INTRODUCTION

The Grant School, located as the red object in Figure 1, has stood in the heart of the George Washington University campus in Washington, D.C. at 2130 G Street NW since 1882 and has housed the School Without Walls since 1977. The "School Without Walls" name comes from the faculties' encouragement for students to use Washington D.C. as an active classroom, thus not restraining learning to the walls of the senior high school.
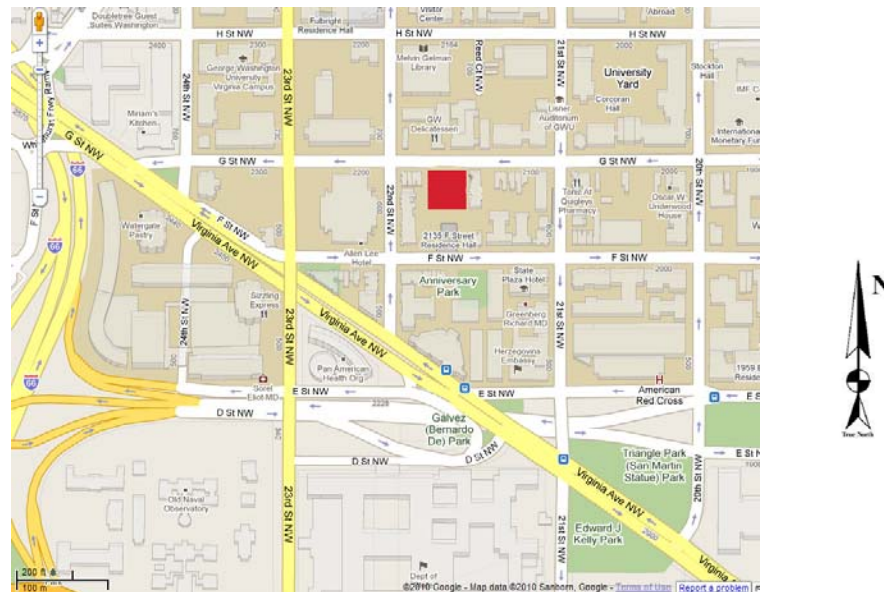

Figure 1: Area Map

The original 32,300 square foot, three story school was in dire need of modernization and expansion due to the increasing number of students and outdated mechanical and electrical equipment. The 68,000 square foot addition and renovation, as seen in blue in Figure 2, blends the 19th century school with a modern design. This is achieved by combining existing brick patterns with glass, steel and curtain walls. The School Without Walls project is expected to receive LEED Gold Certification.

The existing three story school is made up of four large classrooms per floor, one at each corner of the square building. The new addition to the school provides an additional two large classrooms on each floor, an open atrium space, a large student commons, roof terrace area and a library. The basement was also reengineered and redesigned to serve as scientific laboratories for the school.
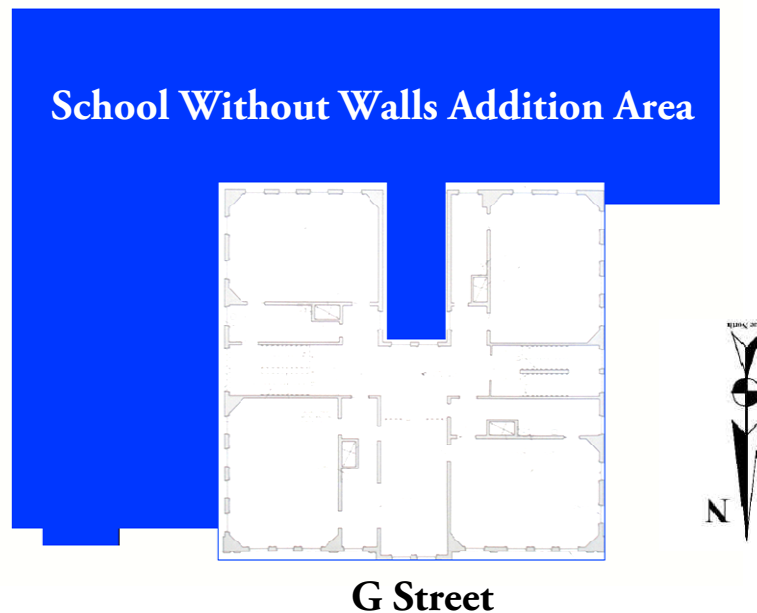
School Without Walls Addition Area

G Street

Figure 2: Floor Plan Showing Expansion

EXISTING STRUCTURAL SYSTEM

The 68,000 square foot addition to the School Without Walls project is located in blue in Figure 2. Due to expansion joints located at the interface of the addition and the existing building, the structural systems of the existing and new building work independently. A detail of this expansion joint can be viewed in Figure 3. As stated in the drawing, the expansion joint along the east side of the existing building is 4", and is 2" along the south side.
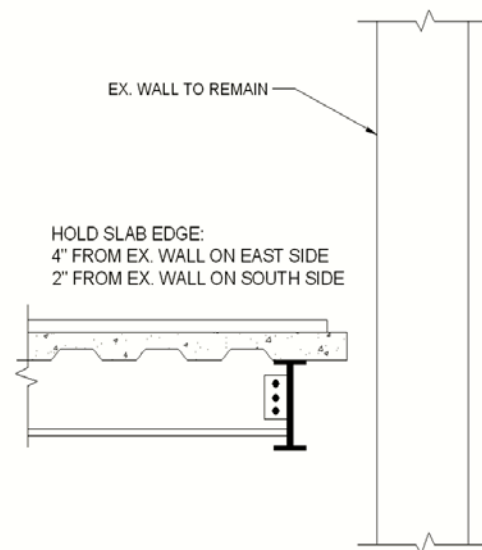

Figure 3: Expansion Joint Detail

The new addition to the School Without Walls itself is divided by an expansion joint. This expansion joint therefore creates a total of three independently acting structural systems. The division of the new addition can be viewed in Figure 4. These separate structural systems, which can be distinguished in Figures 4 and 5, will be referred to as "Area 1" and "Area 2" throughout this report.
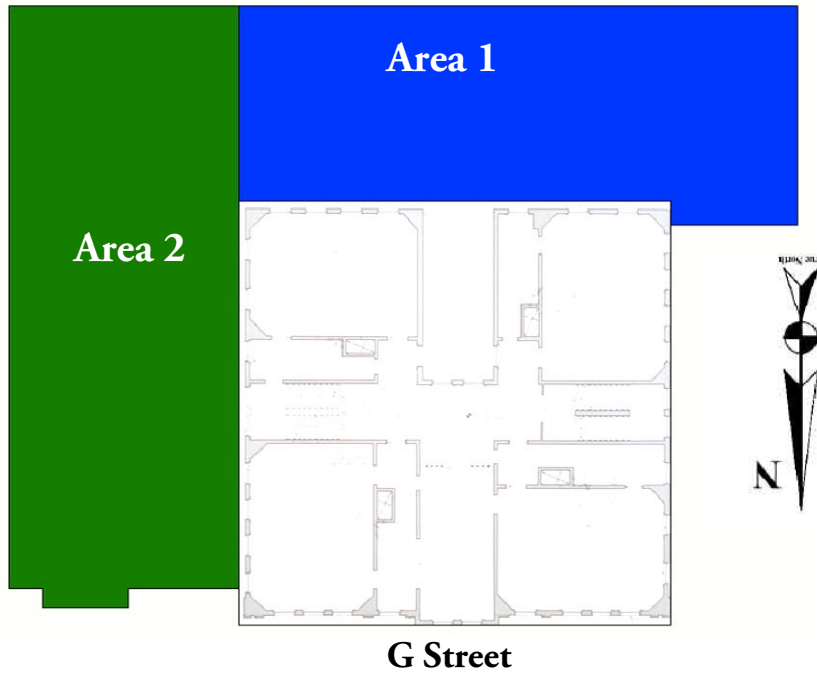
Shaun Kreidel
Structural Option
School Without Walls

April 12, 2010
Advisor: Dr. Hanagan
Final Report

Figure 4: Floor Plan Showing Building Separation

Figure 5: West Elevation

Foundation

The geotechnical engineering study was performed by Thomas L. Brown Associates, P.C. on January 28, 2007. After performing a series of in-situ tests, considering the lab test results, anticipated loads, and settlement analyses, a shallow foundation consisting of reinforced cast-in-place spread footings and grade beams was deemed appropriate. Based on the testing and analysis, the footings should be designed for an allowable bearing capacity of 3.0 ksf.   The addition utilizes typical footings which are 2' 6" wide by 2'0" deep and rest on compacted earth 3'0" below the top of the slab-on-grade. Grade beams are also used in the foundation of the new addition. The beams measure 30"x30" along the east side and 30"x24" along the south side of the building.
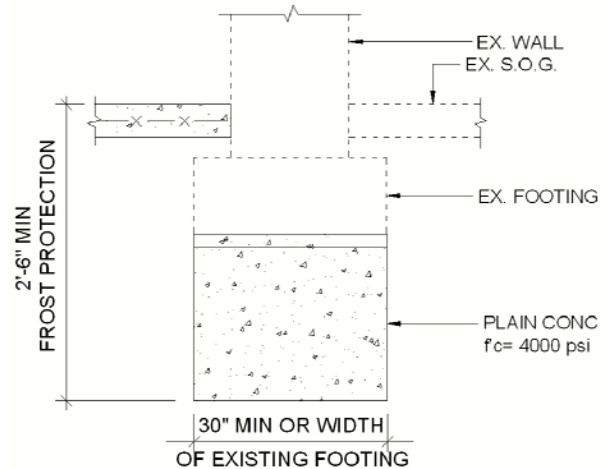

Figure 6: Underpinning Detail

Due to the increased load and the disruption of earth, underpinning the existing footings of the school became necessary. An underpinning detail is located in Figure 6. The underpinning sequence was performed in sections of no larger than 4 feet wide, approximately spaced 12-15 feet apart.

Floor System

The floor system of School Without Walls is a composite steel system. The floor slab of the new addition is 3 ¼" LWC topping over a 2" 20 GA LOK composite steel floor decking, bringing the total floor slab to 5 ¼" thick. Along the top flange of the beam, ¾"x4" long headed shear studs are used for composite action. A section of this floor system is shown above in Figure 7.
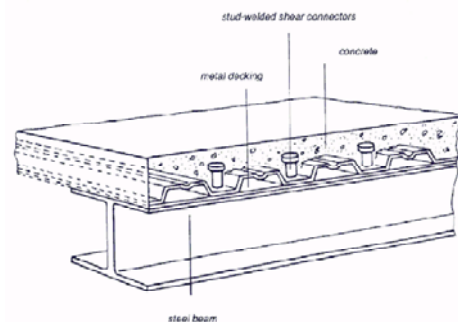

Figure 7: Typical Composite Steel Construction (www.epitech.com)

Lateral System Summary

The lateral system of School Without Walls works as three different systems due to expansion joints as stated before and show above in Figures 4 and 5. Both braced frames and shear walls, located in blue and green respectively in Figure 8, are used to resist lateral loads that are applied to the building.
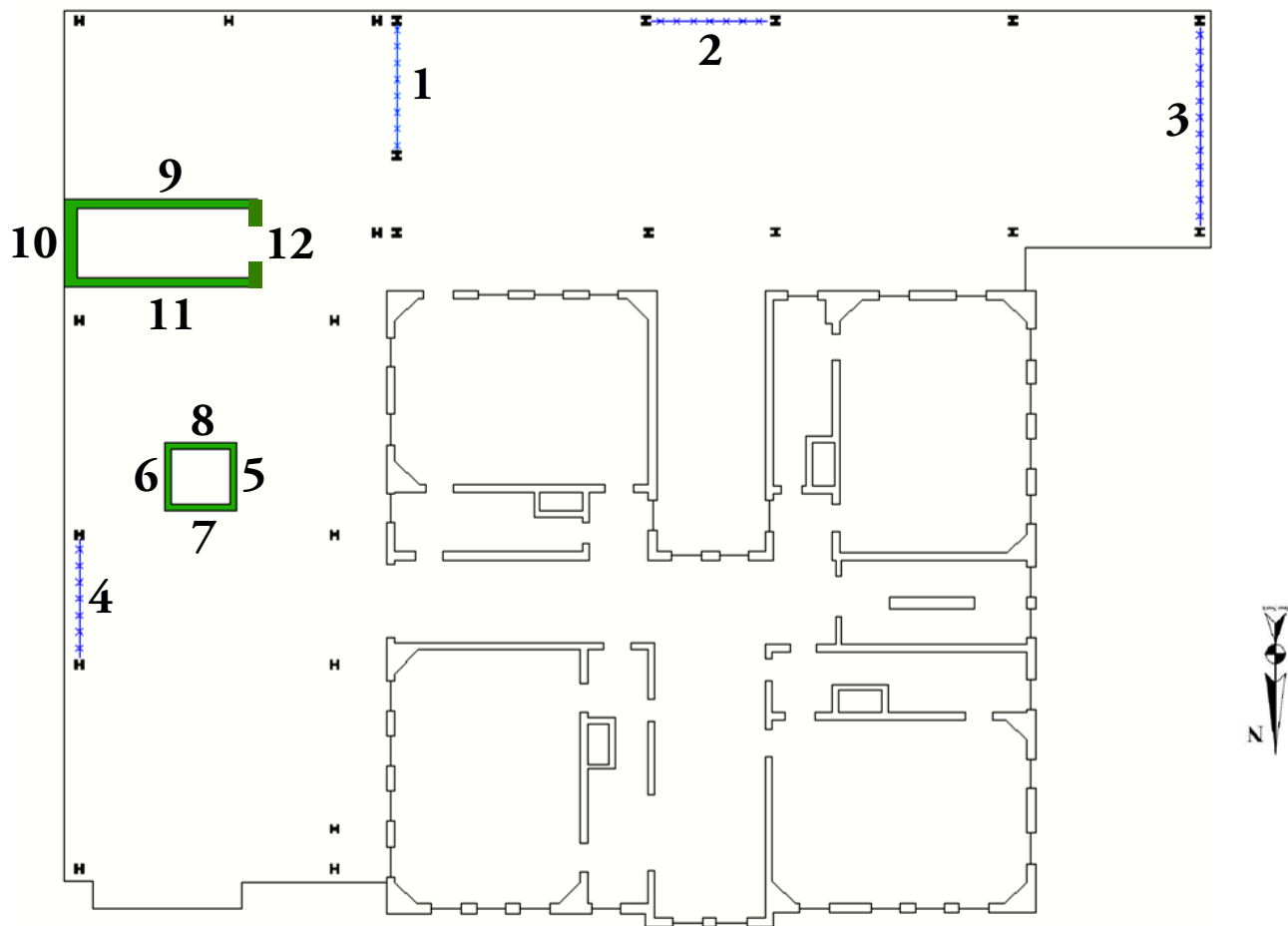

Figure 8: Summary of Lateral Systems

Area 1 Lateral System

The two story structure supporting the outside roof terrace (Area 1) utilizes only braced frames for lateral support. All of the braced frames located in this section of building are comprised of only HSS6x6x3/8 sections. Diagonal, cross, and chevron bracing are utilized in braced frames 1, 2 and 3 respectively as labeled in Figures 9, 10 and 11. All of the braced frames extend the entirety of the two story section of building.
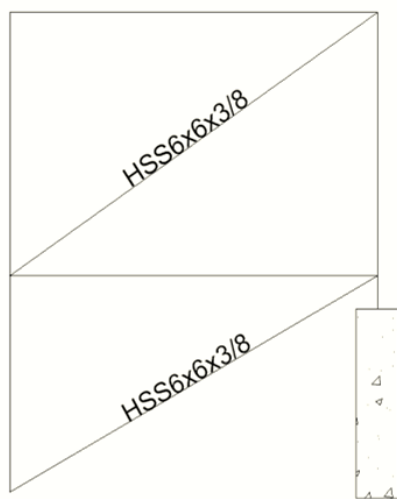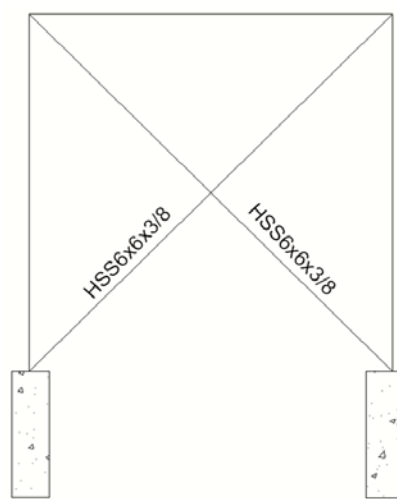

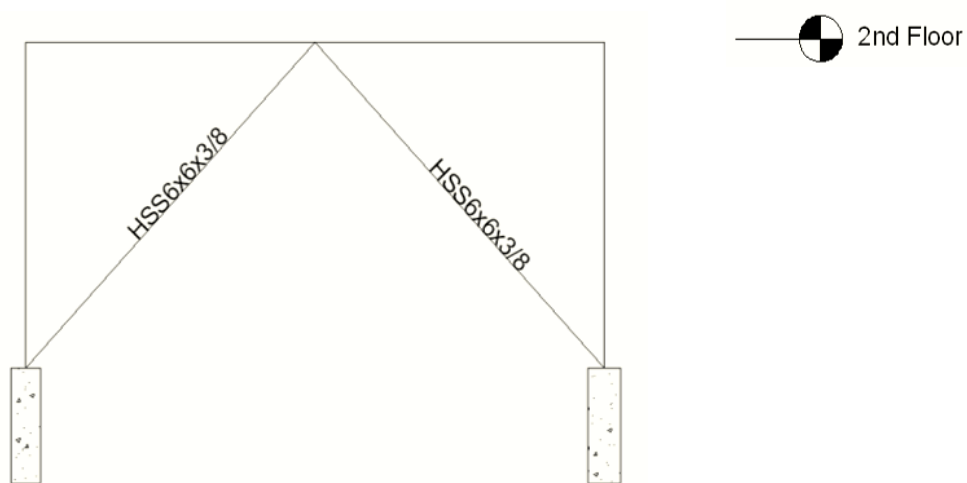*Figure 9: Braced Frame 1*   *Figure 10: Braced Frame 2*

Figure 10: Braced Frame 3
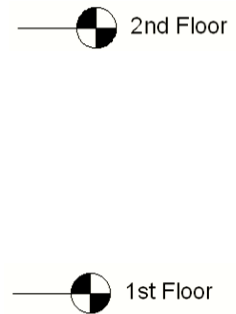
Area 2 Lateral System

The four story structure supporting the library, referred to as Area 2 in this technical report, uses a combination of a braced frame system and a shear wall system to resist lateral loads. The braced frame, comprised of HSS square sections reaches from the ground to the roof level as seen in Figure 11. The shear walls are located around both the elevator core and the stair core. The shear walls surrounding the stair well are all 12" thick and are reinforced with #5 vertical bars spaced at 10" on center and #4 horizontal bars spaced at 12" on center at each face. The shear walls surrounding the elevator core are all 8" thick and are reinforced with #5 vertical bars spaced at 10" on center and #4 horizontal bars spaced at 12" on center at each face.

Figure 11: Braced Frame 4

PROBLEM STATEMENT

Currently, the School Without Walls project current floor system calls for steel wide flange beams which range from W10 to W33 se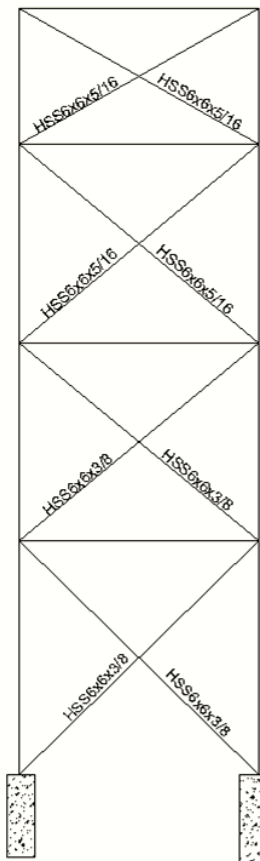ctions. With the addition of a 5 ¼" decking system, the total floor depth amounts to 38 ¼". Due to the updated mechanical system of the existing building and the addition of the science labs a larger clearance between the ceiling and the floor system above would create an easier coordination of the electrical and mechanical systems in the building. In addition to the benefits of a larger ceiling to floor clearance area, the cantilevered sections are typically more easily constructed in concrete than in steel. This is due in part to the welded moment connections required in steel cantilever construction.

SOLUTION METHOD

To limit the total depth of the floor system, the gravity resisting systems were altered from the current steel composite system to a concrete system. An important goal was to keep the column lines relatively in the same position, therefore effectively meeting the layout which was required by the architect.

A post-tensioned concrete system is very applicable to this project because of the relatively large spans the building layout requires and the desire for minimal floor deflection. Post-tensioning of the beams and the slab allowed for the desired cantilever along the face of the existing building and reduced deflections.

Concrete columns were also designed as part of this concrete system and replaced the current steel columns.

The expansion joints which separate the School Without Walls into three different zones remained, therefore, creating three different lateral systems. The lateral systems of both Area 1 and Area 2 were redesigned, introducing concrete moment frames and thus removing the both the braced frames and shear walls which currently resist lateral loads.

ACI 318-05 was referenced throughout in the design of the concrete system. PCA column was utilized as an aid in the structural design of the columns and ADAPT PT was used to assist in the design of the post tensioning in the building. An ETABS model was also built in order to effectively analyze the lateral system of the building. Hand calculations were performed to confirm and verify the accuracy of the computer programs.

ALTERED FLOOR PLANS

Area 1

Throughout the structural redesign of the two story addition of the School Without Walls, the architectural drawings were continually referenced to ensure that the structural components did not interfere with the designed layouts of each floor. In the attempt not to change the architectural layouts, column line positions were altered as little as possible. A revit model was built as an aid to study how the alterations of the structural components affect the architectural layout. Column lines G, 7, and 5, as displayed in Figure 12 were the only column lines which were altered in this portion of the School Without Wall's structural redesign. Column lines G and 7 were both moved 6.25" towards the interior of the building. Column line 5 was moved 14.25" south of the original line provided by the architectural drawings.
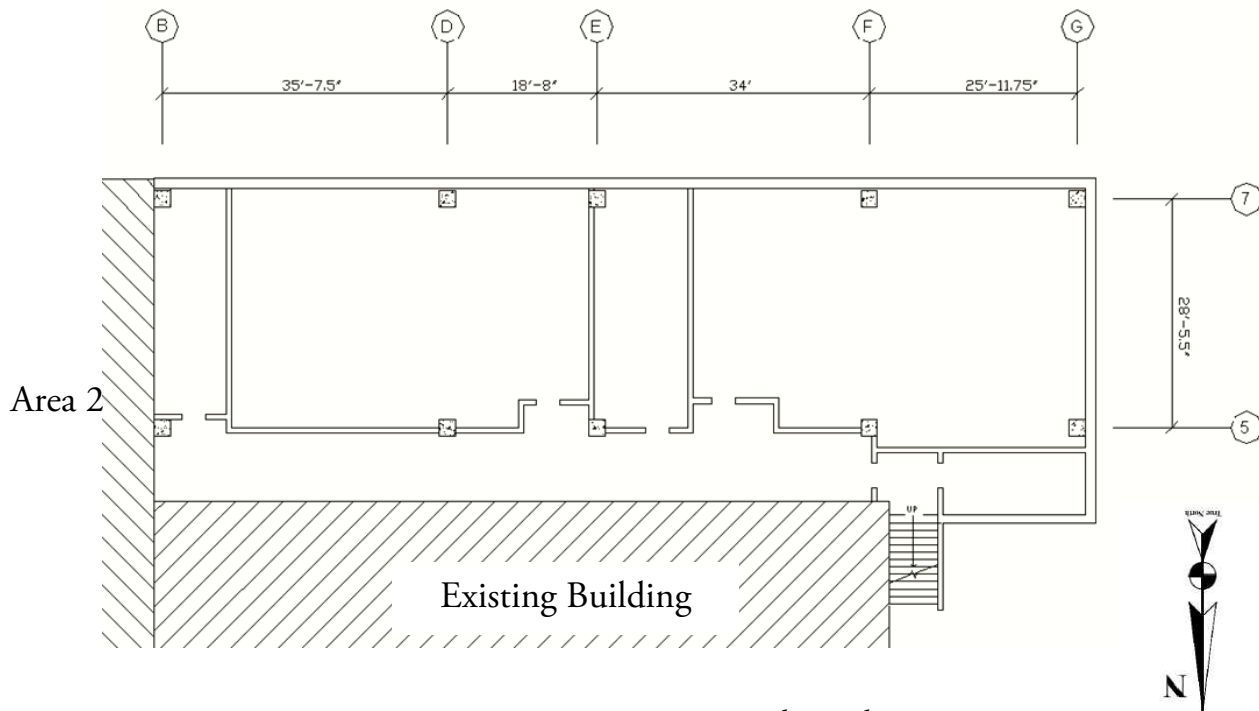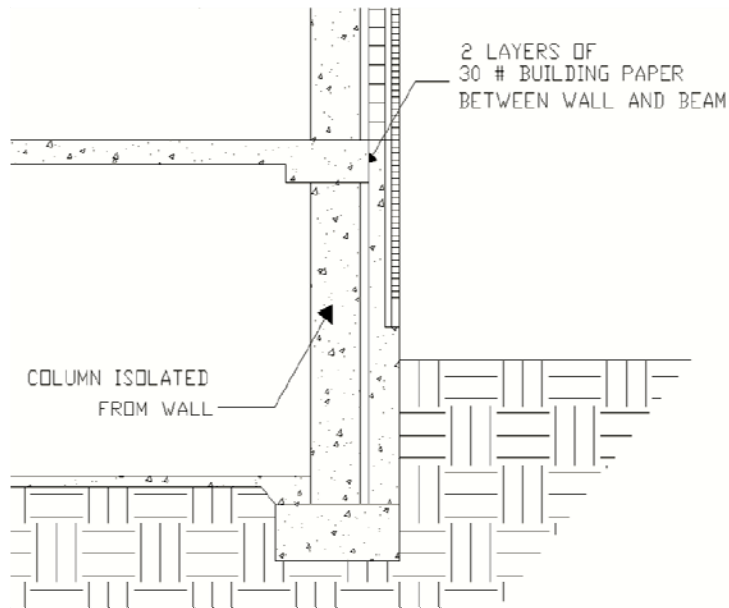

Figure 12: Basement Floor Plan

In post tension design, it is important to allow for the shortening of the slab without restraint. Removing this restraint reduces the amount of cracking due to shrinkage which can be viewed in Figure 14. To avoid restraining the slab, the column lines were displaced from the foundation wall. A typical exterior wall section of Area 1 can be viewed in Figure 13.

Figure 13: Exterior Wall Section

Because of the expansion joint located at the interface of the existing school it was necessary to cantilever the slab on the first floor in the area supporting the Student Commons. The slab cantilevered 9'-2.25" from column line 5 in this area as seen in Figure 15.
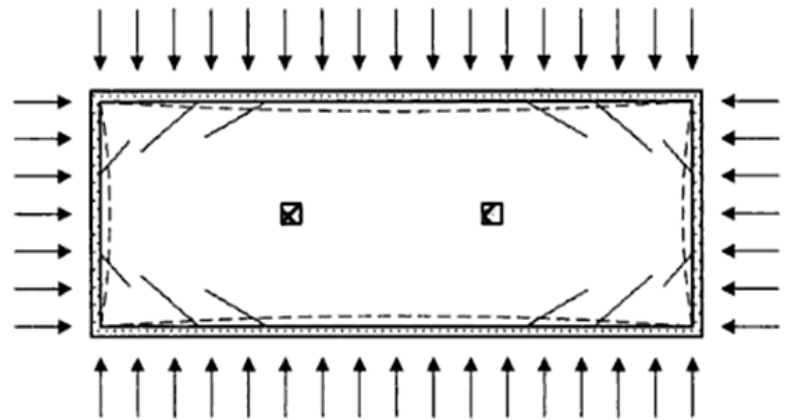
Figure 14: Cracking Due to Wall Restraint

The service area located on the first floor, as stated in Figure 15 works independently from the reinforced loading dock. A section cut is taken at this point in the North-South direction as seen in Figure 16 to display the interaction of the loading dock and the post tensioned floor slab. Because the slab is not restrained to the wall, shortening is possible which limits the amount of cracking due to shrinkage.
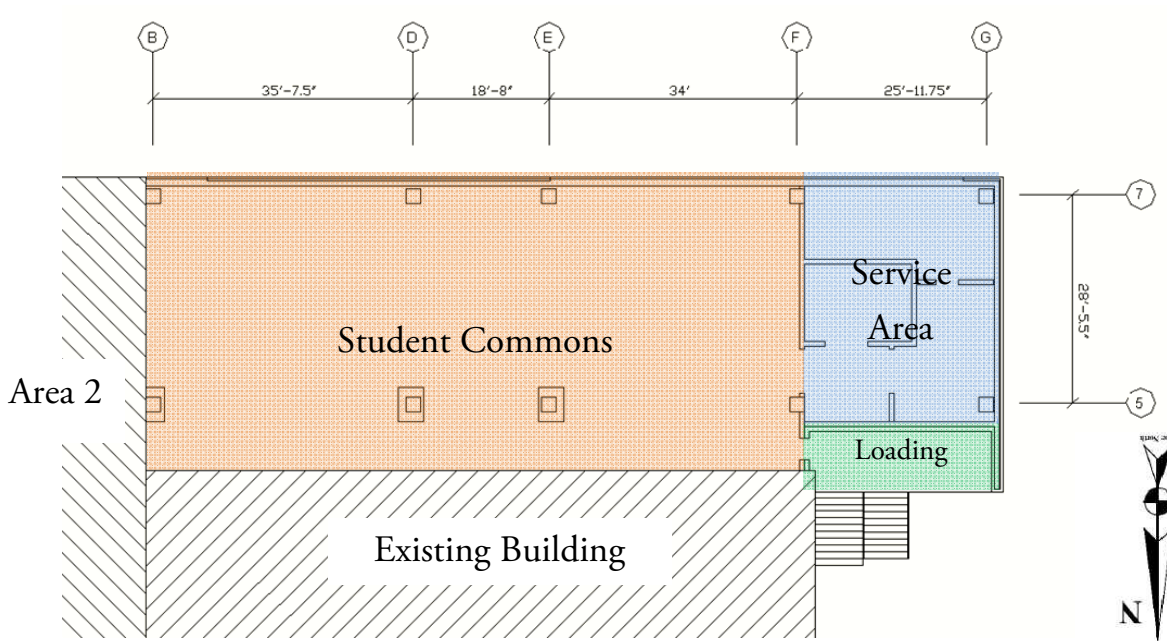
Figure 15: First Floor Plan
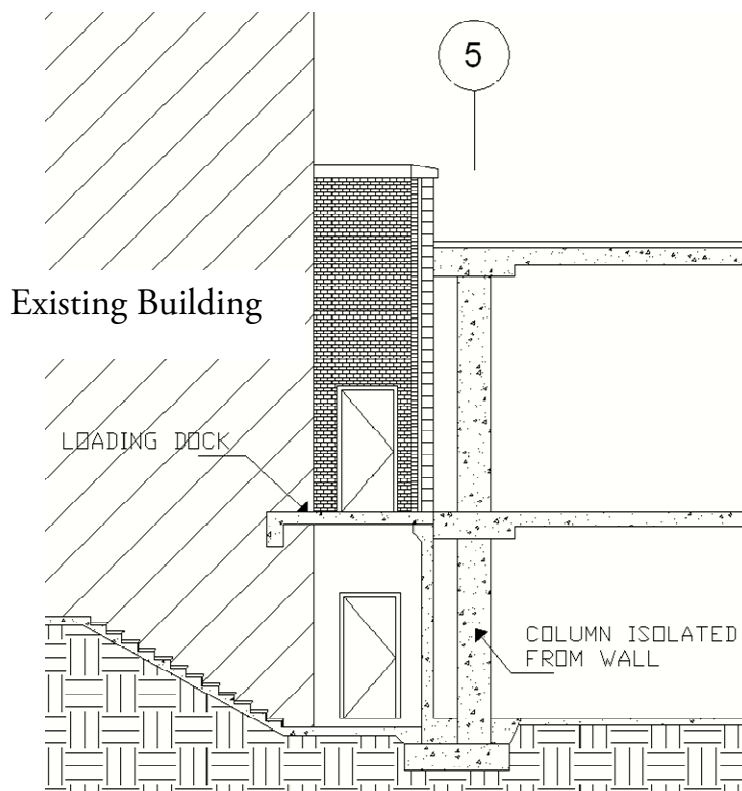
Figure 16: Service Area- Loading Dock Section

Area 2

Likewise as in the structural redesign of Area 1, the architectural drawings were continually referenced in the attempt not to change the architectural layouts. Column lines positions were altered as little as possible. A revit model for Area 2 was built as an aid to study how the alterations of the structural components affects the architectural layout. Column line A was shifted 14" to the west and B was shifted 16" to the east in order to accommodate the beam geometry. Column lines 1 and 7 were both shifted 6" to the south and north respectively. Because of the cantilever capablity of concrete, columns along A.5 and C, located in red, were deleted. The column located in green in Figure 17 was the only column which was added to the School Without Walls. This column necessary to divide the span between 4 and 7 (a span of 46'-6.5").
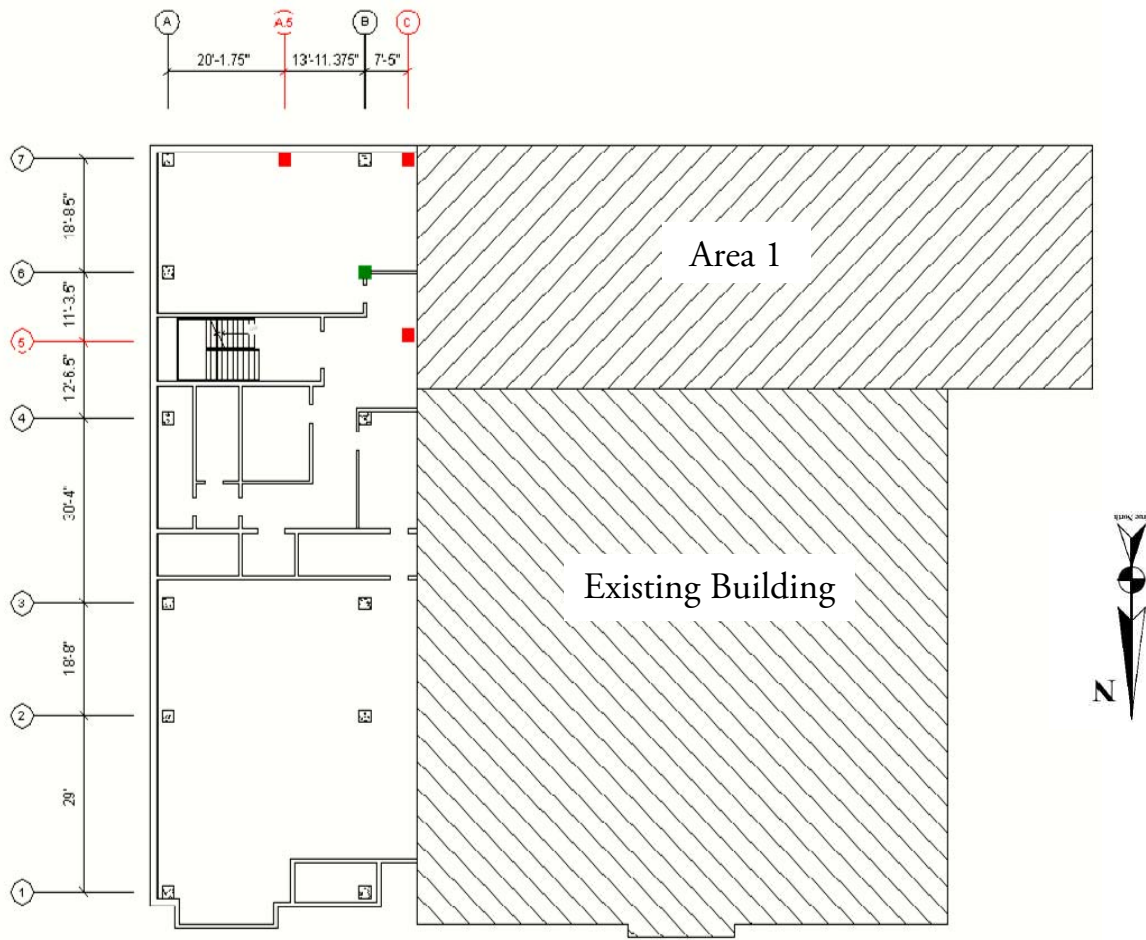
Figure 17: Basement Floor Plan

Because of the expansion joint located at the interface of the existing school it was necessary to cantilever the slab in the East West direction. The slab cantilevers 9' from column line B in this area as seen in Figure 18. Because of the presence of an atrium space, as located as the "X" areas in Figure 18, the slab is simply supported on the second, third and fourth floor. Because of the increased of span and removal of the cantilever in sections, the slab thickness was increased.

Area 2 required an exterior wall construction much like the type shown in Figure 13 in order to allow for the shortening of the slab without restraint.

It must be noted that only the structure supporting floors 1 through 4 were redesigned in concrete. The remaining penthouse level housing the library remained constructed as steel utilizing both wide flange shapes and hollow steel sections. Even though the mixing of trades is not an ideal situation, the complex design of the roof required steel members to be used.
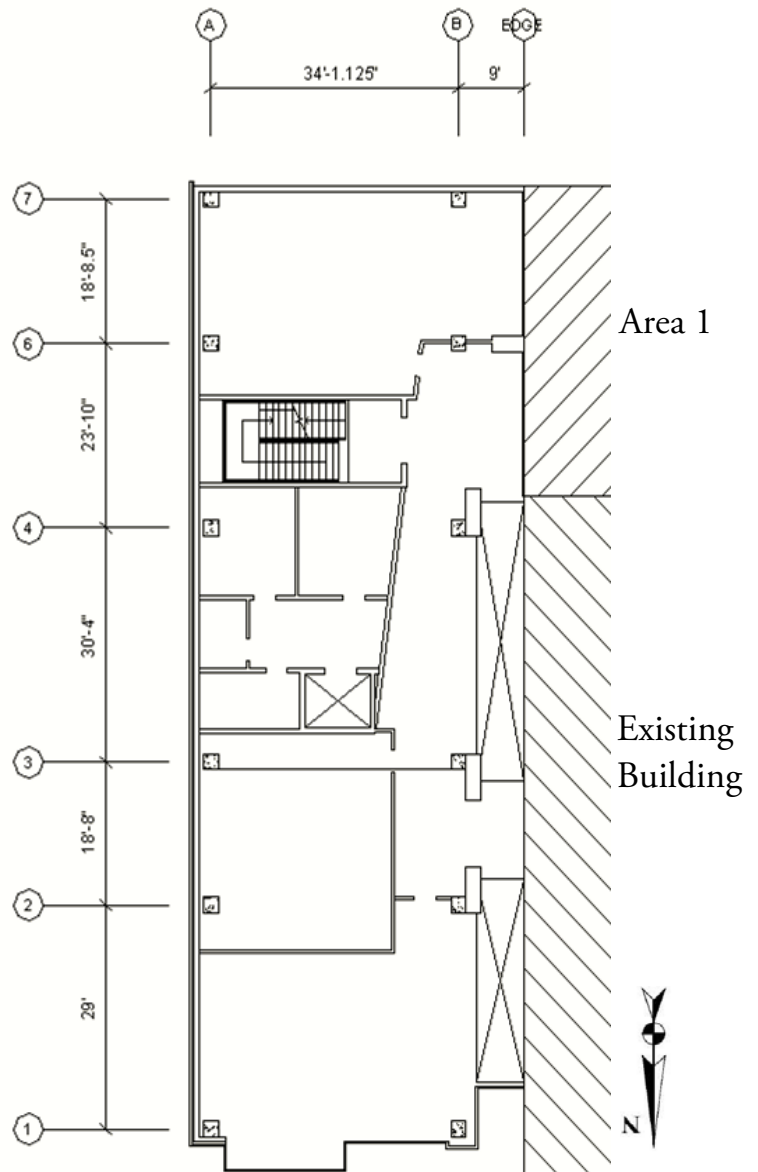
Figure 18: Second Floor Plan

POST TENSIONED DESIGN

The method of post-tensioning involves the tensioning of steel tendons after the concrete has hardened. Unbonded tendons were utilized in all post-tensioning applications in this structural redesign. These tendons are fabricated with a plastic sheathing and grease to prevent a bond with the concrete.   All of the tendons used in design were ½" diameter, 7-wire strands. The tendon properties can be viewed in Table 1.

| Area= .153 in^2 |
|---|
| f_{pu}= 270 ksi |
| Estimated Prestress Loss= 15ksi |
| f_{se}= .7*(f_{pu})- 15ksi= 174 |
| P_{eff}=A* f_{se}= 26.6 kip/tendon |
| P/A: Max=300, Min=150 |

Table 1: Tendon Properties

The beams were modeled as either as T or L sections based on their geometry. The effective flange was calculated in accordance with Chapter 8 of ACI 318-05. ADAPT PT was used as an aid in designing the post-tensioned beams. The beams were designed in accordance with Chapter 18 of ACI 318-05. After analyzing each beam in the ADAPT PT program, the tendon drape was adjusted to best balance the dead loads. The balanced dead loads, even though not a code requirement, were targeted as 70-80% of the dead load for beams and 60-70% of the dead load for slabs. Superimposed dead loads were not included into this percentage because they are not present at the time of stressing.

Area 1 Beams

The four beams located in Area 1 span in the east-west direction as seen in Figure 19. This orientation was chosen because the number of beams can be limited which allowed for more ceiling to floor area. Trial sizes of the beams were determined using an L/d ratio of 30. These trial sizes were adjusted based on the architectural components present.

Figure 19: Isometric View of Column

The finalized design of the beams resulted in 18" deep beams supporting the first floor, located in red and green, and 20" deep beams supporting the roof terrace, located in blue and magenta. The beams spanning in the direction of column line 5 are 56" wide, and the beams in the direction of column line 7 measure 40" wide. A detailed output including tendon profiles and required post-tension forces and required reinforcement for Beams B1-B4 as labeled in Figures 19 and 20 can be viewed in Appendix A.
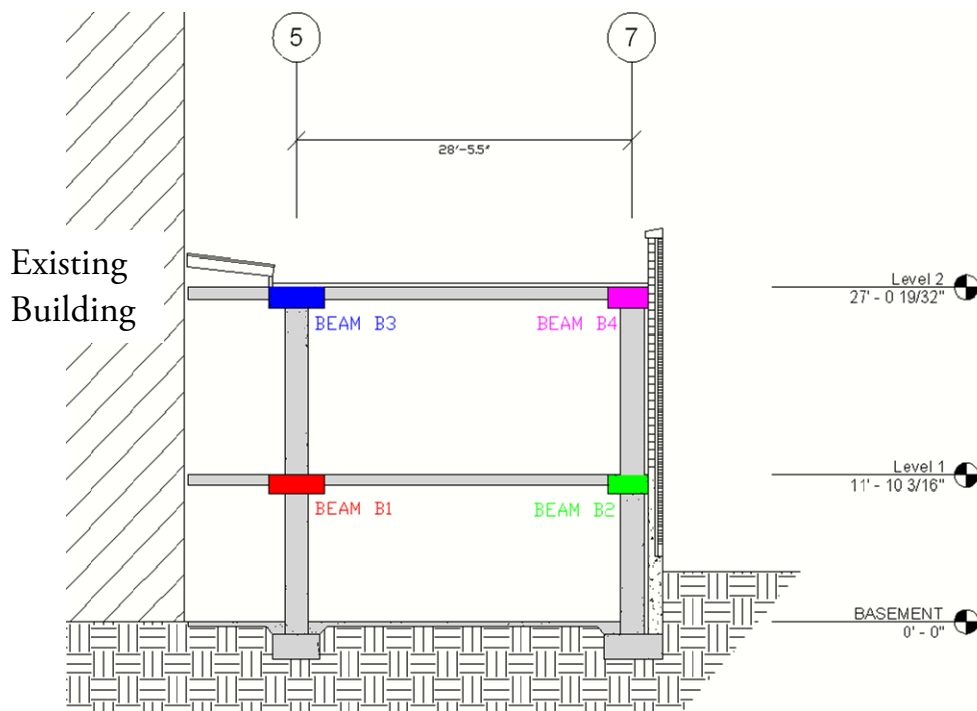
Figure 20: East-West Section Cut of Area 1

Area 1 Slabs

The post-tensioned slab in Area 1 spans in the north-south direction perpendicular to the post tensioned beams located in Figures 19and 20. Design was carried out using ADAPT PT as an aid while complying with Chapter 18 of ACI 318-05.   A spot hand calculation was performed to verify the accuracy of the ADAPT PT program. It was determined that the hand calculation results and computer output were comparable.

Preliminary design for the slab thickness was determined by using an L/d ratio equal to 35. The live loads of these slabs were not be reduced because of section 4.8.4 of ASCE 7-05, which states that live loads of 100 lb/ft^2 or less shall not be reduced in public assembly areas.

During design, the unit strip method was utilized and the one way post tensioned slab was modelled as a 12" wide beam. The transverse beams were modelled into the analysis as well which shortened the clear span of the slab.

After completing the analysis, it was found that the required slab on the first floor measures 10" thick throughout and the slab on the second floor, or roof terrace measures 11". In design, it was assumed that because the roof is acting as an assembly space, therefore there will not be an instance where both the live load and snow load will exist concurrently. The assembly load will control over the snow load, therefore the current design for 100 psf of live load is adequate. The super imposed dead load of the roof terrace was also increased for design to account for the roof pavers that were specified by the architect.

A detailed output of these slabs containing post tension force, profile and rebar specification can be found in Appendix B.

Shaun Kreidel
Structural Option
School Without Walls

April 12, 2010
Advisor: Dr. Hanagan
Final Report

Area 2 Beams

The ten beams located in Area 2 span in the north south direction as seen in Figure 21. This orientation was chosen because the number of beams can be limited which will allow for more ceiling to floor area. Trial sizes of the beams were determined using an L/d ratio of 30 and a b/d ratio of 3. These trial sizes were adjusted based on the architectural components present.
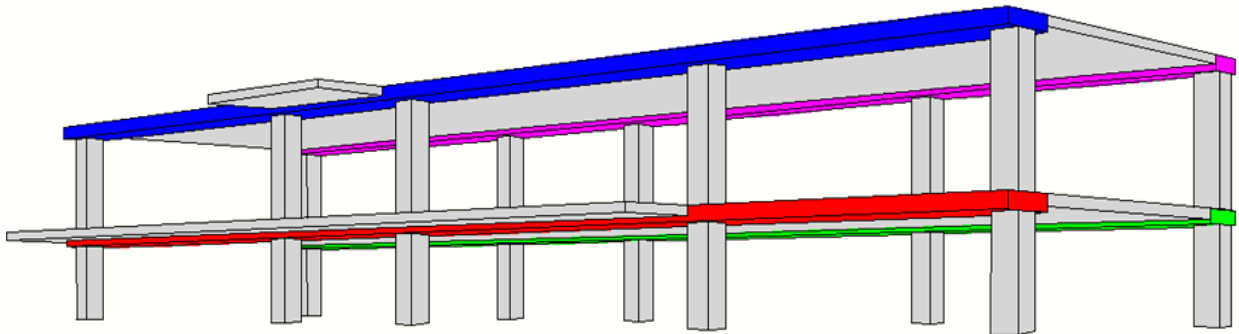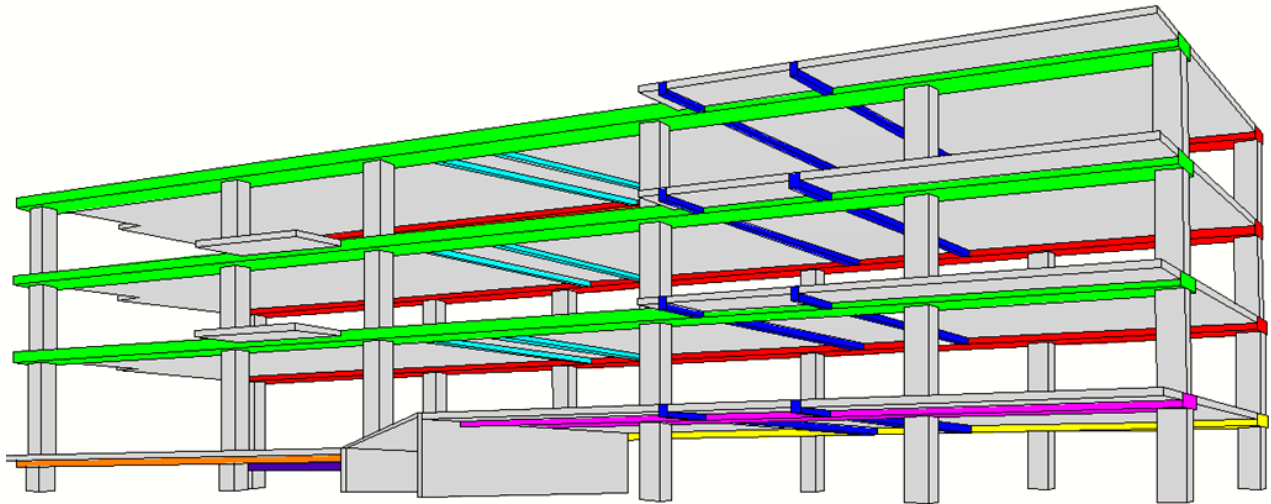

Figure 21: Isometric View of Area 2

The beams supporting the second through fourth floors referred to as Beams 9 and 10 located in green and red respectively in Figure 21, run the extent of the building. Due to the split entrance/first floor level, four beams were designed to carry the loads of the areas. These beams located in Magenta, Yellow, Orange and Purple in Figure 21 are referred to as Beam 5, Beam 6, Beam 7 and Beam 8 respectively throughout the remainder of this report and the Appendices.

The dark blue and cyan colored beams running in the east west direction are necessary to carry the loads of the partition walls from the stairwell and elevator shaft, as well as the one way mildly reinforced slab which spans between each section. These beams were assumed to impose a point load on the main beams. The loads can be viewed in Appendix A.

The finalized design of the beams resulted in 18" depth for beams located on the entrance and first level in Area 2. The beams on the remaining floors in Area 2 all measure 20" deep.

In the direction of column line B, columns measure 60" wide and those spanning in the direction of column line A are 40" wide for all floors. A detailed output including tendon profiles and required post-tension forces for Beams B5-B10 as labeled in Figures 21 and 22 can be viewed in Appendix A.
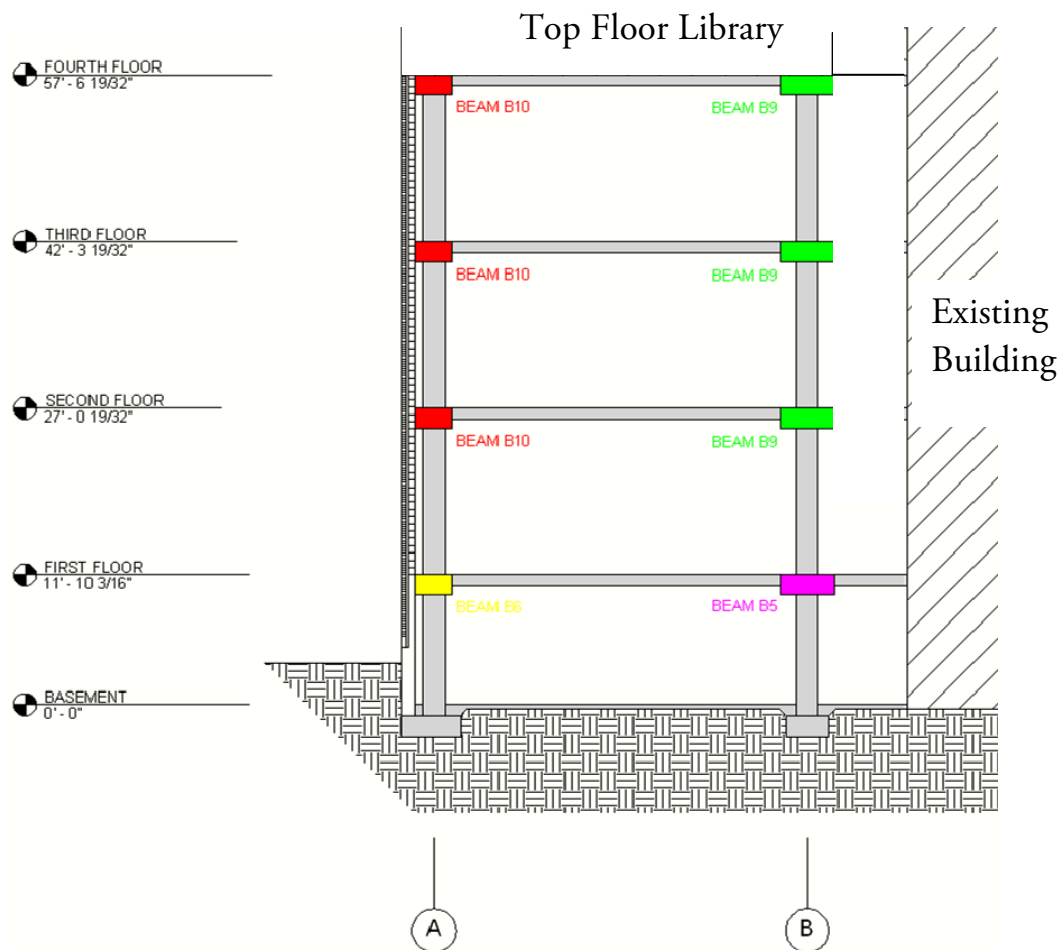
Figure 22: North-South Section

Area 2 Slabs

The post-tensioned slab in Area 2 spans in the east-west direction perpendicular to the post tensioned beams located in Figure 22. Design was carried out using ADAPT PT as an aid and complied with Chapter 18 of ACI 318-05.

Preliminary design for the slab thickness was determined by using an L/d ratio equal to 35. Live loads were reduced according to Section 4.8 of ASCE 7-05 using the equation:

$$L = L_o \left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}} \right) \qquad (4\text{-}1)$$

Similar to the design method used to calculate the slabs in Area 1, the unit strip method was utilized and the one way post tensioned slab was modelled as a 12" wide beam. The transverse beams were modelled into the analysis as well which shortened the clear span of the slab.

It was found that the required slab on the first floor, entrance level and the slab that supports the southern most classroom all measure 10" thick. The simple span areas require a 12" slab. These slab areas can be distiguished in the red and blue areas respectively in Figure 23.

Figure 23: Second Floor Plan Showing Slab Thicknesses

To account for the slab area around the stair wells, a one way 8" reinforced slab is utilized located in orange in Figure 23. This slab spans between the transverse beams indicated in dark blue and cyan in Figure 21.

A detailed output of these slabs containing post tension force, profile and rebar specification can be found in Appendix B.

LATERAL DESIGN

The lateral system was changed from one that utilized shear walls and braced frames to one that relies on concrete moment frames. In the direction in which load is applied parallel to the beams, the post tensioned beams and columns work together to create the moment frame. In the case where the lateral load is being applied perpendicular to the beams, the slab interacts with the columns to create a moment frame.

The stiffness properties of the lateral force resisting members were modified assuming cracked sections. The section properties that were used are:

$$\text{Beams: } I_{eff} = .35I_g$$
$$\text{Columns: } I_{eff} = .70I_g$$

To aid in creating an ETABS model, the ACI Journal entry "Dynamic Responses of Flat Plate Systems with Shear Reinforcement" by Thomas Kang and John Wallace was referenced. From this document, guidelines on modeling and calculating the effective width of the slab were determined and are summarized in Figure 24. The slab was modeled with rigid end zones at the joints.

α: Effective Beam Width Factor= .65
β: Coefficient accounting for Cracking= .5
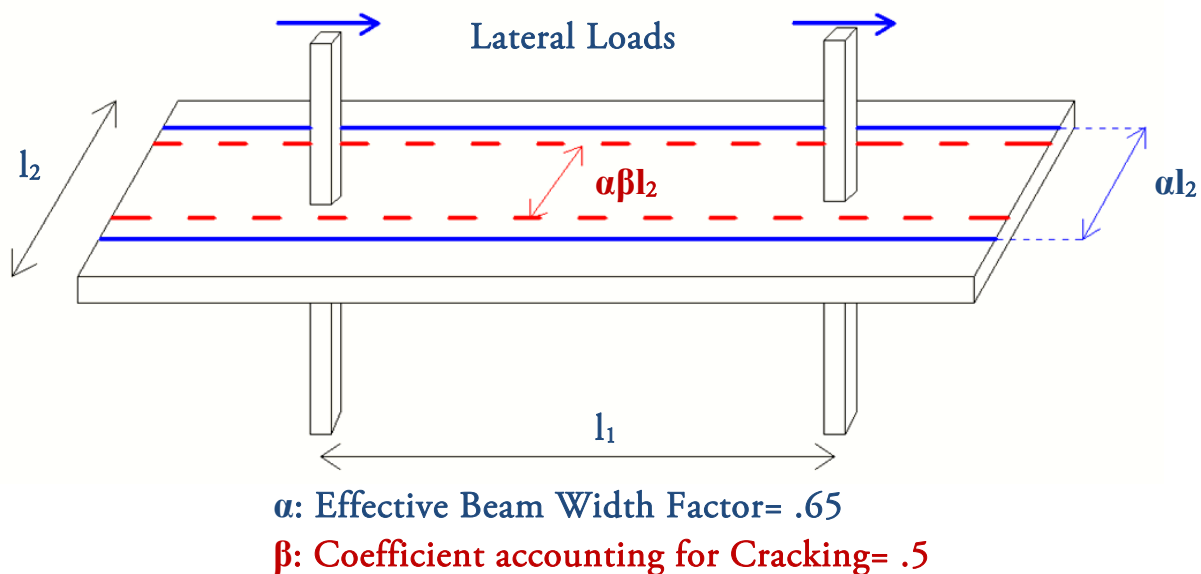
Figure 24: Effective Beam Width Model

Area 1 Lateral Loads

Area 1 has a total height of approximately 22 feet above grade, therefore, it is considered a low rise building and was designed in accordance to Method 1 as listed in Chapter 6 of ASCE 7-05. The applied lateral pressures due to wind are located in Table 2. To simplify calculations, pressures resulting from Zone A, or the End Zone, located in Figure 25 were applied for the entire building. This resulted in a conservative design.
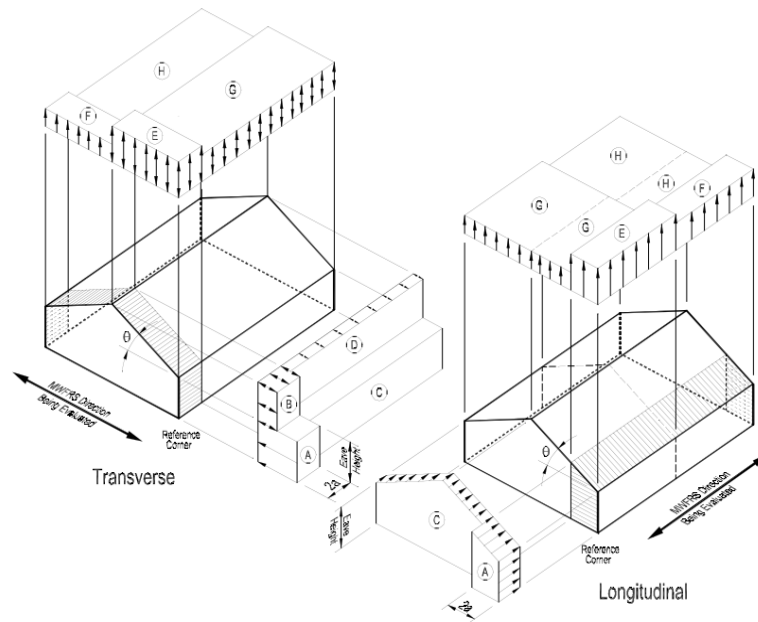
Figure 25: Wind Designation for Area 1

Horizontal Pressures (psf)				Vertical Pressures (psf)			
A	B	C	D	E	F	G	H
12.8	-6.7	8.5	-4.0	-15.4	-8.8	-10.7	-6.8
Adjusted Pressures (psf)				Adjusted Pressures (psf)			
14.7	-7.7	9.8	-4.6	17.71	-10.1	-12.3	-7.8

Table 2: Wind Pressures for Area 1

The seismic loads were calculated using Chapters 11 and 12 of ASCE 7-05. This seismic analysis includes dead loads from beams, slabs, columns and M/E/P equipment. A takedown of the building weight can be viewed in Appendix C. A summary of these forces can be viewed in Table 3, and a detailed breakdown of these calculations can be viewed in Appendix D of this report.

Floor	w_x (kip)	h_x	k	$w_x h_x^k$	$\sum w_i h_i^k$	F_x (kip)	Story Shear V_x (kip)	Moment (k-ft)
2	923	27	1.014	26105.63	37216.05	36.41	--	983.15
1	976	11	1.014	11110.42	37216.05	15.50	36.41	170.47
Total	1899	27	1.014	37216.05	37216.05	51.91	**51.91**	**1153.61**

Table 3: Seismic Loads for Area 1

Area 1 Lateral Analysis

To aid in the analysis of the lateral system, an ETABS model of Area 1 was created as shown in Figure 26. The seismic and wind loads were applied to the model and drift was compared for the four wind cases stated by ASCE 7-05, and seismic loads. It was determined that the seismic loads will control the design of the building in both directions. A summary of the drifts can be found in Table 4. The total building drift has been limited to H/400. It is assumed that spread footings will be adequate for this section of the building. The base supports were modelled as pinned supports to effectively account for the behavior of the spread footings.
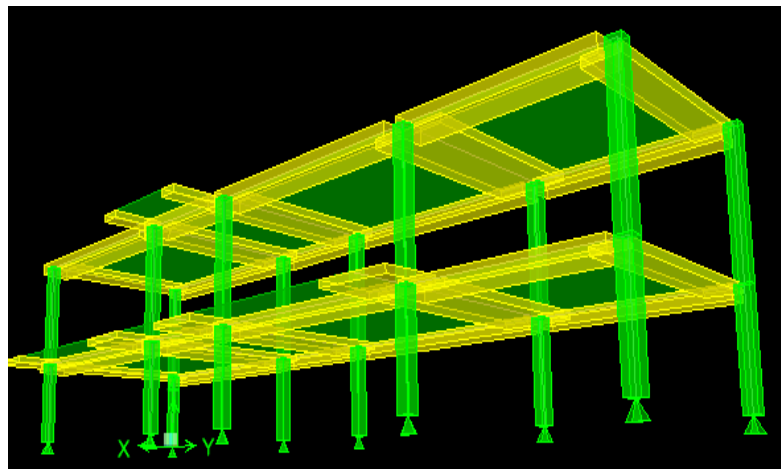
Figure 26: Area 1 ETABS Model

Story	Diaphragm	Load	UX (in)	UY (in)
STORY2	D2	XWIND	0.3308	0
STORY2	D2	YWIND	0.0002	0.091
STORY2	D2	XQUAKE	0.6515	-0.0001
STORY2	D2	YQUAKE	0	0.5129
STORY2	D2	CASE2X	0.2461	-0.0002
STORY2	D2	CASE2Y	0.0004	0.0683
STORY2	D2	CASE3	0.2482	0.0682
STORY2	D2	CASE4	0.2464	0.0681
STORY1	D1	XWIND	0.2121	0.0002
STORY1	D1	YWIND	0.0003	0.0617
STORY1	D1	XQUAKE	0.3827	0.0006
STORY1	D1	YQUAKE	0.0003	0.3133
STORY1	D1	CASE2X	0.1556	-0.0033
STORY1	D1	CASE2Y	0.0007	0.0467
STORY1	D1	CASE3	0.1593	0.0464
STORY1	D1	CASE4	0.1562	0.0434

Table 4: Drifts Due to Lateral Loads

Area 2 Lateral Loads

Method 2 located in Chapter 6 of ASCE 7-05 was used to determine the wind loads acting on Area 2. This method was chosen because the mean height of the building is greater than 60'. The resulting pressure diagrams can be viewed in Figures 27 and 28, and tabulated in Tables 6 and 7.

Classification	Category
V, Basic Wind Speed (Fig. 6-1)	90 mph
K_d (Table 6-4)	0.85
I (Table 6-1)	1.15
Occupancy Category (Table 1-1)	III
Exposure Category	B
K_{zt} (Topographic Factor)	1

Table 5: Wind Classifications

Shaun Kreidel
Structural Option
School Without Walls

April 12, 2010
Advisor: Dr. Hanagan
Final Report

11.67 psf

10.86 psf

9.92 psf

8.45 psf

7.29 psf
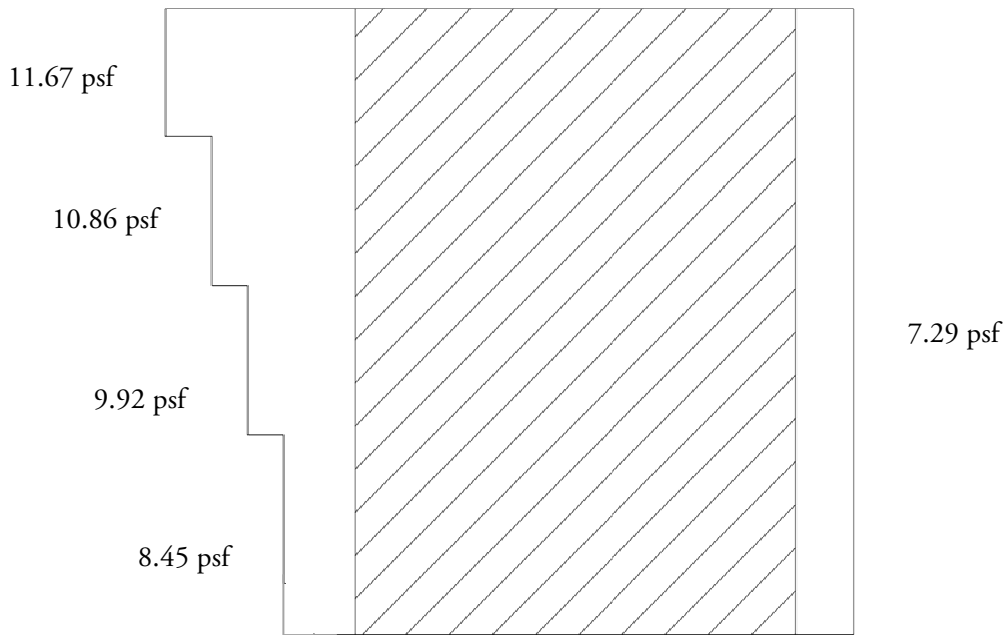
East-West Wind Pressure Diagram (Figure 27)

11.99 psf

11.16 psf

10.20 psf

8.68 psf

3.9 psf
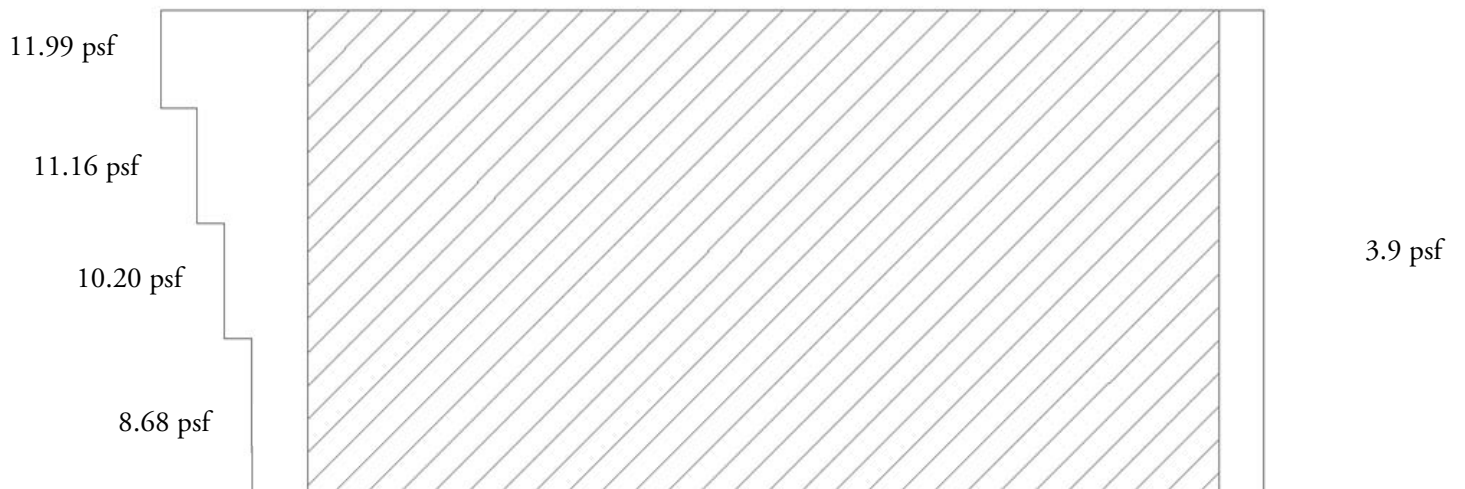
North-South Wind Pressure Diagram (Figure 28)

Wind Forces (North-South Direction)						
				Load (kip)	Shear (kip)	Moment (ft-kip)
Level	Trib Height (ft)	Total Load N-S (psf)	Tributary Width (ft)	N-S	N-S	N-S
Roof	6	15.90	46	4.39	0.00	279.15
4	13.6	15.90	46	9.95	4.39	506.80
3	15.2	15.06	46	10.53	14.34	375.92
2	17.6	14.10	46	11.42	24.87	233.44
Ground	10	12.58	46	5.79	36.28	0.00
					42.07	1395.31

Table 6: North-South Wind Forces

Wind Forces (East-West Direction)						
				Load (kip)	Shear (kip)	Moment (ft-kip)
Level	Trib Height (ft)	Total Load E-W (psf)	Tributary Width (ft)	E-W	E-W	E-W
Roof	6	18.96	129	14.68	0.00	933.48
4	13.6	18.96	129	33.26	14.68	1694.77
3	15.2	18.15	129	35.59	47.94	1270.51
2	17.6	17.21	129	39.07	83.53	799.05
Ground	10	15.74	129	20.30	122.60	0.00
					142.91	4697.82

Table 7: East-West Wind Forces

The seismic loads were calculated in a similar manner to those of Area 1 using Chapters 11 and 12 of ASCE 7-05. A takedown of the building weight can be viewed in Appendix C. A summary of these forces can be viewed in Table 8, and a detailed breakdown of these calculations can be viewed in Appendix D of this report.

Floor	w_x (kip)	h_x	k	$w_x h_x^k$	$\sum w_i h_i^k$	F_x (kip)	Story Shear V_x (kip)	Moment (k-ft)
ROOF	164	69.5	1.014	12067	177100	8.81		612.14
4	1139	57.5	1.014	69329	177100	50.60	8.81	2909.78
3	1204	42.25	1.014	53635	177100	39.15	59.41	1654.06
2	1204	27	1.014	34060	177100	24.86	98.56	671.25
1	1113	7	1.014	8010	177100	5.85	123.42	40.93
Total	4824	69.5	1.014	177100	177100	129.27	129.27	5888.15

Table 8: Seismic Loads for Area 2

Area 1 Lateral Analysis

An ETABS model of Area 2, as seen in Figure 29, was created to aid in the lateral system analysis. The seismic and wind loads were applied to the model and drift was compared for the four wind cases stated by ASCE 7-05, and seismic loads. It was determined that the seismic loads will control the design in the North-South direction, however, wind will control in the East-West direction. This was expected because of the large surface area of the East-West side of the building and the relatively small surface areas of the transverse elevations. A summary of the drifts can be found in Table 9. Because drift is a serviceability check, load factors were not applied to the loads. The total building drift has been limited to H/400. It is assumed that because of the increase in weight of the building, the foundation system will need to change to a mat slab or pile system. Because of this, the base restriants were modelled as fixed connections.
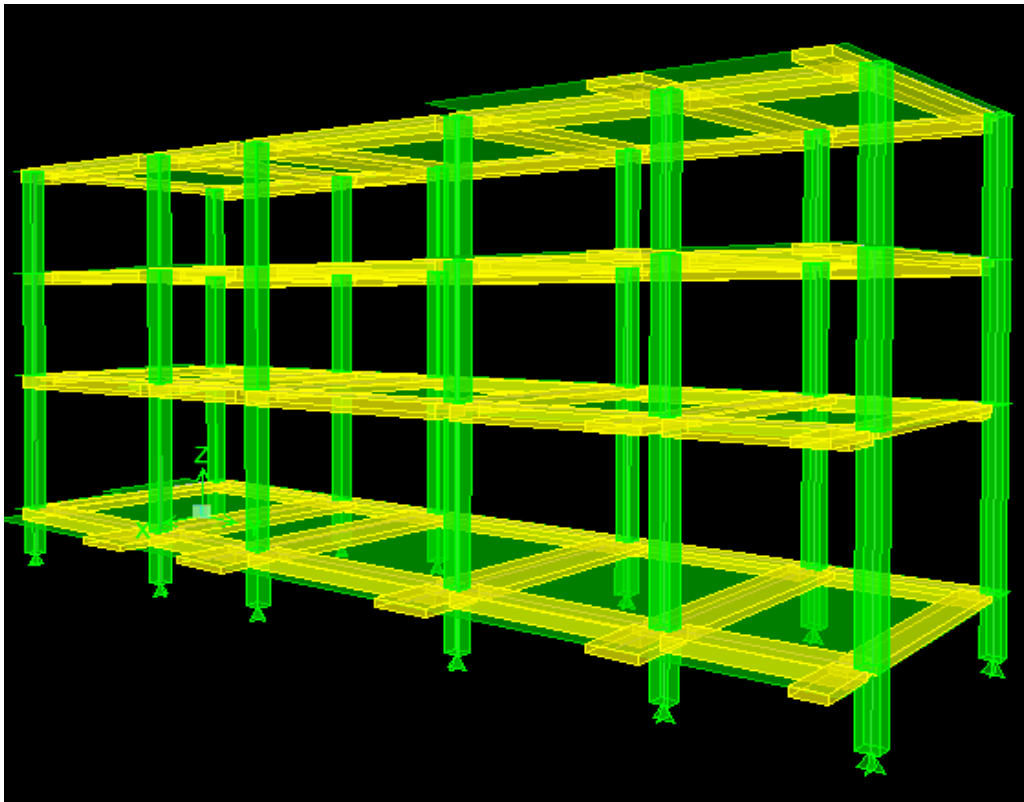

Figure 29: Area 2 ETABS Model

Story	Diaphragm	Load	UX (in)	UY (in)
STORY4	D4	XWIND	1.1218	0.0007
STORY4	D4	YWIND	-0.0012	0.1851
STORY4	D4	XQUAKE	1.331	0.0016
STORY4	D4	YQUAKE	0.0013	0.7267
STORY4	D4	CASE3	0.8405	0.1394
STORY4	D4	CASE2X	0.8772	0.0033
STORY4	D4	CASE2Y	-0.0047	0.1386
STORY4	D4	CASE4	0.8725	0.1418
STORY3	D3	XWIND	0.9105	0.0003
STORY3	D3	YWIND	-0.0008	0.1591
STORY3	D3	XQUAKE	1.0466	0.0003
STORY3	D3	YQUAKE	0.0007	0.6051
STORY3	D3	CASE3	0.6823	0.1196
STORY3	D3	CASE2X	0.705	-0.0002
STORY3	D3	CASE2Y	-0.0029	0.1194
STORY3	D3	CASE4	0.7021	0.1192
STORY2	D2	XWIND	0.5796	0.0004
STORY2	D2	YWIND	-0.0003	0.1097
STORY2	D2	XQUAKE	0.6395	0.0002
STORY2	D2	YQUAKE	0.0003	0.4
STORY2	D2	CASE3	0.4345	0.0826
STORY2	D2	CASE2X	0.4444	-0.0008
STORY2	D2	CASE2Y	-0.0013	0.0824
STORY2	D2	CASE4	0.4431	0.0816
ENTRANCE	D1	XWIND	0.0622	0.0001
ENTRANCE	D1	YWIND	0	0.0116
ENTRANCE	D1	XQUAKE	0.0652	0
ENTRANCE	D1	YQUAKE	0	0.0401
ENTRANCE	D1	CASE3	0.0466	0.0088
ENTRANCE	D1	CASE2X	0.0467	-0.0006
ENTRANCE	D1	CASE2Y	0	0.0088
ENTRANCE	D1	CASE4	0.0467	0.0082

Table 9: Drifts Due to Lateral Loads

COLUMN DESIGN

As seen from the lateral system analysis, the columns were designed to handle the demands of both the gravity system, and to transfer lateral loads.   PCA Column was used as an aid to verify column sizes and rebar requirements. A excel spread sheet was created to determine the axial loads imposed on the each column. The moments exerted on the columns were determined from the ETABS model created. The following load combinations were applied to determine the controlling load factors:

$$1.4D$$
$$1.2D + 1.6L$$
$$\mathbf{1.2D + 1.6W + 0.5L}$$
$$1.2D \pm 1.0E + 0.5L$$
$$0.9D \pm (1.6W \text{ or } 1.0E)$$

D = dead load
L = live load
W = wind load
E = earthquake load

For this report, Column B-2 as designated in Figure 18 was spot checked.   A detailed breakdown of the load combinations can be found in Appendix E. It was found that (8) #10 bars will be adequate to carry the loads imposed on the column. An interaction diagram can be viewed in Figure 30.
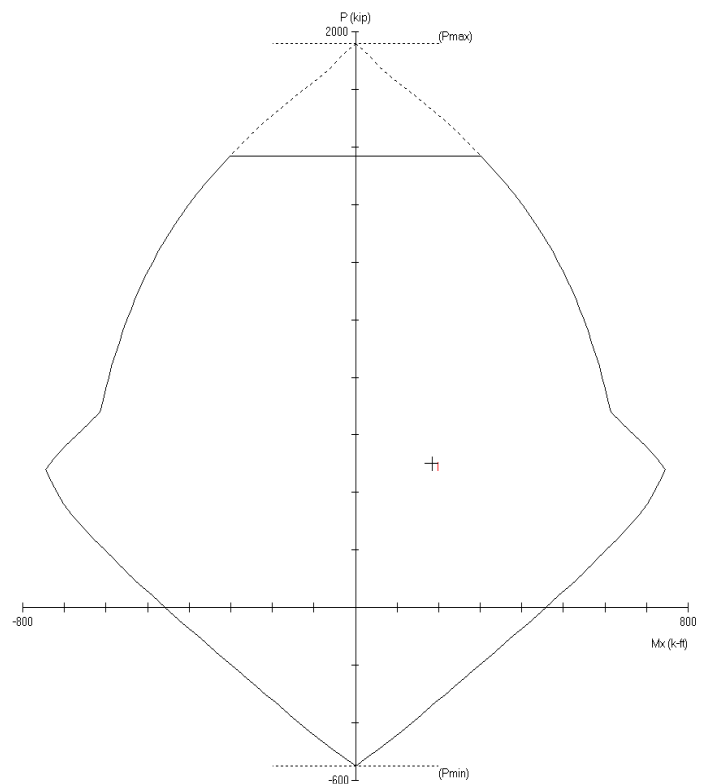

Figure 30: Interaction Diagram

From the interaction diagram, it is clear to see that this column does not utilize all of its strength capacity. A more efficient column based on strength requirements was investigated, however using a smaller column results in an unacceptable drift.

DEPTH STUDY CONCLUSION

The goal of reducing the total depth of the floor system was successfully reached by changing the gravity system from a composite steel deck and steel beam system to a concrete system that utilizes the trade of post tensioning. ADAPT PT was used as an aid in design of both the beams and one ways slabs. The deepest beam required in the redesign measured 20". Spanning the beams in the suggested positions allowed for maximum clearance space. The maximum slab depth measured 12" which was necessary to support the loads over the simple spanned areas.

Redesign of the lateral system was required to complement the gravity system. The original braced frames and shear walls were removed from the lateral system. All lateral loads are resisted by concrete moment frames. An ETABS model of the building was created in order to determine the drift and moments the applied lateral loads create.

Total drift of the building was limited to H/400. From the column analysis, it is clear to see that the columns chosen were not the most efficient section from a strength standpoint, however were necessary to meet the serviceability requirement of drift.

 Because the weight of the building increased upon changing structural systems, the foundation system must be altered. Strip footings should be adequate in the case of Area 1; however, the foundation type supporting Area 2 will most likely change. Because of the existing historic building using piles may not be the best alternative due to the vibration they create on impact. A mat foundation therefore could be a viable option for this foundation system.

CONSTRUCTION MANAGEMENT BREADTH

The goal of the construction management breath was to study the cost and schedule impact when changing the current system to a post tensioned concrete system. To accurately compare the cost and schedule of the two different systems, an analysis for both the concrete and steel buildings was conducted using MC^2 as an aid.

A post tensioned concrete system is a very viable alternative when it comes to geography. Because of the height restrictions of buildings in Washington D.C., post tensioned construction is a common practice in the area.

Four separate cost analysis were conducted in order to effectively make comparisons for both Area 1 and Area 2. A detailed takedown and assumptions for each estimate can be found in Appendices F-I. Because the structural system was the only system under investigation and comparison, the beams, columns floor slabs, and lateral resisting components were the only items inserted into the program. It must be noted that the foundation was not incorporated into the estimate. This will most likely add cost to the concrete estimate, because typically, concrete buildings require large foundations.

From the analysis of the proposed post-tensioned concrete system, it was found that Area 1 would cost approximately $150,000 and Area 2 approximately $350,000. This was compared to the existing steel system which cost approximately $100,000 for Area 1 and $400,000 for Area 2.

From the comparison of the total costs, it was found that both systems cost roughly $500,000. From the detailed cost breakdown of the different elements, it is clear to see that even though the proposed concrete system requires formwork of the columns beams and slab, the eight shear walls, additional fireproofing and steel moment connections are eliminated, which effectively offsets the costs.

When comparing the durations of the tasks, a linear progression using one crew was assumed. It was determined that the concrete system took 40% longer to construct than the existing steel system. Even though the time of erection is significantly longer, the lead time is typically much shorter for concrete construction.

ARCHITECTURAL BREADTH

The existing historic building utilizes 2x6 wood joists to transfer gravity loads to load bearing walls. From Figure 31 it is apparent that the direction in which the joists run varies throughout the building.

For the historic restoration of School Without Walls, it was detailed by the architect that the gypsum wall board ceiling was to be directly attached to the wood joists as seen in Figure 32.
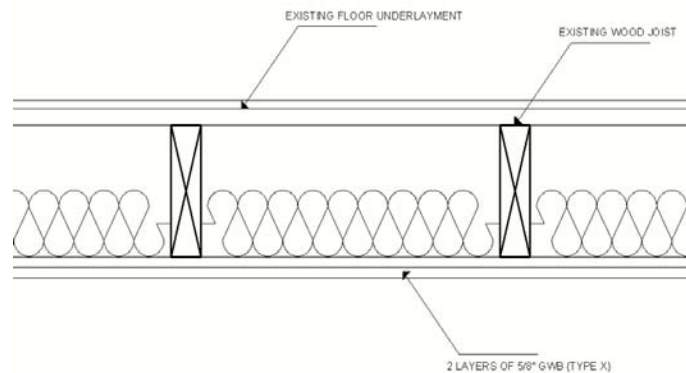

Figure 31: Existing Wood Joist Layout

Figure 32: Existing Ceiling Detail

Due to the upgraded mechanical systems as part of the modernization process, ceiling clearance is very important. The construction team was challenged with the task of containing the electrical and mechanical systems in the space between the 2x6 joists. In addition to the limited space between joists, careful coordination had to occur when joists running in the transverse direction were encountered. The objective of this breadth is to show that the architecture of each classroom is not compromised by dropping the ceiling to allow for more clearance.

The dropped ceiling requires crossties and hangers to be used in order to attach the ceiling to the joists. A detailed section of the proposed ceiling system can be viewed in Figures 33 and 34. The architectural impact was studied if the ceiling is dropped 8".
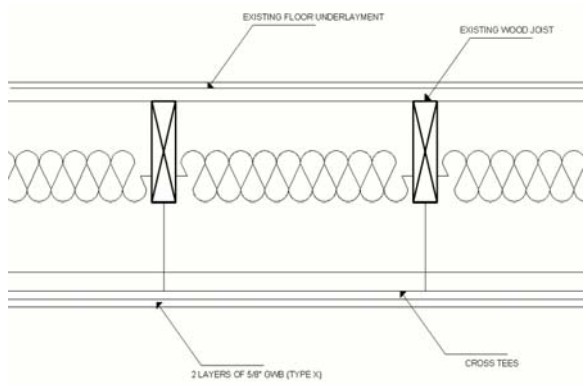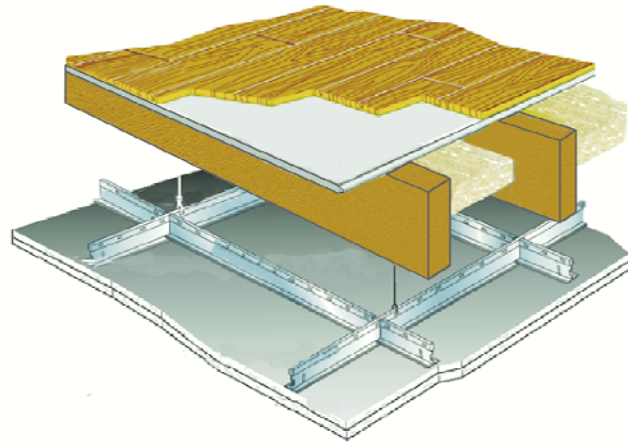

Figure 33: Dropped Ceiling Section

Figure 34: Isometric Ceiling Section

In order to effectively the aesthetic changes, a typical classroom was modeled in REVIT. Renderings were created in different areas of the room showing both the existing ceiling, and proposed dropped ceiling. These renderings can be viewed in Figures 35 through 40.

Figure 35: Existing Classroom Render

Figure 36: Dropped Ceiling Render

Figure 37: Existing Classroom Render

Figure 38: Dropped Ceiling Render

Figure 39: Existing Classroom Render

Figure 40: Dropped Ceiling Render

From the architectural investigation, it appears that the drop ceiling does not interfere with any major architectural elements. Using a dropped ceiling, therefore is a very viable option in this instance. Using this type of construction can eliminate extra costs involving MEP coordination without majorly effecting the architectural appeal of the room.

CONCLUSION

The goal of reducing the total depth of the floor system was successfully reached by changing the gravity system from a composite steel deck and steel beam system to a concrete system that utilizes the trade of post tensioning. ADAPT PT was used as an aid in design of both the beams and one ways slabs. The deepest beam required in the redesign measured 20". Spanning the beams in the suggested positions allowed for maximum clearance space. The maximum slab depth measured 12" which was necessary to support the loads over the simple spanned areas.

The lateral system required redesign to complement the gravity system. The original braced frames and shear walls were removed from the lateral system. All lateral loads are resisted by concrete moment frames. An ETABS model of the building was created in order to determine the drift and moments the applied lateral loads create.

Total drift of the building was limited to H/400. From the column analysis, it is clear to see that the columns chosen were not the most efficient section from a strength standpoint, however were necessary to meet the serviceability requirements.

The change in systems required an analysis of the cost of the project. A cost breakdown was performed for both the steel and concrete systems using MC^2 as an aid. It was found that when comparing the structure cost it was found that the change in systems did not dramatically change the price. The increased labor costs of the concrete building due to formwork were offset by the elimination of shear walls, steel moment connections and required additional fireproofing.

 Because of the increased weight of the building when changing structural systems, the foundation system must be altered. Strip footings should be adequate in the case of Area 1; however, the foundation supporting Area 2 will most likely change. Because of the existing historic building using piles may not be the best alternative due to the vibration they create on impact. A mat foundation therefore could be a viable option for this foundation system.

A benefit of reducing the floor system depth of Area 1 and Area 2 is that it allows an easier coordination of the trades between the ceiling, and floor above. Likewise in the existing historic building, more clearance space allows for easier trade coordination.

The architectural breadth for the school without walls studies the aesthetic impact dropping the ceiling 8". After creating a REVIT model and exporting renderings of the space, it was determined that dropping the ceiling was in fact a viable option from an architectural standpoint.

REFERENCES

1. American Concrete Institute (2008) *Building Code Requirements for Structural Concrete (ACI 318-08)*, ACI, Farmington Hills, MI

2. American Institute of Steel Construction (2005) *Steel Construction Manual,* Thirteenth Edition, AISC, Chicago, IL

3. American Society of Civil Engineers (2005) *ASCE 7-05:Minimum Design Loads for Building and Other Structures,* ASCE, Reston, VA

4. Allred, Brian. *Common Post-Tensioning and Construction Issues.* Structure Magazine, July 2005

5. Kang, T. H.-K., and J. W. Wallace, 2005. Dynamic Responses of Flat Plate Systems with Shear Reinforcement, *ACI Structural Journal*, 102 (5), 763-773.