INTORDUCTION

The Northbrook Corporate Center is located on1500 Northbrook Drive in Philadelphia, PA. The building's construction is scheduled to be completed in the spring of 2006. When completed, the building will be used entirely for office use.

Building provides approximately 26,000 square feet of usable area per story. The height of each story is 14 feet. The total height of the building is 74 feet. Contrary to description proved in technical report 1 and 2, the building is a 5 story building. There was some confusion concerning this fact because of the landfill in front of the building. Building provides a garage space on its lowest floor. Because the entrance into the building is located on second floor, the story number count begins at the second floor level. The levels are marked in the following order: Garage level, first, second, third, fourth levels, roof.

The building's lateral force resisting system consists of moment frames. Moment connection capacities range from 40 ft-k to 15 ft-k, largest ones located on the lower floors, and smallest ones on the fourth floor.

Structural system of the Northbrook Corporate Center consists of steel columns, typically W12x72, steel girders, typically W24x68, and steel joists. All steel girders are connected to columns via moment connection to resist lateral loads only. Steel joist are spaced at 3 feet o.c., and span 30 feet between the girders. Steel joists support 4 inch concrete slab on metal deck.

In this report the lateral load resisting system will be described and analyzed in detail. The wind and seismic load development will be described and compared. The lateral force resisting system will be analyzed under five different subsections: *Strength Check, Drift Check, Overturning, Impact on Foundation, and Torsion.* These sections will be accompanied by spot check of directly involved structural members.

This report will conclude that the lateral force resisting system is sufficient to resist the controlling lateral load applied on a building. Also, the report will layout a basis for further study of Northbrook Corporate Center's lateral force resisting system.

LATERAL LOAD DEVELOMPMENT

Wind and Seismic Loads

SEISMIC

Seismic design is based on ASCE7-02, Section 9.

Because the Northbrook Corporate Center incorporates curved walls and inconvenient angles, the wind and seismic calculations were performed only in the North – South direction. Same procedure was used to calculate the seismic load on the building as in the first technical report; however, the site classification, site classification adjustment, response modification factor, 'T' period, and the building weight were corrected. As a result, the seismic load on the building was found to be much smaller than in technical report 1. In the first technical report the full weight of the garage floor was mistakenly included in the building's total weight. All in-depth calculations are provided in the appendix.

These are the major factors used in the calculation of seismic loads:

OCUPANCY CATEGORY	II	[TABLE 1-1]
SEISMIC GROUP	I	[TABLE 9.1.3]
SITE CLASSIFICATION	D	TABLE 9.4.1.2

ACCELERATION

$S_S = 0.335$ (MAP 9.4.1.1a)

$S_1 = 0.085$ (MAP 9.4.1.1b)

SITE CLASS ADJUSTMENT

$F_A = 1.0$ (TABLE 9.4.1.2.4a)

$F_V = 1.0$ (TABLE 9.4.1.2.4b)

$S_{MS} = 1.0(0.335) = 0.335$

$S_{M1} = 1.0(0.085) = 0.085$

DESIGN SPECTURAL RESPONSE

$S_{DS} = 0.667(0.335) = 0.223$

$S_{D1} = 0.667(0.085) = 0.057$

SEISMIC DESIGN CATEGORY

B (TABLE 9.4.2.1a)

B (TABLE 9.4.2.1b)

SEISMIC BASE SHEAR

$V = C_s W$

$C_s = \frac{S_{DS} \cdot I}{R} < C_s = \frac{S_{D1} \cdot I}{T \cdot R}$

$< C_s = 0.044 I S_{DS}$

RESPONSE MODIFICATION FACTOR

$R = 8$ (TABLE 9.5.2.2) ← ORDINARY STEEL MOMENT FRAMES

I

$I = 1.0$ (TABLE 9.1.4)

T PERIOD

$T_a = 0.505$ (EQ. 9.5.3.3-1) $T_a = 0.02(.74)^{0.75} = 0.505$

Using the factors shown above, the seismic loads were calculated to be the following:

ASCE7-02 SEISMIC LOAD CALCUALATION

k=1.005	Level	Weight (w_x)	h_x	$w_x h_x^k$	C_{vx}	V	F_x(k)	N-S (plf)
Roof	5	650	72	49385.23	0.176281	7.67	1.352075	6.911036
Office	4	1560	56	92473.9	0.330086	18.408	6.07623	31.05822
Office	3	1560	42	69255.74	0.247209	18.408	4.550622	23.26018
Office	2	1560	28	46076.98	0.164472	18.408	3.027604	15.47538
Office	1	1560	14	22958.78	0.081952	18.408	1.508565	7.710921
Total		6890		280150.6	1	81.302	81.302	84.41574

<u>WIND</u>

Wind design was based on ASCE7-02, Section 6. Analytical procedure outlined for method 2 was used to calculate the wind loads. As stated above, due to complexity of the building, the calculations were performed only in the North – South direction of the building. The wind

z (ft)	Kz (T 6-3)	q	P	P total (psf)
0-15	0.57	10.04	6.58624	10.512878
20	0.62	10.93	7.17008	11.0963349
25	0.66	11.64	7.63584	11.5617894
30	0.7	12.34	8.09504	12.0206881
40	0.76	13.4	8.7904	12.7155919
50	0.81	14.28	9.36768	13.2924931
60	0.85	14.99	9.83344	13.7579475
70	0.89	15.69	10.29264	14.2168462
80	0.93	16.39	10.75184	14.675745

forces were distributed in pounds per linear foot at each story level for a tributary column width of 30 feet. Also the distribution of forces was calculated in pounds per square foot for a specific vertical distance in feet, as shown in table above. Finally, the point load at each story level was calculated in kips (see table below). Few corrections were made to calculations done in the first technical report; however the final results were not affected.

The following factors were used in the wind load calculations:

ASCE7-02 WIND LOAD CALCUALATION

Method 2: Analytical Aproach

Location data		**Value**	**Reference**	**Ch/Fg/Tb**
Occupancy Type		2	1.5.1	T 1-1
Importance Factor		1	6.5.5	T 6-1
Surface Roughness		B	6.5.6.2	NA
Exposure Factor		B	6.5.6.3	NA
Topographic		1	6.5.7.2	Fg 6-4
Rigid Structure: natural frequency > 1				

Building Dimensions (ft)		**Value**	**Reference**
Height Above Base		74.33	9.5.5.3
Height Above Ground		74.33	6.3
Horiz. Length Parallel		232.146	6.3
Horizontal Dimension Ratio		1.187	F 6-6
Horiz. Length Perpendicular.		195.64	6.3

Wind Velocity (mph)		**Value**	**Reference**	**Ch/Fg/Tb**
Basic Wind Speed		90	6.5.4	F 6.1
Wind Directionality		0.85	6.5.4.4	T 6-4
3-sec Gust Power Law		7	6.3	T 6-2
Mean Wind Speed Factor		0.25	6.5.8.2	T 6-2
Wind Coefficient (b)		0.45	6.5.8.2	T 6-2
Wind Coefficient (z)		44.6	6.5.8.2	T 6-2
Mean Hourly Wind Speed		64.05	6.5.8.2	Eq 6-14
Height atm Boundary		1200	6.3	T 6-2
Velocity Pressure Exp.		0.90332	6.5.6.6	T 6-3

Integral Length Scale		**Value**	**Reference**	**Ch/Fg/Tb**
Integral Length Scale Factor		320	6.5.8.1	T 6-2
Integral Length Scale Exp.		0.333	6.5.8.1	T 6-2
Integral Length Scale, Turb.		353.796	6.5.8.1	Eq 6-7
Turbulence Intensity Factor		0.3	6.3	T 6-2
Intensity of Turbulence		0.392	6.5.8.1	Eq 6-5

Fundamental Period		Value	Reference	Ch/Fg/Tb
Period Coefficient		0.02	9.5.3.2	T 9.5.5.3.2
Approx. Fund. Period		0.51	9.5.3.2	Eq 9.5.3.3-1
Natural Frequency		1.96	6.5.8.2	1/T

Gust Effect Factor		Value	Reference	Ch/Fg/Tb
Gust Coefficient		3.4	6.5.8.2	N/A
Gust Coefficient		3.4	6.5.8.2	N/A
Background response		0.80821	6.5.8.1	Eq 6-6
Gust Factor		0.819462	6.5.8.2	Eq 6-4

Wind Pressure		Value	Reference	Ch/Fg/Tb
Velocity Pressure		17.6256	6.5.10	Eq 6-15
Volocity Pressure at z		15.99	6.5.12.2	T 6-3

External Pressure Coefficient		Value	Reference	Ch/Fg/Tb
Windward Side		0.8	6.5.11.2	F 6-6
Leeward Side		-0.3	6.5.11.2	F 6-6
Final Pressure (psf)		15.48577	P = qGCp - qGCp	

The point loads on each column line (tributary width = 30ft) at each story level were calculated to be:

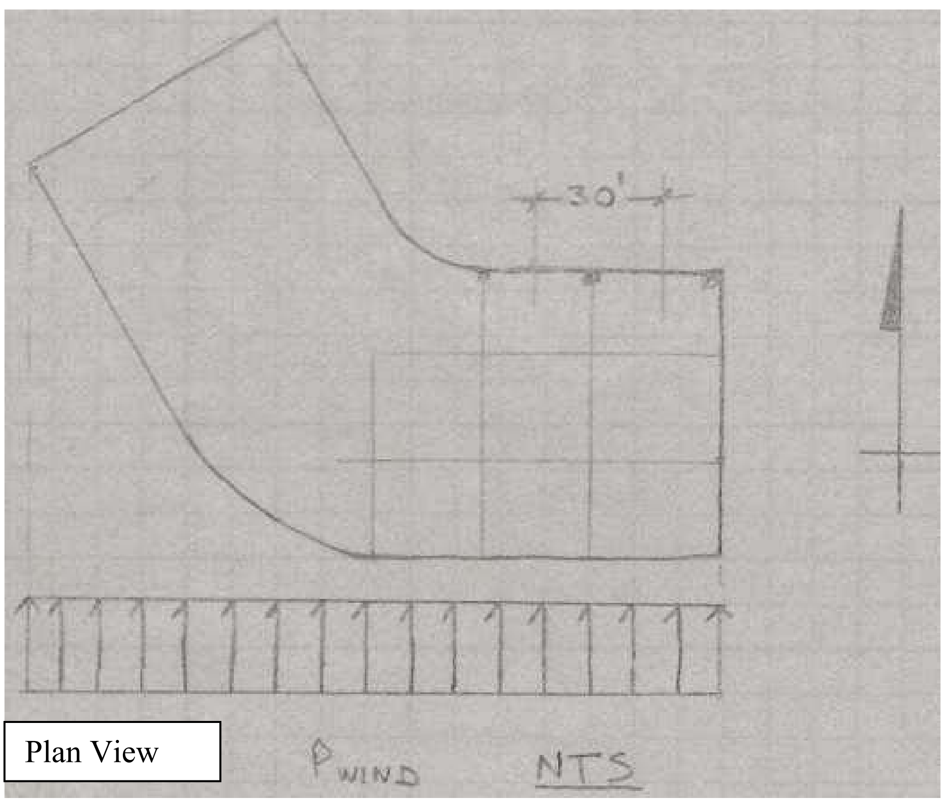
#. STORY / HEIGHT	POINT LOAD (K)
ROOF / 74'	3.6
4th / 56'	5.9
3RD / 42'	5.4
2ND / 28'	4.9
1ST / 14'	4.6

WIND AND SEISMIC LOAD COMPARISON CHART

Wind and Seismic North - South Direction Comparison				
k=1.005	Level	h_x	Seismic (plf)	Wind (plf)
Roof	5	72	6.9	400
Office	4	56	31	421
Office	3	42	23.3	386
Office	2	28	15.5	350
Office	1	14	7.7	328
Total			84.4	1885

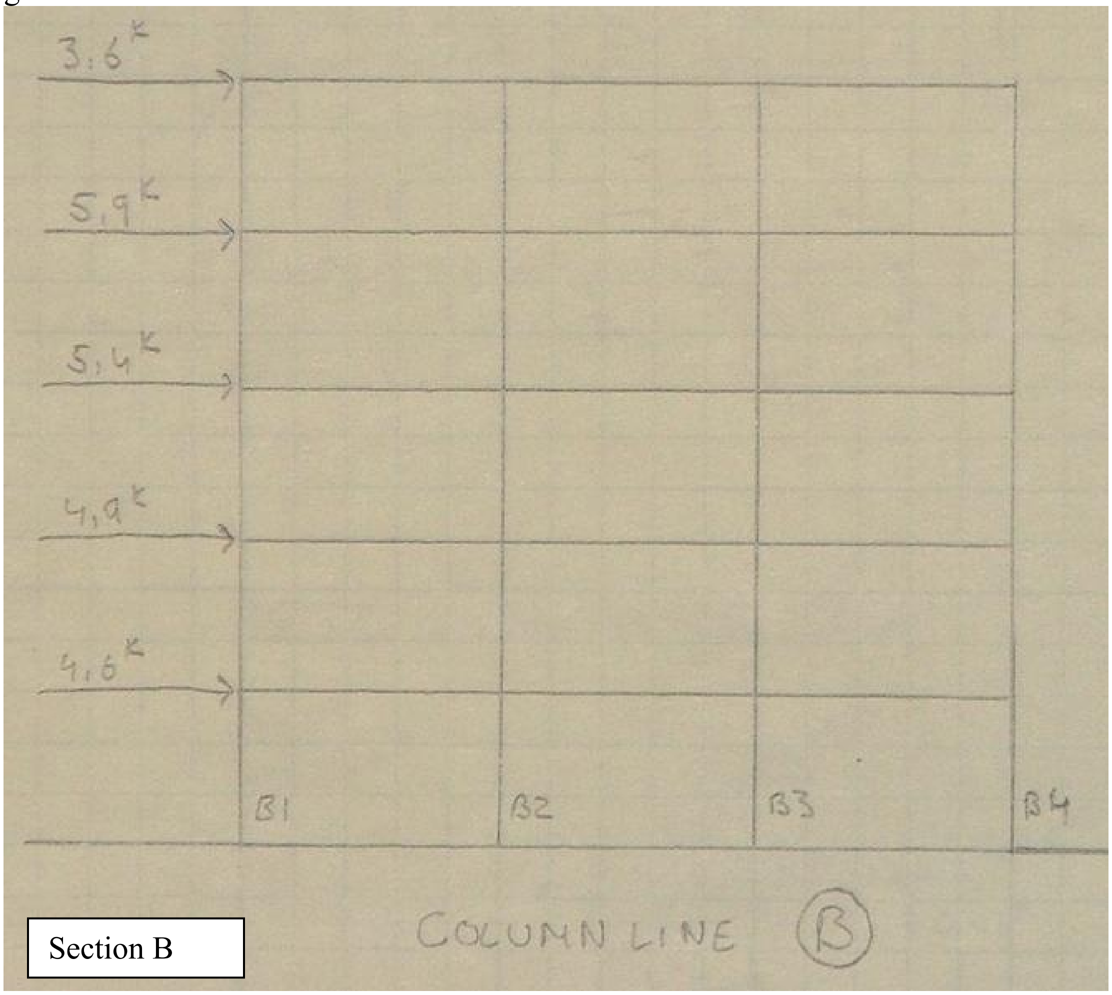
LATERAL LOAD DISTRUBUTION

Wind controls the lateral forces of the design, as shown in comparison chart above. The wind forces were distributed in pounds per linear foot at each story level. The direction and distribution of wind forces is shown by two drawings below. First drawing shows the distribution of wind forces in a plan view. The second drawing shows the calculated point load wind distribution on a section of a column line B. Because all calculations were done by hand, the easiest column line was analyzed in this report. The tributary width of column line B is 30 feet, as shown in plan view drawing below. Because of the complexity of the building, the wind loads were calculated only for column line B.



Plan View

The frame shown bellow is a part of main lateral force resisting system. The system resists lateral forces by incorporating the moment connections between the columns and girders.



Section B

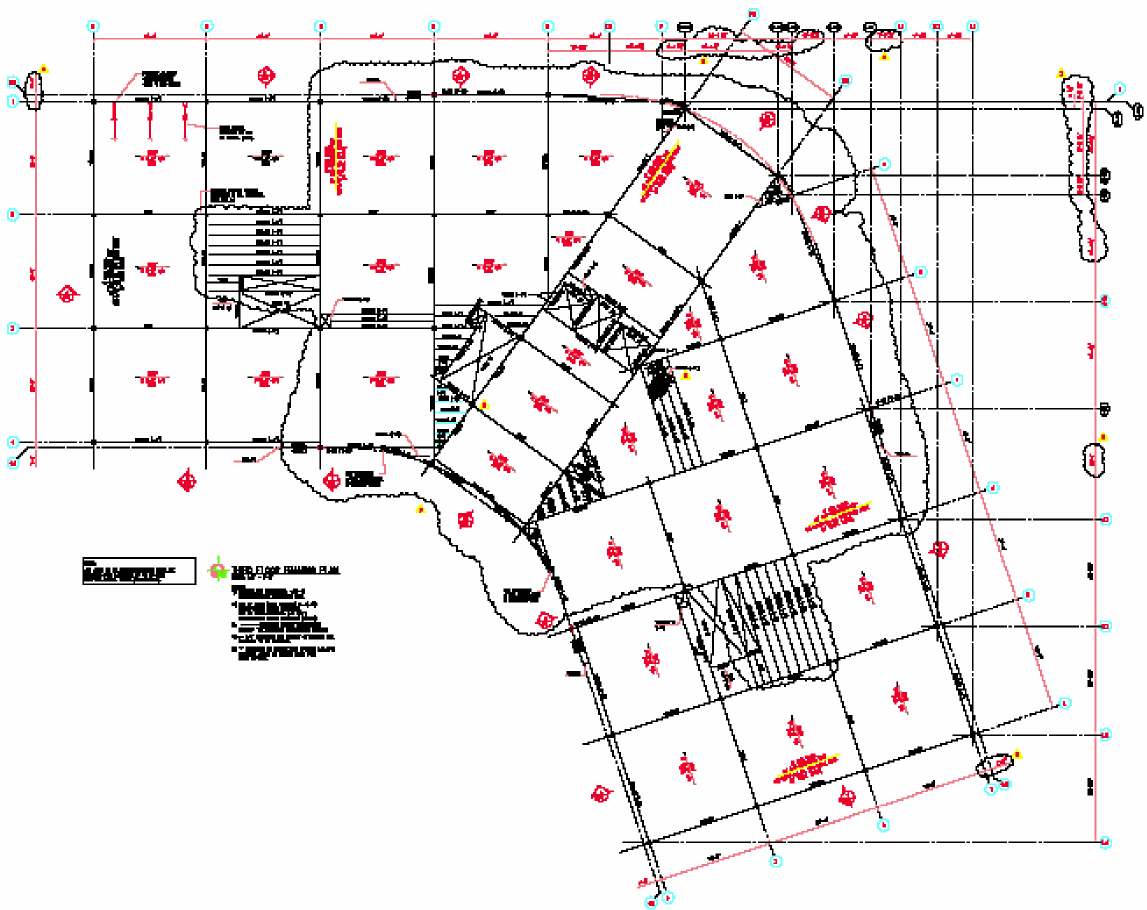
The wind force in pounds per square foot were converted to pounds per linear in the vertical direction along the column line B. Then the point load at each story level was determined. The calculations performed to obtain these results are shown in the appendix.

LATERAL FORCE RESISTING SYSTEM

The Northbrook Corporate Center's lateral force resisting system consists entirely of moment connections. Because the building is only 74 feet high, the accumulation of wind forces is small enough for moment connections to resist. The most columns are spread 30 feet apart in each direction and rest on shallow concrete foundations. Typical column size is W12x72. Almost all columns span from the garage floor to the third story, where they are connected to and continued by a smaller column, typically W12x53. This connection is made 4 feet above the floor of the third story. Typical girder size is W24x68. The girders are connected to the columns through a moment connection, capacity of which ranges from 40 ft-k to 15 ft-k. 40 ft-k moment resistive connections are found on the first and second floors, 30 ft-k moment connections are found on the third floor, and 15 ft-k moment connections are found on the fourth floor. The girder size is satisfactory to carry all gravity loads without the moment connections, thus, the moment connections are used to resist the lateral loads only. This is better illustrated in the lateral system analysis section of this report.

The steel joists rest on girders, and support the concrete slab on metal deck. The joists do not contribute to the lateral force resisting system, and due to the scope of this report, little attention is given to them.

The Northbrook Corporate Center is not a perfectly rectangular building; in fact, the design incorporates curved exterior walls and inconvenient angles, as shown in the drawing below. Hence, the structural layout is not entirely uniform. The bays located in the center region of the building vary in size and proportion. In some instances 'W' shaped beams are used instead of steel joists (see drawing below).



Section of the structural system is shown in lateral force distribution section of this report.

LATERAL SYSTEM ANALYSIS

Due to the complexity of the building's outline, only North – South direction was chosen to perform this analysis. The calculations were done by hand, so the calculations were simplified even more by limiting the analysis to only one column line (B). The outline of the building closely resembles letter L. Since the angle of the building's geometry is close to 90 degrees, it can be roughly estimated that the lateral loads in the East – West direction will be similar to the loads in North - South direction. Nevertheless, due to limitations mentioned above, only the column line B will be analyzed in this report.

The lateral forces are resisted entirely by moment connections as described in the lateral force resisting section of this report. The analysis is divided into four subsections: *Strength Check, Drift Check, Overturning, Impact on Foundation, and Torsion.*

STRENTH CHECK

The impact of lateral load is expressed in resulting end moments applied to columns and girders.

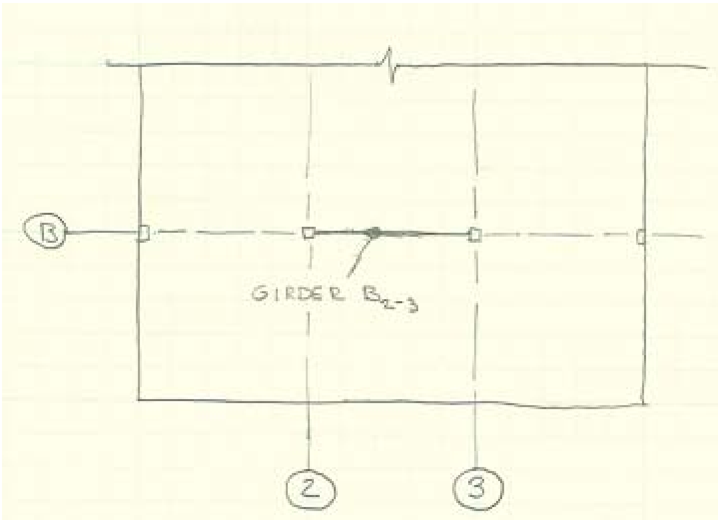
The shear wind forces were distributed to each of the four columns at each story. The interior columns resist point load value that is twice as large as exterior columns. That is because interior columns have moment connections on both sides, while exterior columns have moment connections only on one side. The end moments were calculated and checked against the given moment connection capacity. Every connection proved to be satisfactory. For example, the fourth floor end moment per connection was calculated to be 4.2 ft-k, compared to 15 ft-k moment connection capacity.

Columns were checked for strength. Dead load of 60 psf included concrete slab, metal deck, reinforcement, steel joists, steel girders, columns, ceiling, floor finish, collateral, and partition. Reduced live load was used where applicable. The end moments were incorporated into these calculations, and final axial strength of columns per story was determined as follows:

- Garage 719.7 k
- First Story 565.2 k
- Second Story 417.3 k
- Third Story 246.3 k
- Fourth Story 75.42 k

The axial strength of a W12x72 column for KL = 12 ft is 761 k. The axial strength of a W12x53 column for KL = 12 ft is 518 k. The given values were found in LRFD manual in table 4-2. The columns prove to be satisfactory to withhold all gravity and lateral loads. Detailed calculations are provided in the appendix, at the end of this report.

Virtual work method was used to calculate the vertical displacement of first story interior girder (see the drawing below, column B). The displacement at the midspan of the W24x68 girder was calculated to be 0.03 ft, which is less than 1/400, thus, satisfactory.



The total uniformly distributed load on the girder was found to be 122.4 k. According o table 5.4 in the LRFD, the total uniformly distributed load capacity of W24x68 girder is 180 k, thus girder is satisfactory. The girder was also checked using maximum moment capacity method. The calculated moment was found to be 465.9 ft-k, compared to maximum capacity of 664 ft-k found in LRFD manual table 5-2; thus flexure design is also satisfactory. The in-depth calculations, E and I values used, and all assumption are available in the appendix.

DRIFT CHECK

Drift of each individual story was calculated using the equation Δ=VH/192EI. Values of I were found in LRFD manual. As mentioned before column span from the garage floor to a 4 feet mark above the third floor. The I value of the third story was averaged to avoid complicated calculations. In the table below, the I values represent the columns below the given story level. For example, the column spanning from the 4th floor to the roof has and I value of 425 quadratic inches.

LEVEL	V	H	E (k/in^2)	I	STORY Δ (in)
ROOF	3.6 K	14'	3600	425	2.6×10^{-7}
4	5.9 K	14'	3600	542	3.2×10^{-7}
3	5.4 K	14'	3600	597	2.7×10^{-7}
2	4.9 K	14'	3600	597	2.4×10^{-7}
1	4.6 K	14'	3600	597	2.3×10^{-7}
Total					1.3×10^{-6} in

The total drift of each story is shown in the right column. The total drift of the building due to the wind was calculated to be 0.0000013 inches. The drift ratio was found to be 0.00000027, which is much less than allowable ratio of l/400. The drift check proves to be satisfactory. Detailed calculations are provided in the appendix.

OVERTURNING

The building does not have deep foundations because all wind load moments are resisted by the buildings weight. The wind force moment was calculated and compared to the building's gravity moment, as shown in tables below. The total overturning wind moment was found to be 1,018 ft-k. The total moment due to gravity loads was calculated to be 57,240 ft-k. Using these values the factor of safety was determined to be 56, which is by far larger than 3. Thus, the overturning check proves satisfactory.

OVERTURNING MOMENT CALCULATION

Calculations are only of column line B, tributary area = 30 ft

Overturning Wind Moment Table			
Level	Height (ft)	Wind Force (k)	Moment
Roof	72	3.6	259.2
4	56	5.9	330.4
3	42	5.4	226.8
2	28	4.9	137.2
1	14	4.6	64.4
Total	212	24.4	1018

Gravity Moment Table			
Level	Load (psf)	Weight (k)	Moment
Roof	25	90	5400
4	60	216	12960
3	60	216	12960
2	60	216	12960
1	60	216	12960
Total		954	57240

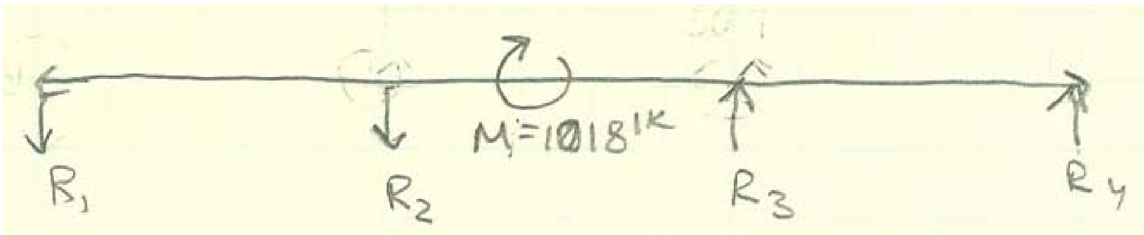
Factor of Safety	56.22789784

****Note***: Precise gravity moment calculation is not necessary because the factor of safety is much larger than 3.

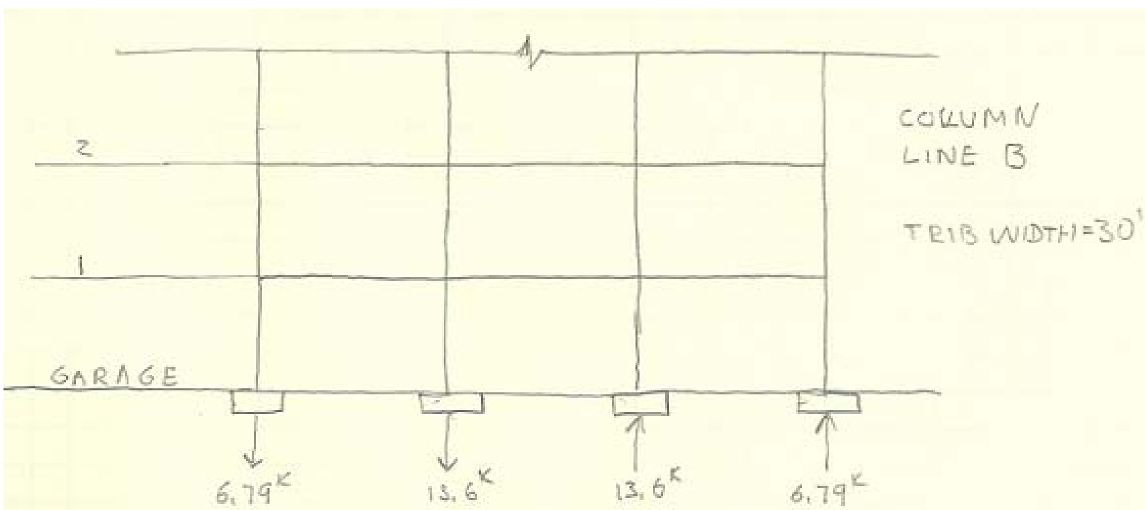
These values are only for column line B. The gravity loads were calculated for the tributary area of 30 ft x 120 ft, with gravity moment arm = 60 ft.

IMPACT ON FOUNDATION

Total overturning wind moment was used to find the resulting reactions on each of the column footings. The gravity forces are not shown in these calculations because the impact of overturning wind moment is very small compared to the weight the building applies on each column. As previously stated, this analysis is done only for column line B.



Shown below are the resulting forces on each foundation. Assuming that the wind blows in the right to left direction, the total weight on the first two footings will be increased, while the forces applied on the second two footings will decrease. Because the wind changes directions, all four footings are designed to resist an increased force.



Because the Northbrook Corporate Center is relatively small building, gravity loads can resist all the tension forces; thus, tension resisting foundations are not necessary.

Due to the limited scope of this report, the detailed check of dimensions and reinforcing of foundations is not performed.

TORSION

The moment resisting connections are uniformly distributed around the building, making the torsion moment arm infinitely small. Also the moment connections used in the building are larger than needed for wind loads; hence, it is assumed that the moment connections are satisfactory to resist the torsion forces applied on the building.

CONCLUSION

The purpose of this report was to describe and analyze the lateral force resisting system of the Northbrook Corporate Center. In the body of this report a detailed description of lateral force development and its effect on the structural system of a building is provided. It was determined that the wind is the controlling force. The report provides a detailed analysis of the lateral force resisting system by the means of spot checks and illustrations. This report concludes that the lateral force resisting system of Northbrook Corporate Center is sufficient to resist all lateral loads applied on the building. This report also provides the foundation for further study of Northbrook Corporate Center's lateral force resisting system.

APENDIX

INCLUDED:

SESIMIC DESIGN
WIND DISTREBUTION
DRIFT CHECK
MOMENT CHECK
COLUMN CHECK
GIRDER CHECK
IMPACT ON FOUNDATION

SEISMIC LOAD DESIGN

LATERAL LOADS

SEISMIC LOAD CALCULATIONS   ASCE 7-02   CH 9

OCUPANCY CATEGORY	II	[TABLE 1-1]
SEISMIC GROUP	I	[TABLE 9.1.3]
SITE CLASSIFICATION	D	TABLE 9.4.1.2

ACCELERATION

$S_C = 0.335$ (MAP 9.4.1.1a)

$S_2 = 0.085$ (MAP 9.4.1.1b)

SITE CLASS ADJUSTMENT

$F_A = 1.0$ (TABLE 9.4.1.2.4a)

$F_V = 1.0$ (TABLE 9.4.1.2.4b)

$S_{ms} = 1.0(0.335) = 0.335$

$S_{m2} = 1.0(0.085) = 0.085$

DESIGN SPECTURAL RESPONSE

$S_{DS} = 0.667(0.335) = 0.223$

$S_{D1} = 0.667(0.085) = 0.057$

SEISMIC DESIGN CATEGORY

B   (TABLE 9.4.2.1a)

B   (TABLE 9.4.2.1b)

SEISMIC BASE SHEAR

$V = C_S W$

$C_s = \frac{S_{DS} \cdot I}{R} < C_s = \frac{S_{DI} \cdot I}{T \cdot R}$

$< C_s = 0.044 I$ SAS

RESPONSE MODIFICATION FACTOR

$R = 8$ (TABLE 9.5.2.2) ← ORDINARY STEEL MONENT FRAMES

I

$I = 1.0$ (TABLE 9.1.4)

T PERIOD

$T_a = 0.505$ (EQ. 9.5.3.3-1) $T_a = 0.02(.74)^{0.75} = 0.505$

SEISMIC LOAD DESIGN (RECALCULATED)

BUILDING DEAD LOADS

AVG FLOOR AREA = 26,000 SF

FLOOR WEIGHT $= (60 \text{ PSF})(26000 \text{ SF}) = 1560^K$

*RECALCULATED FLOOR LOAD = 60 PSF, THAT INCLUDES:

CONCRETE SLAB, METAL DECK, STEEL JSTS, STEEL GIRDERS,

PARTITION, CEILING, FLOOR FINISH, STEEL COLUMNS

ROOF WEIGHT $= (25)(26,000) = 650^K$

TOTAL BUILDING WEIGHT $= 6890^K$

$C_s = \dfrac{0.223}{4(1.0)} = 0.056$

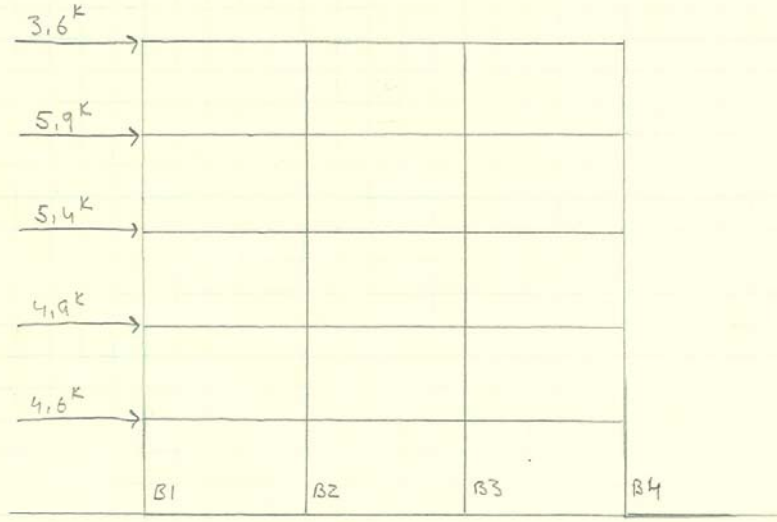
$C_{s\,MAX} = \dfrac{0.057}{0.504(4)/1.0} = 0.028$

$C_{s\,MIN} = 0.044(1.0)(0.223) = 0.010$ ← CONTROLS

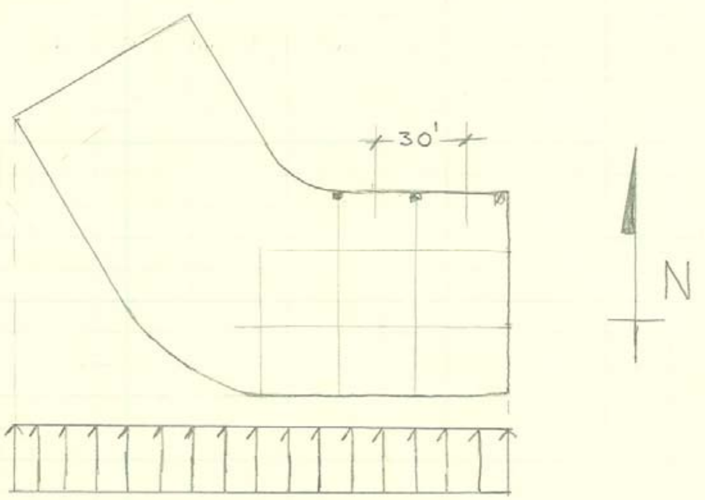
$V = 0.010(6890^K) = 81.3$ BASE SHEAR

$K = 1 + (0.51 - 0.5)/2 = 1.005$ (TABLE 9.5.3.4)

COLUMN TRIB. WIDTH = 30 FT



COLUMN LINE (B)



PWIND NTS

Dimitry Reznik

STORY DRIFT

$$\Delta = \frac{VH}{192EI}$$

LEVEL	V	H	E ($^k/_{in^2}$)	I	STORY Δ (in)
ROOF	3.6 K	14'	3600	425	2.6×10^{-7}
4	5.9 K	14'	3600	542	3.2×10^{-7}
3	5.4 K	14'	3600	597	2.7×10^{-7}
2	4.9 K	14'	3600	597	2.4×10^{-7}
1	4.6 K	14'	3600	597	2.3×10^{-7}
Total					1.3×10^{-6} in

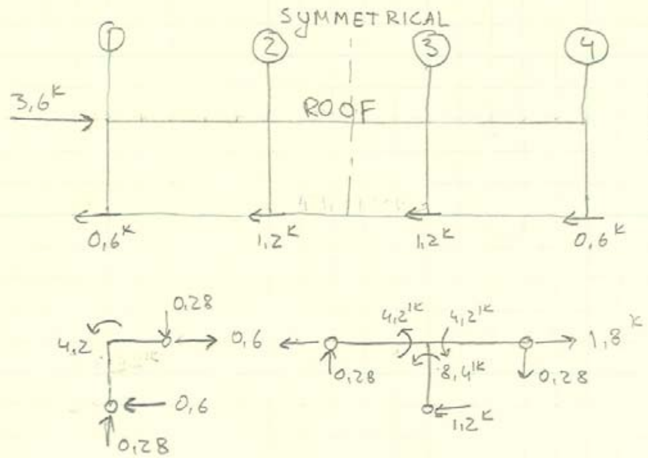
$$\Delta_{ROOF} = \frac{3.6^K(14\text{ FT})(12\ ^{in}/_{ft})}{192(29000\frac{K}{in^2})(425\ in^2} = 2.6\times10^{-7}\text{ in}$$

$$\Delta_4 = \frac{5.9(14)(12)}{192(29000)(542} = 3.2\times10^{-7}\text{ in}$$

$$\frac{1.3\ 10^{-6}/(12)}{72} = 2.2\times10^{-7} \ll 1/400 = 0.0025 \qquad \underline{OK}$$

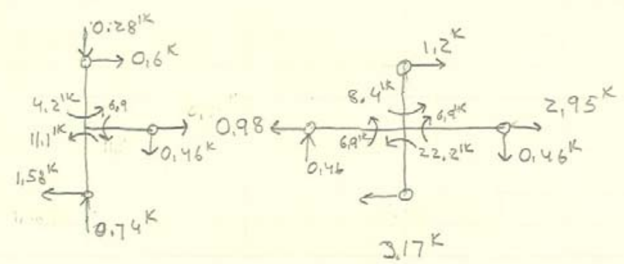
* NOTE: EQUATION DOES NOT ACCOUNT FOR GIRDER I; HOWEVER, IT DOES INCLUDE THE RESISTANCE MOMENTS GIRDERS INDUCE ON COLUMNS.

ROOF MOMENTS CHECK



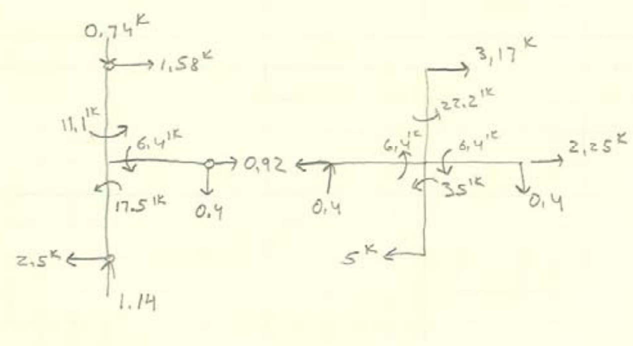
$\boxed{M = 4{,}2^{1K} \leq 15^{1K}}$

FOURTH LEVEL MOMENT CHECK



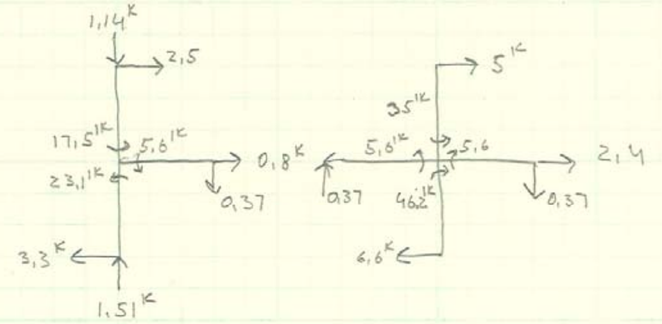
$\boxed{M = 11{,}1^{1K} \leq 30^{1K}}$

THIRD LEVEL MOMENT CHECK



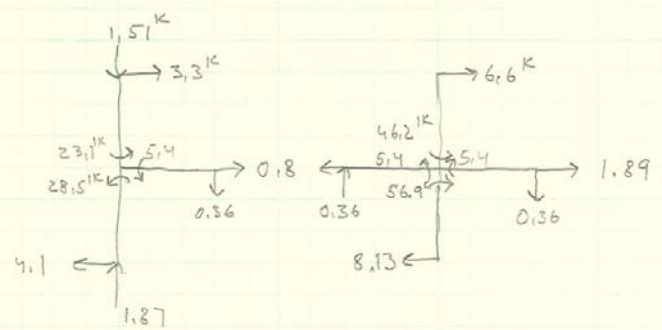
$\boxed{M = 17{,}5^{K} \leq 40^{1K}}$

SECOND FLOOR MOMENT CHECK



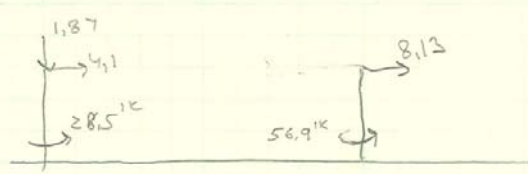
$\boxed{M = 23,1'^{k} < 40'^{k}}$

FIRST FLOOR MOMENT CHECK



$\boxed{M = 28.5 < 40'^{k}}$

GARAGE FLOOR MOMENT CHECK



COLUMN CHECK

	LEVEL	MOMENT	LL R. FACTOR	LOAD Pu (k)	TOTAL LOAD Peff (K)
$\alpha = 2$	GARAGE	113.8 'K	0.4	492.1	779.7
$\alpha = 2$	1	92 'K	0.4	386.2	565.2
$\alpha = 2$	2	70 'K	0.43	277.3	417.3
$\alpha = 2$	3	44.4	0.5	157.5	246.3
$\alpha = 2.4$	4	16.8 'K	0.5 ROOF LL	35.1	75.42

$\alpha = 24/d$

$P_{eff} = P_U + \alpha M$

$LL_R = (0.25 + 15/\sqrt{A_I}) \geq 0.4$

FACTORED LOAD 1.2D + 1.6L controls

GIRDER CHECK - FIRST STORY INTERIOR GIRDER

MOMENTS DO TO LATERAL WIND FORE AND GRAVITY LOADS ARE CONSIDERED IN THIS GIRDER CHECK CALCULATION.

$\text{TRIB AREA} = (30)(30)\ FT^2 = 90\ FT^2$

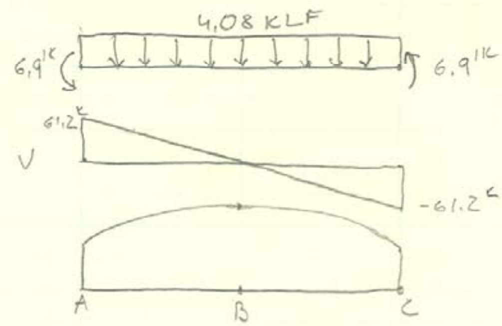
$LL_R = (0.25 + 15/\sqrt{3600}) = 0.5$

$LL = 0.5(80\ PSF) = 40\ PSF$

$\text{FACTORED LOAD} = 1.2(60) + 1.6(40) = 136\ PSF$

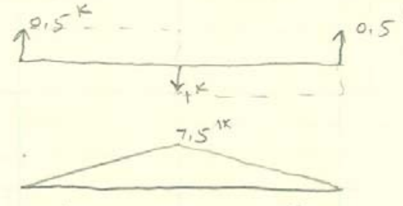
$W_w = (136\ PSF)(30\ FT) = 4080\ PLF = 4.08\ KLF$

REAL



$M_\Delta = 6.9 + 61.2x - \frac{4.08x^2}{2}$

VIRTUAL



$M_{\Delta_P} = 0.5x \qquad M_{\Delta P} = -0.5x$

2 GIRDER CHEK

$$\Delta_B = \frac{2}{EI}\left[\int_0^{7.5}\left(6.9+61.2x-\frac{4.08}{2}x^2\right)(0.5x)\,dx\right.$$

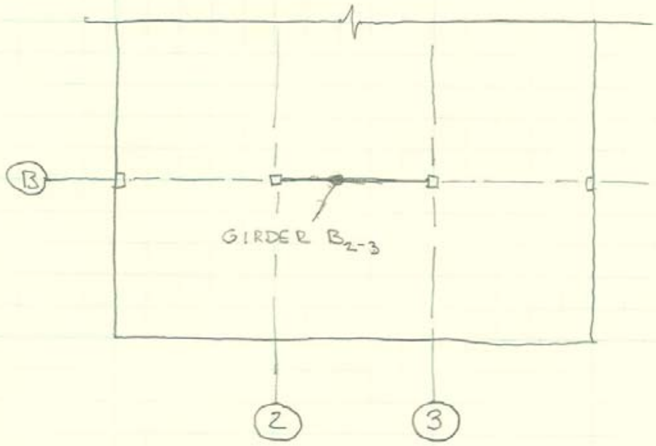
$$= \frac{2}{EI}\left[\int_0^{7.5} 3.45x\,dx + \int_0^{7.5} 30.6x^2\,dx + \int_0^{7.5} 1.02x^3\,dx\right]$$

$$= 2/EI\left[\frac{3.45}{2}x^2\Big|_0^{7.5} + \frac{30.6x^3}{3}\Big|_0^{7.5} + \frac{1.02}{4}x^4\Big|_0^{7.5}\right]$$

$$= \frac{2}{EI}\left[97.03 + 4303.125 + 806.8\right]$$

$$= 10413.91(12^3)/(29000)(1830) = 0.339\text{ in} = 0.03\text{ FT}$$

$$\frac{0.03}{15} = 0.002 \le \frac{1}{400} \qquad \boxed{OK}$$

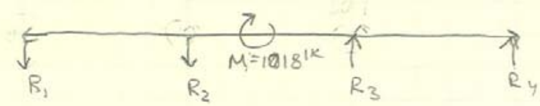


$$M_n = 6.9 + 61.2(15) - \frac{4.08(15^2)}{2} = 465.9^{1K}$$

TOTAL UNIFORMLY DISTRIBUTED LOAD = $122.4^K \le 180^K$

W24x68 is $\boxed{OK}$ TABLE 5.4 - LRFD

WIND LOAD IMPACT ON FOUNDATIONS



1018

$R_1 = 0.5 R_2 = -0.5 R_3 = -R_4$

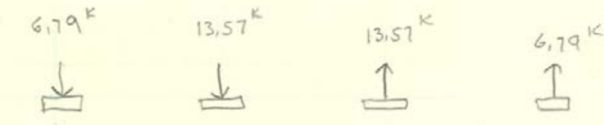
$1018^{1K} = 15(2R_1) + 45R_1 + 15(2R_1) + 45(R_1)$

$R_1 = 6.79^K$

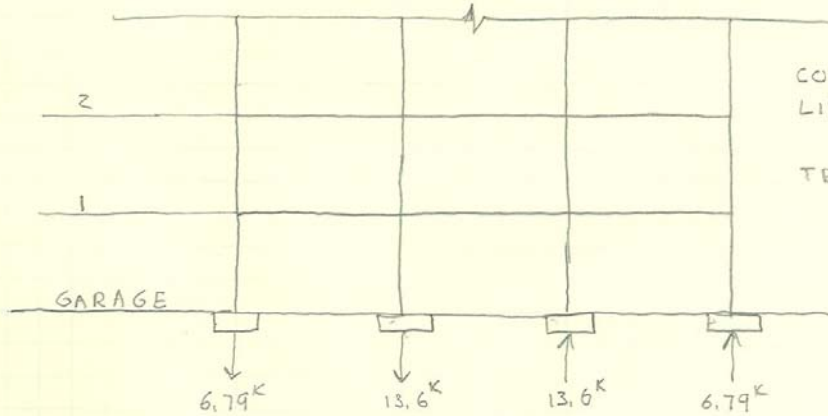
$R_2 = 13.57$

$R_3 = 13.57$

$R_4 = 6.79$



GRAVITY	GRAVITY	GRAVITY	GRAVITY
+	+	−	−
6.79	13.57	13.57^K	6.79^K



COLUMN LINE B

TRIB WIDTH = 30'