Final Report

Student Health Center

Penn State University

Prepared By: Jacob Brambley (Structural Option)
Prepared For: Dr. Richard Behr

4/7/10

Student Health Center @ Penn State

Project Team

Owner:
Pennsylvania State University
Architect:
RMJM Hillier
Structural Engineer:
Greenman - Pedersen, Inc.
Civil Engineer:
Gannett Fleming
MEP Engineer:
BR&A/Bard, Rao, and Athanas
CM Firm:
Whiting - Turner

Building Statistics

5 Stories
64,000 SF
Completed Fall 2008
$26 million LEED Certified Building

Architecture:

Face brick accented with cast stone masonry bands
Glass curtain wall accented with metal panels
- maximizes natural light
Green Roof
- reduces stormwater runoff
- reduces heating/cooling costs
Screenwall around rooftop mechanical equipment

Structural System:

Structural steel frame
Concrete slab on composite steel deck floor system
Partially-restrained moment frame to resist lateral loads
Minipile foundation at 45 ft depth

MEP System:

(2) Air Handling Units on Roof
Airflow regulated by VAV and CAV boxes
Uses 277/480V, 3 phase, 4 wire system mainly for lighting
Utilizes120/208V, 3 phase, 4 wire system for receptacles, etc.

PENNSTATE

Jacob Brambley
Structural Option
http://www.engr.psu.edu/ae/thesis/portfolios/2010/jkb207/

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Executive Summary

The Student Health Center (SHC) is a five story building on the Penn State campus that serves as a health care services and hospital facility. After completion in the fall of 2008, this building now houses University Health Services and Counseling and Psychological Services, two departments of Penn State's Division of Student Affairs.

The facility is 77 feet in height from the first level and is approximately 64,000 SF in area. It has a brick façade rising from the ground with large curtain wall on the south side the building. The structure is held up primarily by a steel frame. The overall structure sits on a mini-pile foundation through use of pile caps, piers, and grade beams. Composite steel with concrete slab on deck is use for the floor system throughout the SHC.

In this final report, the current building statistics is to be discussed, as well as, the proposed redesign. A comparison of the two structures will then be stated.

The redesign changed the building from a primarily steel structure to a concrete supported structure. This was done for one main reason; to reduce floor thickness and research the plausibility of adding another floor to the structure. A post-tensioned floor was designed, as it would allow for the thinnest floor, and the thickness was determined to be 8". Setting the story heights at 11 feet, this floor system allows for the mechanical equipment to fit as per original design. Ceiling heights currently employed in the SHC were kept intact despite the structure change. Because of this another story could be added without changing the original building's overall height.

Gravity columns were designed at 18"x18"; with (12) #11 rebar and were adequate to carry the load. Foundations were also checked for gravity loading. It was found that (4) piles had to be added to resist the heavy concrete structure's loads. Shear walls were designed to replace steel moment frames to resist lateral loads and minimize lateral drift. These were designed with a width of 18".

A CM study was then done, calculating the plausibility of implementing the design. A cost analysis yielded $899,153 construction cost for the concrete structure and $1,358,422 for the steel structure. A schedule estimate yielded 234 days of construction for the concrete superstructure and 177 days for the steel one.

In addition to the structural redesign, a study of shading systems was completed. Two systems were implemented, solar fins and light shelves. The light shelves were then analyzed to determine the effectiveness of light in exterior rooms. This was then converted to show a lighting system savings of $150 per year.

Introduction

The Student Health Center gives off a light and inviting atmosphere through use of a large curtain wall. This curtain wall works to let natural light into the building, as well as, expose the inner structure from the outside. This report is meant to examine how a new structure constructed of concrete instead of steel would perform. Floors, columns, and the lateral system will be redesigned to implement an extra floor to the structure. In addition, a construction management study will be performed to try to access which system is better.

Solar shading devices will also be examined to replace the current indoor shade system. Solar fins and light shelves are to be looked at as alternatives. The light savings due to the light shelves will also be determined and a comparison will show the best design.

Current Structural Systems

Foundation:

The foundation of the SHC is composed of grade beams and piers that are supported by mini-piles with pile caps. The mini-piles are arranged in configurations of 1-5 piles per pile cap. They are to be at a depth of 45 feet and have an 80 ton allowable capacity. The partially-restrained moment frame employed in this building is either connected directly to a pile cap or to a concrete pier. The depth of these mini-piles will counteract the moment of the partially-restrained moment frame caused by lateral loads. Locations of the piles are shown in *Fig. 1*.

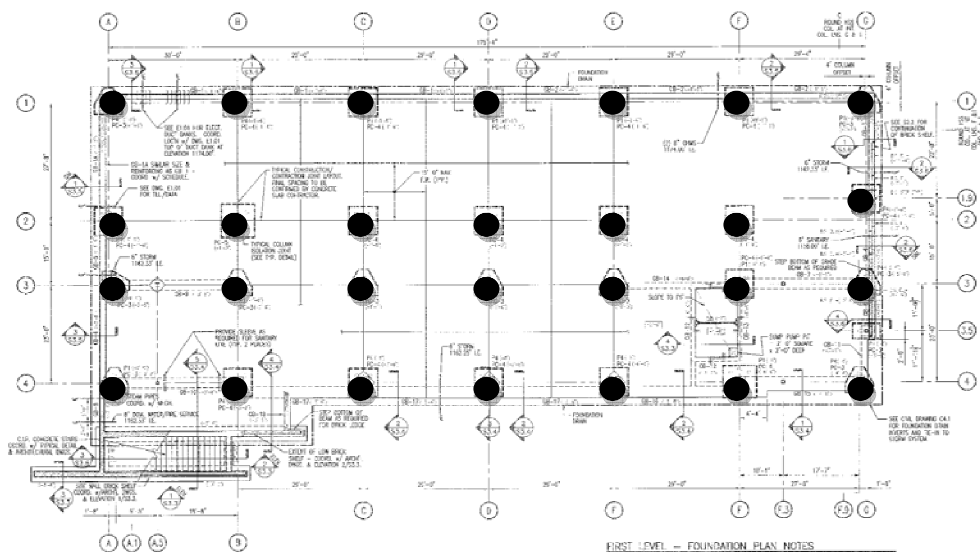


Fig. 1 – Pile Locations

Floor System / Beams:

The floor system used in the SHC is composed of 3 1/4” lightweight concrete fill on 2”-20 gage galvanized composite floor deck LOK floor for a total slab thickness of 5 1/4". Also included are 3/4ϕ x 4” long shear studs equally spaced along the entire lengths of all interior beams and girders that are not part of the partially-restrained moment frame. The shear studs are not on the moment frame because the beams on the frame cannot be too rigid so that they can deform. This composite floor deck is supported by steel W-shape beams spanning between steel columns.

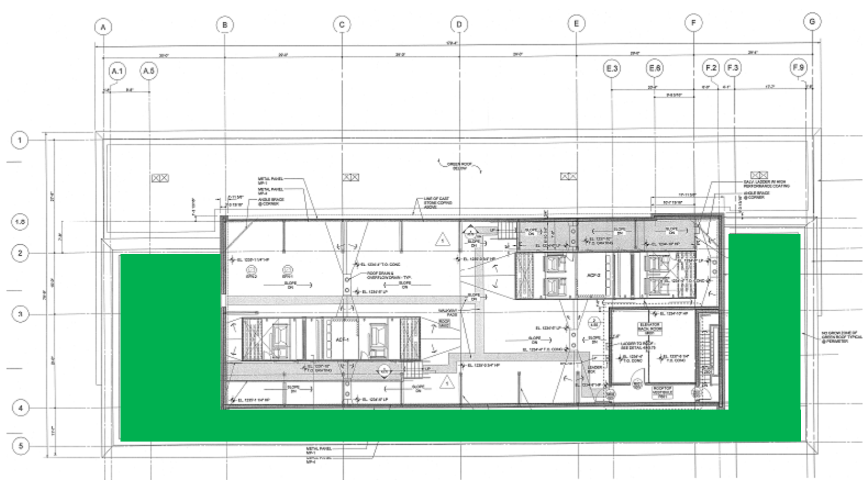
Columns:

The P.R. moment frame consists of W14 steel columns running from the foundation up to the roof level. Columns that are not part of the P.R. moment frame range in size and shape. Round HSS shapes are used both with and without concrete fill, as well as square HSS shapes and W shapes to resist gravity loads.

Roof / Penthouse Level:

The roof system is composed of 5 1/4” normal weight concrete fill on 3”-20 gage galvanized composite floor deck LOK floor for a total slab thickness of 8 1/4". The main roof is at the 6th level with a screen wall around the rooftop mechanical equipment. There is also a green roof around the perimeter of the main roof level *(Fig. 2)*. On the north end of the building, at the 5th level, there is another green roof *(Fig. 3)* that is nearly 20 feet wide and runs the length of the building.

Fig. 2 – Green Roof on Main Roof



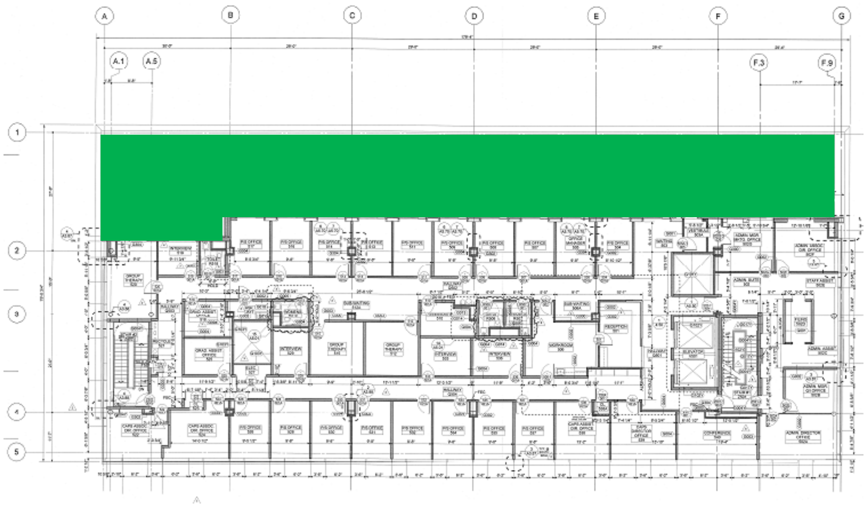


Fig 3 – Green Roof on 5th Floor

Lateral System:

A partially-restrained moment frame is used to resist lateral loads on the SHC. These frames are to have Flexible Moment Connections (FMC) designed by the steel fabricator per Part 11 of the AISC- Load & Resistance Factor Design Manual. A typical beam to column flange connection for these frames is detailed below *(Fig. 4)*. There are eight partially-restrained frames employed in this building, with seven running in the north/south direction, and one in the east/west direction *(Fig. 5)*. These frames run vertically up to the 5th Level or Main Roof Level of the building depending on the location. Frames are shown below in elevation (Fig. 6-8).

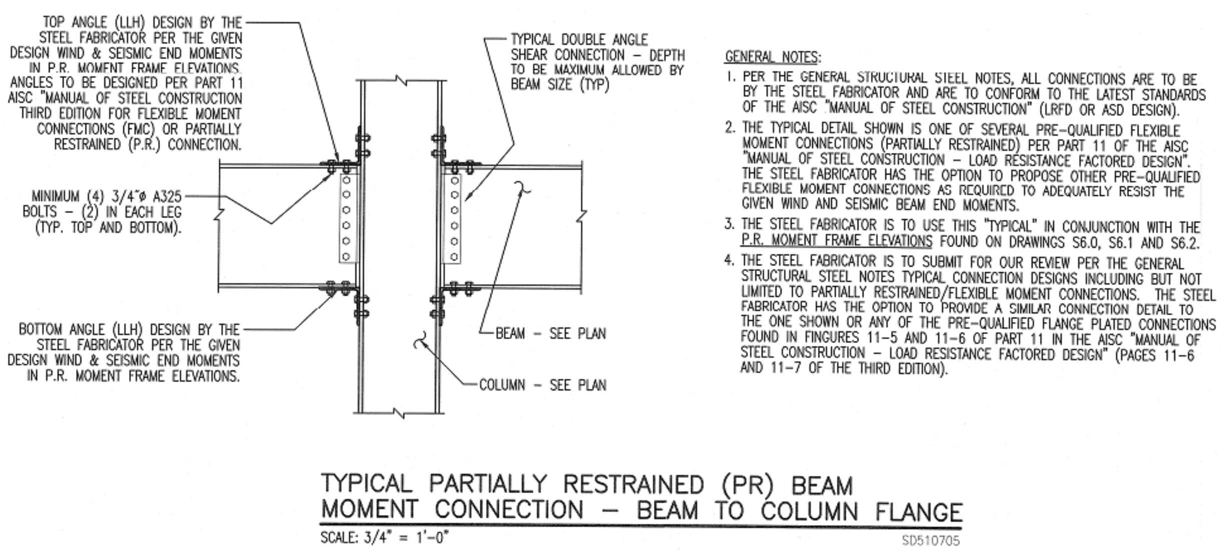


Fig. 4

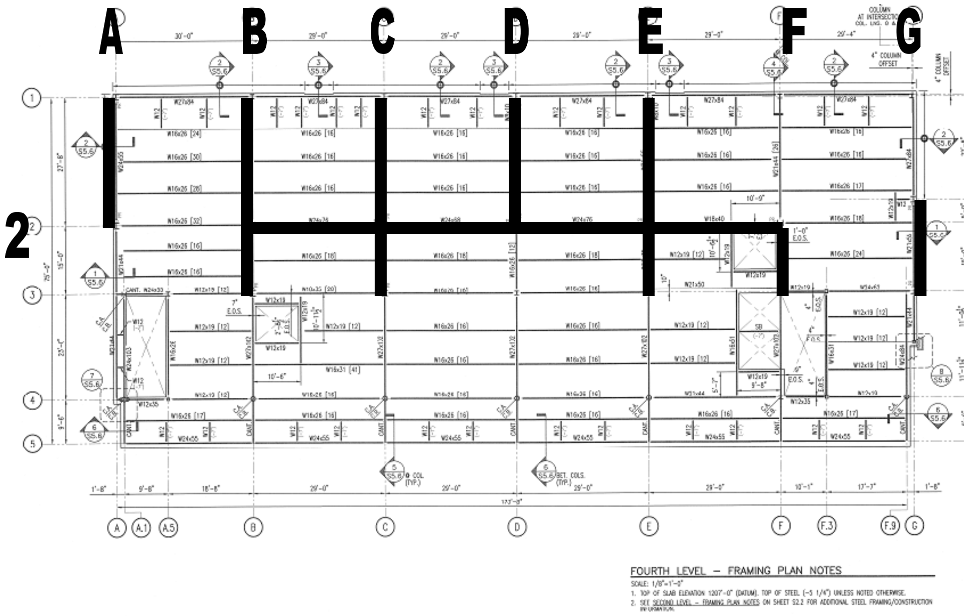


Fig. 5 – Partially-restrained Frame Locations

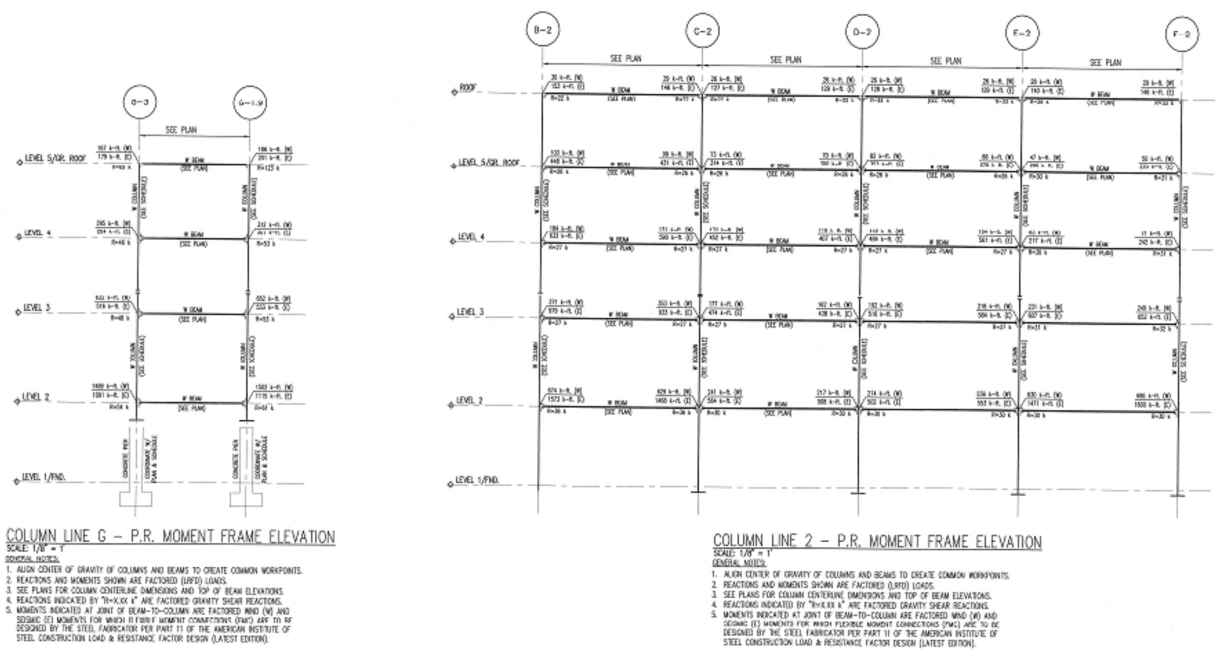


Fig. 6 – P.R. Moment Frame Elevations (G and 2)

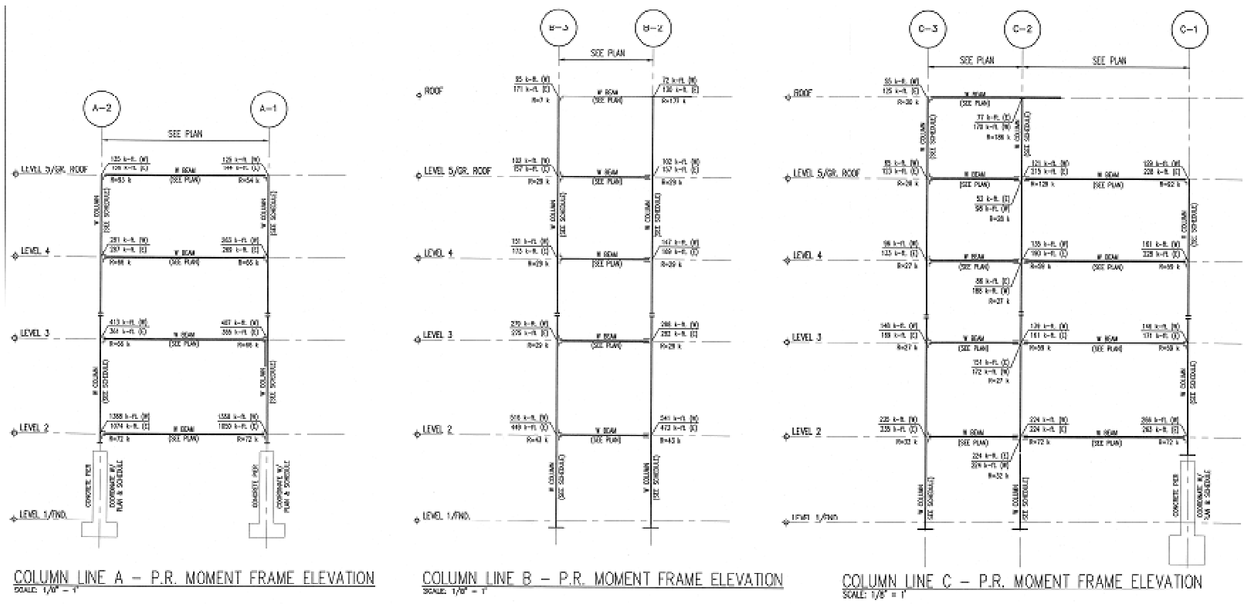


Fig. 7 – P.R. Moment Frame Elevations (A, B, and C)

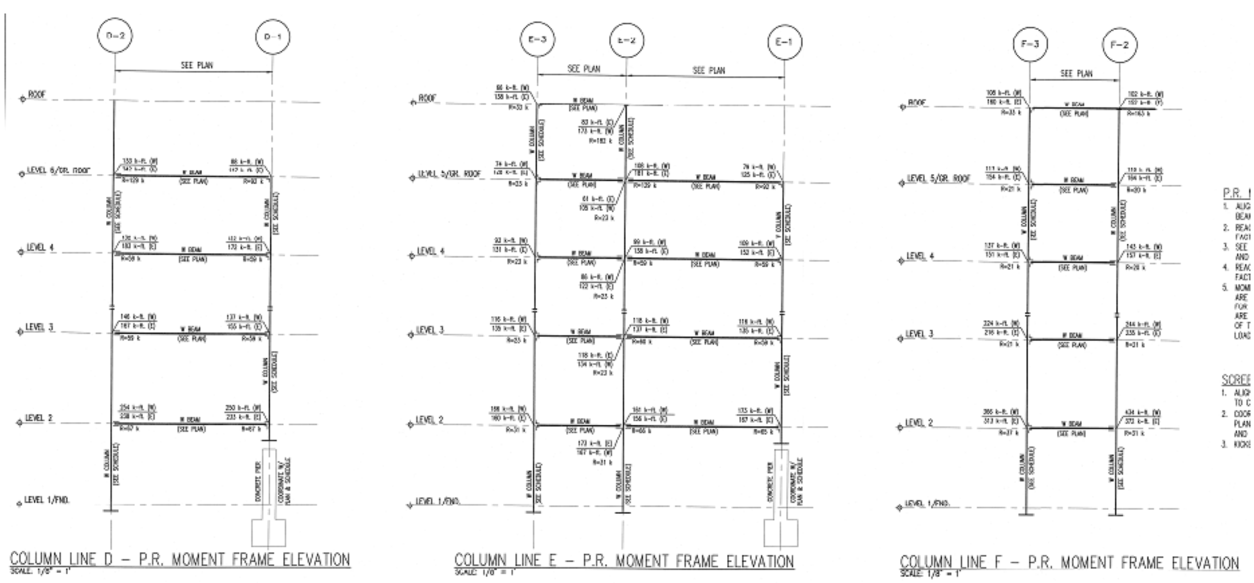


Fig. 8 – P.R. Moment Frame Elevations (D, E, and F)

Code and Design Requirements

Design Codes and References:

Codes used by Project Team:

International Building Code (IBC)/2003 with Borough Amendments
International Mechanical Code (IMC)/2003 with Borough Amendments
International Plumbing Code (IPC)/2003 with Borough Amendments
International Energy Conservation Code (IECC)/2003 with Borough Amendments
International Code Council Electrical Code (ICCEC)/2003
International Fire Code (IFC)/2003
ACI 318-05
AISC "Steel Construction Manual" (13th Edition)
ACI 530.1/ASCE 6/TMS 602 (2005)

Codes used for Thesis:

International Building Code (IBC)/2006
ACI 318-08
AISC "Steel Construction Manual" (13^{th} Edition)
ASCE 7-05

Deflection Criteria:

Maximum Floor Deflections:
- L/360 Live load
- L/240 Total load
- L/240 Roof

Maximum Lateral Deflections:
- L/400 - Drift due to wind
- $0.020h_{sx}$ - Drift due to seismic

Load Combinations:

1.4 (Dead)
1.2 (Dead) + 1.6 (Live) + 0.5 (Roof Live)
1.2 (Dead) + 1.6 (Roof Live) + 1.0 (Live or 0.8 Wind)
1.2 (Dead) + 1.6 (Wind) + 1.0 (Live) + 0.5 (Roof Live)
1.2 (Dead) + 1.0 (Seismic) + 1.0 (Live)
0.9 (Dead) + 1.6 (Wind)
0.9 (Dead) + 1.0 (Seismic)

Material Properties

Material	A.S.T.M.	Minimum Strength
Concrete		
Foundation Walls, Pile Caps, Slab on Grade, Retaining Walls, Footings	-	3000 PSI
Exterior Slabs, Curbs	-	4000 PSI
Reinforcement	A615 (Grade 60)	60 KSI
WWF	A185, A497	70 KSI
Structural Tubing, Round	A500 (Grade B)	42 KSI
Structural Tubing, Shaped	A500 (Grade B)	46 KSI
Steel Pipe	A53 (Type E, Grade B)	35 KSI
Rolled Shapes	A992	50 KSI
Other Rolled Plates	A36	36 KSI
Connection Bolts	A325	92 KSI
Anchor Bolts	A307	-
Threaded Rods	A36	36 KSI
Non-shrink Grout	C1107	8000 PSI
CMU	C90 (lightweight)	2800 PSI

Proposal Information

Problem Statement:

The existing steel frame and composite steel floor system described earlier was constructed with no major problems and has many benefits including having a smaller effect on foundations compared to a heavy concrete frame. One downside to this steel system though, is the 29-inch thickness of a typical floor which in turn reduces the attainable number of stories due to height restrictions. The main reason for the construction of the Student Health Center was to create a new, larger space to house services provided in the then overcrowded Ritenour Building. If a thinner floor system was implemented, then perhaps another story could be added, increasing floor area to the maximum.

Proposed Solution:

A post-tensioned floor system will be studied in detail as a means to decrease floor thickness throughout the SHC. Upon completion of Technical Report 2, it was determined that post-tensioning would provide the smallest floor thickness compared to other systems studied. A new concrete structure will be designed along with this floor system and shear walls will be added to replace the moment frames for resisting lateral loads. Calculations pertaining to the increased self-weight of the building on the foundations will be done to further see the effects of the new superstructure. The possibility of an additional floor will be examined and pros and cons of each system will be quantified to determine plausibility of the new design.

Solution Method:

For the design of the post-tensioned slab, calculations will be completed using ADAPT-Builder and ADAPT-PT and the Equivalent Frame Method (ACI 318-08). After a floor design is finalized, column sizes will be determined using the program PCA Column and checked with hand calculations referencing ACI 318-08. A 3D model using ETABS will be created implementing this data, as well as, loads given through ASCE 7-05 and IBC 2006, to check that several hand calculations are accurate. Also, through use of this model, loads on columns and foundations will be revealed. Validity of the current mini-pile foundation to resist the added dead load due to the concrete structure will be examined using these loads and a redesign will be completed if necessary. Shear walls will need to be implemented in the current layout of the building to increase effectiveness in resisting lateral loads. ETABS will also be used for designing shear walls to ensure the maximum drift does not exceed ASCE 7-05 maximum drift parameters.

Breadth Topics:

The overhaul of the current structural system does not only impact material sizes, weights, and orientation but also impacts cost and schedule effects. Construction management issues such as material lead time and system constructability will be examined as a breadth topic. Comparisons of direct and indirect costs and construction time will be summarized upon completion of calculations.

Another topic separate from the structural system that will be covered is solar shading. Difficulties with the current fabric shades wrinkling and rolling up unevenly due to their size continue. Therefore, a study of alternate systems such as light shelves and overhangs will be performed and comparisons will be drawn between these and the current shading system.

Structural Depth

Floor System:

Implementing a post-tensioned floor system throughout the building will create the biggest reduction in floor thickness. Design of the PT slab was done using ADAPT-Builder and ADAPT-PT and some minor hand calculations. To figure out a thickness to check from the start, the rule of thumb L/h = 45 was used. This resulted in an initial check of an 8 inch slab. An 8 inch slab was then modeled in ADAPT in conjunction with shear walls and columns as shown in Fig. 9.

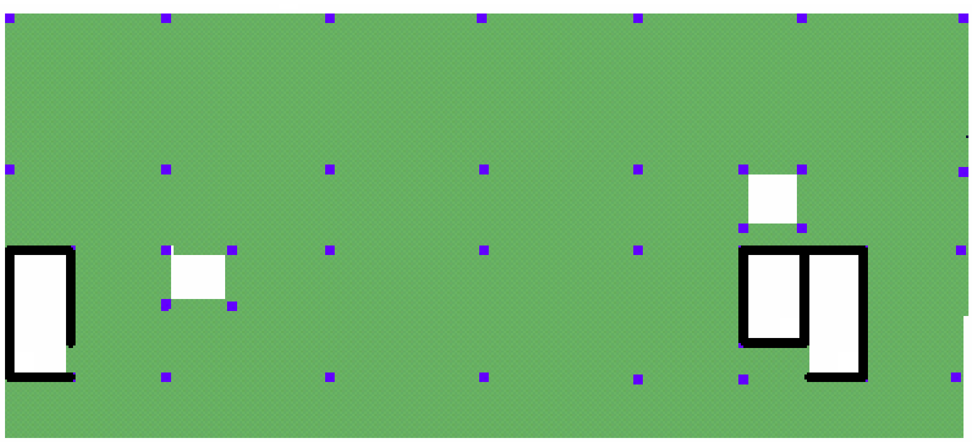


Figure 9 – Slab, shear wall, and columns in plan

Some areas (shown in red in Fig. 10) were cause for concern. These trouble areas are due to the openings being next to the cantilever slab. There must support for the PT tendons to resist loads off of the cantilever.

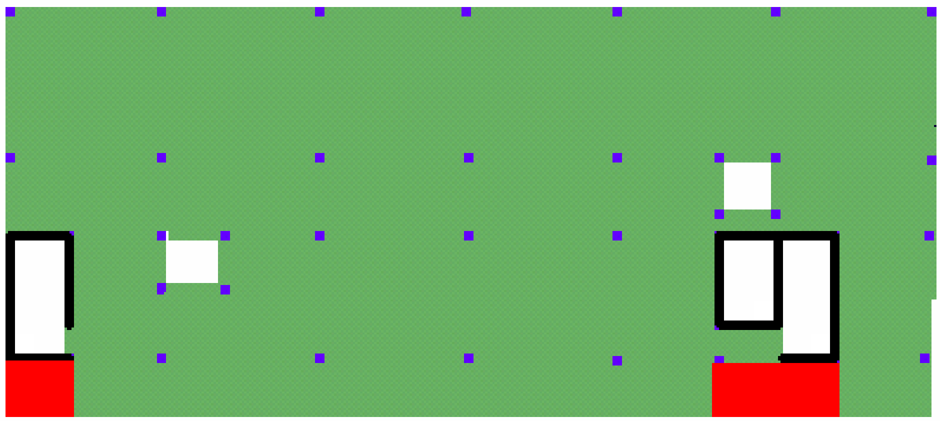


Fig. 10 – Trouble areas for PT slab

These problems were examined and solutions were roughly calculated. More detailed structural analysis would need to be done to verify this design, though. Possible solutions to tendon layout problems in these areas are shown in Figs. 11 and 12. Research into the plausibility of placing PT tendons through shear walls will need to be done.

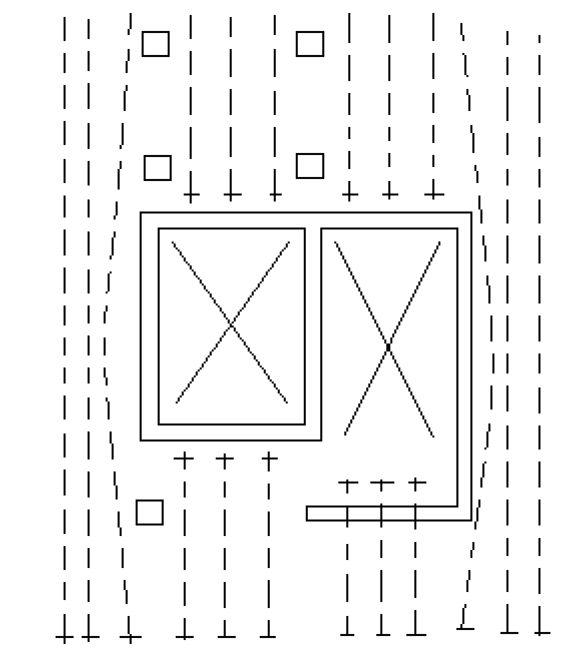


Fig. 11 – Possible tendon layout in Trouble area 1

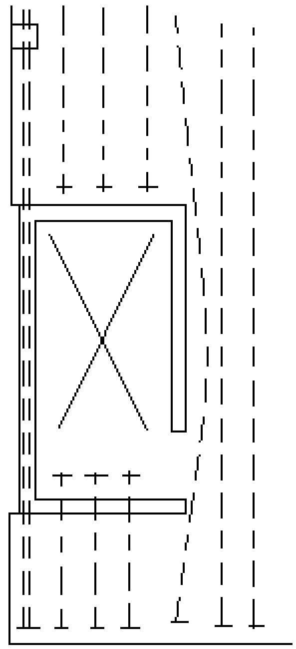


Fig. 12 – Possible tendon layout in Trouble Area 2

All other areas of the slab worked well with the post-tensioning tendons. Design strips were made in ADAPT-Builder in both the x and y directions. Optimum tendon profiles for each design strip were then calculated using ADAPT-PT. A typical design strip in the y-direction contains 14 tendons with a force of 12.0 kips per tendon. The typical design strip in the x-direction contains 11 tendons with a post-tension force of 13.3 kips per tendon. Tendon profiles of each are shown here in Fig. 13.

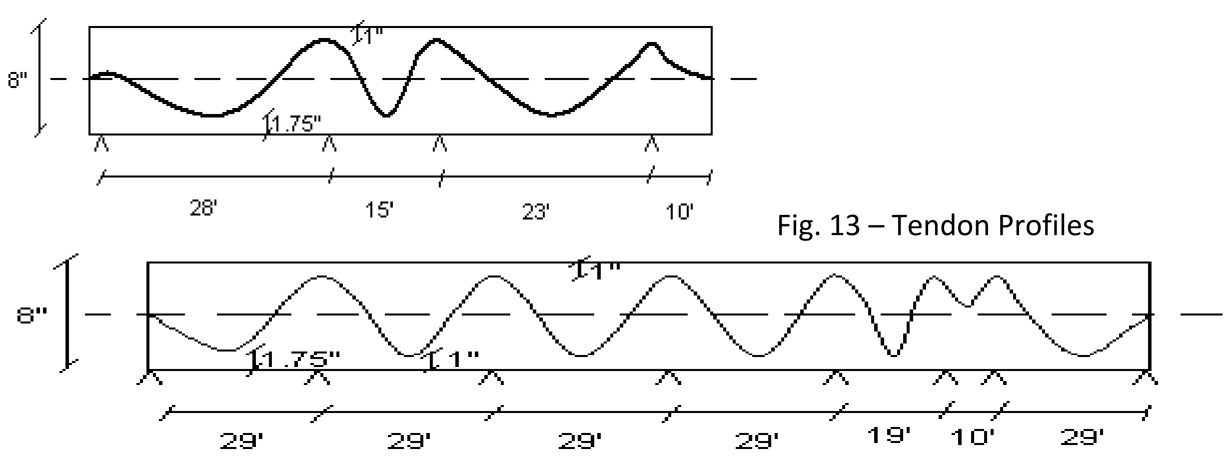


Fig. 13 – Tendon Profiles

Steel reinforcement is also needed in positive and negative moment regions of the slab. Information on the amount and location of reinforcement within each design strip is shown in a design table in Appendix A.

For the last step in the design of the PT floor, deflections were tabulated in ADAPT and checked in relation to code limitations. ACI 318-08 gives allowable deflections for slabs due to live load and total load. The newly designed slab functioned well within code limits. Values for the critical typical long spans are shown in the following table:

Deflection Checks:

	Actual Live Load Deflection	Allowable Live Load Deflection
Longest Span in X-direction	0.107”	L/360 = 0.967”
Longest Span in Y-direction	0.090”	L/360 = 0.933”

	Actual Total Load Deflection	Allowable Total Load Deflection
Longest Span in X-direction	0.327”	L/240 = 1.450”
Longest Span in Y-direction	0.272”	L/240 = 1.400”

Columns:

Next in the design process is the design of the gravity columns. Calculations were done by hand using ACI 318-08 and checked with the StructurePoint Column program to ensure that they could resist loads caused by dead and live loads. Snow load was minimal therefore it was neglected for ease of design calculations. Weight takeoffs were completed using tributary areas and adding the number of floors above the 1st floor columns. The most critical column on the first floor was selected for detailed calculation. It location in plan view and tributary area is shown in Fig. 14.

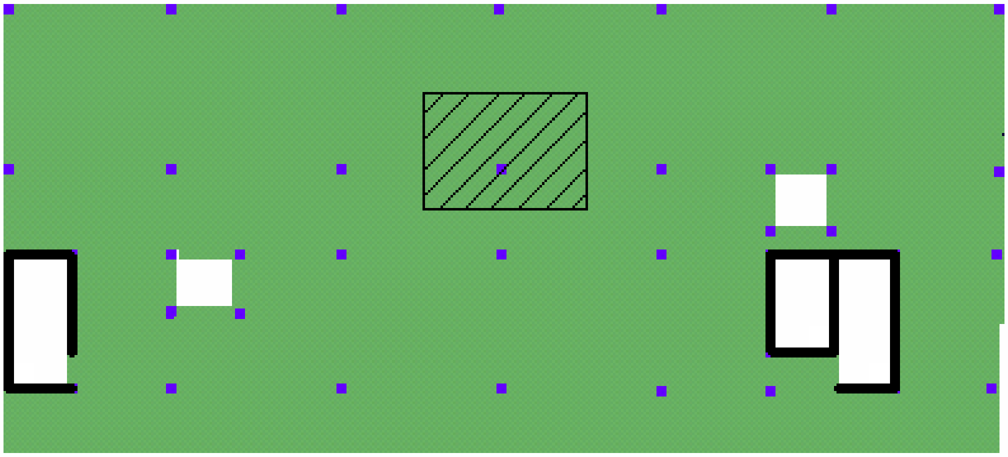
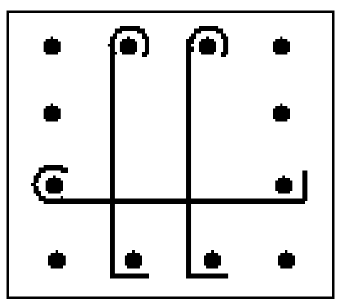


Fig. 14 – Critical Gravity Column Location and Tributary Area

Column dimensions and reinforcement designed is shown in Fig. 15. Using the controlling load combination of 1.2D + 1.6L, the maximum factored axial load on the chosen column was 1065 kips. This load combination is labeled "4" on the interaction diagram shown in Fig. 16. As shown, it is within the limits of the interaction diagram and therefore good. This size column was used for all columns, on all stories, in the design process for continuity and ease. Using this column on upper stories is sufficient although overdesigned.

18" x 18" column



(12) #11 bars (As = 18.72 in^2)
(3) #4 ties @ 24"
Clear cover = 1.5"

Fig. 15 – Column Section

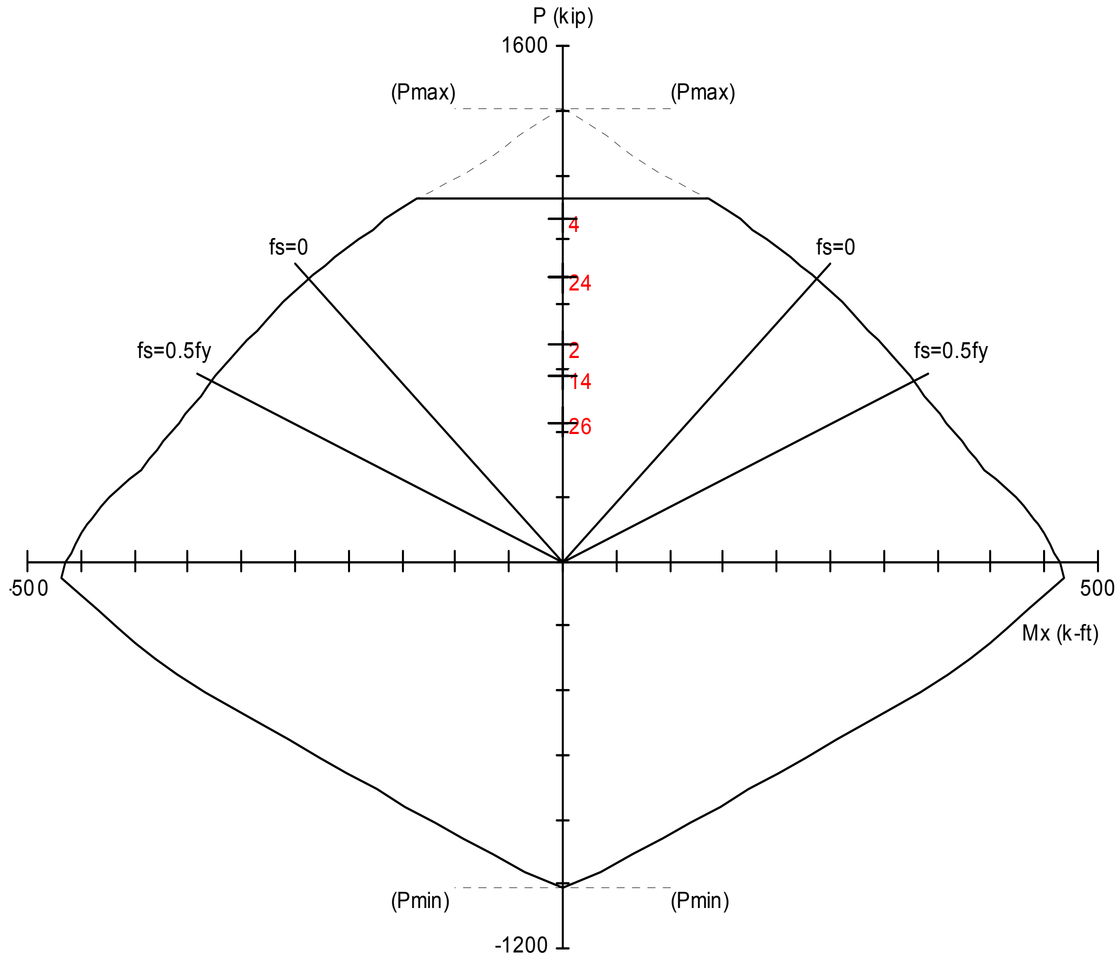


Fig. 16 – Interaction Diagram for gravity column

Foundations:

Foundations needed to be reanalyzed, mainly due the increase in overall building weight. Total building weight was calculated, much in the same way that was done to calculate seismic loading in Technical Report 3. The new building's weight ended up being 11,392 kips. This is substantially more than the current structure's weight of 8222 kips. Detailed takeoffs determining these weights are shown in Appendix C.

The foundation system currently in place consists of mini-piles with pile caps, as mentioned earlier. Allowable gravity load capacities for a 3-pile pile cap, 4-pile pile cap, and 5-pile pile cap currently in place are 743 kips, 991 kips, and 1233 kips respectively. These values were given by the Engineer of Record. For calculation, overall building weight was distributed between pile caps using tributary area methods. Total area was determined and the percentage of the total area was designated to each pile cap. Factored loads (in kips), using the controlling load combination 1.2D + 1.6L, were then calculated for each pile cap and compared with the allowable capacity. A table showing the calculated loads versus the allowable loads is shown below. As you can see (4) 3-pile pile caps were inadequate to carry the new structure's gravity loads. Locations of these piles, in plan view, are shown in Fig. 17. Changing these 3-pile pile caps to ones bearing on 4-piles will easily remedy the problem, bringing their bearing capacity up to 991 kips.

Pile Caps

	1	2	3	4	5	6	7	8	9	10	11	12	13	14
Calculated Load (k)	343	629	629	629	629	629	343	483	934	934	934	934	934	483
Allowable Load (k)	743	991	991	991	991	991	743	991	1233	991	991	991	991	991

	15	16	17	18	19	20	21	22	23	24	25	26	27	28
Calculated Load (k)	444	833	833	833	833	833	444	547	958	958	958	958	958	547
Allowable Load (k)	743	743	743	743	743	991	743	743	991	991	991	991	1233	743

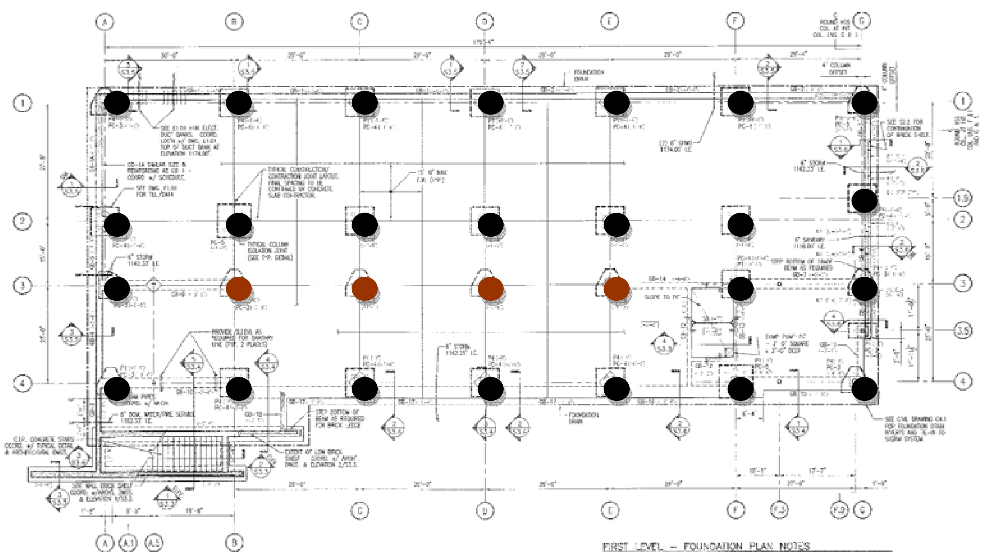


Fig 17 – Pile Redesign in red

Another subject to be discussed pertaining to foundations is overturning moment. This is a problem cause mainly by lateral loads creating moments great enough create uplift at the foundation. In Technical Report 3, I determined that uplift was not a factor in the steel structure. Now that the overall building weight has increased dramatically, foundation overturning is even less of a factor, therefore, a manual check was not completed.

Shear Walls:

In lieu of the steel moment frames to resist lateral loads, reinforced concrete shear walls were designed. To begin the design process, an adequate location for the shear walls had to be found. Being an open plan, with different room layouts at every level, it was difficult to find a location that wasn't disruptive to the current layout. Therefore, shear walls were placed around the two stairwells and the elevator shaft as shown in black on the plan in Fig. 18. Direct shear was calculated for each shear wall using lateral loads shown in Appendix B. Torsional shear did not appear to be too much of a factor and therefore was neglected for ease of calculations.

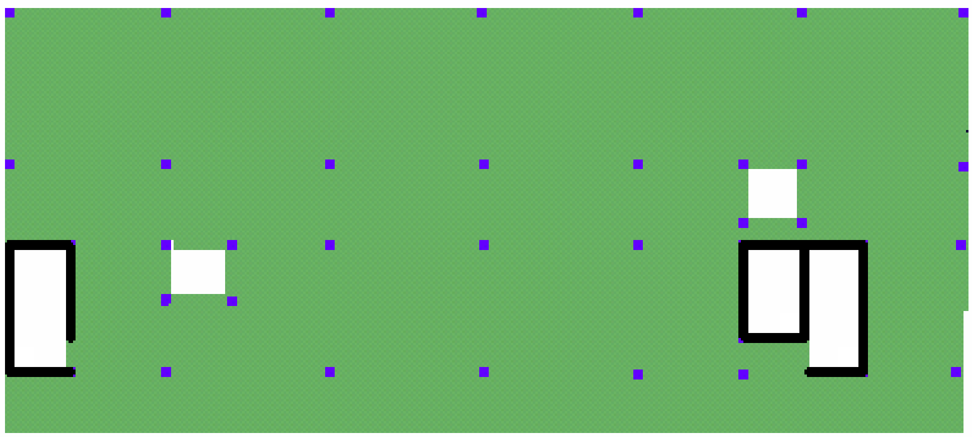


Fig. 18 – Shear walls in plan (shown in black)

The walls currently in those areas are 8" thick. For the shear walls, that thickness was increased to 18". This initial thickness was chosen to create a stiffer shear wall to resist deflections. Detailed calculations were then completed by hand for shear wall number 5 (running in the y-direction on the far right in the above plan), assuming that it was detached on both ends from the x-direction shear walls. This design would be conservative. These calculations yielded the section shown in Fig. 19. This section also shows how two shear walls are connected in the box-shaped and C-shaped configurations using horizontal reinforcement. Detailed conclusions of shear and flexural capacity calculations for this shear wall are shown in Appendix D.

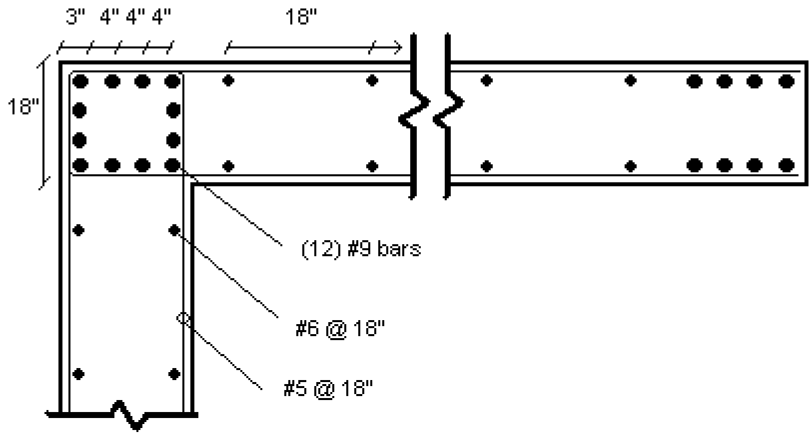


Fig. 19 – Shear wall section

In addition to load capacity considerations, serviceability requirements were examined. Deflections due to lateral loading are a major issue in shear wall design. Shear walls were modeled in ETABS and subjected to lateral loads to determine deflections, in addition to before mentioned strength checks. Story drifts and overall building drift were well within code requirements. A table summarizing actual drift values compared to allowable values for seismic load is shown below.

Controlling Seismic Drift (for Shear Walls)						
Story	Story Ht (ft)	Total Ht (ft)	Story Drift (in)	Allowable Story Drift (in) Δ = 0.020hsx	Total Drift (in)	Allowable Total Drift (in) Δ = 0.020hsx
2nd	11	11	0.185	2.64	0.185	2.64
3rd	11	22	0.28	2.64	0.465	5.28
4th	11	33	0.34	2.64	0.805	7.92
5th	11	44	0.365	2.64	1.170	10.56
6th	11	55	0.355	2.64	1.525	13.2
Roof	11	66	0.325	2.64	1.850	15.84

Depth Conclusion:

It was determined that an 8 inch two-way, post-tensioned slab is adequate to carry gravity loads. Further research is still needed for problem areas. Columns were designed to resist gravity loads and this yielded an 18" x 18" column reinforced with (12) #11 bars. Next, foundation effectiveness was evaluated for the new heavy structure. It was found that the majority of the pile caps supported the new loads, but four were inadequate. These pile caps were redesigned as 4-pile pile caps instead of 3, and this change resulted in an adequate design. The last step of the depth study was the implementation of shear walls to replace the steel moment frames. 18" thick shear walls were designed to both resist loads and deflection.

Construction Management Breadth

Cost Comparison:

For the construction management breadth, costs were determined for construction of the old steel structure and the new concrete structure. These costs were then taken into account in comparison of plausibility of each system. The detailed cost analysis was completed using RS Means Construction Cost Data. Takeoffs of building materials were done in previous reports and were utilized in these calculations. A detailed table of values obtained through analysis is shown in Appendix E, and is summarized below. The original steel superstructure cost $1,358,422 to construct and the new concrete structure costs $899,153. This cost analysis was based on the superstructure, assuming that other building components were similar in both designs. Some added costs were not taken into account, such as, the unavailability of post-tension savvy contractors. Bringing a contractor from farther away adds cost to the construction.

Concrete Structure				
Material	Cost			
	Material	Labor	Equipment	Total
Concrete	159788	0	0	158788
Formwork	162572	288787	0	451359
Reinforcing	173950	61080	0	235030
Placing	0	21907	7995	29902
Finishing	0	9798	0	9798
Post-tensioning	10359	3917	0	14276
Total	505669	385489	7995	899153

Steel Structure				
Material	Cost			
	Material	Labor	Equipment	Total
Framing	924150	118170	0	1042320
Concrete	57876	0	0	57876
Placing	0	13104	4805	17909
Metal Deck	179626	21773	2177	203576
WWF	14424	12519	0	26944
Finish	0	9798	0	9798
Total	1176076	175364	6982	1358422

Schedule Comparison:

Schedule impacts were also looked at. An estimated schedule was created using the program Microsoft Project. Durations for construction of the structure were taken from the RS Means calculations. The construction time for the existing steel structure was calculated to be 177 days while the new concrete structure was 234 days. Discrepancies between calculations and real life may be attributed to crew sizes and/or construction roadblocks. To view screenshots of the Gantt Chart formulated in Microsoft Project, see Appendix F. Lead times and construction document generation were not taken into effect.

CM Conclusion:

After doing cost and schedule calculations, it was determined that the new concrete building is a plausible alternative. Several factors were left out of the analysis though. Topics such as lead time for materials, material and labor availability for PT, and site logistics were omitted from due to time constraints. The concrete structure costs less to build but has a larger impact on the schedule. A summary of total cost and time is shown below.

	Concrete supported structure	Steel supported structure
Schedule	234 days	177 days
Cost	$899,153	$1,358,422

Lighting and Shade Design Breadth

Proposed Systems:

There were two types of shading system examined for the large, south-facing curtain wall. They are needed for mainly for shading purposes. They were implemented to relieve the reliance on the large fabric shades that are currently creating difficulty. The systems analyzed were exterior louvers and light shelves.

Louvers:

Louvers were chosen for design for several reasons. They are readily available, as shown by many nearby buildings implementing them. Also, they would help the SHC fit in even more with the university's master plan. The nearby Chemistry and Life Sciences buildings are fitted with exterior louvers. This design of louver was chosen for the SHC, and is shown in Fig. 20.

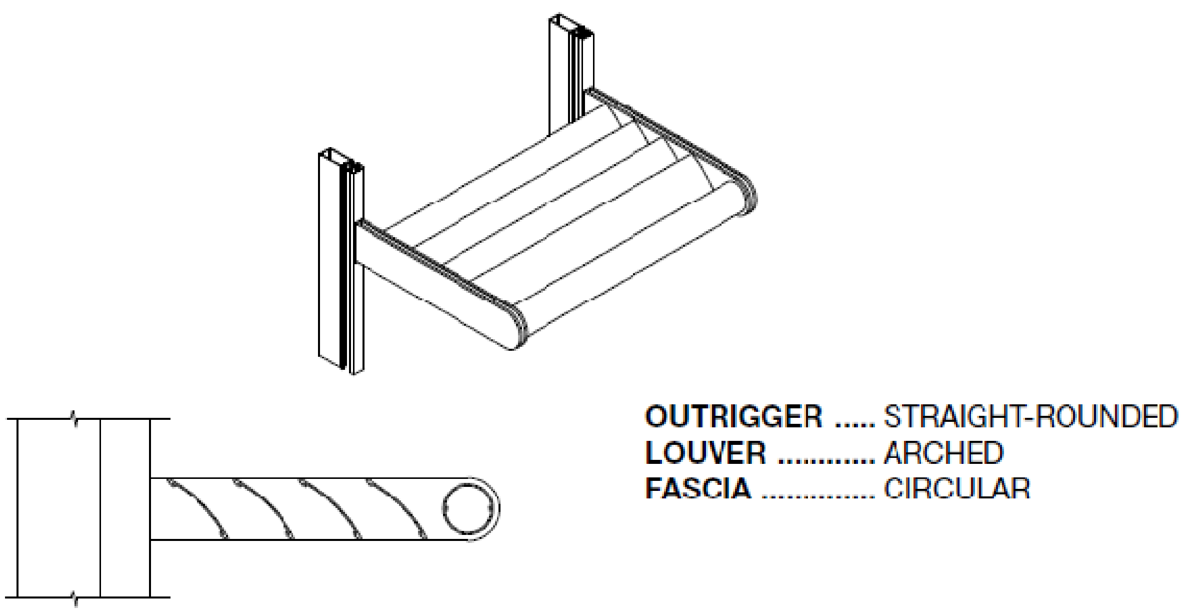


Fig. 20 – Exterior Louver

Solar calculations were done to find the angle of the sun during different times of the year. The maximum and minimum angles are during the summer and winter solstices. During the height of summer the sun is at an angle of 73° and in the winter, it is at 26°. These angles were then shown in a cross-section of the corridor adjacent to the curtain wall. As you can see in Fig. 21, the louver blocks almost all of the high summer sun while letting the lower winter sunlight in. This greatly reduces glare on the window and on interior objects. The louver is set at a height of 8 feet and the drop ceiling is 10 feet high. The maximum horizontal projection of the louver chosen was 30-3/4".

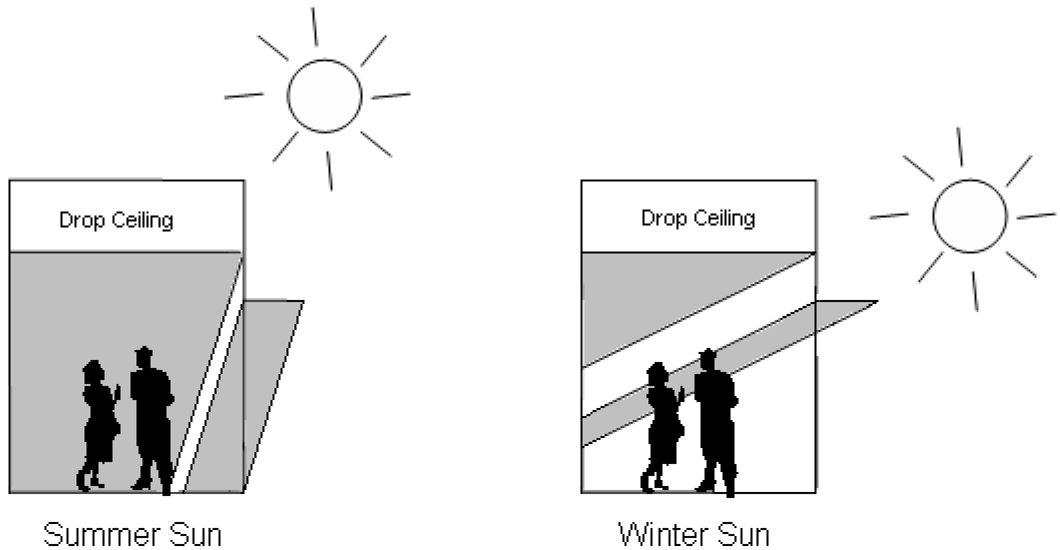


Fig. 21 – Sun shading from louver

Light Shelves:

The other system analyzed was an interior light shelf system. These components were also mounted at a height of 8 feet and extend 30" into the corridor. A light shelf's ability to deflect light farther into a room is extremely desirable. A cross section of the light shelves in the corridor and their effect is shown in Fig. 22. They spread light more evenly throughout the room and create a more desirable lighting atmosphere. This, in turn, reduces the need for some artificial lighting.

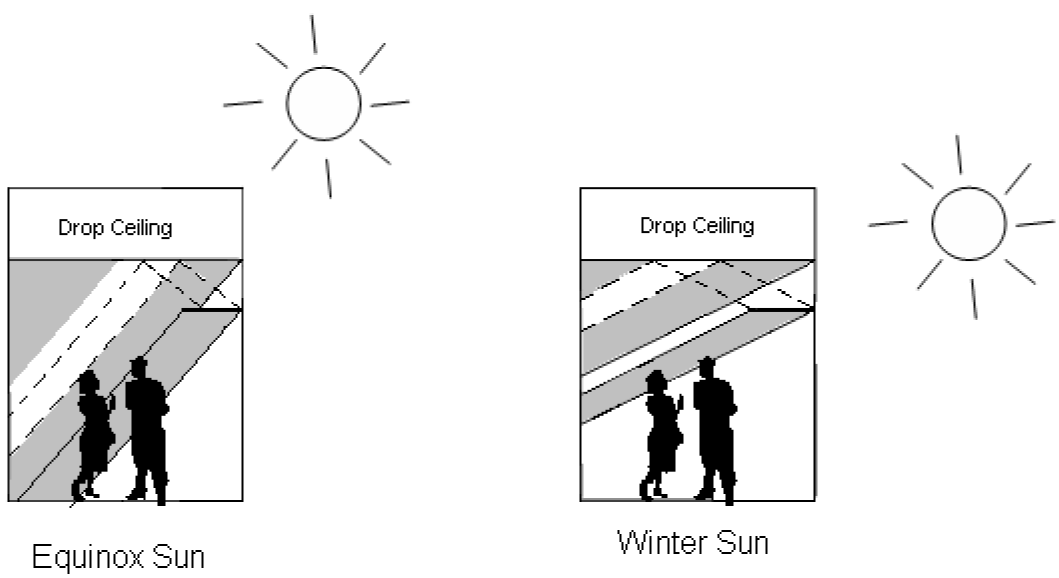


Fig. 22 – Sun shading and reflection from light shelf

Energy Savings:

Several case studies were examined to determine the amount of savings due to the incorporation of light shelves. The Florida Solar Energy Center (FSEC) performed several tests to determine the effectiveness of interior light shelves. The amount of energy savings for the SHC due to the incorporation of light shelves is projected to be around 16%.

I took this percentage and compared it to the amount of energy used by the lighting fixtures in the corridor. A spec of the 32W compact fluorescent downlights used in the corridor is shown in Appendix G. Energy usage and cost equivalents for these lights used in the corridor are shown in Appendix H. This Appendix also shows reduced cost compared to an equivalent incandescent bulb. Total cost of this lighting on two floors was determined to be around $1000. Incorporation of these light shelves would bring about a reduction of around $150 per year in lighting.

The possibility of adding dimming ballasts to these fixtures was investigated also. Even more efficiency in lighting is optimum. Dimming ballasts for compact fluorescents were researched and it was determined that they would not be ideal. Only special compact fluorescents are dimmable and most don't become visually warmer as the dim. Also, low cost ballasts only dim to about 20% before turning off.

Conclusions:

It was determined through findings that both systems have their upsides. Either system would reduce the need for fabric sun shading. When this breadth idea was started, it was to determine which system was better. After research, a combination system seems to be the best idea. The SHC would fit in with surrounding architecture, sufficiently block out the sun, and reduce building cost by reflecting light deeper into the space. A combination system is shown in Fig. 23.

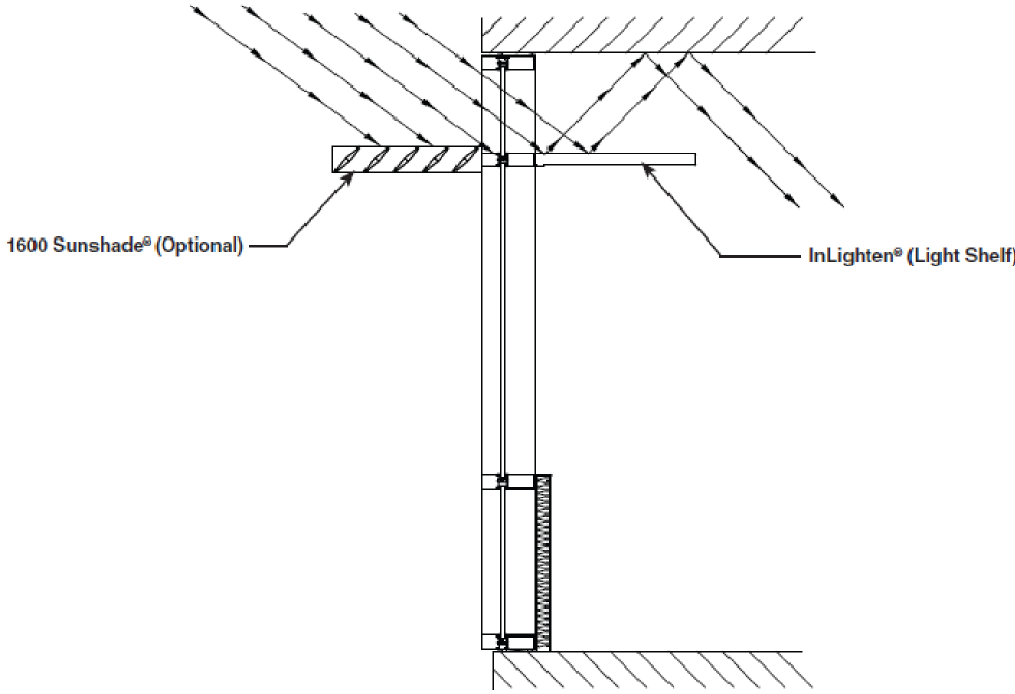


Fig. 23 – Light Shelf/Louver Combination

Thesis Summary:

All in all, I learned a lot from this thesis project. I wanted to steer away from hand calculations and learn to use computer applications more extensively. In practice, I feel that an engineer needs to learn the most efficient way to do calculations, which tends to be electronically. Throughout this semester I learn how to use ADAPT and ETABS applications. Using these programs, I was able to compute data relatively quickly.

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