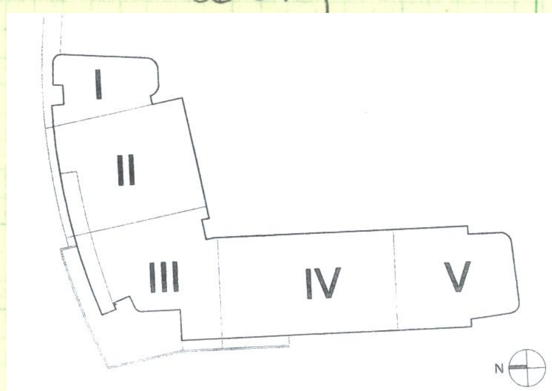
6084 ft²

PART V

$(59' \times 68') + (1/2)(5')(60') + (25' \times 61')$

5687 ft²

CENTER OF PART MASS




PART I : (3, 204)

PART II : (19, 136)

PART III : (38, 59)

PART IV : (141, 53)

PART V : (177, 50)

$$X_m = \frac{\Sigma x A}{\Sigma A} = \frac{(3)(2537) + (19)(7787) + (38)(8824) + (141)(6084) + (177)(5687)}{2537 + 7787 + 8824 + 6084 + 5687}$$

$X_m = 76.2$ ft

$$y_m = \frac{(204)(2537) + (136)(7787) + (59)(8824) + (53)(6084) + (50)(5687)}{2537 + 7787 + 8824 + 6084 + 5687}$$

$y_m = 87.5$ ft