Visteon Village Corporate Center

Van Buren Township, MI

Technical Assignment #3

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Structural Option
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Executive Summary

This report is a technical analysis of the lateral load resisting system of the Visteon Village Corporate Center in Van Buren, MI. RAM Structural System and SAP2000 were the two main structural analysis programs used in this report. Hand calculations were also performed to determine the structural adequacy under combined loading of the members composing the special steel moment frames that account for the lateral system. This report takes a detailed look at both the seismic and wind forces present on the building and how the moment frames distribute these loads. It also analyzes other design considerations including story drifts, relative stiffness of the lateral framing, and overturning moments at the base of the building. The moment frame system in the building was coordinated to minimize the eccentricity between the center of rigidity and loading as efficiently as the design would allow, keeping the torsional shears as small as possible. As previously suspected, the wind loading is the controlling force in both the N-S and E-W direction of analysis, as the building is not located in a region with high seismic activity. While the system is able to adequately handle the loading, some of the independent and overall story displacements do not meet acceptable standards. This topic will be noted as a potential topic of investigation for a thesis proposal.

Introduction: Visteon Corporate Village Center

The Visteon Corporate Village Center is located in the Detroit metro area of Van Burin, MI. The facility is one of many office and laboratory buildings present on the corporate campus of the global automotive supplier. The campus is laid out and styled to provide a village type of atmosphere, complete with sidewalks and streetlights. All master planning, architecture and engineering of the campus and its various buildings was completed by the Detroit office of the SmithGroup.

The Visteon Corporate Village Center is five stories high, with the fifth story penthouse reaching a height of 72'-9" above grade, and has an overall size of 130,000 gross square feet. The building is a steel framed structure consisting of a composite steel decking system resisting gravity loading and a special steel moment frame system for lateral support. The majority of the building consists of 40'-0" x 20'-0" bays providing a large amount of floor area that is uninterrupted by column placement.

The following report is an assessment of the lateral loading to the structure due to wind and seismic forces, as well as the adequacy of the structure to resist these applied forces. The computer analysis programs of RAM Structural System and SAP2000 were used extensively to complete this report, as well as hand calculations to verify the correct member selection.

Existing Framing System

Foundation:

All of the foundation systems for the Visteon Village Corporate Center were designed based upon the findings of a geotechnical investigation performed by Somat Engineering on October 14, 2002**.** There is a deep foundation system to support all building columns, walls, grade beams and other foundation elements. The deep foundation elements are comprised of friction steel H-piles in native medium compact to compact sand. All H-piles consist of 75 foot long HP12x84 sections with concrete pile caps and are of ASTM A992 steel (Fy = 50 ksi). The number of piles for each foundation element range from 1 to 7 providing capacities of 100 kips to 1050 kips respectively. The concrete pile caps are of reinforced concrete construction with their top elevation at a minimum depth of 3’-6” below finished grade as to prevent frost heave. The dimensions of the caps range from 3’x3’ for a single H-pile element up to 13’x11’-8” for a 7 H-pile element. All concrete used in the foundation systems has a minimum compressive strength of 3000 psi.

Columns:

All of the columns of the building are composed of structural steel. The main column system is made up of ASTM A992 wide flange shapes ranging in size from W14x43 to W14x311. Typically, these columns rest upon the deep foundation system and extend 72 feet to the penthouse level with a column splice at an elevation of 52 feet (falling within the third story). These multistory columns are also part of the special moment frame system that resists lateral loading.

Floor and Roof Framing System:

The typical framing system for the Visteon Village Corporate Center is composed of structural steel composite beams and girders. The supported floor consists of 40 foot long ASTM A992 wide flange shapes spanning a column free space. The typical bay for each floor is 40’x20’ with wide flange beams spaced at 10’ on center supporting 3” composite metal floor deck with 3-1/4” light weight concrete fill providing a total slab depth of 6-1/4”. All supporting materials for this system can be found in the appendix.

Design Guides and Criteria

During the analysis of the lateral system used by the Visteon Corporate Village Center, the following design aids were used:

The 2006 International Building Code (IBC 2006)

Building Code Requirements for Structural Concrete 2008, American Concrete Institute (ACI 318-08)

Steel Construction Manual, 13th Edition, American Institute of Steel Construction (AISC)

Minimum Design Loads for Buildings and Other Structures 2005, American Society of Civil Engineers (ASCE 7-05)

Drift Criteria per the 2006 International Building Code

Δ_{wind} = H/400 (Allowable Building Drift)
$\Delta_{seismic}$ = $0.25h_{sx}$ (Allowable Story Drift)

The load cases used during this analysis were taken from section 1605 of the 2006 International Building Code. They included:

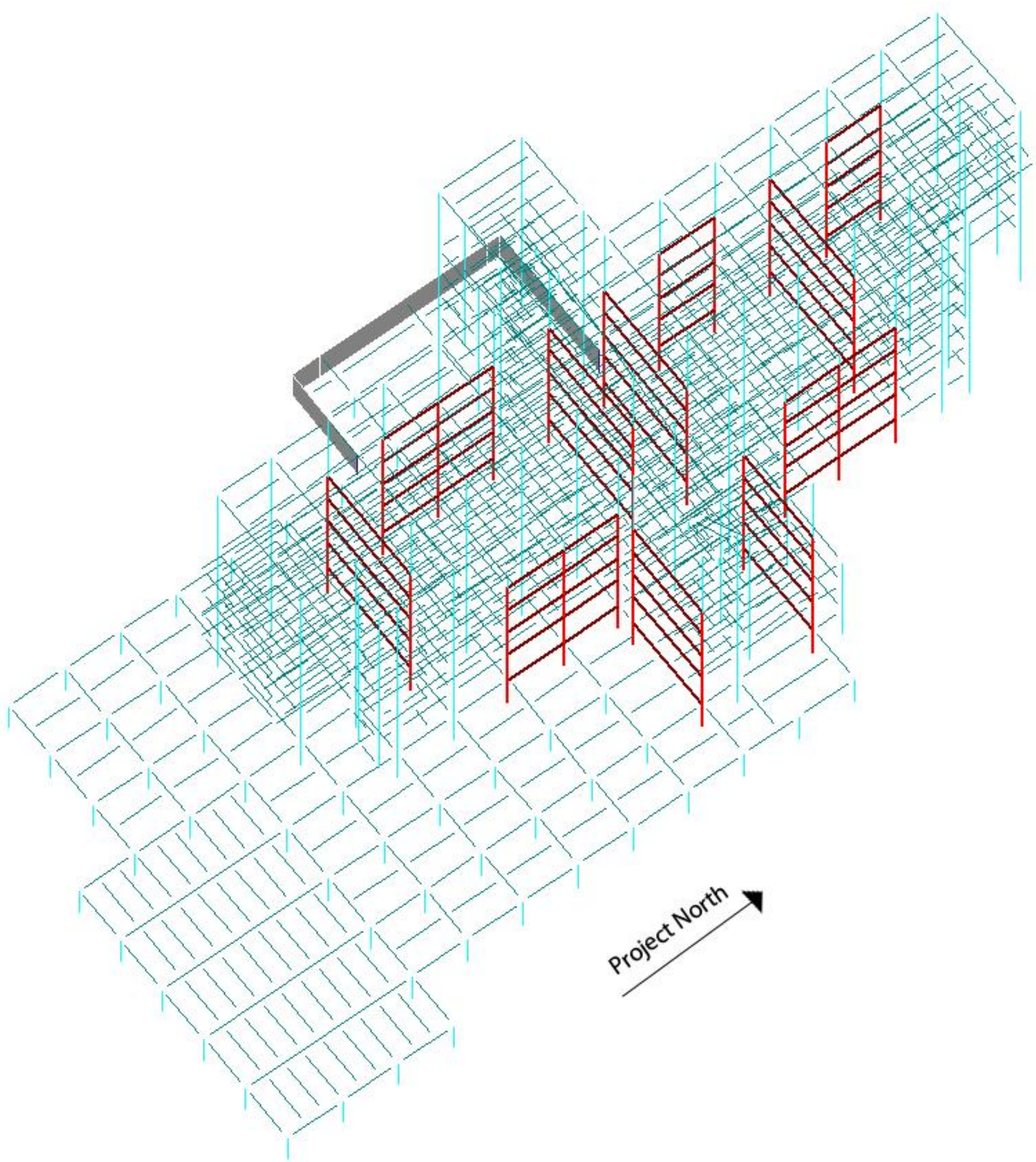
1.4D
1.2D + 1.6L + 0.5Lr
1.2D + 1.6Lr + (1.0L or 0.8W)
1.2D + 1.6W + 1.0L + 0.5Lr
1.2D + 1.0E +1.0L
0.9D + 1.6W
0.9D + 1.0E

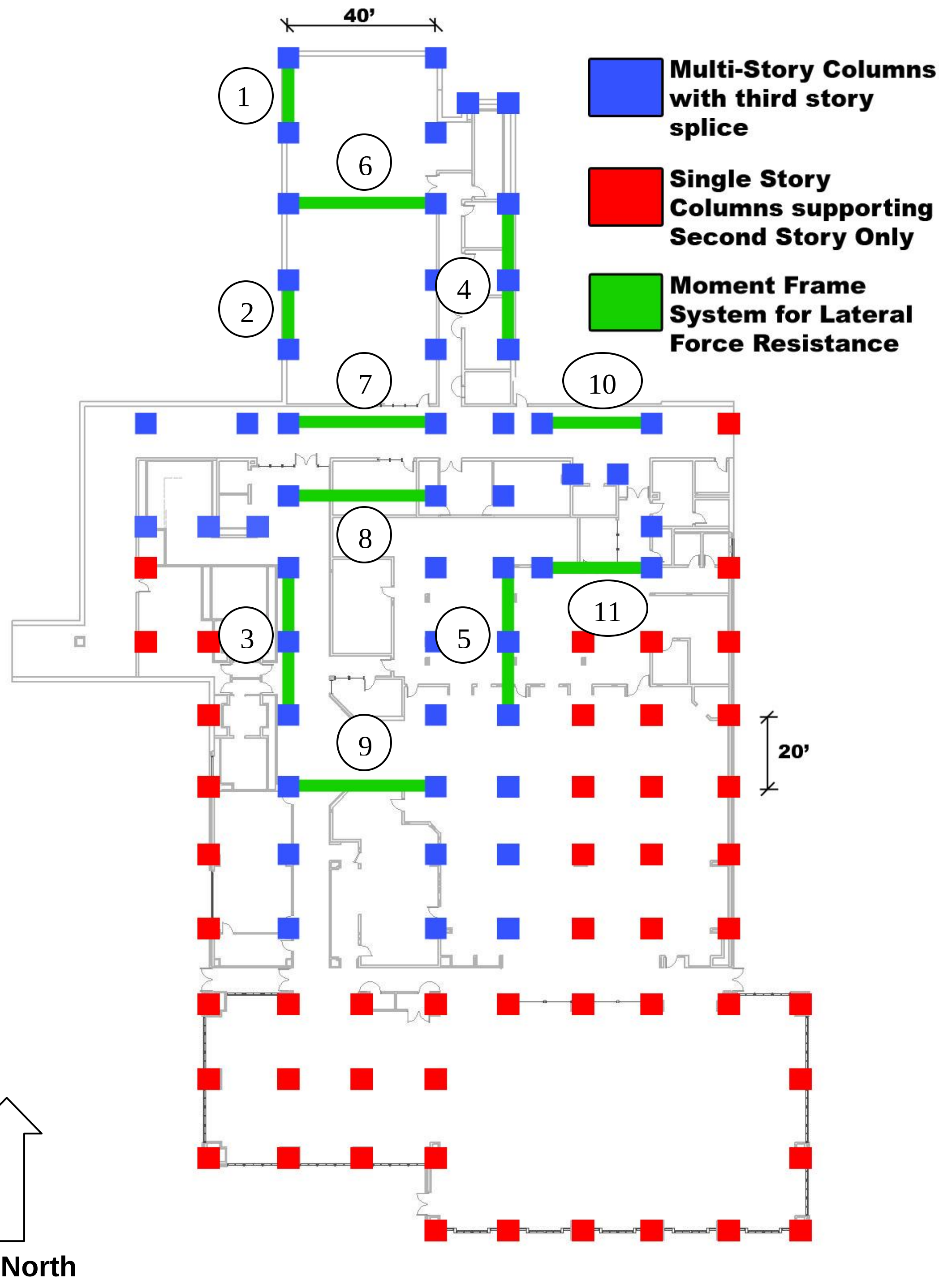
These combinations were analyzed in different directions and applied to various eccentricities during the computer analysis. There were 122 LRFD load combinations that were generated and analyzed. Due to time constraints and simplicity, snow loading was not included in this analysis.

Lateral System

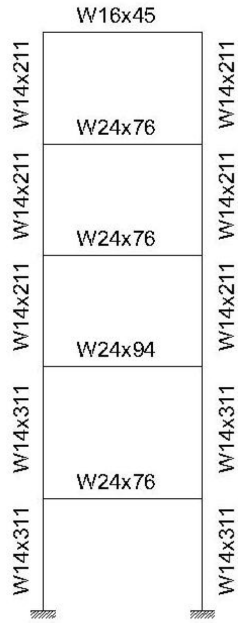
Lateral:

All lateral loads caused by wind and seismic forces are resisted by special steel moment frames. There are five moment frames running in the North/South direction of analysis and six moment frames running in the East/West direction of analysis. Each moment frame consists of multistory wide flange columns and wide flange beams. The columns are spliced at the third story, with the top three stories consisting of a W14x211 section being supported by a W14x311 extending through the lower two stories.

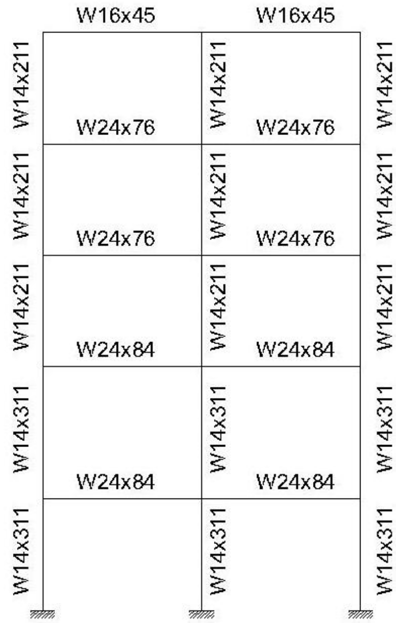



40'
Multi-Story Columns with third story splice
Single Story Columns supporting Second Story Only
Moment Frame System for Lateral Force Resistance
1
2
3
4
5
6
7
8
9
10
11
20'
Plan North

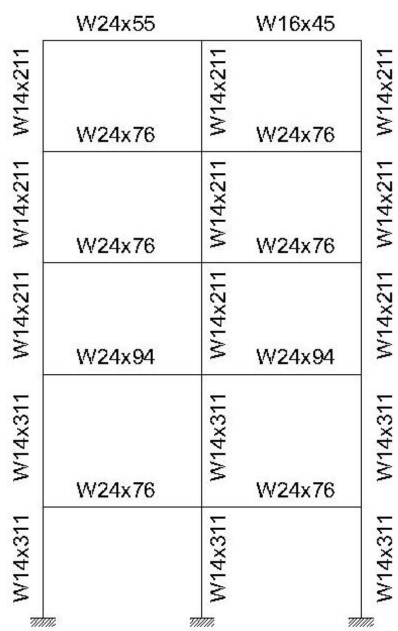
Frame sections of special steel moment frames spanning in the North-South direction:



Frames 1&2



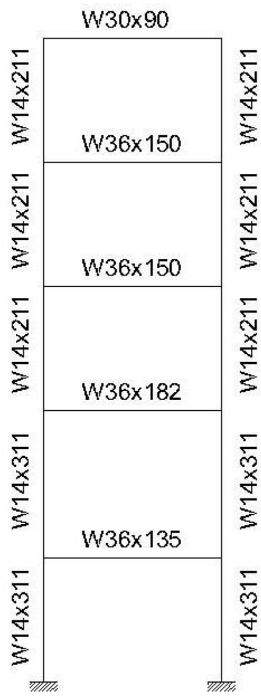
Frames 3&5



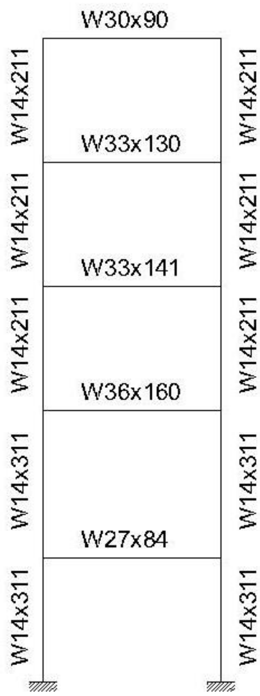
Frame 4

These diagrams are intended to show the frame sections and are not to scale.

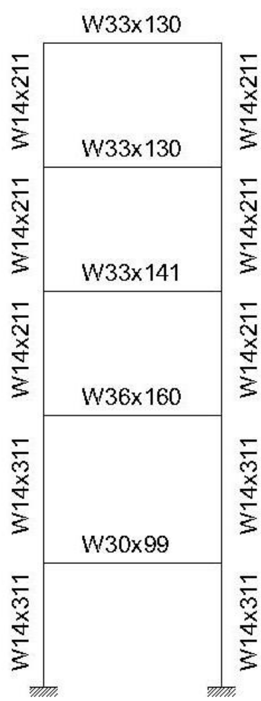
Frame sections of the special steel moment frames spanning in the East-West direction:



Frames 6, 7, 8 & 9



Frame 10



Frame 11

These diagrams are intended to show the frame sections and are not to scale.

Building Loads

Building Gravity Loads

For proper analysis of the lateral system, gravity loading was introduced to enable all appropriate load cases to be assessed. All of the dead and live loads used in the design of the building have been displayed previously in Technical Report one. The values come from the corresponding accepted values from the 13th Edition of the Steel Construction Manual, or are mandated by the 2006 International Building Code.

Dead Loads	Total	Construction
Supported Floor w/ Composite Floor Deck	91 psf	65 psf
Penthouse Composite Floor Deck	89 psf	80 psf
Roof Deck; Low, Concrete Roofs	85 psf	65 psf
Fire-Rated Composite Floor Deck, Penthouse	94 psf	80 psf

Live Loads (MBC, IBC 2000)	
Office Building	100 psf
Penthouse Floor*	150 psf

*Accounts for weight of mechanical equipment pads

Criteria for Wind Loading

Wind loading was analyzed according to ASCE 7-05 section 6.5. The assumptions used to obtain the magnitude of the wind pressures and forces are listed below:

V=	90 mph	**Rigid Structure**
k_d=	0.85	**G=**0.85
I=	1.15	**g_Q=g_v=**3.4
Exposure Category=	B	**z=(0.6)h=**43.6
		I_z=c(33/z)$^{1/6}$=0.286
k_{zt}=	1	**L_z=l(z/33)$^{\epsilon}$=**351.1
z_g=	1200 ft	
α=	7	

$Q=\text{sqrt}(1/(1+0.63((B+h)/L_z)^{0.63}))=$	0.814	NS
	0.768	EW
$G=0.925((1+1.7g_QI_zQ)/(1+1.7g_vI_z))=$	0.818	NS
	0.791	EW
Gc_{pi}=	-0.18	

Criteria for Seismic

Wind loading was analyzed according to ASCE 7-05 chapters 11 and 12. The assumptions used to obtain the magnitude of the seismic and forces are listed below:

Ss=	0.126		
		42.241	**83.433**
S1=	0.046	**N**	**W**
Category=	III		
Site Class=	D		
Fa=	1.6		
Fv=	2.4		
Sms= Fv*Ss=	0.2016		
Sm1= Fv*S1	0.1104		
SDS= 2*Sms/3=	0.1344		
SD1=2*Sm1/3=	0.0736		
Ct=	0.028		
hn=	108		
x=	0.8		
Ta=Ct*hn^x=	1.186		
Ts=Sd1/SDs=	0.548		
0.8*Ts=	0.438		
SDC=	B		
R=	7		
I=	1.25		
TL=	12		

SAP2000

To determine the distribution of forces in the special steel moment frames, a relative stiffness analysis was performed. This consisted of modeling all of the moment frames of one direction in a single plane, and then assigning a diaphragm constraint to each separate z level to ensure that the deflections in each frame would be identical. A unit force of 1000 kips was then placed on the top story joint of one of the frames, and the shear values in each frame was recorded. Using these shear ratios, the relative stiffness of each frame could be identified. The full results of the SAP analysis leading to these values can be found in the appendix.

North-South		East-West	
Frame	Rel. Stiff	Frame	Rel. Stiff
1	0.217	6	0.174
2	0.217	7	0.174
3	0.184	8	0.174
4	0.199	9	0.174
5	0.184	10	0.147
		11	0.157

RAM Structural System

RAM Structural System was used extensively in the analysis of the lateral load resisting system. This program is very powerful because it allows you to model all of the gravity members quickly and efficiently giving you a very realistic representation of the entire structural system. In doing so, the accuracy and efficiency of center of mass and center of rigidity calculations increases dramatically as compared to hand calculations. Another advantage of using RAM is the direct integration of ASCE 7-05 and the 2006 International Building Code load combinations and analysis options. Simply by entering in the criteria listed earlier in this report, the program is able to give you a very accurate and thorough analysis of the lateral system by automatically running iterations of the various seismic and wind load cases. As suspected, the critical load case for both the N-S and E-W directions turned out be wind loading. A summary of the data found is shown below, with more detailed documentation as to how these numbers were obtained found in the appendix.

Total Shear Distribution in the North-South direction of analysis:

x	1	2	3	4	5
5ph	55.9795	55.9795	47.59088	45.04759	41.76238
4	73.75257	73.75257	62.70062	59.34238	55.01467
3	92.46616	92.46616	78.60994	74.40586	68.97961
2	108.2859	108.2859	92.05909	87.27379	80.90911
1	127.918	127.918	108.7493	98.02862	90.87962

Total Shear Distribution in the East-West direction of analysis:

y	6	7	8	9	10	11
5ph	13.3578	18.23812	19.86489	26.37198	15.3563	19.34987
4	17.16339	23.41914	25.50439	33.84538	19.71866	24.84013
3	20.82976	28.39779	30.92047	41.01117	23.91063	30.11029
2	25.74384	33.79169	36.47431	47.20478	28.45223	35.25466
1	26.93449	39.03895	43.07377	59.21306	32.87037	42.41389

Story Drifts

Using the drift values acquired from the RAM analysis, the serviceability of the special steel moment frames was checked. All values obtained from the model were checked against the criteria specified by code for allowable story and total drift.

Story Drifts due to Wind Loading:

Story	Height	Story Drift	H/400	Acceptable	Total Drift	H/400	Acceptable
5ph	72.67	0.319	0.42	Yes	2.055	2.1801	Yes
4	58.67	0.358	0.42	Yes	1.736	1.7601	Yes
3	44.67	0.43856	0.42	No	1.378	1.3401	No
2	30.67	0.62744	0.5001	No	0.93944	0.9201	No
1	14	0.312	0.42	Yes	0.312	0.42	Yes

Story Drifts due to Seismic Loading:

Story	Height	Story Drift	.025hsx	Acceptable	Total Drift	.025hsx	Acceptable
5ph	72.67	0.05793	0.35	Yes	0.494	1.81675	Yes
4	58.67	0.0871	0.35	Yes	0.43607	1.46675	Yes
3	44.67	0.10408	0.35	Yes	0.34897	1.11675	Yes
2	30.67	0.15683	0.41675	Yes	0.24489	0.76675	Yes
1	14	0.08806	0.35	Yes	0.08806	0.35	Yes

The second and third story fail the deflection criteria set forth by code when the critical wind loading is applied. This is most likely due to the difference in story height at this level. The second story is 16’-8” in height which is slightly higher than the 14’-0” heights of the other floors. This provides a longer unbraced length that would decrease the rigidity allowing for larger deflections. A correction to this in an effort to restrict the drift to allowable limits mandated by code could be a potential topic area to investigate for a thesis proposal.

Overturning Effects

The maximum overturning moments were caused by the critical wind loading in the East-West direction. The overturning moment was found to be just under 29,000 ft-k. It is assumed that the friction pile foundation system that extends 75 ft below grade will be adequate to resist these moments. A summary of the overturning moment analysis is shown below:

		E-W dir		N-S dir	
Story	Height	Force	Moment	Force	Moment
5ph	72.67	242.6	17629.74	112.38	8166.655
4	58.67	77.13	4525.217	31.59	1853.385
3	44.67	81.08	3621.844	30.29	1353.054
2	30.67	68.67	2106.109	31.09	953.5303
1	14	71.36	999.04	34.07	476.98
Total			28881.95		12803.6

Member Spot Checks

Spot checks for lateral members on the lowest story of the most heavily loaded frame were analyzed by hand, and compared to the numbers produced by the computer analysis. Using interaction equations for the combined loading on the members, it was determined that the members checked were indeed adequate to carry the loads required. This conclusion agreed with the computer model, which produced slightly different numbers but overall very similar results. The member checks, as well as the RAM analysis results can be found in the appendix.

Conclusion

Reviewing the acquired information taken from RAM Structural System, SAP2000, and the hand calculations, the building is deemed acceptable under code, except for the two stories that failed the drift criteria. In the grand scheme of things, the fact that it failed by such a small amount in the lower floors and still met overall story drift limitations lends to the fact that this issue is not overly critical to the performance of the structure as a whole. Still, this issue will be looked at in greater detail as a possible thesis proposal.

All design and procedures performed were in accordance with the applicable codes and design guides. All members seemed to be well within their strength specifications for both shear and flexural capacities

Appendix

SAP2000 Relative Stiffness Analysis

N-S	Frame 1		Frame 2		Frame 3		Frame 4		Frame 5		Total
P=1000k	V	V/∑V	V	V/∑V	V	V/∑V	V	V/∑V	V	V/∑V	∑V
5 PH	123.26	0.194469	123.26	0.194469	111.5	0.175915	164.31	0.259234	111.5	0.175915	633.83
4	128.68	0.209498	128.68	0.209498	122.68	0.19973	111.51	0.181544	122.68	0.19973	614.23
3	133.02	0.211775	133.02	0.211775	117.92	0.187735	126.24	0.200981	117.92	0.187735	628.12
2	137.04	0.227151	137.04	0.227151	109.41	0.181353	110.4	0.182994	109.41	0.181353	603.3
1	138.92	0.242782	138.92	0.242782	100.37	0.175411	93.62	0.163614	100.37	0.175411	572.2
Total	660.92	0.216576	660.92	0.216576	561.88	0.184122	606.08	0.198605	561.88	0.184122	3051.68

E-W	Frame 6		Frame 7		Frame 8		Frame 9		Frame 10		Frame 11		Total
P=1000k	V	V/∑V	V	V/∑V	V	V/∑V	V	V/∑V	V	V/∑V	V	V/∑V	∑V
5 PH	163.16	0.164911	163.16	0.164911	163.16	0.164911	163.16	0.164911	154.02	0.155673	182.72	0.184681	989.38
4	174.92	0.174637	174.92	0.174637	174.92	0.174637	174.92	0.174637	153.42	0.153172	148.52	0.14828	1001.62
3	169.88	0.169839	169.88	0.169839	169.88	0.169839	169.88	0.169839	161.52	0.161481	159.2	0.159162	1000.24
2	178.9	0.178864	178.9	0.178864	178.9	0.178864	178.9	0.178864	136.52	0.136493	148.08	0.14805	1000.2
1	182.32	0.182378	182.32	0.182378	182.32	0.182378	182.32	0.182378	126.36	0.1264	144.04	0.144086	999.68
Total	869.18	0.174145	869.18	0.174145	869.18	0.174145	869.18	0.174145	731.84	0.146628	782.56	0.15679	4991.12

RAM Structural System Analysis Results

Using RAM structural system, the wind forces were found to be the prevailing lateral loads. Once this was determined, an analysis of the eccentricities was done to analyze the direct shear, torsional shear, and total shear of the individual frames. The following charts show the results of this analysis:

Eccentricities:

	COP		COR		Eccentricity		5% (Used by RAM)	
	x	y	x	y	x	y	x	y
5PH	67.45	207.48	70.09	203.62	2.64	-3.86	7.2	13.76
4	62.57	205.28	69.91	204.15	7.34	-1.13	7.2	13.76
3	70.46	207.84	69.97	204.55	-0.49	-3.29	7.2	13.76
2	69.63	198.58	70.22	205.32	0.59	6.74	7.13	12.44
1	76.74	147.84	70.78	206.84	-5.96	59	9.97	16.13

Direct Shears (N-S):

x	Vtot	1	2	3	4	5
5ph	242.6	52.54129	52.54129	44.66788	48.18166	44.66788
4	319.73	69.24578	69.24578	58.86918	63.50009	58.86918
3	400.82	86.80791	86.80791	73.79959	79.605	73.79959
2	469.49	101.6802	101.6802	86.44322	93.24323	86.44322
1	540.85	117.135	117.135	99.58213	107.4157	99.58213

Direct Shears (E-W):

y	Vtot	6	7	8	9	10	11
5ph	112.38	19.57045	19.57045	19.57045	19.57045	16.4781	17.62011
4	143.97	25.0717	25.0717	25.0717	25.0717	21.11009	22.57312
3	174.26	30.34656	30.34656	30.34656	30.34656	25.55147	27.32231
2	205.35	35.76073	35.76073	35.76073	35.76073	30.11014	32.19692
1	239.42	41.69386	41.69386	41.69386	41.69386	35.10577	37.53877

Distance of Moment Frames to Center of Rigidity:

	Frame 1		Frame 2		Frame 3		Frame 4		Frame 5	
N-S	x	y	x	y	x	y	x	y	x	y
Coord	40	310	40	250	40	160	100	260	100	160
5PH	30.09	-106.38	30.09	-46.38	30.09	43.62	-29.91	-56.38	-29.91	43.62
4	29.91	-105.85	29.91	-45.85	29.91	44.15	-30.09	-55.85	-30.09	44.15
3	29.97	-105.45	29.97	-45.45	29.97	44.55	-30.03	-55.45	-30.03	44.55
2	30.22	-104.68	30.22	-44.68	30.22	45.32	-29.78	-54.68	-29.78	45.32
1	30.78	-103.16	30.78	-43.16	30.78	46.84	-29.22	-53.16	-29.22	46.84

	Frame 6		Frame 7		Frame 8		Frame 9		Frame 10		Frame 11	
E-W	x	y	x	y	x	y	x	y	x	y	x	y
Coord	60	280	60	220	60	200	60	120	123.66	220	123.66	180
5PH	10.09	-76.38	10.09	-16.38	10.09	3.62	10.09	83.62	-53.57	-16.38	-53.57	23.62
4	9.91	-75.85	9.91	-15.85	9.91	4.15	9.91	84.15	-53.75	-15.85	-53.75	24.15
3	9.97	-75.45	9.97	-15.45	9.97	4.55	9.97	84.55	-53.69	-15.45	-53.69	24.55
2	10.22	-74.68	10.22	-14.68	10.22	5.32	10.22	85.32	-53.44	-14.68	-53.44	25.32
1	10.78	-73.16	10.78	-13.16	10.78	6.84	10.78	86.84	-52.88	-13.16	-52.88	26.84

Ki*di and ki*di^2 were obtained for use in the Torsional Shear equation:

Frame 1		Frame 2		Frame 3		Frame 4		Frame 5	
ki*di	ki*di^2	ki*di	ki*di^2	ki*di	ki*di^2	ki*di	ki*di^2	ki*di	ki*di^2
6.516765	196.0895	6.516765	196.0895	5.540217	166.7051	-5.94029	177.674	-5.50708	164.7166
6.477782	193.7505	6.477782	193.7505	5.507075	164.7166	-5.97604	179.8189	-5.54022	166.7051
6.490776	194.5286	6.490776	194.5286	5.518122	165.3781	-5.96412	179.1025	-5.52917	166.041
6.54492	197.7875	6.54492	197.7875	5.564153	168.1487	-5.91447	176.1328	-5.48314	163.2879
6.666203	205.1857	6.666203	205.1857	5.667261	174.4383	-5.80325	169.5709	-5.38003	157.2045

Frame 6		Frame 7		Frame 8		Frame 9		Frame 10		Frame 11	
ki*di	ki*di^2	ki*di	ki*di^2	ki*di	ki*di^2	ki*di	ki*di^2	ki*di	ki*di^2	ki*di	ki*di^2
-13.3012	1015.947	-2.8525	46.72395	0.630406	2.282069	14.56203	1217.677	-2.40177	39.34105	3.703391	87.47409
-13.2089	1001.897	-2.7602	43.74921	0.722703	2.999217	14.65433	1233.161	-2.32406	36.83636	3.78649	91.44372
-13.1393	991.3573	-2.69054	41.56891	0.792361	3.605243	14.72398	1244.913	-2.26541	35.00057	3.849206	94.498
-13.0052	971.2261	-2.55645	37.52873	0.926453	4.928729	14.85808	1267.691	-2.15251	31.59877	3.969934	100.5187
-12.7405	932.0927	-2.29175	30.15946	1.191154	8.147492	15.12278	1313.262	-1.92963	25.39393	4.208256	112.9496

Sum of ki*di^2 results for use in Torsional Shear equation:

5ph	3310.72
4	3308.828
3	3310.522
2	3316.636
1	3333.59
Tot	16580.3

Torsional Shears using the equation (M*ki*di)/(∑ki*di^2):

x	Vtot	M	1	2	3	4	5	6	7	8	9	10	11
5ph	242.6	1746.72	3.438215	3.438215	2.922992	-3.13407	-2.90551	-7.01766	-1.50497	0.332599	7.682857	-1.26716	1.953891
4	319.73	2302.056	4.506797	4.506797	3.831446	-4.15772	-3.8545	-9.18986	-1.92036	0.502807	10.19548	-1.61692	2.63438
3	400.82	2885.904	5.65825	5.65825	4.810351	-5.19914	-4.81998	-11.454	-2.34545	0.69073	12.83544	-1.97484	3.355495
2	469.49	3347.464	6.605754	6.605754	5.61587	-5.96944	-5.5341	-13.126	-2.58021	0.935064	14.99618	-2.17251	4.006834
1	540.85	5392.275	10.78297	10.78297	9.167121	-9.38709	-8.70251	-20.6084	-3.70704	1.92676	24.46196	-3.12129	6.807097

y	Vtot	M	1	2	3	4	5	6	7	8	9	10	11
5ph	112.38	1546.349	3.043807	3.043807	2.587688	-2.77455	-2.57221	-6.21264	-1.33233	0.294446	6.801535	-1.1218	1.729755
4	143.97	1981.027	3.87831	3.87831	3.297139	-3.57791	-3.31698	-7.90831	-1.65256	0.432689	8.773686	-1.39144	2.267008
3	174.26	2397.818	4.701283	4.701283	3.996787	-4.31982	-4.00479	-9.51679	-1.94877	0.573909	10.66461	-1.64084	2.787988
2	205.35	2554.554	5.041057	5.041057	4.285646	-4.55547	-4.22325	-10.0169	-1.96904	0.713577	11.44405	-1.65791	3.05774
1	239.42	3861.845	7.722557	7.722557	6.565318	-6.72286	-6.23257	-14.7594	-2.65491	1.379909	17.51919	-2.23541	4.875113

Total Shears on each Frame (Direct Shear + Torsional Shear):

x	1	2	3	4	5
5ph	55.9795	55.9795	47.59088	45.04759	41.76238
4	73.75257	73.75257	62.70062	59.34238	55.01467
3	92.46616	92.46616	78.60994	74.40586	68.97961
2	108.2859	108.2859	92.05909	87.27379	80.90911
1	127.918	127.918	108.7493	98.02862	90.87962

y	6	7	8	9	10	11
5ph	13.3578	18.23812	19.86489	26.37198	15.3563	19.34987
4	17.16339	23.41914	25.50439	33.84538	19.71866	24.84013
3	20.82976	28.39779	30.92047	41.01117	23.91063	30.11029
2	25.74384	33.79169	36.47431	47.20478	28.45223	35.25466
1	26.93449	39.03895	43.07377	59.21306	32.87037	42.41389

Member Spot Checks

Spot checks were performed on lateral frame members using combined loading interaction equations. The checks were performed using the forces from the critical wind loading case on the first story column and beam of frame 6. A RAM analysis was performed and then checked for accuracy by hand calculations with values out of the 13th Edition Steel Construction Manual.

Beam Spot Check:

Member Code Check

RAM Frame v11.2
DataBase: Lateral Analysis Tech 3 — 11/20/08 02:51:58
Building Code: IBC — Steel Code: AISC LRFD

BEAM INFORMATION:

Story Level = First   Frame Number = 0   Beam Number = 85
Fy (ksi) = 50.00
Beam Size = W36X135

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	40.00	10.00
Lu for Bending (ft)	40.00	10.00
K	1.00	1.00
Braced Against Joint Translation	No	No
Top Flange Continuously Braced	No	
Bottom Flange Continuously Braced	No	

CONTROLLING BEAM SEGMENT FORCES - SHEAR

Segment distance (ft) i - end ______ 0.00
j - end ______ 40.00

Load Combination: 1.200 D + 0.500 Lp + 1.600 W2

SHEAR CHECK:

Vux (kips) =	-104.78	0.90*Vnx (kips) =	576.72	Vux/0.90*Vnx =	0.182
Vuy (kips) =	-0.00	0.90*Vny (kips) =	511.92	Vuy/0.90*Vny =	0.000

CONTROLLING BEAM SEGMENT FORCES - FLEXURE

Segment distance (ft) i - end ______ 0.00
j - end ______ 40.00

Load Combination: 1.200 D + 0.500 Lp + 1.600 W2

CALCULATED PARAMETERS:

Pu (kips) =	-0.00	0.90*Pn (kips) =	1483.62
Mux (kip-ft) =	-1410.69	0.90*Mnx (kip-ft) =	1908.75
Muy (kip-ft) =	0.00	0.90*Mny (kip-ft) =	212.06
Cbx =	1.34		

INTERACTION EQUATION:

$Pu/\phi Pn$ = 0.000
Eq H1-1b: 0.000 + 0.739 + 0.000 = 0.739

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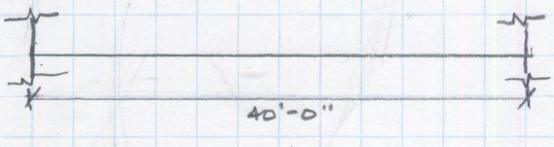
BEAM MEMBER CHECK



W36×135
FRAME 6, LEVEL 1

$P_u = 0$

$M_{ux} = -1410.69\,'^{k}$ (FROM RAM ANALYSIS OF CRITICAL LOAD CASE)

$M_{uy} = 0$ $\quad 1.2D + 1.6W + 0.5L_r$

$V_u = -104.78^{k}$

SHEAR

W36×135
$t_w = 0.60''$
$d = 35.6''$

$\phi V_n = \phi(0.6)F_y d\, t_w$

$\phi V_n = (1.0)(0.6)(50ksi)(35.6'')(0.60'')$

$\phi V_n = 640.8$

$640.8^{k} > 104.78^{k}$ ✓

FLEXURE

$\frac{P_u}{\phi P_n} = 0 < 0.2 \rightarrow H1-1b$ $\qquad \phi M_p = 1910$

STEEL MANUAL $\quad \frac{P_u}{\phi P_n} + \left(\frac{M_{ux}}{\phi M_{nx}} + \frac{M_{uy}}{\phi M_{ny}}\right) = 0 + \left(\frac{1410.69\,'^{k}}{1910\,'^{k}} + 0\right)$

$= 0.739$ (STEEL MANUAL) $= 0.739$ (RAM)

$0.739 < 1 \quad \therefore$ BEAM IS ADEQUATE ✓

Column Spot Check:

Member Code Check

RAM Frame v11.2
DataBase: Lateral Analysis Tech 3
Building Code: IBC

11/20/08 02:51:58
Steel Code: AISC LRFD

COLUMN INFORMATION:

Story Level = First   Frame Number = 0   Column Number = 31
Fy (ksi) = 50.00
Column Size = W14X311

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu (ft)	14.00	14.00
K	1.56	1.00
Braced Against Joint Translation	No	No

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.200 D + 0.500 Lp + 0.500 Rfp - 1.600 W2

Shear	Top	Vux (kips)	106.26
		Vuy (kips)	3.25
Shear	Bot.	Vux (kips)	106.26
		Vuy (kips)	3.25

SHEAR CHECK:

Vux (kips) =	-106.26	0.90*Vnx (kips) =	651.00	Vux/0.90*Vnx =	0.163
Vuy (kips) =	-3.25	0.90*Vny (kips) =	1977.05	Vuy/0.90*Vny =	0.002

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.200 D + 0.500 Lp + 0.500 Rfp - 1.600 W2

Axial		Load (kips)	823.27
Moment	Top	Mux (kip-ft)	303.90
		Muy (kip-ft)	-13.53
Moment	Bot.	Mux (kip-ft)	-1026.14
		Muy (kip-ft)	-59.03

CALCULATED PARAMETERS:

Pu (kips) =	823.27	0.85*Pn (kips) =	3455.06
Mux (kip-ft) =	-1026.14	0.90*Mnx (kip-ft) =	2261.25
Muy (kip-ft) =	-59.03	0.90*Mny (kip-ft) =	1119.38
Cbx =	2.08		

INTERACTION EQUATION:

Pu/φPn = 0.238
Eq H1-1a: 0.238 + 8/9*(0.454 + 0.053) = 0.689

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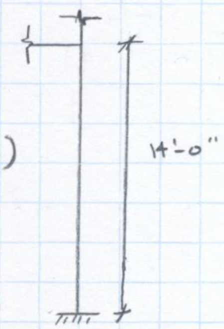
COLUMN MEMBER CHECK

$L_{ux} = 14'$ $L_{uy} = 14'$ FRAME 6, LEVEL 1

$K_x = 1.56$ $K_y = 1.00$



W14×311 (FROM RAM ANALYSIS OF CRITICAL LOAD CASE)

$P_u = 823^k$

$V_u = -106.26^k$

BOT $M_{ux} = -1026.14'^k$ TOP $M_{ux} = 303.9'^k$

$M_{uy} = -59.03'^k$ $M_{uy} = -13.53'^k$

SHEAR CHECK

$\phi V_n = \phi_v (0.5) F_y \cdot d \cdot t_w$

$\phi V_n = (1.0)(0.5)(50\,ksi)(17.1")(1.41")$

$\phi V_n = 602.78^k$

W14×311: $t_w = 1.41"$, $d = 17.1"$

$I_x = 4330\ in^4$

$I_y = 1610\ in^4$

$602.78^k > 106.26^k$ ∴ ✓

FLEXURE CHECK

$B_1 = C_m / \left(1 - \alpha P_r / P_{e_1}\right) \geq 1.0$

X

$C_m = 0.6 - 0.4\left(\frac{303.9'^k}{-1026.16'^k}\right) = 0.718$

$P_e = \frac{\pi^2 (29000)(4330)}{(1.56(14')(12))^2} = 18043.4^k$

$B_1 = \frac{0.718}{\left(1 - 1.0\left(\frac{823.0}{18043.4}\right)\right)} = 0.752 < 1.0 \rightarrow 1.0$

Y

$C_m = 0.6 - 0.4\left(\frac{-13.53'^k}{-59.03'^k}\right) = 0.508$

$P_e = \frac{\pi^2 (29000)(1610)}{(1.0(14)(12))^2} = 16326.9^k$

$B_1 = \frac{0.508}{\left(1 - 1.0\left(\frac{823.0}{16326.9}\right)\right)} = 0.535 < 1.0 \rightarrow 1.0$

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$B_2 = 1/\left(1 - \frac{\alpha P_{nt}}{\Sigma P_{e2}}\right) \geq 1.0$

$\Sigma P_{e2} = R_m \frac{\Sigma HL}{\Delta H}$

$R_m = 1.0$

$\Delta H_x = 0.31206''$

$\Delta H_y = 0.00153''$

$\Sigma P_{e2x} = \frac{1.0(94.39^k)(14)(12)}{0.312} = 50825.4^k$

$\Sigma P_{e,x} = \frac{1.0(0.03)(14)(12)}{0.00153} = 3294.12k$

$B_{2x} = \frac{1}{(1 - 1.0(823/50825.4))} = 1.02 > 1.0$

$B_{2y} = \frac{1}{(1 - 1.0(823/3294.12))} = 1.33 > 1.0$

$P_r = 1.02(823) = 839.46^k$

$M_{rx} = 1.02(1026.14) = 1046.67'k$

$M_{ry} = 1.33(59.03) + 1.02(59.03) = 138.72'k$

$p = 0.273 \times 10^{-3}$
$b_x = 0.393 \times 10^{-3}$ TBL 6-1
$b_y = 0.780 \times 10^{-3}$

$pP_r = (0.273E{-3})(823) = 0.225 > 0.2 \rightarrow H1\text{-}1a$

$0.225 + (0.393E{-3})(1046.67'k) + (0.780E{-3})(138.72'k) = 0.745 < 1.0$ ✓
(STEEL MANUAL)

RAM $= 0.689 < 1.0$ ✓

∴ COLUMN IS ADEQUATE