WESTINGHOUSE ELECTRIC COMPANY CORPORATE HEADQUARTERS

CRANBERRY, PA

Technical Assignment 3

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Structural Option

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EXECUTIVE SUMMARY

The purpose of the technical report is to analyze the lateral system of the Westinghouse Electric Company Corporate Headquarters Building One and the design as is, by LLI Engineering and IKM, Inc. An additional goal of the report is to understand the structural design of the building so it can be correctly analyzed and adjusted for a final project.

The Westinghouse Electric Company Corporate Headquarters will be a three building campus with site features such as asphalt walking paths and volleyball courts on eighty-three acres in Cranberry, PA. For the purpose of the project, only Building One will be analyzed as the other two are considered a separate project by all parties involved. The truncated V-shape building has been given a look of importance with polished concrete block merging into brick stepped-out columns to accentuate the verticality of the five-story 74’-6” tall structure.

The report discusses the main structural framing, foundation types, and building materials. The building is a steel structure with steel girders and composite beams and deck. Lateral load resisting systems have been determined, checked, and are discussed in detail in this report. The system is an ordinary moment frame with wind moment connections on all members of the lateral system. All wind and seismic loads were determined using ACSE 7-05 chapters 6, 11, and 12. The wind and seismic loads were checked in a RAM model. However, RAM basic code uses different assumptions than were used in this report and a different method of determining values for each floor, which is not required by code. Therefore, the values from RAM and the hand-calculated values in this technical report do not match, but are in the same order and essentially confirmed no drastic errors in calculations.

For this report, torsion was considered as a potential amplifier of the lateral loads. As seen in the torsion section, there are some torsion induced effects on the Westinghouse Electric Company Corporate Headquarters. Torsion was determined in a method following stiffness of members of each frame in the building and distributing the load accordingly. Wind loading controls the short direction frames of the building while seismic loading controls the longer direction of the building’s design. After the effects due to torsion were determined, a portal analysis was used to find the moments of four specified frames. Spot checks of several members for lateral and gravity loading using Load and Resistance Factor Design (LRFD) are included in the member check portion of the report. The members were chosen, then the gravity loads were determined on them and the moment determined in the portal analysis from the controlling lateral force on the frame was applied. All members evaluated in this report have been determined to be acceptable. Overturning moment on the building and uplift of the foundations were examined and calculated. The uplift on the building is five times smaller than the axial force on an exterior column in the building, thus ensuring uplift is not an issue for the foundations. However, due to the weight of the building the same conclusion could have been determined through inspection.

INTRODUCTION

WESTINGHOUSE ELECTRIC COMPANY CORPORATE HEADQUARTERS BUILDING ONE

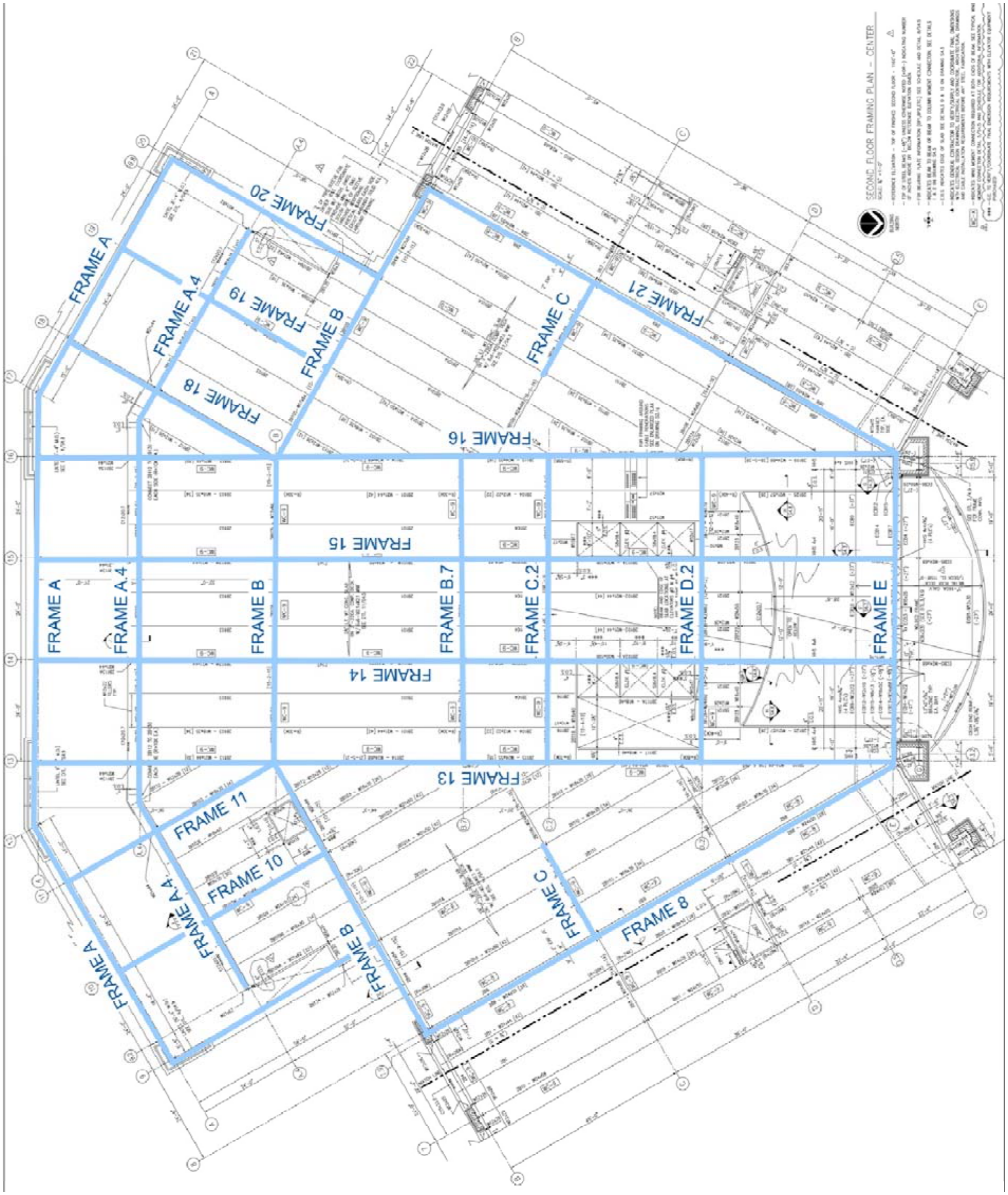
The Corporate Headquarters building for the Westinghouse Electric Company is located in Cranberry, Pennsylvania. Just north of the city in Butler County, the site is on 83 acres in an office park easily accessible by I-79 and PA-228. With five above grade floors and a full 17' high basement, Building One will be the main building on this campus. Complete with cafeteria, gym, locker rooms, offices, and executive conference rooms, the flagship building comes well equipped and diverse. At 434,800 square feet, the building makes quite an architectural statement.

The main building utilizes a powerful entrance with a two-story atrium to express its importance. The first floor also has a height of 18'-0" to emphasize a larger space while floors two through four have floor-to-floor heights of 14'-0". The fifth floor has a height of 14'-6". Building one has a total height of 74'-6" above grade. Aluminum and glass curtain walls add light and make the building feel more open while polished concrete at the base of the brick façade accentuate the height. The foundation system consists of caissons in addition to some spread footings and grade beams. A typical bay is 45'-0" by 24'-0", and uses a steel system with composite beams and deck. In most of the building, the girders are not composite, but the beams framing into the girders have some composite action. The floor system is a 2" 22 gage steel deck with 2-1/2" of lightweight concrete topping. The Westinghouse Electric Company Corporate Headquarters Building One has two expansion joints present, thus creating essentially three structural buildings inside of one. The expansion joints create the East, Center, and West parts of the building. These joints can be seen along column lines 7.9 and 8 between the east and center portions, and column lines 21 and 21.1 between the center and west parts of the building.

The lateral system of this building is comprised of wind moment connections at every column and most of the girders. Wind and seismic loading were recalculated in order to compare to RAM and for general purposes based on ASCE 7-05. Torsion was calculated based on the overall stiffness of the frames in the building. Torsion from wind and seismic forces was taken into account in the member checks of the lateral system. Portal analysis was performed in order to find the moments on the members chosen for more extensive member check analysis. All members chosen were found to be adequate. Uplift was considered and analyzed for potential issues, but it was clearly not an issue due purely to the weight of the building, as can be seen in this report.

Second Floor Plan East with Frames Highlighted

Second Floor Plan Center with Frames Highlighted



STRUCTURAL SYSTEMS

FOUNDATIONS

Sixty-seven caissons are the main elements in the foundation system. Each was designed to carry 8,000 psf. The caissons range from 36" to 84" in diameter and from 8'-0" to 30'-8" in height. On top of each caisson, there is a 2'-6" cap with #6 @8" each way on the top and the bottom as well as base plates for the columns. The 5" slab on grade in the basement bears directly on the soil and the thickened slabs under the non-load bearing walls. On the south side and the east portion of the building, where caissons are not present, there are spread footings or grade beams. The sub-grade walls in the basement (referred to in drawings as grade beams) range from 1'-4" to 1'-8" wide and are 14'-4" deep. The bottom reinforcement in the grade beams is mainly (3) #6, but varies from #6 to #9 and in number. Top reinforcement also varies from #6 to #9 and from two bars to four bars. All end reinforcing bars are #6, but vary from two bars to four bars.

FLOOR SYSTEM

The floor system for the corporate headquarters main building consists of 2" 22 gage metal deck with 2 ½" lightweight concrete topping, for a total slab depth of 4 ½". The typical bay size of this composite steel system is 24'-0" by 45'-0". W21 beams (W21x44 typ.) spaced 24'-0" on center and W18x35 beams spaced 8'-0" on center support the deck and transfer the load to the W24 girders (W24x55 typ.). The girders then continue to transfer the load to the columns. The 5" thick slab-on-grade in the basement of the headquarters is the exception to the typical floors. The roof uses a different system consisting of 1 ½" 20 gage roof deck, steel beams and steel K series joists. However, the penthouse system uses 2" 20 gage metal deck with a 2 ½" lightweight concrete topping. Where the penthouse is absent, roof uses a fully adhered EPDM roofing system including the membrane over ½" protection board over tapered insulation over 5/8" type X GWB over the roof decking.

LATERAL SYSTEM

The Westinghouse Corporate Headquarters Building One uses moment connections at every column to resist lateral loads from wind and seismic forces and torsion forces. Wind moment connections with angles and bolts are provided at all members in the lateral system of the building.

COLUMNS

The columns used in the headquarters are typical for a mid-rise building. The large columns in the basement and first floor of the building are W36x230 at the largest, but typically are W14x90. The W36x230 columns are so large due to the entire front façade of the building bearing on a W36x230 beam and the two columns. On the roof, any columns that do not continue up from the fifth floor are W10x49 or W10x33. The rest of the building is generally the same size, of course with some smaller sizes of columns, such as W10's on the fifth and roof levels. The base plates have four possible layouts and range in thickness from 1 ¾" to 3".

CODE AND DESIGN REQUIREMENTS

These are the design standards, codes, and design criteria used by the design professional and in the calculations for this report.

APPLICABLE DESIGN STANDARDS

THE 2006 INTERNATIONAL BUILDING CODE

ACI 318-05 (REINFORCED CONCRETE DESIGN)

AISC STEEL CONSTRUCTION MANUAL, 13TH EDITION

ACI 530 (MASONRY STRUCTURES)

ASCE 7-05 (MINIMUM DESIGN LOADS FOR BUILDINGS AND OTHER STRUCTURES)

DEFLECTION CRITERIA

FLOOR DEFLECTION CRITERIA

L/240 TOTAL LOAD

L/360 LIVE LOAD

L/600 CURTAIN WALL LOAD

LATERAL DEFLECTION CRITERIA

H/400 TOTAL ALLOWABLE WIND DRIFT

H/400 STORY WIND DRIFT

H/50 TOTAL ALLOWABLE SEISMIC DRIFT (Δ=0.02H_{SX} FROM TABLE 12.12-1 ASCE 7-05)

Materials

The materials used in the Westinghouse Electric Company Corporate Headquarters as listed on the general notes page of the structural drawing set are as follows and were used in design and analysis as appropriate.

Concrete

Freezing Temperature Exposure	Air entrained (6% ±1%)
Slab-on-grade	4,000 PSI
Slab-on-deck	4,000 PSI
Caissons	3,000 PSI
Footings and Caisson Caps	3,000 PSI
Walls and Piers	4,000 PSI
Over excavation fill	2,000 PSI

Reinforcing Steel

Reinforcing Bar	ASTM A-615
Welded Wire Fabric	ASTM A-185

Structural Steel

W-Shapes	ASTM A-992
C-Shapes	ASTM A-36
Steel Pipe	ASTM A-501
Tubes	ASTM A-500 Grade B

Metal Deck

Bolts	ASTM A-325, ¾" diameter
Deck	ASTM A611 Grade C or D
Studs	¾"x 3 ½" headed stud

Masonry

CMU	ASTM C-90
Concrete Brick	ASTM C-55 type N-1
Mortar	ASTM C-270
Grout	ASTM C-476

Gravity and Lateral Loads

The loads on the building are applied as such based on the design professional's specification on the drawings. It is understood these values are conservative since the live load of 80 PSF is used everywhere on the upper floors and a partition load is also used. This is discussed further in other sections of the report. The load combinations from IBC 2006 were taken into consideration and the boxed ones were used for the lateral analysis of the frames in the building.

Dead Loads

Construction Dead Load

Concrete	115 PCF
Steel	490 PCF
Partitions	10 PSF
M.E.P.	5 PSF
Finishes	3 PSF

Live Loads

Public Areas	100 PSF
Lobbies	100 PSF
Corridors above 1st	80 PSF
Office	50 PSF
Mechanical	150 PSF
Stairs	100 PSF

From IBC 2006:

1605.2.1 Basic Load Combinations

(As applied to this Report)

1.4 D	Eq 16-1
1.2D + 1.6L	Eq 16-2
1.2D+1.0L	Eq 16-3
1.2D+0.8W	Eq 16-3
1.2D+1.0L+1.6W	Eq 16-4
1.2D+1.0E+1.0L	Eq 16-5
0.9D+1.6W	Eq 16-6
0.9D+1.0E	Eq 16-7

WIND DESIGN

Wind loads were analyzed using section 6.5 of ASCE 7-05. The loads were evaluated as well as analyzed for the North-South direction and the East-West direction. These loads do not have torsion taken into account in them. These calculations can be viewed in Appendix C, but were calculated using the same method as Technical Report One. The loads with torsion taken into account can be viewed in the torsion section of this report. A RAM analysis was performed for the East section of the building solely to obtain a check of the total loads per floor and details of values can be seen in Appendix C. However, in order to check the calculations against RAM's values, a calculation of the values for just the East portion of the building was required. One of the main discrepancies between the RAM values and the hand-calculated ones is the value for the natural moment. RAM has two values (one for each direction) that are different from the one determined for this report. One of the reasons for the difference is due to computer programs having different assumptions and the capabilities to quickly do a finite element analysis.

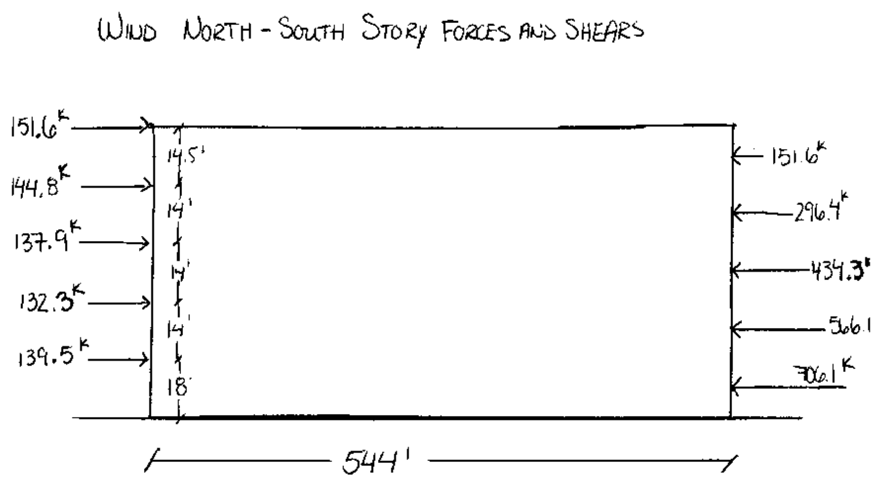
Floor Heights	Level	Total Height	K_Z	q_Z	Wind Pressures (psf)					
					N-S Windward	N-S Leeward	N-S Side Wall	E-W Windward	E-W Leeward	E-W Sidewall
14.5	Roof	74.5	0.908	13.924	11.09	-8.13	-10.38	11.86	-4.84	-10.69
14	5	60	0.85	13.034	10.54	-8.13	-10.38	11.67	-4.84	-10.69
14	4	46	0.79	12.114	9.97	-8.13	-10.38	11.26	-4.84	-10.69
14	3	32	0.712	10.918	9.24	-8.13	-10.38	10.85	-4.84	-10.69
18	2	18	0.59	9.201	7.89	-8.13	-10.38	10.64	-4.84	-10.69

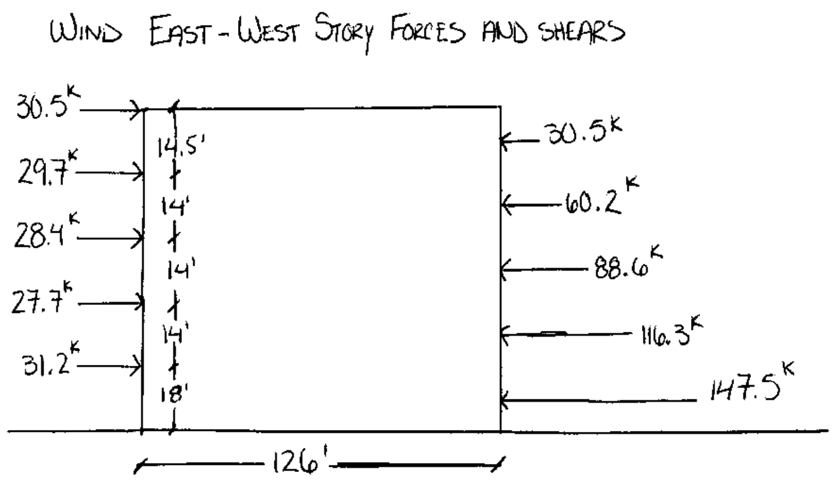
Wind Pressure with respect to height before torsion

Level	Wind Design					
	Load (kips)		Shear (kips)		Moment (ft-k)	
	N-S	E-W	N-S	E-W	N-S	E-W
Roof	151.6	30.5	0	0	2198.6	442.4
5	144.8	29.7	151.6	30.5	2026.7	415.2
4	137.9	28.4	296.4	60.2	1930.7	397.7
3	132.3	27.7	434.3	88.6	1852.1	387.5
2	139.5	31.2	566.6	116.3	2511.1	562.0
Total	706.1	147.5	706.1	147.5	10519.2	2204.8

Note: Total Base Shear includes load from Windward and Leeward pressures

The wind story forces are summarized in these pictures of each side of the building. The story forces are on the left and the story shears are on the right side of the pictures.





These values are not extraordinary. The RAM checked values are different from the calculated ones from the point where q_z values come into the picture. They may be different based on different assumptions, or RAM may have actually calculated the values instead of using Table 6-3 in ASCE 7-05.

From Table 6-3

H (ft)	K_z	q_z
74.5	0.908	13.9235
60	0.85	13.0341
46	0.79	12.1141
32	0.712	10.918
18	0.59	9.20056
0	0.57	8.74054

From RAM

H (ft)	K_z	q_z
74.5	0.909	16.013
60	0.854	15.053
46	0.792	13.952
32	0.714	12.578
18	0.605	10.671
0	0.575	10.130

Since the wind pressures do not start out with the same q value, it can hardly be expected that the RAM wind pressures would coincide with the calculations done for the same portion of the building by hand. RAM also calculated the Gust Factor (G) for each floor, and this was not done by hand since it is not a code requirement.

Design Wind Pressures p in E-W Direction (Table 5.41)						
Location	Height above Ground Level z (ft)	q(psf)	External Pressure qGC_p (psf)	Internal Pressure q_hGC_{pi}	Net Pressure p (psf)	
					+Gcpi	-Gcpi
Windward	74.5	13.92	9.35	2.51	6.85	11.86
	60	13.03	8.76	2.51	6.25	11.26
	46	12.11	8.14	2.51	5.63	10.64
	32	10.92	7.33	2.51	4.83	9.84
	18	9.20	6.18	2.51	3.67	8.69
	15	8.74	5.87	2.51	3.37	8.38

Design Wind Pressures p in N-S Direction (Table 5.41)						
Location	Height above Ground Level z (ft)	q(psf)	External Pressure qGC_p (psf)	Internal Pressure q_hGC_{pi}	Net Pressure p (psf)	
					+Gcpi	-Gcpi
Windward	74.5	13.92	9.23	2.51	6.73	11.74
	60	13.03	8.64	2.51	6.14	11.15
	46	12.11	8.03	2.51	5.53	10.54
	32	10.92	7.24	2.51	4.73	9.74
	18	8.74	5.80	2.51	3.29	8.30
	15	8.74	5.80	2.51	3.29	8.30

Design Wind Pressures p in RAM (Table 5.41)						
Location	Height above Ground Level z (ft)	q(psf)	External Pressure qGC_p (psf)	Internal Pressure q_hGC_{pi}	Net Pressure p (psf)	
					+Gcpi	-Gcpi
Windward	74.5	16.01	10.62	2.51	8.11	17.93
	60	15.05	9.98	2.51	7.47	17.43
	46	13.95	9.25	2.51	6.74	16.64
	32	12.58	8.34	2.51	5.83	15.65
	18	10.67	7.07	2.51	4.57	9.58
	0		0.00	2.51	-2.51	2.51

All of these calculations were performed using the same method as was used in the whole building part, with the lengths changed to the appropriate numbers.

Floor Heights	Level	Total Height	Hand Calculated Values				RAM Calculated Values		
			K_Z	q_Z	N-S Windward	E-W Windward	K_Z	q_Z	Windward
14.5	Roof	74.5	0.908	13.924	11.74	11.86	0.909	16.013	17.93
14	5	60	0.85	13.034	11.15	11.67	0.854	15.053	17.43
14	4	46	0.79	12.114	10.54	11.26	0.792	13.952	16.64
14	3	32	0.712	10.918	9.74	10.85	0.714	12.578	15.65
18	2	18	0.59	9.201	8.30	10.64	0.605	10.671	2.51

(Floor Heights and Total Height in feet, Windward forces in PSF)

SEISMIC DESIGN AND ANALYSIS

Seismic loads were calculated using ASCE 7-05, Chapters 11 and 12. Since the design professional used a response modification factor of 3.5, R= 3.5 was used. However, because not all structures are constructed to the higher standards used for this building, an R-value of 3 was also used for comparison. The site falls into site class D, seismic category B, and occupancy category II. Factors and accelerations were calculated using ASCE 7-05. After analysis, a response factor of R=3.0 would result in higher story forces and therefore higher shears and moments than the R=3.5 from the analysis performed and the design professionals' analysis. Since the R=3.0 created higher shears and moments, the forces from the R=3.0 were used in the torsion calculation for the building.

This portion of the building was also checked against values from RAM and recalculated just for the East portion of the building. The RAM assumptions and calculated values can be seen in Appendix D.

Seismic Design Values, ASCE 7-05		
Occupancy	II	Table 1-1
Importance Factor	I= 1	Table 11.5-1
Site Class	D	Table 20.3-1
Spectral Response Acceleration, short	S_S= 0.12	Figure 22-1
Spectral Response Acceleration, 1 sec	S_1= 0.046	Figure 22-2
Site Coefficient F_a	F_a= 1.6	Table 11.4-1
Site Coefficient F_V	F_V= 2.4	Table 11.4-2
MCE Spectral Response Acceleration, short	S_{MS}= 0.192	Eq. 11.4-1
MCE Spectral Response Acceleration, 1 sec	S_{M1}= 0.1104	Eq. 11.4-2
Design Spectral Acceleration, short	S_{DS}= 0.128	Eq. 11.4-3
Design Spectral Acceleration, 1 sec	S_{D1}= 0.0736	Eq. 11.4-4
Seismic Design Category	B	Table 11.6-1

Seismic Design Values, ASCE 7-05			
Response Modification Coefficient	R= 3	R= 3.5	Table 12.2-1
Coefficient	C_U= 1.7	C_U= 1.7	Table 12.8-1
Fundamental Period	T= 1.497	T= 1.497	Sec. 12.8.2
Seismic Response Coefficient	C_S= 0.016	C_S= 0.014	Eq. 12.8-3
Building Height (above grade)	h= 74.5	h= 74.5	

The values for all the seismic coefficients were determined using ASCE 7-05 equations and tables. The building was first confirmed as Seismic design category B by using Table 11.6-2 of ASCE 7-05. Once the design category had been confirmed, the approximate period was calculated by using equation 12.8-7 and table 12.8-2. Since ASCE 7-05 section 11.6 requires where an S_1 value is less than 0.75 the Seismic Design Category can be determined solely on table11.6-1 and 11.6-2 when T_a> $0.8T_S$, the period used to calculate drift is less than T_S, equation 12.8-2 is used to find C_S, and rigid diaphragms are present.

Calculated Values

S_S=	0.12	(From Figure 22-1)	
S_1=	0.046	(From Figure 22-2)	
S_{MS}=	F_a*S_S =	0.192	
S_{M1}=	F_V*S_1 =	0.1104	
S_{DS}=	$2S_{MS}/3$=	0.128	A (Table 11.6-1)
S_{D1}=	$2S_{M1}/3$=	0.0736	B (Table 11.6-2)

USGS Website Values

S_S=	0.125
S_1=	0.048
S_{MS}=	0.2
S_{M1}=	0.116
S_{DS}=	0.133
S_{D1}=	0.077

C_T =	0.028	(From Table 12.8-2)
X =	0.8	(From Table 12.8-2)

T_a=	$C_t h_n^{\ x}$ =	0.8808116
T_s=	S_{D1}/S_{DS} =	0.575
$0.8T_s$=	0.46	<T_a therefore must use Table 11.6-1,2

T_L= 12 (From Fig. 22-15 p. 228 ASCE 7-05)

C_S values were calculated according to Section 12.8.1.1 equations 12.8-2, 12.8-3, and 12.8-4 and checked against the minimum requirement from EQ 12.8-5 of C_S≥0.01. Equation 12.8-3 is a maximum for this structure, and equation 12.8-4 does not apply since equation 12.8-3 does. The values were then compared based on R and what the professional calculated.

		R=3	R=3.5	R=3.5, consultant
	$S_{DS}/(R/I)$ =	0.0427	0.0366	0.0382
C_S= MAX	$S_{D1}/(T{*}R/I)$ =	0.0164	0.0140	0.0147
for $T>T_L$	$S_{D1}T_L/(T^2R/I)$ =	0.3795	0.3253	0.1181

T= C_U*T_a = 1.4973798

k= 1.499 W= 28502.4

V= C_s*W

466.99 R=3.0
400.28 R=3.5
419.85 R=3.5 from consultant

Floor # 3				
Approx. Area:	72932 ft^2		Floor to Floor Height:	14 ft
Slab:				
22 GA Deck	2			
Thickness=	2.5			
unit weight=	34 PSF			
total weight=	2480 kips			
Columns:				
Shape	Quantity	Unit Weight (lb/ft)	Column Height (ft)	Total Weight
W14x99	19	99	14	26.3 kips
W14x132	6	132	14	11.1 kips
W14x120	18	120	14	30.2 kips
W14x90	8	90	14	10.1 kips
W14x109	6	109		
W14x145	12	145	14	24.4 kips
W14x159	6	159	14	13.4 kips
W14x82	12	82	14	13.8 kips
Total Weight=	129.2 kips			
Beams Connections, Bracing, etc.				
allowance=	11 psf			
Total Weight=	802.3 kips			
Super Imposed Dead:				
partitions=	10 psf			
MEP=	5 psf			
Finishes=	3 psf			
Total Weight=	1312.8 kips			
TOTAL FLOOR WEIGHT:		4724.0 kips	or	64.8 psf

Typical floor weight calculation.

The floor weights used for the seismic calculations were calculated using a 34 PSF deck weight from United Steel Deck over the entire area, added to the column weights. Also, the superimposed loads were added and a bracing allowance to account for beams as part of the floor system.

The different response modification coefficients yield different story forces, story shears, and moments as seen below.

Floor	w_x (k)	h_x (ft)	h_x^k (ft)	$w_x h_x^k$	C_{vx}	Story Force F_x (k)	Story Shear V_x (k)	Moment at Floor (ft-k)
Roof	4240.5	74.5	639.41	2711449	0.359	167.42	0	12473.065
5	4713.6	60	462.27	2178985	0.288	134.55	167.42	8072.7394
4	4726.5	46	310.43	1467216	0.194	90.60	301.97	4167.4204
3	4724.0	32	180.20	851252	0.113	52.56	392.57	1681.9916
2	4653.4	18	76.08	354028	0.047	21.86	445.13	393.48265
1	5444.4						466.99	
Sum	28502.4	74.5	1668.39	7562930	1.000	466.99	466.99	26788.699

Story Shears, Forces and Moments for R=3.0

Floor	w_x (k)	h_x (ft)	h_x^k (ft)	$w_x h_x^k$	C_{vx}	Story Force F_x (k)	Story Shear V_x (k)	Moment at Floor (ft-k)
Roof	4240.5	74.5	639.41	2711449	0.359	143.51	0	10691.199
5	4713.6	60	462.27	2178985	0.288	115.32	143.51	6919.4909
4	4726.5	46	310.43	1467216	0.194	77.65	258.83	3572.0746
3	4724.0	32	180.20	851252	0.113	45.05	336.48	1441.7071
2	4653.4	18	76.08	354028	0.047	18.74	381.54	337.27085
1	5444.4						400.28	
Sum	28502.4	74.5	1668.39	7562930	1.000	400.28	400.28	22961.742

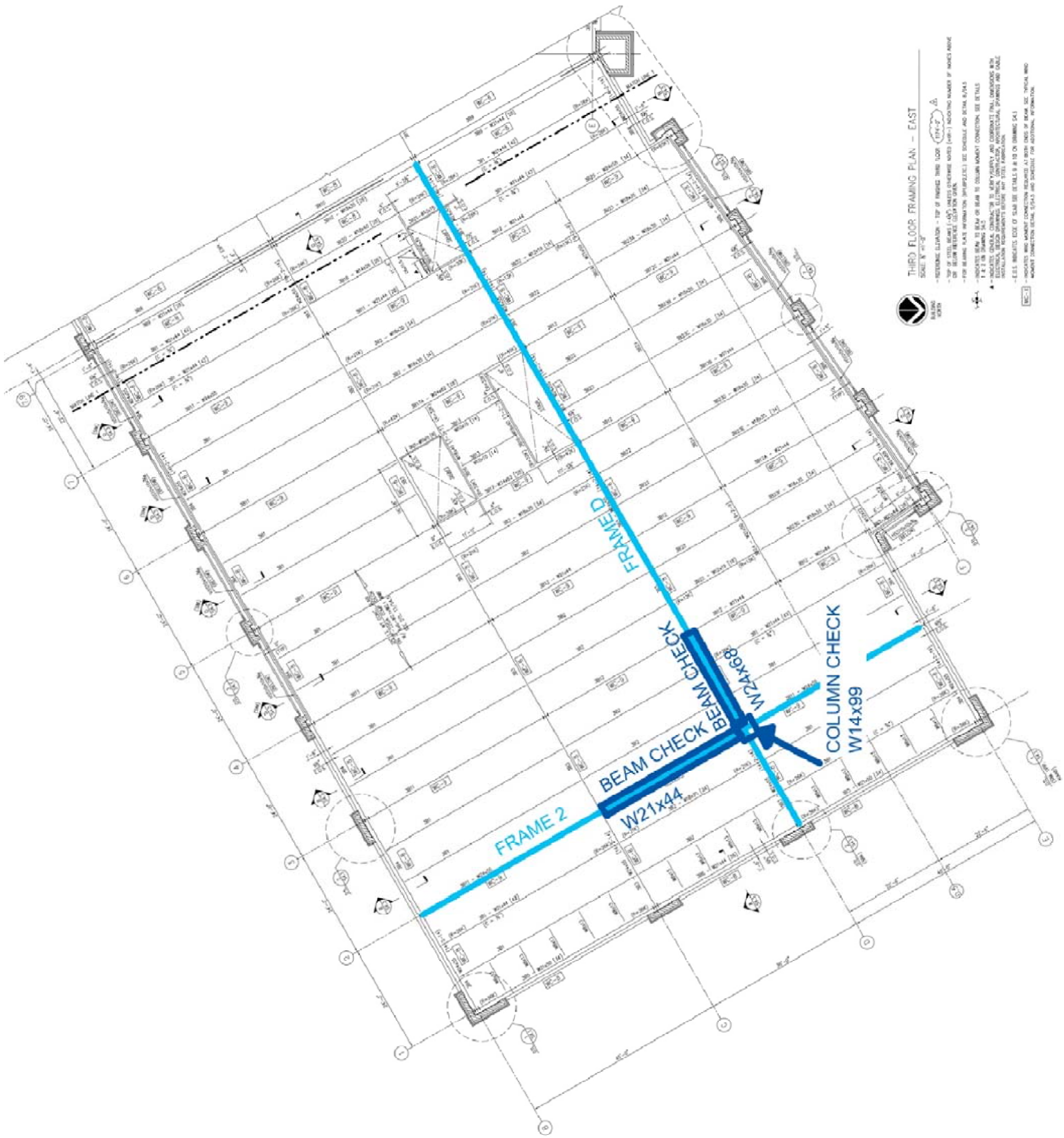
Story Shears, Forces, and Moments for R=3.5

Floor	w_x (k)	h_x (ft)	h_x^k (ft)	$w_x h_x^k$	C_{vx}	Story Force F_x (k)	Story Shear V_x (k)	Moment at Floor (ft-k)
Roof	4240.5	74.5	639.41	2711449	0.359	150.53	0	11214.138
5	4713.6	60	462.27	2178985	0.288	120.97	150.53	7257.9442
4	4726.5	46	310.43	1467216	0.194	81.45	271.49	3746.7956
3	4724.0	32	180.20	851252	0.113	47.26	352.94	1512.2254
2	4653.4	18	76.08	354028	0.047	19.65	400.20	353.76779
1	5444.4						419.85	
Sum	28502.4	74.5	74.5	7562930	1.000	419.85	419.85	24084.871

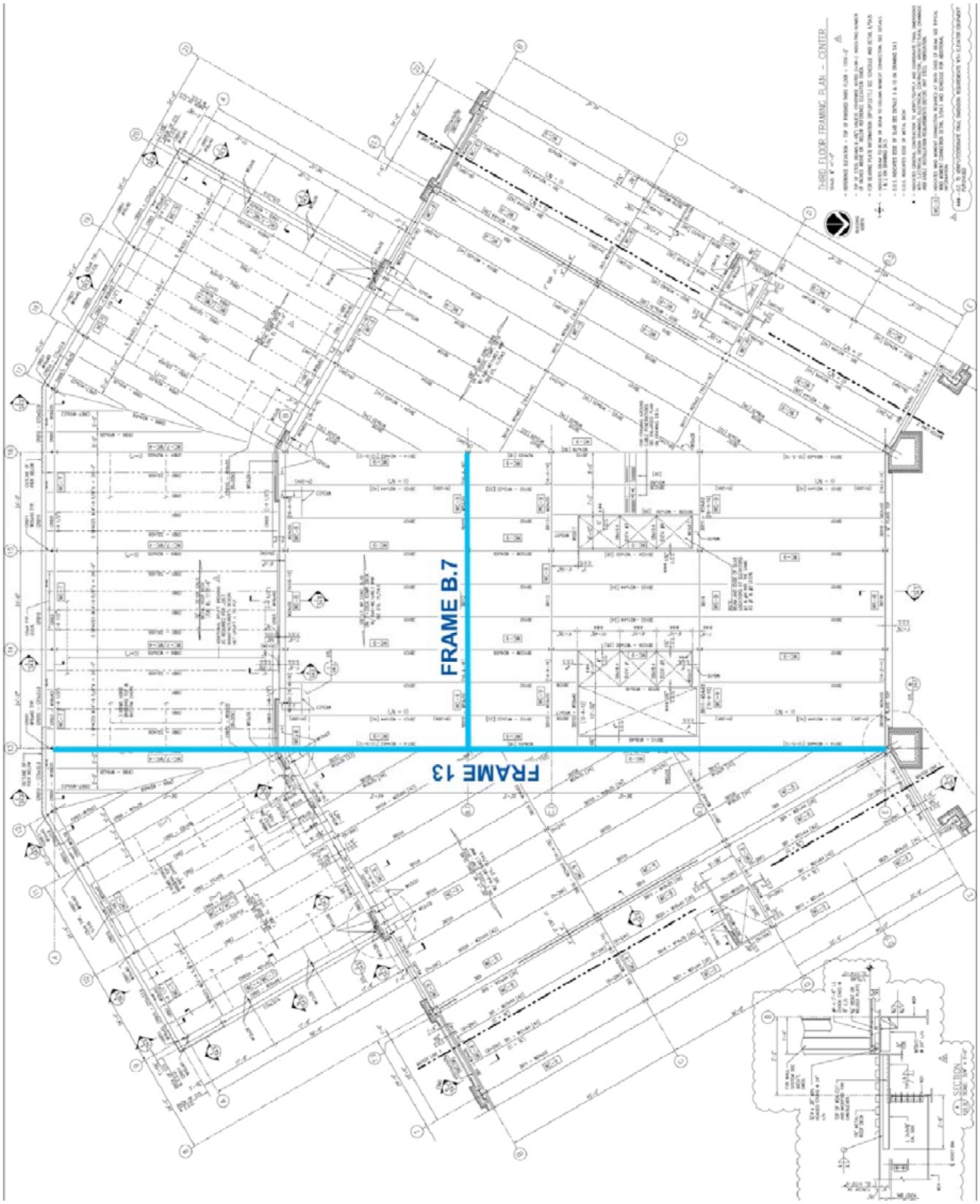
Story Shears, Forces, and Moments, R=3.5 and C_s from design professional

Results: The story forces, shears, and moments are similar between all three different coefficients. The R=3.0 values are the most conservative, as was expected.

THIRD FLOOR LAYOUT EAST FOR TORSION VALUES CALCULATED AND PORTAL FRAMES INDICATED IN LIGHT BLUE AND DARK BLUE ON THE FRAME LINES AND THE SPOT CHECKED MEMBERS HIGHLIGHTED.



THIRD FLOOR LAYOUT CENTRAL FOR TORSION CALCULATED FRAMES AND PORTAL ANALYSIS DETERMINED FRAMES INDICATED IN LIGHT BLUE WITH DARK BLUE TEXT.



TORSION

Torsion needs to be accounted for in lateral systems due to the possibility for twisting and portions of the building being loaded in a non-uniform manner. To find the torsion effects, stiffness of the building needed to be calculated for each of the frames. The stiffness is used as a basis to distribute the lateral loading through the building frames. Since expansions joints are present in the Westinghouse Electric Company Corporate Headquarters building, the building can be structurally treated as three separate buildings based on the locations of the joints. Since the building is symmetrical on the East and West portions, the values obtained on the East calculations would be identical to the ones for the West. The values for the calculations in the center of the building are available upon request and the values for the remaining floors in the East portions of the building can be found in Appendix B. The K factors were determined using the deflections from a 1 kip load in a STAAD analysis of each frame. From the stiffness in the frame direction, a distribution factor was obtained for each direction by finding the percent the stiffness would contribute out of a total of one.

5TH FLOOR EAST BUILDING (SAME AS WEST)

Frame	Load (K)	K in the X Direction	K in the Y Direction	Distance from X 0rigin (ft)	Distance from Y 0rigin (ft)	Distribution Factor	MID OF FRAME X	MID OF FRAME Y
B	1	45.4545	0.0000	0	126	0.272		
C	1	52.6316	0.0000	0	81	0.314		
D	1	52.6316	0.0000	0	45	0.314	83.25	45
D.5	1	0.0000	0.0000	0	22.5	0.000		
D.8	1	0.0000	0.0000	0	7	0.000		
E	1	16.6667	0.0000	0	0	0.100		
F	1	0.0000	0.0000	105	-14	0.000		
0.7	1	0.0000	43.4783	0	0	0.159		
0.8	1	0.0000	43.4783	12.5	0	0.159		
0.9	1	0.0000	43.4783	36.5	0	0.159		
1	1	0.0000	13.8889	57	0	0.051		
2	1	0.0000	13.8889	81	0	0.051	63	81
3	1	0.0000	13.8889	105	0	0.051		
4	1	0.0000	24.3902	129	0	0.089		
5	1	0.0000	20.8333	153	0	0.076		
6	1	0.0000	20.8333	177	0	0.076		
7	1	0.0000	20.8333	201	0	0.076		
7.9	1	0.0000	13.8889	223.5	0	0.051		

The centers of rigidity for each direction were determined by taking the sum of the stiffness in one direction multiplied by the distance from the perpendicular distance from the origin (distance from the opposite origin) all divided by the sum of the stiffness for each frame used in the calculation. For example, the center of rigidity in the Y direction was calculated by taking the K in the X direction and multiplying it by the distance from the Y origin, and finding the sum of all the values divided by the sum of the K's in the X direction (Kix*dix/ΣKix). The Ix was calculated by taking the sum of the K's in the X direction multiplied by the distance from the Y origin squared ($\Sigma kix*yi^2$). The Iy was calculated using the same method.

Center of Rigidity in Y= Kix*dix/Σkix
Center of Rigidity in X= Kiy*dix/Σkiy
$Ix= \Sigma kix*yi^2$
$Iy= \Sigma kiy*xi^2$

Center of Rigidity in Y Direction (ft) Kix*dix/ΣKix	Center of Rigidity in X Direction (ft) Kiy*diy/Σkiy	Ix $\Sigma kix*yi^2$	Iy $\Sigma kiy*xi^2$
73.84	83.62	1173531	3435814

After the K values were determined, the K in the direction of the force was divided by the sum of the K's in the same direction and multiplied by the force in the specific direction. The torsion induced moment in each direction was determined differently for wind and for seismic.

For seismic forces, the torsion induced moment was calculated by taking the force at the specified story multiplied by the center of rigidity subtracted from center of mass in the direction perpendicular to the force.

Torsion Induced Moment X= (Force)*(Center of Mass in Y direction- Center of Rigidity in X direction)

Story	Force (K)	Direct Force on Frame 2 (K) Fiy=(Kiy/ΣKiy)F	Torsional Force on Frame 2 (K-ft) Fix=((Ki*xi)/Ip)M	Torsional Force on Frame 2 (K-ft) Fiy=((Ki*yi)/Ip)M	Total Force on Each Story (K) F=DF+TF	Center of Mass in X Direction (ft)	Center of Mass in Y Direction (ft)	Torsional Moment (K-ft) X	Torsional Moment (K-ft) Y
5	167.42	8.52	0.000	0.208	8.73	141.14	64.32	9630	1593
4	134.55	6.85	0.000	0.163	7.01	139.68	60.38	7543	1810
3	90.6	4.61	0.000	0.110	4.72	139.87	60.26	5096	1230
2	52.56	2.68	0.000	0.064	2.74	139.87	60.31	2956	711
1	21.86	1.11	0.000	0.025	1.14	136.33	59.94	1152	304

For wind, it was determined by taking the force perpendicular to the specified moment multiplied by the center of rigidity subtracted from center of the wall.

Torsion Induced Moment X= (force in Y)*(Center of Wall in X- Center of Rigidity in X)

Story	Force X (K)	Force Y (K)	Direct Force on Frame 2 (K) Fix=(Kix/ΣKix)F	Direct Force on Frame 2 (K) Fiy=(Kiy/ΣKiy)F	Torsional Force on Frame 2 (K-ft) Fix=((Ki*xi)/Ip)M	Torsional Force on Frame 2 (K-ft) Fiy=((Ki*yi)/Ip)M	Total Force on Each Story (K) $F=DF+TF_X+TF_Y$	Center of Mass in X Direction (ft)	Center of Mass in Y Direction (ft)	Torsional Moment (K-ft) X	Torsional Moment (K-ft) Y
5	30.5	151.6	0.00	7.72	0.00	0.07	12.45	141.14	64.32	3126	1086
4	29.7	144.8	0.00	7.37	0.00	0.06	11.90	139.68	60.38	2986	1037
3	28.4	137.9	0.00	7.02	0.00	0.06	11.33	139.87	60.26	2844	988
2	27.7	132.3	0.00	6.73	0.00	0.06	10.87	139.87	60.31	2728	948
1	31.2	139.5	0.00	7.10	0.00	0.06	11.46	136.33	59.94	2877	999

These calculations were performed for each floor where the frames change, which in this case is floor levels 5, 4, 2, and 1.

The final lateral forces on each frame are seen below with the controlling force highlighted in red. These frames are the ones chosen for the portal analysis, which can be seen in Appendix E and the following section.

Frame 2		
Story	Seismic	Wind
5	8.73	12.45
4	8.91	11.00
3	6.00	10.48
2	3.48	10.56
1	1.16	10.59

Frame 13		
Story	Seismic	Wind
5	36.28	53.43
4	26.87	47.76
3	18.10	45.50
2	10.22	43.73
1	0.58	5.99

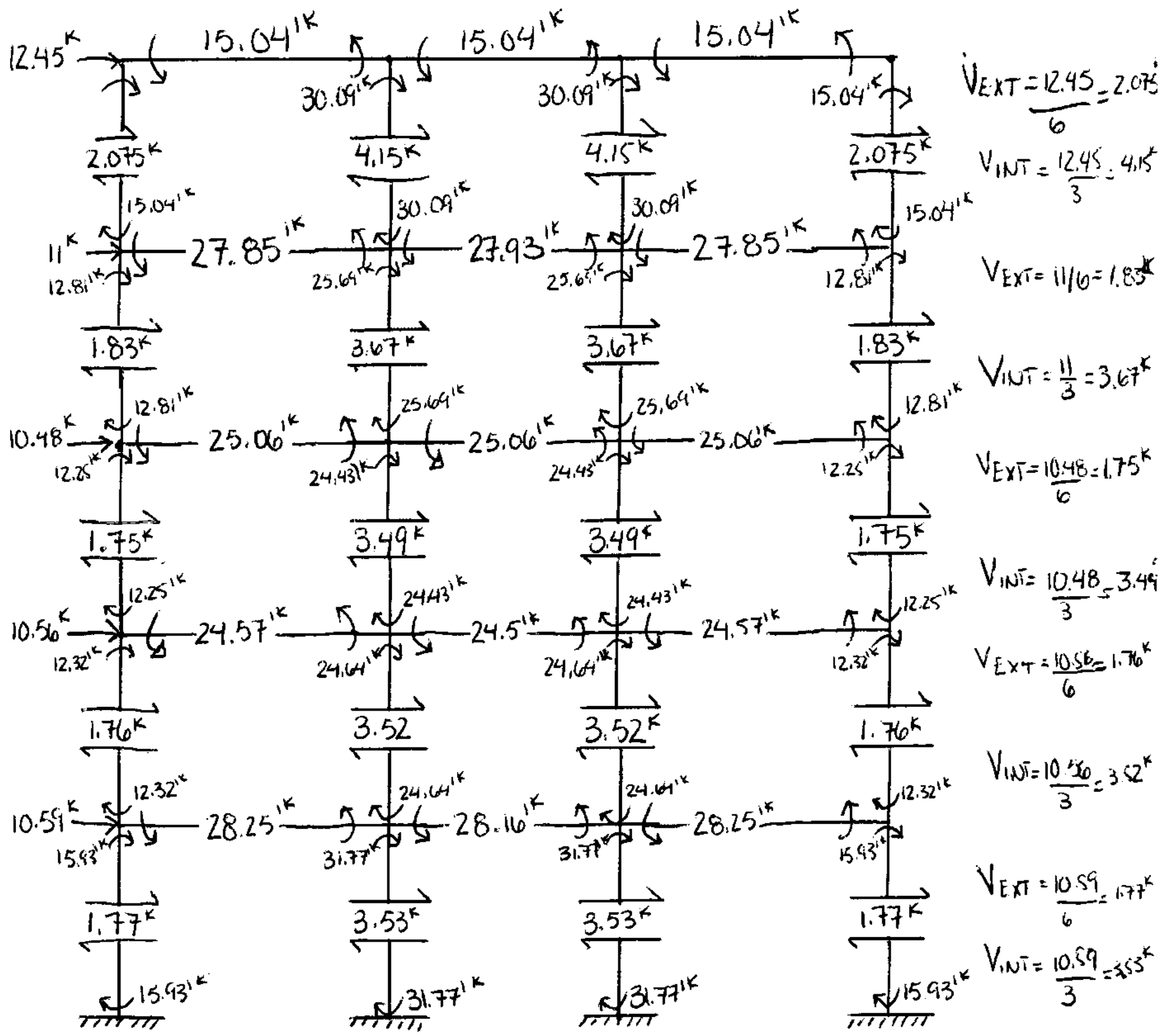
Frame D		
Story	Seismic	Wind
5	52.65	15.37
4	29.84	10.52
3	20.08	10.06
2	11.65	10.53
1	16.59	35.39

Frame B.7		
Story	Seismic	Wind
5	28.14	8.42
4	22.53	8.06
3	15.17	7.71
2	8.84	7.48
1	2.45	6.48

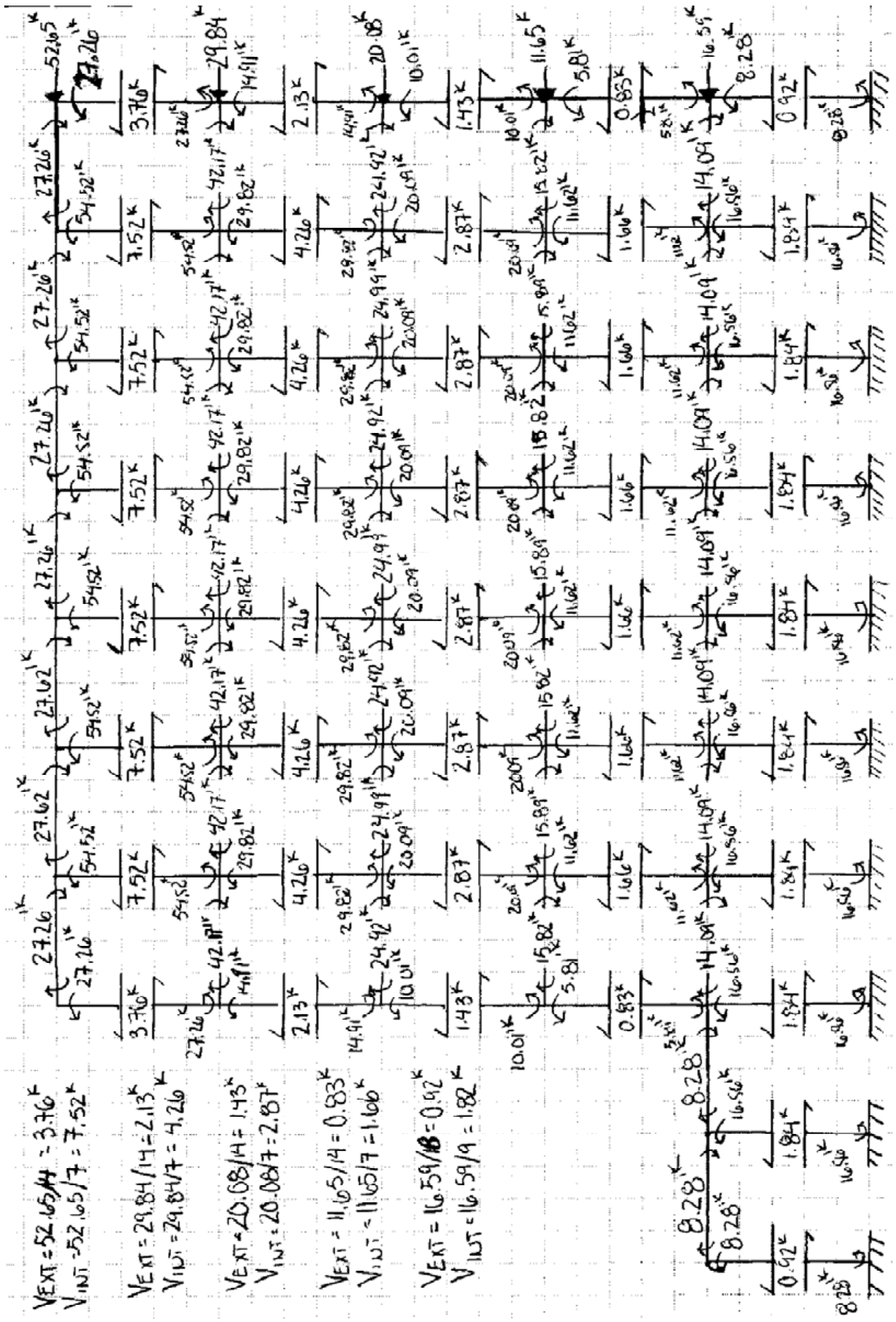
PORTAL METHOD

A portal method analysis was performed to find the moments and shear forces in the members of four frames (two from each the East and Center portions). This analysis was performed using the controlling force of either wind or seismic on the individual frame as determined through the torsion calculation section.

Frame 2 East Building



Frame D East Building



MEMBER CHECKS

For this portion of the report, members evaluated were a W21x44 girder, a W24x68 girder, and a W14x99 column shown in the 3rd floor drawing in the previous Section and in Appendix A. For the member checks, first, the distributed loads were calculated and from those values the moments were found. After determining the moments on the girders, the lateral moment generated was added (depending on which force controlled for the frame the member is part of) and then proceeded to recalculate the distributed loads, with and without load combination factors. Once the moments and shears had been determined, the values were checked the values against the AISC table for moment and shear capacity of steel members (Table 3-2). Using a live load of 80 PSF and a partition load of 10 PSF is extremely conservative since the 80 PSF live is treating the entire floor as if it were a corridor, and the floor will be more than just a corridor. Office loading requires 50 PSF live and then a partition load of 15 or 20 PSF would be more acceptable (IBC 2006 1607.1 and 1607.5). The loads are comparable, but the current loads could potentially be reduced as the designer chooses.

Loading:

Live Load=	80	psf
Superimposed Dead=	18	psf
+selfweight		
Wind=	24.5	k-ft
Seismic=	15.89	k-ft
Roof Live=	20	psf
W_U=	149.6	psf

Materials:

f'_C=	4000	psi
f_y=	50	ksi (beams/girders)
f_y=	60	ksi

2" Metal Deck, 22 Gage
2 1/2" LWC
3/4" Diameter, 4" studs

W 24x55, A_S=	16.2	in^2
W 18x53, A_S=	10.3	in^2
W 24x68, A_S=	20.1	in^2
W 21x44, A_s=	13	in^2

MEMBER CHECKS CONTINUED:

Check Composite Deck:

W_U= 101.6 psf

From United Steel Deck, Inc Catalog (P. 38)

Max Unshored span allowed=8.56' for 3 spans >8' OK

Max uniformed live load for 8' span= 235 PSF> 105 PSF OK

Check Girder (W 21x44) on Frame 2:

W_U= 0.813 klf, w/o load factor, w= 0.78

M_U= 131.7 k-ft +wind moment

156.2 k-ft

W_U= 0.964 klf, w/o load factor, w= 0.88

V_U= 17.35 k

ϕM_p= 358 k-ft > 156.2 OK Table 3-2 AISC

ϕV_n= 217 k > 17.4 OK Table 3-2 AISC

I_{LB}= 843 in^4 Table 1-1 AISC

Δ_{MAX}= $5wl^4(1728)/(384EI)$= 1.36

Δ_{D+L}= l/240= 1.8 OK

MEMBER CHECKS CONTINUED:

Check Girder (W 24x68) on Frame D:

P_U=	14.63	k w/o load factors P=		14.112	
M_U=	87.78	k-ft	+wind moment		
	103.7	k-ft			
P_U=	17.28	k w/o load factors P=		16.76	
V_U=	8.639	k, w/o load factors V=		8.38	
ϕM_p=	664	k-ft >	103.7	OK	Table 3-2 AISC
ϕV_n=	295	k >	8.6	OK	Table 3-2 AISC
I=	1830	in^4		Table 1-1 AISC	
Δ_{MAX}=	$PL^3/(48EI)$=		0.157		
Δ_{D+L}=	L/240=	1.2			
	Max Deflection< Allowable			OK	

For the column check, column D2 was chosen. This column is also on the third floor of the building. At this part of the building, the wind moment is greater than the seismic moment, so the wind moment was utilized. A live load reduction calculation was performed, and the equation was greater than the minimum value ($0.4*L_O$), and was therefore used during the remainder of the check. The tributary area and influence areas were also calculated in this portion of the report. The live load was determined by using the reduced live load multiplied by the tributary area for each floor and the roof live load over the tributary area on the roof. The dead load was calculated using the weight per square foot of the deck and concrete from the United Steel Deck manual (page 38) for 2"-22 gage deck with 2.5" of 115 PCF concrete and then the superimposed loads (for partitions, finishes, and MEP) and beam weights carrying load to the column were added. The dead load was calculated for the two floors of load the column carries by using the tributary area and the calculated distributed load. For the factored P, the 1.2D+1.0L was used since the wind is added to it with a factor of 1.6 applied. For the moments on the column, it was assumed that one half of the moment from the load combination would follow the load path into the column and the other half would go to the beam. The moment was calculated using the equation $M=wL^2/8$ and then the wind moment (with the 1.6 multiplier applied) was added to it. This final moment value for the load combination 1.2D+1.0L+1.6W was checked in Table 6-1 of the AISC Steel Construction Manual. The interaction equation $p*P_U+b_x*M_{UX}$ was checked and the member was found to be adequate.

MEMBER CHECKS CONTINUED:

Check Column (W14x99) D2 at 3rd Floor:

Live Load:

Area=24*(45+36)/2= 972

A_T = 2*24*(45+36)/2 = 1944 ft^2

A_I= 4*A_T= 7776 ft^2

$L=L_O[0.25+15/\sqrt{(K_{LL}A_I)}]$= 33.61   0.4*$L_O$= 32

$L<0.4L_O$ Therefore $L=0.4L_O$= 33.61

P_L= A_T*L+L_R*A= 280.8 k

Dead Load:

Dead= Imposed+Deck= 63 psf

P_D= 2 Floors* A*L_D= 122.5 kips

Wind Load:

M_W= 24.43 k-ft

Factored Load:

P_U= 1.2D+1.0L= 427.7

Moments:

Assume 1/2 of $M_{1.2D+1.0L}$ from beam into Column Below= 19.69 k-ft

$M_{1.6W}$= 24.43 k-ft

M_{MAX}= 44.12 k-ft

KL=Lb=14'

From Table 6-1 AISC: Wind Moment= 24.43

p= 0.886 $x10^{-3}$ 1/k

b_x= 1.38 $x10^{-3}$ 1/(k-ft)

p*P_U+ b_x*M_{UX}= 0.44 < 1   Therefore Column is OK

Drift

ASCE 7-05 was used to determine the appropriate allowable drifts for wind and seismic effects. Due to the fact that drift involves serviceability rather than strength, ASCE 7-05 requires in section CC.1.2 the drift be less than H/400 on a wall or frame. The story drift was determined through RAM Analysis and checked against the ASCE 7-05 requirements. RAM Analysis was performed on the east portion of the building. The drift was only checked for the east and west portions of the building for the controlling wind and controlling seismic loads.

Controlling Wind										
Story	Story height (ft)	Story Drift (in)	Allowable Story Drift (in) Δ_{Wind}= H/400			Total Drift (in)	Allowable Total Drift (in) Δ_{Wind}=H/400			
Roof	74.5	0.127	<	0.435	Acceptable	1.02425	<	2.235	Acceptable
5	60.0	0.187	<	0.42	Acceptable	0.89767	<	1.8	Acceptable
4	46.0	0.247	<	0.42	Acceptable	0.71044	<	1.38	Acceptable
3	32.0	0.257	<	0.42	Acceptable	0.46336	<	0.96	Acceptable
2	18.0	0.207	<	0.54	Acceptable	0.20662	<	0.54	Acceptable

For seismic drift, table 12.12-1 was used to find the maximum drift of $0.02h_{sx}$, since the structure falls into the “All other structures” category of the table. This was then converted into an elastic drift ratio using equation 12.8-15 as follows so the values could be compared to RAM output, which is available upon request.

$\delta_x = C_d * \delta_{xe}/I$

$0.02h_{sx}=(3*\delta_{xe})/1.0=$ 0.06

drift ratio= $\delta_{xe}/h_{sx}=0.02*1.0/3=$ 0.0066667

Controlling Seismic				
Story	Story height (ft)	Acutal Drift Ratio	Allowable Total Drift Ratio $\delta_{xe}/h_{sx}=0.02*1.0/3$	
Roof	74.5	0.0011	<	0.006667
5	60.0	0.0013	<	0.006667
4	46.0	0.0014	<	0.006667
3	32.0	0.0012	<	0.006667
2	18.0	0.0006	<	0.006667

Comparing the drift ratios for wind and seismic forces to the allowable drift, it can be concluded that drift is not an issue for either load.

OVERTURNING MOMENT

Overturning moment was calculated using the wind direct forces after the effects from torsion were taken into consideration. The moment was calculated by using the moment arm of the height of the building to the direct forces.

$M_{WN\text{-}S}$= 1.6*Moment from Wind Design=	4157	k-ft
M_E = ΣH(ft)*Load (k)=	1128	k-ft
P_{Uplift}=M/L=	33	k
Load on Opposite Columns= 0.9P_D=	155	k

The load on the opposite columns supports more than enough to resist the uplift generated by the wind moments on the frame. Thus, overturning moment and uplift are not issues.

CONCLUSIONS

From the calculations performed for this report, it can reasonably be concluded that the Westinghouse Electric Company Corporate Headquarters has no issues with lateral systems. Far from having potential lateral issues, the building could be considered overdesigned for the assumptions utilized in this report. Each frame was simply analyzed in order to obtain stiffness for the frame. There are two expansion joints separating the building into parts, which were used throughout this report.

While a live load of 80 PSF was used over the entire upper floors since tenant fit-out drawings are not available at this time, this value is excessive if a partition load is also used. According to the IBC 2006, an office building requires a 50 PSF live load, and would then require a 10-15 PSF partition load depending on the exact partition used. While this would lower the required load on the building, one of the goals was to find the same members are the most efficient for the building as the design professional. For the actual thesis project, this smaller office and partition loading will be used and explored more thoroughly. One of the reasons the building is being analyzed and seen as overdesigned could be due to a heavier dead load than has been assumed from equipment or a computer load throughout the building.

A RAM model was built to use as a check for the lateral forces and to find the drift ratios for the building. The model, however, only includes the East portion of the building. The RAM data for wind and seismic varied from the hand-calculated values, which were based from ASCE 7-05. RAM may have performed a finite element analysis and taken into consideration factors potentially changing at different heights in the building, which are not required by code and were therefore not done in the hand calculation. The checks proved to be of very little value, since they required further hand calculations to be performed in order for a comparison to take place and they did not correlate with the hand-calculated values. For wind, a spreadsheet from a structural design company was used as a third value to check accuracy. The spreadsheet took more into consideration other than this report did. The spreadsheet is not in this report, but a PDF of the values is available upon request. The values confirmed that the hand checked values would be acceptable by design firms.

Torsion can greatly impact a lateral system in a building and was taken into consideration in the lateral analysis in this report. It was calculated for the wind and seismic loads for four frames. Torsion was determined through a stiffness calculation, and then load was distributed throughout the lateral system based on the stiffness of each frame. Based on this analysis, in the long direction of the Westinghouse Electric Company Corporate Headquarters building seismic loads control, but in the shorter direction, where the North-South direction wind blows, the wind load controls. When the overturning moment was calculated, it was evaluated for a frame in the shorter direction and thus with wind load on it. The uplift caused by the overturning moment was resisted by the moment connections and the weight of the building, which was found to be five times the force of uplift.

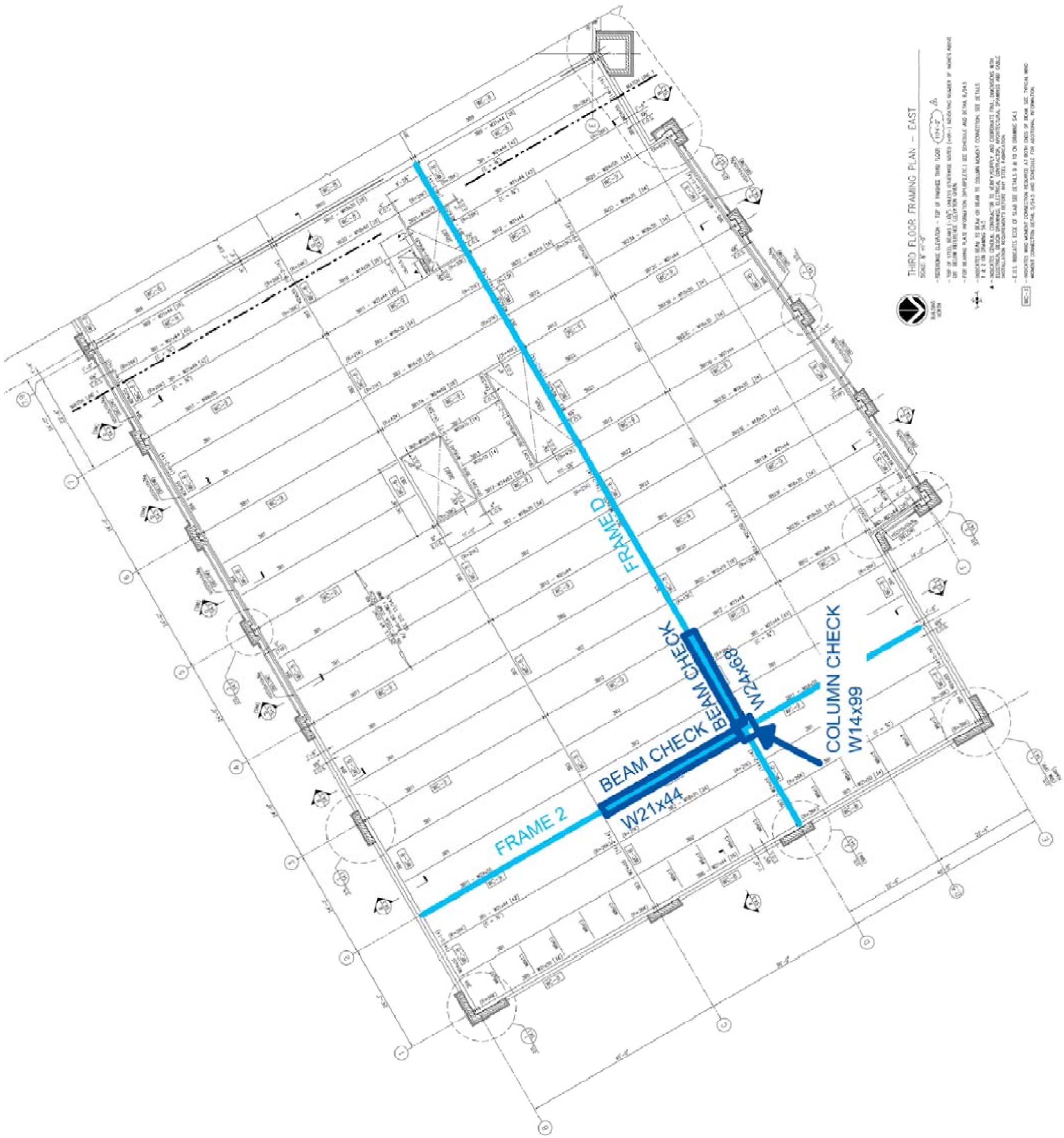
A portal method analysis was performed on four frames in the building, two from each the east and center building portions. The analysis was done so additional moments could be applied in a hand check of the girders and columns in the system.

Further calculations can be found in the appendices. Additional calculations for torsion and wind and seismic loading are available upon request.

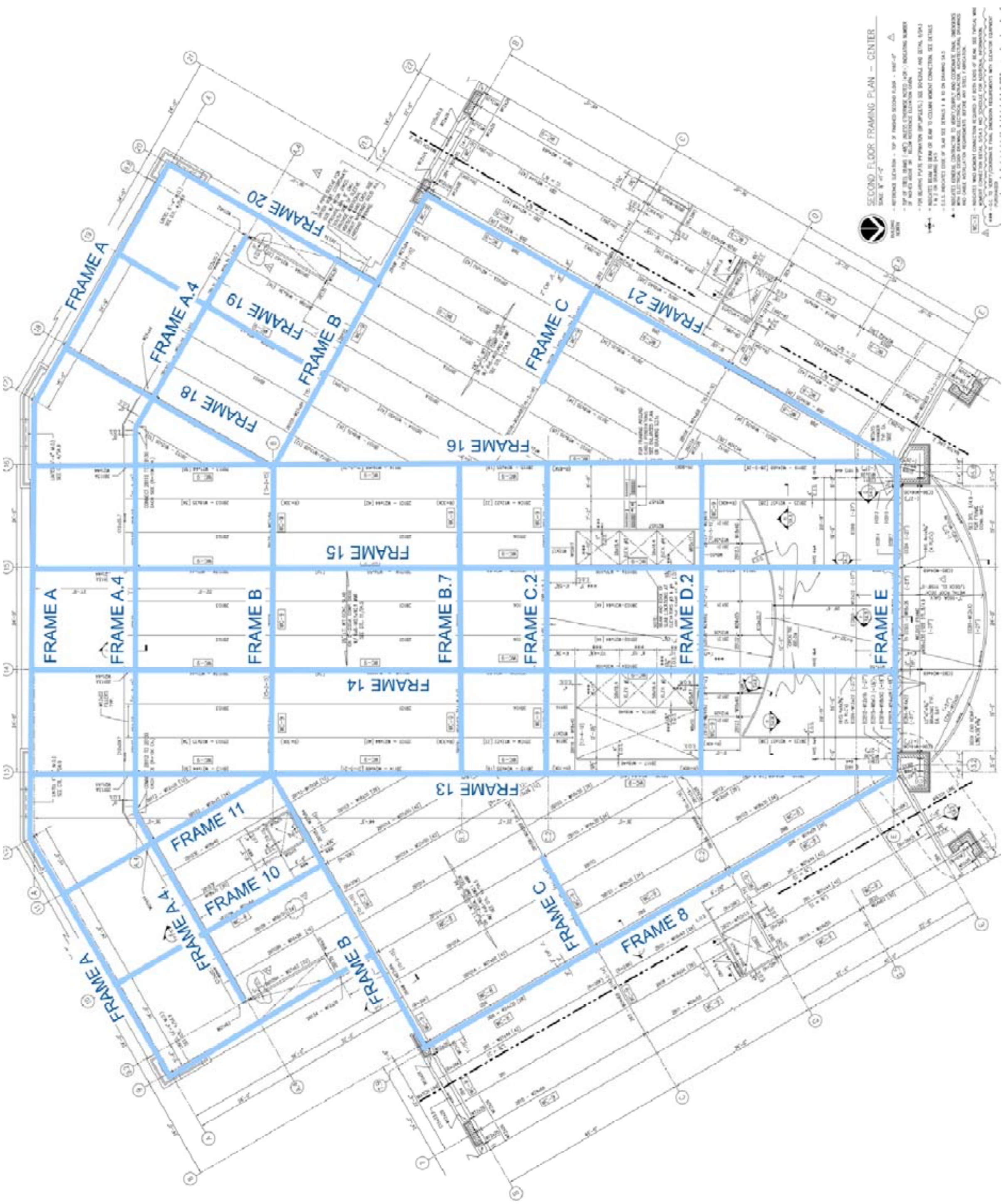
APPENDIX A: TYPICAL FLOOR LAYOUT

SECOND FLOOR LAYOUT OF EAST OF BUILDING

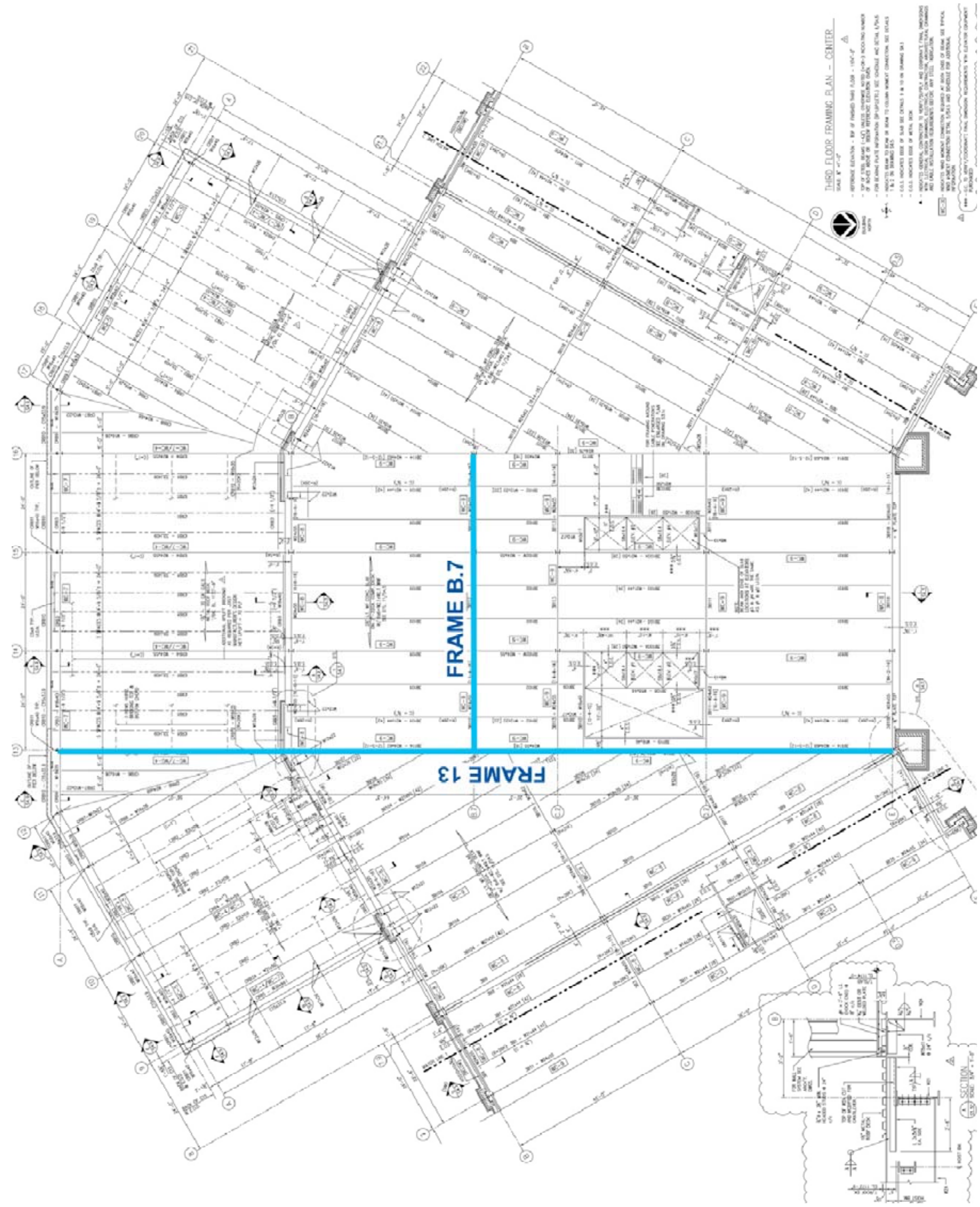
THIRD FLOOR LAYOUT EAST WITH PORTAL FRAMES INDICATED IN LIGHT BLUE AND DARK BLUE ON THE FRAME LINES AND THE SPOT CHECKED MEMBERS HIGHLIGHTED.



SECOND FLOOR LAYOUT OF CENTER OF BUILDING



THIRD FLOOR LAYOUT CENTER WITH PORTAL ANALYSIS FRAMES HIGHLIGHTED



Appendix B: Torsion Effects Calculations

Stiffness Calculations For Each Frame Using Deflection From 1 Kip Load in STAAD for Wind and Seismic.

Building	same frame	Column Line	Node	Top of Floor	Δ	K=1/Δ K
E		B	12	5	0.022	45.5
E	C	D	12	5	0.019	52.6
E	C	D	2	1	0.002	500.0
E	0.7, 0.8	0.9	8	5	0.023	43.5
E	1, 3, 7.9	2	30	5	0.072	13.9
E		4	37	5	0.041	24.4
E		4	35	4	0.037	27.0
E		5	37	5	0.048	20.8
E		5	35	4	0.034	29.4
E		6	37	5	0.048	20.8
E		6	35	4	0.034	29.4
E		7	37	5	0.048	20.8
E		E	54	5	0.06	16.7
E		E	53	4	0.05	20.0
E		D.5	41	4	0.028	35.7
E		D.8	2	1	0.034	29.4
E		F	41	4	0.022	45.5
E		7	35	4	0.034	29.4
C	14, 15, 16	13	18	5	0.035	28.6
C	14, 15, 16	13	9	2	0.009	111.1
C	14, 15, 16	13	46	1	0.003	333.3
C	B, B.7, B, A.4, D.2, E	C.2	66	5	0.05	20.0
C	20	9	18	5	0.218	4.6
C	20	9	9	2	0.034	29.4
C	19	10	9	2	0.06	16.7
C	19	10	26	1	0.015	66.7
C	18	11	9	2	0.063	15.9
C	18	11	26	1	0.016	62.5
C	21	8	30	5	0.096	10.4
C		A	75	2	0.017	58.8

The stiffness factor, K, was calculated by taking the deflection on the frame from STAAD for each frame with the exact members used by the design professional. K=1/Δ was used to determine the stiffness.

TORSION CALCULATIONS CONTINUED:

5TH FLOOR EAST BUILDING (SAME AS WEST)

Frame	Load (K)	K in the X Direction	K in the Y Direction	Distance from X Origin (ft)	Distance from Y Origin (ft)	Distribution Factor	MID OF FRAME X	MID OF FRAME Y
B	1	45.4545	0.0000	0	126	0.272		
C	1	52.6316	0.0000	0	81	0.314		
D	1	52.6316	0.0000	0	45	0.314	83.25	45
D.5	1	0.0000	0.0000	0	22.5	0.000		
D.8	1	0.0000	0.0000	0	7	0.000		
E	1	16.6667	0.0000	0	0	0.100		
F	1	0.0000	0.0000	105	-14	0.000		
0.7	1	0.0000	43.4783	0	0	0.159		
0.8	1	0.0000	43.4783	12.5	0	0.159		
0.9	1	0.0000	43.4783	36.5	0	0.159		
1	1	0.0000	13.8889	57	0	0.051		
2	1	0.0000	13.8889	81	0	0.051	63	81
3	1	0.0000	13.8889	105	0	0.051		
4	1	0.0000	24.3902	129	0	0.089		
5	1	0.0000	20.8333	153	0	0.076		
6	1	0.0000	20.8333	177	0	0.076		
7	1	0.0000	20.8333	201	0	0.076		
7.9	1	0.0000	13.8889	223.5	0	0.051		

Center of Rigidity in Y Direction (ft) $K_{ix}*d_{ix}/\Sigma K_{ix}$	Center of Rigidity in X Direction (ft) $K_{iy}*d_{iy}/\Sigma k_{iy}$	Ix $\Sigma k_{ix}*y_i^2$	Iy $\Sigma k_{iy}*x_i^2$
73.84	83.62	1173531	3435814

TORSION CALCULATIONS CONTINUED:

4TH AND 3RD FLOORS EAST BUILDING (SAME AS WEST)

Frame	Load (K)	K in the X Direction	K in the Y Direction	Distance from X 0rigin (ft)	Distance from Y 0rigin (ft)	Distribution Factor	Wall in X Direction (ft)	Wall in Y Direction (ft)
B	1	45.4545	0.0000	0	126	0.183		
C	1	52.6316	0.0000	0	81	0.212		
D	1	52.6316	0.0000	0	45	0.212	83.25	45
D.5	1	35.7143	0.0000	0	22.5	0.144		
D.8	1	0.0000	0.0000	0	7	0.000		
E	1	16.6667	0.0000	0	0	0.067		
F	1	45.4545	0.0000	105	-14	0.183		
0.7	1	0.0000	43.4783	0	0	0.144		
0.8	1	0.0000	43.4783	12.5	0	0.144		
0.9	1	0.0000	43.4783	36.5	0	0.144		
1	1	0.0000	13.8889	57	0	0.046		
2	1	0.0000	13.8889	81	0	0.046	63	81
3	1	0.0000	13.8889	105	0	0.046		
4	1	0.0000	27.0270	129	0	0.090		
5	1	0.0000	29.4118	153	0	0.098		
6	1	0.0000	29.4118	177	0	0.098		
7	1	0.0000	29.4118	201	0	0.098		
7.9	1	0.0000	13.8889	223.5	0	0.046		

Center of Rigidity in Y Direction (ft) $K_{ix} * d_{ix} / \Sigma K_{ix}$	Center of Rigidity in X Direction (ft) $K_{iy} * d_{iy} / \Sigma K_{iy}$	Ix $\Sigma k_{ix} * y_i^2$	Iy $\Sigma k_{iy} * x_i^2$
50.40	45.16	1191611	4295836

TORSION CALCULATIONS CONTINUED:

2ND FLOOR EAST BUILDING (SAME AS WEST)

Frame	Load (K)	K in the X Direction	K in the Y Direction	Distance from X Origin (ft)	Distance from Y Origin (ft)	Distribution Factor	Wall in X Direction (ft)	Wall in Y Direction (ft)
B	1	45.4545	0.0000	0	126	0.183		
C	1	52.6316	0.0000	0	81	0.212		
D	1	52.6316	0.0000	0	45	0.212	83.25	45
D.5	1	35.7143	0.0000	0	22.5	0.144		
D.8	1	0.0000	0.0000	0	7	0.000		
E	1	16.6667	0.0000	0	0	0.067		
F	1	45.4545	0.0000	105	-14	0.183		
0.7	1	0.0000	43.4783	0	0	0.144		
0.8	1	0.0000	43.4783	12.5	0	0.144		
0.9	1	0.0000	43.4783	36.5	0	0.144		
1	1	0.0000	13.8889	57	0	0.046		
2	1	0.0000	13.8889	81	0	0.046	63	81
3	1	0.0000	13.8889	105	0	0.046		
4	1	0.0000	27.0270	129	0	0.090		
5	1	0.0000	29.4118	153	0	0.098		
6	1	0.0000	29.4118	177	0	0.098		
7	1	0.0000	29.4118	201	0	0.098		
7.9	1	0.0000	13.8889	223.5	0	0.046		

Center of Rigidity in Y Direction (ft) Kix*dix/ΣKix	Center of Rigidity in X Direction (ft) Kiy*diy/Σkiy	Ix $\Sigma kix*yi^2$	Iy $\Sigma kiy*xi^2$
50.40	45.16	1191611	803849

TORSION CALCULATIONS CONTINUED:

1ST FLOOR EAST BUILDING (SAME AS WEST)

Frame	Load (K)	K in the X Direction	K in the Y Direction	Distance from X 0rigin (ft)	Distance from Y 0rigin (ft)	Distribution Factor	Center of Wall in X Direction (ft)	Center of Wall in Y Direction (ft)
B	1	45.4545	0.0000	0	126	0.062		
C	1	52.6316	0.0000	0	81	0.072		
D	1	500.0000	0.0000	0	45	0.686	111.75	45
D.5	1	35.7143	0.0000	0	22.5	0.049		
D.8	1	29.4118	0.0000	0	7	0.040		
E	1	20.0000	0.0000	0	0	0.027		
F	1	45.4545	0.0000	105	-14	0.062		
0.7	1	0.0000	43.4783	0	0	0.144		
0.8	1	0.0000	43.4783	12.5	0	0.144		
0.9	1	0.0000	43.4783	36.5	0	0.144		
1	1	0.0000	13.8889	57	0	0.046		
2	1	0.0000	13.8889	81	0	0.046	63	81
3	1	0.0000	13.8889	105	0	0.046		
4	1	0.0000	27.0270	129	0	0.090		
5	1	0.0000	29.4118	153	0	0.098		
6	1	0.0000	29.4118	177	0	0.098		
7	1	0.0000	29.4118	201	0	0.098		
7.9	1	0.0000	13.8889	223.5	0	0.046		

Center of Rigidity in Y Direction (ft) $K_{ix} * d_{ix} / \Sigma K_{ix}$	Center of Rigidity in X Direction (ft) $K_{iy} * d_{iy} / \Sigma k_{iy}$	Ix $\Sigma k_{ix} * y_i^2$	Iy $\Sigma k_{iy} * x_i^2$
45.97	45.16	2098973.69	4295836.28

APPENDIX C: WIND LOAD CALCULATIONS

MAIN WIND-FORCE RESISTING SYSTEM (ASCE 7-05)

Basic Wind Speed (V) mph	90
Exposure Category	B
Importance Factor (I)	1
Wind Directionality Factor (Kd)	0.85
Topographic Factor (Kzt)	1

WHOLE BUILDING L/B AND VALUES

		L/B	C_p
East-West Direction			
	Windward	4.317	0.8
	Leeward	4.317	-0.2
	Sidewall	4.317	-0.7
North-South Direction			
	Windward	0.232	0.8
	Leeward	0.232	-0.5
	Sidewall	0.232	-0.7

EAST PORTION OF BUILDING FOR RAM COMPARISON

		L/B	C_p
East-West Direction			
	Windward	1.321	0.8
	Leeward	1.321	-0.4358
	Sidewall	1.321	-0.7
North-South Direction			
	Windward	0.757	0.8
	Leeward	0.757	-0.5
	Sidewall	0.757	-0.7

	Wind Direction	
Variable	N-S	E-W
Stiffness	Flex	Flex
B	544	126
L	126	544
h	74.5	74.5
z	30'	30'
ℓ	320	320
ϵ	0.333	0.333
α	0.25	0.25
β	0.05	0.05
V	90	90
V_z	64.082	64.082
L_z	354.06	354.06
n_1	0.706	0.706
N_1	3.90	3.90
R_n	0.059	0.059
R_h	0.230	0.230
R_b	0.036	0.144
R_L	0.046	0.011
b	0.45	0.45
R	0.073	0.145
I_z	0.285	0.285
g_R	4.106	4.106
q_p	13.924	13.924
g_V	3.4	3.4
Q	0.726	0.833
G_f	0.771	0.839

WIND CALCULATIONS CONTINUED:

ENTIRE BUILDING CALCULATIONS:

$q_p = 0.00256\ K_h K_{zt} K_d V^2 I =$ 13.924

GCpn= 1.5 -1

$Pp = q_p GCpn =$ 20.885 -13.924

$n_1 = \dfrac{22.2}{H^{0.8}} =$ 0.706

$g_Q =$ $g_V =$ 3.4

$g_R = \sqrt{(2\ln(3600n_1))} + 0.577/(\sqrt{(2\ln(3600n1))} =$ 4.106

$z = 0.6h =$ 44.7

$z_{min} =$ 30'

$I_z = c(33/z)^{1/6} =$ 0.285

$L_z = l(z/33)^{\epsilon} =$ 354.06

$Q_{N\text{-}S} = \sqrt{(1/(1+0.63(B+h/L_z)^{0.63}))} =$ 0.726

$Q_{E\text{-}W} = \sqrt{(1/(1+0.63(B+h/L_z)^{0.63}))} =$ 0.833

$V_z = b(z/33)^{\alpha}V(88/60) =$ 64.082

$N_1 = n_1 L_z/V_z =$ 3.90

$R_n = 7.47N_1/(1+10.3N_1)^{5/3} =$ 0.059

$R_h = 1/h - 1/(2h^2)(1-e^{-2h}) =$ 0.230

$h = 4.6n_1h/V_z =$ 3.774

WIND CALCULATIONS CONTINUED:

ENTIRE BUILDING:

As can be seen in the table at the bottom of the page, RAM used a different method to find q_z than was specified as a minimum in ASCE 7-05.

N-S

$R_b = 1/h - 1/(2h^2)(1-e^{-2h}) =$ 0.036

$h = 4.6n_1B/V_z =$ 27.558

$R_L = 1/h - 1/(2h^2)(1-e^{-2h}) =$ 0.046

$h = 15.4n_1L/V_z$ 21.36901

$R = \sqrt{((1/b)(R_nR_hR_b(0.53+0.47R_L)} =$ 0.073

$G_f = 0.925\,[\,(1+1.7I_z\sqrt{(g_Q^2Q^2+g_R^2R^2)})/(1+1.7g_vI_z)] =$ 0.771

E-W

$R_b = 1/h - 1/(2h^2)(1-e^{-2h}) =$ 0.144

$h = 4.6n_1B/V_z =$ 6.383

$R_L = 1/h - 1/(2h^2)(1-e^{-2h}) =$ 0.011

$h = 15.4n_1L/V_z$ 92.25985

$R = \sqrt{((1/b)(R_nR_hR_b(0.53+0.47R_L)} =$ 0.145

$G_f = 0.925[(1+1.7I_z\sqrt{(g_Q^2Q^2+g_R^2R^2)})/(1+1.7g_vI_z)] =$ 0.839

$g_z = 0.00256K_zK_{zt}K_dV^2I =$ 15.33427 K_z

From Table 6-3

H (ft)	K_z	q_z
74.5	0.908	13.9235
60	0.85	13.0341
46	0.79	12.1141
32	0.712	10.918
18	0.59	9.20056
0	0.57	8.74054

From RAM

H (ft)	K_z	q_z
74.5	0.909	16.013
60	0.854	15.053
46	0.792	13.952
32	0.714	12.578
18	0.605	10.671
0	0.575	10.130

Wind Calculations Continued:

Entire Building

Design Wind Pressures p in E-W Direction (Table 5.41)						
Location	Height above Ground Level z (ft)	q(psf)	External Pressure qGC_p (psf)	Internal Pressure q_hGC_{pi}	Net Pressure p (psf)	
					+Gcpi	-Gcpi
Windward	74.5	13.92	9.35	2.51	6.84	11.86
	70	13.65	9.17	2.51	6.66	11.67
	60	13.03	8.75	2.51	6.25	11.26
	50	12.42	8.34	2.51	5.84	10.85
	46	12.11	8.14	2.51	5.63	10.64
	40	11.65	7.83	2.51	5.32	10.33
	32	10.92	7.33	2.51	4.83	9.84
	30	10.73	7.21	2.51	4.70	9.72
	25	10.12	6.80	2.51	4.29	9.30
	20	9.51	6.39	2.51	3.88	8.89
	18	9.20	6.18	2.51	3.67	8.69
	15	8.74	5.87	2.51	3.36	8.38
Leeward	All	13.92	-2.34	2.51	-4.84	0.17
Side	All	13.92	-8.18	2.51	-10.69	-5.68

Design Wind Pressures p in N-S Direction (Table 5.41)						
Location	Height above Ground Level z (ft)	q(psf)	External Pressure qGC_p (psf)	Internal Pressure q_hGC_{pi}	Net Pressure p (psf)	
					+Gcpi	-Gcpi
Windward	74.5	13.92	8.58	2.51	6.08	11.09
	70	13.65	8.41	2.51	5.91	10.92
	60	13.03	8.03	2.51	5.53	10.54
	50	12.42	7.66	2.51	5.15	10.16
	46	12.11	7.47	2.51	4.96	9.97
	40	11.65	7.18	2.51	4.68	9.69
	32	10.92	6.73	2.51	4.22	9.24
	30	10.12	6.24	2.51	3.73	8.74
	25	9.51	5.86	2.51	3.35	8.37
	20	8.74	5.39	2.51	2.88	7.89
	18	8.74	5.39	2.51	2.88	7.89
	15	8.74	5.39	2.51	2.88	7.89
Leeward	All	14.61	-5.63	2.51	-8.13	-3.12
Side	All	14.61	-7.88	2.51	-10.38	-5.37

WIND CALCULATIONS CONTINUED:

ENTIRE BUILDING

Floor Heights	Level	Total Height	K_Z	q_Z	Wind Pressures (psf)					
					N-S Windward	N-S Leeward	N-S Side Wall	E-W Windward	E-W Leeward	E-W Sidewall
14.5	Roof	74.5	0.908	13.924	11.09	-8.13	-10.38	11.86	-4.84	-10.69
14	5	60	0.85	13.034	10.54	-8.13	-10.38	11.67	-4.84	-10.69
14	4	46	0.79	12.114	9.97	-8.13	-10.38	11.26	-4.84	-10.69
14	3	32	0.712	10.918	9.24	-8.13	-10.38	10.85	-4.84	-10.69
18	2	18	0.59	9.201	7.89	-8.13	-10.38	10.64	-4.84	-10.69

Level	Wind Design					
	Load (kips)		Shear (kips)		Moment (ft-k)	
	N-S	E-W	N-S	E-W	N-S	E-W
Roof	151.6	30.5	0	0	2198.6	442.4
5	144.8	29.7	151.6	30.5	2026.7	415.2
4	137.9	28.4	296.4	60.2	1930.7	397.7
3	132.3	27.7	434.3	88.6	1852.1	387.5
2	139.5	31.2	566.6	116.3	2511.1	562.0
Total	706.1	147.5	706.1	147.5	10519.2	2204.8

Note: Total Base Shear includes load from Windward and Leeward pressures

WIND CALCULATIONS CONTINUED:

FRAME 2 WIND LOADS- TORSION INCLUDED

FIFTH FLOOR LOADS

Story	Force X (K)	Force Y (K)	Direct Force on Frame 2 (K) Fix=(Kix/ΣKix)F	Direct Force on Frame 2 (K) Fiy=(Kiy/ΣKiy)F	Torsional Force on Frame 2 (K-ft) Fix=((Ki*xi)/Ip)M	Torsional Force on Frame 2 (K-ft) Fiy=((Ki*yi)/Ip)M	Total Force on Each Story (K) $F=DF+TF_X+TF_Y$	Center of Mass in X Direction (ft)	Center of Mass in Y Direction (ft)	Torsional Moment (K-ft) X	Torsional Moment (K-ft) Y
5	30.5	151.6	0.00	7.72	0.00	0.07	12.45	141.14	64.32	3126	1086
4	29.7	144.8	0.00	7.37	0.00	0.06	11.90	139.68	60.38	2986	1037
3	28.4	137.9	0.00	7.02	0.00	0.06	11.33	139.87	60.26	2844	988
2	27.7	132.3	0.00	6.73	0.00	0.06	10.87	139.87	60.31	2728	948
1	31.2	139.5	0.00	7.10	0.00	0.06	11.46	136.33	59.94	2877	999

FOURTH AND THIRD FLOOR LOADS

Story	Force X (K)	Force Y (K)	Direct Force on Frame 2 (K) Fix=(Kix/ΣKix)F	Direct Force on Frame 2 (K) Fiy=(Kiy/ΣKiy)F	Torsional Force on Frame 2 (K-ft) Fix=((Ki*xi)/Ip)M	Torsional Force on Frame 2 (K-ft) Fiy=((Ki*yi)/Ip)M	Total Force on Each Story (K) $F=DF+TF_X+TF_Y$	Center of Mass in X Direction (ft)	Center of Mass in Y Direction (ft)	Torsional Moment (K-ft) X	Torsional Moment (K-ft) Y
5	30.5	151.6	0.00	6.99	0.00	0.21	11.52	141.14	64.32	2705	4640
4	29.7	144.8	0.00	6.68	0.00	0.20	11.00	139.68	60.38	2584	4431
3	28.4	137.9	0.00	6.36	0.00	0.19	10.48	139.87	60.26	2461	4220
2	27.7	132.3	0.00	6.10	0.00	0.18	10.05	139.87	60.31	2361	4049
1	31.2	139.5	0.00	6.43	0.00	0.19	10.60	136.33	59.94	2489	4269

SECOND FLOOR LOADS

Story	Force X (K)	Force Y (K)	Direct Force on Frame 2 (K) Fix=(Kix/ΣKix)F	Direct Force on Frame 2 (K) Fiy=(Kiy/ΣKiy)F	Torsional Force on Frame 2 (K-ft) Fix=((Ki*xi)/Ip)M	Torsional Force on Frame 2 (K-ft) Fiy=((Ki*yi)/Ip)M	Total Force on Each Story (K) $F=DF+TF_X+TF_Y$	Center of Mass in X Direction (ft)	Center of Mass in Y Direction (ft)	Torsional Moment (K-ft) X	Torsional Moment (K-ft) Y
5	30.5	151.6	0.00	6.99	0.00	0.58	12.10	141.14	64.32	2705	4640
4	29.7	144.8	0.00	6.68	0.00	0.55	11.56	139.68	60.38	2584	4431
3	28.4	137.9	0.00	6.36	0.00	0.52	11.01	139.87	60.26	2461	4220
2	27.7	132.3	0.00	6.10	0.00	0.50	10.56	139.87	60.31	2361	4049
1	31.2	139.5	0.00	6.43	0.00	0.53	11.14	136.33	59.94	2489	4269

FIRST FLOOR LOADS

Story	Force X (K)	Force Y (K)	Direct Force on Frame 2 (K) Fix=(Kix/ΣKix)F	Direct Force on Frame 2 (K) Fiy=(Kiy/ΣKiy)F	Torsional Force on Frame 2 (K-ft) Fix=((Ki*xi)/Ip)M	Torsional Force on Frame 2 (K-ft) Fiy=((Ki*yi)/Ip)M	Total Force on Each Story (K) $F=DF+TF_X+TF_Y$	Center of Mass in X Direction (ft)	Center of Mass in Y Direction (ft)	Torsional Moment (K-ft) X	Torsional Moment (K-ft) Y
5	30.5	151.6	0.00	6.99	0.00	0.21	11.51	141.14	64.32	2705	5310
4	29.7	144.8	0.00	6.68	0.00	0.20	11.00	139.68	60.38	2584	5072
3	28.4	137.9	0.00	6.36	0.00	0.19	10.47	139.87	60.26	2461	4830
2	27.7	132.3	0.00	6.10	0.00	0.18	10.05	139.87	60.31	2361	4634
1	31.2	139.5	0.00	6.43	0.00	0.19	10.59	136.33	59.94	2489	4886

Wind Calculations Continued:

East Building For RAM Comparison

(from Figure 6-6)

	L/B	C_p
East-West Direction		
Windward	1.321	0.8
Leeward	1.321	-0.4358
Sidewall	1.321	-0.7
North-South Direction		
Windward	0.757	0.8
Leeward	0.757	-0.5
Sidewall	0.757	-0.7

	Wind Direction	
Variable	N-S	E-W
Stiffness	Flex	Flex
B	166.5	126
L	126	166.5
h	74.5	74.5
z	30'	30'
ℓ	320	320
ϵ	0.333	0.333
α	0.25	0.25
β	0.05	0.05
V	90	90
V_z	64.082	64.082
L_z	354.06	354.06
n_1	0.706	0.706
N_1	3.90	3.90
R_n	0.059	0.059
R_h	0.230	0.230
R_b	0.112	0.144
R_L	0.046	0.035
b	0.45	0.45
R	0.130	0.147
I_z	0.285	0.285
g_R	4.106	4.106
q_p	13.924	13.924
g_V	3.4	3.4
Q	0.818	0.833
G_f	0.829	0.840

From Table 6-3

H (ft)	K_z	q_z
74.5	0.908	13.9235
60	0.85	13.0341
46	0.79	12.1141
32	0.712	10.918
18	0.59	9.20056
0	0.57	8.74054

From RAM

H (ft)	K_z	q_z
74.5	0.909	16.013
60	0.854	15.053
46	0.792	13.952
32	0.714	12.578
18	0.605	10.671
0	0.575	10.130

WIND CALCULATIONS CONTINUED:

EAST BUILDING FOR RAM COMPARISON

LOAD CASE: Wind RAM

Wind ASCE 7-05/IBC2006
Exposure: B
Basic Wind Speed (mph): 90.0 Importance Factor: 1.000
Apply Directionality Factor, Kd = 0.85
Use Topography Factor, Kzt: 1.00
Use Calculated Frequency for X-Dir.
Use Calculated Frequency for Y-Dir.
Gust Factor for Flexible Structures, G: Use Calculated G for X-Dir.
Gust Factor for Flexible Structures, G: Use Calculated G for Y-Dir.
Damping Ratio for Flexible Structures= 0.01
Mean Roof Height (ft): Top Story Height = 74.50
Ground Level: Base

WIND PRESSURES:

X-Direction: Natural Frequency = 0.583 Structure is Flexible
Y-Direction: Natural Frequency = 0.742 Structure is Flexible
CpWindward = 0.80 qLeeward (qh) = 16.01 psf
GCpn (Parapet): Windward = 1.50 Leeward = -1.00

Height	Kz	Kzt	qz	Gust Factor G		CpLeeward		Pressure (psf)	
ft			psf	X	Y	X	Y	X	Y
74.50	0.909	1.000	16.013	0.906	0.856	-0.436	-0.500	17.929	17.833
60.00	0.854	1.000	15.053	0.896	0.856	-0.463	-0.500	17.430	17.168
46.00	0.792	1.000	13.952	0.897	0.856	-0.462	-0.500	16.635	16.414
32.00	0.714	1.000	12.578	0.897	0.856	-0.462	-0.500	15.650	15.473
18.00	0.605	1.000	10.671	0.896	0.837	-0.381	-0.500	13.111	13.854
0.00	0.575	1.000	10.130	0.896	0.837	-0.381	-0.500	12.723	13.491

As these tables show, RAM values are around the same change from floor to floor as the hand calculations.

Design Wind Pressures p in E-W Direction (Table 5.41)						
Location	Height above Ground Level z (ft)	q(psf)	External Pressure qGC_p (psf)	Internal Pressure q_hGC_{pi}	Net Pressure p (psf)	
					+Gcpi	-Gcpi
Windward	74.5	13.92	9.35	2.51	6.85	11.86
	60	13.03	8.76	2.51	6.25	11.26
	46	12.11	8.14	2.51	5.63	10.64
	32	10.92	7.33	2.51	4.83	9.84
	18	9.20	6.18	2.51	3.67	8.69
	15	8.74	5.87	2.51	3.37	8.38

Design Wind Pressures p in N-S Direction (Table 5.41)						
Location	Height above Ground Level z (ft)	q(psf)	External Pressure qGC_p (psf)	Internal Pressure q_hGC_{pi}	Net Pressure p (psf)	
					+Gcpi	-Gcpi
Windward	74.5	13.92	9.23	2.51	6.73	11.74
	60	13.03	8.64	2.51	6.14	11.15
	46	12.11	8.03	2.51	5.53	10.54
	32	10.92	7.24	2.51	4.73	9.74
	18	8.74	5.80	2.51	3.29	8.30
	15	8.74	5.80	2.51	3.29	8.30

Design Wind Pressures p in RAM (Table 5.41)						
Location	Height above Ground Level z (ft)	q(psf)	External Pressure qGC_p (psf)	Internal Pressure q_hGC_{pi}	Net Pressure p (psf)	
					+Gcpi	-Gcpi
Windward	74.5	16.01	10.62	2.51	8.11	17.93
	60	15.05	9.98	2.51	7.47	17.43
	46	13.95	9.25	2.51	6.74	16.64
	32	12.58	8.34	2.51	5.83	15.65
	18	10.67	7.07	2.51	4.57	9.58
	0		0.00	2.51	-2.51	2.51

Wind Calculations Continued:
East Building For RAM Comparison

RAM values were compared further to the hand calculation method results used throughout the report and used in technical report one. The main differences have already been addressed, such as the possibility of a finite element analysis. The Wind Design table below is the final story force and story shears for the east portion of the building.

Calculated Values

Floor Heights	Level	Total Height	K_Z	q_Z	Wind Pressures (psf)					
					N-S Windward	N-S Leeward	N-S Side Wall	E-W Windward	E-W Leeward	E-W Sidewall
14.5	Roof	74.5	0.908	13.924	11.74	-8.56	-10.98	11.86	-7.60	-10.69
14	5	60	0.85	13.034	11.15	-8.56	-10.98	11.26	-7.60	-10.69
14	4	46	0.79	12.114	10.54	-8.56	-10.98	10.64	-7.60	-10.69
14	3	32	0.712	10.918	9.74	-8.56	-10.98	9.84	-7.60	-10.69
18	2	18	0.59	9.201	8.30	-8.56	-10.98	8.69	-7.60	-10.69

RAM Calculated Values

Floor Heights	Level	Total Height	K_Z	q_Z	Wind Pressures (psf)					
					N-S Windward	N-S Leeward	N-S Side Wall	E-W Windward	E-W Leeward	E-W Sidewall
14.5	Roof	74.5	0.909	16.013	17.93	-2.51	-2.51	17.93	-2.51	-2.51
14	5	60	0.854	15.053	17.43	-2.51	-2.51	17.43	-2.51	-2.51
14	4	46	0.792	13.952	16.64	-2.51	-2.51	16.64	-2.51	-2.51
14	3	32	0.714	12.578	15.65	-2.51	-2.51	15.65	-2.51	-2.51
18	2	18	0.605	10.671	2.51	-2.51	-2.51	9.58	-2.51	-2.51

Level	Wind Design					
	Load (kips)		Shear (kips)		Moment (ft-k)	
	N-S	E-W	N-S	E-W	N-S	E-W
Roof	49.0	35.6	0	0	710.5	515.6
5	46.8	33.9	49.0	35.6	654.6	474.2
4	44.5	32.2	95.8	69.4	623.2	450.6
3	42.7	30.8	140.3	101.6	597.3	430.8
2	44.9	32.8	182.9	132.4	808.5	591.1
Total	227.9	165.2	227.9	165.2	3394.1	2462.2

Note: Total Base Shear includes load from Windward and Leeward pressures

Appendix D: Seismic Load Calculations

Seismic Design Values, ASCE 7-05			
Response Modification Coefficient	R= 3	R= 3.5	Table 12.2-1
Coefficient	C_U= 1.7	C_U= 1.7	Table 12.8-1
Fundamental Period	T= 1.497	T= 1.497	Sec. 12.8.2
Seismic Response Coefficient	C_S= 0.016	C_S= 0.014	Eq. 12.8-3
Building Height (above grade)	h= 74.5	h= 74.5	

LOAD CASE: seismic RAM 3

Seismic ASCE 7-05 / IBC 2006 Equivalent Lateral Force
Site Class: D Importance Factor: 1.00 Ss: 0.050 g S1: 0.050 g TL: 12.00 s
Fa: 1.600 Fv: 2.400 SDs: 0.053 g SD1: 0.080 g
Occupancy Category: II Seismic Design Category: B
Provisions for: Force
Ground Level: Base

Dir	Eccent	R	Ta Equation	Building Period-T
X	+ And -	3.0	Std,Ct=0.028,x=0.80	Calculated
Y	+ And -	3.0	Std,Ct=0.028,x=0.80	Calculated

Dir	Ta	Cu	T	T-used	Eq12.8-2	Eq12.8-3	Eq12.8-5	k
X	0.881	1.700	1.715	1.497	0.018	0.018	0.0100	1.499
Y	0.881	1.700	1.348	1.348	0.018	0.020	0.0100	1.424

F_a Values (Table 11.4-1 ASCE 7-05)					
	S_S≤0.25	S_S=0.5	S_S=0.75	S_S=1.0	S_S≥1.25
D	1.6	1.4	1.2	1.2	1

F_V Values (Table 11.4-2 ASCE 7-05)					
	S_1≤0.1	S_1=0.3	S_1=0.3	S_1=0.4	S_1≥0.5
D	2.4	2	1.8	1.6	1.5

SEISMIC CALCULATIONS CONTINUED:

The values in this portion of the report were calculated using the method described in the Seismic design portion of this report and the same method used in Technical Report One.

R=3

Floor	w_x (k)	h_x (ft)	h_x^k (ft)	$w_xh_x^k$	C_{vx}	Story Force F_x (k)	Story Shear V_x (k)	Moment at Floor (ft-k)
Roof	4240.5	74.5	639.41	2711449	0.359	167.42	0	12473.065
5	4713.6	60	462.27	2178985	0.288	134.55	167.42	8072.7394
4	4726.5	46	310.43	1467216	0.194	90.60	301.97	4167.4204
3	4724.0	32	180.20	851252	0.113	52.56	392.57	1681.9916
2	4653.4	18	76.08	354028	0.047	21.86	445.13	393.48265
1	5444.4						466.99	
Sum	28502.4	74.5	1668.39	7562930	1.000	466.99	466.99	26788.699

R=3.5

Floor	w_x (k)	h_x (ft)	h_x^k (ft)	$w_xh_x^k$	C_{vx}	Story Force F_x (k)	Story Shear V_x (k)	Moment at Floor (ft-k)
Roof	4240.5	74.5	639.41	2711449	0.359	143.51	0	10691.199
5	4713.6	60	462.27	2178985	0.288	115.32	143.51	6919.4909
4	4726.5	46	310.43	1467216	0.194	77.65	258.83	3572.0746
3	4724.0	32	180.20	851252	0.113	45.05	336.48	1441.7071
2	4653.4	18	76.08	354028	0.047	18.74	381.54	337.27085
1	5444.4						400.28	
Sum	28502.4	74.5	1668.39	7562930	1.000	400.28	400.28	22961.742

R=3.5 CONSULTANT

Floor	w_x (k)	h_x (ft)	h_x^k (ft)	$w_xh_x^k$	C_{vx}	Story Force F_x (k)	Story Shear V_x (k)	Moment at Floor (ft-k)
Roof	4240.5	74.5	639.41	2711449	0.359	150.53	0	11214.138
5	4713.6	60	462.27	2178985	0.288	120.97	150.53	7257.9442
4	4726.5	46	310.43	1467216	0.194	81.45	271.49	3746.7956
3	4724.0	32	180.20	851252	0.113	47.26	352.94	1512.2254
2	4653.4	18	76.08	354028	0.047	19.65	400.20	353.76779
1	5444.4						419.85	
Sum	28502.4	74.5	74.5	7562930	1.000	419.85	419.85	24084.871

Seismic Calculations Continued:

These values are a continuation of the method described in the Torsion section of this report. Additional Calculations are available upon request.

Frame 2 Seismic Loads- Torsion Included

Fifth Floor Loads

Story	Force (K)	Direct Force on Frame 2 (K) Fiy=(Kiy/ΣKiy)F	Torsional Force on Frame 2 (K-ft) Fix=((Ki*xi)/Ip)M	Torsional Force on Frame 2 (K-ft) Fiy=((Ki*yi)/Ip)M	Total Force on Each Story (K) F=DF+TF
5	167.42	8.52	0.000	0.208	8.73
4	134.55	6.85	0.000	0.163	7.01
3	90.6	4.61	0.000	0.110	4.72
2	52.56	2.68	0.000	0.064	2.74
1	21.86	1.11	0.000	0.025	1.14

Center of Mass in X Direction (ft)	Center of Mass in Y Direction (ft)	Torsional Moment (K-ft) X	Torsional Moment (K-ft) Y
141.14	64.32	9630	1593
139.68	60.38	7543	1810
139.87	60.26	5096	1230
139.87	60.31	2956	711
136.33	59.94	1152	304

Fourth And Third Floor Loads

Story	Force (K)	Direct Force on Frame 2 (K) Fiy=(Kiy/ΣKiy)F	Torsional Force on Frame 2 (K-ft) Fix=((Ki*xi)/Ip)M	Torsional Force on Frame 2 (K-ft) Fiy=((Ki*yi)/Ip)M	Total Force on Each Story (K) F=DF+TF
5	167.42	7.72	0.000	3.423	11.14
4	134.55	6.20	0.000	2.709	8.91
3	90.6	4.18	0.000	1.828	6.00
2	52.56	2.42	0.000	1.060	3.48
1	21.86	1.01	0.000	0.425	1.43

Center of Mass in X Direction (ft)	Center of Mass in Y Direction (ft)	Torsional Moment (K-ft) X	Torsional Moment (K-ft) Y
141.14	64.32	16070	2331
139.68	60.38	12718	1343
139.87	60.26	8581	894
139.87	60.31	4978	521
136.33	59.94	1993	209

Second Floor Loads

Story	Force (K)	Direct Force on Frame 2 (K) Fiy=(Kiy/ΣKiy)F	Torsional Force on Frame 2 (K-ft) Fix=((Ki*xi)/Ip)M	Torsional Force on Frame 2 (K-ft) Fiy=((Ki*yi)/Ip)M	Total Force on Each Story (K) F=DF+TF
5	167.42	7.72	0.000	3.423	11.14
4	134.55	6.20	0.000	2.709	8.91
3	90.6	4.18	0.000	1.828	6.00
2	52.56	2.42	0.000	1.060	3.48
1	21.86	1.01	0.000	0.425	1.43

Center of Mass in X Direction (ft)	Center of Mass in Y Direction (ft)	Torsional Moment (K-ft) X	Torsional Moment (K-ft) Y
141.14	64.32	16070	2331
139.68	60.38	12718	1343
139.87	60.26	8581	894
139.87	60.31	4978	521
136.33	59.94	1993	209

First Floor Loads

Story	Force (K)	Direct Force on Frame 2 (K) Fiy=(Kiy/ΣKiy)F	Torsional Force on Frame 2 (K-ft) Fix=((Ki*xi)/Ip)M	Torsional Force on Frame 2 (K-ft) Fiy=((Ki*yi)/Ip)M	Total Force on Each Story (K) F=DF+TF
5	167.42	7.72	0.000	1.222	8.94
4	134.55	6.20	0.000	0.968	7.17
3	90.6	4.18	0.000	0.653	4.83
2	52.56	2.42	0.000	0.379	2.80
1	21.86	1.01	0.000	0.152	1.16

Center of Mass in X Direction (ft)	Center of Mass in Y Direction (ft)	Torsional Moment (K-ft) X	Torsional Moment (K-ft) Y
141.14	64.32	16070	3071
139.68	60.38	12718	1938
139.87	60.26	8581	1294
139.87	60.31	4978	753
136.33	59.94	1993	305

Seismic Calculations Continued:

Frame D Seismic Loads- Torsion Included

Fifth Floor Loads

Story	Force (K)	Direct Force on Frame D (K) Fix=(Kix/ΣKix)F	Torsional Force on Frame D (K-ft) Fix=((Ki*xi)/Ip)M	Torsional Force on Frame D (K-ft) Fiy=((Ki*yi)/Ip)M	Total Force on Each Story (K) $F=DF+TF_X+TF_Y$	Center of Mass in X Direction (ft)	Center of Mass in Y Direction (ft)	Torsional Moment (K-ft) X	Torsional Moment (K-ft) Y
5	167.42	52.64	0.007	0.00	52.65	141.14	64.32	9630	1593
4	134.55	42.31	0.008	0.00	42.31	139.68	60.38	7543	1810
3	90.6	28.49	0.005	0.00	28.49	139.87	60.26	5096	1230
2	52.56	16.53	0.003	0.00	16.53	139.87	60.31	2956	711
1	21.86	6.87	0.001	0.00	6.87	136.33	59.94	1152	304

Fourth And Third Floor Loads

Story	Force (K)	Direct Force on Frame D (K) Fix=(Kix/ΣKix)F	Torsional Force on Frame D (K-ft) Fix=((Ki*xi)/Ip)M	Torsional Force on Frame D (K-ft) Fiy=((Ki*yi)/Ip)M	Total Force on Each Story (K) $F=DF+TF_X+TF_Y$	Center of Mass in X Direction (ft)	Center of Mass in Y Direction (ft)	Torsional Moment (K-ft) X	Torsional Moment (K-ft) Y
5	167.42	35.45	2.342	0.00	37.79	141.14	64.32	16070	2331
4	134.55	28.49	1.350	0.00	29.84	139.68	60.38	12718	1343
3	90.6	19.18	0.898	0.00	20.08	139.87	60.26	8581	894
2	52.56	11.13	0.524	0.00	11.65	139.87	60.31	4978	521
1	21.86	4.63	0.210	0.00	4.84	136.33	59.94	1993	209

Second Floor Loads

Story	Force (K)	Direct Force on Frame D (K) Fix=(Kix/ΣKix)F	Torsional Force on Frame D (K-ft) Fix=((Ki*xi)/Ip)M	Torsional Force on Frame D (K-ft) Fiy=((Ki*yi)/Ip)M	Total Force on Each Story (K) $F=DF+TF_X+TF_Y$	Center of Mass in X Direction (ft)	Center of Mass in Y Direction (ft)	Torsional Moment (K-ft) X	Torsional Moment (K-ft) Y
5	167.42	35.45	2.342	0.00	37.79	141.14	64.32	16070	2331
4	134.55	28.49	1.350	0.00	29.84	139.68	60.38	12718	1343
3	90.6	19.18	0.898	0.00	20.08	139.87	60.26	8581	894
2	52.56	11.13	0.524	0.00	11.65	139.87	60.31	4978	521
1	21.86	4.63	0.210	0.00	4.84	136.33	59.94	1993	209

First Floor Loads

Story	Force (K)	Direct Force on Frame D (K) Fix=(Kix/ΣKix)F	Torsional Force on Frame D (K-ft) Fix=((Ki*xi)/Ip)M	Torsional Force on Frame D (K-ft) Fiy=((Ki*yi)/Ip)M	Total Force on Each Story (K) $F=DF+TF_X+TF_Y$	Center of Mass in X Direction (ft)	Center of Mass in Y Direction (ft)	Torsional Moment (K-ft) X	Torsional Moment (K-ft) Y
5	167.42	114.88	15.993	0.00	130.87	141.14	64.32	16070	3071
4	134.55	92.33	10.092	0.00	102.42	139.68	60.38	12718	1938
3	90.6	62.17	6.739	0.00	68.91	139.87	60.26	8581	1294
2	52.56	36.07	3.923	0.00	39.99	139.87	60.31	4978	753
1	21.86	15.00	1.590	0.00	16.59	136.33	59.94	1993	305

Seismic Calculations Continued:

Frame 13 Seismic Loads- Torsion Included

Fifth Floor Loads

Story	Force (K)	Direct Force on Frame13 (K) Fiy=(Kiy/ΣKiy)F	Torsional Force on Frame 13 (K-ft) Fix=((Ki*xi)/Ip)M	Torsional Force on Frame 13(K-ft) Fiy=((Ki*yi)/Ip)M	Total Force on Each Story (K) $F=DF+TF_X+TF_Y$	Center of Mass in X Direction (ft)	Center of Mass in Y Direction (ft)	Torsional Moment (K-ft) X	Torsional Moment (K-ft) Y
5	167.42	35.73	0.00	0.55	36.28	36	83.17	2220	1291
4	134.55	28.72	0.00	0.44	29.16	36	83.17	1785	1038
3	90.6	19.34	0.00	0.30	19.63	36	83.17	1202	699
2	52.56	11.22	0.00	0.17	11.39	36	99.57	697	1267
1	21.86	4.67	0.00	0.07	4.74	35.92	113.42	288	830

Fourth and Third Floor Loads

Story	Force (K)	Direct Force on Frame13 (K) Fiy=(Kiy/ΣKiy)F	Torsional Force on Frame 13 (K-ft) Fix=((Ki*xi)/Ip)M	Torsional Force on Frame 13(K-ft) Fiy=((Ki*yi)/Ip)M	Total Force on Each Story (K) $F=DF+TF_X+TF_Y$	Center of Mass in X Direction (ft)	Center of Mass in Y Direction (ft)	Torsional Moment (K-ft) X	Torsional Moment (K-ft) Y
5	167.42	33.15	0.00	0.29	33.44	36	83.17	1253	1291
4	134.55	26.64	0.00	0.23	26.87	36	83.17	1007	1038
3	90.6	17.94	0.00	0.16	18.10	36	83.17	678	699
2	52.56	10.41	0.00	0.09	10.50	36	99.57	393	1267
1	21.86	4.33	0.00	0.04	4.37	35.92	113.42	162	830

Second Floor Loads

Story	Force (K)	Direct Force on Frame13 (K) Fiy=(Kiy/ΣKiy)F	Torsional Force on Frame 13 (K-ft) Fix=((Ki*xi)/Ip)M	Torsional Force on Frame 13(K-ft) Fiy=((Ki*yi)/Ip)M	Total Force on Each Story (K) $F=DF+TF_X+TF_Y$	Center of Mass in X Direction (ft)	Center of Mass in Y Direction (ft)	Torsional Moment (K-ft) X	Torsional Moment (K-ft) Y
5	167.42	31.57	0.00	0.97	32.54	36	83.17	1481	1291
4	134.55	25.37	0.00	0.78	26.15	36	83.17	1190	1038
3	90.6	17.09	0.00	0.52	17.61	36	83.17	801	699
2	52.56	9.91	0.00	0.30	10.22	36	99.57	465	1267
1	21.86	4.12	0.00	0.12	4.25	35.92	113.42	192	830

First Floor Loads

Story	Force (K)	Direct Force on Frame13 (K) Fiy=(Kiy/ΣKiy)F	Torsional Force on Frame 13 (K-ft) Fix=((Ki*xi)/Ip)M	Torsional Force on Frame 13(K-ft) Fiy=((Ki*yi)/Ip)M	Total Force on Each Story (K) $F=DF+TF_X+TF_Y$	Center of Mass in X Direction (ft)	Center of Mass in Y Direction (ft)	Torsional Moment (K-ft) X	Torsional Moment (K-ft) Y
5	167.42	4.26	0.00	0.17	4.42	36	83.17	1125	4576
4	134.55	3.42	0.00	0.13	3.56	36	83.17	904	3678
3	90.6	2.30	0.00	0.09	2.39	36	83.17	609	2477
2	52.56	1.34	0.00	0.05	1.39	36	99.57	353	575
1	21.86	0.56	0.00	0.02	0.58	35.92	113.42	149	64

Seismic Calculations Continued:

Frame B.7 Seismic Loads- Torsion Included

Fifth Floor Loads

Story	Force (K)	Direct Force on Frame B.7 (K) Fix=(Kix/ΣKix)F	Torsional Force on Frame B.7 (K-ft) Fix=((Ki*xi)/Ip)M	Torsional Force on Frame B.7 (K-ft) Fiy=((Ki*yi)/Ip)M	Total Force on Each Story (K) $F=DF+TF_X+TF_Y$	Center of Mass in X Direction (ft)	Center of Mass in Y Direction (ft)	Torsional Moment (K-ft) X	Torsional Moment (K-ft) Y
5	167.42	27.90	0.24	0.00	28.14	36	83.17	2220	1291
4	134.55	22.43	0.19	0.00	22.62	36	83.17	1785	1038
3	90.6	15.10	0.13	0.00	15.23	36	83.17	1202	699
2	52.56	8.76	0.23	0.00	8.99	36	99.57	697	1267
1	21.86	3.64	0.15	0.00	3.80	35.92	113.42	288	830

Fourth And Third Floor Loads

Story	Force (K)	Direct Force on Frame B.7 (K) Fix=(Kix/ΣKix)F	Torsional Force on Frame B.7 (K-ft) Fix=((Ki*xi)/Ip)M	Torsional Force on Frame B.7 (K-ft) Fiy=((Ki*yi)/Ip)M	Total Force on Each Story (K) $F=DF+TF_X+TF_Y$	Center of Mass in X Direction (ft)	Center of Mass in Y Direction (ft)	Torsional Moment (K-ft) X	Torsional Moment (K-ft) Y
5	167.42	27.90	0.13	0.00	28.03	36	83.17	1253	1291
4	134.55	22.43	0.10	0.00	22.53	36	83.17	1007	1038
3	90.6	15.10	0.07	0.00	15.17	36	83.17	678	699
2	52.56	8.76	0.12	0.00	8.88	36	99.57	393	1267
1	21.86	3.64	0.08	0.00	3.72	35.92	113.42	162	830

Second Floor Loads

Story	Force (K)	Direct Force on Frame B.7 (K) Fix=(Kix/ΣKix)F	Torsional Force on Frame B.7 (K-ft) Fix=((Ki*xi)/Ip)M	Torsional Force on Frame B.7 (K-ft) Fiy=((Ki*yi)/Ip)M	Total Force on Each Story (K) $F=DF+TF_X+TF_Y$	Center of Mass in X Direction (ft)	Center of Mass in Y Direction (ft)	Torsional Moment (K-ft) X	Torsional Moment (K-ft) Y
5	167.42	27.90	0.09	0.00	27.99	36	83.17	1481	1291
4	134.55	22.43	0.07	0.00	22.49	36	83.17	1190	1038
3	90.6	15.10	0.05	0.00	15.15	36	83.17	801	699
2	52.56	8.76	0.08	0.00	8.84	36	99.57	465	1267
1	21.86	3.64	0.06	0.00	3.70	35.92	113.42	192	830

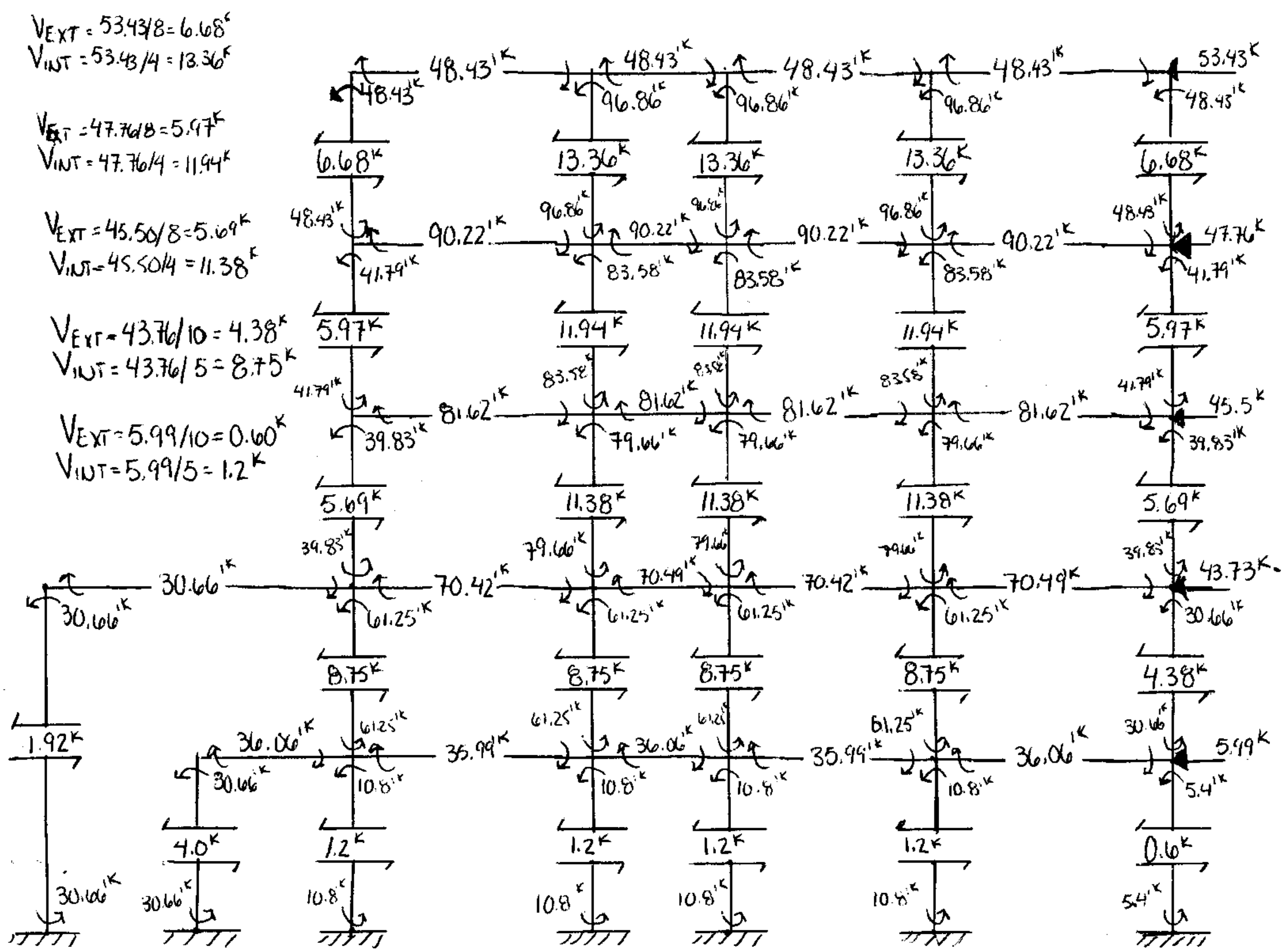
First Floor Loads

Story	Force (K)	Direct Force on Frame B.7 (K) Fix=(Kix/ΣKix)F	Torsional Force on Frame B.7 (K-ft) Fix=((Ki*xi)/Ip)M	Torsional Force on Frame B.7 (K-ft) Fiy=((Ki*yi)/Ip)M	Total Force on Each Story (K) $F=DF+TF_X+TF_Y$	Center of Mass in X Direction (ft)	Center of Mass in Y Direction (ft)	Torsional Moment (K-ft) X	Torsional Moment (K-ft) Y
5	167.42	18.72	0.02	0.00	18.75	36	83.17	1125	4576
4	134.55	15.05	0.02	0.00	15.07	36	83.17	904	3678
3	90.6	10.13	0.01	0.00	10.15	36	83.17	609	2477
2	52.56	5.88	0.01	0.00	5.89	36	99.57	353	575
1	21.86	2.44	0.003	0.00	2.45	35.92	113.42	149	64

APPENDIX E: PORTAL METHOD

This section shows the calculations and portal method analysis for the frames indicated in Appendix A of this report for the center portion of the building. These frames were not shown in the Portal Analysis section in the main body of the report.

Frame 13 Center Building



Frame B.7 Center Building