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Kettler Capitals Iceplex
Arlington, Virginia

Structural Option
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TECHNICAL REPORT III
LATERAL SYSTEM ANALYSIS AND CONFIRMATION DESIGN

EXECUTIVE SUMMARY

The Kettler Capitals Iceplex is the practice facility for the NHL franchise, Washington Capitals. It is located in Arlington, Virginia just outside Washington D.C. The Iceplex was constructed on top of the existing parking structure for the Ballston Mall in Arlington.

The gravity system of the Iceplex consists of mildly reinforced concrete on floors 1-5, post-tensioned concrete on level 6, and composite steel on floors 7-9.

The lateral framing system of the Iceplex is very complex and consists of concrete moment frames, steel moment frames, and steel braced frames. The goal of this report was to analyze the entire lateral system and understand how it works together as a whole.

Three analysis procedures were used for the wind load case. First, wind was analyzed using BOCA 1999, which was the analysis procedure used by the engineer of record, RGA. Second, wind was analyzed using IBC 2003. Finally, a less conservative approach was completed. This approach used IBC 2006 to find wind story forces on each level of the structure taking into account that windward and leeward pressures would not exist on some areas of the building. The absence of some pressures is due to the interference of adjacent structures which block wind forces.

Two analysis procedures were used to analyze the seismic load case. The first analysis allowed RAM to generate seismic forces using IBC 2003. The second analysis used IBC 2006 to calculate seismic story forces which were distributed as point loads on the structure. The engineer of record did not complete a seismic analysis because it was evident that wind would control using the BOCA code.

The distribution of lateral forces to each frame was estimated using the concept of relative stiffness. Each frame was modeled separately and a unit load was applied at the top floor. Using deflections from this load, distribution factors were calculated.

The structure was analyzed for strength, drift, and torsion. Hand calculations were used to check the design of the computer generated model. As expected, the member sizes could be decreased when using the less conservative wind load case. However, it was determined that drift controlled the design in most cases and these members needed to be upsized in order to limit deflections. Additional shear due to torsion was calculated and found to be significant, especially on the 9th floor where the center of mass and the center of rigidity had a large eccentricity.

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INTRODUCTION

The Kettler Capitals Iceplex is the practice facility for the National Hockey League team, Washington Capitals. It is located at the Ballston Common Mall in Arlington, Virginia at the intersection of Glebe Road and Randolph Street. This 137,000 square foot facility was built on top an existing parking structure and houses two regulation sized ice rinks, corporate offices, a training facility, and a pro shop. At 60 ft. above street level, the Kettler Capitals Iceplex is the home of the highest ice rink above street level in the United States.

Design for the Iceplex began in 2000; however, this was the third time the Ballston parking garage has been expanded. The original facility, which dates back to the 1950s, was a five story cast-in-place concrete structure reinforced with mild steel. Then in the 1980s, the parking garage was expanded two more times. In 1981, a five story L-shaped addition was constructed of cast-in-place post-tensioned concrete. Then in 1986, the existing five level structure was topped with two more levels, one post-tensioned concrete and the other composite steel. See Figure 1 for a schematic phasing diagram of these additions.

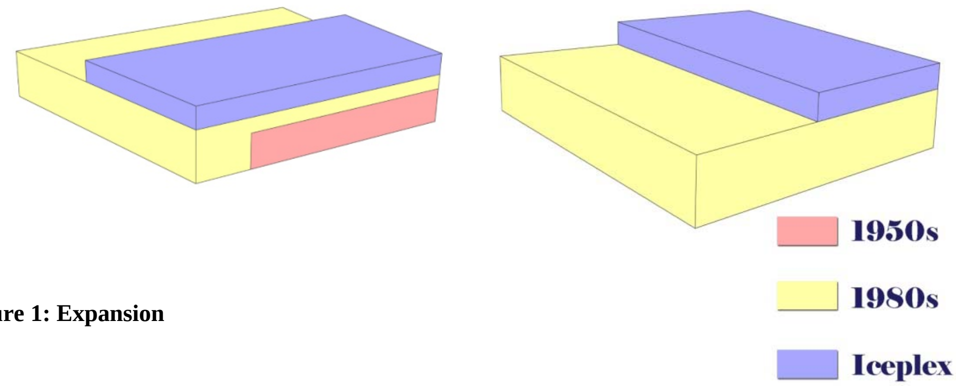


Figure 1: Expansion

There were several challenges when designing the Iceplex. The initial challenge was figuring out how to safely build an ice rink and roof weighing a total of 235 psf dead load plus 130 psf live load over an existing structure that was designed for a total expansion of 60 psf dead load and 50 psf live load. Another challenge was controlling deflection over the long 200 ft. span of each ice rink. A consultant recommended that the deflection be as close to zero as possible in order to prevent the ice from cracking. The need for large column-free spaces limited the locations where lateral members could be placed.

This report describes in detail the lateral framing system of the Iceplex and analyzes it for both wind and seismic loads. Three different distributions of wind forces are used: RAM generated BOCA wind loads, which is what the engineer of record, RGA, used; RAM generated IBC 2003 wind loads; and a less conservative distribution of wind forces taking into account adjacent structures that will block windward pressures and eliminate leeward pressures. Two seismic load conditions will also be examined: RAM generated IBC 2003 forces and hand calculated seismic story forces.

CODES USED FOR ANALYSIS AND DESIGN

The lateral system of the Kettler Capitals Iceplex was designed by the engineer of record, RGA, using Building Officials and Code Administrators, Inc (BOCA), 1996. Concrete design used American Concrete Institute, ACI 318-95 and the Manual of Steel Construction –Allowable Stress Design, 9th Edition, 1989 was used for the steel design.

This report uses a newer version of code to analyze the lateral framing system. The International Building Code (IBC 2003 and 2006) was used to determine wind and seismic loads and design procedures. The analysis and design checks of the steel lateral members use AISC Manual of Steel Construction –Load Reduction Factor Design, 13th Edition.

GRAVITY FRAMING SYSTEM

There were two expansion joints used in the construction of the new Iceplex, one running in the north-south direction and the other in the east-west direction. See Figure 2 for the locations of these joints. Expansion joint A, running north-south, separates the 8th floor parking structure from the 8th floor of the Iceplex. Expansion Joint B, running east-west, separates the ice rinks from team facility including the team offices and locker rooms. Both these joints span vertically the entire height of the building.

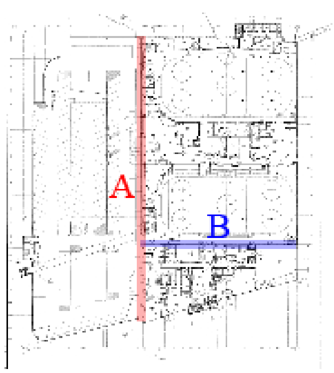
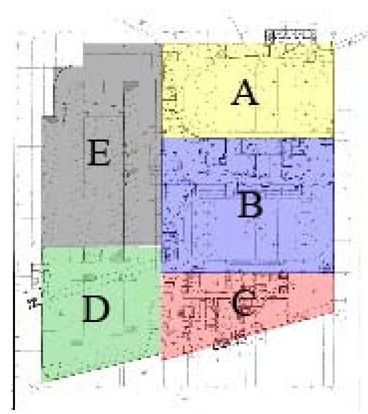


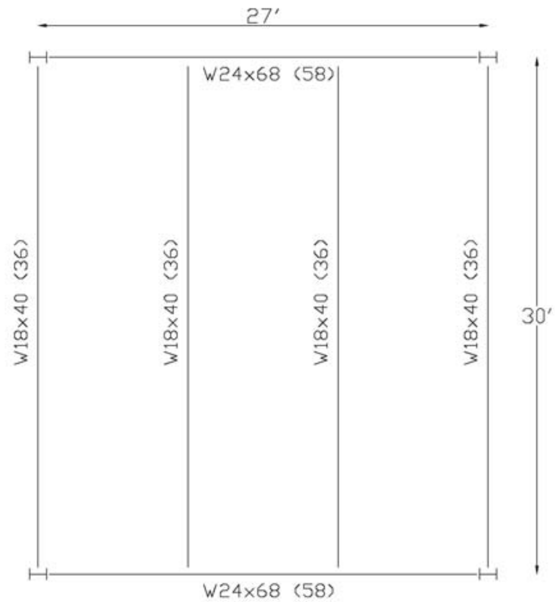
Figure 2: Location of Expansion Joints



Keyplan

The first five levels of Areas A and B are constructed of mildly reinforced cast-in-place concrete consisting of 26” and 28” diameter columns. The two-way slab is 10½” thick with 5¼ ” drop panels and column capitals. Levels six and seven are constructed of 27’x30’ composite steel bays with W16x26s spanning the 27’ direction and W24x55s spanning the 30’ direction. Levels eight and nine of the Iceplex also consist of composite steel framing with the same 27’-0” x 30’-0” bay. Figure 3 shows a typical bay framing of level eight supporting the ice rinks.

Figure 3: Enlarged Framing Plan



LATERAL FRAMING SYSTEM

The lateral system of Areas A and B is somewhat complicated due to the several expansions the structure has encountered over the years and the various materials that were used.

The first five levels of concrete were cast monolithically creating continuous concrete moment frames in each direction throughout the building footprint. In general, this lateral system has proven very stiff and efficient for resisting lateral loads but creates potential problems in seismic regions because of its heavy weight.

When the structure was expanded both horizontally and vertically in the 1980s, reinforcement of the lateral system was needed. The original lateral system is shown in yellow in Figure 4. Areas A and B on levels 7 and 8 were framed using composite steel with moment connections. There are ten moment frames spanning the east-west direction along the exterior of the building. Two frames spanning the north-south direction run the entire width of the building at both sides of the structure.

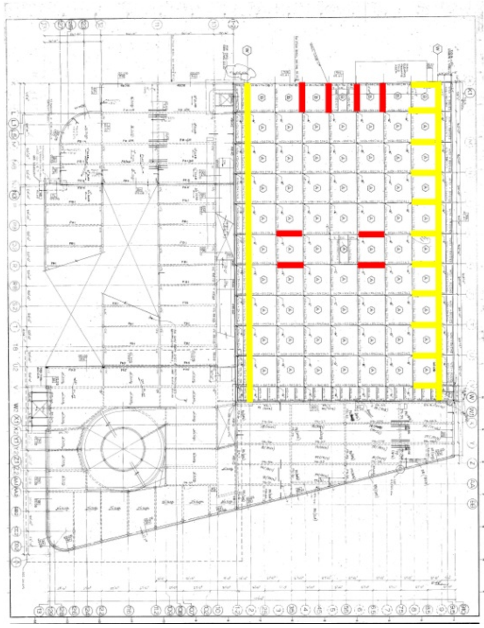


Figure 4: 7th Floor Lateral System

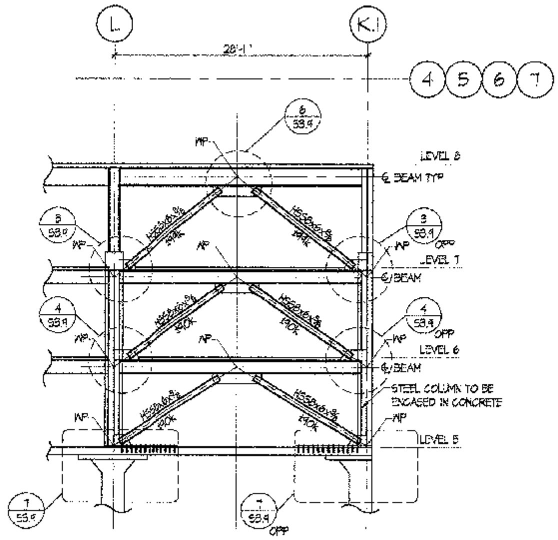


Figure 4A: Braced Frame Detail

Finally, when the Iceplex was added onto the parking structure, a mix of braced frames and moment connections was used. Eight braced frames were constructed on the 7th level reinforcing the existing structure for additional lateral forces. HSS8x6x3/8 tubes were used for all cross bracing. These frames are shown in red in Figure 4 and a detail of these braced frames is shown in Figure 4A. On the 8th level, there are a total of eight braced frames, four in each direction. These frames use the same tube sections and are shown in blue in Figure 5. Eight moment frames were constructed and were spaced evenly throughout with the exception of the voided areas from the ice rinks. These are shown in green in Figure 5.

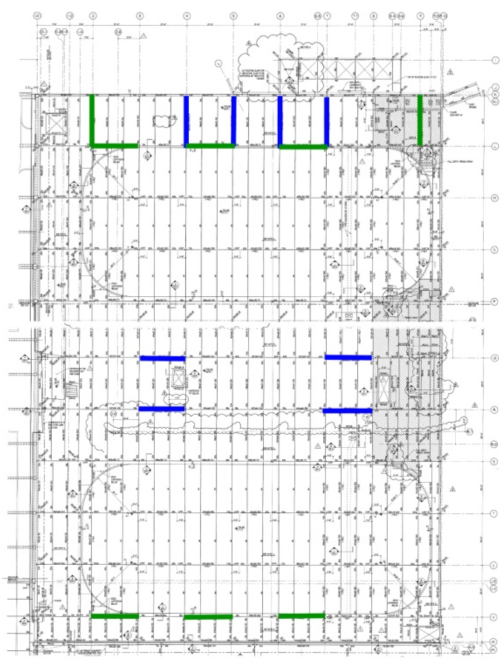


Figure 5: 8th Floor Lateral System

All lateral resisting members on the 9th level in this area are located in Area 9B. Seven moment frames span the north-south direction and four span the east-west direction. W24s and W33s are typical of the moment frames on the 9th level. Figure 6 shows the location of all lateral resisting frames in Area 9B.

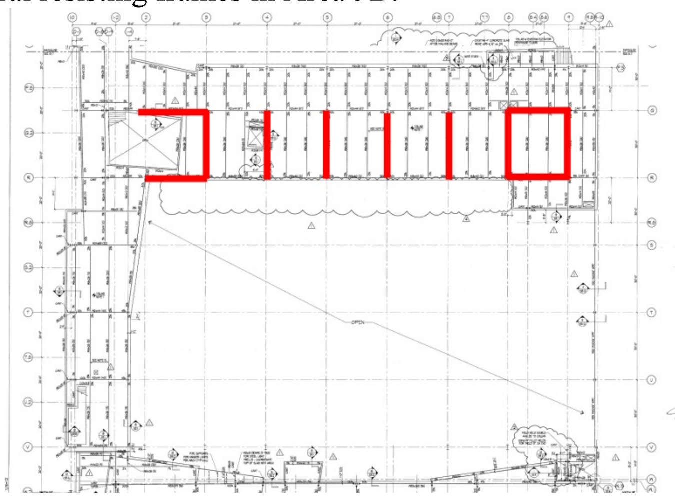


Figure 6: 9th Floor Lateral System

The lateral resisting system of Areas A and B may be difficult to understand in 2-dimensions. Figure 7 shows the entire lateral system in 3D which may help to explain how the various systems work together to resist wind and seismic loads.

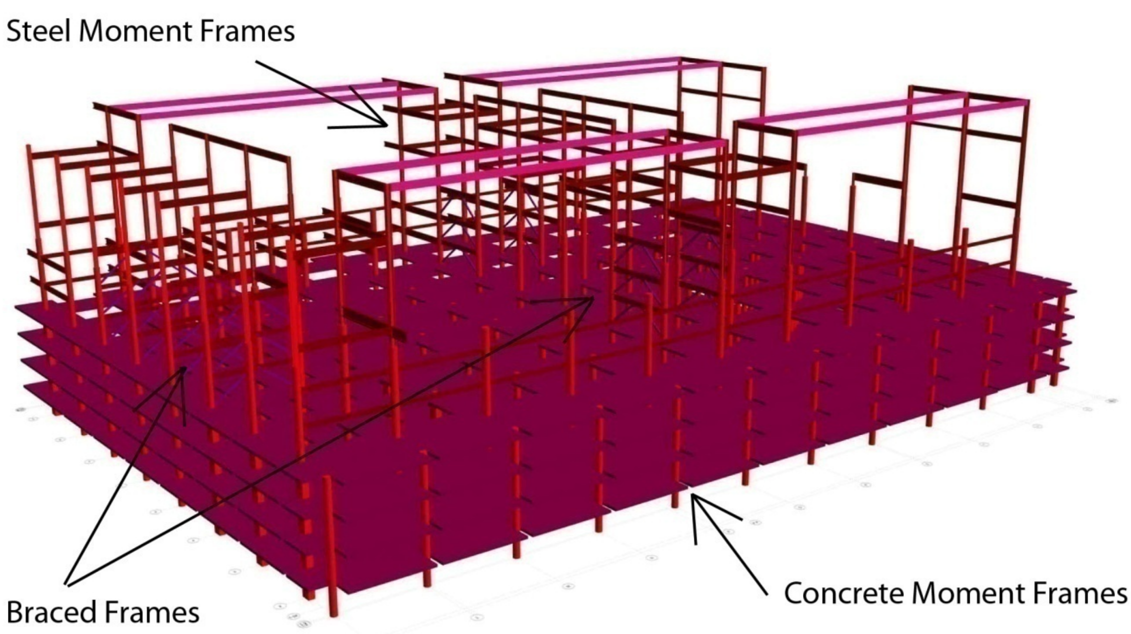


Figure 7: 3D Lateral Resisting System

ANALYSIS PROCEDURES

Wind Loads

Three methods of calculating and distributing wind loads were used during the analysis of the Iceplex lateral system.

1) RAM generated wind loads using BOCA 1999 (used by engineer of record, RGA)
2) RAM generated wind loads using IBC 2003
3) Hand calculated wind story forces using ASCE7-05 taking into consideration adjacent buildings

Both methods 1 and 2 use the structural design software, RAM Structural System (Frame Module), to generate wind loads using a certain code. When the building model was built in the program, the adjacent structures located on the other side of expansion joints were not modeled. This means that RAM included both windward and leeward pressures on *all* faces and stories of the building.

But in reality, much of the building exterior is blocked by these adjacent structures; therefore, no windward or leeward pressures would result. The hand calculated wind forces take this into consideration. For example, the west side of the building is blocked by the adjacent parking structure on floors 1-7 which will result in no windward or leeward pressures. Similarly, the south side of the building is blocked on all floors, resulting in zero windward and leeward pressures. Here is a list of input parameters used when calculating wind pressures:

- Basic Wind Speed (V) — 90 mph
- Wind Directionality Factor (K_d) — 0.85
- Importance Factor (I) — 1.15
- Exposure Category — B
- Internal Pressure Coefficient (C_{pi}) — 0.18
- Topographic Factor (K_{zt}) — 1.0
- External Pressure Coefficient ($C_{p,w}$) — 0.8
- External Pressure Coefficient ($C_{p,l}$) — -0.5
- External Pressure Coefficient ($C_{p,s}$) — -0.7

The tables below show how the wind forces will be distributed for the four different wind directions. The grayed out areas show where there will be no windward/leeward pressure. See the appendix for the calculation of pressures. It can be seen that east-west will control in the transverse direction and north-south will control in the longitudinal direction.

Table 1: East-West Wind

E-W Wind Distribution							
Level	Leeward Pressure (psf)	Windward Pressure (psf)	Wall Area-Leeward (SF)	Wall Area-Windward (SF)	Total Leeward Load (kips)	Total Windward Load (kips)	Total Load to be Applied (kips)
2		11.08		3240		35.90	**35.90**
3		11.75		3200		37.59	**37.59**
4		12.73		3200		40.74	**40.74**
5		13.51		3200		43.22	**43.22**
6		14.15		3200		45.29	**45.29**
7		14.71		3440		50.61	**50.61**
8	1.21	15.28	2000	3840	2.42	58.67	**61.09**
9	1.21	15.83	4960	4960	6.00	78.49	**84.49**
Roof	1.21	16.54	2960	2960	3.58	48.95	**52.53**

Table 2: West-East Wind

W-E Wind Distribution							
Level	Leeward Pressure (psf)	Windward Pressure (psf)	Wall Area-Leeward (SF)	Wall Area-Windward (SF)	Total Leeward Load (kips)	Total Windward Load (kips)	Total Load to be Applied (kips)
2	1.21		3240		3.92		**3.92**
3	1.21		3200		3.87		**3.87**
4	1.21		3200		3.87		**3.87**
5	1.21		3200		3.87		**3.87**
6	1.21		3200		3.87		**3.87**
7	1.21		3440		4.16		**4.16**
8	1.21	15.28	3840	2000	4.65	30.56	**35.20**
9	1.21	15.83	4960	4960	6.00	78.49	**84.49**
Roof	1.21	16.54	2960	2960	3.58	48.95	**52.53**

Table 3: North-South Wind

N-S Wind Distribution							
Level	Leeward Pressure (psf)	Windward Pressure (psf)	Wall Area-Leeward (SF)	Wall Area-Windward (SF)	Total Leeward Load (kips)	Total Windward Load (kips)	Total Load to be Applied (kips)
2		11.18		2430		27.18	**27.18**
3		11.86		2400		28.46	**28.46**
4		12.86		2400		30.86	**30.86**
5		13.64		2400		32.74	**32.74**
6		14.30		2400		34.32	**34.32**
7		14.87		2580		38.36	**38.36**
8		15.44		2880		44.47	**44.47**
9		16.00		3720		59.50	**59.50**
Roof		16.72		2220		37.11	**37.11**

Table 4: South-North Wind

S-N Wind Distribution							
Level	Leeward Pressure (psf)	Windward Pressure (psf)	Wall Area-Leeward (SF)	Wall Area-Windward (SF)	Total Leeward Load (kips)	Total Windward Load (kips)	Total Load to be Applied (kips)
2	4.54		2430		11.03		**11.03**
3	4.54		2400		10.90		**10.90**
4	4.54		2400		10.90		**10.90**
5	4.54		2400		10.90		**10.90**
6	4.54		2400		10.90		**10.90**
7	4.54		2580		11.71		**11.71**
8	4.54		2880		13.08		**13.08**
9	4.54		3720		16.89		**16.89**
Roof	4.54		2220		10.08		**10.08**

The tables below show the story forces and story shears that result from all three analysis procedures. The hand calculated story forces were inserted as point loads in RAM at the center of pressure. It can be seen that the forces and shears calculated by hand are significantly less than that of the RAM generated loads. This was to be expected since this is a much less conservative analysis. The RAM generated loads using BOCA and IBC are reasonable comparable.

Table 5: Wind Story Forces

Wind Applied Story Forces (kips)						
Story	RGA (BOCA)		Hand Calcs		RAM (IBC 2003)	
	Longitudinal	Transverse	Longitudinal	Transverse	Longitudinal	Transverse
R	179.9	250.15	37.11	52.53	150.32	201.56
9	74.17	96.2	59.5	84.49	62.49	78.4
8	79.87	97.51	44.47	61.09	68.79	80.38
7	51.76	84.44	38.36	50.61	45.31	68.09
6	45.56	72.32	34.32	45.29	40.63	61.38
5	42.96	68.94	32.74	43.22	39.01	59.45
4	39.98	64.74	30.86	40.74	37.09	56.86
3	36.83	60.1	28.46	37.59	34.96	53.88
2	33.72	55.33	27.18	35.9	32.44	50.14

Table 6: Wind Story Shears and Overturning Moment

Wind Story Shears (kips)						
Story	RGA		Hand Calcs		RAM (IBC 2003)	
	Longitudinal	Transverse	Longitudinal	Transverse	Longitudinal	Transverse
R	185.23	253.74	39.92	54.38	154.78	204.45
9	355.11	474.1	105.36	153.59	297.21	382.83
8	440.48	570.74	149.46	207.99	370.68	462.56
7	487.18	644.51	184.93	254.97	411.88	524.54
6	543.46	728.43	223.65	305.73	461.58	595.39
5	592.09	809.73	261.2	354.61	505.68	665.23
4	647.52	892.42	299.63	403.84	556.13	737.03
3	689.84	958.8	331.3	444.8	595.97	796.28
2	701.25	987.81	348.04	468.61	609.25	824.71
Base	701.25	987.81	348.04	468.61	609.25	824.71
Overturning Moment (ft-kip)	**201,792**	**271,529**	**75,936**	**104,475**	**171,040**	**221,620**

Seismic Loads

Two methods of calculating and distributing seismic loads were used to analyze the Iceplex lateral system.

1) RAM generated seismic loads using IBC 2003
2) Hand calculated seismic loads using ASCE7-05

The engineer of record, RGA did not conduct an in-depth seismic analysis because it was clear that wind would control using the BOCA code. Hand calculated seismic story forces were inserted in the RAM model as point loads at the center of mass. The calculation of these loads can be found in the appendix. The second form of analysis was a RAM generated load case. Parameters using ASCE7-05 were input into the software. The table below shows the story shears that result from both analyses. Here is a list of the parameters used for seismic analysis:

- Ss 0.154
- S1 0.0051
- Site Class D
- Occupancy Category III
- F_a 1.6
- F_v 2.4
- Importance Factor (I) 1.25
- Response Modification Coefficient 3 (most conservative)
- Approximate Period (T_a) 0.65

Table 7: Seismic Story Shears

Seismic Story Shear (kips)			
Story	**RGA (BOCA)**	**Hand Calcs**	**RAM (IBC 2003)**
R		25.31	59.46
9		43.28	115.99
8		124.84	294.97
7		175.66	370.98
6		228.96	435.09
5		349.97	530.38
4		471.62	604.25
3		608.93	643.8
2		724.93	635.97
Overturning Moment (ft-k)		**85,407**	**140,443**

It can be seen that the overturning moments are significantly different. The RAM generated loads created an overturning moment that was about 40% higher than the hand calculations. As in any hand calculation, there is always the possibility of human error during analysis. The most likely cause of the differing seismic loads is a miscalculation of building weight. Heavier buildings will create greater seismic forces. The weight of the structure may have been underestimated during hand analysis.

Summary of Lateral Loads

Overturning moment was calculated by ∑(story shears x story heights above base). When comparing the overturning moments for each lateral load, wind controlled the design in every case except when using the less conservative wind force hand calculation. In this situation, seismic will control whether using the hand calculated seismic loads or the computer generated loads.

DISTRIBUTION OF FORCES

Lateral forces are distributed throughout the resisting framing system by relative stiffness, or rigidity. Calculating the distribution of lateral forces for the Iceplex was completed by modeling individual frames using the structural modeling software, SAP2000. Due to the complexity of the lateral system of the structure, only the six frames running in the transverse direction were modeled. Each joint of every floor was assigned an equal joint constraint which allows the program to analyze each level as a rigid diaphragm. This means that the deflection of every joint at each floor will be the same. After modeling each frame the entire length of the building, a unit load of 1000 kips was placed at the top story. SAP then calculated the deflection of each story and the inverse of this deflection was taken as the stiffness of that level. Distribution factors were then calculated from these stiffnesses. These distribution factors can be used to calculate how much of the story force will go to each frame. The six frames analyzed are shown below. The tables showing the calculations for each frame at every level running in the transverse direction of the building can be found in the appendix.

Figure 8A: Frame L

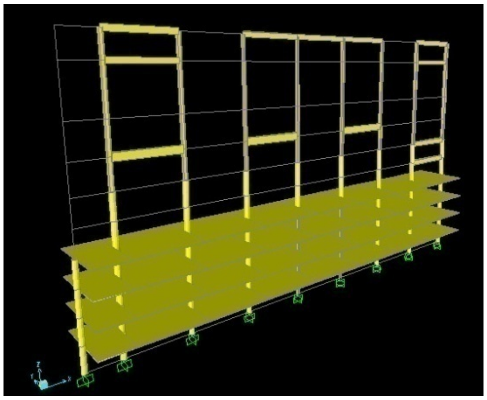


Figure 8B: Frame P

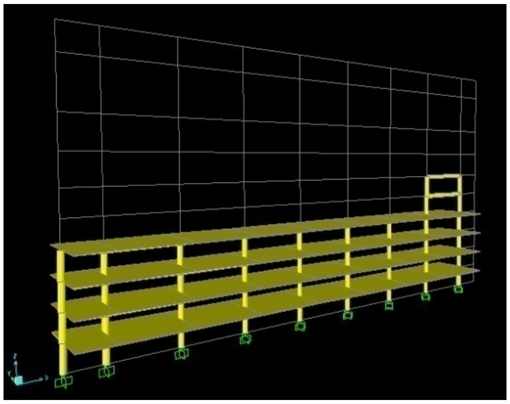


Figure 8C: Frames Q and R

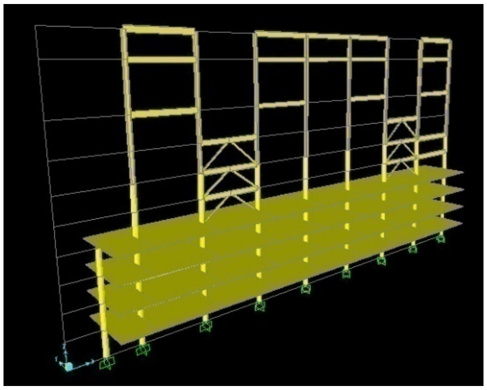


Figure 8D: Frame S

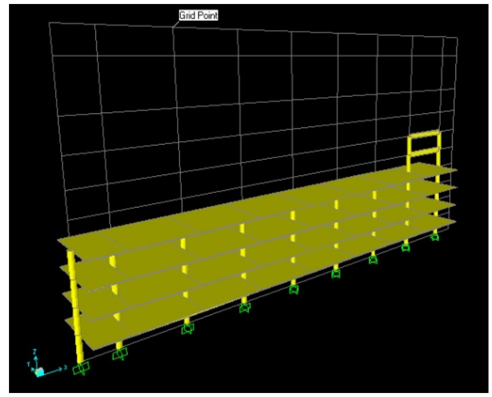
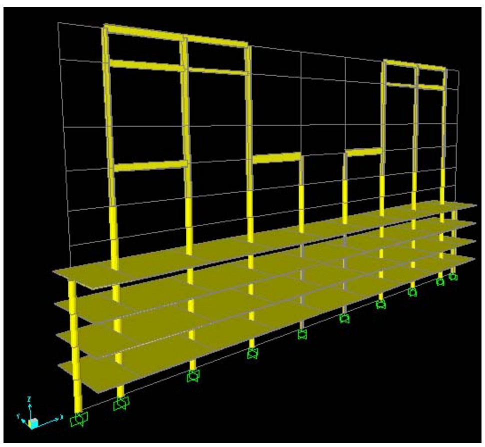


Figure 8E: Frame V



At each floor, it was concluded that Frames Q and R had the highest distribution factors consequently taking more load than any other frame. This was to be expected; it is obvious by visual inspection that these frames are stiffer than the others running in the transverse direction. Using the member forces command in RAM Frame confirms these results within 10%.

STRENGTH CHECK

Both hand calculations and RAM were used to check the strength of the lateral resisting system. Four load cases were analyzed using RAM: wind, BOCA; wind, IBC; wind, hand calculations; and seismic, IBC. In all cases, most members encountered very little stress, as you can see all the blue members in the figures below. There were a few overstressed members in the IBC and BOCA load cases. Theses members are highlighted in red. These members were typically the lateral bracing steel tubes and were overstressed by a maximum of 20% for wind cases and only 4% for the seismic case.

Figure 9A: BOCA Wind Member Code Check

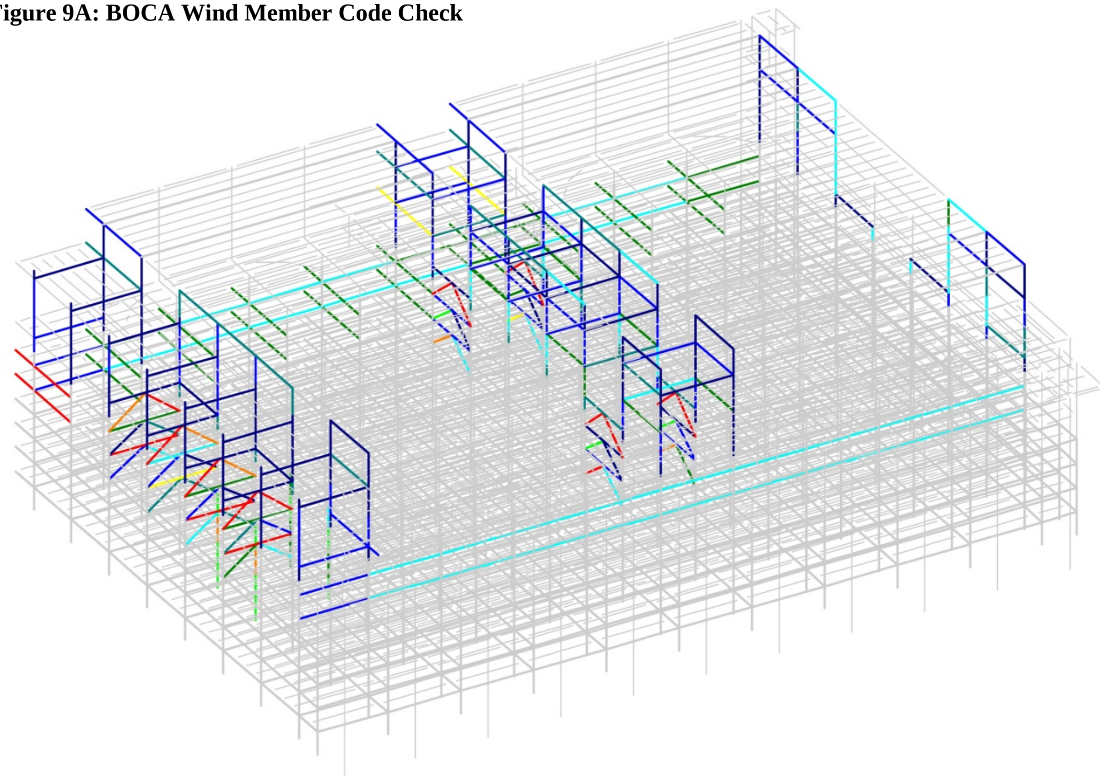


Figure 9B: IBC Wind Member Code Check

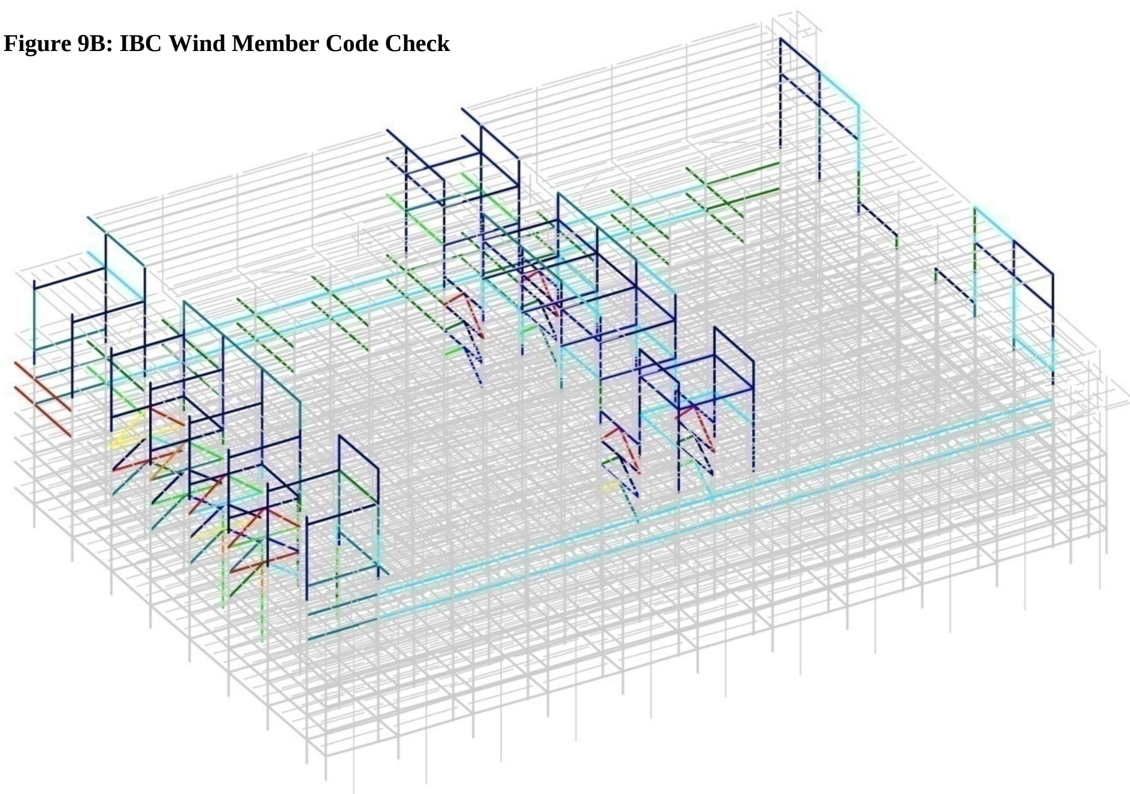


Figure 9C: Hand Calculation Wind Member Code Check

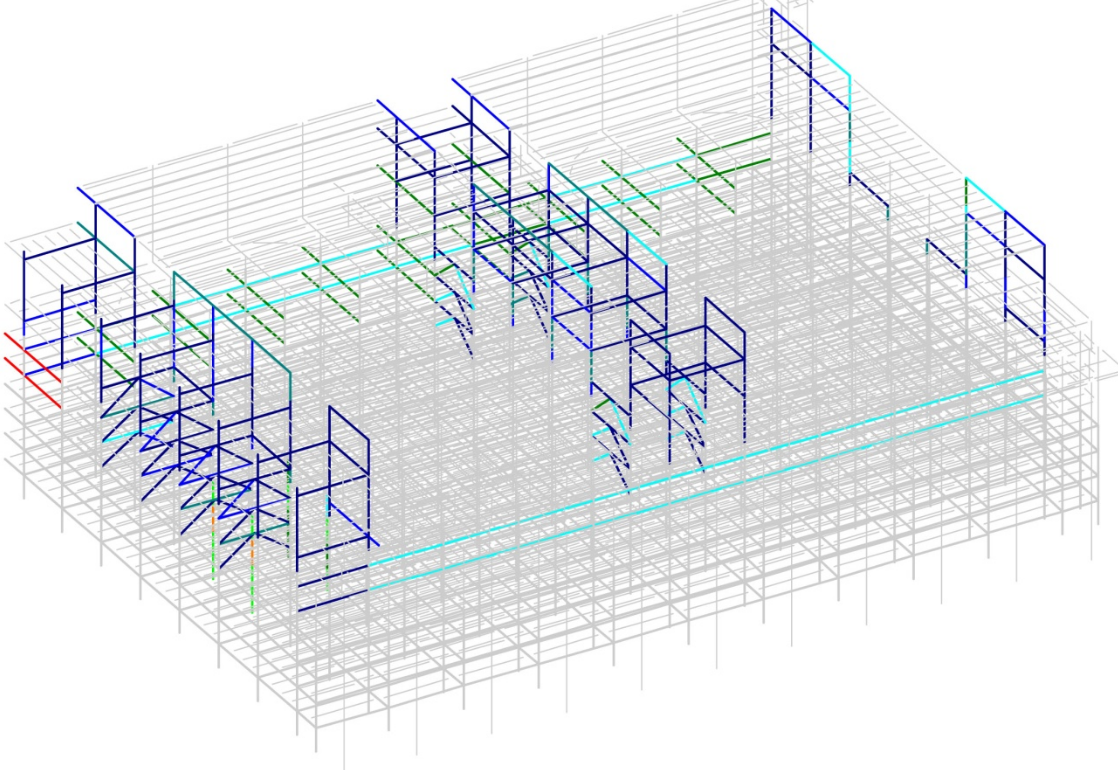
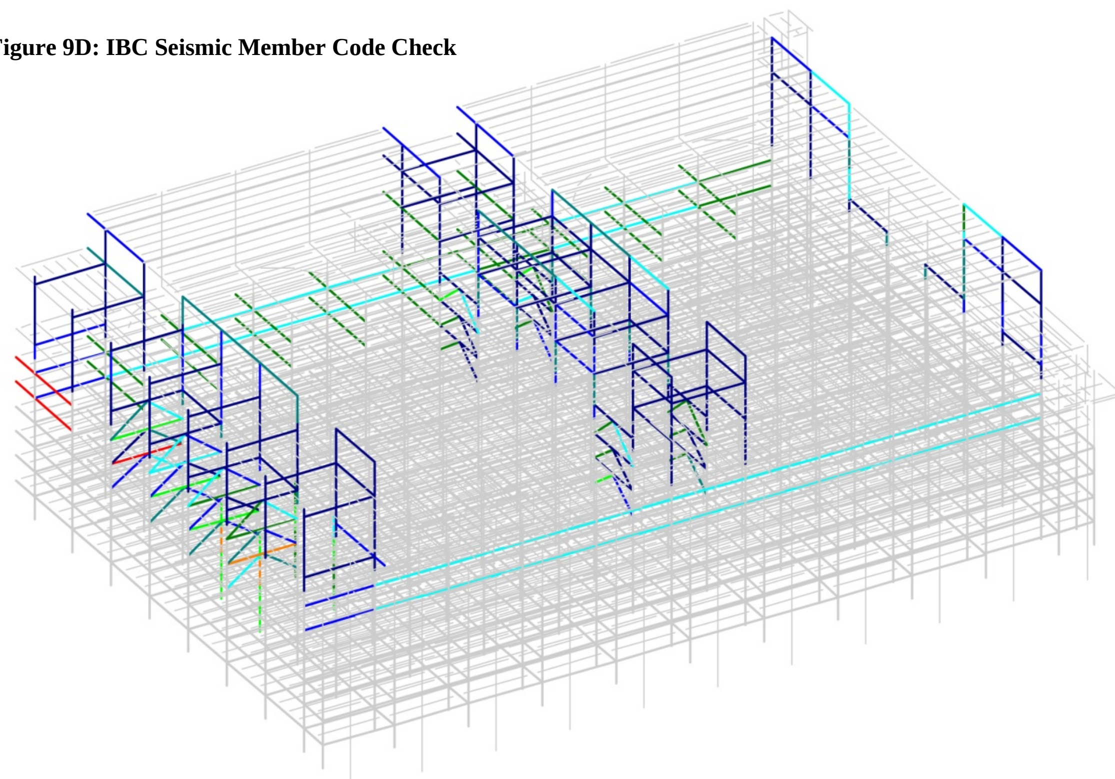


Figure 9D: IBC Seismic Member Code Check



The strength of three lateral members was also checked by hand. The member loads for the less conservative wind load case were found using RAM and were applied to the members. This will show how much the member sections can decrease from that of the more conservative approach used by the engineer of record. As expected, all three members can be decreased significantly based on strength requirements. The lateral braced member that was checked could be downsized from an HSS8x6x3/8 to an HSS 4.5x4.5x3/8 and the column design changed from a W14X120 to a W12x58. The analyzed beam was actually upsized based on engineering judgment. A W18x35 was used in the original design; however, a W18x55 was concluded to be the smallest size that should be used in a moment frame. As previously mentioned, this design was based on strength alone ignoring drift. Drift will most definitely control in this case making it necessary to upsize the members significantly. See the appendix for the hand calculations.

DRIFT

As previously mentioned, most members had very low stress. This means that the design was controlled by something other than strength. This controlling factor is most likely drift. Structural modeling software, such as RAM, make it extremely easy to determine drift, which would otherwise be a complicated calculation.

Drift, lateral deflection, is a serviceability issue and should be minimized in order to avoid uncomfortable conditions for building users. If the owner does not have a specific deflection requirement, the traditional criterion is to limit drift to H/400, where H is the total building height. In the case of the Iceplex, Δ_{max} = 102.75ft / 400 = 3.08". The table below shows the maximum story displacements of the existing design for all load cases analyzed throughout this report. It can be seen that the existing design is adequate to limit drift to H/400.

Table 8: Drift

<table>
<tr><th colspan="8">Maximum Drift Displacements (in)</th></tr>
<tr><th colspan="2"></th><th>Longitudinal</th><th>Transverse</th><th colspan="2"></th><th>Longitudinal</th><th>Transverse</th></tr>
<tr><td rowspan="3">Wind</td><td>RGA (BOCA)</td><td>2.231</td><td>2.714</td><td rowspan="3">Seismic</td><td>RGA</td><td></td><td></td></tr>
<tr><td>Hand Calcs</td><td>0.654</td><td>0.800</td><td>Hand Calcs</td><td>0.629</td><td>0.530</td></tr>
<tr><td>RAM (IBC)</td><td>1.878</td><td>2.201</td><td>RAM (IBC)</td><td>1.147</td><td>0.986</td></tr>
</table>

Interstory drift should also be considered as a serviceability issue. The table below shows the interstory drift for the controlling load case, Wind-BOCA. Using the 8^{th}-9^{th} story height of 18.5', Δ_{max} = 18.5' / 400 = 0.555". As shown in the table, interstory drift is not adequate based on the existing design. However, this could be because there is actually a low roof level in the structure. This should allow the interstory drift between the roof and low roof to be adequate.

Table 9: Interstory Drift

Controlling Load Case Interstory Drift		
Floor	**Drift**	**Interstory Drift**
R	2.71365	
9	1.73821	0.97544
8	0.90348	0.83473
7	0.68751	0.21597
6	0.59804	0.08947
5	0.47071	0.12733
4	0.33999	0.13072
3	0.19269	0.1473
2	0.05981	0.13288

Drift from lateral loads is extremely important in structures with expansion joints, such as the Iceplex. It is essential to keep lateral deflections smaller than the size of the joint. Otherwise, the two structures could clash which could cause some major issues. Both expansion joints in the Iceplex are 4" wide. The width of the expansion should be calculated using the following equation:

$$w = \sqrt{d1^2 + d2^2}$$

where d1 and d2 are the lateral displacements of the two structures on either side of the joint. Assuming that both structures will have a maximum deflection of 2.71", the width of the joint should be greater than 3.82". Since a joint width of 4" was used in the original design, it can be concluded that both the lateral system and joint size are adequate.

TORSION

Wind loads are applied at the center of pressure (geometric center of the building) and seismic forces are applied at the center of mass. The lateral system's center of rigidity is the point at which applied loads will not create a torsional rotation. Therefore, if the center of mass and center of rigidity have a large eccentricity, the lateral system will withstand an additional shear from torsion. It is essential to take that into consideration when analyzing and designing a lateral resisting system. Table 10a shows the distances between the centers of mass and centers of rigidity that will be used to calculate the torsional shear for the seismic load case. Table 10b shows the distances between the centers of pressure and centers of rigidity that will be used to calculate torsional shear for the wind load case. It can be seen that the seismic eccentricity on the 9th floor is significant.

Table 10a: COM and COR Eccentricities

Eccentricity Between COM and COR According to RAM	
Floor	**Distance (ft)**
1	15.2
2	15.5
3	13.8
4	14.7
5	16.7
6	13.5
7	10.6
8	2.1
9	**37.5**
R	4.4

Table 10b: COP and COR Eccentricities

Eccentricity Between COP and COR According to RAM	
Floor	**Distance (ft)**
1	15.1
2	**15.1**
3	12
4	11.7
5	13.1
6	6.6
7	3.6
8	2.5
9	14.3
R	5.3

Torsional shear can be calculated by the following equation:

$$T = \frac{V.e.d_i.R_i}{J}$$

where V = story shear, e = eccentricity, d_i = perpendicular distance from COR to member, J = torsional moment of inertia, and R_i = stiffness of member. Table 11a shows the torsional shear on the 9th floor for the IBC seismic load case. All shears in the transverse direction are within 10% of the total story shear. However, Frame A in the longitudinal direction accounts for 39% of the total story shear. This is a considerable percentage and should be taken into consideration during design. Table 11b shows the torsional shear for the engineer of record wind load case on floor two. All shears in both directions are within 5%, reasonably insignificant.

Figure 10: Frames Analyzed for Torsion

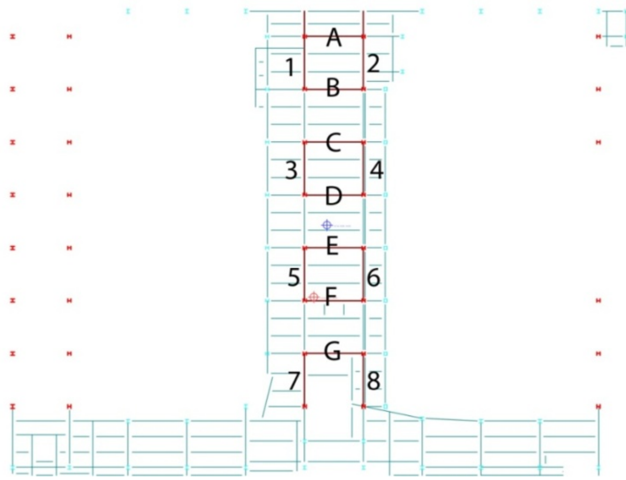


Table 11a: Seismic Torsional Shear

9th Floor Seismic Torsional Shear per Frame							
	Frame	Size	d (ft)	R (k/in)	R.d^2		T (k)
Transverse Members							
	1	W24x94	17.93	315.46	101,415		9.09
	2	W24x94	12.07	315.46	45,957		6.12
	3	W24x62	17.93	154.80	49,765		4.46
	4	W24x55	12.07	148.81	21,679		2.89
	5	W24x62	17.93	154.80	49,765		4.46
	6	W24x55	12.07	148.81	21,679		2.89
	7	W27x84	17.93	320.51	103,040		9.24
	8	W24x68	12.07	281.69	41,038		5.47
Longitudinal Members							
	A	W24x76	96.35	293.26	2,722,382		45.14
	B	W24x76	69.35	293.26	1,410,388		32.49
	C	W24x76	42.35	168.07	301,432		11.37
	D	W24x76	15.35	168.07	39,600		4.12
	E	W24x76	11.65	168.07	22,811		3.13
	F	W24x76	38.5	168.07	249,118		10.34
	G	W24x68	65.65	317.46	1,368,229		33.30
					6,548,300	**= J**	

IBC Seismic Story Shear=	116.73	Trans.	9th floor (e = 37.5')
	115.99	Long.	

Table 11b: Wind Torsional Shear

2nd Floor Wind Torsional Shear per Frame							
	Frame	Size	d (ft)	R (k/in)	R.d^2		T (k)
Transverse Members	1	W24x94	17.93	315.46	101,415		12.88
	2	W24x94	12.07	315.46	45,957		8.67
	3	W24x62	17.93	154.80	49,765		6.32
	4	W24x55	12.07	148.81	21,679		4.09
	5	W24x62	17.93	154.80	49,765		6.32
	6	W24x55	12.07	148.81	21,679		4.09
	7	W27x84	17.93	320.51	103,040		13.09
	8	W24x68	12.07	281.69	41,038		7.74
Longitudinal Members	A	W24x76	96.35	293.26	2,722,382		45.69
	B	W24x76	69.35	293.26	1,410,388		32.89
	C	W24x76	42.35	168.07	301,432		11.51
	D	W24x76	15.35	168.07	39,600		4.17
	E	W24x76	11.65	168.07	22,811		3.17
	F	W24x76	38.5	168.07	249,118		10.46
	G	W24x68	65.65	317.46	1,368,229		33.70
					6,548,300	**= J**	

RGA Wind Story Shear= 987.81 Trans.
701.25 Long.

2nd floor (e = 15.1')

CONCLUSIONS

- RAM generated wind load cases using BOCA and IBC gave comparable story shears.
- Using a less conservative wind load approach, story shears were decreased significantly. This approach takes into account the interference of adjacent buildings that block wind loads.
- Wind controlled in all cases except that of the less conservative wind analysis, in which seismic controlled.
- Drift controlled the design over strength. In order to limit drift to H/400, members needed to be upsized much larger than what was needed for strength.
- Torsional shear for the seismic load case was found to be significant in the longitudinal direction and must be taken into consideration.
- Torsional shear for the wind load case was proven to be insignificant in both the longitudinal and transverse directions.

APPENDIX

Wind

Main Windforce Resisting System

CODE:

International Building Code 2000 / ASCE 7-98

INPUT:

Building Height (H):	103 ft		
Building Depth (L):	240 ft		
Building Width (B):	320 ft	Wind on Broad Face of Building	
Number of Stories (N):	9		
Basic Wind Speed (V):	90 MPH	Figure 6-1 (pg 34)	
Wind Directionality Factor (K_d):	0.85	Table 6-6 (pg 61)	
Building Category:	III	Table 1-1 (pg 4)	
Importance Factor (I):	1.15	Table 6-1 (pg 55)	
Exposure Category:	B	6.5.6.1 (pg 28)	
Topographic Factor (K_{zt}):	1.00	6.5.7 (pg 29)	
Gust Effect Factor (G or Gf): Use Calculated?	Yes	6.5.8 (pg 29)	Frequency (Hz) = 1.11
Internal Pressure Coefficients (+/-GC_{pi}):	0.18	Table 6-7 (pg 62)	
External Pressure Coefficient (C_p windward):	0.8	Figure 6-3 (pg 42)	
External Pressure Coefficient (C_p leeward):	-0.3	Figure 6-3 (pg 42)	L/B = 0.75
External Pressure Coefficient (C_p sidewall):	-0.7	Figure 6-3 (pg 42)	

FORMULAS:

$p = qGC_p - q_i(GC_{pi})$ Equation 6-15 (6.5.12.2 pg 31)

$q_z = 0.00256(K_z)(K_{zt})(K_d)(V\text{^}2)(I)$ Equation 6-13 (6.5.10 pg 30)

CALCULATIONS:

Gust Effect Factor:

$z =$ 61.8 ft

frequency (n_1) = 1.11 Hz *Rigid*

$I_z =$ 0.270

$L_z = 394.4$

$g_Q = g_V = 3.4$

$Q = 0.777$

Rigid Structures: $G = 0.925[(1+1.7g_Q I_z Q)/(1+1.7g_V I_z)]$ Equation 6-2 (6.5.8.1 pg 29)

Flexible Structures: $G_f = 0.925[(1+1.7I_z\text{*sqrt}(g_Q\text{^}2Q\text{^}2+g_R\text{^}2(R\text{^}2)))/(1+1.7g_V I_z)]$ Equation 6-6 (6.5.8.2 pg 29)

$g_R = 4.215$
$N_1 = 6.307$
$\eta_B = 23.54$
$R = 0.110$

$b = 0.45$
$R_n = 0.044$
$R_B = 0.042$

$\alpha = 0.25$
$\eta_h = 7.58$
$\eta_L = 59.10$

$V_z = 69.49$
$R_h = 0.123$
$R_L = 0.017$

Velocity Pressure and Wind Force Summary

Location	Height (ft)	K_z	q_z	G	Ext. Pres. qGC_p	Internal Pressure q_i	Internal Pressure $q_i(GC_{pi})$	Combined Pressure (+GC_{pi})	Combined Pressure (-GC_{pi})	Design Load Ww + Lw Height (ft)	Design Load Ww + Lw Load (psf)
Windward	10.25	0.575	11.65	0.799	7.45	20.19	3.63	3.81	11.08	10.25	12.28
	20.25	0.626	12.69	0.799	8.11	20.19	3.63	4.48	11.75	20.25	12.95
	30.25	0.702	14.23	0.799	9.10	20.19	3.63	5.46	12.73	30.25	13.94
	40.25	0.762	15.44	0.799	9.87	20.19	3.63	6.24	13.51	40.25	14.71
	50.25	0.812	16.46	0.799	10.52	20.19	3.63	6.88	14.15	50.25	15.36
	60.25	0.855	17.33	0.799	11.08	20.19	3.63	7.44	14.71	60.25	15.92
	71.75	0.899	18.22	0.799	11.64	20.19	3.63	8.01	15.28	71.75	16.48
	84.25	0.941	19.07	0.799	12.19	20.19	3.63	8.56	15.83	84.25	17.03
	102.75	0.996	20.19	0.799	12.90	20.19	3.63	9.27	16.54	102.75	17.74
Leeward	ALL	0.996	20.19	0.799	-4.84	20.19	3.63	-8.47	-1.21		
Side Walls	ALL	0.996	20.19	0.799	-11.29	20.19	3.63	-14.92	-7.66		

Main Windforce Resisting System

CODE:

International Building Code 2000 / ASCE 7-98

INPUT:

Building Height (H):	103 ft		
Building Depth (L):	320 ft		
Building Width (B):	240 ft	Wind on Narrow Face of Building	
Number of Stories (N):	9		
Basic Wind Speed (V):	90 MPH	Figure 6-1 (pg 33)	
Wind Directionality Factor (K_d):	0.85	Table 6-6 (pg 61)	
Building Category:	III	Table 1-1 (pg 4)	
Importance Factor (I):	1.15	Table 6-1 (pg 55)	
Exposure Category:	B	6.5.6.1 (pg 28)	
Topographic Factor (K_{zt}):	1.00	6.5.7 (pg 29)	
Gust Effect Factor (G or Gf): Use Calculated?	Yes	6.5.8 (pg 29)	Frequency (Hz) = 1.11
Internal Pressure Coefficients (+/-GC_{pi}):	0.18	Table 6-7 (pg 62)	
External Pressure Coefficient (C_p windward):	0.8	Figure 6-3 (pg 42)	
External Pressure Coefficient (C_p leeward):	-0.5	Figure 6-3 (pg 42)	L/B = 1.33
External Pressure Coefficient (C_p sidewall):	-0.7	Figure 6-3 (pg 42)	

FORMULAS:

$p = qGC_p - q_i(GC_{pi})$ Equation 6-15 (6.5.12.2 pg 31)

$q_z = 0.00256(K_z)(K_{zt})(K_d)(V^2)(I)$ Equation 6-13 (6.5.10 pg 30)

CALCULATIONS:

Gust Effect Factor: frequency (n_1) = 1.11 Hz *Rigid* $g_Q = g_V = 3.4$

$z =$ 61.8 ft $I_z =$ 0.270 $L_z = 394.4$ $Q = 0.796$

Rigid Structures: $G = 0.925[(1+1.7g_QI_zQ)/(1+1.7g_vI_z)]$ Equation 6-2 (6.5.8.1 pg 29)

Flexible Structures: $G_f = 0.925[(1+1.7I_z*sqrt(g_Q\text{^}2Q\text{^}2+g_R\text{^}2(R\text{^}2)))/(1+1.7g_vI_z)]$ Equation 6-6 (6.5.8.2 pg 29)

$g_R = 4.215$	$b = 0.45$	$\alpha = 0.25$	$V_z = 69.49$
$N_1 = 6.307$	$R_n = 0.044$	$\eta_h = 7.58$	$R_h = 0.123$
$\eta_B = 17.65$	$R_B = 0.055$	$\eta_L = 78.80$	$R_L = 0.013$
$R = 0.126$			

Velocity Pressure and Wind Force Summary

Location	Height (ft)	K_z	q_z	G	Ext. Pres. qGC_p	Internal Pressure q_i	$q_i(GC_{pi})$	Combined Pressure (+GC_{pi})	(-GC_{pi})	Design Load Ww + Lw Height (ft)	Load (psf)
Windward	10.25	0.575	11.65	0.810	7.55	20.19	3.63	3.92	11.18	10.25	15.73
	20.25	0.626	12.69	0.810	8.23	20.19	3.63	4.59	11.86	20.25	16.40
	30.25	0.702	14.23	0.810	9.23	20.19	3.63	5.59	12.86	30.25	17.40
	40.25	0.762	15.44	0.810	10.01	20.19	3.63	6.38	13.64	40.25	18.19
	50.25	0.812	16.46	0.810	10.67	20.19	3.63	7.03	14.30	50.25	18.84
	60.25	0.855	17.33	0.810	11.23	20.19	3.63	7.60	14.87	60.25	19.41
	71.75	0.899	18.22	0.810	11.81	20.19	3.63	8.17	15.44	71.75	19.98
	84.25	0.941	19.07	0.810	12.36	20.19	3.63	8.73	16.00	84.25	20.54
	102.75	0.996	20.19	0.810	13.08	20.19	3.63	9.45	16.72	102.75	21.26
Leeward	ALL	0.996	20.19	0.810	-8.18	20.19	3.63	-11.81	-4.54		
Side Walls	ALL	0.996	20.19	0.810	-11.45	20.19	3.63	-15.08	-7.81		

Seismic

SEISMIC LOADING CALCULATIONS

REF: ASCE7-05

General Information

Ss=	0.154	Fa=	1.6	SDS=	0.164
S1=	0.0051	Fv=	2.4	SD1=	0.008
Site Class:	D				
Occ. Cat.	III			Seismic Design Category:	A

Seismic Response Coefficient

I=	1.25	CSa=	0.068	CS=	0.005
R=	3.00	CSmax=	0.005		
Ta=	0.65	CSmin=	0.010		

Equivalent Lateral Force Calculation

Fx = 0.01Wx
Fx = story forces

Vx = story shear
W = 78245.0
$V=C_S$*W 411.2

Floor(x)	Height(ft.)	Wx(kips)	Fx(kips)	Vx(kips)	Mx (ft.-kips)
Roof	102.75	3000.0	30.0	30.0	0.0
9	84.25	1340.0	13.4	43.4	3082.5
8	71.75	9920.0	99.2	142.6	4211.5
7	60.25	5245.0	52.5	195.1	11329.1
6	50.25	10680.0	106.8	301.9	14489.2
5	40.25	10680.0	106.8	408.7	19855.9
4	30.25	10680.0	106.8	515.5	24154.6
3	20.25	10680.0	106.8	622.3	27385.3
2	10.25	10680.0	106.8	729.1	29548.0
1	0.00	5340.0	53.4	782.5	30642.7
total		78245.0			

Stiffness

1st Floor Transverse Stiffness

FRAME	DEFLECTION	STIFFNESS	D.F.
L	0.1472	6.7935	0.1793
P	0.1459	6.8540	0.1809
Q	0.1208	8.2781	0.2185
R	0.1208	8.2781	0.2185
S	0.1414	7.0721	0.1867
V	1.6352	0.6115	0.0161
		37.8875	1.0000

2nd Floor Transverse Stiffness

FRAME	DEFLECTION	STIFFNESS	D.F.	FORCE TO FRAME (KIPS) ENGINEER OF RECORD	FORCE TO FRAME (KIPS) HAND CALCS
L	0.4763	2.0995	0.1769	9.79	6.35
P	0.4803	2.0820	0.1754	9.71	6.30
Q	0.3749	2.6674	0.2248	12.44	8.07
R	0.3749	2.6674	0.2248	12.44	8.07
S	0.4616	2.1664	0.1825	10.10	6.55
V	5.411	0.1848	0.0156	0.86	0.56
		11.8675	1.0000		

	RGA	Hand
WIND=	55.33	35.9

3rd Floor Transverse Stiffness

FRAME	DEFLECTION	STIFFNESS	D.F.	FORCE TO FRAME (KIPS) ENGINEER OF RECORD	FORCE TO FRAME (KIPS) HAND CALCS
L	0.8964	1.1156	0.1715	10.31	6.45
P	0.92	1.0870	0.1671	10.05	6.28
Q	0.6536	1.5300	0.2353	14.14	8.84
R	0.6536	1.5300	0.2353	14.14	8.84
S	0.8726	1.1460	0.1762	10.59	6.62
V	10.5783	0.0945	0.0145	0.87	0.55
		6.5030	1.0000		

	RGA	Hand
WIND=	60.1	37.59

4th Floor Transverse Stiffness

FRAME	DEFLECTION	STIFFNESS	D.F.	FORCE TO FRAME (KIPS) ENGINEER OF RECORD	FORCE TO FRAME (KIPS) HAND CALCS
L	1.3424	0.7449	0.1617	10.46	6.59
P	1.415	0.7067	0.1534	9.93	6.25
Q	0.8562	1.1680	0.2535	16.40	10.33
R	0.8562	1.1680	0.2535	16.40	10.33
S	1.3155	0.7602	0.1650	10.68	6.72
V	16.8525	0.0593	0.0129	0.83	0.52
		4.6071	1.0000		

	RGA	Hand
WIND=	64.71	40.74

5th Floor Transverse Stiffness

FRAME	DEFLECTION	STIFFNESS	D.F.	FORCE TO FRAME (KIPS) ENGINEER OF RECORD	FORCE TO FRAME (KIPS) HAND CALCS
L	1.7572	0.5691	0.1483	10.22	6.41
P	2.0935	0.4777	0.1245	8.58	5.38
Q	0.914	1.0941	0.2851	19.65	12.32
R	0.914	1.0941	0.2851	19.65	12.32
S	1.7797	0.5619	0.1464	10.09	6.33
V	24.5072	0.0408	0.0106	0.73	0.46
		3.8376	1.0000		

	RGA	Hand
WIND=	68.94	43.22

6th Floor Transverse Stiffness

FRAME	DEFLECTION	STIFFNESS	D.F.	FORCE TO FRAME (KIPS) ENGINEER OF RECORD	FORCE TO FRAME (KIPS) HAND CALCS
L	2.1328	0.4689	0.1322	9.56	5.99
P	2.7272	0.3667	0.1034	7.48	4.68
Q	0.9443	1.0590	0.2986	21.60	13.52
R	0.9443	1.0590	0.2986	21.60	13.52
S	1.7797	0.5619	0.1584	11.46	7.18
V	32.4701	0.0308	0.0087	0.63	0.39
		3.5462	1.0000		

	RGA	Hand
WIND=	72.32	45.29

7th Floor Transverse Stiffness

FRAME	DEFLECTION	STIFFNESS	D.F.	FORCE TO FRAME (KIPS) ENGINEER OF RECORD	FORCE TO FRAME (KIPS) HAND CALCS
L	2.4964	0.4006	0.1651	13.44	8.35
P	0	0.0000	0.0000	0.00	0.00
Q	0.9997	1.0003	0.4122	33.57	20.86
R	0.9997	1.0003	0.4122	33.57	20.86
S	0	0.0000	0.0000	0.00	0.00
V	39.4496	0.0253	0.0104	0.85	0.53
		2.4265	1.0000		

	RGA	Hand
WIND=	81.44	50.61

8th Floor Transverse Stiffness

FRAME	DEFLECTION	STIFFNESS	D.F.	FORCE TO FRAME (KIPS) ENGINEER OF RECORD	FORCE TO FRAME (KIPS) HAND CALCS
L	3.0653	0.3262	0.1278	12.46	7.80
Q	1.2298	0.8131	0.3184	31.05	19.45
R	1.2298	0.8131	0.3184	31.05	19.45
V	1.6637	0.6011	0.2354	22.95	14.38
		2.5536	1.0000		

	RGA	Hand
WIND=	97.51	61.09

9th Floor Transverse Stiffness

FRAME	DEFLECTION	STIFFNESS	D.F.	FORCE TO FRAME (KIPS) ENGINEER OF RECORD	FORCE TO FRAME (KIPS) HAND CALCS
L	4.1484	0.2411	0.1324	33.11	6.95
Q	1.6949	0.5900	0.3240	81.05	17.02
R	1.6949	0.5900	0.3240	81.05	17.02
V	2.5003	0.4000	0.2196	54.94	11.54
		1.8210	1.0000		

	RGA	Hand
WIND=	250.15	52.53

Roof Transverse Stiffness

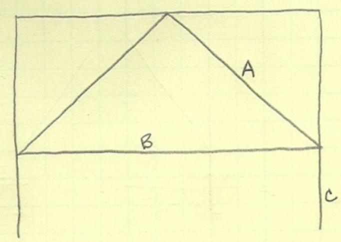
FRAME	DEFLECTION	STIFFNESS	D.F.	FORCE TO FRAME (KIPS) ENGINEER OF RECORD	FORCE TO FRAME (KIPS) HAND CALCS
L	4.4683	0.2238	0.1341	33.54	7.04
Q	1.8494	0.5407	0.3239	81.03	17.02
R	1.8494	0.5407	0.3239	81.03	17.02
V	2.7476	0.3640	0.2180	54.54	11.45
		1.6692	1.0000		

	RGA	Hand
WIND=	250.15	52.53

Member Checks

Lateral Frame Member Check:

using less conservative wind analysis / loads



A: $P_D = 3.55^k$
$P_L = 2.9^k$
$P_W = 35.69^k$

B: $P_D = 6.7^k$
$P_L = 2.81^k$
$P_W = 16.74^k$

$M_D = 14.2$ ft·k
$M_L = 11.18$ ft·k

C: $P_D = 237.1^k$
$P_L = 193^k$
$P_W = 22.96^k$

$M_D = 25.4$ ft·k
$M_L = 23.5$ ft·k
$M_W = 27.6$ ft·k

$V_D = 1.88^k$
$V_L = 2.58^k$
$V_W = 4.52^k$

from RAM member forces

Design of Braced Frame A:

$P_u = 1.2D + 1.6W + 0.5L = 1.2(3.55) + 1.6(35.69) + 0.5(2.9) = 62.8^k$

$KL = 18.5$ ft.

Try HSS = $4\tfrac{1}{2} \times 4\tfrac{1}{2} \times 3/8$
$\phi P_n = 66.4^k$ for $KL = 19'$

Manual Table 4-4

Check Buckling:

$$\frac{KL_x}{r_x} = \frac{18.5(12'')}{1.67''} = 133$$

$$\frac{KL_y}{r_y} = \frac{KL_x}{r_x} = 133 < 200 \quad \text{OK} \checkmark$$

$\boxed{\text{use HSS } 4\tfrac{1}{2} \times 4\tfrac{1}{2} \times 3/8}$

original = HSS $8 \times 6 \times 3/8$

Design of Beam B:

$P_u = 1.2D + 1.6W + 0.5L = 1.2(6.7) + 1.6(16.74) + 0.5(2.81) = 36.2^k$

$M_u = 1.2D + 0.5L = 1.2(14.2) + 0.5(11.18) = 22.6$ ft·K

Try 18×55 → smallest member I would use for moment frame

$p = 2.88 \times 10^{-3}$
$b_x = 2.89 \times 10^{-3}$ 〉Table 6-1 $KL = 13.5$ ft (use 14')

$pP_u = 2.88 \times 10^{-3}(36.2) = 0.104 < 0.2 \therefore$ H1-1b

$\frac{1}{2} p P_u + \frac{9}{8} b_x M_u \leq 1.0$

$\frac{1}{2}(2.88 \times 10^{-3})(36.2) + \frac{9}{8}(2.89 \times 10^{-3})(22.6) = 0.126 << 1.0$

A smaller size could be used because the beam is only 12% stressed; however, based on my engineering judgement & knowledge a W18×55 should be the smallest size used for a moment frame member.

$\boxed{\text{use W18×55}}$

original = W18×35

Design of Column C:

$P_u = 1.2(237.1) + 1.6(22.96) + 0.5(193) = 418^k$

$V_u = 1.2(1.88) + 1.6(4.52) + 0.5(2.58) = 10.8^k$

$M_u = 1.2(25.4) + 1.6(27.6) + 0.5(23.5) = 86.4$ ft·K

$L_b = 10$

Try W12×170 (guess)

$p = 0.492 \times 10^{-3}$
$b_x = 0.862 \times 10^{-3}$ 〉Table 6-1

$pP_u = 0.492 \times 10^{-3}(418) = 0.206 > 0.2 \therefore$ H1-1a

$0.206 + b_x M_u = 0.206 + 0.862 \times 10^{-3}(86.4) = 0.28 < 1.0$

$\phi V_n = 404^k >> V_u$

Try smaller size:

Try W12x58

$p = 1.54 \times 10^{-3}$
$b_x = 2.80 \times 10^{-3}$

$pP_u = 1.54 \times 10^{-3} (418) = 0.644 > 0.2 \therefore$ H1-1a

$pP_u + b_x M_u = 0.644 + 2.8 \times 10^{-3} (86.4) = 0.89 < 1.0$ OK ✓

Shear: $\phi V_n = 132^k >> V_u$ OK ✓

$\boxed{\text{use W12x58}}$

original = W14x130