Crossroads at Westfields

Building II

Chantilly, Va

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Structural option

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Technical Report 3

EXECUTIVE SUMMARY

The following report is a detailed examination of the lateral system of the Crossroads at Westfields Building II. This five story building resists lateral forces through four moment frames positioned in each direction. The building was modeled using RAM Structural for an overall 3D model and SAP 2000 to model the frames individually. The lateral loads from wind and seismic were compared from the first Technical Report and the output from RAM which is based off of over 300 load combinations according to code. Both techniques of analysis verified that wind controlled in the North-South direction and seismic controlled in the East-West direction. The Hand calculations computed in Tech Report I were very conservative due to certain assumptions made while the output from RAM was much more accurate due all the possible load cases used and a more precise modeling of the building. The rest of the analysis throughout the report was conducted using the output from RAM for this reason.

Using the SAP, the frames resisting load in each direction were modeled in the same plane. Each floor was constrained by connecting them with a rigid diaphragm so all of the floors displaced the same distance. This was done to find the relative stiffness of each frame to easily show the load distribution throughout the building. A torsion analysis was completed and the results were coupled with the direct shear. The torsional and direct shears were then distributed accordingly to find the total shear at each respective level of the frame. Strength and serviceability checks were performed to validate member size and confirm that the members were within acceptable code limits. Spot checks revealed that drift governed member design, although the seismic drift did not meet the drift criteria and should be analyzed further. While the building meets code provisions for strength and drift due to wind, a different lateral framing system may be required to meet seismic loading. Also, the foundations will be further analyzed after failing in the computer model.

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OVERALL INTRODUCTION

The Crossroads at Westfields are two identical office buildings mirroring each other on site. Although the project is currently on hold, these two buildings will offer over 300,000 GSF of office space to future tenants. Located in the Westfields Corporate Center in Chantilly, Virginia, the site is located at the crossing of the Stonecroft Blvd. and Lee Rd., hence the name.

Building II, identical to Building I, is a 5-story office building with floor plans that offer spans of over 41 feet. The large open floor plan creates long spans that require the beams to be cambered to pass deflection criteria. The structure consists of composite steel beam framing with ordinary moment frames to resist lateral loading. The roof is supported by joists and steel decking, and the future mechanical units will have composite slab pads similar to each floor.

Typical Floor Plan

This report will describe in-depth the overall lateral system designed to resist seismic and wind loads. Through computer modeling and hand calculations, analysis will be conducted to verify controlling load cases and combinations and to see how the loads are distributed through the buildings lateral resisting system. Checks for strength and serviceability will be conducted to verify the design of the lateral system meets certain code criteria. Some checks will include overall strength to certain members, story drift, overall building drift, overturning moments and the impact they may have on foundations. A torsion analysis will also be conducted to see if there are issues on the building.

EXISTING STRUCTURAL SYSTEMS

FOUNDATION SYSTEMS

The Foundation system consists of reinforced cast-in-place concrete spread footings. According to the Geotechnical report recommendations prepared by ECS, Ltd the allowable soil bearing values vary throughout the site. Foundations bearing on the natural 'weathered rock' soil classification will be designed with an allowable soil bearing of 6000 psf while foundations bearing on engineered fill will be designed for soil bearing of 3000 psf. The concrete strength shall be 3000 psi.

According to recommendations in the Geotechnical Report, the Slab on Grade will bear on the natural soil. The slab is a 4" thick cast-in-place concrete with 6x6–10/10 welded wire mesh (WWM), laid on a 6-mil fiberglass reinforced polyethylene vapor barrier and 4" of washed gravel. Interior SOG will have a compressive strength of 3000 psi, while exterior SOG will have a strength of 4500 psi.

FLOOR SYSTEMS

A typical floor in the Building II consists of 3" 20 gauge composite steel deck with 3-1/4" lightweight concrete slab totaling a total slab thickness of 6-1/4". The slab shall be reinforced with 6X6-10/10 WWM and have a compressive strength of 3000 psi. The floor is supported by A992 wide flange beams with studs dimensioned at ¾" in diameter and 5 ¼" in length. The beams are spaced at 10' o/c and span 41'-8" in a typical exterior bay and 30'-0" in a typical interior bay, as you can see in Figure 2 below. Depending on the floor, the beams will be cambered from an 1" to 1½" and will vary in size and weight. Typical interior girders are W24-62 spanning 30'-0", while typical exterior girders vary in size and also span 30'-0".

ROOF SYSTEM

As seen in Figure 3, the roof system is comprised of 1-1/2" 22 gauge Type B wide rib galvanized roof deck, on K series bar joists and steel girders. Light-gage framing makes up the 4' parapet and the screen wall encompassing the roof. Precast panels frame into each floor including the roof.

Rooftop Mechanical pads for future tenant equipment shall be constructed similar to the typical floor system consisting of 3" 20 gauge composite steel deck with 3-1/4" lightweight concrete slab totaling a total slab thickness of 6-1/4". The slab shall be reinforced with 6X6-10/10 WWM and have a compressive strength of 3000 psi.

COLUMN SYSTEM

Having a very uniform design layout the column system consists of typical exterior bays of 30'-0" x 41'-8" and interior bays of 30'-0" x 30'-0". All of the columns consist of either a gravity resisting member or a combined lateral and gravity resisting member. Each columns is spliced at 4 feet past the third floor, regardless of its resisting system. All columns vary in size depending on location and load resistance capabilities.

LATERAL SYSTEM

The lateral resisting system for wind and seismic loads consists of a number of structural steel moment frames running in both directions. Lateral loading is transferred from precast panels (connected at each floor) to each individual floor. Once transferred into the floor system, the load is transferred into composite beams which make up the framing and then into the columns. The columns and beams are connected by a moment connection seen in Figure 1 the columns transfer the rest of the load into the foundation.

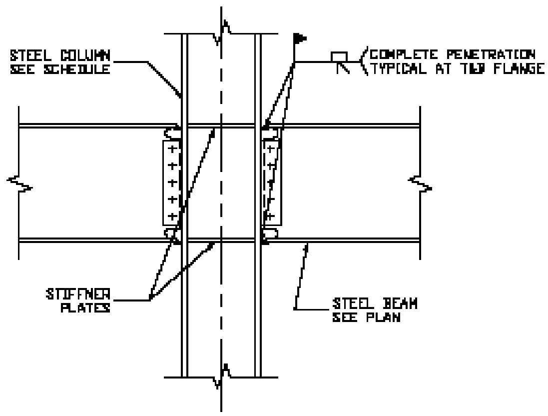


FIGURE 1 – Typical Beam to Column Moment connection

Figure 2 clearly shows the four moment frames positioned in each direction, North-South and East-West, supporting the building laterally. In both directions the moment frames are positioned symmetrically about the center axis. The North-South lateral system is 2 sets of parallel moment frames anchoring each end bay. The East-West lateral system is a set of 2 moment frames on each exterior side of the building. The beam sizes vary. An elevation view of each frame can be found in the Appendix.

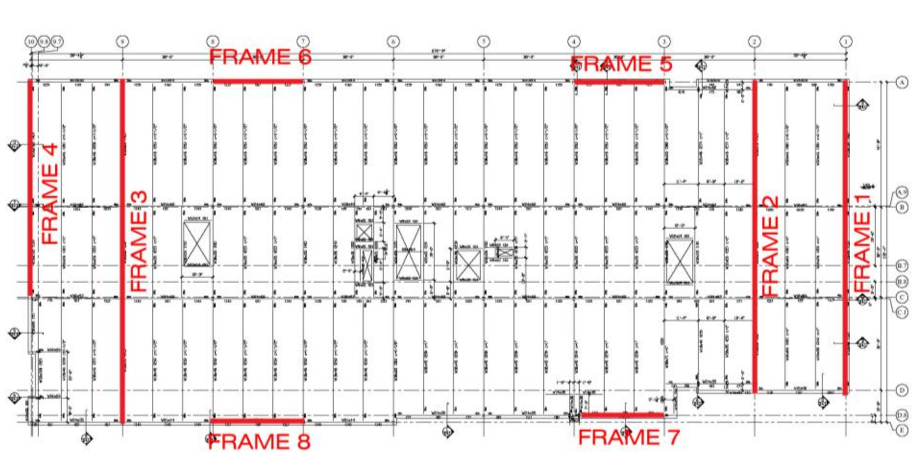


FIGURE 2 – Typical Floor plan with moment frames

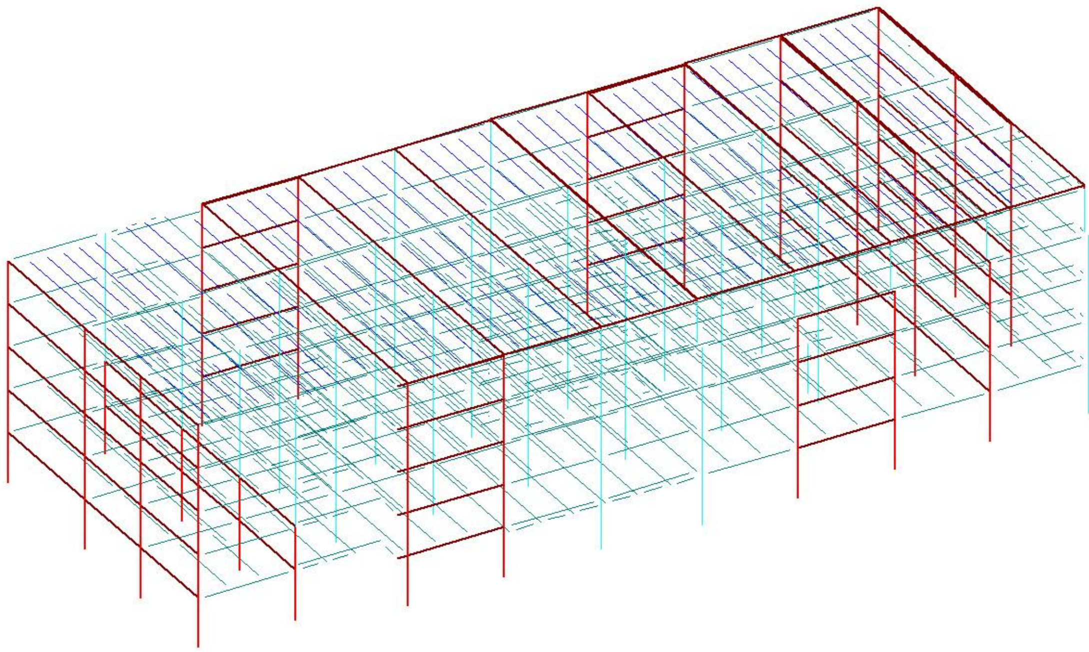


FIGURE 3 – Overall 3D RAM Model with highlighted moment frames

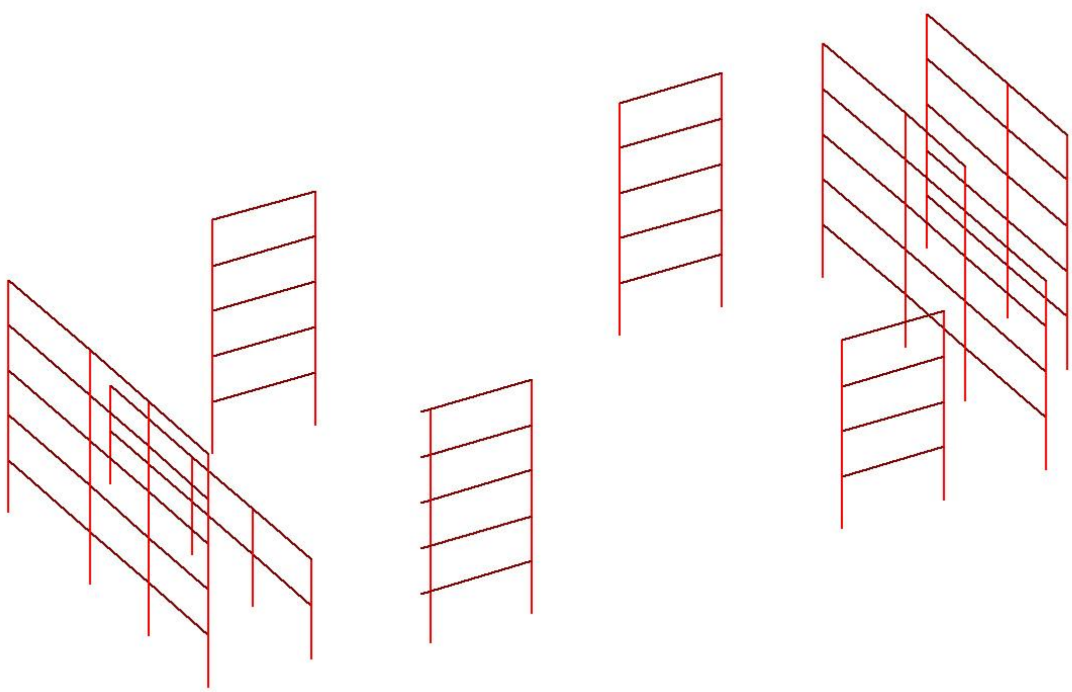


FIGURE 4 – 3D RAM Model with only moment frames

APPLICABLE CODE

Design Codes used for Original Design:

- International Building Code, 2003 Edition
- Viginina Uniform State Building Code, 2003
- American Society of Civil Engineers (ASCE)
  - ASCE 7 – 02, Minimum Design Loads for Buildings and Other Structures
- American Institute of Steel Construction (AISC)
  - Steel Construction Manual, Ninth Edition (LRFD)
- American Concrete Institute (ACI)
  - Building Code Commentary 318-02

Code Substitutions/ Additional References used for Thesis Design:

- International Building Code, 2006 Edition
- American Society of Civil Engineers (ASCE)
  - ASCE 7 – 05, Minimum Design Loads for Buildings and Other Structures
- American Institute of Steel Construction (AISC)
  - Steel Construction Manual, Thirteenth Edition (LRFD)
- American Concrete Institute (ACI)
  - Building Code Commentary 318-08

Load Cases and Combinations per IBC 2006/ ASCE 7-05

1.) 1.4D
2.) 1.2D + 1.6L+ 0.5(Lr or S or R)
3.) 1.2D + 1.6(Lr or S or R) + (L or 0.8W)
4.) 1.2D + 1.6W + L + 0.5(Lr or S or R)
5.) 1.2D + 1.0E + L + 0.2S
6.) 0.9D + 1.6W + 1.6H
7.) 0.9D + 1.0E + 1.6H

These are just a few of the 313 total load combinations generated by RAM for LRFD Design. Different load cases were added to the wind and seismic lateral loads and depending on the direction and eccentricity several combinations controlled.

Deflection Criteria per ASCE 7-05

$\Delta_{wind} = H/400$

$\Delta_{seismic} = .02H_{sx}$

MATERIALS AND PROPERTIES

Steel:	
Wide flange shapes	50 ksi (A992)
Square or Rectangular Tubes	46 ksi (A500 Grade B)
Round Pipes	42 ksi (A500 Grade B)
Miscellaneous Steel	36 ksi (A36)
Bolts	36/45 ksi (A325N/A490N)
Steel Studs	60 ksi (A108)
Weld Strength	70 ksi (E70XX)
Concrete:	
Foundations, Int. Wall & Int. SOG	f'c = 3000 psi
Ext. SOG and Pads	f'c = 4000 psi
Deck supported slabs (lightweight)	f'c = 3000 psi
Reinforcement:	
Stirrups and Ties	40 ksi (A615)
All other	60 ksi (A615)
Welded Wire Fabric:	70 ksi (A185)
Cold-Formed Steel Framing:	
20 Gage	33 ksi (A653)
18 Gage	33 ksi (A653)
16 Gage	50 ksi (A653)

Note: Material strengths are based on American Society for Testing and Materials (ASTM) Standard ratings.

DESIGN LOADS

Gravity Loads

The Design loads for were calculated in Technical Report I and were calculated referencing ASCE 7-05: *Minimum Design Loads for Buildings and other structures.* The actual design loads referenced IBC 2003 and there wasn't much discrepancy other than calculating the dead load per floor, as seen in Table 1 below. Live loads and Snow loads were calculated the using the same references therefore resulting in very similar results compared to the design.

Design Loads					
Live Loads					
Area	Actual Design		Thesis Design		Code/Table
Lobby	100	psf	100	psf	100 (ACSE Min.)
Office	100	psf	100	psf	50 (ASCE Min.)
Corridors	100	psf	100	psf	80 (ASCE Min.)
Roof	20	psf	20	psf	20 (ASCE Min.)

Dead Loads					
Area	Actual Design		Thesis Design		Code/Table
Floor	79.3	psf	90.0	psf	Table 1a (Appendeix)
Roof	28.5	psf	30.0	psf	Table 1b (Appendix)

Snow Loads					
Value	Actual Design		Thesis Design		Code/Table
Pg	25.0	psf	25.0	psf	ASCE 7-05 Chapter 7
Ce	1.0		1.0		
Ct	1.0		1.0		
Cs	1.0		1.0		
I	1.0		1.0		
Pf calculated	17.5	psf	17.5	Psf	
Pf	20.0	psf	20.0	Psf	

TABLE 1 – Design Loads

Lateral Loads

Lateral loads were calculated in Technical Report I using ASCE 7-05 and were compared to the actual design results. A comparison of the loads calculated in the first Tech Report will be compared to the results from computer modeling output in this report, as seen on the following page. The building was modeled using RAM Structural for an overall 3D model and SAP 2000 to model the frames individually. From the results, the controlling load combination will be determined and the design check will be conducted to verify the design of the lateral system meets certain code criteria. These checks will include overall strength to certain members, story drift, overall building drift, overturning moments and the impact they may have on foundations. A torsion analysis will also be conducted to see if there are issues on the building.

Wind Analysis

Basic Wind Speed ***V***.. 90 mph
Exposure Category... C
Importance Factor.. 1.0
Building Category... II
Internal Pressure Coefficient ***GCpi***.............................. +/- 0.18

Seismic Analysis

Seismic Use Group.. II
Importance Factor... 1.0
Spectral Response Accelerations Ss........................... 0.183
S1... 0.064
Site Class..C
Site Class Factors Fa... 1.2
Fv.. 1.7
SMS.. 0.220
SM1.. 0.109
SDS.. 0.146
SD1.. 0.073
Seismic Design Category... A
Response Modification Factor..................................... 3.0
Seismic Period Coefficient (Ct)..................................... 0.028
Seismic Period Coefficient (Cs).................................... 0.03
Period Coefficient (x).. 0.8

CONTROLLING LOAD CASES

Controlling NS Direction - Wind							
		RAM Output			Hand Results		
Height (ft)	Level	Force (K)	Story Shear	Moment (ft*K)	Factored Force	Story Shear	Moment
76.5	PP	52.60	52.60	4024			
68	Roof	41.32	93.92	2810	50.1	50.1	3407
54	5	68.90	162.82	3721	95.8	145.9	5173
40.75	4	68.94	231.76	2809	90.7	236.6	3696
27.5	3	61.88	293.64	1702	85.8	322.4	2360
14.25	2	57.41	351.05	818	80.8	403.2	1151
	Base Shear	351.05	O.M.	15883	403.2		15787

Controlling EW Direction - Seismic							
		RAM Output			Hand Results		
Height (ft)	Level	Force (K)	Story Shear	Moment (ft*K)	Factored Force	Story Shear	Moment
76.5	PP	0.58	0.58	44			
68	Roof	56.31	56.89	3829	81.79	81.79	5561.72
54	5	80.10	136.99	4325	144.17	144.17	7785.18
40.75	4	61.10	198.09	2490	80.10	80.10	3264.075
27.5	3	37.74	235.83	1038	37.82	37.82	1040.05
14.25	2	5.14	240.97	73	10.11	10.11	144.0675
	Base Shear	240.97	O.M.	11800	353.99		17795

TABLE 2 – Controlling Load Cases

RAM vs. TECH REPORT 1 Results

The building was modeled using RAM Structural for an overall 3D model and SAP to model the frames individually. The lateral loads from wind and seismic were compared from Technical Report I and the output from RAM which is based off of over 300 load combinations according to code. Both techniques of analysis verified that *wind* controlled in the North-South direction and *seismic* controlled in the East-West direction. The Hand calculations were very conservative due to certain assumptions while the output from RAM was much more accurate due to all possible load cases used and a more accurate modeling of the building. This can be seen in the comparison of the overturning moment, the moments are very similar for wind because the RAM model took into account the 8.5' tall parapet screen wall basically adding another floor. The rest of the analysis throughout this report was conducted using the output from RAM for this reason.

DISTRIBUTION OF LATERAL LOADS

Distribution of lateral forces is based on frame relative stiffness. The building was modeled in RAM and SAP and the outputs from both were used to achieve relative stiffness. For coordinate references, the location of Column E-10 was used as the x-coordinate of 0 and y-coordinate of 0.

A determination of each frame's relative stiffness was completed using SAP 2000. The frames resisting load in each direction were modeled in the same plane. The floors were then constrained at each level with a rigid diaphragm so the floors displaced the same distance. For simplicity, a 1000 kip load was applied at the roof level. The stiffness of the specified floor was then determined by taking the inverse of the measured deflection. Since all of the frames don't extend the complete extents of the building, individual floor rigidities were computed. This was completed by multiplying the unit load by the stiffness. This procedure was repeated at each floor and for each frame. Using the rigidity of each frame at that level, a determination could be made as to how much story shear each frame experienced.

The lateral force system is controlled by wind in the North-South Direction and by seismic in the East-West direction. To compute torsion analysis the Center of Mass (COM), Center or Rigidity (COR) and Center of Geometry (COG) are needed (output can be found in Appendix). To simply these calculations and have the results more accurate the RAM model output was used. Eccentricities were computed using 5% of the buildings total width in each direction. This is a conservative approach done by RAM due to the fact that the eccentricities in the N-S direction are very small. The torisonal rigidity was then found using the rigidities of each floor along with the COR for seismic loads and COG for wind loads.

Frame Rigidity (k/in)							
Load (K)	delta (in)	R (K/in)	Level	Frame 1	Frame 2	Frame 3	Frame 4
1000	9.0876	110.04	Roof	50.84	0.00	21.59	37.62
1000	6.164	162.23	5.00	47.75	0.00	45.08	69.40
1000	4.1141	243.07	4.00	103.66	0.00	48.49	90.91
1000	2.2682	440.88	3.00	148.90	78.42	55.62	157.94
1000	0.9614	1040.15	2.00	339.87	168.74	201.18	330.36
		1996.37	Total	691.01	247.17	371.96	686.23
	Frame Relative Stiffness			34.61%	12.38%	18.63%	34.37%

TABLE 3 – Frame Rigidity and Relative Stiffness N-S Direction

Frame Rigidity (k/in)							
Load (K)	delta (in)	R	Level	Frame 5	Frame 6	Frame 7	Frame 8
1000	15.9267	62.79	Roof	21.40	17.71	23.68	0.00
1000	11.0279	90.68	5	21.18	14.09	21.25	34.16
1000	7.5122	133.12	4	32.30	34.09	34.46	32.27
1000	4.3942	227.57	3	57.44	57.26	55.86	57.02
1000	1.7606	567.99	2	141.64	141.96	142.16	142.22
		1082.14		273.96	265.12	277.40	265.66
	Frame Relative Stiffness			25.32%	24.50%	25.63%	24.55%

TABLE 4– Frame Rigidity and Relative Stiffness E-W Direction

Direct Shear ($V*R_i$ / ΣR)										
Level	Controlling Wind NS - RAM Output					Controlling Seismic EW - RAM Output				
	V (k)	Frame 1	Frame 2	Frame 3	Frame 4	V (k)	Frame 5	Frame 6	Frame 7	Frame 8
roof	41.32	19.09	0.00	8.11	14.13	56.31	19.19	15.88	21.24	0.00
5	68.9	20.28	0.00	19.14	29.47	80.1	18.71	12.45	18.77	30.17
4	68.94	29.40	0.00	13.75	25.79	61.1	14.83	15.65	15.82	14.81
3	61.88	20.90	11.01	7.81	22.17	37.74	9.53	9.50	9.26	9.46
2	57.41	18.76	9.31	11.10	18.23	5.14	1.28	1.28	1.29	1.29
BASE	298.45	108.43	20.32	59.91	109.79	240.39	63.53	54.76	66.37	55.72

Torsional Shear $V*e*R_i*C$ / $\Sigma R*C^2$										
Level	Controlling Wind NS - RAM Output					Controlling Seismic EW - RAM Output				
	V (k)	Frame 1	Frame 2	Frame 3	Frame 4	V (k)	Frame 5	Frame 6	Frame 7	Frame 8
roof	41.32	2.09	0.00	0.66	1.49	56.31	1.88	1.56	2.27	0.00
5	68.9	2.33	0.00	1.64	3.27	80.1	1.78	1.18	1.76	2.83
4	68.94	3.23	0.00	1.13	2.74	61.1	1.87	1.97	1.99	1.87
3	61.88	2.39	0.99	0.67	2.45	37.74	1.38	1.37	1.37	1.40
2	57.41	2.20	0.86	0.98	2.07	5.14	0.22	0.22	0.23	0.23
BASE	298.45	12.25	1.84	5.08	12.02	240.39	7.12	6.30	7.62	6.32

Total Shear										
Level	Controlling Wind NS					Controlling Seismic EW				
	Total V (k)	Frame 1	Frame 2	Frame 3	Frame 4	Total V (k)	Frame 5	Frame 6	Frame 7	Frame 8
roof	41.25	21.18	0.00	7.44	12.63	55.14	17.31	14.32	23.51	0.00
5	66.32	22.61	0.00	17.50	26.20	81.73	16.93	11.27	20.53	33.00
4	68.30	32.63	0.00	12.62	23.05	61.12	12.96	13.67	17.81	16.68
3	62.14	23.29	11.99	7.14	19.71	37.76	8.15	8.12	10.64	10.86
2	57.42	20.96	10.17	10.13	16.17	5.16	1.06	1.06	1.52	1.52
BASE	295.43	120.68	22.16	54.83	97.76	240.91	56.41	48.45	73.99	62.05

TABLE 5 – Distributed Shear (Torsion and Direct)

TORSION IMPACT

As stated in the introduction torsion is determined by using an eccentricity between the center of mass and either the center of rigidity for seismic loads or the center of geometry for wind loads. The force is applied at the eccentricity off of the center of mass and a rotation or torsion is applied to the building. When modeling the building in RAM I used 5% of the total width of the building in each respective direction to estimate the eccentricity. This is a conservative approach because in the short direction (N-S) the eccentricity is very small. After solving the torsion shears applied to the frames it is resolved that torsion has little effect because of the symmetry of the frame layout.

OVER-TURNING MOMENT AND FOUNDATION IMPACT

Overturning Moments were calculated by multiplying the force at each floor by the respective story height in feet and summed for an overall moment. The controlling lateral forces, seismic for the East-West direction and wind for the North-South direction, produced moments of 11,800 ft*k and 15,883 ft*k, respectively. These moments are relatively low due to the height of the building only being 68'. Once modeled in RAM, the actual design for spread footing throughout the building did not resist the uplift force at the frame locations. This calls for further inspection and possibly a proposal in the spring semester.

Lateral Member Strength Checks:

LATERAL MEMBER CHECK - FRAME 1, LEVEL 3 BEAM

BEAM - W33 × 130 $K = 1.0$

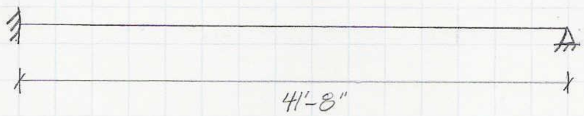
$L_u = 41.67$

$P_u = 0^K$

$M_{ux} = -540.86'K$

$M_{uy} = 0'K$

$V_u = -48.98$



* ALL FORCES & MOMENTS ARE RESULTS FROM RAM ANALYSIS FOR THE CONTROLLING LOAD CASE

SHEAR CHECK (CONTROLLING LOAD CASE: $1.2D + 1.6L_{point} + .5S_{now}$)

$\phi V_n = \phi(.6)F_y\, d\, t_w$ $\quad t_w = .58"$

$\phi V_n = (1.0)(.6)(50\text{ ksi})(33.1")(.58")$ $\quad d = 33.1"$

$\phi V_n = 575.94^K > V_u = 48.98^K \therefore$ OK ✓ $\quad \frac{48.98}{575.94} = 8.5\%$

FLEXURE CHECK (CONTROLLING LOAD CASE: $1.2D + .5L_P + 1.6W_{IND\,Y}$)

FROM TABLE 6-1: $p = 5.05 \times 10^{-3}\ K\cdot ft^{-1}$, $b_x = 1.97 \times 10^{-3}\ K\cdot ft^{-1}$

$b_y = 3.98 \times 10^{-3}\ K\cdot ft^{-1}$

$\frac{P_U}{\phi P_n} = \frac{0}{.9(219)} = 0 < .2 \therefore$ H1-1b GOVERNS

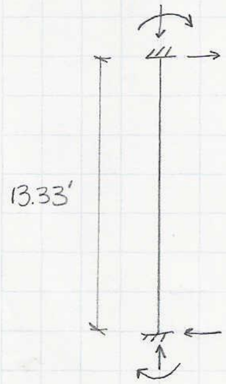
$$H1\text{-}1b \Rightarrow \frac{1}{2}\frac{P_u}{\phi P_n} + \left(\frac{M_{ux}}{\phi M_{nx}} + \frac{M_{uy}}{\phi M_{ny}}\right) \le 1.0$$

$$= 0 + \left((540.86/.9(1026)) + 0\right)$$

$= .586 \le 1.0 \therefore$ BEAM IS ADEQUATE ✓

LATERAL MEMBER CHECK - FRAME 1, LEVEL 3 COLUMN

COLUMN - W18×119 *OUTPUT FROM RAM ANALYSIS



$P_u = 404.3^K$
$V_u = 40.36^K$
$M_{ux} = -203.11'^K$
$M_{uy} = -.93'^K$

	X-AXIS	Y-AXIS
L_u:	13.33	13.33
K:	1.6	1.0

$\frac{KL}{r_y} = \frac{1.0(13.33)12''/_{1'}}{2.69} = 59.46 < 200 \therefore OK$

$P_u = 404.3^K$
$V_u = 40.36^K$
$M_{ux} = 223.56'^K$
$M_{uy} = .28'^K$

CHECK SHEAR

$\phi V_n = \phi_v(.5) F_y d t_w$ $\qquad t_w = .655''$
$\phi V_n = (1.0).5(50 ksi)(19'')(.655'')$ $\qquad d = 19''$

$\phi V_n = 311^K > V_u = 40.36^K \therefore OK$ ✓ $\qquad \frac{40.36}{311} = 13\%$

COMBINED LOADING CHECK

$B_1 = {C_m}/{1-\alpha {P_r}/{P_{e1}}} \geq 1.0$

$C_m = .6 - .4\left(\frac{-203.11}{223.56}\right) = .963 \qquad C_{my} = .6 - .4\left(\frac{-.93}{.28}\right) = 1.93$

$P_{ex} = \frac{\pi^2 (29000 ksi)(2190)}{(1.61(13.33)12)^2} = 9451^K \quad P_{ey} = \frac{\pi^2 (29000 ksi)(253)}{(1.0 \cdot 13.33 \cdot 12)^2} = 2830^K$

$$B_{1x} = \frac{.963}{1 - \frac{(1.0)404.3}{9451^K}} = 1.0 \geq 1.0 \qquad B_{1y} = \frac{1.93}{1 - \frac{(1.0)(404.3)}{2830^K}} = 2.25 \geq 1$$

$$\therefore B_{1x} = 1.0 \qquad \therefore B_{1y} = 2.25$$

$$B_2 = \frac{1}{1 - \frac{\alpha P_{nt}}{\sum P_{e_2}}} \geq 1.0 \qquad \sum P_{e_2} = R_m \frac{\sum HL}{\Delta_H} \qquad R_m = 1.0$$

$$\Delta H_y = .00007'' \qquad \Delta H_x = .781''$$

$$\sum P_{e_{2x}} = \frac{1.0\,(100.1^K)(13.33 \cdot 12)}{.781''} = 20502^K$$

$$\sum P_{e_{2y}} = \frac{1.0\,(.02^K)(13.33 \cdot 12)}{.00007''} = 45703^K$$

$$B_{2x} = \frac{1}{1 - \frac{(404.3)}{20502^K}} = 1.02 \qquad B_{2y} = \frac{1}{1 - \frac{(404.3)}{45703}} = 1.01$$

$$P_r = (1.02)\,404.3^K = 412.4^K \qquad p = .822\,E^{-3}\ K \cdot ft^{-1}$$

$$M_{rx} = 1.02\,(223.56) = 228'^K \qquad b_x = .961\,E^{-3}\ K \cdot ft^{-1}$$

$$M_{ry} = 2.25\,(.93) + 1.01\,(.93) = 3.03'^K \qquad b_y = 3.43\,E^{-3}\ K \cdot ft^{-1}$$

TABLE 6-1

$$pP_r = (.822E^{-3})(412.4) = .339 > .2 \quad \therefore H1-1a$$

$$H1-1a \Rightarrow pP_r + b_x M_{rx} + b_y M_{ry} = .339 + .961E^{-3}(228) + 3.43E^{-3}(3.03)$$

$$= .568 < 1.0 \quad \therefore$$ COLUMN IS ADEQUATE

DRIFT CRITERIA

Criteria for seismic and wind drift were compared with the drift values determined by RAM Frame. Wind drift was compared against Δ_{WIND} = H/400 for the entire building and seismic drift was compared against $\Delta_{SEISMIC}$ = $0.02h_{SX}$ at each floor as seen in Table 6 below.

Controlling Wind Drift - NS Direction							
Level	Story height (Ft)	Story Drift (in)	Allowable drift (h/400)		Total Drift (in)	Allowable total (H/400)	
roof	68	0.299	0.420	ok	1.851	2.040	ok
5	54	0.347	0.398	ok	1.552	1.620	ok
4	40.75	0.424	0.398	not	1.205	1.223	ok
3	27.5	0.401	0.398	not	0.781	0.825	ok
2	14.25	0.380	0.428	ok	0.380	0.428	ok

Controlling Seismic Drift - EW Direction							
Level	Story Height (ft)	Strory Drift (in)	Allowable drift (.020h)		Total Drift (in)	Allowable total (.02h)	
roof	68	0.349	0.280	ok	2.398	1.360	not
5	54	0.458	0.265	not	2.049	1.080	not
4	40.75	0.579	0.265	not	1.591	0.815	not
3	27.5	0.582	0.265	not	1.012	0.550	not
2	14.25	0.430	0.285	not	0.430	0.285	not

TABLE 6 – Drift criteria per ASCE 7-05

Clearly from the charts the Total Drift in the N-S direction, which is controlled by wind is acceptable. However, the overall drift for seismic fails at every level and overall. There can be many reasons for this, including a possible modeling error. Further investigation of this problem will be analyzed in the spring semester.

CONCLUSION

This report used two computer models along with hand calculations to analyze the lateral systems of the Crossroads at Westfields building II. The results confirm that the existing system works with a few discrepancies. SAP and RAM both produced very similar relative frame stiffness verifying that the existing load distribution is accurate. The controlling load cases between Tech Repot I and the RAM model clearly show that the assumptions used for Tech Report I were very conservative and therefore the output from RAM was used throughout this report. Torsion was analyzed and was found not to be an issue. The two main problems occur with the overall drift of the building and effect the overturning moments will have on the foundation.

The overall drift of the building met the code requirements for the wind loads which control in the N-S direction. However, according to the RAM output the Seismic drifts failed at each level and for the overall drift of the building raising an issue. Another conclusion from the drift was that most of the lateral members were designed with drift being the controlling factor. This is seen in the member checks, as both the beam and column required only about 50% of their capacity. The fact that the members were oversized because of drift and the drift didn't meet the code requirements proves there is a flaw somewhere in the design and needs to be re-analyzed.

The second discrepancy occurred with the foundation design and effects the overturning moment had on it. After modeling the building, the spread foundation failed due to uplift forces caused by the overturning moment. One solution may be to add piles to resist this but regardless further analysis must be conducted.

APPENDIX

WIND PRESSURES AND FORCES

Lateral Forces E-W Direction, Width = 115'						
	Force	Factored Force	Story Shear	Factored Shear	Moment	Factored Moment
Floor	Fx, (k)	Fx * 1.6 (K)	V, (k)	V, (k)	M (ft-k)	M, (ft-k)
Roof	11.2	17.9	-	-	761.6	1218.56
5.0	21.5	34.4	11.2	17.9	1161.0	1857.6
4.0	20.3	32.5	32.7	52.3	827.2	1323.56
3.0	19	30.4	53.0	84.8	522.5	836
2.0	17.3	27.7	72.0	115.2	246.5	394.44
-	-	-	89.3	142.9	-	
Base Shear	89.3	142.88		Overturning Moment	3518.9	4411.6

Lateral Forces N-S Direction, Width = 275'						
	Force	Factored Force	Story Shear	Factored Shear	Moment	Factored Moment
Floor	Fx, (k)	Fx * 1.6 (K)	V, (k)	V, (k)	M (ft-k)	M, (ft-k)
Roof	31.3	50.1	-	-	2128.4	3405.44
5.0	59.9	95.8	31.3	50.1	3234.6	5175.36
4.0	56.7	90.7	91.2	145.9	2310.5	3696.84
3.0	53.6	85.8	147.9	236.6	1474.0	2358.4
2.0	50.5	80.8	201.5	322.4	719.6	1151.4
-	-	-	252.0	403.2	-	-
Base Shear	252	403.2		Overturning Moment	9867.2	15787.4

Design Wind Pressures E-W					
Floor	Height	q (psf)	windward q	leeward q	total pressure q
Roof	68.00	15.55	9.94	3.62	13.56
5	54.00	14.56	9.31	3.62	12.93
4	40.75	13.46	8.61	3.62	12.23
3	27.50	11.63	7.44	3.62	11.06
2	14.25	10.05	6.43	3.62	10.05

Design Wind Pressures N-S					
Floor	Height	q (psf)	windward q	leeward q	total pressure q
Roof	68.00	15.55	10.34	6.21	16.55
5	54.00	14.56	9.68	6.21	15.89
4	40.75	13.46	8.95	6.21	15.16
3	27.50	11.63	7.74	6.21	13.95
2	14.25	10.05	6.68	6.21	12.89

SEISMIC FORCES

Seismic Force Story Distribution						
Floor	w_x	h_x	k	$w_x h_x^k$	$\Sigma\ w_i h_i^k$	C_{vx}
Base	--	--	--	--	--	--
2	2760.10	14.25	2.00	560472.81	19622948.41	0.029
3	2771.10	27.50	2.00	2095644.38	19622948.41	0.107
4	2739.70	40.25	2.00	4438485.23	19622948.41	0.226
5	2739.70	54.00	2.00	7988965.20	19622948.41	0.407
Roof	981.70	68.00	2.00	4539380.80	19622948.41	0.231

Floor	F_x (Kips)	Story Shear Vx	Moment (k-ft)
Roof	81.92	-	5570.51
5	144.17	81.92	7785.27
4	80.10	226.09	3223.96
3	37.82	306.19	1040.02
2	10.11	344.01	144.13
-	-	354.12	-
Base	354.12	**Overtunring Moment (k-ft)**	17763.89

TORSION ANALYSIS

Torsion Rigidity (J) & C (COG) - Controlling Wind NS									
Level	Frame 1 (C)	R*C²	Frame 2 (C)	R*C²	Frame 3 (C)	R*C²	Frame 4 (C)	R*C²	J=ΣR*C²
roof	139.89	994829.7975	0	0	104.61	236216.45	135.11	686705.7	1917751.95
5	139.89	934507.662	0	0	104.61	493286.15	135.11	1266906.2	2694700.04
4	139.89	2028531.46	0	0	104.61	530681.94	135.11	1659594.4	4218807.82
3	139.89	2913785.21	109.39	938446.3	104.61	608667.6	135.11	2883086	7343985.10
2	139.89	6650927.77	109.39	2019200	104.61	2201581.6	135.11	6030605.6	16902315.13

Torsion Rigidity (J) & C (COR) - Controlling Seismic EW									
Level	Frame 5 (C)	R*C²	Frame 6 (C)	R*C²	Frame 7 (C)	R*C²	Frame 8 (C)	R*C²	J=ΣR*C²
Roof	55.04	64823.23	55.04	53638.94	59.96	85146.96	0.00	0.00	203609.12
5.00	57.88	70959.18	57.88	59317.16	57.12	77272.02	57.12	111440.17	318988.53
4.00	57.53	106906.74	57.53	58601.95	57.47	78221.89	57.47	106569.42	350299.99
3.00	56.79	185236.92	56.79	57104.07	58.21	80249.27	58.21	193197.34	515787.59
2.00	56.29	448810.93	56.29	56102.96	58.71	81633.81	58.71	490227.63	1076775.33

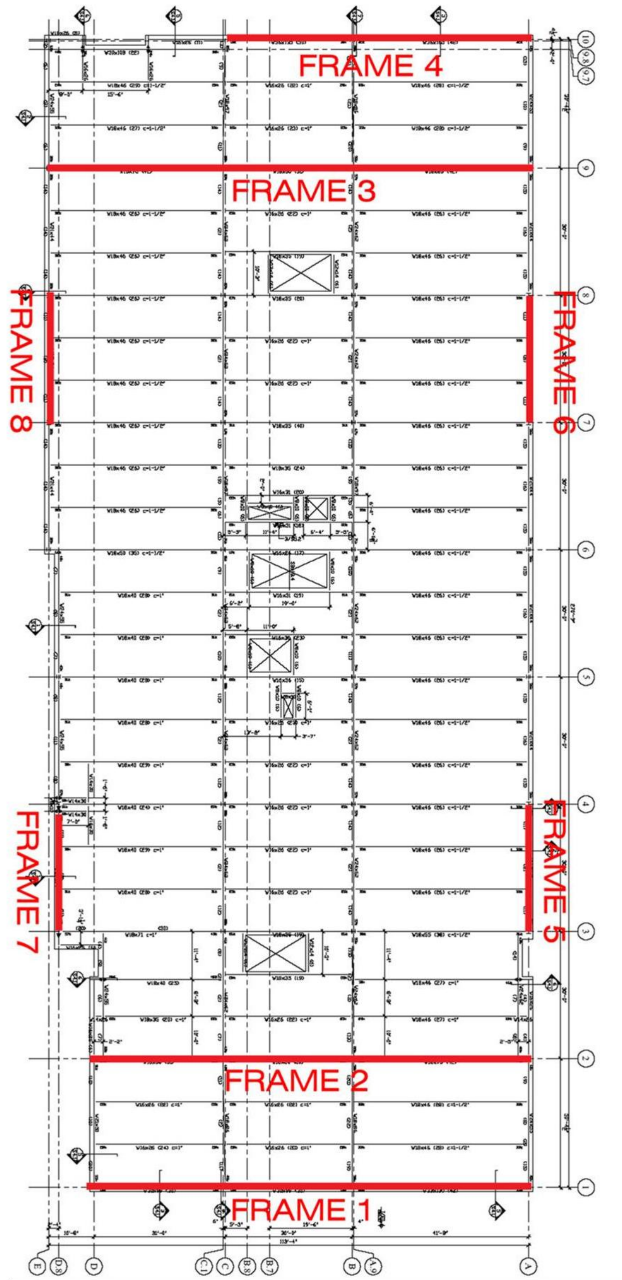
Center of Geometry - Hand Calculated					
Level	Σa	Σa*x	Σa*y	x (ft)	y (ft)
Roof	30879	4172020	1717514	135.11	55.62
5	30879	4172020	1717514	135.11	55.62
4	30879	4172020	1717514	135.11	55.62
3	30879	4172020	1717514	135.11	55.62
2	30879	4172020	1717514	135.11	55.62

Center of Rigidity - RAM Output		
Level	x (ft)	y (ft)
Roof	154.71	59.96
5	152.18	57.12
4	146.58	57.47
3	143	58.21
2	141.71	58.71

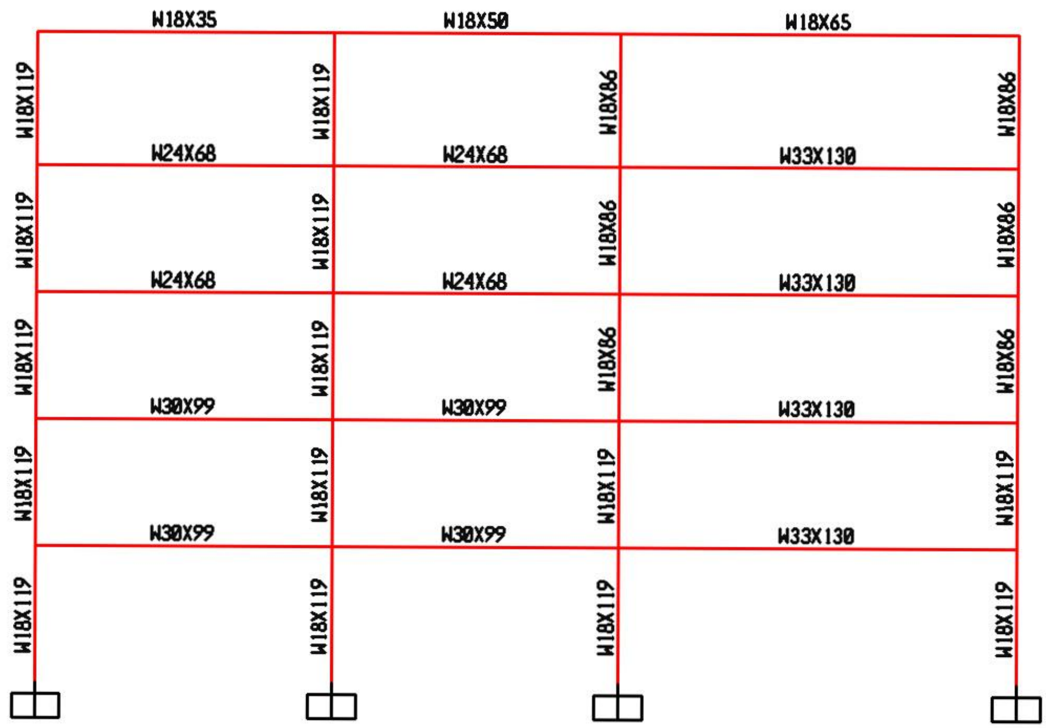
Center of Mass - RAM Output		
Level	x (ft)	y (ft)
Roof	136.19	61.75
5	135.47	59.83
4	135.2	59.67
3	135.21	59.57
2	135.24	59.29

Eccentricity, e				
	Actual		RAM Output	
Level	x	y	x (ft)	y (ft)
Roof	18.52	-1.79	13.64	5.77
5	16.71	-2.71	13.64	5.77
4	11.38	-2.2	13.64	5.77
3	7.79	-1.36	13.64	5.77
2	6.47	-0.58	13.64	5.77
			5% of 274'	5% of 115'

OVERALL LATERAL RESISTING SYSTEM



FRAME MEMBERS



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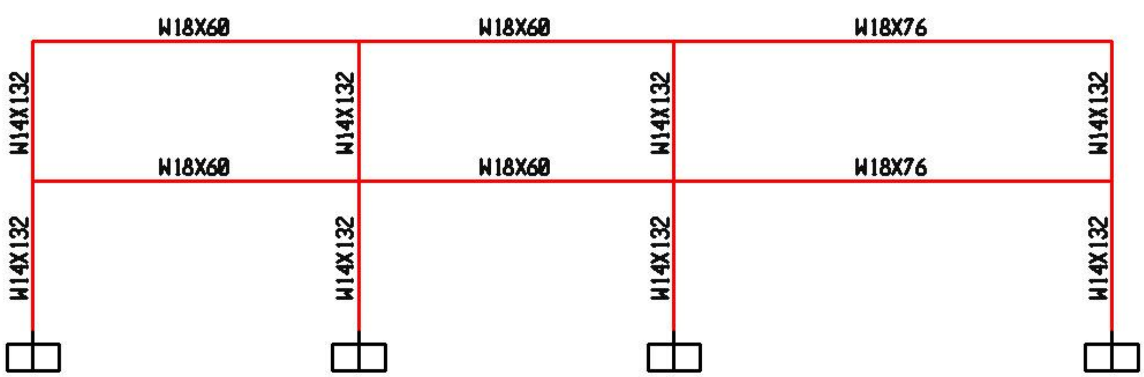

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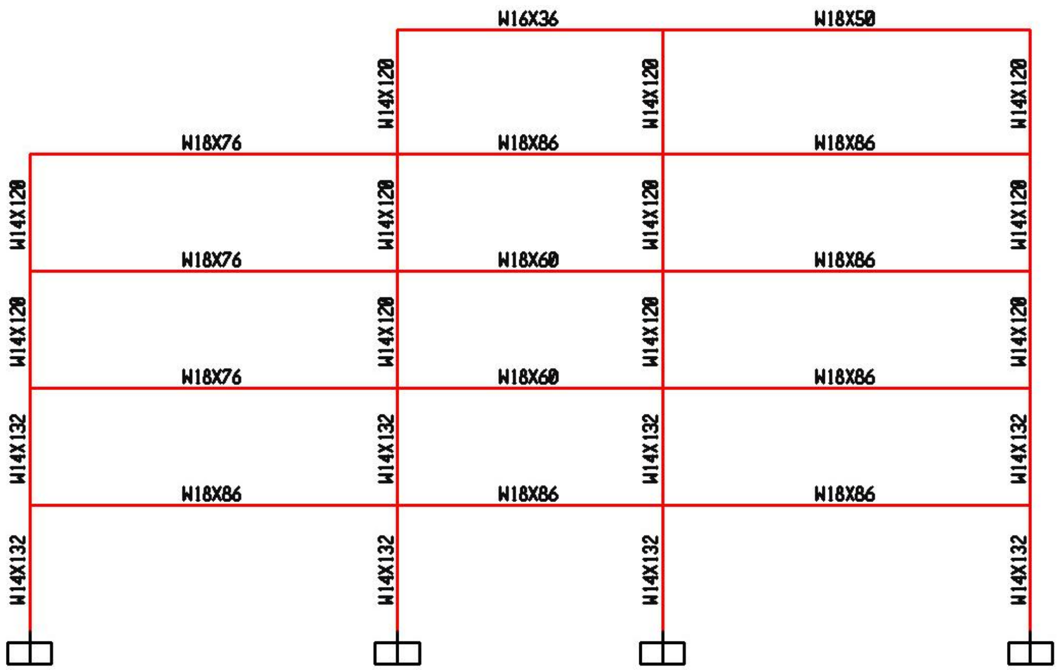

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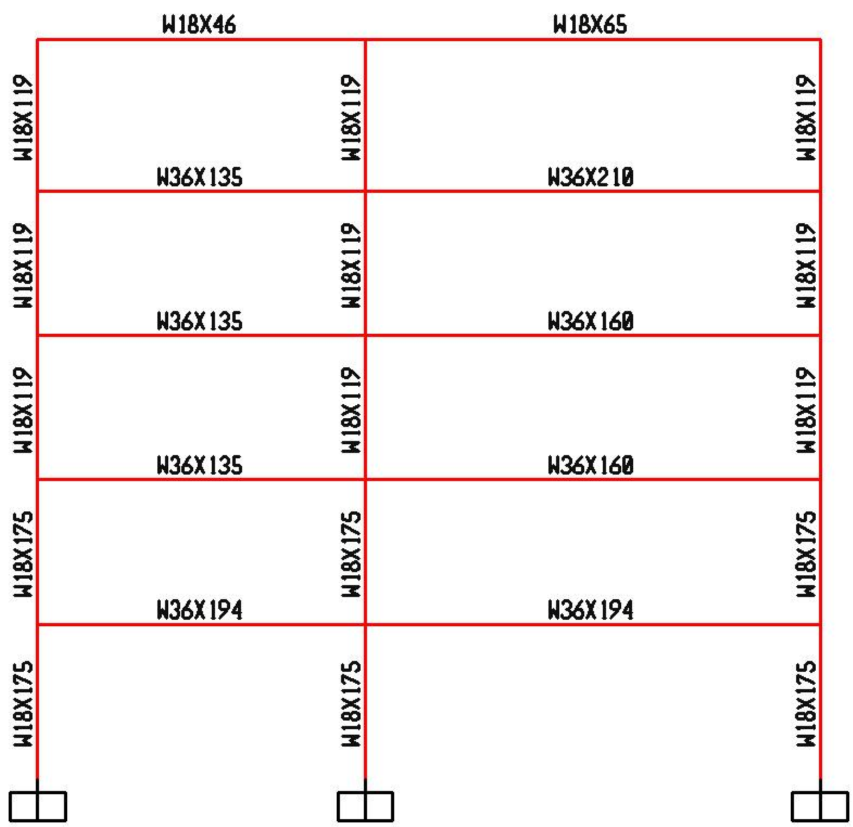

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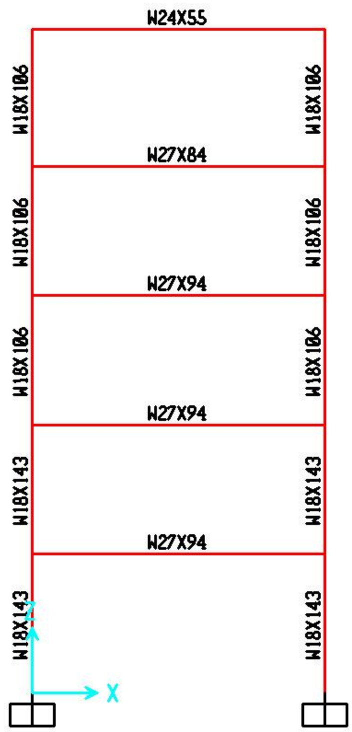

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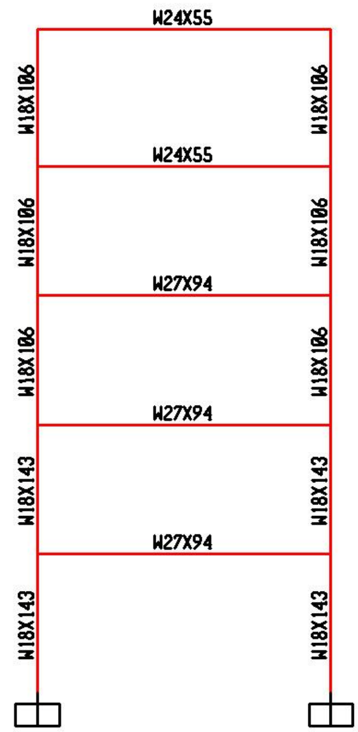

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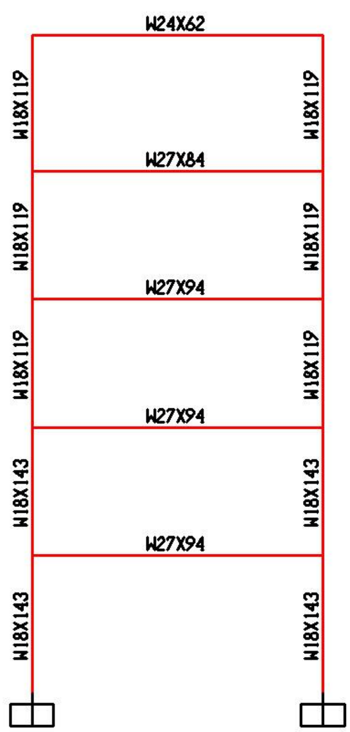

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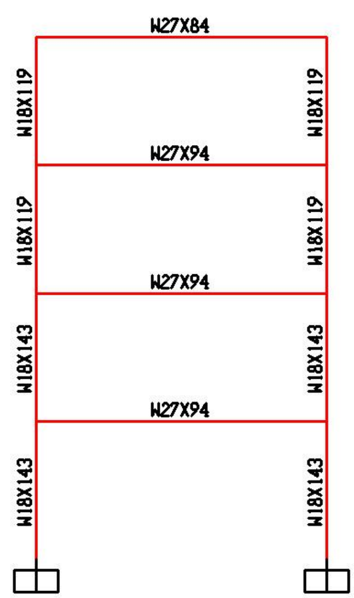

FRAME 8