UNIVERSITY OF ROCHESTER
BME / OPTICS BUILDING

TECHNICAL ASSIGNMENT 1

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ARCHITECTURAL ENGINEERING
STRUCTURAL OPTION
AE 481W – SENIOR THESIS
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Executive Summary

The purpose of this report is to outline the structural system of the University of Rochester BME/O building and analyze the procedures used in its structural design.

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 needs of these highly regarded programs. The facility, currently known as the BME/O Building, began construction in January of 2005, with a scheduled completion of December 2006.

It is strategically located on the south end of the U of R River Campus, across the street from the Medical Center, and adjacent on two sides to the current Biomedical and Optics facility, the Wilmot Building. 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. Key architectural features of the building include channel glass façade at stairways and an 80' atrium inside the main entrance to be lit by skylights. The structure is 5 stories plus a mechanical penthouse and partial basement, and consists of laboratory, classroom, and office space.

This report provides a detailed description of the structural system used in this building, as well as the codes and design guides used. The BME/O building is a steel framed structure, using composite beams in a majority of the floor systems. Strict vibration and deflection criteria in laboratory spaces were an important design consideration in the floor systems. Steel columns transfer the floor and roof loads down to the foundation – steel H-piles driven to bedrock in various configurations. Another important design consideration was the existing steam and utility tunnel running under the building's footprint. Grade beams frame around the tunnel, as there is a strict requirement to keep the tunnel fully operational throughout construction.

The lateral system resists wind and seismic loads through braced frames in the east-west direction, and moment frames in the north-south direction. Since the BME/O Building is adjacent to Wilmot Hall on two sides with isolation joints, accurate lateral deflection calculations were necessary to determine proper clearance.

In order to better understand the design concepts used in this building, several calculations were made in this report. Gravity loads, both live and dead, were determined and used to spot check beams and columns. Also, wind and seismic loads, along with their distributions, were calculated and are displayed in various charts and diagrams. They were then used to analyze the design of a lateral element. In performing these calculations, several conclusions were made in regards to the structural design. It was determined that the design methods and calculations were similar to those I used, although some conservative assumptions were made.

Structural System Description

Foundation

The foundation system used in this building consists of pile caps supported by 50 ksi steel H-piles bearing on bedrock. There are several different pile configurations, but they all use HP8x36 (Design capacity = 66.3 tons), HP10x42 (77.5 tons / 13 kips design uplift), and/or HP10x57 (105 tons). The pile caps have a design lateral load capacity of 4 kips each.

The foundation system also uses grade beams at different sections of the building. All exterior walls are supported by grade beams, typically 16”x 48”, with some variations in size. An existing steam/utility tunnel running under the footprint of the building is framed around by grade beams 24”x 54” and 18”x 24”. Since this tunnel supplies several buildings on this section of campus, its complete functionality throughout construction of BME/O was an important design consideration. Concrete for pile caps and grade beams is normal weight with design strength of 4000 psi. All reinforcement conforms to ASTM A615 Grade 60.

Gravity System

The columns used in this building are predominately W12 shapes, ranging in weight from 40 lb/ft supporting the roof to 120 lb/ft at moment frames. Most of the columns supporting the roof above the mechanical penthouse level do not line up with columns below. W21 transfer girders distribute this roof load to nearby columns.

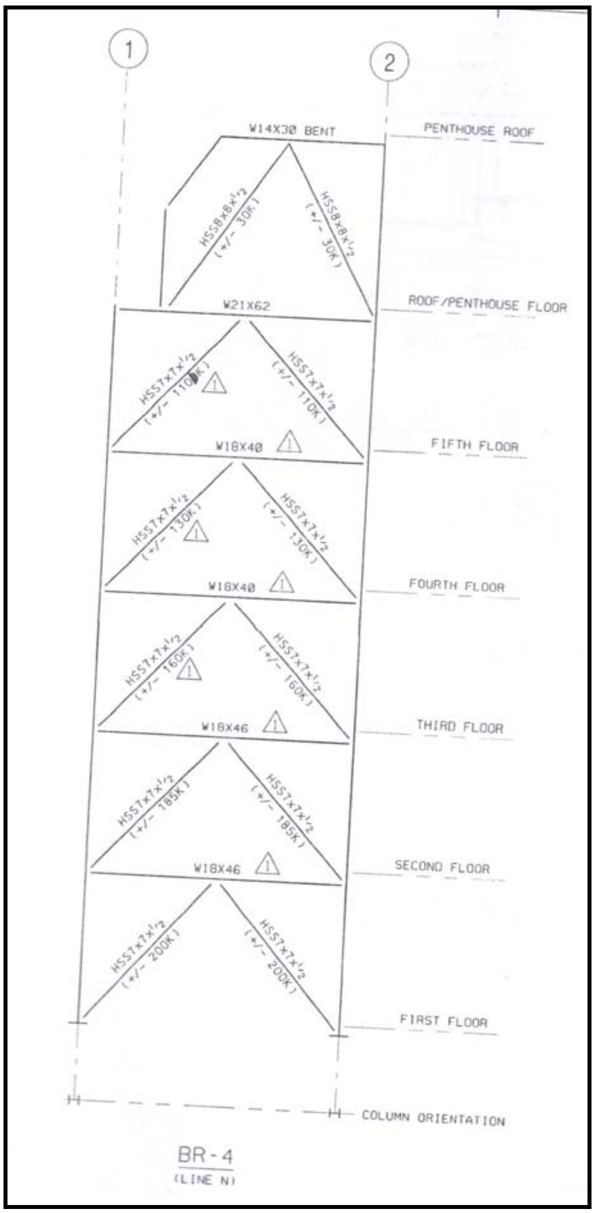
The typical floor system consists of 4 ½” concrete slabs on 3” composite metal deck. The load is distributed from the slab to composite steel beams, then to steel girders, and finally down to the columns and foundation. Although the loads are relatively constant throughout the building, the steel shapes vary in size due to varying spans. This is because of the irregular shape of the building designed to meet architectural and spatial challenges. Although the floors as a whole are similar, there is no “typical bay” redundancy in this building. Also, the strict vibration criterion in laboratory spaces is cause for some variation in beam sizes throughout the building. Beams vary in size from W10x12 to composite W18x35, and girders range from W16x26 to W21x50.

All structural steel is A992 Grade 50, with the exception of a few HSS columns that are ASTM A500 Grade B (46 ksi). The floor slabs are 4000 psi normal weight concrete, reinforced with 6x6 W2.9xW2.9 welded wire fabric. Slab on grade is 5” thick, 3000 psi concrete with similar WWF reinforcing.

Lateral System

Lateral forces due to wind and seismic loading were important design considerations for the BME/O Building. Since it was built adjacent to the existing Wilmot Hall on two sides, the lateral deflection was especially important. Isolation joints increase in size from the ground to the roof. Accuracy in lateral calculations was necessary to determine proper clearance.

The system designed by LeMessurier consultants uses concentric braced frames in the short (E-W) direction, and ordinary moment frames in the long (N-S) direction. There are four similar braced frames in the building using HSS7x7x1/2, 46 ksi steel shapes in the form of chevron bracing. A Typical braced frame is shown below.



Codes / Design Guides

The following codes and guidelines were used in the structural design and construction of The University of Rochester BME / Optics Building, as outlined in the general notes section of the design documents:

Building Code *"The Building Code of New York State" (2002)*
Referencing: International Building Code, IBC 2000
ASCE 7-98

Concrete "Building Code Requirements for Reinforced Concrete" (ACI 318-02)
"Specifications for Structural Concrete for Buildings" (ACI 301-99)

Reinforcement "Building Code Requirements for Reinforced Concrete" (ACI 318-02)
"ACI Detailing Manual-1994" (SP-66)
"CRSI Manual of Standard Practice" (MSP1-97)
"Structural Welding Code - Reinforcing Steel" (AWS D1.4-92)

Structural Steel "Specification for Structural Steel Buildings – Allowable Stress Design and Plastic Design" (AISC 1989)
"Code of Standard Practice for Steel Buildings & Bridges" (AISC 1992)
"Seismic Provisions for Structural Steel Buildings" (AISC March 7, 2000)
"Structural Welding Code – Steel" (AWS D1.1-96)

Deck/Shear Studs "Specification for Design of Light Gage Cold-Formed Steel Structural Members" (AISI-1996)
"Specification for Structural Steel Buildings" (AISC 1989)
"Structural Welding Code – Steel" (AWS D1.1-96)
"Structural Welding Code – Sheet Steel" (AWS D1.3-89)

Calculations / Conclusions

In order to develop a better understanding of the design of the BME/O Building, I performed several calculations of the structural system. In doing this, I was able to make some conclusions about the concepts and assumptions used by the engineers. Although the adopted ordinance is the Building Code of New York State, which references IBC 2000 and ASCE 7-98, I used the more recent IBC 2003 and ASCE 7-02 in my calculations. I found my loads and member sizes to be consistent with the current design, although some possible conservative assumptions by the engineer, as is common practice, were noted.

Gravity Loads

Live

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

Based on the geometry of the roof, and the use of a mechanical penthouse rather than rooftop equipment, there were no notable areas of snow drift. In this climatic region, snow drift is often an important structural design consideration. As common practice in New York, roof live load is omitted in lieu of high snow loads that control design. Also, the lack of specific information about mechanical equipment led to the conservative assumption of 150 pounds per square foot in the equipment area of the mechanical penthouse floor.

Dead

Typical Floor System / Concrete Roof (minimal occurrence)

7 ½“ Floor Slab	94 psf (based on 150 pcf reinf., normal wt. concrete)
3” Steel Deck	3 psf
Framing	10 psf (or beam self-weight where applicable)
Ceiling	2 psf
MEP Allowance	5 psf
` Total:	115 psf

Metal Roof

Built-Up Roofing	15 psf
½“ Cover Board	2 psf
3” Roof Deck	3 psf
Framing	10 psf
MEP Allowance	5 psf
Total:	35 psf

Lateral Loads

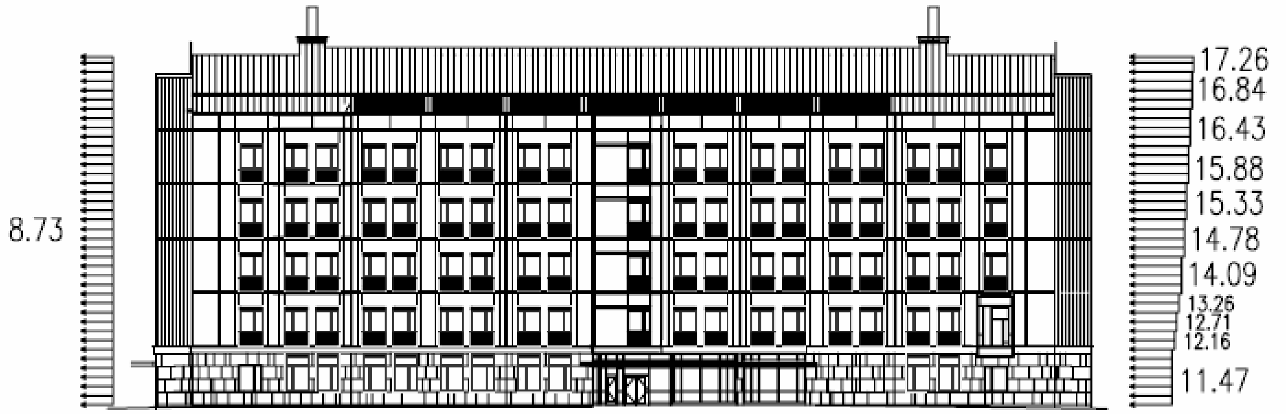
Wind

The wind load calculation was determined using Method 2 – Analytical Procedure as outlined in ASCE 7-02. In doing this, pressures were found at various height intervals. Using tributary area, the effective wind load at each floor could be determined, as well as the overall base shear and overturning moment on the building. An important assumption was neglecting the effects of the adjacent Wilmot Building. In doing this, I was conservative. This building acts as a shield, causing no leeward wind pressure with wind blowing from the north, and no windward pressure plus a reduction in leeward pressure with wind blowing from the south. In the east-west direction, this building causes similar effects, although to a much lesser scale, since it only lies on a small percentage of the BME/O footprint in this direction. The reason for neglecting these effects was for conservative purposes, and more importantly for future considerations. BME/O was built as a stand-alone building. Neglecting the effects of Wilmot allows for BME/O to remain an independent building regardless of future plans for the U of R campus. It was determined that seismic loading controlled in the north-south direction anyway.

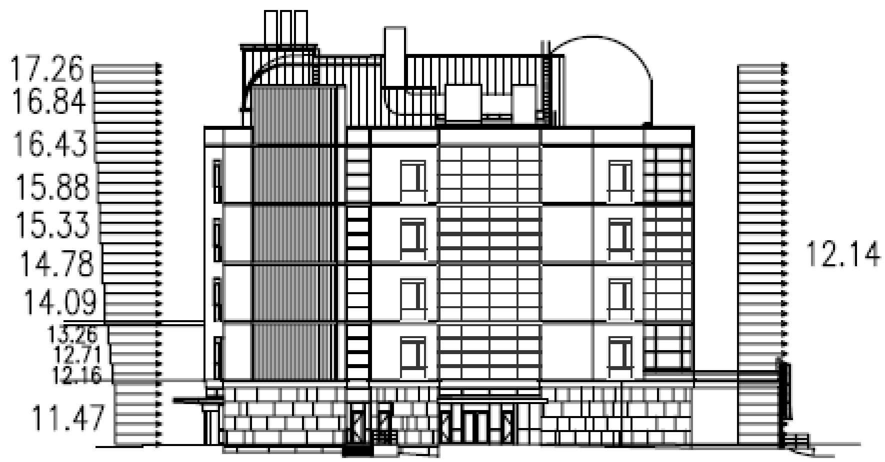
Below are the wind pressures and distribution determined through my calculations. For a more detailed analysis, see Appendix A.

Basic Wind Speed V = 90 mph
Exposure Category B
Importance Factor I =1.15

	North-South			East-West		
Height (z)	**p (Windward)**	**p (Leeward)**	**p (Total)**	**p (Windward)**	**p (Leeward)**	**p (Total)**
0-15	11.47	-8.73	**20.20**	11.47	-12.14	**23.61**
20	12.16	-8.73	**20.89**	12.16	-12.14	**24.30**
25	12.71	-8.73	**21.44**	12.71	-12.14	**24.85**
30	13.26	-8.73	**21.99**	13.26	-12.14	**25.40**
40	14.09	-8.73	**22.82**	14.09	-12.14	**26.23**
50	14.78	-8.73	**23.51**	14.78	-12.14	**26.92**
60	15.33	-8.73	**24.06**	15.33	-12.14	**27.47**
70	15.88	-8.73	**24.61**	15.88	-12.14	**28.02**
80	16.43	-8.73	**25.16**	16.43	-12.14	**28.57**
90	16.84	-8.73	**25.57**	16.84	-12.14	**28.99**
100	17.26	-8.73	**25.99**	17.26	-12.14	**29.40**



Wind Pressures, North-South Direction



Wind Pressures, East-West Direction

Seismic

The Equivalent Lateral Force Procedure, as outlined in ASCE 7-02, was used in determining the seismic loading for this building. The total dead load weights, along with partition allowances, were determined for each floor. From this, the distribution of seismic load to each floor could be determined and modeled as a lateral force. Again, base shear in each direction was calculated. Seismic load was found to control in the north-south direction, and wind load controls east-west. For detailed calculations, see Appendix B.

Seismic Use Group:	II
Importance Factor:	I = 1.25
Equivalent Seismic Weight:	12,000 kips
North-South:	Ordinary Moment Frames R = 3.5 Cs = 0.066 V = 800 kips
East-West:	Concentric Braced Frames R = 5 Cs = 0.046 V = 550 kips

Spot Checks

Beam

A typical composite beam was checked with the design loads I determined in order to analyze and draw conclusions about the structural design technique. The beam is located at the northwest end of the building in laboratory space, with a span of 26'-6" and a typical spacing of 10'-6". In performing a design analysis, I determined a non-composite W18x35 beam to be sufficient in strength to resist the moment. However, the deflection criteria of L/360 for live load and L/240 for total load, with a maximum of 1" were not met. It was concluded that deflection controlled this member, which is probably the reason for use of composite action. The engineer decided it would be better to use the concrete slab to aid in strength rather than increasing beam size to reduce deflection. I therefore did an analysis of a composite beam with a constraint on moment of inertia to meet deflection criteria. An added deflection constraint of L/280 or 1" in the beam for construction load was also used for the period before the concrete is cured and composite action can occur. The design I found was a W18x35 with 16 shear studs required. The actual design was the same, only with 24 studs. Upon further investigation, it appears the engineer or detailer may have used a minimum of 1 stud per foot in all composite beams. For calculations, see Appendix C.

Column

A typical column at location L2 was also checked. This column has a height of 16' supporting the second floor. Gravity loads were accumulated using tributary areas from the 2nd-5th floors and mechanical penthouse level. Since the columns supporting the high roof do not line up, I calculated the load on these columns and distributed it through a transfer girder to the column in question. Upon completion, I determined the required size to be a W12x79, just a few sizes smaller than the actual W12x96. As noted in the calculations, the engineer may have conservatively assumed the base of the column to be a pin connection rather than fixed, resulting in a k value of 1.0, and a larger required column size. See Appendix D for calculations.

Lateral

A braced frame in the east-west direction was the final calculation made in this report. As noted earlier, I found wind load to control in this direction. In doing an analysis at the base of the frame, I found axial forces in the bracing members to be 226 kips, resulting in HSS 7x7x½ bracing members. This is the same as the actual design. An important conservative assumption was in the distribution of lateral load to the frame. There are 4 braced frames in this direction, but they are not all the same. Based on their layout and geometry, I made the conservative assumption that ½ of the lateral load was distributed to the frame in question, and the frame held. Advisement as to the actual distribution may be necessary for further analysis of the lateral system. See Appendix E for calculations and a rough frame plan layout.

Future Considerations

Some future considerations may be necessary as the project develops. One is the foundation system, especially as it relates to the existing steam tunnel. Another is the vibration criteria in laboratory areas. This will be especially important in making decisions about an alternative structural system. Also, there are "floating" stairs cantilevered into the atrium space which will affect my decision. Finally, the pedestrian bridge to the CSB building uses firetrol columns, and further analysis may be necessary.

APPENDIX A: WIND CALCULATIONS

Basic Wind Speed V = 90 mph
Exposure Category B
Importance Factor I =1.15

		North-South				
Height (z)	**Kz**	**qz**	**qh**	**p (Windward)**	**p (Leeward)**	**p (Total)**
0-15	0.57	11.55	20.07	11.47	-8.73	**20.20**
20	0.62	12.57	20.07	12.16	-8.73	**20.89**
25	0.66	13.38	20.07	12.71	-8.73	**21.44**
30	0.7	14.19	20.07	13.26	-8.73	**21.99**
40	0.76	15.40	20.07	14.09	-8.73	**22.82**
50	0.81	16.42	20.07	14.78	-8.73	**23.51**
60	0.85	17.23	20.07	15.33	-8.73	**24.06**
70	0.89	18.04	20.07	15.88	-8.73	**24.61**
80	0.93	18.85	20.07	16.43	-8.73	**25.16**
90	0.96	19.46	20.07	16.84	-8.73	**25.57**
100	0.99	20.07	20.07	17.26	-8.73	**25.99**

		East-West				
Height (z)	**Kz**	**qz**	**qh**	**p (Windward)**	**p (Leeward)**	**p (Total)**
0-15	0.57	11.55	20.07	11.47	-12.14	**23.61**
20	0.62	12.57	20.07	12.16	-12.14	**24.30**
25	0.66	13.38	20.07	12.71	-12.14	**24.85**
30	0.7	14.19	20.07	13.26	-12.14	**25.40**
40	0.76	15.40	20.07	14.09	-12.14	**26.23**
50	0.81	16.42	20.07	14.78	-12.14	**26.92**
60	0.85	17.23	20.07	15.33	-12.14	**27.47**
70	0.89	18.04	20.07	15.88	-12.14	**28.02**
80	0.93	18.85	20.07	16.43	-12.14	**28.57**
90	0.96	19.46	20.07	16.84	-12.14	**28.99**
100	0.99	20.07	20.07	17.26	-12.14	**29.40**

WIND FORCE DISTRIBUTION

NORTH - SOUTH

FLOOR	ELEV	TRIB HEIGHT	LOAD	
1	0'-0"	8'	(20.2 PSF)(8')(138') =	22.3^k
2	16'-0"	15.33'	[(20.2 PSF)(7') + (20.89 PSF)(5') + (21.44 PSF)(3.33')](138') =	43.8^k
3	30'-8"	14.67'	[(21.44 PSF)(1.67') + (21.99 PSF)(5') + (22.82 PSF)(8')](138') =	45.3^k
4	45'-4"	14.67'	[(22.82 PSF)(2') + (23.51 PSF)(10') + (24.06 PSF)(2.67')](138') =	47.6^k
5	60'-8"	14.67'	[(24.06 PSF)(7.33') + (24.61 PSF)(7.33')](138') =	49.2^k
6	74'-8"	17.33'	[(24.61 PSF)(2.67') + (25.16 PSF)(8.67')](138') + [(25.16 PSF)(1.33') + (25.57 PSF)(4.67')](85') =	52.2^k
ROOF	94'-8"	10'	[(25.57 PSF)(5.83') + (25.99 PSF)(4.67')](85') =	21.9^k

BASE SHEAR = 282^k

$$M = 22.3^k(8') + 43.8^k(16') + 45.3^k(30.67') + 47.6^k(45.33') + 49.2^k(60.67') + 52.2^k(74.67') + 21.9^k(94.67') = 13204'k$$

OVERTURNING MOMENT = 13,400'k

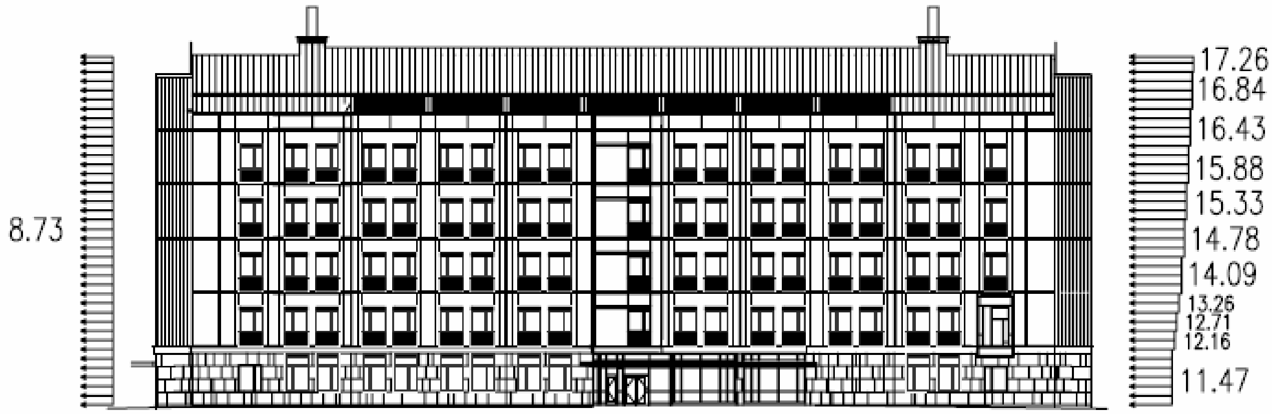
EAST-WEST

FLOOR	LOAD	
1	(23.61 PSF)(8')(254') =	48.0^k
2	[(23.61 PSF)(7') + (24.30 PSF)(5') + (24.85 PSF)(3.33')](254') =	93.9^k
3	[(24.85 PSF)(1.67') + (25.40 PSF)(5') + (26.23 PSF)(8')](254') =	96.1^k
4	[(26.23 PSF)(2') + (26.92 PSF)(10') + (27.47 PSF)(2.67')](254') =	100.3^k
5	[(27.47 PSF)(7.33') + (28.02 PSF)(7.33')](254') =	103.3^k
6	[(28.02 PSF)(2.67') + (28.57 PSF)(10') + (28.99 PSF)(4.67')](254') =	126.0^k
ROOF	[(28.99 PSF)(5.33') + (29.40 PSF)(4.67')](254') =	74.1^k

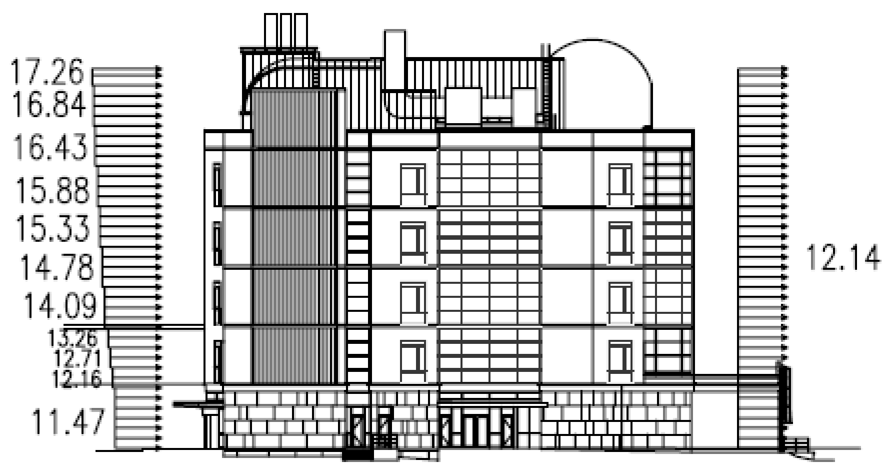
BASE SHEAR = 642^k

$$M = 48^k(8') + 93.9^k(16') + 96.1^k(30.67') + 100.3^k(45.33') + 103.3^k(60.67') + 126^k(74.67') + 74.1^k(94.67') = 32071'k$$

OVERTURNING MOMENT = 32,000'k



Wind Pressures, North-South Direction



Wind Pressures, East-West Direction

APPENDIX B: 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

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

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

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

EQUIVALENT SEISMIC WEIGHT

TYPICAL FLOOR 2-5

DEAD LOAD = 115 PSF AREA = 18,000 SQ FT
PARTITIONS = 20 PSF

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

PENTHOUSE

DEAD LOAD = 150 PSF ROOF DEAD LOAD + 20% SNOW = 40 PSF + 0.2(35 PSF) ≈ 50 PSF
FLOOR AREA = 9,000 FT^2 ROOF AREA = 7,000 FT^2

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

HIGH ROOF DEAD LOAD = 50 PSF
AREA = 9,000 FT^2

$w = (50\text{ PSF})(9000\text{ FT}^2) = 450^k$

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

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

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

APPENDIX C: COMPOSITE BEAM CHECK

SPOT CHECK - BEAM

THIRD FLOOR FRAMING, TYPICAL COMPOSITE BEAM AT WEST END

SPAN = 26.5'

SPACING = 10.5'

DEAD LOAD

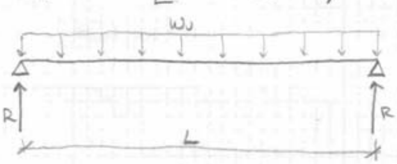
CONCRETE SLAB	150 PCF (7.5")(1/12)	= 94 PSF
METAL DECK		= 3 PSF
MEP		= 5 PSF
		≈ 105 PSF
BEAM SELF-WEIGHT - ASSUME		35 lb/ft

LIVE LOAD

LABORATORY SPACE 80 PSF

1.2D + 1.6L

$W_U = 1.2\left[(105\text{ PSF})(10.5') + 35\text{ PLF}\right] + 1.6(80\text{ PSF})(10.5') = 2.7\ ^k/ft$



REACTIONS

$R = \frac{W_U L}{2} = \frac{(2.7\ ^k/ft)(26.5')}{2} = 35.8^k$

MAX MOMENT

$M_U = \frac{W_U L^2}{8} = \frac{(2.7\ ^k/ft)(26.5')}{8} = \boxed{237'k}$

COMPOSITE BEAM DESIGN BASED ON ANSI/AISC 360-05 AND 13TH EDITION STEEL CONSTRUCTION MANUAL

ASSUME $a = 1.0$ $Y2 = 7.5" - \frac{1.0"}{2} = 7"$

SHEAR STUDS: NORMAL WT. CONCRETE $f'_c = 4$ ksi DECK SPANS ⊥ TO BEAM 3/4" DIA STUDS 17.2 k/STUD

* ACTUAL BEAM DESIGN IS COMPOSITE W18×35. SINCE THIS SIZE WORKS AS NON-COMPOSITE ($\phi M_P = 249'k$), AND RELATIVELY LONG SPAN, ASSUME DEFLECTION CONTROLS

DEFLECTION

$L/360 = \frac{26.5'(12)}{360} = 0.88"$ $L/240 = 1.3"$ USE 1.0" $L/280 = 1.1"$ USE 1.0"

LIVE LOAD $\Delta_{LL} = \frac{5wL^4}{384EI}$ $I \geq \frac{5wL^4}{384E\Delta_{LL}} = \frac{5(80\text{ PSF})(10.5')(26.5')^4(1728)}{384(29000\text{ ksi})(0.88")(1000)} = 364\text{ IN}^4$

TOTAL LOAD $I \geq \frac{5(80+110)(10.5')(26.5')^4(1728)}{384(29{,}000)(1")(1000)} = \boxed{763\text{ IN}^4}$ (COMPOSITE)

CONSTRUCTION - $I \geq \frac{5[94\text{ PSF}(10.5') + 35\ lb/ft](26.5')^4(1728)}{384(29{,}000)(1")(1000)} = \boxed{391\text{ IN}^4}$ (NON-COMPOSITE)

TRY W16×36

$I_{NON\text{-}COMPOSITE} = 448^{IN^4} > 391^{IN^4}$ CONSTRUCTION DEFLECTION OK

$I_{COMPOSITE} = 919^{IN^4} > 763^{IN^4}$ TOTAL LOAD DEFLECTION OK

$\phi M_n = 366'k > M_u = 237'k$ STRENGTH OK

$\Sigma Q_n = 210^k \rightarrow$ 26 STUDS

EQUIV. WEIGHT = $36^{lb/ft}(26.5') + 26\,(10^{lb/stud}) = 1214$

W18×35

$I_{NON\text{-}COMPOSITE} = 510 > 391$ OK

$I_{COMPOSITE} = 1030 > 763$ OK

$\phi M_n = 382'k > 237'k$ OK

$\Sigma Q_n = 129^k \rightarrow$ 16 STUDS

EQUIV. WEIGHT = $35(26.5) + 16(10) = \underline{1088}$

CHECK ASSUMPTIONS

$b_{eff} \le \frac{26.5'}{4} = \underline{80''}$

$b_{eff} \le 10.5' = 126''$

$$a = \frac{A_s F_y}{0.85 f'_c b_{eff}} = \frac{(10.3\,in^2)(50^{ksi})}{0.85(4^{ksi})(80'')} = 1.89'' \Rightarrow \text{USE } a = 2'',\ Y2 = 6.5''$$

W18×35

$I = 996^{IN^4}$ @ $\Sigma Q_n = 129^k$ (16 STUDS)

$\phi M_n = 377'k$ OK

$\boxed{\text{USE W18×35, (16) 3/4" DIAMETER SHEAR STUDS}}$

ACTUAL DESIGN: W18×35 + (24) 3/4"φ STUDS

APPENDIX D: COLUMN CHECK

SPOT CHECK - COLUMN

COLUMN L2 @ FIRST FLOOR

INTERIOR STEEL COLUMN

TRIB. AREA $= (21')\left(\frac{26.5'}{2} + \frac{23'}{2}\right) = 520\ \text{FT}^2$

TYPICAL LOAD FLOORS 2-5

DEAD:	SLAB	94 PSF	LIVE: OFFICE/LAB	80 PSF
	DECK	3 PSF		
	MEP	5 PSF		
	FRAMING	10 PSF		
		115 PSF		

$[1.2(115\ \text{PSF}) + 1.6(80\ \text{PSF})](4)(520\ \text{FT}^2) = \underline{553^k}$

MECH. PENTHOUSE LOAD

DEAD: 115 PSF LIVE: 150 PSF

$[1.2(115\ \text{PSF}) + 1.6(150\ \text{PSF})](520\ \text{FT}^2) = \underline{197^k}$

ROOF

*COLUMN NOT CONTINUOUS TO ROOF. TRANSFER GIRDER CARRIES LOAD FROM W8×31 COLUMN (L3)

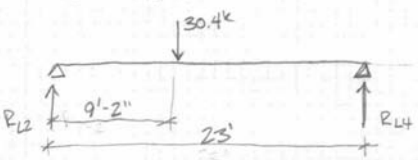
COLUMN L3

TRIB AREA $= \left(\frac{26.25'}{2} + \frac{12.83'}{2}\right)\left(\frac{31.67'}{2}\right) = 310\ \text{FT}^2$

DEAD:	FRAMING	10 PSF	SNOW: 35 PSF
	DECK	3 PSF	
	BUILT-UP ROOF	15 PSF	
	1/2" COVER BOARD	2 PSF	
	MEP	5 PSF	
		35 PSF	

$[1.2(35\ \text{PSF}) + 1.6(35\ \text{PSF})](310\ \text{FT}^2) = 30.4^k$

TRANSFER GIRDER

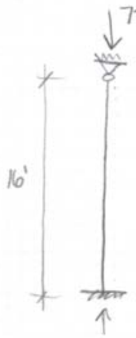


$\Sigma M = 0$

$(30.4^k)(13.83') - R_{L2}(23') = 0$

$R_{L2} = \underline{18.3^k}$

TOTAL AXIAL COLUMN LOAD = 770^k



$K_x = K_y = 0.80$ (RECOMMENDED – AISC MANUAL TABLE C-C2.1)

$L = 16'$

$K_y L_y = 12.8'$

TABLE 4-2

$\boxed{W12 \times 79}$ $\phi P_n = 814^k$

ACTUAL DESIGN: W12x96

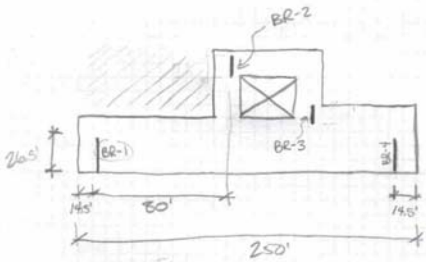
NOTE: CONSERVATIVELY ASSUMING $K = 1.0$ ($KL_y = 16'$)
GIVES W12x87 ($\phi P_n = 817^k$), CLOSER TO ACTUAL DESIGN

APPENDIX E: BRACED FRAME CHECK

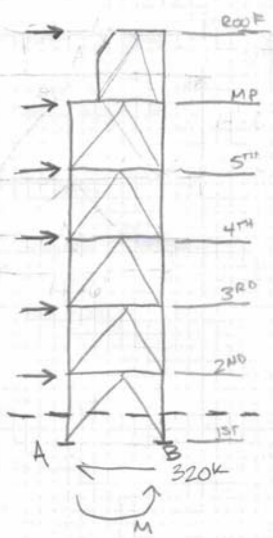
LATERAL CHECK

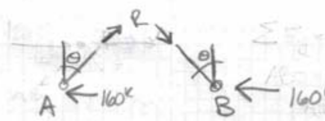
BRACED FRAME BR-1 (WIND CONTROLS IN THIS DIRECTION)
BETWEEN 1ST AND 2ND FLOORS
$V = 642^k$

NOTE: THERE ARE 4 BRACED FRAMES IN E-W DIRECTION AT THIS LEVEL



BASED ON GEOMETRY,
CONSERVATIVELY USE $V/2 = 320^k$



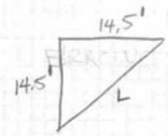


$\theta = 45°$

$R = \frac{160}{\cos 45°}$

$R = 226^k$

BRACING REACTION $= \pm 226^k$



$L = 20.5'$
$k = 1.0$
$kL = 20.5$

TABLE 4-6 → USE $\boxed{\text{HSS } 7 \times 7 \times 1/2}$
$\phi P_n = 252^k$

ACTUAL BRACING DESIGN: HSS $7 \times 7 \times \frac{1}{2}$
NOTE: USING $V/2$ WAS VERY CONSERVATIVE