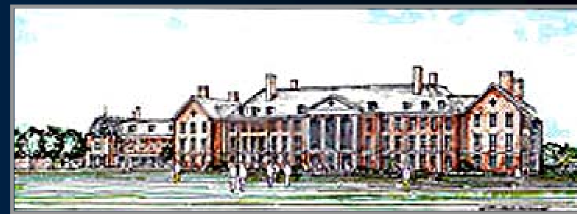
Senior Thesis Project
Department of Architectural Engineering
The Pennsylvania State University
Spring 2006

Ann & Richard Barshinger Life Science & Philosophy Building
Franklin & Marshall College – Lancaster, PA

Michael A. Hebert
Structural Option
Linda Hanagan, PhD, P.E. (Advisor)

Barshinger Life Science & Philospohy Building

Lancaster, Pennsylvania

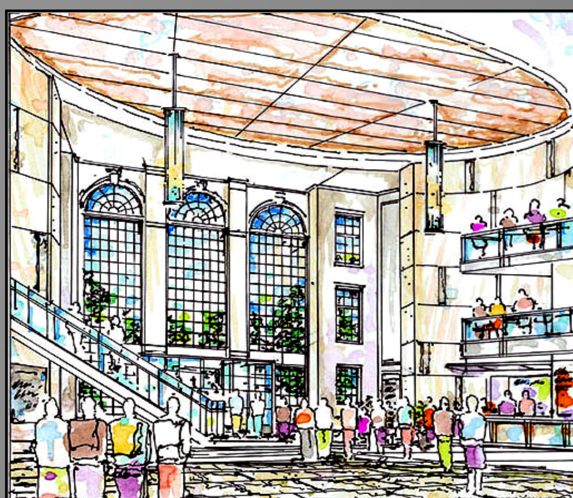


Project Team

- Owner - Franklin & Marshall College
- Architect - Einhorn Yaffee Prescott
- Structural & MEP - Einhorn Yaffee Prescott
- General Contractor - Turner Construction

Project Info

- Size - 100,000 GSF
- Number of Stories - 3+
- Total Project Cost - $40 million
- Construction Dates - Oct. 2005–June 2007
- Delivery Method - Design-Bid-Build
- Contract Type - Guaranteed Max Price

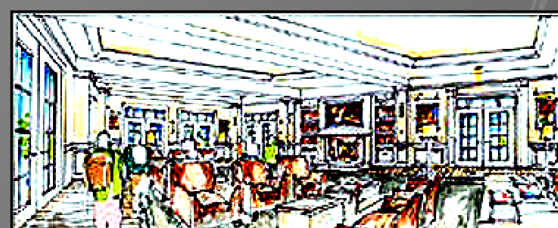


Architecture

- Colonial-Revival Style.
- Red brick façade w/ slate shingled roofs.
- 3-story central atrium featuring grand staircase, café, soft seating, study tables, and direct entry to departmental offices.
- Faculty offices, classrooms, teaching labs, research labs, vivarium, lecture hall, and Humanities Common Room.

Structural

- Conventional framing with composite decking and composite wide-flange beams.
- Wide-flange columns supported by concrete piers and shallow foundations.
- The lateral force resisting system is composed of steel concentrically braced frames in the main building and moment frames in the vivarium wing.
- 14-foot typical floor-to-floor heights.



Mechanical

- Steam and chilled water supplied by the campus's Central Utility Plant.
- Two double-wall custom AHUs with VAV controls serve the main building.
- One dedicated double-wall 100% outdoor air AHU serves the vivarium.

Electrical

- Substation with a 2000/2666kVA, 12.47kVA to 480/277V, 3-phase, 4-wire dry transformer and a 4000A, 277/480V distribution.
- 208/120V local receptacle panels.
- 480/277V lighting panels.

Michael A. Hebert
Structural Option

http://www.arche.psu.edu/thesis/eportfolio/current/portfolios/mah409/

FRANKLIN & MARSHALL

EYP/ Einhorn Yaffee Prescott Architecture & Engineering P.C.

Senior Thesis Project
Department of Architectural Engineering
Spring 2006

Table of Contents

Acknowledgements

I would like to thank all the people that helped make this project possible, especially:

My parents, Dorothy Brayley and Andy Hebert, for all their love and support.

Frank Lancaster, PE, RA and Andy Hebert, PE, LEED of Einhorn, Yaffee, Prescott Architecture & Engineering P.C. for providing design documents and an abundance of helpful advice.

My professors, Linda Hanagan, PhD, PE and Kevin Parfitt, PE, for their aid and expertise.

My friends, for making Penn State my home for five years.

My fellow classmates, for sharing the journey.

1.0 Executive Summary

The Ann and Richard Barshinger Life Science and Philosophy Building is designed to house the life science and philosophy departments in a conscious effort to "encourage interdisciplinary interactions between the natural sciences and the humanities" at Franklin and Marshall College in historic Lancaster, Pennsylvania. The three-story, 100,000 gross square foot Colonial Revival style structure has quite a status to live up to. I made it my goal for this project to develop more efficient structural systems for this locally important building, while also finding an opportunity to satisfy my curiosity when presented with unfamiliar structural devices.

I utilized numerous resources including course textbooks, nationally recognized design aids, and the structural modeling software *SAP2000* and *RAM Structural Systems* to fulfill my project goal. The lateral force resisting system was streamlined for efficiency. A new foundation system of drilled concrete piers was designed to carry the building loads directly into the intact limestone less than 25-feet below finish grades. The structural device that is the Vierendeel truss was evaluated based on the design potential of the best alternative. After the designs were complete, I reached into construction management, architecture, and HVAC ductwork design to properly compare the new systems with their existing counterparts. In the end, I drew the following conclusions from my results:

- The existing lateral force resisting system of ten concentrically braced frames is oversized for the calculated seismic loads.

- Replacing traditional spread footings with drilled concrete piers is not cost effective. Although the geotechnical engineers mapped the intact rock depth as enticingly close to the planned ground floor level, the drilled pier system ($325,000) was estimated to cost twice as much as the spread footing system ($163,000). However, the drilled pier system did reduce steel reinforcing quantities by 24% and concrete consumption by 35%. Geotechnical investigation is an extremely difficult job to complete accurately. As a result, excavation contractors are wary of deep foundation systems and charge accordingly.

- The Vierendeel truss is extremely effective and efficient for carrying significant loads over a large span. It can be manipulated to accommodate various configurations of rectangular openings, such as windows in a façade. A system of long span steel joists can be designed as an alternative to the rigidly-connected and weighty Vierendeel truss. However, the long span joists required bay in-fill members that were significantly deeper than those used with the truss. Those deeper members created a heavily congested above-ceiling plenum space and required that some HVAC ducts be resized to preserve the prescribed ceiling height.

- HVAC ducts can be flattened out without increasing friction loss or system energy consumption by simply maintaining the hydraulic diameter.

- For each individual building system that is changed, ten other systems are subsequently affected…

2.0 Existing Building Description

2.1 The Building Program

The Ann and Richard Barshinger Life Science and Philosophy Building is the largest construction project in the long history of Lancaster, Pennsylvania's Franklin and Marshall College. The three-story Georgian Revival structure will house the departments of biology, psychology, and philosophy, as well as two interdisciplinary programs in biological foundations of behavior and scientific and philosophical students of mind. At a total cost of $45 million, the 102,000 square-foot building will include state-of-the-art classrooms and laboratories, a greenhouse, a multi-story atrium, a 125-seat lecture hall, a Humanities Common Room for meetings and gatherings, and a basement vivarium for the study of primates, rodents, and other small animals.

2.2 Superstructure

The building superstructure is comprised of composite slab-on-deck in combination with composite wide-flange steel beams supported by wide-flange columns bearing on concrete piers and shallow footings. The framing system is divided into approximately 20'x30' bays. Floor-to-floor heights are typically found to be 14-feet. A typical floor frame consists of 2-inch composite metal deck with 4 ½-inches of normal weight concrete above the flutes. The composite slab is then carried by W16x26 filler beams spaced 7-feet apart. Interior girders, of size W18x40, are typically carried by W12x65 columns, sized for ease of fabrication and erection considering the OSHA-required four anchor bolt pier connection.

2.3 Lateral Force Resisting Systems

The structure's main lateral force resisting system is composed of ten concentrically braced steel frames of varying sizes. These frames utilize wide-flange shapes for the vertical and horizontal members with ½-inch thick HSS shapes for the diagonal braces. The ten frames are located throughout the structure according to the Figure 2.3.1 below. The basic structure of each frame is depicted in Figure 2.3.2 on the next page.

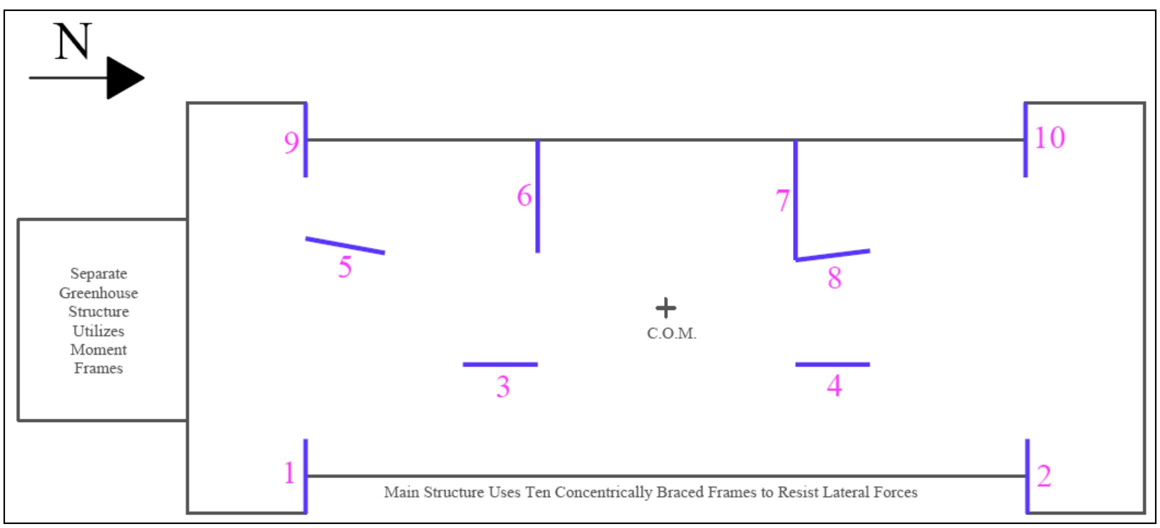


Figure 2.3.1 Layout of the 10 Concentrically Braced Frames

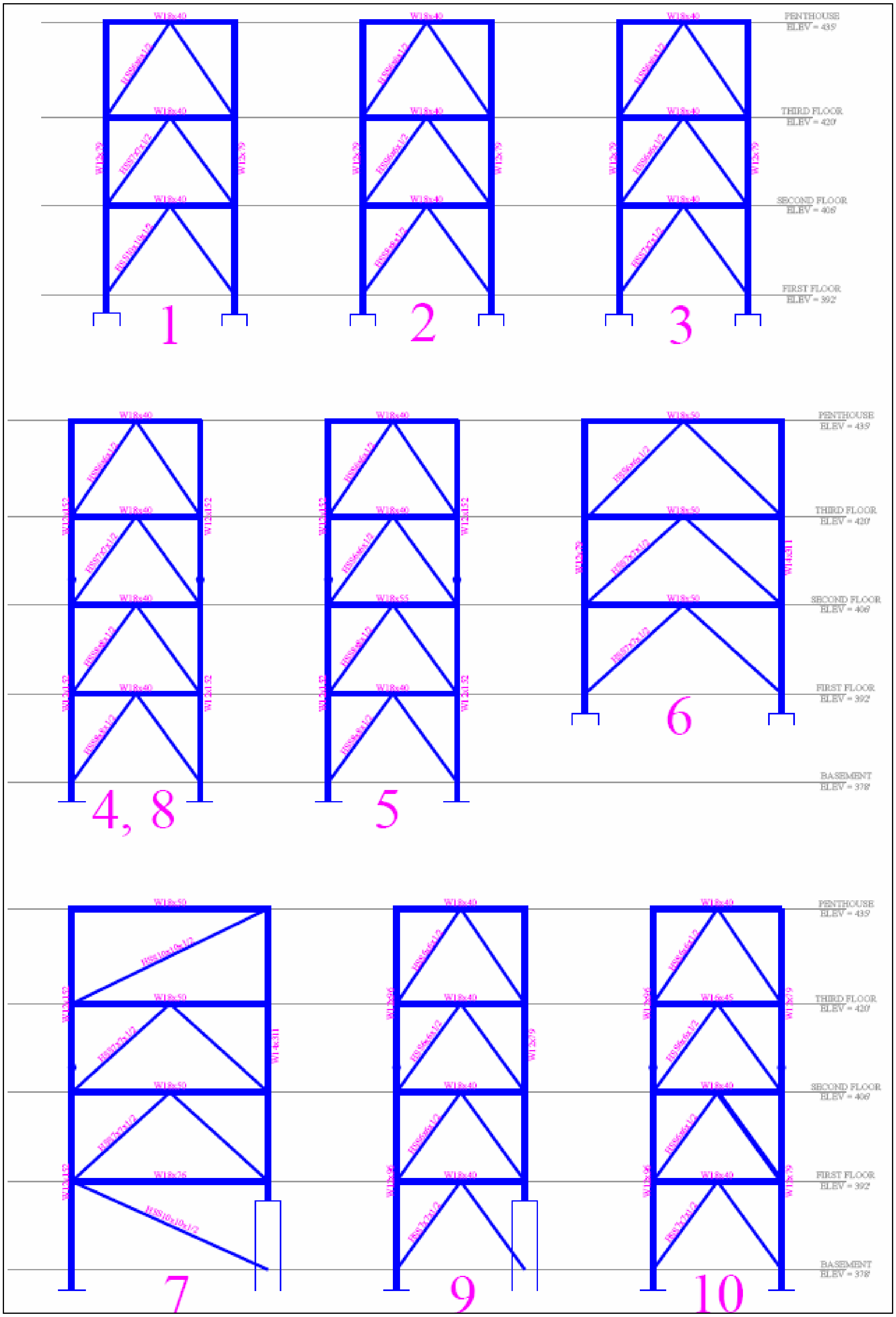


Figure 2.3.2 The 10 Concentrically Braced Frames in the Main Lateral Force Resisting System

The greenhouse wing on the building's southern exposure uses aluminum moment frames to resist the lateral forces. Large areas of glass were necessary to create the light, airy, and habitable space necessary for its greenhouse function. Moment frames were chosen over of the clumsier-looking braced frames due to the glass requirements as well as the lightweight nature of the structure that includes a glass and an aluminum-framed barrel roof. The greenhouse wing is separated from the main building by an expansion joint in order to keep the lateral resisting systems separate.

2.4 Foundations

The superstructure of the Barshinger Building rests upon shallow foundations, specifically spread footings. In the geotechnical report for the site, Advanced GeoServices Corp. of West Chester, Pennsylvania recommended that the foundations not exceed an allowable bearing pressure of 3,000 pounds per square foot (psf). Large footings will be necessary to transfer the loads from the braced frames into the ground and to resist the potential overturning moments. Test borings encountered intact rock at depths ranging from 3 to 23.5 feet. The intended construction method will involve excavating the rock where necessary and supplying a soil cushion beneath the footings in the excavated areas to discourage issues with differential settlement.

2.5 Cladding

The building employs a relatively heavy cladding system. The red brick façade is backed by concrete masonry units and certainly increased the seismic design loads on the structure. However, the cladding system is consistent with all of the other buildings on the Franklin and Marshall College campus.

2.6 Unique Structural Feature – Vierendeel Truss

The building has one peculiar structural feature: a Vierendeel truss. This statically indeterminate truss is comprised of rigid upper and lower girders, connected by vertical beams using rigid joints. The configuration of elements creates bending moments in all the members under gravity loading. Trusses of this type are found in some bridges, and were also used in the frame of the World Trade Center's Twin Towers. The vertical beams create regular openings for rectangular windows in the western facade. The truss, illustrated in Figure 2.6.1, spans nearly 70-feet over the large 125-seat lecture hall to create an open and uninterrupted space for the audience to enjoy. However, the truss requires exceptionally large wide-flange members that could present difficult erection issues for the contractor, including the need for a special crane that is larger than necessary for the rest of the job.

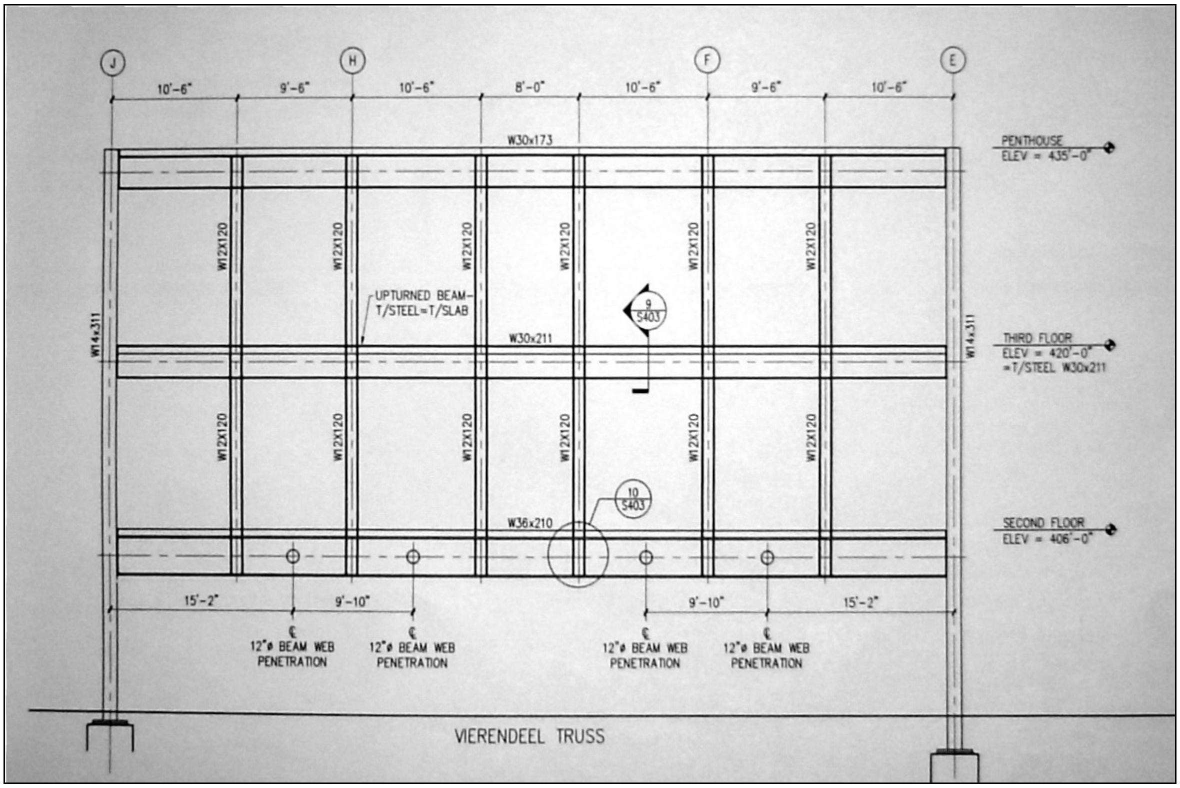


Figure 2.6.1 Vierendeel Truss

2.7 Material Strengths

The desired material strengths listed below in Figure 2.7.1 have been taken from the General Notes page of the Structural Drawings provided by Einhorn Yaffee Prescott, PC (EYP).

Concrete	f'_c	Unit Weight
Footings	3000 psi	150 pcf
Foundation Walls, Piers	4000 psi	150 pcf
Concrete on Metal Deck (Floor)	3500 psi	150 pcf
Concrete on Metal Deck (Roof)	3500 psi	150 pcf
Slabs on Grade	3500 psi	150 pcf
All Other Concrete	4000 psi	150 pcf
Reinforcing		
Typical Bars	ASTM A615	Grade 60
Welded Bars	ASTM A706	Grade 60
Welded Wire Fabric	ASTM A185	
Metal Deck Properties		
Roof Deck	3" Type "N"	20-gage
Composite Floor Deck	2" Type "B"	18-gage
Steel Members		
Wide-Flange Shapes	ASTM A992	
Channels & Angles	ASTM A36	
Pipe	ASTM A53	Grade B
Tubular Shapes	ASTM A500	Grade B
Base Plates	ASTM A36	

All Other Steel Members	ASTM A36
Steel Connections	
High Strength Bolts	ASTM A325 or A490
Nuts & Washers	(Min. ¾" Diameter)
Anchor Rods	ASTM F-1554 Grade 55
Welding Electrode	E70XX
Metal Deck Welding Electrode	E60XX Min.
Masonry Properties	
Mortar	Type S
CMU Strength	F'_m = 1500 psi

Figure 2.7.1 Material Strengths & Properties for Design

2.8 Major Design Codes & Standards

The Barshinger Life Science and Philosophy Building was designed using the following major design codes and standards.

- International Building Code (IBC), 2000
- ASCE 7-98
- ACI 315 "Manual of Standard Practice for Detailing Reinforced Concrete Structures"
- ACI 318 "Building Code Requirements for Reinforced Concrete"
- ACI 530 "Building Code Requirement for Masonry Structures"
- ACI 531 "Specifications for Masonry Structures"
- AISC "Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings"

2.9 Design Loads

Design building loads were obtained from the General Notes page of the Structural Drawings provided by EYP. However, I also verified the values through simplified calculations using ASCE7-02 and the International Building Code (IBC) 2000 to determine the live, dead, snow, wind, and seismic loads acting on the building. The resulting load values are summarized in Figure 2.9.1 below. The verifying calculations are available for review in Appendix A.

Live	Offices	50 psf (+20 psf partitions)
	Laboratories	60 psf
	Public Spaces	100 psf
Dead	Floor Loads	120 psf
	Exterior Walls	45 psf
Snow	Flat Roof	25 psf
	Sloped Roof	28 psf
Wind	N-S Base Shear	65.5 k
	E-W Base Shear	143.2 k
Seismic	Base Shear	895 k

Figure 2.9.1 Building Loads for Design

3.0 Project Proposal

3.1 Goal

The stated goal of this project is to improve the efficiency of certain aspects of the structural system.

3.2 Depth Analysis

The depth analysis for this project will primarily investigate the new design options for structural system components. I hope to streamline the lateral force resisting system through a reduction in the number of braced frames necessary to resist the calculated environmental loads. I will also explore an alternative foundation system of concrete caissons drilled into rock. In addition, I will attempt to design an alternative system to replace the Vierendeel truss.

3.3 Breadth Analyses

The two breadth analyses will focus on assessing the impact of the depth analysis results on construction management, architectural integrity, and mechanical systems. The streamlined lateral force resisting system and the alternative foundation system will be evaluated based on raw material quantities and cost impact using RS Means 2006. The truss replacement will be assessed additionally for its architectural and mechanical impact on the building.

3.4 Task Breakdown & Methodology

Lateral Force Resisting System Efficiency Evaluation/Alteration

- Develop improved structural models of the existing braced frames using SAP 2000 computer software. Create new models as necessary for any altered/new braced frame configurations.
- Create Excel spreadsheets to analyze the efficiency of the existing and altered systems.
- Determine foundation requirements for the altered system using hand calculations and Excel spreadsheets.

Foundation Systems

- Research the design of drilled caisson foundations.
- Redesign the current foundations to include any changes made to the lateral system using hand calculations, Excel spreadsheets, and my foundations textbook.

Spanning the Lecture Hall - Vierendeel Truss Options

- Research the origin and use of Vierendeel trusses.
- Explore potential options including, but not limited to, long span steel joists and triangular trusses.

Constructability Management

- Determine quantities of steel, concrete, and excavation material from the depth analyses.
- Calculate cost impact using R.S. Means 2006.
- Research general construction issues related to the existing and new designs.
- Compare the existing and new designs.

Architectural/Mechanical Impact

- Create AutoCAD drawings of the exterior façade that is impacted by the Vierendeel truss and the alternative options.
- Assess the impacts to the both interior and exterior appearances.

4.0 Depth Analysis – Lateral Force Resisting System

4.1 Existing System – Braced Frames

The existing lateral force resisting system was previously assessed for its load carrying capacity and potential for improvement. As described in Section 2.3, the existing system is composed of ten concentrically braced frames spaced throughout the building. Computer models were created and analyzed using SAP2000 to determine the characteristic stiffness of each frame. This information was dumped into an Excel spreadsheet (Figure 4.1.1) to distribute the seismic base shear to the individual frames according to the equivalent lateral force method as described in ASCE 7-02. The SAP2000 models are not provided in this report.

To further deconstruct the braced frames, I distributed the lateral story forces to the diagonal bracing members using an Excel spreadsheet. The members were checked for allowable compression and tension strengths using the design tools in the *Manual of Steel Construction: Load and Resistance Factored Design (LRFD), 3rd Edition* published by the American Institute for Steel Construction (AISC). In addition, total story drift was calculated using design procedures described in *The Seismic Design Handbook, 2nd Edition* by Farzad Naeim for undamped Multi-Degree of Freedom (MDOF) systems under static loading. Stiffness matrices (Appendix B) were created from the calculated axial stiffness values of the bracing members. The results of the force distribution, allowable strength comparisons, and total story drifts are available for review in Figure 4.1.2.

Upon review, the existing system was adequate to resist the calculated seismic load. Overall, the capacity of the system is underutilized and presents the opportunity for streamlining, which is described in the next section.

Base Shear	895	k	
Torsion (E-W)	-30445	in-k	$M = V^*e_x$
Torsion (N-S)	-13508	in-k	$M = V^*e_y$
H/400 Drift Limit	1.29	in	

C_{s1}	0.230
C_{s2}	0.444
C_{s3}	0.325

h_1	168	in
h_2	336	in
h_3	516	in

Shear Distribution by Equivalent Lateral Force Method

Frame	Direction	x-coord. (in.)	y-coord. (in.)	in/kip (STAAD)	k (k/in)	% Direct Load	Direct Shear (k)	d (in)	k*d	k^*d^2	$\frac{(kd)}{SUM(kd^2)}$	Eccentric Shear (k)	Total Shear (k)	Overturning Moment (in-k)
1	E-W	380	-	0.00437	228.78	10.83%	96.9	1126	257603	290056869	0.0002	-5.9	96.9	34480
2	E-W	2700	-	0.00466	214.55	10.15%	90.9	1194	256172	305873351	0.0002	-5.9	90.9	32335
6	E-W	1126	-	0.00191	523.56	24.78%	221.8	380	198944	75595481	0.0001	-4.6	221.8	78907
7	E-W	1954	-	0.00127	784.93	37.15%	332.5	448	351661	157550150	0.0003	-8.1	332.5	118299
9	E-W	380	-	0.00398	251.00	11.88%	106.3	1126	282626	318232574	0.0002	-6.5	106.3	37830
10	E-W	2700	-	0.00909	110.00	5.21%	46.6	1194	131341	156822757	0.0001	-3.0	46.6	16578
3	N-S	-	480	0.00452	221.24	30.28%	271.0	167	36926	6163296	0.0000	-0.4	271.0	96422
4	N-S	-	480	0.00586	170.65	23.36%	209.0	167	28483	4753942	0.0000	-0.3	209.0	74373
5	N-S	-	840	0.00595	168.10	23.01%	205.9	193	32458	6267395	0.0000	-0.3	205.9	73261
8	N-S	-	840	0.00586	170.65	23.36%	209.0	193	32951	6362582	0.0000	-0.3	209.0	74373
	C.O.R.	1506	647											
	C.O.M.	1540	662											
	e	-34	-15											

Figure 4.1.1 Seismic Base Shear Distribution According to the Equivalent Lateral Force Method

Diagonal Bracing Member Forces

Frame	Story Level	HSS Brace Size	Length (in)	Area (in^2)	cos(theta)	Member Stiffness (k/in)	Story Shear (k)	Brace Axial Force (k)	Allowable Tensile Strength (k)	Allowable Compress. Strength (k)	Strength Design Efficiency	Total Story Drift (in)
1	1	10x10x$^1/_2$	206	17.2	0.581	816.2	22.3	83.4	712.1	557	15.0%	0.12
	2	7x7x$^1/_2$	206	11.6	0.581	550.5	43.1	64.2	480.2	303	21.2%	0.25
	3	6x6x$^1/_2$	216	9.74	0.555	401.7	31.5	28.4	403.2	203	14.0%	0.33
2	1	8x8x$^1/_2$	206	13.5	0.581	640.6	20.9	78.2	558.9	390	20.0%	0.14
	2	6x6x$^1/_2$	206	9.74	0.581	462.2	40.4	60.2	403.2	217	27.7%	0.29
	3	6x6x$^1/_2$	216	9.74	0.555	401.7	29.5	26.6	403.2	203	13.1%	0.37
3	1	7x7x$^1/_2$	206	11.6	0.581	550.5	62.5	233.1	480.2	303	76.9%	0.49
	2	6x6x$^1/_2$	206	9.74	0.581	462.2	120.4	179.4	403.2	217	82.7%	0.94
	3	6x6x$^1/_2$	216	9.74	0.555	401.7	88.1	79.4	403.2	203	39.1%	1.16
4	1	8x8x$^1/_2$	206	13.5	0.581	640.6	48.2	179.8	558.9	390	46.1%	0.33
	2	7x7x$^1/_2$	206	11.6	0.581	550.5	92.9	138.4	480.2	303	45.7%	0.62
	3	6x6x$^1/_2$	216	9.74	0.555	401.7	67.9	61.2	403.2	203	30.2%	0.79
5	1	8x8x$^1/_2$	206	13.5	0.581	640.6	47.5	177.1	558.9	390	45.4%	0.32
	2	6x6x$^1/_2$	206	9.74	0.581	462.2	91.5	136.3	403.2	217	62.8%	0.66
	3	6x6x$^1/_2$	216	9.74	0.555	401.7	66.9	60.3	403.2	203	29.7%	0.83
6	1	7x7x$^1/_2$	248	11.6	0.727	717.3	51.1	152.6	480.2	245	62.3%	0.31
	2	7x7x$^1/_2$	248	11.6	0.727	717.3	98.6	117.4	480.2	245	47.9%	0.55
	3	6x6x$^1/_2$	256	9.74	0.703	545.6	72.1	51.3	403.2	161	31.8%	0.68
7	1	7x7x$^1/_2$	248	11.6	0.727	717.3	76.6	228.8	480.2	245	93.4%	0.46
	2	7x7x$^1/_2$	248	11.6	0.727	717.3	147.8	176.0	480.2	245	71.9%	0.82
	3	10x10x$^1/_2$	406	17.2	0.443	241.4	108.1	243.8	712.1	317	76.9%	1.27
8	1	8x8x$^1/_2$	206	13.5	0.581	640.6	48.2	179.8	558.9	390	46.1%	0.33
	2	7x7x$^1/_2$	206	11.6	0.581	550.5	92.9	138.4	480.2	303	45.7%	0.62
	3	6x6x$^1/_2$	216	9.74	0.555	401.7	67.9	61.2	403.2	203	30.2%	0.79
9	1	6x6x$^1/_2$	206	9.74	0.581	462.2	24.5	91.5	403.2	217	42.1%	0.23
	2	6x6x$^1/_2$	206	9.74	0.581	462.2	47.3	70.4	403.2	217	32.4%	0.41
	3	6x6x$^1/_2$	216	9.74	0.555	401.7	34.6	31.2	403.2	203	15.3%	0.49
10	1	6x6x$^1/_2$	206	9.74	0.581	462.2	10.7	40.1	403.2	217	18.5%	0.10
	2	6x6x$^1/_2$	206	9.74	0.581	462.2	20.7	30.8	403.2	217	14.2%	0.18
	3	6x6x$^1/_2$	216	9.74	0.555	401.7	15.1	13.7	403.2	203	6.7%	0.22
						$(AE\cos^2\bullet)/L$			*$0.9F_yA_g$*	*LRFD Table 4-6*		*MDOF*

Figure 4.1.2 Diagonal Brace Member Force Distribution, Strength Design Efficiency, and Total Story Drift

4.2 Updated System – Less Frames

The review of the existing lateral force resisting system manifested the opportunity to streamline the existing system and create a new system with a more efficient use of member capacities and total drift limits. I adjusted the Excel spreadsheets from Figure 4.1.1 and Figure 4.1.2 through trial and error to find the best combination of frames and diagonal member sizes. Allowable member strengths were cut-off at 85% to provide some liberty for connection design. The resulting spreadsheets are shown in Figure 4.2.1 and Figure 4.2.2, while the new stiffness matrices are found in Appendix B.

The revised system involves the removal of four braced frames (Frames 4, 5, 9, and 10) and the alteration of three of the remaining frames (Frames 3, 7, and 8). The new frames were re-modeled in SAP2000 to determine the new characteristic stiffness. The reduction in the number of frames placed additional seismic loads on the remaining frames' foundations, but the existing spread footings have enough additional capacity to handle the increased loads satisfactorily.

Base Shear	895	k	
Torsion (E-W)	46847	in-k	$M = V^*e_x$
Torsion (N-S)	-15651	in-k	$M = V^*e_y$
H/400 Drift Limit	1.29	in	

C_{s1}	0.230
C_{s2}	0.444
C_{s3}	0.325

h_1	168	in
h_2	336	in
h_3	516	in

Shear Distribution by Equivalent Lateral Force Method

Frame	Direction	x-coord. (in.)	y-coord. (in.)	in/kip (SAP2000)	k (k/in)	% Direct Load	Direct Shear (k)	d (in)	k*d	k^*d^2	$\frac{(kd)}{SUM(kd^2)}$	Eccentric Shear (k)	Total Shear (k)	Overturning Moment (in-k)
1	E-W	380	-	0.00437	228.78	13.06%	116.9	1212	277361	336256017	0.0003	15.6	132.5	47142
2	E-W	2700	-	0.00466	214.55	12.25%	109.6	1108	237644	263227738	0.0003	13.4	123.0	43759
6	E-W	1126	-	0.00191	523.56	29.89%	267.5	466	244159	113861571	0.0003	13.7	281.2	100059
7	E-W	1954	-	0.00127	784.93	44.81%	401.0	362	283875	102665569	0.0003	16.0	417.0	148364
9	[redacted]													
10														
3	N-S	-	480	0.00369	271.30	54.30%	486.0	165	44632	7342499	0.0001	-0.8	486.0	172914
4	[redacted]													
5														
8	N-S	-	840	0.00438	228.31	45.70%	409.0	195	44632	8724945	0.0001	-0.8	409.0	145516
	C.O.R.	1592	645											
	C.O.M.	1540	662											
	e	52	-17											

Figure 4.2.1 Seismic Base Shear Distribution According to the Equivalent Lateral Force Method

Diagonal Bracing Member Forces

Frame	Story Level	HSS Brace Size	Length (in)	Area (in^2)	cos(theta)	Member Stiffness (k/in)	Story Shear (k)	Brace Axial Force (k)	Allowable Tensile Strength (k)	Allowable Compress. Strength (k)	Strength Design Efficiency	Total Story Drift (in)
1	1	10x10x1/2	206	17.2	0.581	816.2	30.5	114.0	712.1	557	20.5%	0.16
	2	7x7x1/2	206	11.6	0.581	550.5	58.9	87.7	480.2	303	28.9%	0.35
	3	6x6x1/2	216	9.74	0.555	401.7	43.1	38.8	403.2	203	19.1%	0.45
2	1	8x8x1/2	206	13.5	0.581	640.6	28.3	105.8	558.9	390	27.1%	0.19
	2	6x6x1/2	206	9.74	0.581	462.2	54.7	81.4	403.2	217	37.5%	0.40
	3	6x6x1/2	216	9.74	0.555	401.7	40.0	36.0	403.2	203	17.8%	0.50
3	1	10x10x1/2	206	17.2	0.581	816.2	112.0	418.1	712.1	557	75.1%	0.60
	2	10x10x1/2	206	17.2	0.581	816.2	216.0	321.7	712.1	557	57.8%	1.05
	3	10x10x1/2	216	17.2	0.555	709.4	158.0	142.4	712.1	557	25.6%	1.28
4	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
5	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
6	1	7x7x1/2	248	11.6	0.727	717.3	64.8	193.5	480.2	245	79.0%	0.39
	2	7x7x1/2	248	11.6	0.727	717.3	125.0	148.9	480.2	245	60.8%	0.69
	3	6x6x1/2	256	9.74	0.703	545.6	91.4	65.0	403.2	161	40.4%	0.86
7	1	10x10x1/2	248	17.2	0.727	1063.6	96.1	286.9	712.1	450	63.8%	0.39
	2	8x8x1/2	248	13.5	0.727	834.8	185.3	220.8	558.9	315	70.1%	0.78
	3	6x6x1/2	256	9.74	0.703	545.5	135.6	96.4	403.2	161	59.9%	1.02
8	1	10x10x1/2	206	17.2	0.581	816.2	94.3	351.8	712.1	557	63.2%	0.50
	2	10x10x1/2	206	17.2	0.581	816.2	181.8	270.7	712.1	557	48.6%	0.89
	3	10x10x1/2	216	17.2	0.555	709.4	132.9	119.8	712.1	557	21.5%	1.07
9	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
10	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
						$(AE\cos^2 \bullet)/L$			$0.9F_yA_g$	LRFD Table 4-6		MDOF

Figure 4.2.2 Diagonal Brace Member Force Distribution, Strength Design Efficiency, and Total Story Drift

5.0 Depth Analysis – Foundations

5.1 Existing System – Spread Footings

The existing foundations are comprised of numerous shallow, spread footings in a system recommended by the geotechnical engineer of record. Designed with a maximum soil bearing capacity of 3000 pounds per square foot (psf), the majority of the footings are 7’x7’ to 9’x9’. However, the column footings range in size from the smallest, 4’x4’x1’, to the largest combined footing, 17’x38’x4’. That largest footing requires more than 105 cubic yards of concrete!

The largest cast-in-place (CIP) footings support the lateral force resisting braced frames. The column footing schedule for the braced frames is tabulated below in Figure 5.1.1. After improving the braced frame system, I thought it would be rational to assess the foundation system’s potential for improvement.

Frame	Dimensions			Bottom Steel		Top Steel		
	Width	Length	Depth	Short Bars	Long Bars	Short Bars	Long Bars	
1	17	38	4	(38) #9	(18) #9	(38) #9	(18) #9	COMBINED FTG
2	17	38	4	(38) #9	(18) #9	(38) #9	(18) #9	COMBINED FTG
3	14	14	3	(13) #8	(13) #8	(13) #8	(13) #8	
4	16	38	3	(38) #9	(16) #9	(38) #9	(16) #9	COMBINED FTG
5	16	38	3	(38) #9	(16) #9	(38) #9	(16) #9	COMBINED FTG
6	16	16	3	(14) #9	(14) #9	(14) #9	(14) #9	
7	16	16	3	(14) #9	(14) #9	(14) #9	(14) #9	
8	16	38	3	(38) #9	(16) #9	(38) #9	(16) #9	COMBINED FTG
9	14	14	3	(13) #8	(13) #8	(13) #8	(13) #8	
10	16	38	3	(38) #9	(16) #9	(38) #9	(16) #9	COMBINED FTG

Figure 5.1.1 Braced Frame Column Footing Schedule

5.2 Alternative System - Drilled Concrete Piers

In searching for an alternative foundation system, I re-examined the Geotechnical Investigation Report from the geotechnical engineer of record, Advanced Geoservices Corporation (AGC) of West Chester, Pennsylvania. During the investigation, six test borings were drilled and analyzed to approximate the soil conditions of the building site. Intact rock was encountered in all six borings at depths ranging from 3 feet in the center of the building footprint to 23.5 feet in the southeast corner of the main structure. The rock is described as medium hard gray limestone with graphitic shale laminations and earned a Rock Quality Designation (RQD) of 55%, indicating that the rock is sound with numerous fractures/joints. Using straight-line interpolation between the test borings, I created an approximate three-dimensional rock contour map with the lowest floor elevations intersecting the limestone where rock excavation will be necessary. A plan view of this map is depicted in Figure 5.2.1. An additional three-dimensional perspective view and the original contour map provided by AGC can be found in Appendix B. The three-dimensional views helped to approximate the rock depth below the lowest floor elevations for the analysis of an alternative foundation system. Based on the gathered information, I decided that a system of drilled concrete piers extending into the rock base should prove to be an attractive alternative to the CIP spread footings. The project’s lead structural engineer, Frank Lancaster of EYP, also suggested a concrete caisson system as the best option to replace the spread footings.

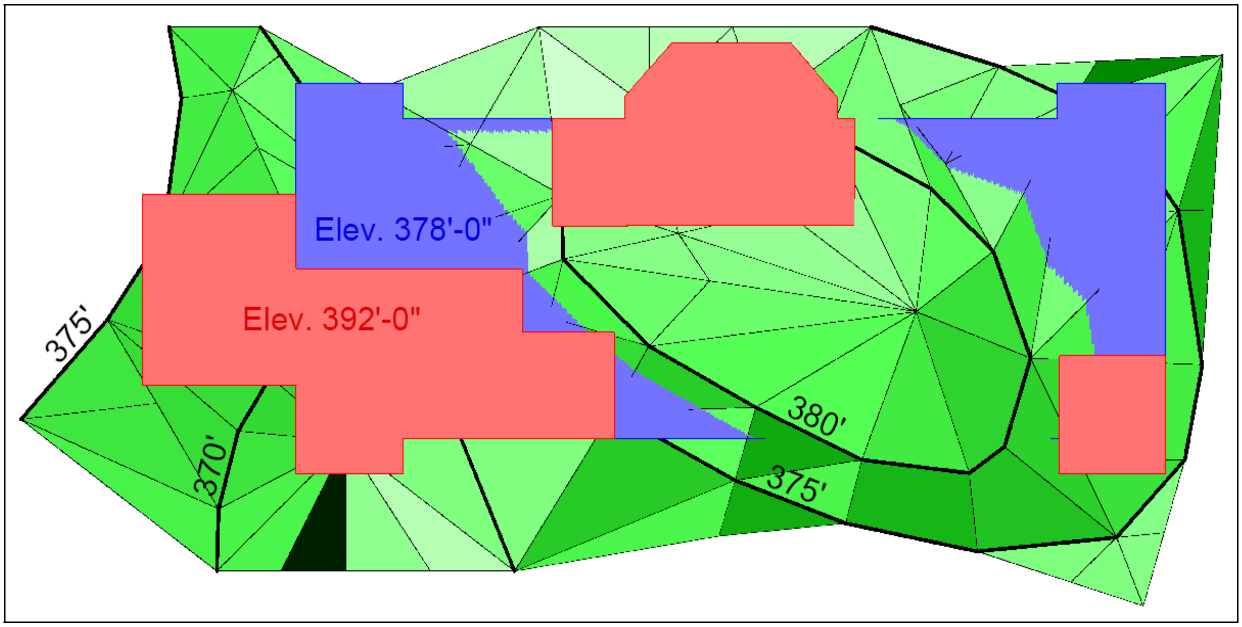


Figure 5.2.1 AutoCAD Approximation of Intact Rock Depth

Given that the footing were largest under the braced frames, these foundations were individually re-designed as drilled piers to assess the overall potential of a new foundation system. All other piers were designed for an anticipated column load of 250 kilo-pounds. To design the new system, I employed a step-by-step procedure to estimate the ultimate bearing capacity of drilled shafts extending into rock from *Principles of Foundation Engineering, 5th Edition* by Braja M. Das, which is available for review in Appendix B.

Unfortunately, the geotechnical report did not include estimated values for the Young's Modulus or the unconfined compression capacity of the local rock. It proved to be very difficult piece of information to garner from libraries or the internet, but I eventually found three sets of limestone strength properties in some very interesting sources. The sources for the information are a technical note entitled "Evaluation of Mechanical Rock Properties" from the *International Journal of Rock Mechanics and Mining Science* and a report entitled "Strength and Deformation Properties of Granite, Basalt, Limestone and Tuff at Various Loading Rates" published by the U.S. Army Corps of Engineers in 1969. The found properties are displayed in the Figure 5.2.2 below.

Rock Description	Young's Modulus (psi)	Unconfined Compression Capacity (psi)
Cordoba Limestone	1.6×10^6	4600
Indiana Limestone	3.8×10^6	9000
Light Oilve-Gray, Dense, Very Fine Grained w/ Some Stylolite Seams	11.23×10^6	11180

Figure 5.2.2 Found Strength Properties of Limestone

Due to the unknown nature of the limestone encountered on the site, the most conservative values were used to design the drilled pier system for the Barshinger Life Science and Philosophy Building. The design calculations were organized and computed in an Excel spreadsheet (Figure 5.2.3). In an attempt to maintain constructability, shaft diameters were limited to one-foot incremental sizes and the shaft depths into rock were restricted to five-foot increments.

Material Constants

Rock Quality Designation, RQD	55	%
Unconfined Compression Strength (rock), $q_{u,rock}$	4.6	ksi
Unconfined Compression Strength (concrete), $q_{u,conc}$	3	ksi
Young's Modulus (rock core), E_{core}	1600	ksi
Young's Modulus (rock mass), E_{mass}	280.0	ksi
Young's Modulus (concrete), E_c	3000	ksi
E_c/E_{mass}	10.71	
Ultimate Unit Side Resistance, f	136.931	psi
Factor of Safety, FS	3	

Braced Frame Foundation Requirements

Frame	Max. Applied Column Load (kips)	Shaft Depth Into Rock, L (ft)	Diameter of Shaft, D_s (ft)	Shaft Area, A_c (in^2)	Min. Vertical Reinforcing, $0.01A_c$ (ft^2)	Embedment Ratio, L/D_s	Settlement Influence Factor, I_ρ	Ultimate Capacity (Side Resistance Only), Q_u (kips)	Shaft Settlement, s_e (in)	Allowable Capacity, Q_{all} (kips)
1	518.3	10	3	1018	0.07	3.3	0.45	1858.4	2/16	619
2	504.2	10	3	1018	0.07	3.3	0.45	1858.4	2/16	619
3	1061.1	15	4	1810	0.13	3.8	0.43	3716.8	4/16	1239
4	487.2	10	3	1018	0.07	3.3	0.45	1858.4	2/16	619
5	487.2	10	3	1018	0.07	3.3	0.45	1858.4	2/16	619
6	1004.6	15	4	1810	0.13	3.8	0.43	3716.8	4/16	1239
7	1137.3	15	4	1810	0.13	3.8	0.43	3716.8	4/16	1239
8	1065.0	15	4	1810	0.13	3.8	0.43	3716.8	4/16	1239
9	429.0	10	3	1018	0.07	3.3	0.45	1858.4	2/16	619
10	429.0	10	3	1018	0.07	3.3	0.45	1858.4	2/16	619
OTHER	*250.0*	*5*	*3*	*1018*	*0.07*	*1.7*	*0.5*	*929.2*	*1/16*	*310*

Figure 5.2.3 Drilled Pier Design

6.0 Depth Analysis – Spanning the Lecture Hall

6.1 Existing System – Vierendeel Truss

The existing system uses a Vierendeel truss (Figure 2.6.1) to span 69-feet over the large lecture hall on the ground floor. The truss carries half of the lecture hall roof load as well as a 15-foot width of classroom spaces on the two upper stories and the main roof. A partial second floor framing plan, Figure 6.1.1, depicts how the truss is incorporated into the floor system. The truss utilizes rigidly connected vertical members to unite the three large girders into one great load carrying system. The truss uses vertical members instead of diagonal members to ensure that the exterior wall openings are not obstructed, thereby maintaining the symmetry of the main façade.

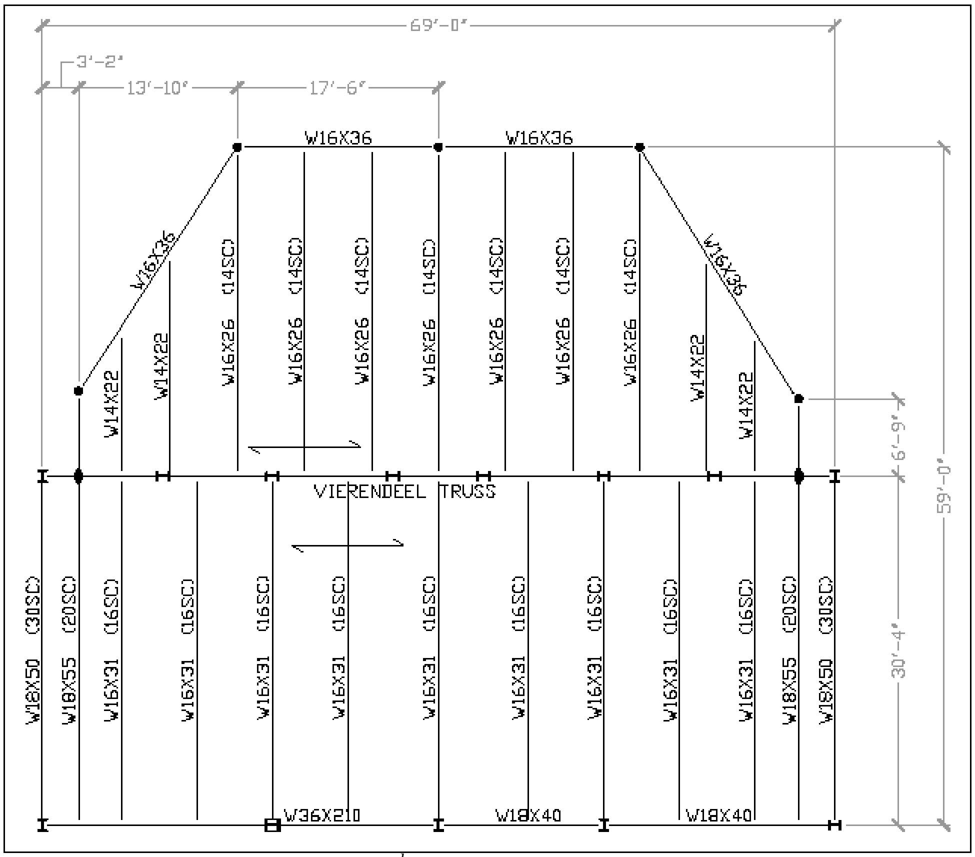


Figure 6.1.1 Partial 2nd Floor Framing Plan – Existing System

6.2 Alternative System – Long Span Steel Joists

This project has exposed me to the Vierendeel truss for the first time. Therefore, I took the opportunity to assess the effectiveness of this structural feature by designing an alternative that will fulfill the structural and architectural duties of the Vierendeel truss. This section will evaluate the structural requirements and Section 8 will discuss the architectural impact.

Three possible alternatives arose from a conversation with the building's primary structural engineer: (1) moving the lecture hall entirely into the main building envelope, (2) a 3-story diagonally braced truss and (3) a new floor diaphragm using long span steel joists. Since I did not want to alter the symmetrical façade or the interior space configuration, I was left with Option 3.

Using RAM Steel computer software and *The New Columbia Joist Company Catalog 2002-1*, I was able to design an alternative structural system (Figure 6.2.1) to span across the large lecture hall. However, the load carrying capacity of the joists precipitated an alteration of other floor diaphragm components. The steel joists, spaced approximately 3-feet center to center, must span the long direction, forcing the composite metal deck to be oriented in a direction perpendicular to the existing system. The long span joists must be a minimum of 40-inches deep to support the required dead and live loads across the entire span.

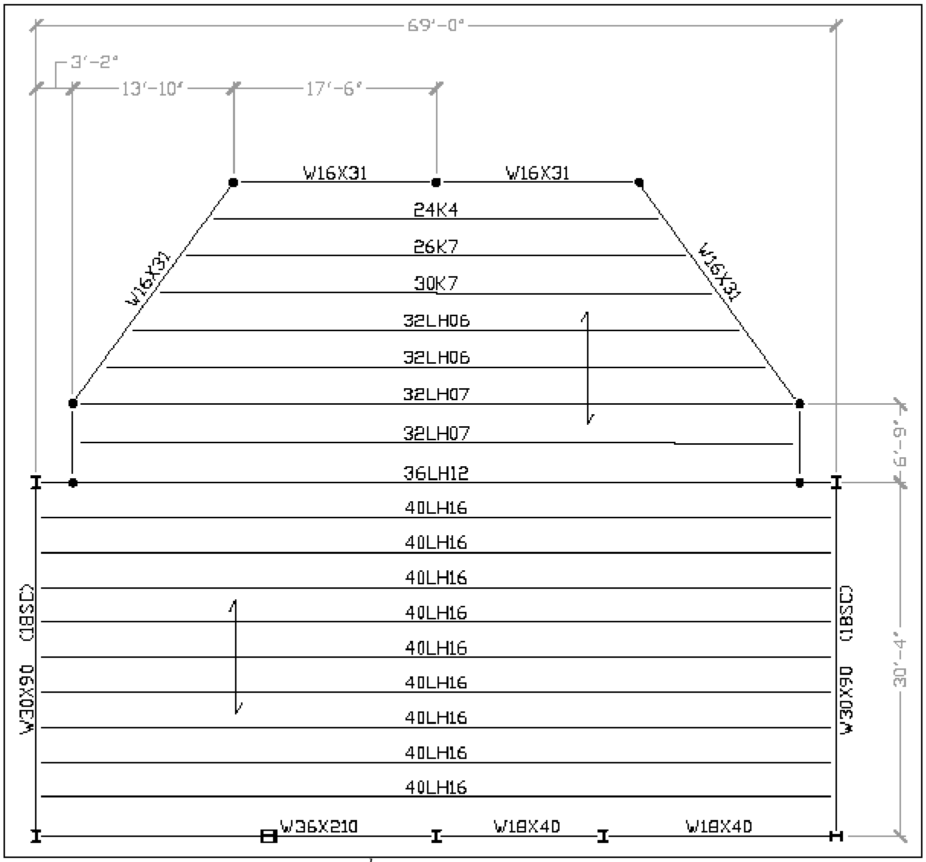


Figure 6.2.1 Partial 2nd Floor Framing Plan – Long Span Joists

The purpose of designing the long span joist system was to determine if there was another system existed that could replace the Vierendeel truss system without dramatically changing the basic shape and configuration of the lecture hall space below. The joist system has the load-carrying ability to do just that. However, the architectural impact of the new design will ultimately decide its practicality as an alternative to the existing design.

7.0 Breadth Analysis – Constructability

7.1 Raw Material Quantities

The simplest test of an alternative structural system is to quantify the basic materials necessary for construction and compare the values with the existing system. Streamlining the braced frame system involved the removal of four braced frames and the alteration of three others. Material savings were calculated to be nearly ten tons steel HSS-shapes using Excel (Figure 7.1.1).

Steel Savings

Frame	HSS Brace Size	Total Length (ft)	lb/ft	Weight (lb)
3	10x10x1/2	(104.9)		
	7x7x1/2	34.4		
	6x6x1/2	70.5		
4	8x8x1/2	68.8		
	7x7x1/2	34.4		
	6x6x1/2	36.1		
5	8x8x1/2	68.8		
	6x6x1/2	70.5		
7	10x10x1/2	(7.4)		
	8x8x1/2	(41.3)		
	7x7x1/2	82.6		
	6x6x1/2	(42.7)		
8	10x10x1/2	(104.9)		
	8x8x1/2	34.4		
	7x7x1/2	34.4		
	6x6x1/2	36.1		
9	7x7x1/2	34.4		
	6x6x1/2	104.9		
10	7x7x1/2	34.4		
	6x6x1/2	104.9	lb/ft	Weight (lb)
Total	10x10x1/2	(217.2)	62.3	(13530.9)
	8x8x1/2	130.8	48.7	6368.3
	7x7x1/2	220.2	41.9	9226.3
	6x6x1/2	380.1	35.1	13342.3
	Weight Savings (tons of steel)			7.70

Figure 7.1.1 Steel Savings for Updated Braced Frame System

Designing a new foundation system greatly reduced the amount of concrete and reinforcing steel needed for construction. The building materials for existing spread-footing system for the braced frames are quantified in Figure 7.1.2. The building materials for the new drilled pier system for the braced frames are quantified in Figure 7.1.3 for comparison. In an attempt to make a fair comparison, I increased the spread-footing materials by 25% and the drilled pier materials by 50% to account for the relative uncertainty of the drilling conditions. Basically, the new system represents a 38% concrete savings and a 24% rebar savings over the existing system. The other fifty column spread footings were tallied and their concrete volumes summed to get the "OTHER" values in Figure 7.1.2. In order to quantify materials, the drilled piers for the other columns were designed to support a typical 250 kilo-pound load. The "other" column footings are not as massive as the braced frame footings; therefore the material savings were not as dramatic. In fact, other columns footings accounted for only sixteen of the four hundred cubic yards of concrete that could be saved by employing a drilled pier foundation system.

note: frame 7 & 8 share a column					
	Dimensions			Total Concrete	Steel
Frame	Width	Length	Depth	Volume (yd^3)	Reinforcing (ft^3)
1	17	38	4	95.7	17.95
2	17	38	4	95.7	17.95
3	14	14	3	21.8	3.88
4	16	38	3	67.6	16.41
5	16	38	3	67.6	16.41
6	16	16	3	28.4	6.05
7	16	16	3	28.4	6.05
8	16	38	3	67.6	16.41
9	14	14	3	21.8	3.88
10	16	38	3	67.6	16.41
OTHER	*50 Ftgs. of Varying Size*			277.9	67.52
			TOTALS	**839.9**	**188.9**
			+ 25%	**1049.9**	**236.1**

Figure 7.1.2 Building Materials for Existing Spread Footings

note: 2 piers per frame (except 7 & 8 b/c they share a column)					
Frame	Approx. Shaft Length Above Rock (ft)	Shaft Depth Into Rock, L (ft)	Diameter of Shaft, D_s (ft)	Total Concrete Volume (yd^3)	Minimum Reinforcing (cu. ft.)
1	25	10	3	18.3	4.95
2	15	10	3	13.1	3.53
3	15	15	4	27.9	7.54
4	10	10	3	10.5	2.83
5	15	10	3	13.1	3.53
6	10	15	4	23.3	6.28
7	10	15	4	23.3	6.28
8	10	15	4	23.3	6.28
9	20	10	3	15.7	4.24
10	20	10	3	15.7	4.24
OTHER	*15*	*5*	3	261.8	70.69
			TOTALS	**434.3**	**120.40**
			+ 50%	**651.4**	**180.60**

Figure 7.1.3 Building Materials for New Drilled Piers

7.2 Cost Impact

The material savings are great statistics, but ultimately the potential of the newly designed systems boils down to cost. I used R.S. Means 2006: Heavy Construction Cost Data to approximate the raw material and construction costs for each major activity affected by the two foundation systems. The cost breakdown for the existing spread footing foundation system is tabulated in Figure 7.2.1. For comparison, the cost estimate for the new drilled pier foundation system is tabulated in Figure 7.2.2. A fear of the unknown clearly manifests itself in the cost estimate of the drilled pier system, leading to an estimate that is practically double the estimate for the basic spread footing assembly.

A1010 210 SPREAD FOOTING ASSEMBLY - includes excavation, backfill, forms, all reinforcement, 3000 psi concrete (chute placed), and screed finished

	2006 Bare Costs			Quantity	Total Costs 2006
	Materials	Installation	Total		
Bulk Excavation	*Per Cubic Yard*				
		4.24	4.24	1469.9	6232
Hand Trim	*Per Square Foot*				
		6.57	6.57	9352.0	61443
Compacted Backfill	*Per Cubic Yard*				
		0.79	0.79	630.0	498
Formwork (4 uses)	*Per Square Foot Perimeter*				
	7.80	48.36	56.16	713.0	40042
Reinforcing, f_y = 60 ksi	*Per Ton*				
	5.37	5.94	11.31	21.0	238
Anchor Bolt Templates	*Per Linear Feet*				
	5.52	20.04	25.56	1584.0	40487
Concrete f_c = 3000 psi	*Per Cubic Yard*				
	31.52		31.52	839.9	26475
Place Concrete, chute	*Per Cubic Yard*				
		6	6	839.9	5040
Screed Finish	*Per Square Foot*				
		4.05	4.05	9352.0	37876
		TOTAL			175414
Note: Overhead & Profit Not Included		**TOTAL w/ regional adjustment factor (Lancaster, PA - 0.929)**			**$163,000**

Figure 7.2.1 Cost Estimate for Existing Spread Footing Foundation System

02465 DRILLED CAISSONS - includes excavation, concrete, 50 lbs. reinforcing steel per C.Y.

	2006 Bare Costs				Quantity	Total Costs 2006
	Materials	Labor	Equipment	Total		
Caisson into Stable Soil	*Per Vertical Linear Foot*					
36"	28.50	11.65	27	67.15	960.0	64464
48"	50.50	14.55	34	99.05	80.0	7924
Caisson into Rock	*Per Vertical Linear Foot*					
36"	28.50	191	286	505.50	370.0	187035
48"	50.50	286	430	766.50	105.0	80483
Mobilization (50 miles)	*Per Drilling Rig*					
36"		730	1700	2430	2	4860
48"		995	2075	3070	1	3070
Excess Material Disposal	*Per Cubic Yard*					
2 miles		1.27	2.68	3.95	434.3	1715
			TOTAL			349551
Note: Overhead & Profit Not Included			**TOTAL w/ regional adjustment factor (Lancaster, PA - 0.929)**			**$325,000**

Figure 7.2.2 Cost Estimate for New Drilled Pier Foundation System

The cost estimate for the streamlined lateral force resisting system is not as dramatic, but it does represent a potential savings over the existing system. I used R.S. Means 2006: Heavy Construction Cost Data to approximate the total cost per ton of HSS-shapes, including basic erection costs. Estimating the cost of the connections proved more difficult. The bracing members are slotted and welded to steel plates with fillet welds. The plates are then welded to the wide-flange columns or beams. Typical connection details are illustrated in Figure 7.2.4, which were taken from the structural drawings provided by EYP.

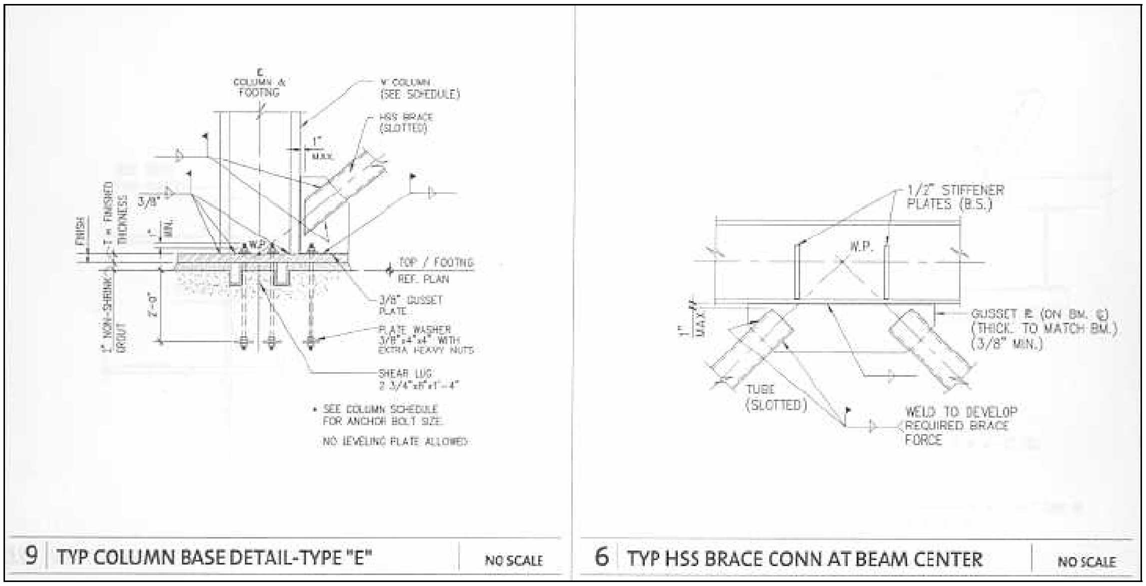


Figure 7.2.4 Typical HSS Bracing Member Connections

Charlie Carter of AISC suggested that an installed fillet weld would cost about $35 per pound of welded metal. I added 10% to that estimate to account for the connection plates. To determine the welding material quantities, I used the tabulated member forces in Figures 4.1.2 and 4.2.2, the basic connection configurations as depicted in Figure 7.2.4, and the minimum weld sizes and lengths as explained in the *Lecture Notes for AE 597E: Design and Analysis of Steel Connections*. Basically, the minimum weld length ($L_{weld} >= 4t_{weld}$ with $L_{weld} = {}^{1}/_{4}L_{real}$) controlled the weld size in every connection. The Excel spreadsheets generated in the connection design processes for both the existing and revised systems are available in Appendix C. The connections savings were then added to the steel savings to produce an overall estimate of the money saved by revising the lateral force resisting system. The savings are tabulated below in Figure 7.2.5.

Steel Cost Savings						
	2006 Bare Costs				Quantity	Total Savings
	Materials	Labor	Equipment	Total		
05120 STRUCTURAL STEEL	Per Ton					
Structural Tubing (HSS)	2100.0	43.5	28.5	2172.0	8	16731
WELDED CONNECTIONS	Per Pound					
E70XX 1/4" fillet welds				38.5	43.1	1661
Note: Overhead & Profit Not Included			TOTAL w/ regional adjustment factor (Lancaster, PA - 0.929)			**$18,000**

Figure 7.2.5 Savings Estimate for Revising the Lateral Force Resisting System

8.0 Breadth Analysis – Architectural/Mechanical Impact

8.1 Façade Impact

The Vierendeel truss is particularly ingenious for its ability to cooperate with the rectangular openings of the building's façade. The Western façade of the Barshinger Life Science and Philosophy Building is depicted in Figure 8.1.1 with the Vierendeel truss location expressed in light blue. The symmetry of the Colonial Revival-style façade is easily recognizable and should be preserved at all costs.

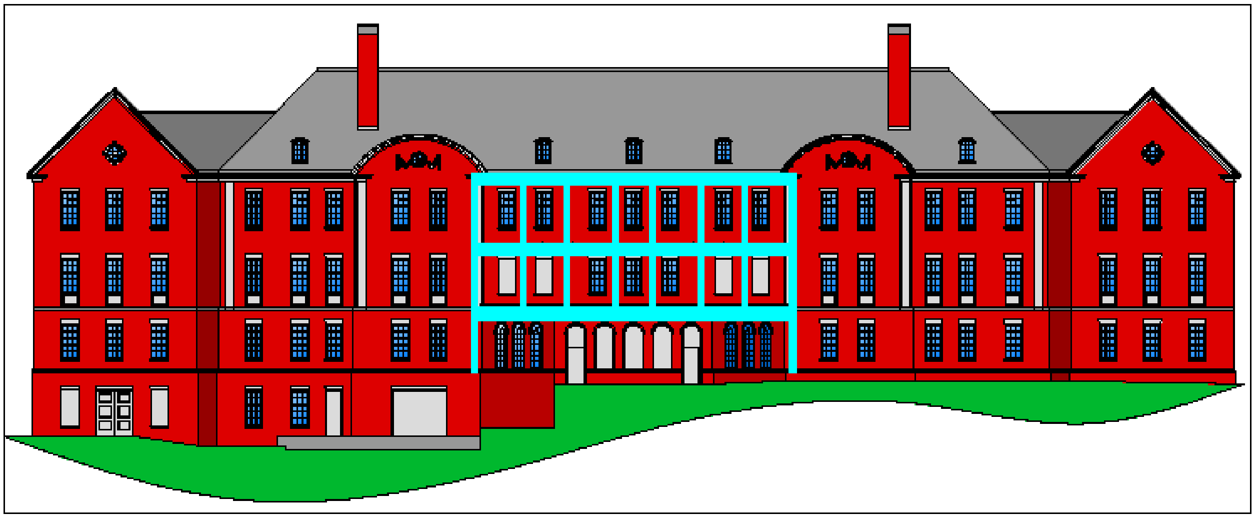


Figure 8.1.1 West Façade with Vierendeel Truss

The long span joist system, as pictured in Figure 8.1.2, also protects the integrity of the façade's architecture. The joists that lie within the façade have the same nominal depth as the girders in the Vierendeel truss. The joist members also have the added advantage of open webs, which create spaces for the four 12-inch web penetrations required in the lowest girder of the truss (see Figure 2.6.1).

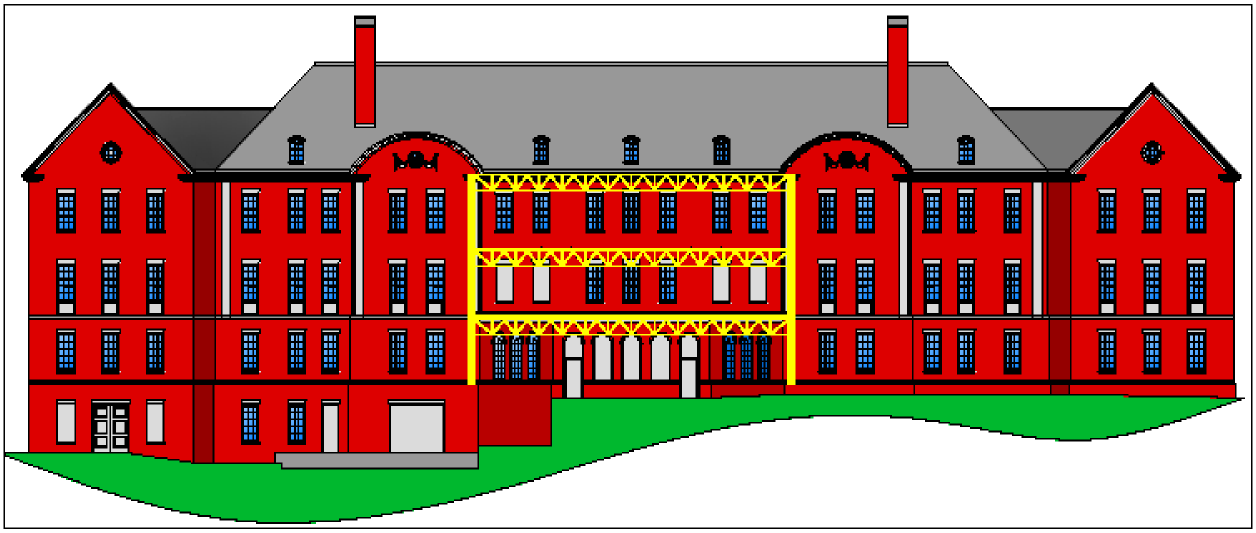


Figure 8.1.2 West Façade with Long Span Joists

8.2 Interior Space – Above Ceiling Assessment

The potential problem with the long span joist system lies within the plenum space above ceiling. The existing system uses W16x31 beams to span the transverse direction from the typical framing at the center of the building to the Vierendeel truss at the exterior. The new system of long span joists has 40LH16 members spanning across the lecture hall on the ground floor and across teaching labs and classrooms on the upper two floors. There is a nominal difference in depth of 24-inches. The rooms are designed with a typical 9-foot ceiling height and a total above ceiling plenum depth of 53-inches. If the ceiling height is to be maintained, there would only be 13-inches for mechanical ductwork in the long span joist system.

The ductwork needed to be investigated in order to properly assess the alternative structural system. If all the ductwork can be reduced to a maximum depth of 10-inches, then the ceiling height would only have to decrease by maximum of 4-inches and the long span joist system could be a viable option. Partial HVAC Ductwork plans provided by EYP are available in Appendix C. Five ducts need to be altered for the long span joist system: a 30x18 return duct on the first floor and two 24x18 supply ducts on each of the two upper floors. Using the design tools in Fundamentals of Thermal-Fluid Sciences by Yunus A. Cengel, I was able find 10-inch ducts that have the same fundamental friction loss. The new duct sizes are listed in Figure 8.2.1. By maintaining the same friction loss, I ensured that only the ducts, and not the mechanical equipment, were resized. If the friction loss was greater for the altered duct, then the fan would use more energy to supply air to the spaces at the prescribed exit rate. However, the newly-sized ducts have a much higher aspect ratio then the existing ducts, which means more sheet metal to enclose and a more expensive duct. The widths of the new ducts are also a cause for concern as the plenum space is going to be very congested with only 17 inches of free space in which to fit numerous utilities. However, the bottom line is that the long span joist system can be made viable with a little extra money and a few changes to the HVAC ductwork.

Friction Factors of Fully Developed Laminar Flow		
w/d	f	F
1	56.92/Re	56.92
2	62.20/Re	62.20
3	68.36/Re	68.36
4	72.92/Re	72.92
6	78.80/Re	78.80
8	82.32/Re	82.32
inf.	96.00/Re	96.00

$F = f \times Re$

$Re = V_m * D_h * v$

$D_h = (2wd)/(w+d)$

Try to keep V_m & v constant.

Find New w-value that Produces Same F-value as Existing Duct.

Duct Size		Aspect Ratio	Hydraulic	F
Depth	Width	w/d	Diameter, D_h	
18	32	1.78	23.0	1406.3
10	70	7.00	17.5	1409.8
18	24	1.33	20.6	1207.1
10	44	4.40	16.3	1207.5

Figure 8.2.1 Design of Equivalent Flattened Duct Sizes

9.0 Conclusions & Recommendations

Through both the depth and breadth topics, I have been able to analyze and assess a few of the many systems at work throughout the Barshinger Life Science and Philosophy Building at Franklin and Marshall College in Lancaster, Pennsylvania. The lateral force resisting system was streamlined for efficiency. A new foundation system of drilled concrete piers was designed to carry the building loads directly into the intact limestone less than 25-feet below finish grades. The structural device that is the Vierendeel truss was assessed based on the design of a potential alternative. After the designs were complete, I reached into construction management, architecture, and HVAC ductwork design to properly compare the new systems with their existing counterparts. The following conclusions were made based on the analyses from this senior thesis project:

- The existing lateral force resisting system of ten concentrically braced frames is oversized for the calculated seismic loads. The number of frames can be safely reduced to six, saving nearly $18,000 in material and construction costs.

- Replacing traditional spread footings with drilled concrete piers is not cost effective. Although the geotechnical engineers mapped the intact rock depth as enticingly close to the planned ground floor level, the drilled pier system ($325,000) was estimated to cost twice as much as the spread footing system ($163,000). However, the drilled pier system did reduce steel reinforcing quantities by 24% and concrete consumption by 35%. Geotechnical investigation is an extremely difficult job to complete accurately. As a result, excavation contractors are wary of deep foundation systems and charge accordingly.

- The Vierendeel truss is extremely effective and efficient for carrying significant loads over a large span. It can be manipulated to accommodate various configurations of rectangular openings, such as windows in a façade. A system of long span steel joists can be designed as an alternative to the rigidly-connected and weighty Vierendeel truss. However, the long span joists required bay in-fill members that were significantly deeper than those used with the truss. Those deeper members created a heavily congested above-ceiling plenum space and required that some HVAC ducts be resized to preserve the prescribed ceiling height.

- HVAC ducts can be flattened out without increasing friction loss or system energy consumption by simply maintaining the hydraulic diameter.

- For each individual building system that is changed, ten other systems are subsequently affected…

Appendix

Appendix	Description
A	**Building Loads**
	Dead Loads
	Live Loads
	Snow Loads
	Wind Loads
	Seismic Loads
B	**Depth Analysis**
	MDOF System Stiffness Matrices
	Drilled Pier Design Procedure
	Intact Rock Contour Drawings
C	**Breadth Analyses**
	Bracing Connections
	Partial HVAC Ductwork Plans

Appendix A

Dead & Live Load Requirements / Weight of Building Calculations

GRAVITY LOADS (ASCE 7-02, IBC 2000, + SOME EDUCATED GUESSES)

- DEAD LOADS
  - $6\frac{1}{2}$" NML. WT. CONCRETE SLAB : 12 PSF / IN $\times 6\frac{1}{2}$ = 78 PSF
  - METAL DECK : 3 PSF
  - FRAMING MEMBERS : 10 PSF
  - MEP EQUIPMENT : 10 PSF
  - EXT WALLS : 45 PSF
  - CARPET : 1 PSF
- PARTITIONS : 20 PSF
- LIVE LOADS
  - OFFICES : 50 + 20 PSF
  - LABORATORIES : 60 PSF
  - STAIRS / CORRIDORS : 100 PSF
- SNOW : 30 PSF (GROUND)
- ROOF DEAD : 60 PSF

FLOOR AREAS

$114' \times 260' \rightarrow 30{,}000$ SF PER FLOOR

$\rightarrow 748'$

STRUCTURE WEIGHT (FOR SEISMIC)

$W_{ROOF} = (30000)(60) + (14/2)(45)(748) = 2010$ K

$W_1 = (30000)(122) + (15/2)(45)(748) = 3885$ K

$W_2 = (30000)(122) + (15/2 + 14/2)(45)(748) = 4094$ K

$W_3 = (30000)(122) + (14/2 + 14/2)(45)(748) = 4079$ K

$W = W_{ROOF} + W_1 + W_2 + W_3 = 14{,}100$ K

Appendix A

Snow Load Analysis

SNOW LOAD (ASCE 7-02)

$p_g = 30$ PSF GROUND SNOW LOAD

$C_e = 1.0$ PARTIALLY EXPOSED

$C_t = 1.0$ FOR FLAT ROOF

$= 1.2$ FOR SCREEN ROOFS

$I = 1.1$ CATEGORY III BUILDING

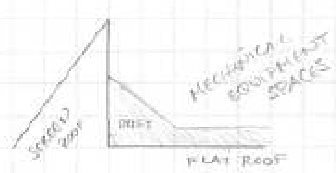
$p_f = 0.7 C_e C_t I p_g = 0.7(1.0)(1.0)(1.1)(30) = \boxed{23.1 \text{ PSF}}$

* A VALUE OF 25 PSF WAS USED FOR DESIGN

$p_s = C_s p_f = (1.0)[(23.1)(1.2)] = \boxed{27.7 \text{ PSF}}$

* A VALUE OF 28 PSF WAS USED FOR DESIGN

DRIFT – SCREEN ROOF – FLAT ROOF PROJECTION (SEC. 7.8)

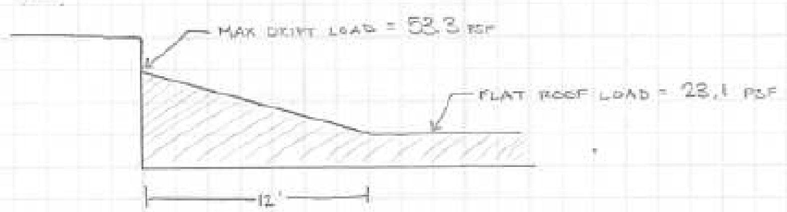


$\gamma = 0.13(30) + 14 = 17.9$ PCF ≤ 30 PCF

$h_d = \begin{cases} 1.5' & \text{FIGURE 7-9} \\ W/4 = 12'/4 = 3' & \leftarrow \text{CONTROLS} \end{cases}$

$h_b = (23.1)/(17.9) = 1.3'$

$h_d / h_b = 2.3$



* THE DESIGNED LOADS ARE

25 PSF FLAT ROOF LOAD

75 PSF MAX. DRIFT LOAD

Appendix A

Wind Load Analysis

WIND LOAD CALCULATIONS : NORTH-SOUTH DIRECTION [EAST-WEST]

- DESIGNED VALUES FROM GENERAL NOTES OF STRUCTURAL DRAWINGS
  - BASIC WIND SPEED, V_{3s} : 90 MPH
  - WIND IMPORTANCE FACTOR, I_W : 1.15
  - WIND EXPOSURE : B
  - HEIGHT & EXPOSURE ADJUSTMENT FACTOR : 1.19
  - P_{ROOF} : +15.9 / -17.3 PSF, FIELD
  - P_{ROOF} : +15.9 / -20.3 PSF, EDGE
  - P_{ROOF} : +15.9 / -20.3 PSF, CORNER
  - P_{WALL} : +17.4 / -18.8 PSF, FIELD
  - P_{WALL} : +17.4 / -23.3 PSF, CORNER

$K_d = 0.85$ (TABLE 6-4)

C_p : WINDWARD → $C_p = 0.8$
LEEWARD → $C_p = -0.5$ [-0.3]

$K_{zt} = 1.0$

$G = 0.830$ [0.799]

K_z : (TABLE 6-3)

0'-15'	0.57
20'	0.62
25'	0.66
30'	0.70
40'	0.76
50'	0.81
60'	0.85

K_h : 0.83 FOR $h = 55'$

$q_z = 0.00256 K_z K_{zt} K_d V^2 I$

0'-15'	11.6 PSF
20'	12.6
25'	13.4
30'	14.2
40'	15.4
50'	16.4
60'	17.2

$q_h = 0.00256 K_h K_{ht} K_d V^2 I = 16.8$ PSF

Appendix A

Wind Load Analysis (cont'd)

WIND LOAD CALCS (CONT'D) [EAST-WEST VALUES]

$P_W = qGC_p = q(0.83)(0.8)$

0-15'	7.7 PSF	[7.4]
20'	8.3	[8.0]
25'	8.9	[8.6]
30'	9.4	[9.0]
40'	10.2	[9.8]
50'	10.9	[10.5]
60'	11.4	[11.0]

$P_e = (16.8)(0.83)(-0.5) = -7.0 \quad [-6.7]$

NET PRESSURE (P_{NET})

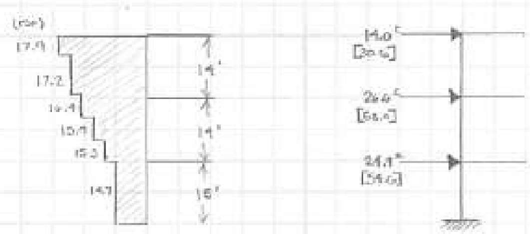
0-15'	(7.7) + (7.0) = 14.7 PSF	[14.1]
-20'	= 15.3	[14.7]
-25'	= 15.9	[15.3]
-30'	= 16.4	[15.7]
-40'	= 17.2	[16.5]
-50'	= 17.9	[17.2]
-60'	= 18.4	[17.7]

• FLOOR SHEAR LOADS

$F_1 = [(14.7)(7.5) + (15.3)(5) + (15.9)(2)](114) = 24.9\text{ k} \quad [54.6^k]$

$F_2 = [(15.9)(3) + (16.4)(5) + (17.2)(6)](114) = 26.6\text{ k} \quad [58.0^k]$

$F_3 = [(17.2)(4) + (17.9)(3)](114) = 14.0\text{ k} \quad [30.6^k]$



BASE SHEAR
65.5 k [143.2 k]

BASE MOMENT
1746.9 'k [3816.8'k]

Appendix A

Seismic Load Analysis

SEISMIC LOAD CALCULATIONS (ASCE 7-02)

- DESIGN VALUES FROM GENERAL NOTES OF STRUCTURAL DRAWINGS
  - SEISMIC USE GROUP : II
  - SEISMIC DESIGN CATEGORY : B
  - $S_{DS} = 0.19$
  - $S_{D1} = 0.05$
  - SITE CLASS : B
  - DESIGN BASE SHEAR : 895 KIPS
  - SEISMIC RESISTING SYSTEM : CONCENTRICALLY BRACED FRAMES (STRUCTURAL STEEL SYSTEM NOT SPECIFICALLY DESIGNED FOR SEISMIC RESISTANCE.)
  - ANALYSIS PROCEDURE : EQUIVALENT LATERAL FORCE PROCEDURE

$I = 1.25$ (TABLE 9.1.4)

$S_{MS} = 25\ \%g$ (FIGURE 9.4.1.1 (a))

$S_{M1} = 6\ \%g$ (FIGURE 9.4.1.1 (b))

$F_a = F_v = 1.0$ (TABLE 9.4.1.2.4)

$S_{DS} = \frac{2}{3} S_{MS} = 0.167\ g$

$S_{D1} = \frac{2}{3} S_{M1} = 0.04\ g$

$T_o = 0.2\ S_{D1}/S_{DS} = 0.048\ s$

$T_s = S_{D1}/S_{DS} = 0.24\ s$

$$\left.\begin{array}{ll} R = 5 & \text{RESPONSE MOD. FACTOR} \\ W_o = 2 & \text{SYSTEM OVERSTRENGTH FACTOR} \\ C_d = 4.5 & \text{DEFLECTION AMP. FACTOR} \end{array}\right\}$$

(TABLE 9.5.2.2 ORDINARY STEEL CONCENTRICALLY BRACED FRAMES

$C_s = \frac{S_{DS}}{R/I} = 0.06$

$T = T_a = C_t h_n^x = (0.02)(43)^{(0.75)} = 0.336 < C_u(0.1N) = 0.51$

$V = C_s W$

Appendix A

Seismic Load Analysis (cont'd)

SEISMIC LOAD CALCS (CONT'D)

$V = C_s W = (0.06)(14100) = 846\ k$ BASE SHEAR

* VERY COMPARABLE TO DESIGN VALUE OF 895 k

$C_{vx} = \frac{w_x h_x^k}{\Sigma w_i h_i^k}$ $k = 1.0$ FOR $T \le 0.5s$

$C_{ROOF} = \frac{(2010)(43)}{(266131)} = 0.325$

$C_3 = \frac{(4079)(29)}{(266131)} = 0.444$

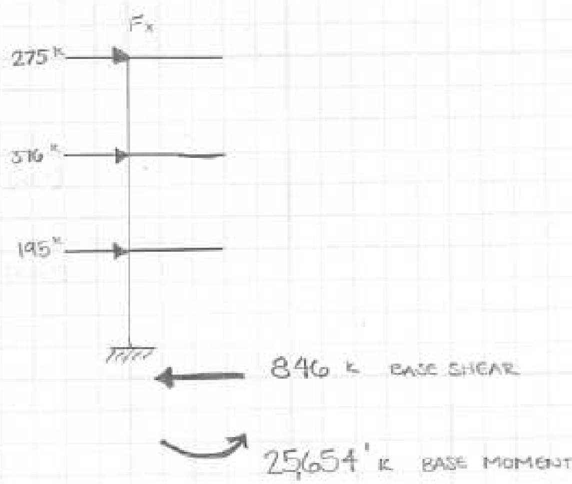
$C_2 = \frac{(4094)(15)}{(266131)} = 0.231$

$F_x = C_{vx} V$

$F_{ROOF} = 275\ k$

$F_3 = 376\ k$

$F_2 = 195\ k$



Appendix B

MDOF System Stiffness Matrices

Existing System

1

2679	-1079	0
-1079	1983	-904
0	-904	904

2

2162	-906	0
-906	1810	-904
0	-904	904

3

1985	-906	0
-906	1810	-904
0	-904	904

4

2335	-1079	0
-1079	1983	-904
0	-904	904

5

2162	-906	0
-906	1810	-904
0	-904	904

6

1250	-625	0
-625	1170	-546
0	-546	546

7

1250	-625	0
-625	866	-241
0	-241	241

8

2335	-1079	0
-1079	1983	-904
0	-904	904

9

1812	-906	0
-906	1810	-904
0	-904	904

10

1812	-906	0
-906	1810	-904
0	-904	904

Revised System

1

1367	-550	0
-550	952	-402
0	-402	402

2

1103	-462	0
-462	864	-402
0	-402	402

3

1632	-816	0
-816	1526	-709
0	-709	709

4

[redacted]

5

[redacted]

6

1435	-717	0
-717	1263	-546
0	-546	546

7

1898	-835	0
-835	1380	-545
0	-545	545

8

1632	-816	0
-816	1526	-709
0	-709	709

9

[redacted]

10

[redacted]

Appendix B

Drilled Pier Design Procedure

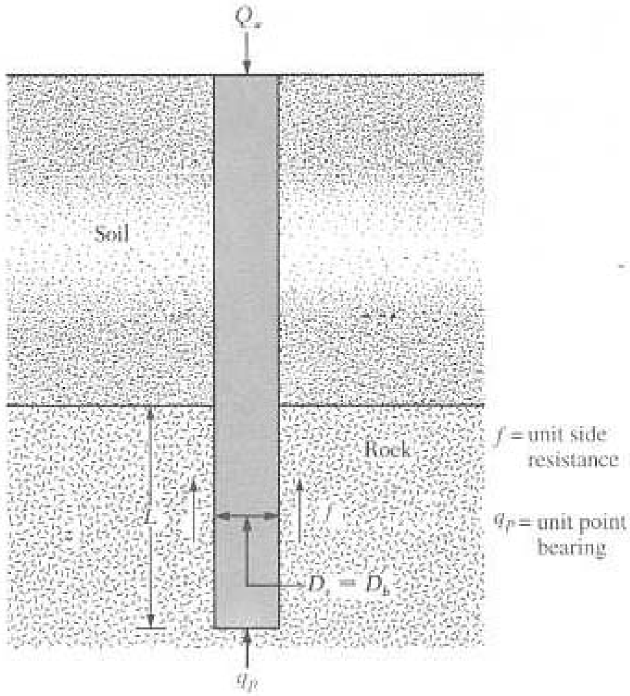


Figure 12.27 Drilled shaft socketed into rock

1. Calculate the ultimate unit side resistance as

$$f\,(\text{lb/in}^2) = 2.5q_u^{0.5} \leq 0.15q_u \tag{12.62}$$

where q_u = unconfined compression strength of a rock core of NW size or larger, or of the drilled shaft concrete, whichever is smaller (in lb/in^2)

In SI units, Eq (12.62) can be expressed as

$$f\,(\text{kN/m}^2) = 6.564q_u^{0.5}(\text{kN/m}^2) \leq 0.15q_u\,(\text{kN/m}^2) \tag{12.63}$$

2. Calculate the ultimate capacity based on side resistance only, or

$$Q_u = \pi D_s L f \tag{12.64}$$

3. Calculate the settlement s_e of the shaft at the top of the rock socket, or

$$s_e = s_{e(s)} + s_{e(b)} \tag{12.65}$$

where $s_{e(s)}$ = elastic compression of the drilled shaft within the socket, assuming no side resistance

$s_{e(b)}$ = settlement of the base

However,

$$s_{e(s)} = \frac{Q_u L}{A_c E_c} \tag{12.66}$$

Das, Braja M.. Principles of Foundation Engineering. 5th Edition.

Appendix B

Drilled Pier Design Procedure (cont'd)

and

$$s_{e(b)} = \frac{Q_u I_f}{D_s E_{\text{mass}}} \tag{12.67}$$

where Q_u = ultimate load obtained from Eq. (12.62) or Eq. (12.63) (this assumes that the contribution of the overburden to the side shear is negligible)

A_c = cross-sectional area of the drilled shaft in the socket (12.68)

$= \frac{\pi}{4} D_s^2$

E_c = Young's modulus of the concrete and reinforcing steel in the shaft

E_{mass} = Young's modulus of the rock mass into which the socket is drilled

I_f = elastic influence coefficient (see Figure 12.28)

The magnitude of E_{mass} can be determined from the average plot shown in Figure 12.29. In this figure, E_{core} is the Young's modulus of intact specimens of rock cores of NW size or larger. However, unless the socket is very long (O'Neill, 1997),

$$s_e \approx s_{e(b)} = \frac{Q_u I_f}{D_s E_{\text{mass}}} \tag{12.69}$$

4. If s_e is less than 10 mm ($\approx$0.4 in.), then the ultimate load-carrying capacity is that calculated by Eq. (12.64). If $s_e \geq$ 10 mm. (0.4 in.), then go to Step 5.

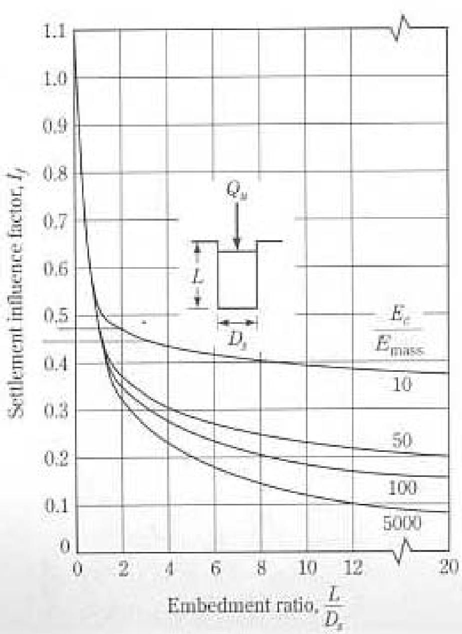


Figure 12.28 Variation of I_f (after Reese and O'Neill, 1989)

Das, Braja M.. Principles of Foundation Engineering. 5th Edition.

Appendix B

Drilled Pier Design Procedure (cont'd)

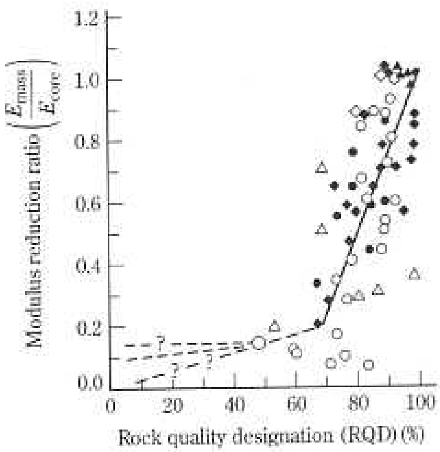


Figure 12.29 Plot of E_{mass}/E_{core} vs. RQD (after Reese and O'Neill, 1989)

5. If $s_e \geq 10$ mm (0.4 in.), there may be rapid, progressive side shear failure in the rock socket, resulting in a complete loss of side resistance. In that case, the ultimate capacity is equal to the point resistance, or

$$Q_u = 3A_p\left[\frac{3 + \frac{c_s}{D_s}}{10\left(1 + 300\frac{\delta}{c_s}\right)^{0.5}}\right]q_u \qquad (12.70)$$

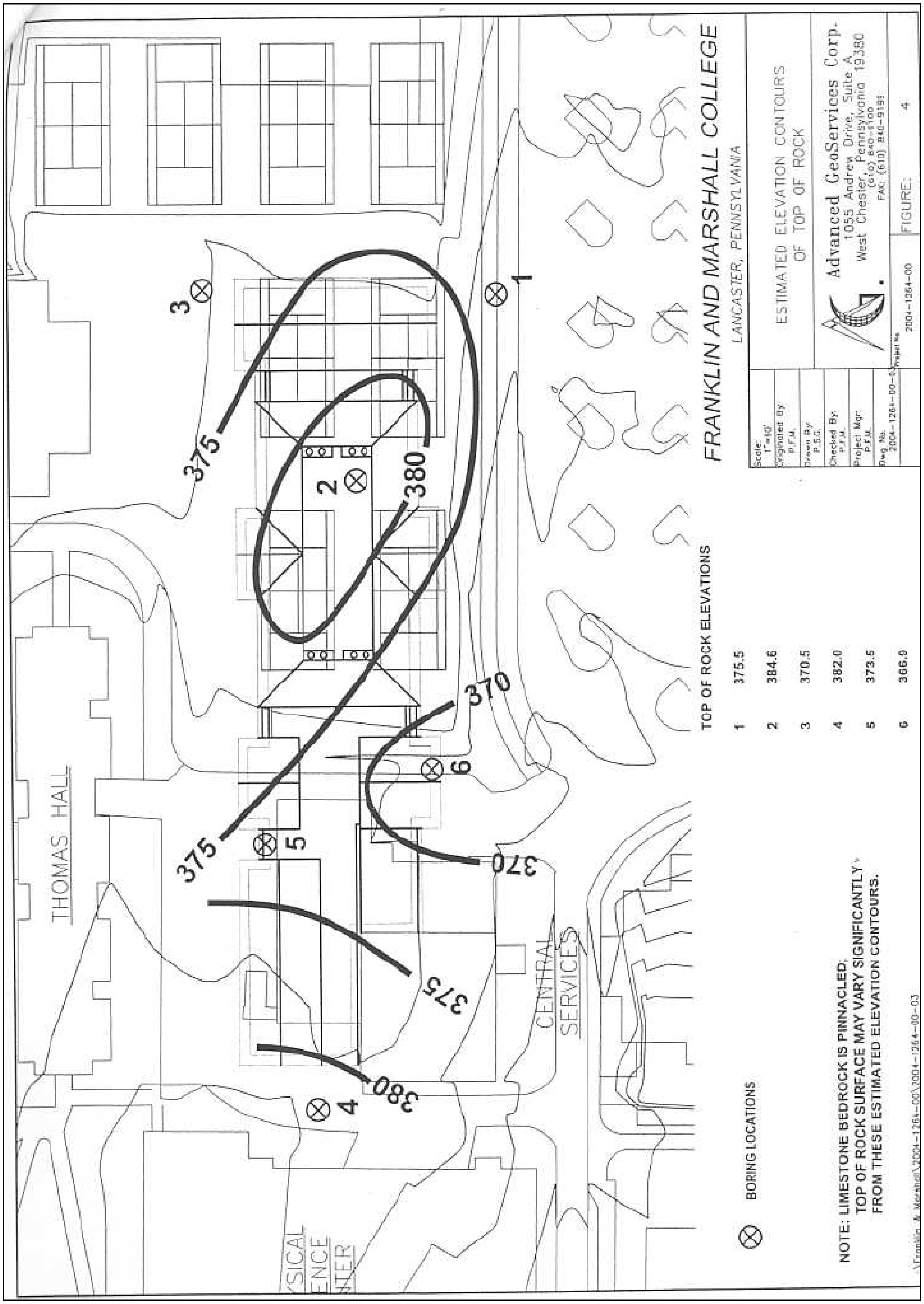
where c_s = spacing of discontinuities (same unit as D_s)
δ = thickness of individual discontinuity (same unit as D_s)
q_u = unconfined compression strength of the rock beneath the base of the socket, or the drilled shaft concrete, whichever is smaller

Note that Eq. (12.70) applies for horizontally stratified discontinuities with $c_s > 305$ mm (12 in.) and $\delta < 5$ mm(0.2 in.).

Das, Braja M.. Principles of Foundation Engineering. 5th Edition.

Appendix B

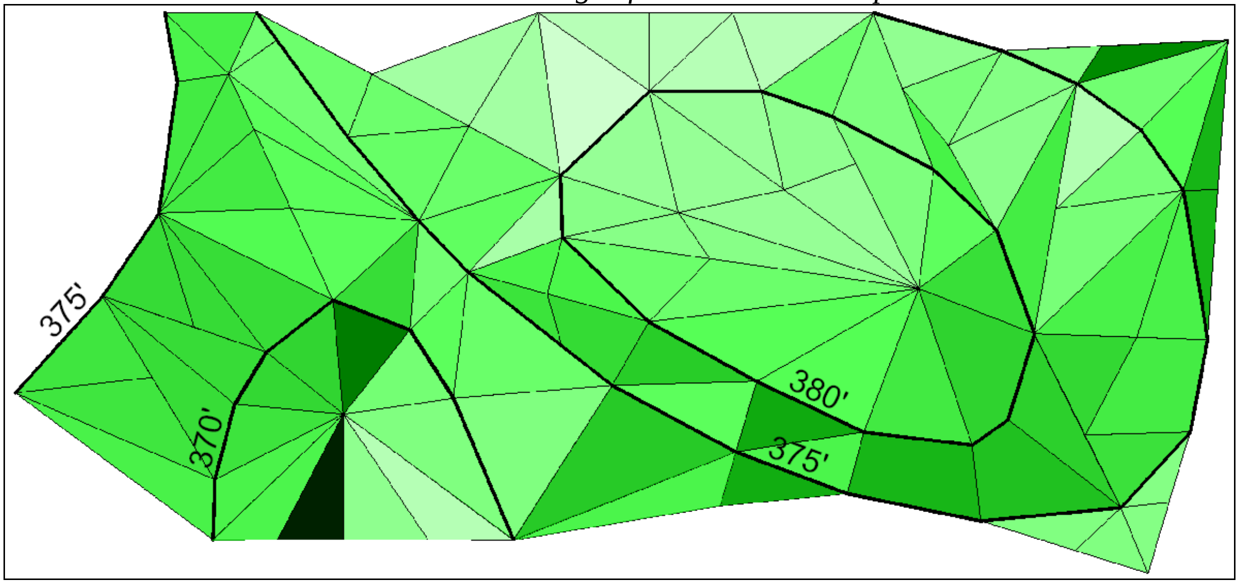
Intact Rock Contour Drawings



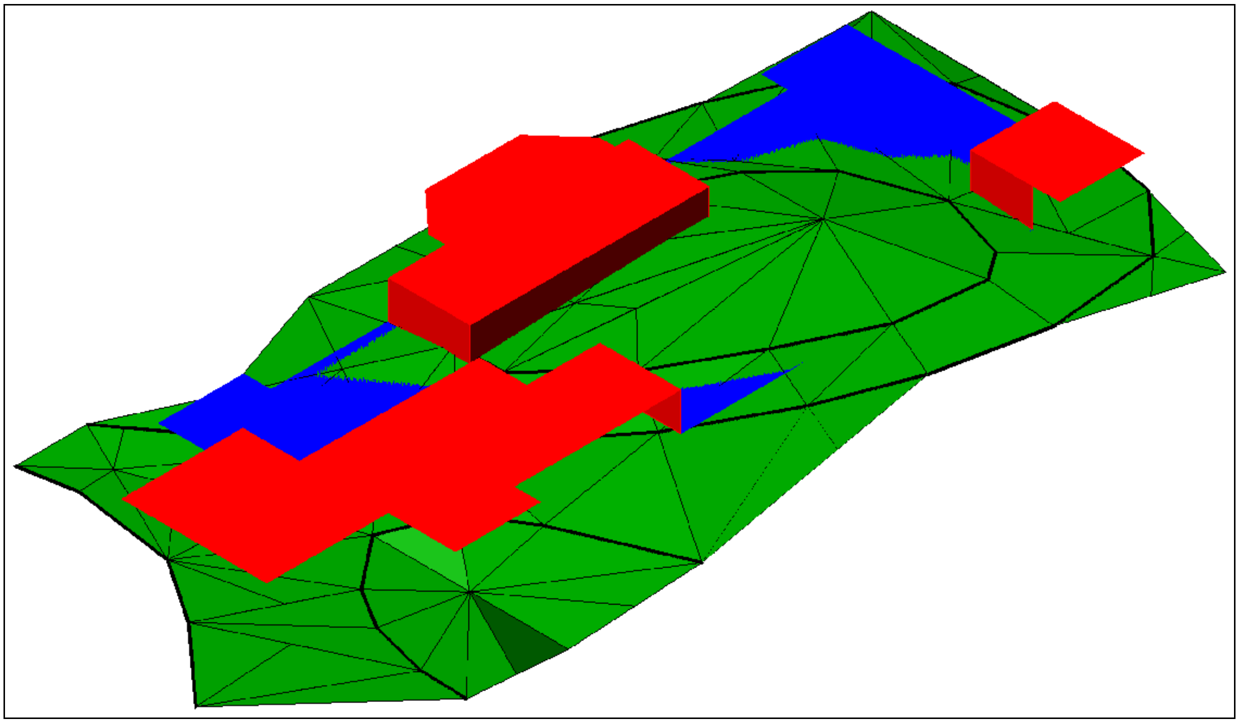
Appendix B

Intact Rock Contour Drawings (cont'd)

3-D CAD Drawings of AGC Contour Map



Plan View



SW Isometric View

Appendix C

Bracing Connections

Weld Size	4/16	*Assumes >= 1/2" Thick Connector Plates		Brace End Req'd Weld Area (in^2)	Beam Bottom Req'd Weld Area (in^2)	Column Req'd Weld Area (in^2)
Frame	(theta)	Story Shear (k)	Brace Axial Force (k)			
1	0.951	22.3	83.4	20.0	10.0	20.0
	0.951	43.1	64.2	20.0	10.0	20.0
	0.983	31.5	28.4	20.0	10.0	20.0
2	0.951	20.9	78.2	20.0	10.0	20.0
	0.951	40.4	60.2	20.0	10.0	20.0
	0.983	29.5	26.6	20.0	10.0	20.0
3	0.951	62.5	233.1	20.0	10.0	20.0
	0.951	120.4	179.4	20.0	10.0	20.0
	0.983	88.1	79.4	20.0	10.0	20.0
4	0.951	48.2	179.8	20.0	10.0	20.0
	0.951	92.9	138.4	20.0	10.0	20.0
	0.983	67.9	61.2	20.0	10.0	20.0
5	0.951	47.5	177.1	20.0	10.0	20.0
	0.951	91.5	136.3	20.0	10.0	20.0
	0.983	66.9	60.3	20.0	10.0	20.0
6	0.757	51.1	152.6	20.0	10.0	20.0
	0.757	98.6	117.4	20.0	10.0	20.0
	0.791	72.1	51.3	20.0	10.0	20.0
7	0.757	76.6	228.8	20.0	10.0	20.0
	0.757	147.8	176.0	20.0	10.0	20.0
	1.112	108.1	243.8	20.0	10.0	20.0
8	0.951	48.2	179.8	20.0	10.0	20.0
	0.951	92.9	138.4	20.0	10.0	20.0
	0.983	67.9	61.2	20.0	10.0	20.0
9	0.951	24.5	91.5	20.0	10.0	20.0
	0.951	47.3	70.4	20.0	10.0	20.0
	0.983	34.6	31.2	20.0	10.0	20.0
10	0.951	10.7	40.1	20.0	10.0	20.0
	0.951	20.7	30.8	20.0	10.0	20.0
	0.983	15.1	13.7	20.0	10.0	20.0
	# of Connections Per Story			4	1	2
	TOTAL WELD AREA (in^2)				3900.0	
	TOTAL WELD VOLUME (in^3)				1950.0	
	TOTAL WELD MATERIAL (lbs)				**107.8**	

Welded Connection Design for the Existing Bracing System

Appendix C

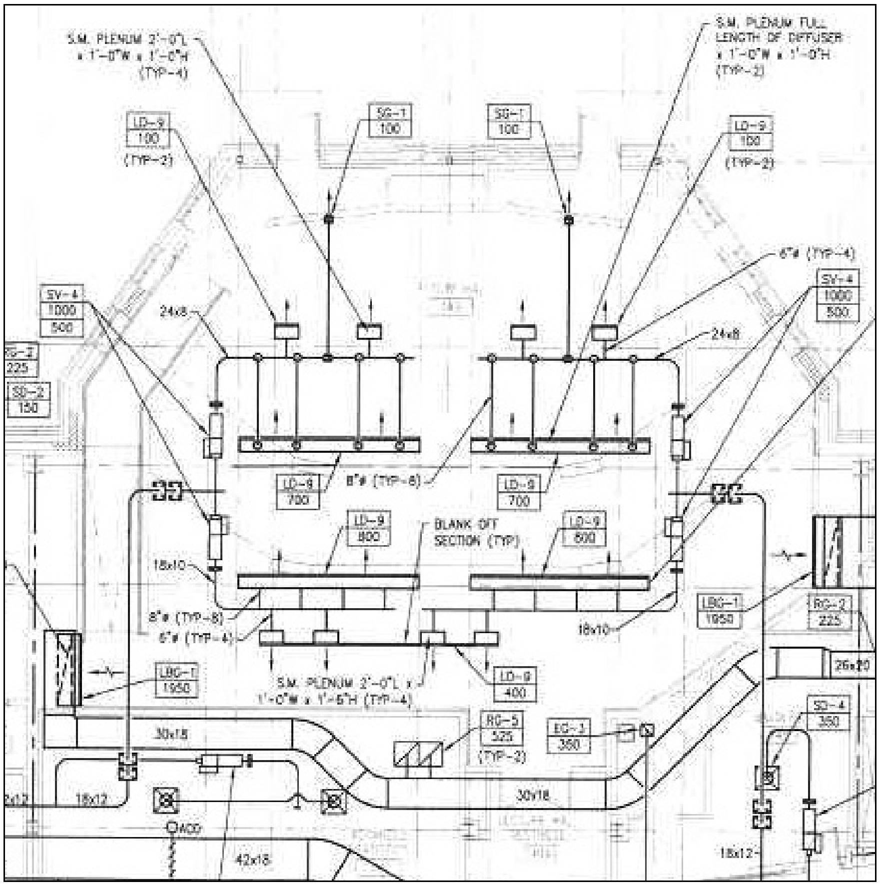
Bracing Connections (cont'd)

Weld Size	4/16	*Assumes >1/2" Thick Connector Plates		Brace End Req'd Weld Area (in^2)	Beam Bottom Req'd Weld Area (in^2)	Column Req'd Weld Area (in^2)
Frame	(theta)	Story Shear (k)	Brace Axial Force (k)			
1	0.951	30.5	114.0	20.0	10.0	20.0
	0.951	58.9	87.7	20.0	10.0	20.0
	0.983	43.1	38.8	20.0	10.0	20.0
2	0.951	28.3	105.8	20.0	10.0	20.0
	0.951	54.7	81.4	20.0	10.0	20.0
	0.983	40.0	36.0	20.0	10.0	20.0
3	0.951	112.0	418.1	20.0	10.0	20.0
	0.951	216.0	321.7	20.0	10.0	20.0
	0.983	158.0	142.4	20.0	10.0	20.0
6	0.757	64.8	193.5	20.0	10.0	20.0
	0.757	125.0	148.9	20.0	10.0	20.0
	0.791	91.4	65.0	20.0	10.0	20.0
7	0.757	96.1	286.9	20.0	10.0	20.0
	0.757	185.3	220.8	20.0	10.0	20.0
	0.791	135.6	96.4	20.0	10.0	20.0
8	0.951	94.3	351.8	20.0	10.0	20.0
	0.951	181.8	270.7	20.0	10.0	20.0
	0.983	132.9	119.8	20.0	10.0	20.0
	# of Connections Per Story			4	1	2
	TOTAL WELD AREA (in^2)				2340.0	
	TOTAL WELD VOLUME (in^3)				1170.0	
	TOTAL WELD MATERIAL (lbs)				**64.7**	

Welded Connection Design for Revised Bracing System

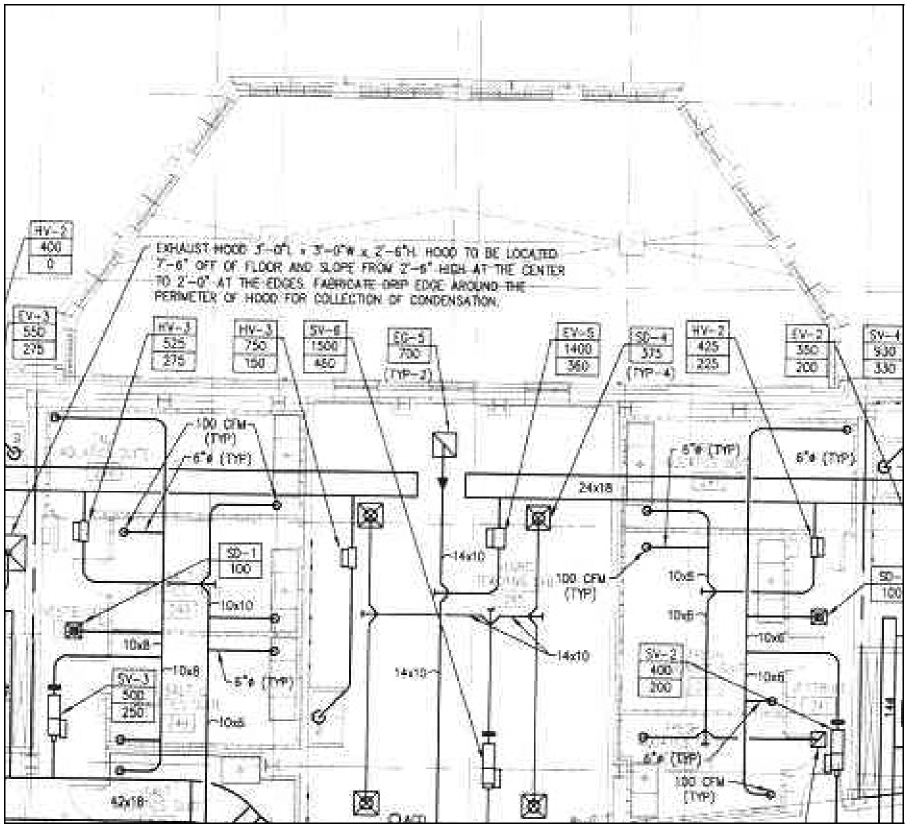
Appendix C

Partial HVAC Ductwork Plans

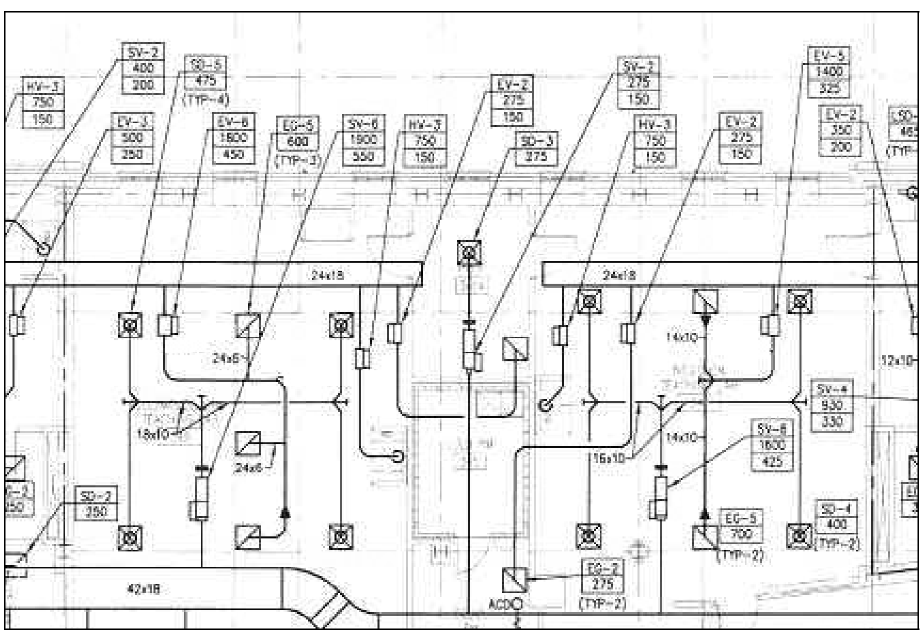


First Floor

Appendix C
Partial HVAC Ductwork Plans (cont'd)



Second Floor



Third Floor

References

ASCE Standard: Minimum Design Loads for Buildings and Other Structures. American Society of Civil Engineers, 2003. (A.k.a. ASCE7-02)

Cengel, Yunus A., Fundamentals of Thermal-Fluid Sciences. Boston, MA: McGraw-Hill, 2001.

Das, Braja M.. Principles of Foundation Engineering. 5th Edition. USA: Brooks/Cole, 2004.

Hanagan, Linda. Lecture Notes for AE 597E: Design and Analysis of Steel Connections. 2005.

Katz, O., Z. Reches and J-C. Roegiers. Evaluation of Mechanical Rock Properties Using a Schmidt Hammer. *International Journal of Rock Mechanics and Mining Science*. <http://earth.es.huji.ac.il/reches/Oded/Schmidt.htm>. 2000.

Manual of Steel Construction: Load and Resistance Factor Design. 3rd Edition. USA: American Institute of Steel Construction, Inc., 2003.

Naeim, Farzad. The Seismic Design Handbook. 2nd Edition. Boston, MA: Kluwer Academic Publishers, 2003.

Stowe, R. L.. Strength and Deformation Properties of Granite, Limestone and Tuff at Various Loading Rates. Defense Atomic Support Agency, 1969.