UNIVERSITY OF ROCHESTER
BME / OPTICS BUILDING

TECHNICAL ASSIGNMENT 3

Lateral System Analysis and Confirmation of Design

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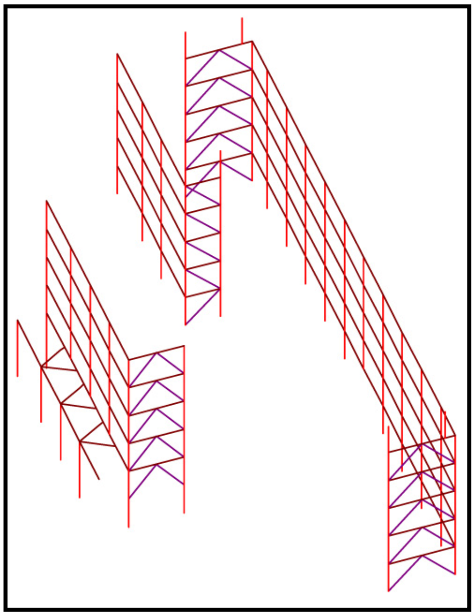
Executive Summary

The University of Rochester BME/Optics Building is a new laboratory and office facility for their highly regarded Optics Department, and up-and-coming Biomedical Engineering Program. It is five stories, with an additional mechanical penthouse, partial basement, and large atrium. The structure consists of composite steel framing, concentrically braced frames in the East-West direction, and moment-resisting frames in the North-South direction.

The purpose of this report is to describe the lateral system of the BME/Optics Building in detail, analyze the loads and their distribution, and confirm the lateral system design. It includes:

- Detailed description of all lateral resisting elements and the load path
- Determination of lateral loads using ASCE 7-02
- Approximate distribution of loads to each frame
- Analysis of torsional effects
- Confirmation of design using building model in RAM Structural System
- Spot checks of critical lateral members using RAM Advanse, Excel spreadsheets, and hand calculations
- Drift analysis



From these analyses, the University of Rochester BME / Optics Building was found to be well designed to resist lateral loads and meet the architectural needs and challenges of the building. The use of braced frames in the East-West direction is very efficient, while the use of moment frames in the North-South direction utilizes the long building dimension and allows for windows at exterior faces.

The elements in both directions are designed, through balance in geometry and stiffness, to resist torsional forces. The eccentricity between the centers of mass and rigidity is less than 5% of the building dimension in both directions, at all floor levels. This often proves to be difficult in buildings with an irregular-shaped footprint.

Through strength checks, the lateral system for the University of Rochester BME / Optics building was confirmed. By analyzing the lateral system and performing strength checks, the concepts used in the building's design could be understood. It provides a strong basis for exploring a structural redesign in the future.

Introduction / Scope

The purpose of this report is to describe the lateral system of the BME/Optics Building in detail, analyze the loads and their distribution, and confirm the lateral system design.

This report explores the lateral system of University of Rochester's BME/Optics Building with the intention of understanding the concepts used in its design. A previous report, titled *Technical Report #1: Existing Structural Conditions*, analyzed the loads of the building and provided a preliminary lateral system analysis and spot check. As a continuation, this report will describe the lateral system in greater detail, summarize and distribute the lateral loads found, and provide a more in-depth, accurate analysis of the lateral system design for the BME/Optics Building. This includes strength spot checks of critical members, drift analysis, and torsional effects. The purpose of these checks is to understand the concepts used in the building's design.

Background

The Institute of Optics at the University of Rochester was founded in 1929 as the first optics education program in the United States. Almost 80 years later, it remains a cutting edge program and one of the finest educational and research institutions in the country. The Institute of Optics, along with the Biomedical Engineering Department, are currently obtaining a new facility to cater to the increasing needs of these highly regarded programs. The facility, currently known as the BME/Optics Building, began construction in January of 2005, with a scheduled completion of December 2006.

Architecture

The BME/Optics Building is strategically located on the south end of the U of R River Campus, across the street from the Medical Center. It is built adjacent on two sides (with pedestrian access on two floors) to the current Biomedical and Optics facility, the Wilmot Building. Also, a second floor pedestrian bridge connects the new BME/O Building to the nearby CSB Building to provide access to computer lab and library services.

The façade of the building is primarily clay brick with limestone at the first floor level. Key architectural features of the building include channel glass façade at stairwells and an 80' atrium inside the main entrance to be lit by skylights. The 100,000 square foot structure is 5 stories above grade plus a mechanical penthouse and partial basement, and consists of laboratory, classroom, and office space.

Structure

The foundation system used in this building consists of concrete pile caps supported by 50 ksi steel H-piles bearing on bedrock. There are several different pile configurations, but each has a design lateral load capacity of 4 kips. The foundation system also uses concrete grade beams at different sections of the building. These include support of the exterior façade and framing around an existing steam/utility tunnel running under the footprint of the building. Since this tunnel supplies several campus buildings, its complete functionality throughout construction of the BME/Optics Building was an important design consideration.

The typical floor system consists of 4 ½" concrete slabs on 3" composite metal deck. The load is distributed from the slab to composite steel beams and girders, and finally down to steel columns and the foundation. Appendix A shows the column layout of a typical floor. Although the loads are relatively constant throughout the building, the steel shapes vary in size due to varying spans. This is because of the irregular shape of the building designed to meet architectural and spatial challenges.

Lateral forces due to wind and seismic loading were important design considerations for the BME/Optics Building. Since it was built adjacent to the existing Wilmot Hall on two sides, the lateral deflection was especially important. At these locations, the steel framing cantilevers out from the columns to form isolation joints. These joints increase in size from 8" at the first floor to 1'-6" at the fifth floor. Accuracy in lateral calculations was necessary to determine proper clearance.

The main lateral force resisting system designed for this building uses four concentric braced frames in the short (E-W) direction, and three ordinary moment frames in the long (N-S) direction. The lateral system will be described in detail and analyzed later in this report.

Gravity Loads

The structural design of the BME/Optics Building used the Building Code of New York State, which references IBC 2000 and ASCE 7-98. As an upgrade, the loads calculated for *Technical Report #1* used IBC 2003 and ASCE 7-02. For a more thorough listing of the design guides used and more detailed analysis of gravity loads, refer to *Technical Report #1.*

<u>Live Load</u>

Laboratory Space	80 psf
Office Space	80 psf
Main Lobby, Stairs	100 psf
Mechanical Room	150 psf (or equip. weight)
Flat Roof Snow	35 psf

<u>Dead Load</u> (Typical Floor)

7 ½" Floor Slab	94 psf
3" Steel Deck	3 psf
Framing	10 psf (or beam self-weight where applicable)
Flooring/Ceiling	2 psf
MEP Allowance	<u>10 psf</u>
` Total:	120 psf

Lateral Loads

Wind

The wind load for the BME/Optics Building was determined using ASCE 7-02 for a previous report titled *Technical Report #1: Existing Structural Conditions.* Tables 1 and 2 show the design wind pressures at various elevations for each direction of wind. These wind pressures, along with effective story forces are shown graphically in Figures 1 and 2. Story force calculations can be found in Appendix B.

The parameters for wind calculations are as follows:

Wind Load *(Analytical Method, ASCE 7-02)*

Basic Wind Speed	**V = 90 mph**
Importance Factor	**I = 1.15**
Exposure Category	**B**
Building Height	**h = 95’**
Building Classification	**Rigid, Enclosed**
Directionality Factor	**K_d = 0.85**
Gust Effect Factor	**G = 0.85** (approximated)
Internal Pressure Coeff.	**GC_{pi} = ± 0.18**
External Pressure Coeff.	**C_p = 0.8** Windward
	C_p = - 0.5 Leeward (E-W)
	C_p = - 0.3 Leeward (N-S)

Wind Pressures, East-West Direction						
Height (z)	**Kz**	**qz**	**qh**	**p (Windward)**	**p (Leeward)**	**p (Total)**
0-15	0.57	11.55	20.07	7.86	-8.53	**16.39**
20	0.62	12.57	20.07	8.55	-8.53	**17.08**
25	0.66	13.38	20.07	9.10	-8.53	**17.63**
30	0.7	14.19	20.07	9.65	-8.53	**18.18**
40	0.76	15.40	20.07	10.48	-8.53	**19.00**
50	0.81	16.42	20.07	11.16	-8.53	**19.69**
60	0.85	17.23	20.07	11.72	-8.53	**20.25**
70	0.89	18.04	20.07	12.27	-8.53	**20.80**
80	0.93	18.85	20.07	12.82	-8.53	**21.35**
90	0.96	19.46	20.07	13.23	-8.53	**21.76**
100	0.99	20.07	20.07	13.65	-8.53	**22.18**

Table 1: Design Wind Pressures, E-W – All pressures are in (lb / ft^2)

Wind Pressures, North-South Direction						
Height (z)	**K_z**	**qz**	**qh**	**p (Windward)**	**p (Leeward)**	**p (Total)**
0-15	0.57	11.55	20.07	7.86	-5.12	**12.97**
20	0.62	12.57	20.07	8.55	-5.12	**13.66**
25	0.66	13.38	20.07	9.10	-5.12	**14.21**
30	0.7	14.19	20.07	9.65	-5.12	**14.77**
40	0.76	15.40	20.07	10.48	-5.12	**15.59**
50	0.81	16.42	20.07	11.16	-5.12	**16.28**
60	0.85	17.23	20.07	11.72	-5.12	**16.83**
70	0.89	18.04	20.07	12.27	-5.12	**17.38**
80	0.93	18.85	20.07	12.82	-5.12	**17.94**
90	0.96	19.46	20.07	13.23	-5.12	**18.35**
100	0.99	20.07	20.07	13.65	-5.12	**18.76**

Table 2: Wind Pressures, N-S – All pressures are in (lb / ft^2)

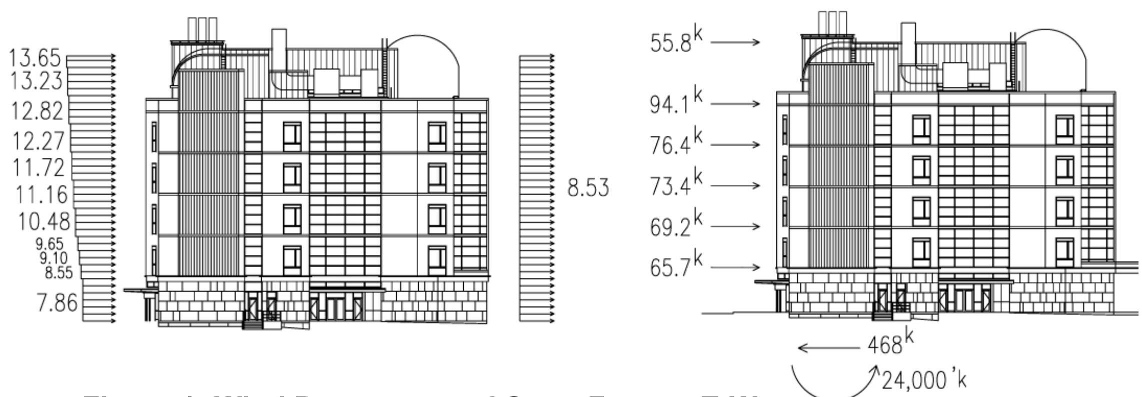


Figure 1: Wind Pressures and Story Forces, E-W

Figure 2: Wind Pressures and Story Forces, N-S

Seismic

Similarly, seismic calculations were performed with regards to IBC 2003 and ASCE 7-02 using the Equivalent Lateral Force Procedure as shown below. Tables 3 and 4 give the distribution of lateral forces by floor, and Figures 3 and 4 display the results graphically. See Appendix C for calculations.

Seismic Load *(Equivalent Lateral Force Procedure, ASCE 7-02)*

Seismic Use Group	**II**
Importance Factor	**I = 1.25**
Equivalent Seismic Weight	**W = 12,000 kips**
Spectral Response Acceleration	**S_{DS} = 0.267g**
	S_{D1} = 0.112g
Approximate Fundamental Period	**T_a = 0.61 sec**
	k = 1.06
North-South Direction:	**Ordinary Steel Moment Frames**
Response Modification Factor	**R = 3.5**
Seismic Design Coefficient	**C_s = 0.066**
Base Shear	**V = 800 kips**
East-West Direction:	**Concentric Braced Steel Frames**
Response Modification Factor	**R = 5**
Seismic Design Coefficient	**C_s = 0.046**
Base Shear	**V = 550 kips**

Floor	2	3	4	5	Penthouse	Roof
Weight, w (kips)	2430	2430	2430	2430	1700	450
Height, h (ft)	16	30.67	45.33	60	74.67	94.67
k	1.06	1.06	1.06	1.06	1.06	1.06
wh^k	45917	91521	138476	186400	164431	55975
Distribution Factor, Cvx	0.067	0.134	0.203	0.273	0.241	0.082
Base Shear, V (kips)	550	550	550	550	550	550
Story Force Fx (kips)	**37.0**	**73.7**	**111.6**	**150.2**	**132.5**	**45.1**

Table 3: Vertical Distribution of Seismic Forces, E-W

Floor	2	3	4	5	Penthouse	Roof
Weight, w (kips)	2430	2430	2430	2430	1700	450
Height, h (ft)	16	30.67	45.33	60	74.67	94.67
k	1.06	1.06	1.06	1.06	1.06	1.06
wh^k	45917	91521	138476	186400	164431	55975
Distribution Factor, Cvx	0.067	0.134	0.203	0.273	0.241	0.082
Base Shear, V (kips)	800	800	800	800	800	800
Story Force Fx (kips)	**53.8**	**107.2**	**162.3**	**218.4**	**192.7**	**65.6**

Table 4: Vertical Distribution of Seismic Forces, N-S

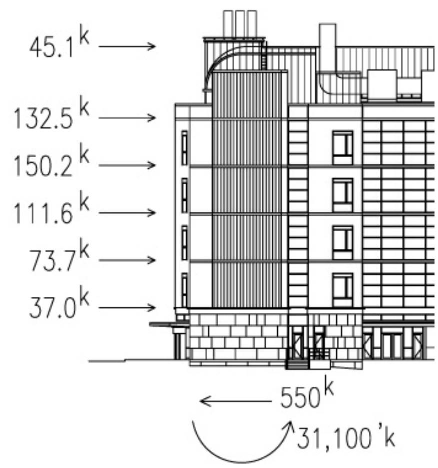


Figure 3: Vertical Distribution of Equivalent Seismic Forces, E-W

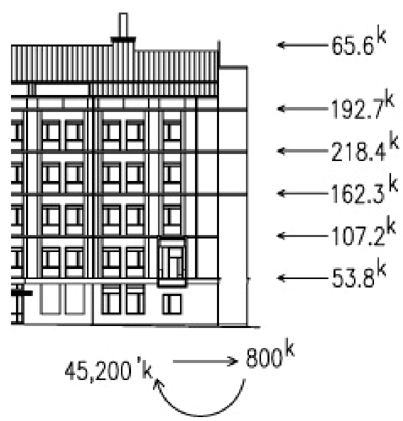


Figure 4: Vertical Distribution of Equivalent Seismic Forces, N-S

Critical Lateral Forces

Comparing the lateral forces calculated, seismic forces clearly controlled for the North-South direction, with the values of base shear, overturning moment, and all story forces being significantly greater for seismic than wind. This is due to the narrow dimension of the building providing a relatively small surface area for wind loading. These controlling forces are shown graphically in Figure 4.

The East-West direction is a little more complicated. Base shear, overturning moment, and many of the story forces are greater for the seismic case, except when considering load factors (1.6 for wind vs. 1.0 for seismic in LRFD, similar for ASD), in which case the wind load controls. Multiple load cases and combinations will need to be considered in order to most effectively analyze the building's frames in this direction.

Description of Lateral System

East-West Direction

The structural system designed for the University of Rochester BME/Optics Building consists of four concentrically braced frames to support lateral forces in the East-West direction. All members use HSS 7x7x1/2 bracing members up to the roof/penthouse floor level. Three of the main braced frames use chevron bracing, while the fourth uses diagonal bracing due to its narrower dimension. Lateral load is transferred to the frames from the concrete floor slab by ¾" diameter shear studs, with a minimum of one stud per foot. The locations and elevations of the frames are given in Figures 5 and 6, respectively.

The mechanical penthouse, which is taller than the typical floor level (20' rather than 14'-8") uses a slightly different lateral system. Three of the braced frames (BR-1, BR-3, and BR-4) continue to the penthouse roof with HSS 8x8x1/2 bracing members. In addition, there are a series of moment frames. These frames consist of bent W18x40 members with moment-resisting connections to the W12 columns that support them. The location of the lateral elements and a typical elevation of a penthouse roof frame are given in Figures 7 and 8, respectively. These additional moment frames are necessary because of the lack of a floor system to act as a diaphragm. The penthouse roof consists of 3" roof deck spanning across W12 beams. Without a concrete slab, there is no way to adequately transfer lateral load to the braced frames, which are spaced very far apart. The moment frames are necessary at regular intervals to resist the wind load on the walls and curved portion of the roof as shown in Figure 8.

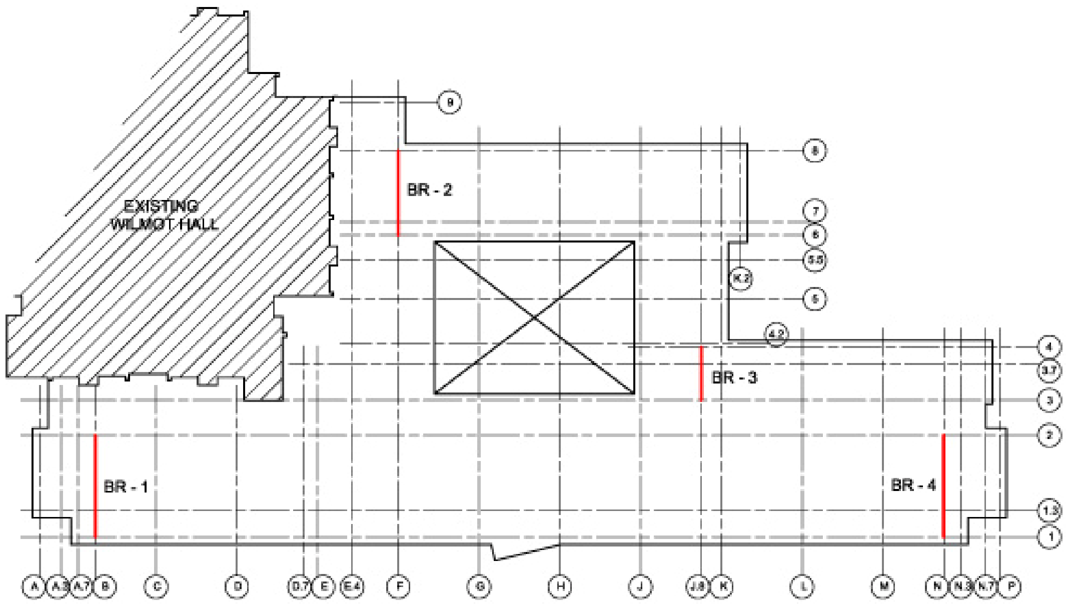
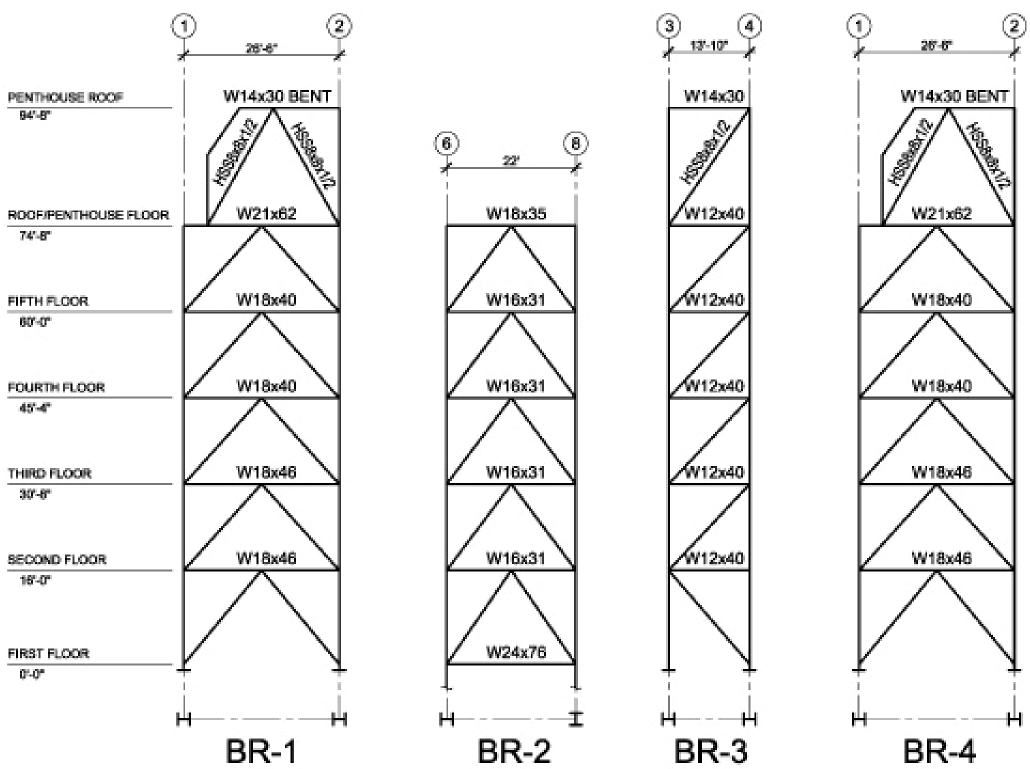


Figure 5: Braced Frames, East-West Direction
Typical, first floor to penthouse floor



NOTES:

1. All bracing members are HSS 7x7x ½ unless otherwise noted.
2. Working point for chevron bracing is at mid-point between supports.
3. Minimum of 1 shear stud per foot on all frame beams to transfer lateral load.

Figure 6: Braced Frame Elevations

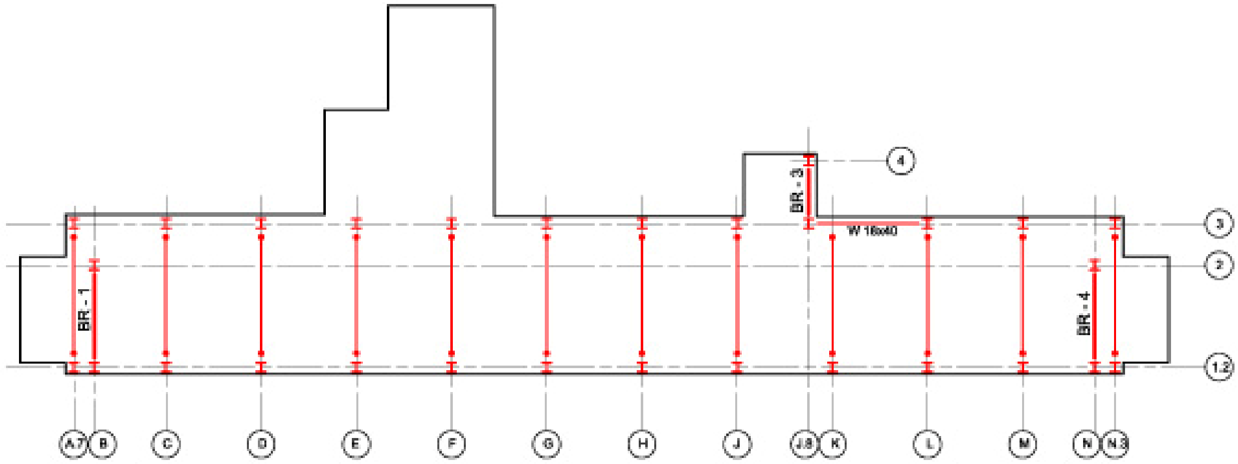


Figure 7: Location of Lateral Elements for Mechanical Penthouse, E-W Direction
Solid circles indicate moment connections

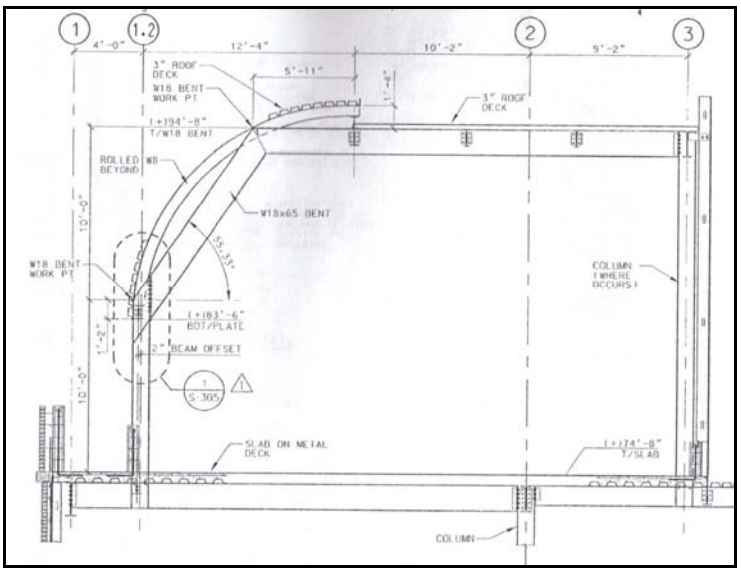


Figure 8: Typical Moment Frame at Mechanical Penthouse, Elevation

North-South Direction

In the longer, narrower direction of the BME/Optics Building, ordinary steel moment frames are utilized to provide lateral stability. In all, there are four moment frames, with the largest at the west face of the building spanning almost the entire length of the building. Locations and elevations of these frames are shown in Figure 9 and Appendix D, respectively.

The most interesting of these frames is located at the east face of the building. The columns of this frame are not continuous. The lecture hall at the first floor, which requires a larger column free space, has columns set wider than the floors above, with large (W33x318) transfer girders to transfer the gravity load from the columns above. Since the columns above are part of a moment frame, additional consideration was needed to transfer lateral loads and maintain continuity of the frame. To achieve this, W8x10 diagonal brace beams form a sort of horizontal truss, as shown in Appendix D. This structural aspect of the building is one of many design challenges that required unique engineering solutions and make this building different than a simpler building with typical bays and a more regularly shaped footprint.

The geometry and location of the mechanical penthouse did not allow the moment frames to support it in this direction. Instead, a series of concentrically braced frames using HSS 7x7x1/2 bracing members in a chevron configuration is utilized. The lateral loads are transferred, through diaphragmatic action, down to the moment frames below. The locations of the braced frames are shown in Figure 10.

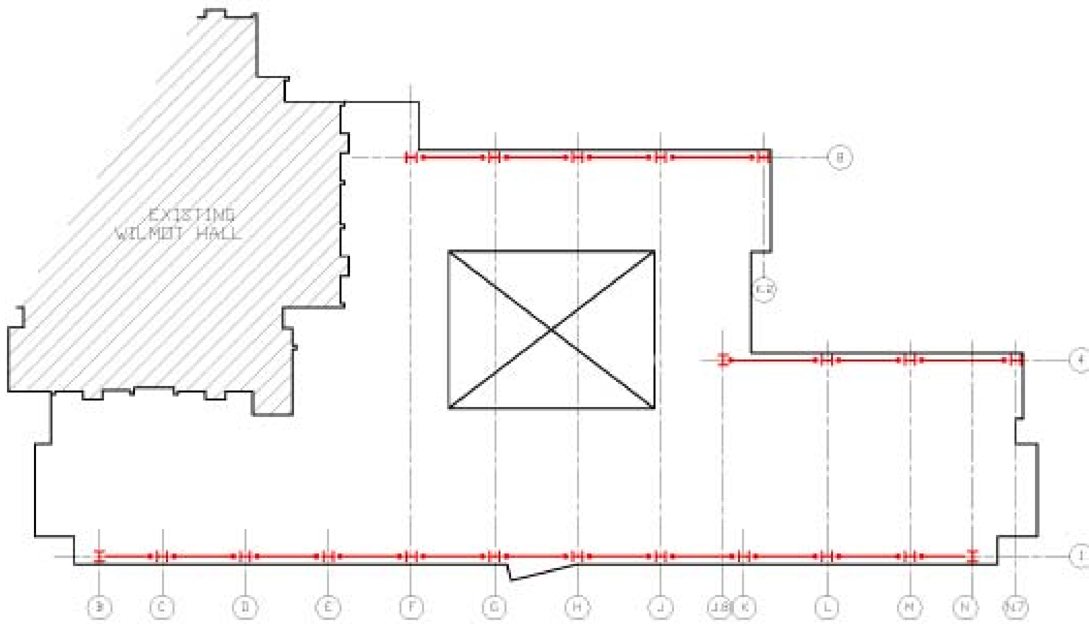


Figure 9: Location of Moment Frames, N-S Direction
Solid circles indicated moment connections

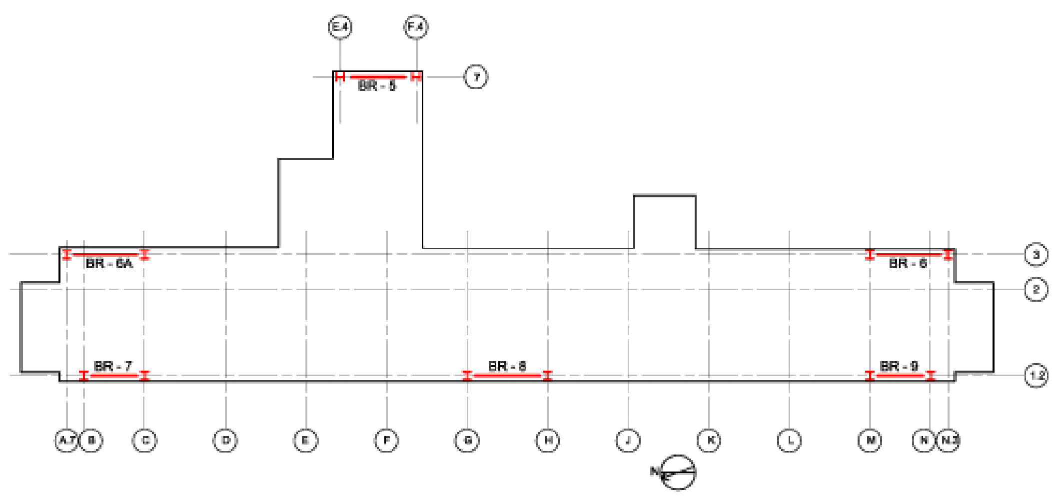


Figure 10: Location of Braced Frames for Mechanical Penthouse, N-S Direction
These are concentrically braced frames using HSS 7x7x1/2 chevron bracing

Distribution of Lateral Loads

The lateral loads for the U of R BME/Optics Building are distributed to the concrete floor slabs, which act as rigid diaphragms. Shear studs in the slab then transfer the lateral load to the building's frames, which are designed to resist these lateral loads and provide stability to the building. The amount of load resisted by a given frame is a result of its stiffness with relation to other frames, along with the geometric layout of the frames in the building. The base shear and overturning moment produced by lateral loads is transferred from the building's frames to the foundation system, which consists of steel H-piles and concrete pile caps designed to resist these types of forces.

In order to better understand the distribution of lateral forces to each frame, the relative stiffness method was used as an approximation. Each frame was modeled individually in RAM Advan*se* with a 100 kip unit load applied at the penthouse floor level. The deflection of each frame was determined, with the inverse of the deflection giving a stiffness value relative to the other frames in that direction. From this, the percent of lateral forces distributed to each frame could be approximated. The values obtained are shown in Tables 5 and 6 below, and an example of the RAM Advanse output is given in Figure 12.

A 3-dimensional computer model of the entire building using RAM Structural System was used to confirm the lateral load distribution approximated by the relative stiffness method. The computer software allows a more in-depth, detailed analysis and is therefore more accurate. The analysis from the RAM Model allowed the percent of lateral load distributed to each frame to be determined, given in Table 7. The values from this building model are reasonably close to those from the relative stiffness method. Sources of discrepancies include torsional effects, number and location of shear studs, and unique geometry of the concrete diaphragms due to the large atrium space. Since the RAM building model is more accurate, the distribution in Table 7 will be used for the strength checks later in this report.

Frame	Deflection (in)	Relative Stiffness, k	% Lateral Load
BR-1	1.00	1.00	31.8
BR-2	1.14	0.88	27.9
BR-3	3.76	0.27	8.5
BR-4	1.00	1.00	31.8

Table 5: Distribution of Lateral Load, Relative Stiffness Method (E-W)
Deflections calculated from a 100 kip unit load

Moment Frame	Deflection (in)	Relative Stiffness, k	% Lateral Load
Column Line 1	2.80	0.36	53.9
Column Line 4	8.84	0.11	17.1
Column Line 8	5.20	0.19	29.0

Table 6: Distribution of Lateral Load, Relative Stiffness Method (N-S)
Deflections calculated from a 100 kip unit load

Analysis Results

Translations

Node	Translations [in] TX	TY	TZ	Rotations [Rad] RX	RY	RZ
Condition L100=100 kip Lateral Load						
12	1.00310	-0.15698	0.00000	0.00000	0.00000	-0.00127

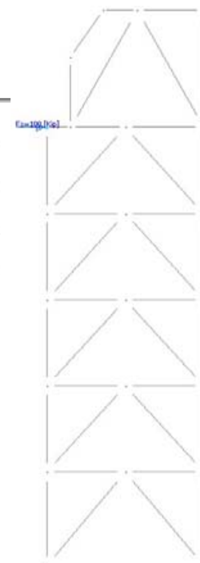


Figure 11: Example of Relative Stiffness Calculation
Modeled in RAM Advanse

E-W Direction (Braced Frames)		N-S Direction (Moment Frames)	
Frame	**% Lateral Load**	**Col Line**	**% Lateral Load**
BR-1	34.4	1	61
BR-2	16.5	4	19.7
BR-3	10.7	8	19.3
BR-4	38.4		

Table 7: Distribution of Lateral Loads
From building model in RAM Structural System

Torsional Effects

The layout of the lateral force resisting elements of a building is an important design consideration. The locations and relative stiffnesses of the frames can be analyzed to determine the center of rigidity of the building. If this location differs significantly from the center of mass, a twisting, or torsion of the building can occur. This torsional moment causes additional lateral forces in certain frames (depending on the direction) along with the lateral forces expected from stiffness distributions.

According to ASCE 7-02, an “accidental” torsional moment should be included in designs with rigid diaphragms. This accidental torsional moment is found by using the resultant force of the lateral load at a distance from the center of mass equal to 5% of the overall building dimension in that direction. This accidental torsion is added to any of the torsion resulting from differences between the center of mass and center of rigidity. Since this building is in Seismic Design Category B, no amplification factor is necessary. Once the torsional moment is determined, torsional force can then be found for a given frame, to be added to the direct shear on the frame from the lateral force.

The RAM model of the BME/Optics Building provided the locations of the center of mass and center of rigidity by floor, as shown in Appendix E. The eccentricities were relatively small, less than the 5% of the overall building dimension in all cases. Despite small eccentricities, the torsional forces produced are significant, and cannot be ignored. In analyzing braced frame BR-1 in the following section, torsional forces accounted for about 20% of the total lateral load on the frame. This is due to the large distance between the frame and the center of rigidity of the building. Detailed calculations of the torsional effects on critical frames are given in the following section and in Appendix F.

Strength Check of Critical Members

In order to accurately analyze the lateral resistance of this somewhat irregular-shaped building, a 3-dimensional computer model of the building using RAM Structural System was used. The computer model uses all of the member sizes, both for gravity and lateral force resistance, that were designed for the actual U of R BME/Optics Building. The model provides accurate analysis of lateral strength, stability, drift, and accounts for torsional effects. A rendering of the lateral frames from the RAM model is shown in Figure 12 below.

This computer model confirms the lateral system design for this building. Based on design loads using several load combinations, all member sizes were sufficient to resist lateral loads. The computer model accounted for several different wind loading patterns and used eccentricity for both wind and seismic calculations to account for torsion.

In addition, hand calculations of two of the building's frames were performed for comparison with the computer model. The braced frame BR-1 and the moment frame at column line 1 were analyzed, as these frames carry the highest percent of lateral load in their respective directions. Calculations are given in Appendices F & G.

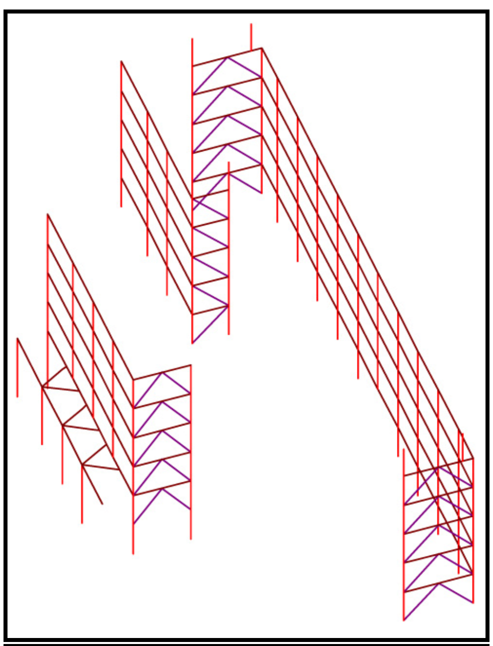


Figure 12: Main Lateral Force Resisting Elements
3-D Building model using RAM Structural System

East-West Direction: Braced Frame BR-1

Braced frame BR-1 (see Figure 5 & 6, p. 9) was analyzed for both wind and seismic load combinations in accordance with ASCE 7-02. Because of the geometric relationship and the large distance between this frame and the center of rigidity, torsional forces were found to be quite significant. Torsional eccentricities and moments were relatively small, as the frame layout of this building was well designed. However, the forces generated in frame BR-1 due to torsional effects accounted for about 20% of the total equivalent seismic forces and close to 15% of the total wind forces. This is largely in part to its location near the edge of the building, about 110 feet from the center of rigidity.

After the story forces were determined, frame BR-1 was analyzed using RAM Advanse, using the following load combinations (LRFD):

- 1.6W
- 1.0E
- 1.2D + 1.6W + L +0.5S
- 1.2D + 1.0E + L + 0.2S

A hand check of a critical bracing member using the 1.6W load case (for simplicity) was also performed to verify the RAM Advanse output for the same load case.

As expected, wind loads controlled over seismic for all members in this lateral direction. Because of the nature of concentrically braced frames, shear and moment values were negligible, and member sizes are based on axial forces. The maximum axial force in a bracing member was found to be 229 kips, which is allowable for the HSS 7x7x1/2 members designed for this frame (ΦP_n = 245 k in compression when KL = 21'). Critical columns were also analyzed. The only notable discrepancy was in the size of the column between the 4th floor and the roof. It was designed as a W12x53, whereas a W12x40 would have been sufficient by my calculations. Possible causes of this discrepancy are snow drift and controlling lateral displacement (which will be considered later in this report). All calculations are given in Appendix F.

North-South Direction: Moment Frame at Column Line 4

Although the RAM model verified all member sizes for this building, an individual analysis of one of the moment frames was necessary for the purposes of this report. By modeling the moment frame at column line 4 (See Appendix D, p. 22) individually in RAM Advanse with all calculated gravity and lateral loads applied to it, the forces in each member could be analyzed more easily and compared to allowable values.

The load combination used for the lateral analysis of this frame was 1.0E + 1.2D + L +0.2S. The gravity loads on this frame were modeled as point loads, as the girders and columns that form this moment frame have intermediate beams framing into them. For the lateral seismic load, equivalent story forces were determined based on earlier seismic load and relative stiffness calculations.

This frame was previously determined to carry 19.7% of the total lateral load in this direction. However, analysis was a bit complicated because the moment frames only extend to the penthouse floor, not to the high roof. As discussed earlier, the lateral forces in this direction are resisted by a series of braced frames at the roof level that transfer lateral forces through the floor diaphragm to the frames below (See Figure 10). In this strength check, the effective shear and moment from the braced frames above were added to the penthouse level of the moment frame.

Upon analysis in RAM Advanse, the member forces for the moment frame could be determined. For this spot check, two of the columns at the base of the structure were analyzed for the combined loading effects. The effective forces were compared to Equation H1-1a from the AISC Manual of Steel Construction:

$$P_u/\Phi P_n + 8/9\ (M_u/\Phi M_n) \leq 1.0$$

Both columns were found to pass the combined loading criteria from this equation. In common practice, engineers would usually want the combined loading effect to be 80% or less as an additional safety factor. By my calculations, the columns in question were at about 90% capacity. It should be noted, however, that I was quite conservative in seismic load calculations, and the column sizes designed for this building are sufficient.

Through an overall building model and detailed spot checks of critical lateral members, the design for the lateral system of the University of Rochester BME/Optics Building has been confirmed.

Drift

As a rule of thumb, building drift is usually limited to 1/400 of the building height in common practice, unless unique conditions exist. For the BME / Optics Building, this h/400 value equals 2.85”. Because of the nature of the mechanical penthouse and the lack of public access, the drift at the high roof is not an issue. Therefore, the drift was calculated at the penthouse floor/low roof level, 74.67’ above grade. Based on analysis from RAM Frame, the building drift at the mechanical penthouse floor level is 2.0” in the North-South direction and 1.2” in the East-West direction, within the H/400 value of 2.24”.

Though the drift for this building is minimal, it was still a concern in its design. The BME / Optics Building, as described earlier, is built adjacent to Wilmot Hall with expansion joints that increase in size from up to the roof. The critical condition for these joints is the seismic load case. If the two buildings have different periods, they can converge towards each other under seismic loading. Since the joints are 12” at the penthouse floor level, they are sufficient to allow a reasonably large drift from both buildings. Another reason for these large expansion joints is for the 2-hour fire rating that they were detailed for.

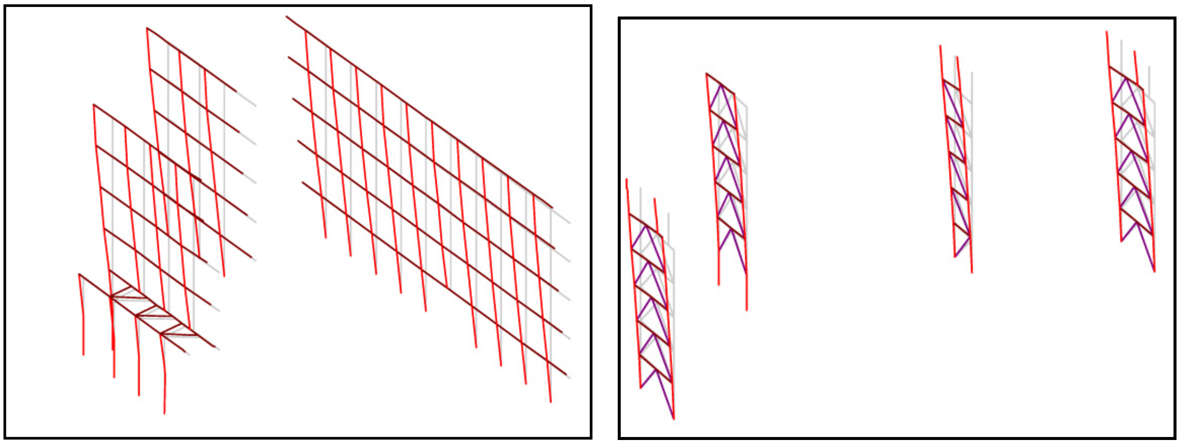


Figure 12: Deflected Shape of Lateral Elements

Summary / Conclusions

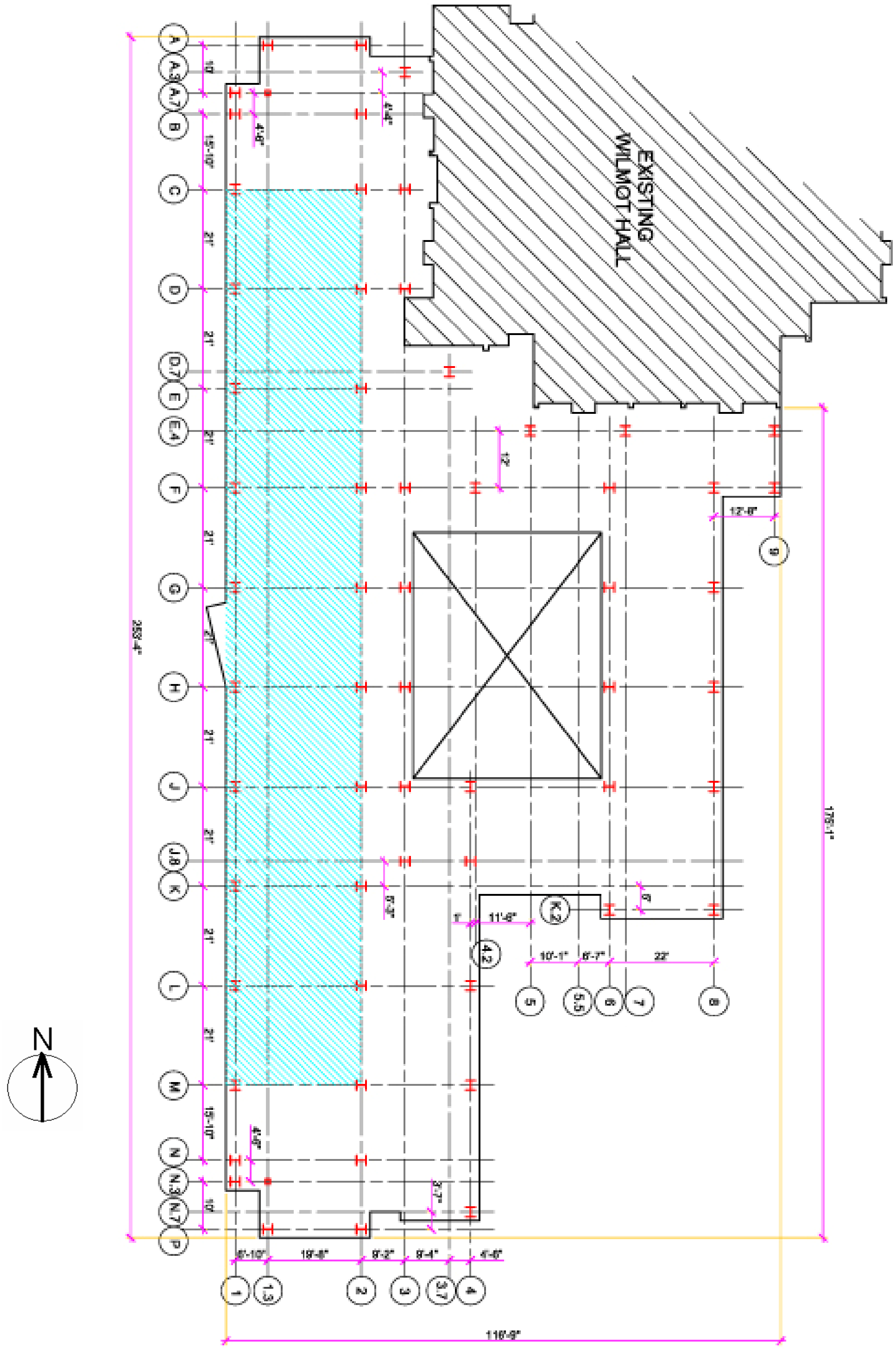
The University of Rochester BME / Optics Building is well designed to resist lateral loads and meet the architectural needs of the building. The use of braced frames in the East-West direction is very efficient, while the use of moment frames in the North-South direction utilizes the long building dimension and allows for windows at exterior faces.

The elements in both directions are designed, through balance in geometry and stiffness, to resist torsional forces. The eccentricity between the centers of mass and rigidity is less than 5% of the building dimension in both directions, at all floor levels. This often proves to be difficult in buildings with an irregular-shaped footprint.

The lateral system also meets several design challenges for this building. For one, the columns of one of the moment frames are not continuous to the foundation. Instead, transfer girders carry the load to nearby columns to provide column free space for a lecture hall. To provide continuity of the moment frame, a horizontal truss transfers lateral forces to nearby columns, and then to the foundation. Other design challenges include isolation joints with an adjacent building and a change in lateral system for the mechanical penthouse / high roof.

In this report, lateral loads were calculated using ASCE 7-02, and were distributed to lateral elements through the relative stiffness method. Strength checks of the lateral system were performed, both using a 3-D building model in RAM Structural System, and by spot checks using RAM Advanse, spreadsheets, and hand calculations. Through this, the lateral system for the University of Rochester BME / Optics Building was explored in detail and confirmed.

Appendix A: Typical Floor Plan – Column Layout



Appendix B: Wind Distribution Calculations

WIND FORCE DISTRIBUTION

NORTH - SOUTH

FLOOR	ELEV	TRIB HEIGHT	LOAD	
1	0'-0"	8'	(12.97 PSF)(8')(138') =	14.3^k
2	16'-0"	15.33'	[(12.97 PSF)(7') + 13.66 PSF(5') + 14.21 PSF(3.33')] (138') =	28.5^k
3	30'-8"	14.67'	[(14.21 PSF)(1.67') + 14.77 PSF(5') + 15.59 PSF(8')] (138') =	30.7^k
4	45'-4"	14.67'	[15.59 PSF (2') + 16.28 PSF(10') + 16.83 PSF(2.67')] (138') =	33.0^k
5	60'-8"	14.67'	[16.83 PSF (7.33') + 17.38 PSF (7.33')] (138') =	34.6^k
PENTHOUSE	74'-8"	17.33'	[17.38 PSF (2.67') + 17.94 PSF (8.67')] (138') + [17.94 PSF (1.33') + 18.35 PSF (4.67')] (85') =	37.2^k
ROOF	94'-8"	10'	[18.35 PSF (5.33') + 18.76 PSF (4.67')] (85') =	15.8^k
			BASE SHEAR =	194^k

$M = 14.3^k(8') + 28.5^k(16') + 30.7^k(30.67') + 33^k(45.33') + 34.6^k(60.67') + 37.2^k(74.67') + 15.8^k(94.67') = 9381'k$

OVERTURNING MOMENT = 9400'k

EAST-WEST

FLOOR	LOAD	
1	16.39 PSF (8')(254') =	33.3^k
2	[16.39 PSF (7') + 17.08 PSF (5') + 17.63 PSF (3.33')] (254') =	65.7^k
3	[17.63 PSF (1.67') + 18.18 PSF (5') + 19 PSF (8')] (254') =	69.2^k
4	[19 PSF (2') + 19.69 PSF (10') + 20.25 PSF (2.67')] (254') =	73.4^k
5	[20.25 PSF (7.33') + 20.8 PSF (7.33')] (254') =	76.4^k
PENTHOUSE	[20.8 PSF (2.67') + 21.35 PSF (10') + 21.76 PSF (4.67')] (254') =	94.1^k
ROOF	[21.76 PSF (5.33') + 22.18 PSF (4.67')] (254') =	55.8^k
	BASE SHEAR =	468^k

$M = 33.3^k(8') + 65.7^k(16') + 69.2^k(30.67') + 73.4^k(45.33') + 76.4^k(60.67') + 94.1^k(74.67') + 55.8^k(94.67') = 23711'k$

OVERTURNING MOMENT = 24000'k

Appendix C: Seismic Calculations

SEISMIC LOAD

$S_S = 0.25$ $S_1 = 0.07$ (FIG 9.4.1.1 a,b)

$F_a = 1.6$ $F_V = 2.4$ (TABLES 9.4.1.2 a,b)

$S_{MS} = 0.4$ $S_{M1} = 0.168$

$S_{DS} = 0.267$ $S_{D1} = 0.112$

OCCUPANCY CATEGORY III

SEISMIC USE II $I = 1.25$

NORTH-SOUTH: ORDINARY MOMENT FRAMES $R = 3.5$

$$C_S = \frac{S_{DS}}{R/I} = \frac{0.267}{3.5/1.25} = 0.095$$

EAST-WEST: STEEL CONCENTRIC BRACED FRAMES $R = 5$

$$C_S = \frac{0.267}{5/1.25} = 0.067$$

$C_t = 0.02$ $x = 0.75$

$T_a = 0.02(95')^{0.75} = 0.61$

$k = 1.06$ (INTERPOLATION

$$C_{S,\,MAX\ N\text{-}S} = \frac{S_{D1}}{T(R/I)} = \frac{0.112}{0.61\left(\frac{3.5}{1.25}\right)} = \underline{0.066}$$

$$C_{S,\,MAX\ E\text{-}W} = \frac{0.112}{0.61(5/1.25)} = \underline{0.046}$$

$$C_{S,min} = 0.044\,S_{DS}\,I = 0.044(0.267)(1.25) = 0.015 \ \underline{OK}$$

EQUIVALENT SEISMIC WEIGHT

TYPICAL FLOOR 2-5

DEAD LOAD = 115 PSF AREA = 18,000 SQ FT

PARTITIONS = 20 PSF

$$W = (135\ PSF)(18{,}000\ FT^2) = 2430^k$$

PENTHOUSE

DEAD LOAD = 150 PSF

FLOOR AREA = 9,000 FT²

ROOF DEAD LOAD + 20% SNOW = 40 PSF + 0.2(35 PSF) ≈ 50 PSF

ROOF AREA = 7,000 FT²

$$W = (150\ PSF)(9{,}000\ FT^2) + (50\ PSF)(7{,}000\ FT^2) = 1700^k$$

HIGH ROOF

DEAD LOAD = 50 PSF

AREA = 9,000 FT²

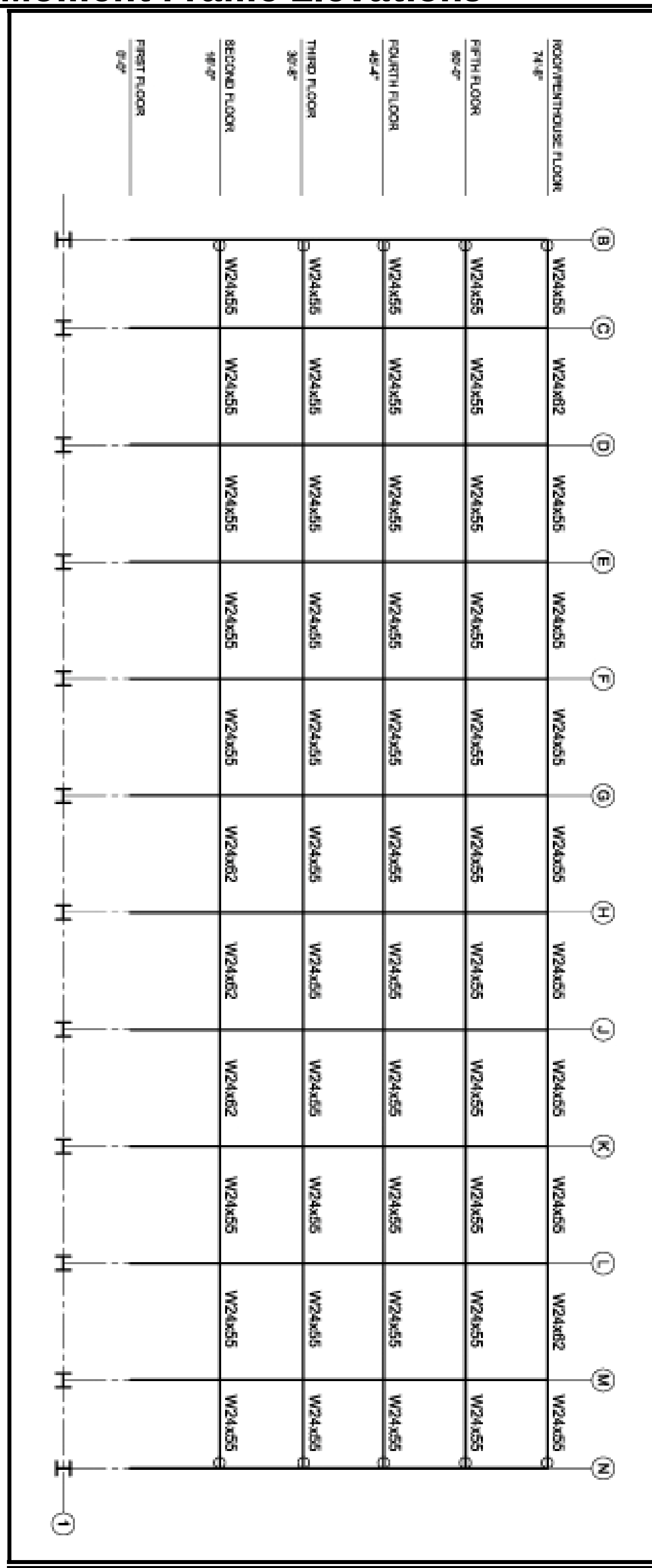
$$W = (50\ PSF)(9000\ FT^2) = 450^k$$

$$W = 4(2430^k) + 1700^k + 450^k = 12{,}000^k$$

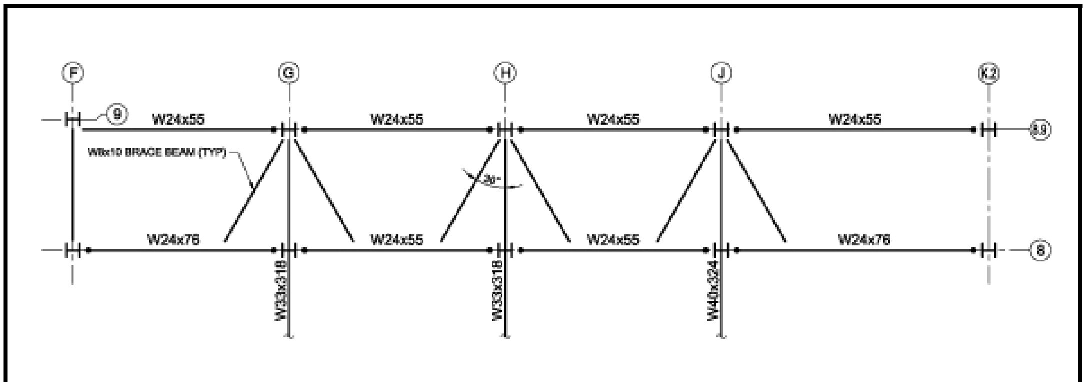
$$V = C_S W \qquad V_{N\text{-}S} \approx \boxed{800^k}$$

$$V_{EW} \approx \boxed{550^k}$$

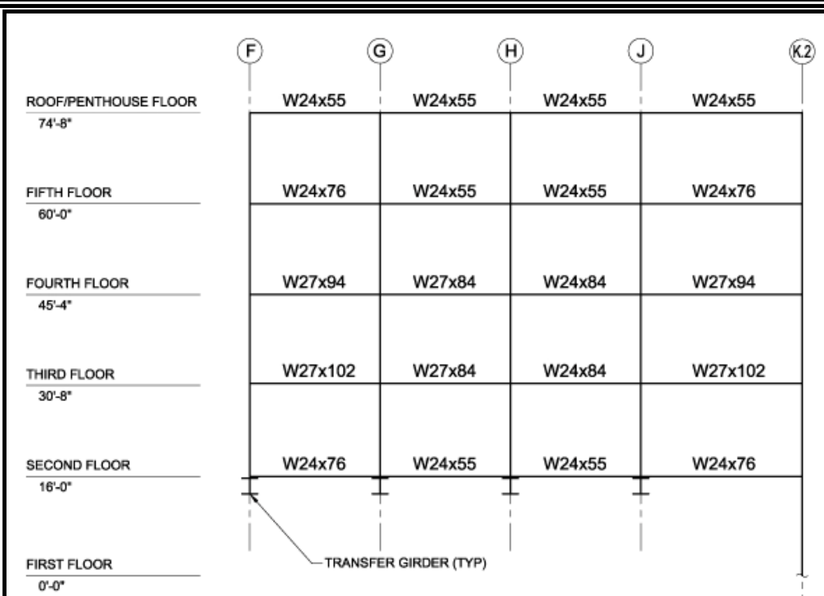
Appendix D: Moment Frame Elevations



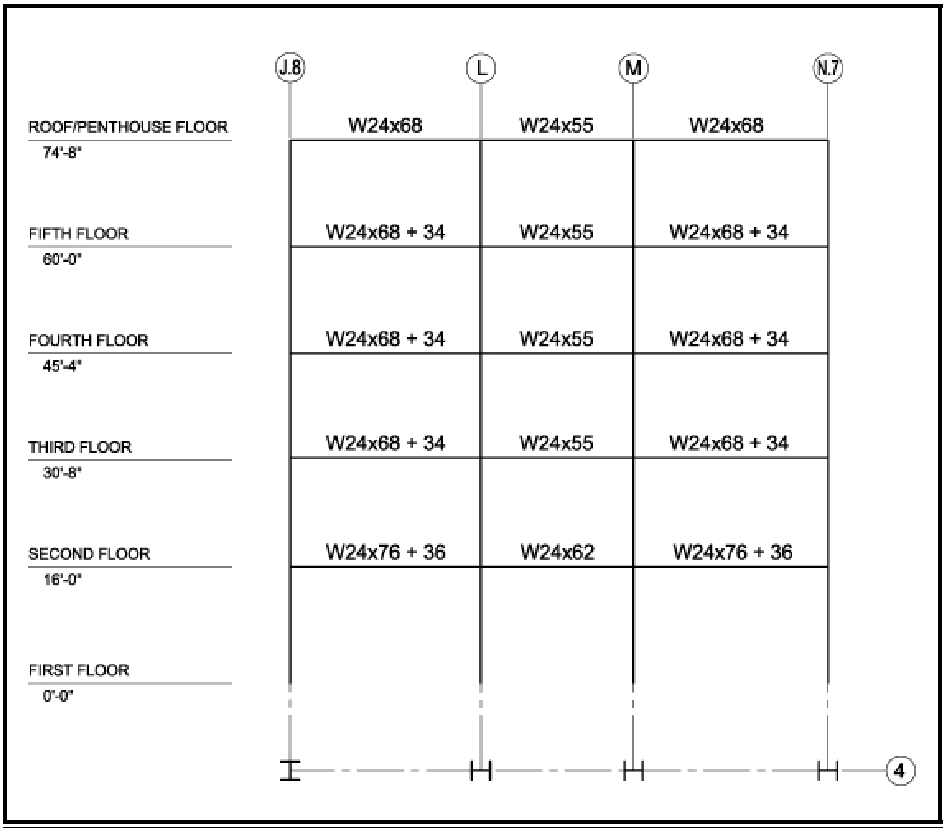
NOTE: Open circles represent pinned (free) connections



PLAN AT 2nd FLOOR



ELEVATION


J.8
L
M
N.7
ROOF/PENTHOUSE FLOOR
74'-8"
W24x68
W24x55
W24x68
FIFTH FLOOR
60'-0"
W24x68 + 34
W24x55
W24x68 + 34
FOURTH FLOOR
45'-4"
W24x68 + 34
W24x55
W24x68 + 34
THIRD FLOOR
30'-8"
W24x68 + 34
W24x55
W24x68 + 34
SECOND FLOOR
16'-0"
W24x76 + 36
W24x62
W24x76 + 36
FIRST FLOOR
0'-0"
4

Appendix E: Torsion Calculations

Center of Rigidity

RAM Frame v11.0
DataBase: BMEO

CRITERIA:

Rigid End Zones: Include Effects: 0.00% Reduction
Member Force Output: At Face of Joint
P-Delta: No
Ground Level: 1st flr
Wall Mesh Criteria :
Wall Element Type : Shell Element with No Out-of-Plane Stiffness
Max. Allowed Distance between Nodes (ft) : 8.00

		Centers of Rigidity		Centers of Mass	
Level	**Diaph. #**	**Xr**	**Yr**	**Xm**	**Ym**
		ft	**ft**	**ft**	**ft**
PENT ROOF HIGH	1	125.45	26.06	122.67	25.40
SKYLIGHT	1	116.00	78.67	129.69	49.68
PENT FLOOR	1	124.24	36.44	129.55	31.82
5TH FLR	1	125.11	37.56	135.09	38.98
4TH FLR	1	125.95	37.36	135.00	38.98
3RD FLR	1	126.83	35.82	135.63	38.56
2nd flr	1	128.32	32.17	136.73	39.04
LECTURE HALL	1	136.55	112.22	123.05	92.05
1st flr	1	124.85	82.01	124.85	82.01

Torsional Moment, N-S Direction (Seismic)						
Story	**Center of Mass (y)**	**Center of Rigidity (y)**	**Ecc. (ft)**	**5 % Bldg Dim**	**Story Force (k)**	**Torsional Moment (ft-k)**
2	39.04	32.17	6.87	6.9	53.8	741
3	38.56	35.82	2.74	6.9	107.2	1033
4	38.98	37.36	1.62	6.9	162.3	1383
5	38.98	37.56	1.42	6.9	218.4	1817
PH	31.82	36.44	-4.62	6.9	192.7	439
R	25.4	26.06	-0.66	6.9	65.6	409
						5823

Torsional Moment, E-W Direction (Seismic)						
Story	**Center of Mass (x)**	**Center of Rigidity (x)**	**Ecc. (ft)**	**5% Bldg Dim**	**Story Force (k)**	**Torsional Moment (ft-k)**
2	136.73	128.32	8.41	12.7	37	781
3	135.63	126.83	8.8	12.7	73.7	1585
4	135	125.95	9.05	12.7	111.6	2427
5	135.09	125.11	9.98	12.7	150.2	3407
PH	129.55	124.24	5.31	12.7	132.5	2386
R	122.67	125.45	-2.78	12.7	45.1	447
						11033

Torsional Moment, E-W Direction (Wind)						
Story	**Wind Load Resultant (x)**	**Center of Rigidity (x)**	**Ecc. (ft)**	**5% Bldg Dim**	**Story Force (k)**	**Torsional Moment (ft-k)**
2	126.5	128.32	-1.82	12.7	65.7	715
3	126.5	126.83	-0.33	12.7	69.2	856
4	126.5	125.95	0.55	12.7	73.4	973
5	126.5	125.11	1.39	12.7	76.4	1076
PH	126.5	124.24	2.26	12.7	94.1	1408
R	126.5	125.45	1.05	12.7	55.8	767
						5795

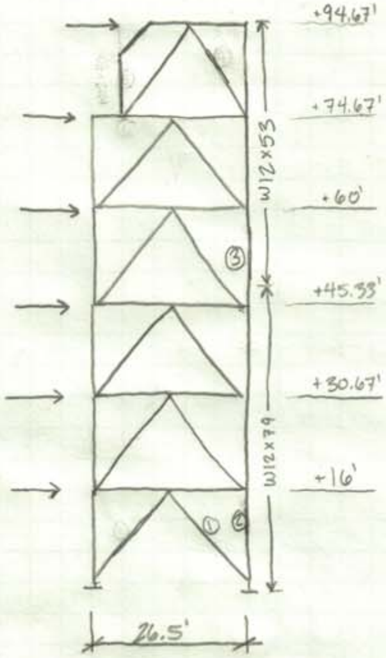
Appendix F: Strength Check of Braced Frame BR-1

J-value for Determining Torsional Forces				
Frame	BR-1	BR-2	BR-3	BR-4
Relative stiffness, k	34.4	16.5	10.7	38.4
Distance to center of rigidity (avg), d	110.0	31.0	-47.8	-110.8
kd^2	416240	15857	24448	471423
J = ∑kd^2 =	***928000***			

Total Design Forces on Frame BR-1 (Seismic)						
Story	2	3	4	5	Penthouse	Roof
Torsional Moment, M (ft-k)	781	1585	2427	3407	2386	447
kd/J	0.004	0.004	0.004	0.004	0.004	0.004
Torsional Force (k)	**3.1**	**6.3**	**9.7**	**13.6**	**9.5**	**1.8**
Building Story Force (k)	37	73.7	111.6	150.2	132.5	45.1
% Story Force on BR-1	34.4	34.4	34.4	34.4	34.4	34.4
Direct Shear on BR-1 (k)	**12.7**	**25.4**	**38.4**	**51.7**	**45.6**	**15.5**
Total Design Forces on Frame BR-1 (k)	***15.9***	***31.7***	***48.1***	***65.3***	***55.1***	***17.3***

Total Design Forces on Frame BR-1 (Wind)						
Story	2	3	4	5	Penthouse	Roof
Torsional Moment, M (ft-k)	715	856	973	1076	1408	767
kd/J	0.004	0.004	0.004	0.004	0.004	0.004
Torsional Force (k)	**2.9**	**3.4**	**3.9**	**4.3**	**5.6**	**3.1**
Building Story Force (k)	65.7	69.2	73.4	76.4	94.1	55.8
% Story Force on BR-1	34.4	34.4	34.4	34.4	34.4	34.4
Direct Shear on BR-1 (k)	**22.6**	**23.8**	**25.2**	**26.3**	**32.4**	**19.2**
Total Design Forces on Frame BR-1 (k)	***25.5***	***27.2***	***29.1***	***30.6***	***38.0***	***22.3***

BRACED FRAME BR-4



- ALL BRACING MEMBERS HSS 7×7×½
  EXCEPT @ ROOF HSS 8×8×½
- CRITICAL MEMBERS NUMBERED 1-3
- ASSUME BEAMS <u>OK</u>
- LOAD COMBINATIONS CONSIDERED:
  1. 1.6 W
  2. 1.0 E
  3. 1.6 W + 1.2D + L + 0.5S
  4. 1.0E + 1.2D + L + 0.2S
- LATERAL LOADS CALCULATED PREVIOUSLY (INCLUDE TORSIONAL EFFECTS)
- <u>GRAVITY LOADS</u>
  DEAD: 120 PSF (FLOOR) / 35 PSF (ROOF)
  LIVE: 80 PSF (FLOOR) / 150 PSF (PENTHOUSE)
  SNOW: 35 PSF

*ADDITIONAL 57^k/FLOOR AXIAL LOAD ON EACH COLUMN FROM GIRDER REACTIONS
64^k @ PENTHOUSE

<u>BEAMS</u>

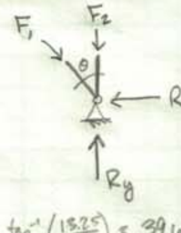
*TYP FLOOR
$[1.2(120\,PSF) + 80\,PSF](4'\ \text{TRIB WIDTH}) = \underline{0.9\ KLF}$

*PENTHOUSE
$[1.2(120\,PSF) + 150\,PSF](4') = \underline{1.2\ KLF}$

*ROOF
$[1.2(35\,PSF) + 0.5(35\,PSF)](4') = \underline{240\ PLF}$
$[1.2(35\,PSF) + 0.2(35\,PSF)](4') = \underline{196\ PLF}$

- <u>MEMBER ①</u> HSS 7×7×½ L = 20.8'
  FROM RAM ADVANSE:
  MAX AXIAL COMP. FROM LOAD COMBINATION 3 → $\underline{P_u = 229^k}$
  FROM LRFD, $\phi P_n = 245^k > 229^k$ <u>OK</u> ∴ <u>ALL BRACING MEMBERS OK</u>

* <u>SPOT CHECK</u> USING LOAD COMBINATION 1



$\theta = \tan^{-1}\left(\frac{13.25}{16}\right) = 39.6°$

BASE SHEAR $= (25.5^k + 27.2 + 29.1 + 30.6 + 38.0 + 22.3)(1.6)$
$= 276.3^k$

$R_x = 276.3^k/2 = 138^k$

OVERTURNING MOMENT $= 14950'k$
$R_y = 14950'k/26.5' = 564^k$

$F_1 = 138/\sin 39.6° = \underline{216.5^k}$

$F_2 = 564 - 216.5\cos 39.6° = \underline{397.2^k}$

FROM RAM ADVANSE:
$F_1 = 217.2^k$ OK
$F_2 = 397.2^k$ OK

MEMBER ② W12×79 $L = 16'$

$P_u = 748^k$

$\phi P_n = 737^k$ X

HOWEVER, MODELED AS A PINNED BASE (CONSERVATIVE)
IF FIXED, $k = 0.7 \rightarrow kL = 11.2'$, $\phi P_n = 860^k$ OK

MEMBER ③ W12×53 $L = 14.67'$

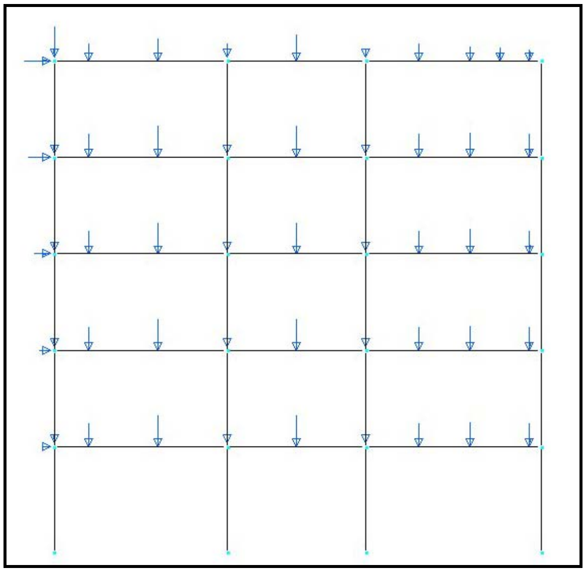
$P_u = 225^k$

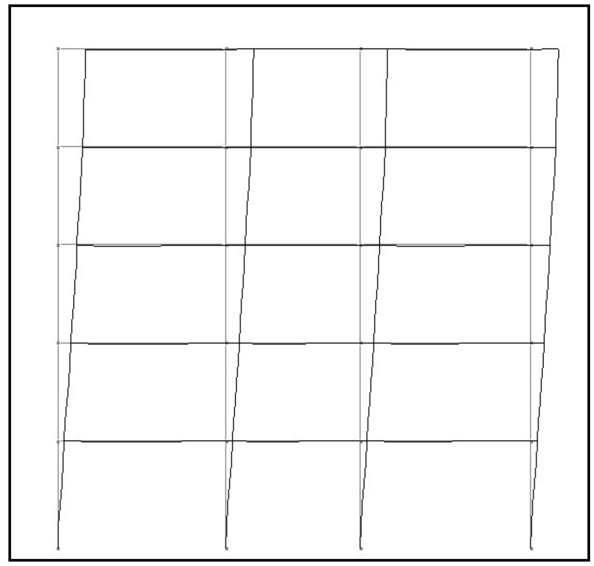
$\phi P_n = 451^k$ OK

Appendix G: Strength Check of Moment Frame, Col. Line 4

J Value for Determining Seismic Forces			
Column Line	1	4	8
Relative stiffness, k	61	19.7	19.3
Distance to center of rigidity (avg), d	35.9	13.6	64.8
kd^2	78617	3644	81041
J = ∑kd^2 =	***163300***		

Total Design Forces on Moment Frame at Col. Line 4 (Seismic Case Controls)						
Story	2	3	4	5	Penthouse	Roof
Torsional Moment, M (ft-k)	741	1033	1383	1817	439	409
kd/J	0.00164	0.00164	0.00164	0.00164	0.00164	0.00164
Torsional Force (k)	**1.2**	**1.7**	**2.3**	**3.0**	**0.7**	**0.7**
Building Story Force (k)	53.8	107.2	162.3	218.4	192.7	65.6
% Story Force on Frame	19.7	19.7	19.7	19.7	19.7	19.7
Direct Shear on Frame (k)	**10.6**	**21.1**	**32.0**	**43.0**	**38.0**	**12.9**
Total Design Forces on Frame (k)	***11.8***	***22.8***	***34.2***	***46.0***	***38.7***	***13.6***





Analysis Results

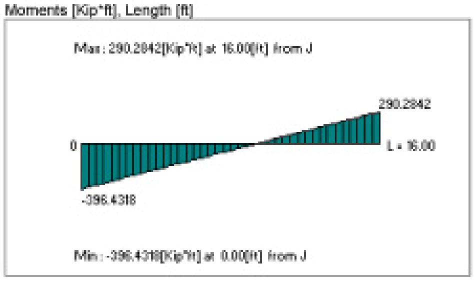
Forces diagram printout

Load conditions
E1=1.0E+1.2D+L+0.2S

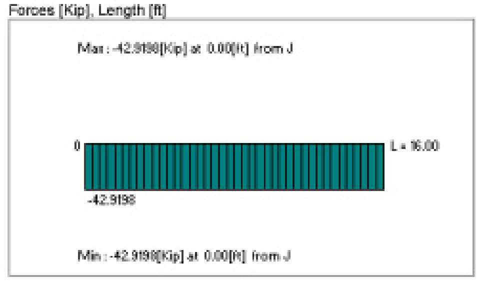
MEMBER	: 16	Length	:	16.000 [ft]	Node J	:	4
Material	: A992 Gr50	Section	:	W 12X96	Node K	:	8

Condition : E1=1.0E+1.2D+L+0.2S

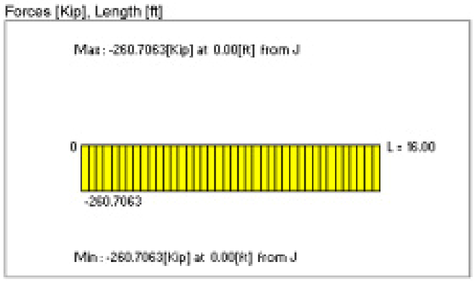
M33 bending moment
Moments [Kip*ft], Length [ft]



V2 shear forces
Forces [Kip], Length [ft]



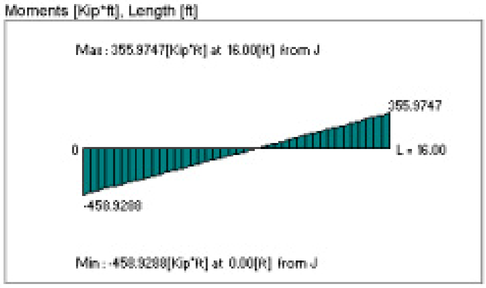
Axial forces
Forces [Kip], Length [ft]



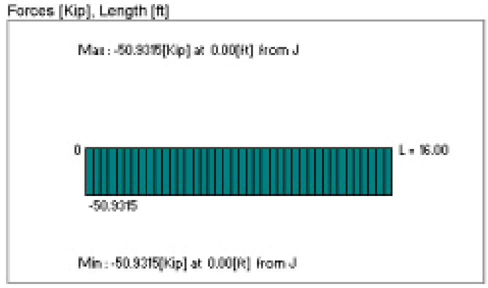
MEMBER : 11 Length : 16.000 [ft] Node J : 3
Material : A992 Gr50 Section : W 12X106 Node K : 7

Condition : E1=1.0E+1.2D+L+0.2S

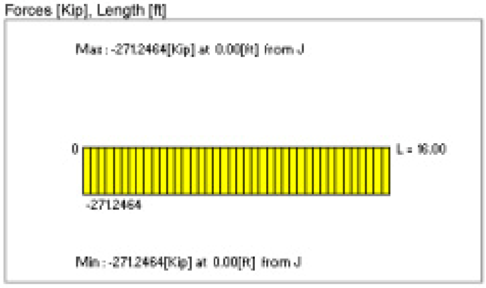
M33 bending moment
Moments [Kip*ft], Length [ft]



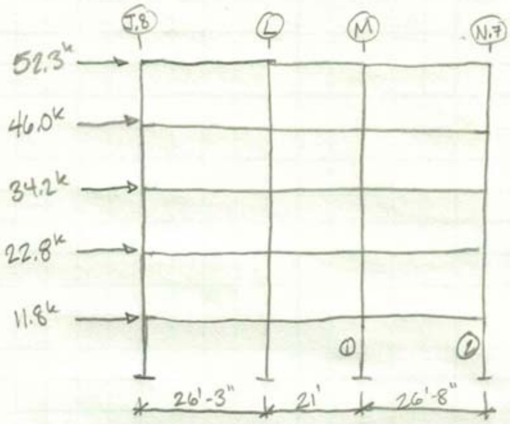
V2 shear forces
Forces [Kip], Length [ft]



Axial forces
Forces [Kip], Length [ft]



MOMENT FRAME @ COL. LINE 4



- CRITICAL LOAD COMBINATION
  $1.0E + 1.2D + L + 0.2S$

- SPOT CHECK OF COLUMNS MARKED 1 & 2

COLUMN 1 → $P_U = 260.7^k$, $M_U = 396'k$ (FROM RAM ADVANSE)

W12x96 $\quad r_x/r_y = 1.76$

$K = 0.5$

$L = 16'$

$KL_{EFF} = 0.5(16')/1.76 = 4.5'$

$\phi P_n = 1160^k$

$P_U/\phi P_n = \frac{260.7}{1160} = 0.225$

$M_U/\phi M_n = \frac{396}{525} = 0.754$

$P_U/\phi P_n + 8/9\left(\frac{M_U}{\phi M_n}\right) \leq 1.0$

$0.225 + 8/9(0.754) = 0.895 < 1.0$ OK

COLUMN 2 → $P_U = 271^k$, $M_U = 458'k$

W12x106 $\quad r_x/r_y = 1.76$

$KL_{EFF} = 0.5(16')/1.76 = 4.5'$

$\phi P_n = 1300^k$

$P_U/\phi P_n = 0.208$

$M_U/\phi M_n = \frac{458}{588} = 0.779$

$0.208 + 8/9(0.779) = 0.90$ OK