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
						$(AEcos^2 \bullet)/L$			*$0.9F_yA_g$*	*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
Torsion (N-S)	-15651	in-k
H/400 Drift Limit	1.29	in

$M = V^*e_x$

$M = V^*e_y$

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*d²	(kd) / SUM(kd²)	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	10x10x$^1/_2$	206	17.2	0.581	816.2	30.5	114.0	712.1	557	20.5%	0.16
	2	7x7x$^1/_2$	206	11.6	0.581	550.5	58.9	87.7	480.2	303	28.9%	0.35
	3	6x6x$^1/_2$	216	9.74	0.555	401.7	43.1	38.8	403.2	203	19.1%	0.45
2	1	8x8x$^1/_2$	206	13.5	0.581	640.6	28.3	105.8	558.9	390	27.1%	0.19
	2	6x6x$^1/_2$	206	9.74	0.581	462.2	54.7	81.4	403.2	217	37.5%	0.40
	3	6x6x$^1/_2$	216	9.74	0.555	401.7	40.0	36.0	403.2	203	17.8%	0.50
3	1	10x10x$^1/_2$	206	17.2	0.581	816.2	112.0	418.1	712.1	557	75.1%	0.60
	2	10x10x$^1/_2$	206	17.2	0.581	816.2	216.0	321.7	712.1	557	57.8%	1.05
	3	10x10x$^1/_2$	216	17.2	0.555	709.4	158.0	142.4	712.1	557	25.6%	1.28
4	[redacted]											
5	[redacted]											
6	1	7x7x$^1/_2$	248	11.6	0.727	717.3	64.8	193.5	480.2	245	79.0%	0.39
	2	7x7x$^1/_2$	248	11.6	0.727	717.3	125.0	148.9	480.2	245	60.8%	0.69
	3	6x6x$^1/_2$	256	9.74	0.703	545.6	91.4	65.0	403.2	161	40.4%	0.86
7	1	10x10x$^1/_2$	248	17.2	0.727	1063.6	96.1	286.9	712.1	450	63.8%	0.39
	2	8x8x$^1/_2$	248	13.5	0.727	834.8	185.3	220.8	558.9	315	70.1%	0.78
	3	6x6x$^1/_2$	256	9.74	0.703	545.5	135.6	96.4	403.2	161	59.9%	1.02
8	1	10x10x$^1/_2$	206	17.2	0.581	816.2	94.3	351.8	712.1	557	63.2%	0.50
	2	10x10x$^1/_2$	206	17.2	0.581	816.2	181.8	270.7	712.1	557	48.6%	0.89
	3	10x10x$^1/_2$	216	17.2	0.555	709.4	132.9	119.8	712.1	557	21.5%	1.07
9	[redacted]											
10	[redacted]											
						$(AEcos^2 \cdot)/L$			$0.9F_yA_g$	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.

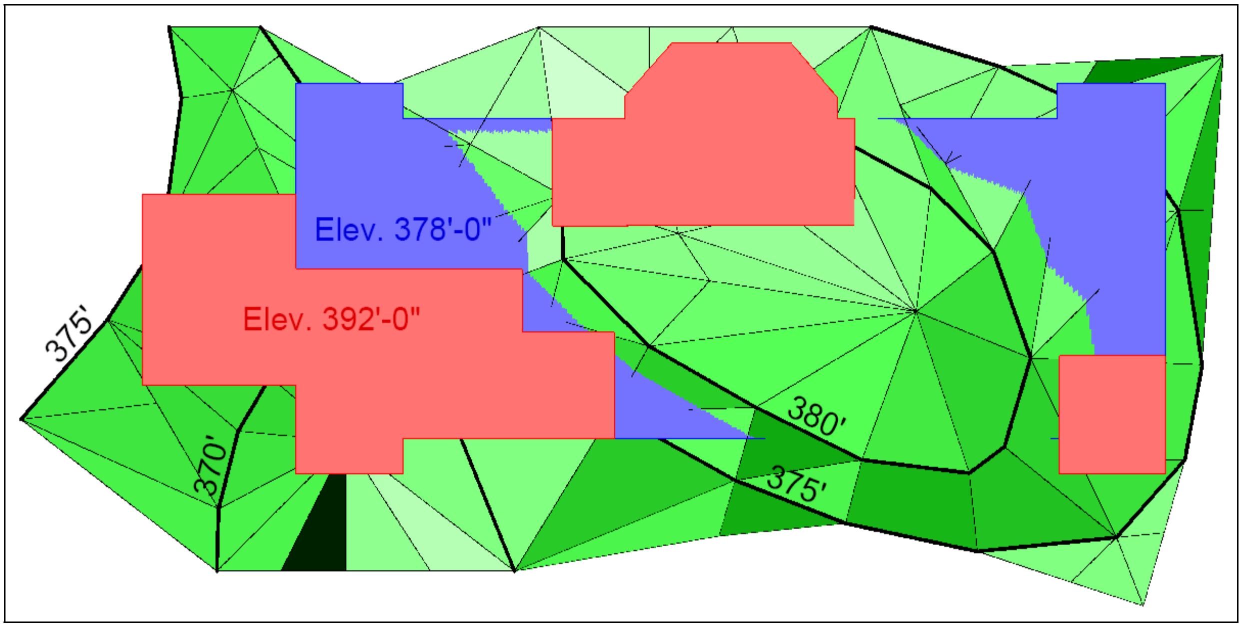

Figure 5.2.1 AutoCAD Approximation of Intact Rock Depth

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, 5^th^ 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_f	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.

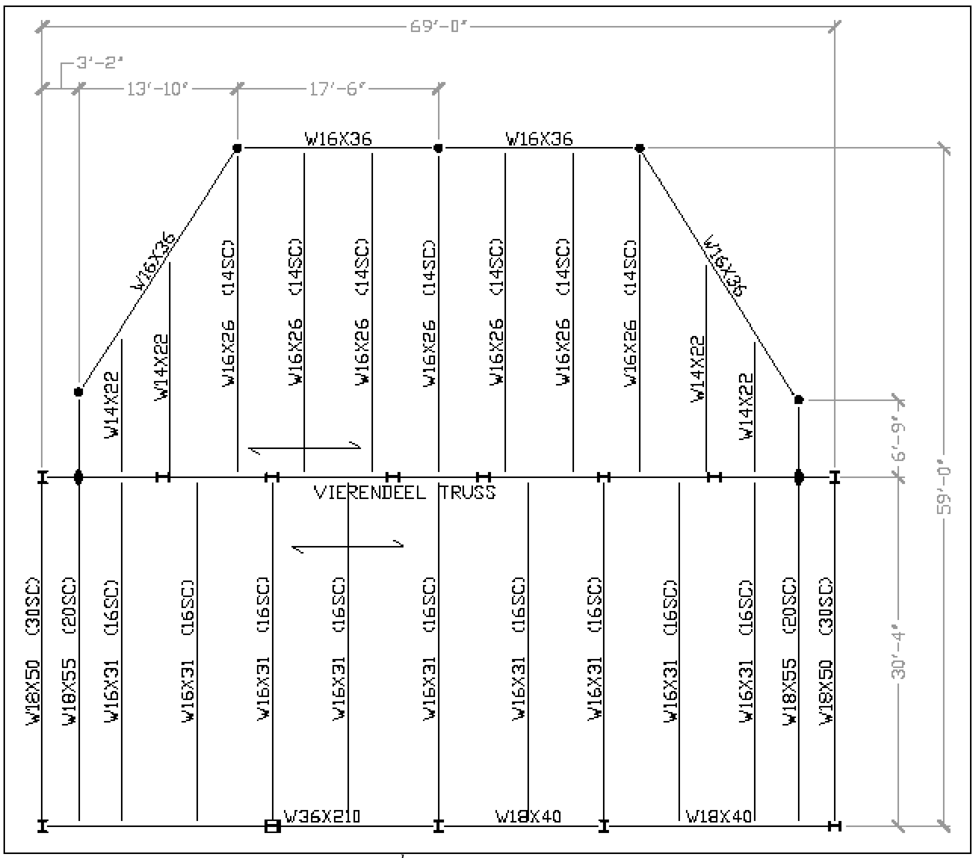

Figure 6.1.1 Partial 2nd Floor Framing Plan – Existing System

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.

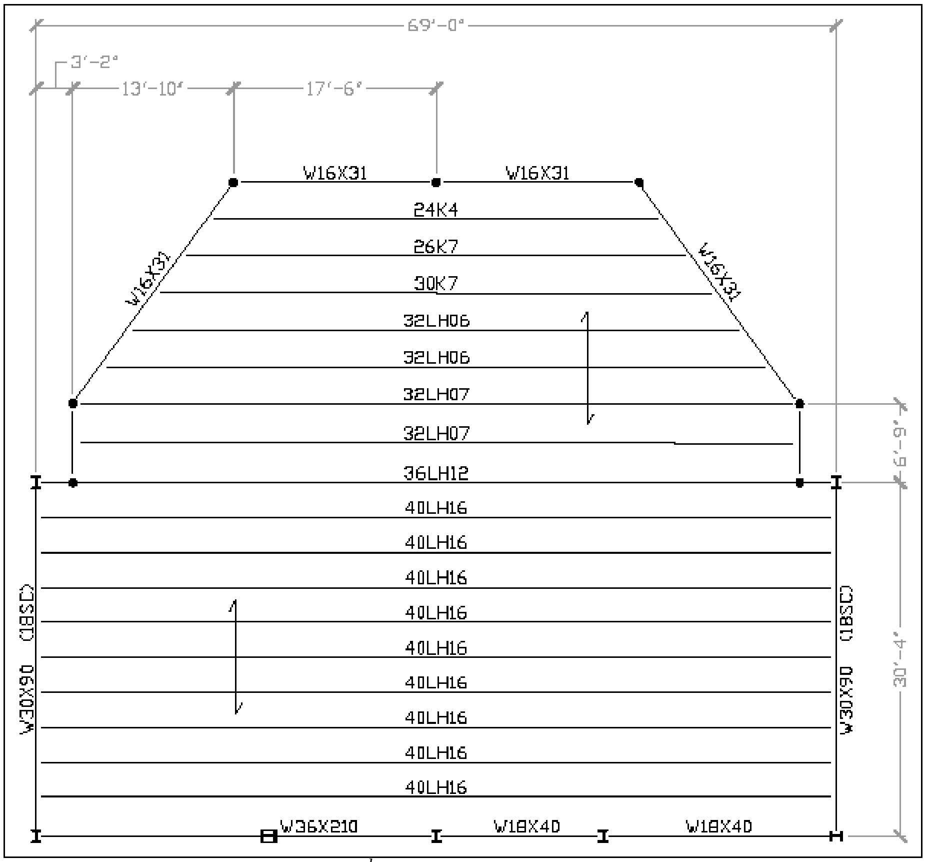

Figure 6.2.1 Partial 2nd Floor Framing Plan – Long Span Joists

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.