FINAL REPORT

Structural Redesign of Hershey Medical Center Children's Hospital

Penn State Hershey Medical Center Children's Hospital

Hershey, Pennsylvania

Matthew V Vandersall

The Pennsylvania State University

Architectural Engineering

Structural Option

Adviser: Dr. Richard Behr

April 7, 2011

PENNSTATE HERSHEY
Children's Hospital

Hershey, Pennsylvania

Matthew V. Vandersall
Structural Option

BUILDING STATISTICS

Location: 500 University Drive, Hershey, PA 17033

Size: 263,556 SF

Number of Stories: 5 Above Grade / 1 Below Grade

Dates of Construction: 3/17/2010 – 8/20/2012

Contracted GMP Amount: $115,726,613

Delivery Method: Design-Bid-Build

PROJECT TEAM

-**Owner**: PSU Hershey Medical Center

-**Construction Manager**: L.F. Driscoll Company, LLC

-**Architect**: Payette Associates Inc.

-**Structural Engineer**: Gannett Fleming Inc.

-**Civil Engineer**: Gannett Fleming Inc.

-**M/E/P Engineer**: Bard, Rao + Athanas Consulting Engineers, LLC

-**Landscape Architect**: Hargreaves Associates

ARCHITECTURE

-Curvilinear façade ties into existing Cancer Institute Building

-Aluminum Curtain Wall system with spandrel glass

-Granite and Limestone Cladding façade along lower levels

-Out-door courtyard between Cancer Institute and Children's Hospital

-Integrated L.E.D. lighting in curtain wall mullions

-LEED certification upon completion

MEP SYSTEMS

-(5) AHU supplying 70,000 CFM each

-(2) Primary Chilled Water Pumps @ 3300 GPM each

-(2) Primary Hot Water Pumps @ 1200 GPM each

-Electrical power supplied from 15 KV feeder

-13.8 KV "K" Dry Type Transformer on 480/277V 3-Phase system

STRUCTURAL SYSTEM

Foundation: Column piers and grade beams on micropiles, 6" Slab on Grade

Gravity System: Composite floor system with 2" metal decking with 4 1/2" topping, steel frame transfer beams and columns

Lateral System: Chevron bracing and moment connections along with a composite floor system

Additional: Structure is designed to accommodate 2 additional floors

CPEP WEBSITE: http://www.engr.psu.edu/ae/thesis/portfolios/2011/mvv5009/index.html

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Figures and Tables

Acknowledgements

I would like to thank the following companies and individuals for their expertise and guidance throughout this report:

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The Pennsylvania State University

Dr. Richard Behr

Professor M. Kevin Parfitt

Professor Robert Holland

The entire AE faculty and staff

Fellow Architectural Engineering Colleagues

I dedicate this thesis to my friends and family who have supported and encouraged me every step of the way.

Executive Summary

The Hershey Medical Center Children's Hospital is located at 500 University Drive in Hershey, Pennsylvania. The existing structure consists of a composite steel deck floor system utilizing steel moment frames and concentric braced frames. Pile caps comprised of several micropiles provide foundation support for the superstructure. The overall building dimensions are 359.1 feet by 124.25 feet with a total height of 85.5 feet above grade.

The overall focus of this report was to investigate the feasibility of utilizing a reinforced concrete structure over the existing steel design. The secondary focus was to include the effects on the structure caused by the addition of two stories for the future expansion of the Children's Hospital. From this report, it was determined that a 9" flat-slab floor system utilizing 5000 psi reinforced concrete would be adequate for the floor design. Shear caps with a depth of 4.5" help to resist punching shear around each column face. The columns for all levels were determined to be 24" x 24", 20"x20", and 18"x18" square columns with various reinforcing. Lateral resistance is primarily through 16" reinforced concrete shear walls.

The effects of these changes then could be compared by performing a cost analysis for both the existing and proposed designs. It was determined that the proposed design cost more than the existing structure when taking into account only five stories of the proposed design. With the additional two floors, the total project cost was determined to be $8,137,696.81. Since both construction processes involve different tasks, the estimated project length was calculated to determine which project has a longer time frame. For the existing structural work, it was estimated that it would take 155 days for erection. The proposed design was estimated to take 289 days for the completion of the structural elements.

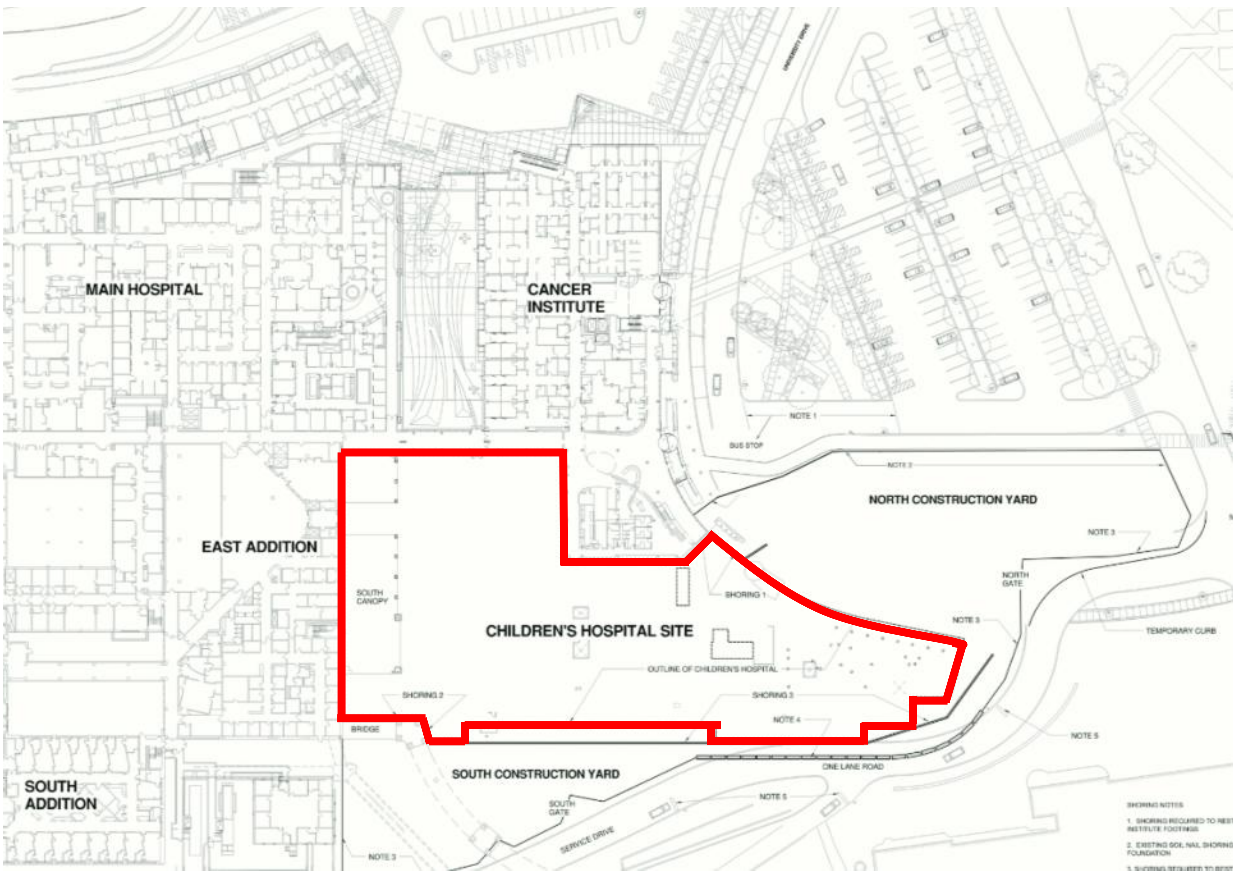
Through both these studies it can be determined that the proposed reinforced concrete system is a viable option and could have been considered for the overall design. The selection of using structural steel by the design team is unconfirmed. Other constraining factors such as time frames and proposed budgets at the time may have influenced the selection of the five story steel design rather than a 7 story reinforced concrete design.

The curtain wall on the north elevation was also redesigned as part of the building enclosure breadth. The existing curtain wall system consists of vision and spandrel insulating glass units. The heat flow rate was calculated to determine the energy transmitted through the system. An alternative "shadow box" design was proposed which consists of a monolithic glass unit, a 2" air cavity, and 2" rigid insulation. The difference in heat flow between the two designs was quantified into energy savings of $155,055.60 for the proposed "shadow box" design for the entire curtain wall section. These savings only reflect the results of the heat transfer analysis. Other factors such as manufacturing costs, structural integrity through testing, and the cost due to building life maintenance must be taken into account.

Building Overview

The new Penn State Hershey Medical Center Children's Hospital is located at 500 University Drive in Hershey, Pennsylvania. The Children's Hospital is an expansion project on the existing Cancer Institute and Main Hospital. The overall project plan calls for a five story, 263,556 square-foot addition which will contain a number of operating rooms, offices, and patient rooms specializing in pediatric care. The exterior of the building utilizes vision glass and an aluminum curtain wall system. The main curve of the façade helps to tie the building into the existing curve along the Cancer Institute. A vegetated roof garden will be situated on the third level above the existing Cancer Institute. See Figure 1 for a site plan of the Children's Hospital.

The dates of construction for the Children's Hospital are scheduled for March 2010 to August 2012. The drawing specifications for the Children's Hospital note that an additional two floors of occupancy are intended for a later date.



(Courtesy of: Payette Architects)

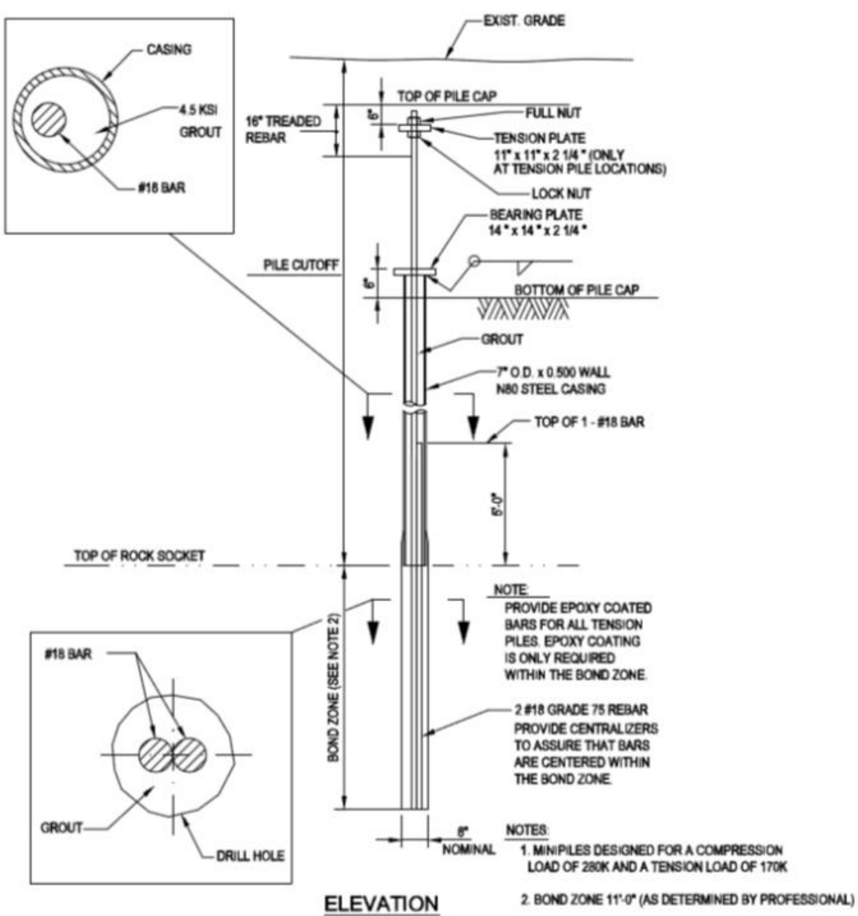
Figure 1 – Site Plan

Introduction to Structural System

The primary structural system comprises of structural steel framing integrated with a composite floor system. The composite floor consists of metal decking with normal weight concrete topping. Shear studs are welded to the supporting beam and embedded into the slab allowing interaction between the two elements. Transfer girders help to transmit the gravity loads from the beams to the columns. All of the columns consist of W14 members which allows for easier constructability. The lateral force resisting system consists of moment connected frames along the East-West direction while diagonal bracing members assist in North-South bracing.

Foundation

Due to the potential for excessive settlement, micropiles were utilized as recommended in the Geotechnical Report provided by CMT Laboratories. Micropiles consist of a casing that is injected with grout to create a friction bond within the bond zone. The piles that are used in the design are specified for a compression load of 280kips and a tension capacity of 170 kips. There are over 600 micropiles that were used in the foundation of the structure. See Figure 2 for a detail section of a typical micropile.



(Courtesy of: Gannett Fleming)

Figure 2 - Micropile Detail

The micropiles are grouped into various sizes of pile caps ranging from 3'0" x 3'0" to 10'0" x 15'0" with a depth ranging from 3' 6" to 6' 0". An example of a typical pile cap can be seen in Figure 4. Typical strut beams of 1' 6" wide by 2' 8" deep span between all pile caps to provide resistance to lateral column base movement. See "Figure 3 – Typ. Strut Beam" below.

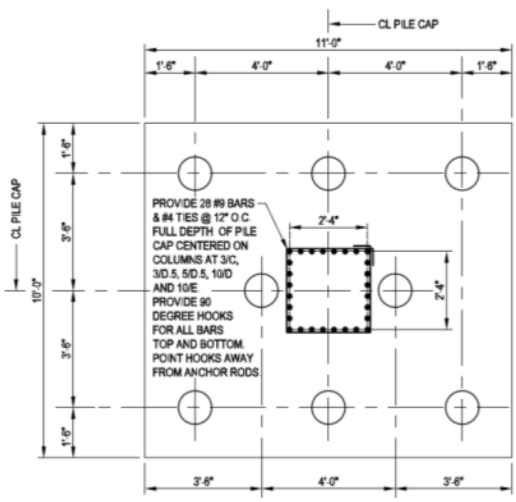
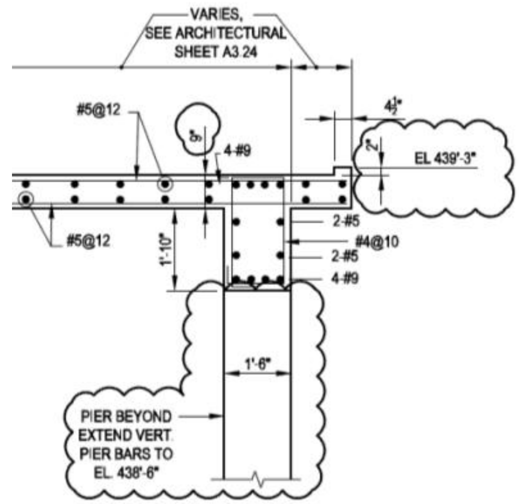


Figure 4 - P8 Pile Cap Plan



(Courtesy of: Gannett Fleming)

Figure 3 - Typ. Strut Beam

The floor at the ground level is a 5" concrete slab while in heavier load areas such as elevator pits and mechanical rooms a slab thickness of 6" is used. Below is an overview of the West End foundation plan.

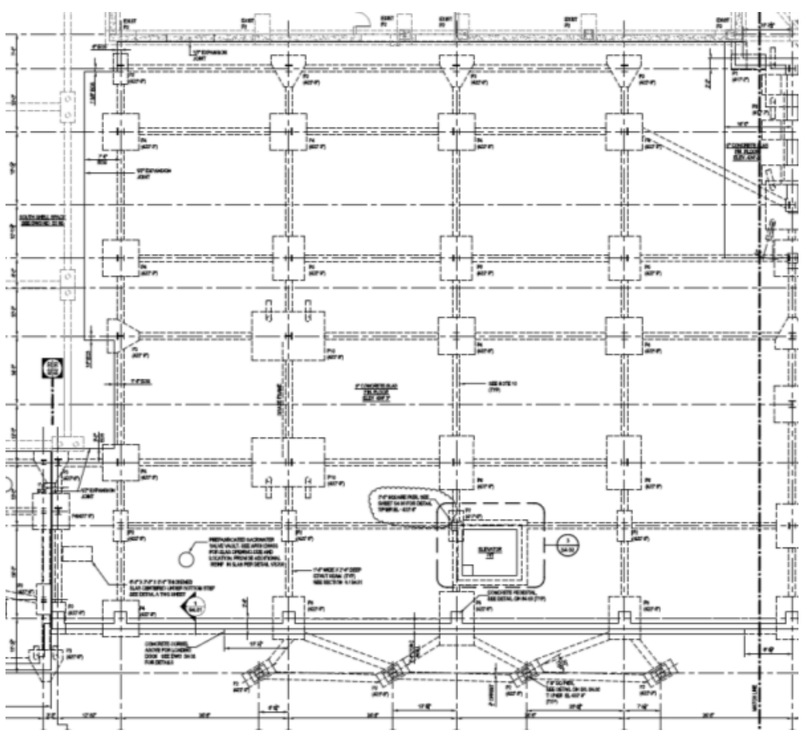


Figure 5 - West End Foundation Plan (Courtesy of: Gannett Fleming)

Floor System

The typical floor slab throughout all five stories consists of a composite floor system denoted on structural drawings as S1 TYP. This slab type is comprised of a 2" deep, 20-gage composite metal deck with a 4 ½" topping thickness. The reinforcement within the slab is 6x6 W2.1xW2.1 Welded Wire Fabric. The only change in slab thickness occurs at an area on Level 2 marked as having a slab type of S2 TYP (see Figure 6). Here, a 6" concrete slab sits on a 2" deep, 20 gage composite deck with 6x6 W2.9xW2.9 Welded Wire Fabric. The main reason behind increasing the slab thickness in this area is to account for a future MRI space where the live load is considered to be 215 PSF. All floor slabs are connected to wide flange beams using ¾" diameter shear studs where the number of studs is listed on each beam in the framing plans. The typical span for a wide flange beam is 34' 6".

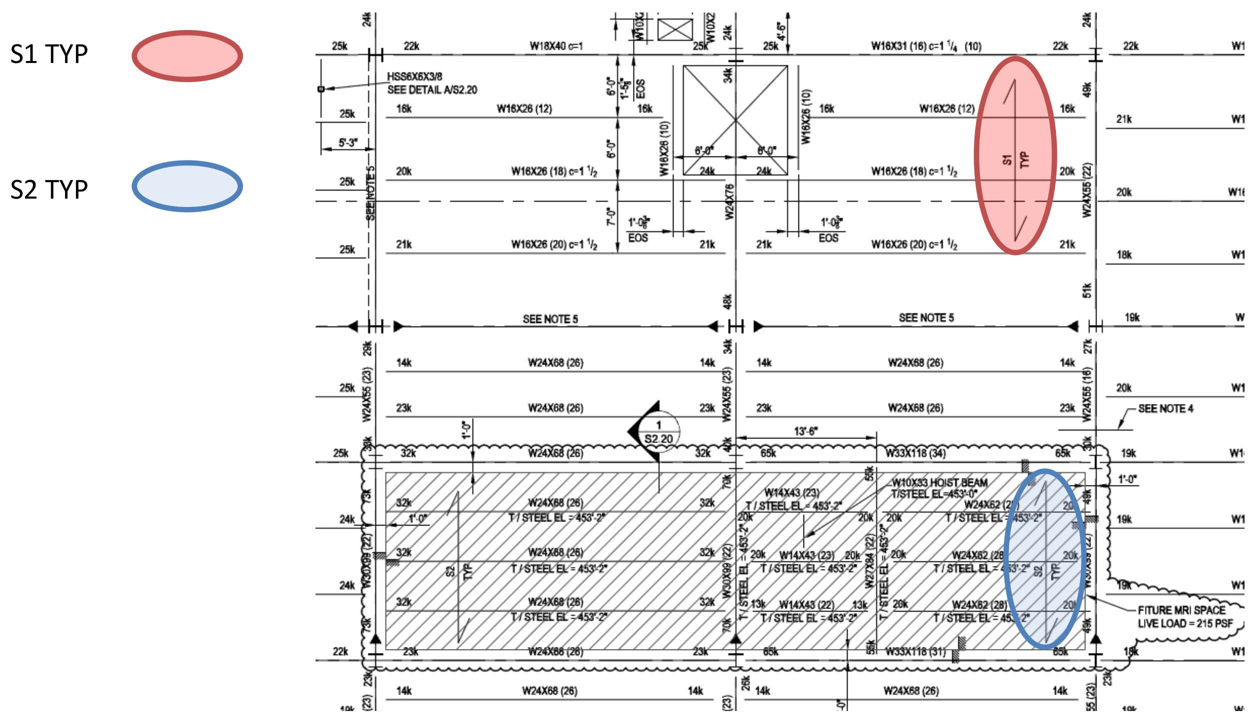
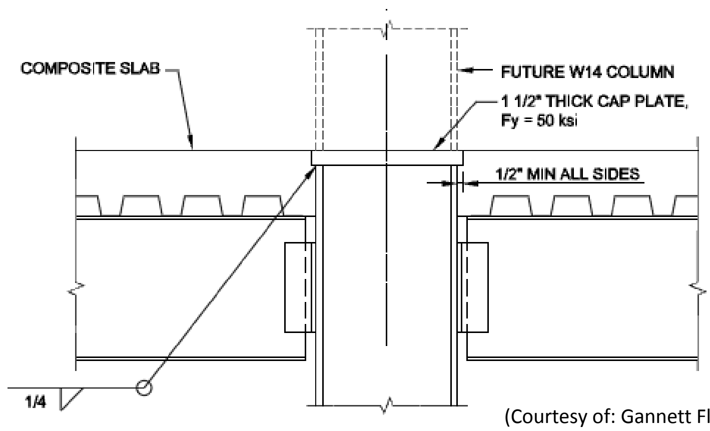


Figure 6 - Level 2 Framing Plan (Courtesy of: Gannett Fleming)

Roof System

The roof system for the Children's Hospital utilizes the same construction as the S1 TYP floor designation. Future plans call for an additional two stories of occupied space to be constructed above the current roof level. Figure 7 shows how the columns for the future sixth floor are to be attached to the existing columns. The roofing material consists of a multiple-ply built-up roofing membrane on top of insulation. Surrounding the roof is an 8" thick parapet wall that rises 1' 4" above the top of the composite slab.



(Courtesy of: Gannett Fleming)

Figure 7 - Top of Column at Future Sixth Floor

Lateral System

The main lateral force resisting system is composed of several moment frames located at the interior of the floor plan. These moment frames run in the East-West direction along the floor plan and are represented in Figure 8 with red. The purpose in placing the moment frames in these locations is to allow for a consistent and open floor space which is important for the functionality of a hospital. Running perpendicular to the moment frames are diagonally braced frames which are represented with blue in Figure 8. The locations of these braced frames are set in locations where space requirements are not as significant such as partitions to the elevator banks.

The main lateral members used in the moment frame system are wide flange sections, primarily W24x229 and W24x176 while the columns are W14x342 and W14x283. The braced frames used in the structure are comprised of W10x112 and W10x88 bracing members.

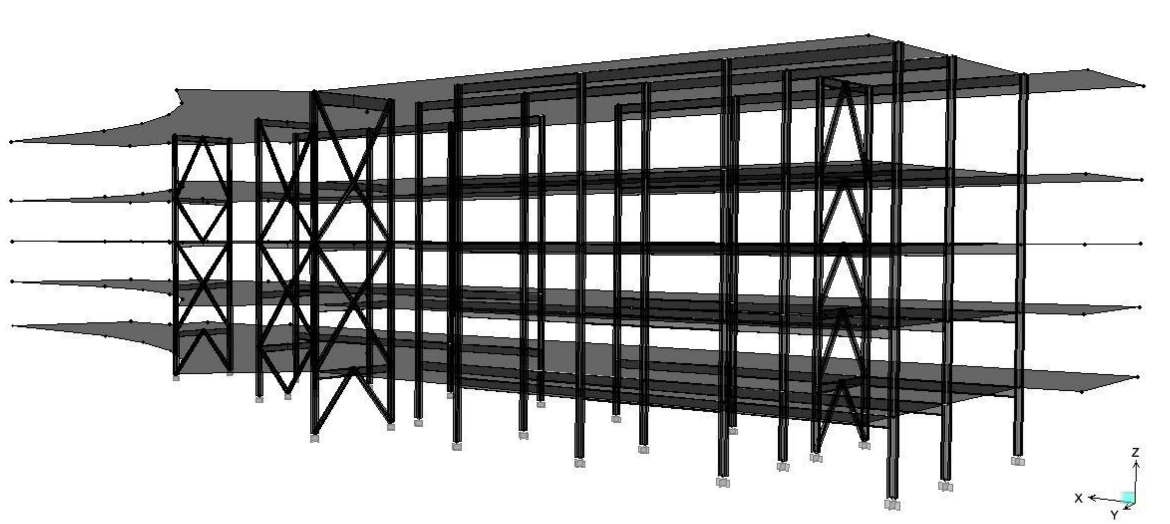


Figure 8 - ETABS model of Lateral Force Resisting System

Figure 9 – Framing Plan (Courtesy of: Gannett Fleming)

Thesis Proposal

Structural Depth

Problem 1: Structural redesign of the existing structure

The existing structure is composed of a composite floor system with structural steel framing members. From the preliminary technical reports, the structure was determined to have been designed adequately to resist the required lateral and gravity loads. Despite this, there are a few disadvantages to designing the structure using steel. The depth of the wide flanges supporting the floor can become fairly deep in certain areas depending on the applied loads. This increase in depth takes away from the floor to ceiling heights within the building. Since the existing system is structural steel all exposed members will require fire retardant spray which adds to the overall building cost. From Technical Report 2, it was determined that there were some advantages to using an alternative structural design using a reinforced concrete system.

Problem 1 Solution

During the analysis of "Technical Report 2: Structural Study of Alternate Floor Systems", it was determined that the existing structure has potential to be designed more efficiently. From Technical Report 2, it was determined that the structure could be made more efficient by switching to a reinforced concrete. As a result the overall floor to floor space could be increased. Another advantage for a concrete structure is the inherent fireproofing. This will save cost for fire retardant spray needed for the existing structure. Although formwork and lead times will adjust the costs and schedule of the project, it is estimated that these changes will make the structure more efficient.

Problem 2: Future expansion of Children's Hospital

The design for the Children's Hospital includes plans for a future expansion of the existing structure. By adding more floors to the structure, this would allow for more patient rooms, operating rooms and more office space for hospital staff. This addition would affect the existing design by increasing the lateral and gravity loads seen by the existing structural design. One of the goals will be to analyze and design the vertical expansion of the hospital in the proposed revisions.

Problem 2 Solution

The owners of the Children's Hospital would like to have flexibility for future expansion. With this in mind, additional floors will be designed for the proposed structure. The effects due to wind and seismic loads will increase due to the change in overall building height. Along with the solution to problem one, the loads due to the expansion will be analyzed to size members adequately.

Construction Management Breadth

The redesign of the structure using reinforced concrete instead of structural steel will have significant impact on the cost and schedule for the project. Direct costs associated with the redesign will include items such as base material cost, additional labor teams, and formwork. An alternative schedule will be necessary to account for the new construction process. An accurate detailed analysis of these changes in cost and project schedule will be necessary to determine the effects of the proposed changes compared with the existing design.

Building Enclosure Breadth

Due to the large amount of north facing glass on the façade of the Children's Hospital, a heat transfer analysis will be performed to analyze the efficiency of the existing curtain wall system. Based on the analysis, an alternative configuration will be proposed to decrease heat loss due to the exposed glass curtain wall. Comparisons will be made with the existing curtain wall system to quantify the energy savings of the proposed system.

Graduate Course Integration

The redesign of the structural system for the Children's Hospital will be modeled using AE 597A (Computer Modeling). An ETABS model for the concrete design will be used to determine member forces. AE 542 (Building Enclosure Science and Design) will be referenced in the design of the proposed curtain wall design. A heat transfer analysis will be used to determine the heat flow rate through both the existing and proposed systems.

Building Codes

The building codes used by the structural engineer in the design of the structural system as listed in the specifications are listed as the following:

"International Building Code, 2006 Edition"

SEI/ASCE 7-05, Third Edition – "Minimum Design Loads for Buildings and Other Structures"

AISC – "Manual of Steel Construction – Load and Resistance Factor Design"

AISC 360-05 – "Specification for Structural Steel Buildings"

AISC 303-05 – "Code of Standard Practice for Steel Buildings and Bridges"

ACI 318-05 – "Building Code Requirements for Structural Concrete"

The building codes that will be referenced throughout the research, calculations, and findings of this report are as follows:

"International Building Code, 2009 Edition"

AISC – Steel Construction Manual, 13th Edition

ACI 318-08 – "Building Code Requirements for Structural Concrete"

SEI/ASCE 7-10 – "Minimum Design Loads for Buildings and Other Structures"

Allowable Building Drift: $\Delta_{wind} = H/400$

Allowable Story Drift: $\Delta_{seismic} = 0.020h_{sx}$

Materials

Structural Steel	
Wide Flanges	ASTM A992 Grade 50
Plates, Bars, and Angles	ASTM A36
HSS Rectangular Members	ASTM A500 Grade B
HSS Round Members	ASTM A500 Grade B
Anchor Rods	ASTM F1554 Grade 36
¾" High-Strength Bolts	ASTM A325-X
Welding Electrode	E70XX

Concrete	
Pile Caps	f'c = 4000 psi
Slab on Grade	f'c = 4000 psi
Foundation Walls	f'c = 4000 psi
Column Pedestals	f'c = 4000 psi
Strut Beams	f'c = 4000 psi
Note: all concrete is normal weight concrete (145 pcf)	

Reinforcement	
Reinforcing Bars	ASTM A615 Grade 60
Welded Wire Fabric	ASTM A185

Decking	
Floor Deck	2" Composite Metal Deck, 20 Ga.
Roof Deck	1 ½" Metal Roof Deck, 20 Ga.
¾" Shear Studs	ASTM A108
Masonry	
Grout (micropiles)	f'c = 4500 psi

Table 1 - Material Specification

Structural Depth

The main scope of the structural depth will focus on the redesign of the Children's Hospital from structural steel to concrete. The existing column layout will be used in accordance with the new reinforced concrete columns. The slab design will consist of a two-way reinforced flat slab system. Shear caps will provide additional shear capacity for the columns to assist in resisting punching shear. Concrete edge beams will run along the perimeter of the slab to increase the stiffness of the exterior columns. The main lateral force resisting system will be changed to reinforced concrete shear walls for both principle directions. These will occupy the same space as the existing concentric braced frames as well as existing stairwells and elevator shafts to minimize impact on the architectural layout.

The existing structure had plans for a future expansion to be included at a later date after completion. The proposed redesign of the structure will also include the design and effects due to the two additional floors. These will be assumed to mirror the third and fourth levels which are occupied primarily by patient rooms. After verifying the proposed structural design, it will be necessary to compare it with the existing design to determine the feasibility. This will involve performing a cost analysis as well as comparing construction schedules. These can be found in the Construction Management Breadth section of this report.

Gravity - Live Loads

For the design of the structure, the following live loads were determined using ASCE 7-10. The design loads cited in the drawing specifications are also listed to provide comparison between those that the design team used and what the code provides. In most instances, in order to provide a fair comparison in building design, the original design loads were applied to corresponding areas.

Live Loads		
Occupancy or Use	**Original Design**	**ASCE 7-10**
Lobbies/Moveable Seat Areas	100 psf	100 psf
Corridors (First Floor)	100 psf	100 psf
Corridors (Above First Floor)	80 psf	80 psf
Classrooms, Scientific Labs, Offices, Etc.	80 psf	60 psf
Electrical and Mechanical Rooms	250 psf	N/A
Stairs and Landings	100 psf	100 psf
Storage Areas: Light Storage	125 psf	125 psf
Storage Areas: Heavy Storage	250 psf	250 psf
Computer Rooms	100 psf	100 psf
Courtyards	100 psf	100 psf
Future MRI Space	215 psf	N/A

Table 2 - Live Loads

Gravity - Dead Loads

Building dead loads and a description are listed below in Table 3. The superimposed dead load includes various MEP systems as well as architectural elements such as ceiling tiles and other finishes. These elements are generally fastened directly to the slab and assumed for all areas.

Dead Loads	
Normal Weight Concrete	150 pcf
Structural Steel	490 pcf
Superimposed Dead Load	30 psf
External Curtain Wall	25 psf

Table 3 - Dead Loads

Column Layout

It was determined that the existing grid layout would be generally sufficient for the initial column layout. For efficiency of the design for a two-way flat slab system, the span ratios should be about a 1:1 ratio. This will be important when designing the reinforcement for the two-way slab for the column strips and middle strips. The column lines were adjusted slightly to create a relatively even layout for the structural design. It was checked to make sure the impact to the architectural floor plans was minimal and would not disrupt any occupied spaces. The general layout for the columns can be seen in Figure 10 below. All columns extend the full height of the structure and there are no offsets of columns between floors.

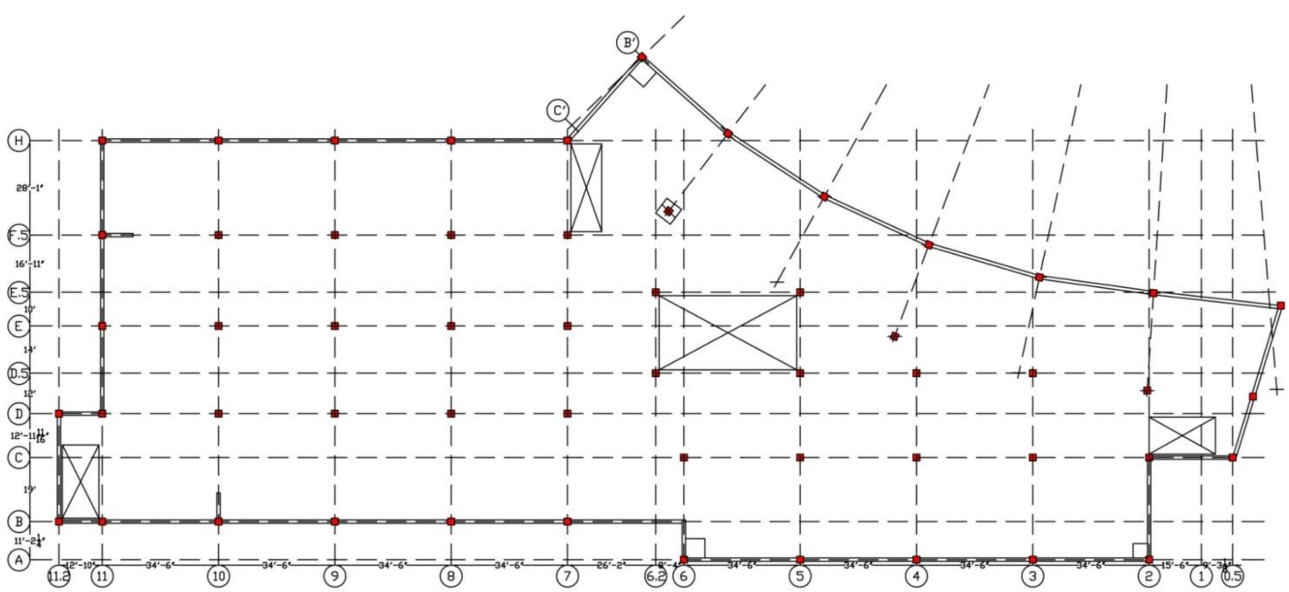


Figure 10 - Column Layout

The overall building dimensions are 359 feet by 124 feet. Due to the addition of two floors, the new structural height will be 115 feet as opposed to 85 feet previously. From Figure 10, it is apparent that the west end of the layout is fairly regular and uniform with bay sizes approximately 30' x 34.5'. The east end of the layout deviates due to the geometric irregularity of the façade. All the columns in this section are positioned in non-critical locations that do not interfere with the architectural plans.

Two-Way Flat Slab Design

From analyzing alternative floor systems in technical report 2, the use of a concrete structural system was determined to be feasible. Based on the proposed column layout, it was determined that the use of a two-way flat slab system would be adequate. Due to the geometry of the floor plans and floor openings, a two-way system could accommodate the offset of columns around the curved section relatively efficiently. In order to design the floor slab, only one critical floor section was taken into account by hand. It is assumed that all other frame sections would be designed using the same principal method. RAM Concept, a finite element modeling program, was selected to aid in the verification of the design as a whole.

Calculations were performed for Frame 9, which can be located from Figure 10, to design the floor slab. The hand calculations for the slab design can be found in Appendix E. The assumptions for the design were the use of a 9" thick slab with drop panels extending 4.5" below the slab. Since the spans are rather large, a compressive strength of 5000 psi was used to help minimize the overall depth of the floor slab. Moment distribution was performed to determine the design moments at the supports and mid-span for the slab. Design aids taken from MacGregor 2009 were used to determine the moment distribution coefficients for the equivalent slab-beam and columns. These moments were distributed to the column strip and middle strip of the slab. Reinforcement was then designed for the column strip and middle strip for each span. Table 4 shows the determination of reinforcement for one of the joints in the frame. For reinforcement design at all sections along the span, refer to Appendix E.

Joint 1 Reinforcement	***Middle Strip***	***Column Strip***	***Middle Strip***
Strip Width, ft	8.625	17.25	8.625
Exterior Negative Moment (kip-ft)		-455.7	
Moment Coefficient	0.033	0.934	0.033
Distributed Moments	-15.0381	-425.6238	-15.0381
Required A_s (in^2)	0.49	13.96	0.49
Minimum A_s (in^2)	1.68	3.35	1.68
Selected Steel	6 #5 bars	24 #7 bars	6 #5 bars
Provided A_s (in^2)	1.86	14.4	1.86

Table 4 - Example of Slab Reinforcement

The amounts of reinforcing varies from span to span but bar sizes were kept as consistently as possible. For the middle strips, #5 reinforcing bars were selected since moments in these spans were much less than in the column strips. For the greater moments in the column strips, #7 and #8 bars were selected to provide greater areas of steel reinforcing. This procedure was performed for one direction of the floor slab to demonstrate the approach. For the two-way system it would be necessary to perform the calculation for the perpendicular spans. From this the reinforcement for the two-way action could be developed. For ease of design, a computer model was generated to design and verify the two-way slab for the entire design.

RAM Concept Model

A model was constructed using RAM Concept to design reinforcement for the entire floor slab. This would allow for an optimal slab design at critical sections that would have been difficult to calculate by hand. The compressive strength of all elements was set to 5000 psi concrete. Punching shear checks were selected to be performed for all columns. RAM would design additional shear stud reinforcement using ¾" diameter stud rails where needed. The initial slab thickness for all floors was designed using a 9" slab depth determined from hand calculations. Shear caps were used to increase the shear capacity around the columns. The dimensions for the shear caps are generally 8 feet by 8 feet varying slightly in areas which failed in punching shear. The depth of all shear caps was kept constant at 4.5".

The initial step was to determine the orientation of the design spans used to generate the reinforcement layout in the column strips and middle strips. For the rectilinear areas, the design spans were selected to run orthogonally to the floor plan. In non-uniform areas, the design spans were selected to run parallel to the geometry of the space. Figure 12 shows the projected column strips and middle strips used in RAM Concept to design the reinforcement.

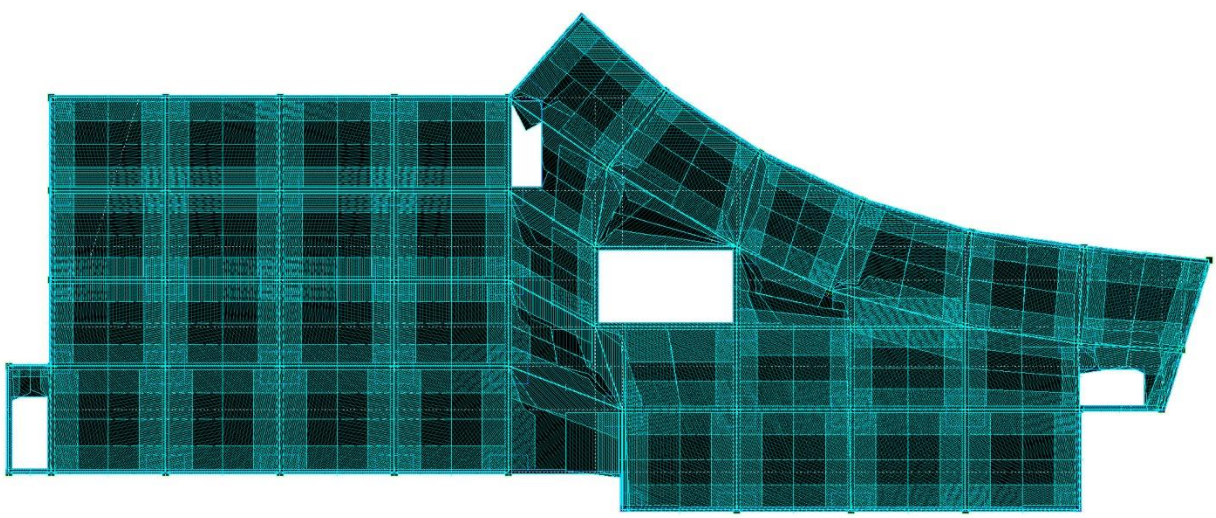


Figure 11 - Column Strip and Middle Strip Layout

Load combinations were used as defined in ASCE 7-10. Since lateral loads are not considered in the design of the floor slab, only the following load combinations were considered:

- 1.4D
- 1.2D + 1.6L + 0.5L_r

Gravity loads were applied to the model as cited previously in the Gravity Loads section. A live load of 80 psf was used for the patient rooms and corridors while 100 psf was used for lab spaces and computer areas. According to ACI 318-08, live load patterns should be used to determine the maximum moments at the column faces. Figure 13 below shows that alternatively loaded bays were used as the load patterns in RAM Concept.

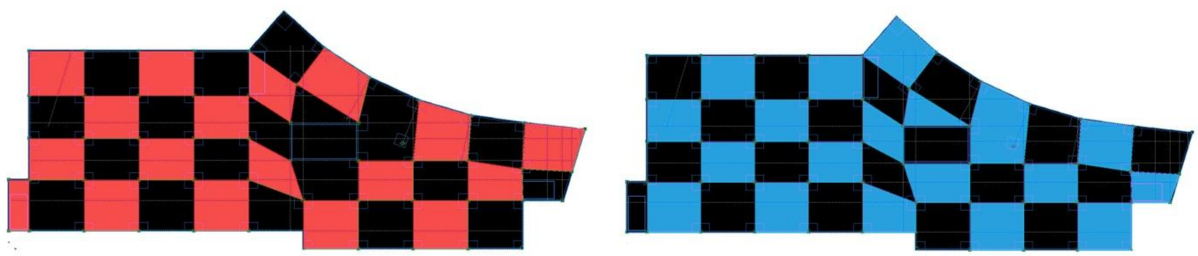


Figure 12 - Live Load Patterns

From the RAM Concept model, the design was adjusted at areas where issues were determined. For the final design the use of a 9" floor slab with shear caps meets the design criteria. The reinforcement used throughout the floor slab were #5 bars for the middle strips while column strips utilized #7 to #9 bars. This design was checked for other floor types in the building. Issues arose when checking the loads for the penthouse level. Figure 14 shows the resulting deflection at all points of the floor slab. In this diagram, darker colors indicate areas with greater deflections, as shown in the scale. The purple area along the curved section shows that the maximum deflection is 1.608 inches. The deflection criterion for the floor slab under total load deflection is L/240. The span for this section is 34.5 feet, which means that the maximum allowable deflection is 1.725 inches. As a result the slab satisfies the maximum allowable deflection for serviceability.

The penthouse level holds most of the mechanical equipment and therefore, the live load for this space was 250 psf. This is a considerable increase in live load from all previous floors. The use of a 9" slab with 5000 psi concrete was not sufficient to withstand the extra loading. As a result, the penthouse level will utilize an 11" slab with 6000 psi concrete for all structural elements including columns and shear caps on that level. These increases in material strengths satisfied all design requirements of the penthouse floor slab.

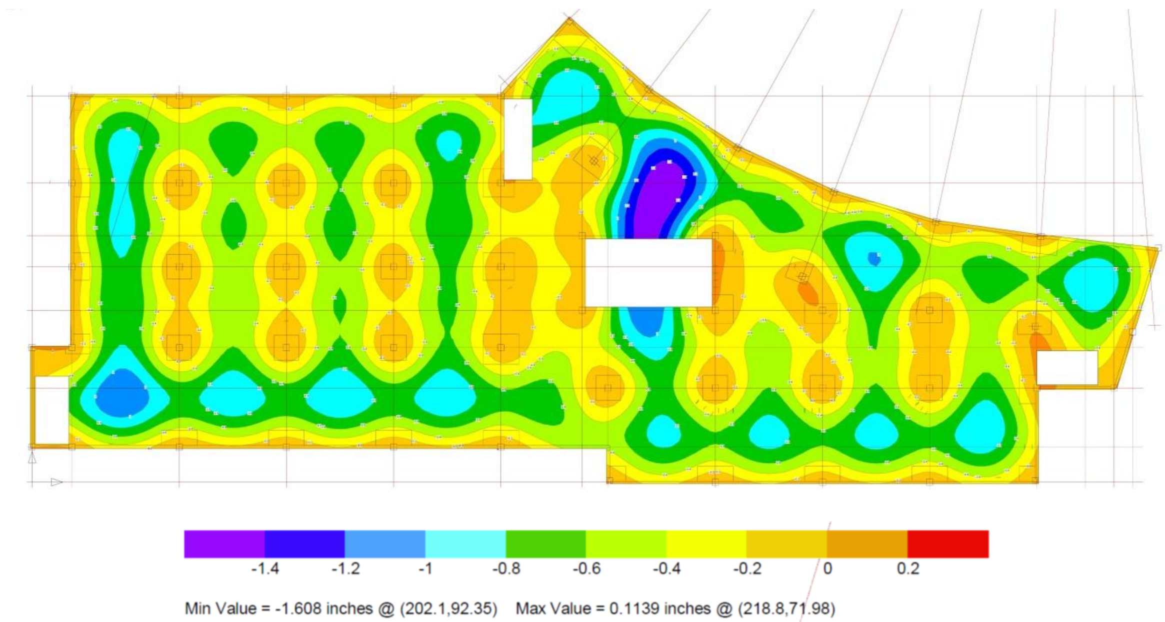


Figure 13 - Maximum Slab Deflections

Results and Discussion

For the final design, it was determined that a 9" thick reinforced concrete slab using 5000 psi concrete was satisfactory for most floors. The penthouse level will be designed using an 11" thick reinforced concrete slab using 6000 psi concrete. All slab sections were found to have adequate strength to resist the applied gravity loads. Shear caps are used around all columns to increase the shear capacity. Sizes of shear caps vary slightly, but the overall dimensions are generally 8 feet by 8 feet with a depth of 4.5". Through punching shear checks, it was determined that all sections were satisfactory against punching shear. With the floor slabs successfully designed, it was then necessary to design the reinforced columns for the structure.

Column Design

The columns for the Children's Hospital were designed using the computer program RAM Structural System. There are a total of three various column sizes that were used in the design for the Children's Hospital. The columns on the bottom two stories are supported by 24"x24" square columns. The columns on floors three and four are supported by 20"x20" square columns. The fifth story, penthouse level, and roof are supported by 18"x18" columns. To provide sufficient reinforcement for each column, three bar pattern groups were considered:

- 14 bars (4 x 3), longitudinal:#6 - #10, transverse: #3
- 16 bars (5 x 3), longitudinal:#6 - #10, transverse: #3
- 20 bars (6 x 4), longitudinal:#6 - #10, transverse: #3

These reinforcement pattern combinations would be used to optimize the design of each column. With this is mind the model was constructed using these assumptions. Since RAM Structural System was not used to design the floor systems, only the type of concrete system was allowed to be specified. For this, the floor was selected to observe two-way action between columns. The importance of this feature is how it will affect the distribution of area loads to the surrounding columns. Applicable load combinations were generated within the program to be applied to each column. From the results of the analysis, the critical load combination was used for the design of each column. Generally it was determined that the 1.2D + 1.6L + $0.5L_r$ load combination controlled.

Results and Discussion

From the analysis results generated from RAM, the load capacity ratios show which columns satisfy the interaction diagram for each column. Figure 15 shows a rendering graphically representing the load capacities. The more critical columns which failed under the critical load combination are colored red. Columns which are more than satisfactory under the controlling load combination are shown in blue. To satisfy the interaction diagrams, columns which were found to have failed were then modified by changing reinforcement bars or patterns.

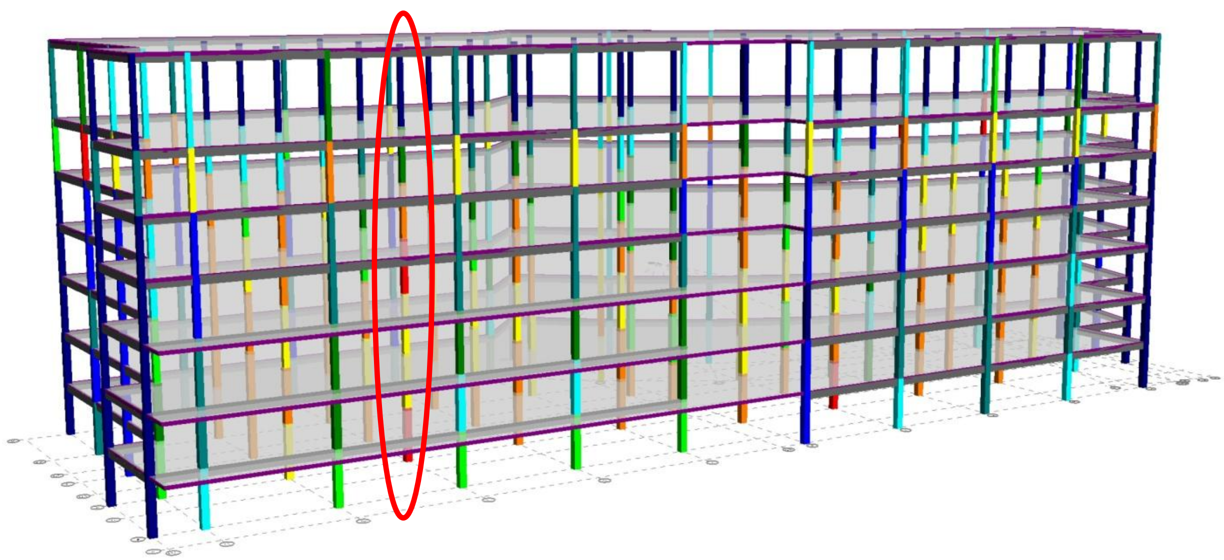
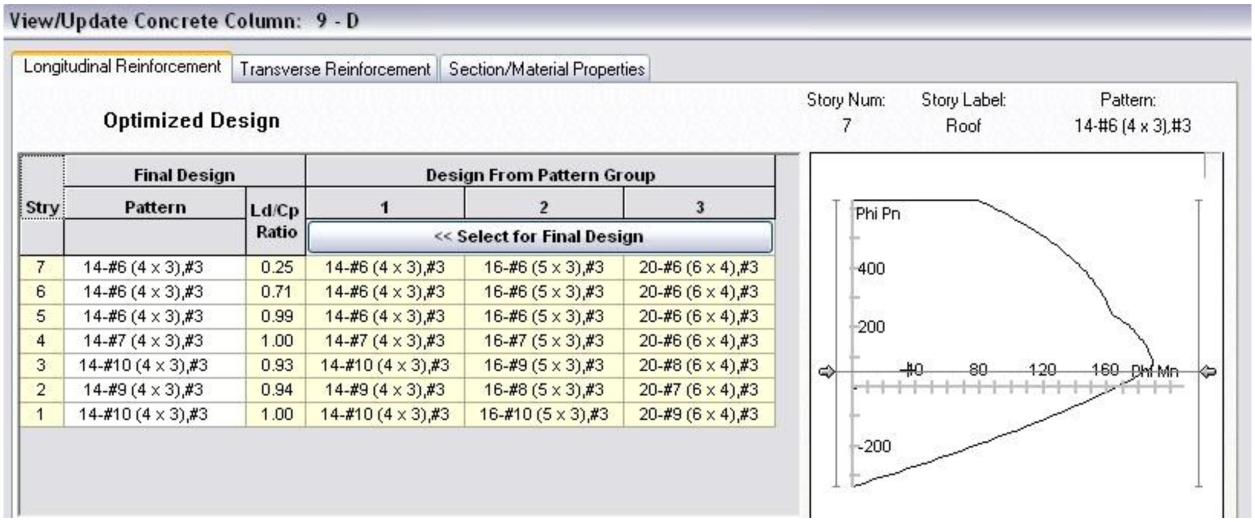


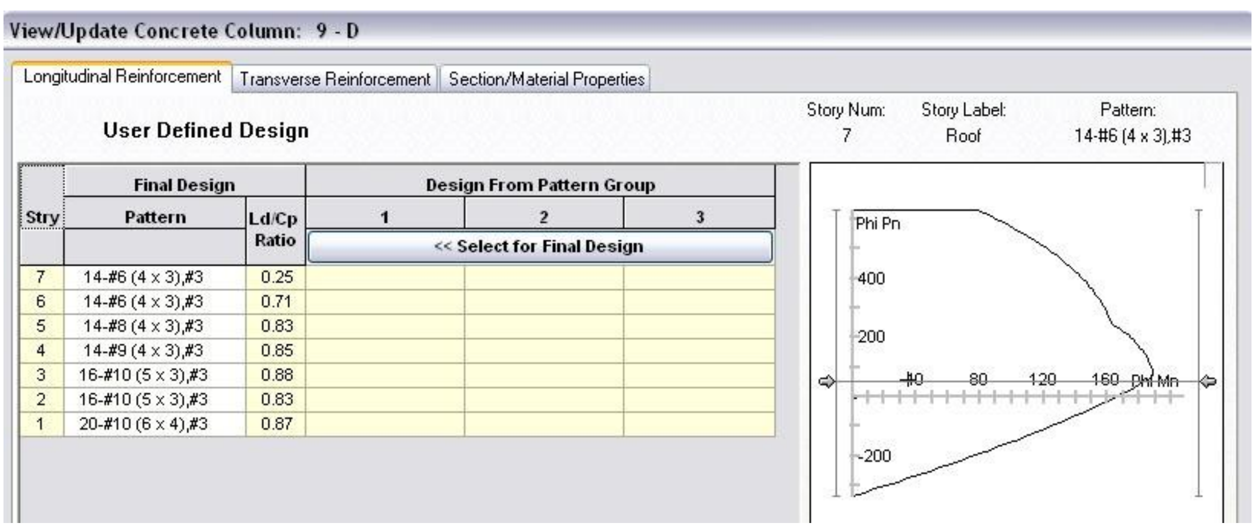
Figure 14 - Column Design Interaction Values

As a representation of the process, Column D-9 was determined to have failed at the ground floor level and the fourth floor level as seen highlighted in Figure 15. The design window for this column location can be seen in Figure 16. The load capacity ratios of multiple columns had either failed or were approaching failure. The reinforcement which the program selected was (14) #9 and (14) #10 bars. The final design reinforcement pattern was modified to provide additional load resistance for the column. Figure 17 shows that the redesigned column using (16) #10 and (20) #10 bars. With these changes the column was found to have sufficient capacity. This process was repeated for other similar cases throughout the model. Figure 18 shows the graphical representation of the changes made to the column reinforcement. It was then determined that all columns were of sufficient capacity.



Stry	Final Design Pattern	Ld/Cp Ratio	Design From Pattern Group 1	2	3
7	14-#6 (4 x 3),#3	0.25	14-#6 (4 x 3),#3	16-#6 (5 x 3),#3	20-#6 (6 x 4),#3
6	14-#6 (4 x 3),#3	0.71	14-#6 (4 x 3),#3	16-#6 (5 x 3),#3	20-#6 (6 x 4),#3
5	14-#6 (4 x 3),#3	0.99	14-#6 (4 x 3),#3	16-#6 (5 x 3),#3	20-#6 (6 x 4),#3
4	14-#7 (4 x 3),#3	1.00	14-#7 (4 x 3),#3	16-#7 (5 x 3),#3	20-#6 (6 x 4),#3
3	14-#10 (4 x 3),#3	0.93	14-#10 (4 x 3),#3	16-#9 (5 x 3),#3	20-#8 (6 x 4),#3
2	14-#9 (4 x 3),#3	0.94	14-#9 (4 x 3),#3	16-#8 (5 x 3),#3	20-#7 (6 x 4),#3
1	14-#10 (4 x 3),#3	1.00	14-#10 (4 x 3),#3	16-#10 (5 x 3),#3	20-#9 (6 x 4),#3

Figure 15 - Column D-9 Design Window



Stry	Final Design Pattern	Ld/Cp Ratio	Design From Pattern Group 1	2	3
7	14-#6 (4 x 3),#3	0.25			
6	14-#6 (4 x 3),#3	0.71			
5	14-#8 (4 x 3),#3	0.83			
4	14-#9 (4 x 3),#3	0.85			
3	16-#10 (5 x 3),#3	0.88			
2	16-#10 (5 x 3),#3	0.83			
1	20-#10 (6 x 4),#3	0.87			

Figure 16 - Column D-9 Revised Design Window

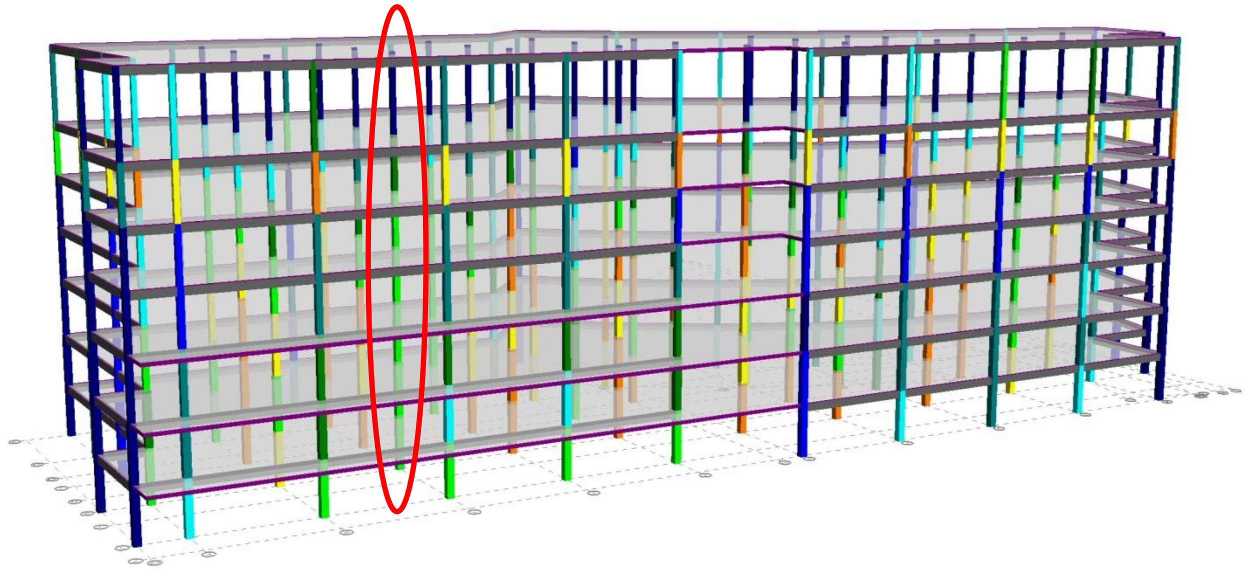


Figure 17 – Revised Column Design Interaction Values

These results were also checked by hand calculations which can be found in Appendix F. A typical interior column, edge column, and corner column were designed to show the design process. The maximum combined axial and moment was used from the gravity and lateral models in determining the capacity of each column. Design aids using MacGregor 2009 were referenced to determine the reinforcement ratio for the column dimensions under the given loads. Reinforcement was then selected to satisfy the necessary reinforcement ratio. This was then checked using the computer analysis program "pcaColumn" to develop the interaction diagram for each column type. In addition, the maximum spacing was checked along with the transverse reinforcement spacing to ensure the reinforcement was not exceeding the dimensions of the column. From the hand calculations, it was verified that all the column types provided sufficient strength capacity to resist the maximum factored load combinations.

Lateral Design (MAE Coursework)

With consideration of the future expansion of the Children's Hospital, two additional floors were included in the structural redesign. This increase in height would increase the lateral forces experienced in the structure due to wind and seismic loading. The existing lateral force resisting system was a combination of steel moment frames and concentrically braced frames. With the redesigned structure using concrete, the main lateral force resisting system will be switched to concrete shear walls. The following subsections show the determination of wind pressures and seismic forces at each floor level along with the corresponding base shears and overturning moments.

ETABS Model

Using knowledge learned in AE 597A "Computer Modeling," an ETABS model was constructed to best represent the structural redesign of the lateral system. Concrete shear walls were chosen as the main lateral force resisting system which will be cast-in-place monolithically with rest of the structure. Figure 19 shows the layout of the shear walls. The locations of shear walls 1 to 4 make use of the existing braced frame locations. Therefore no impact on the existing architectural floor plans will occur. The locations of shear walls A to D were incorporated into areas where impact on the layout would be minimal such as elevator walls and stairwells.

Since it is assumed that the lateral loads applied at each level will cause each point to displace the same, all points were constrained using a rigid diaphragm for each floor level. The lateral loads were applied to each diaphragm and would act at the center of mass at each floor level.

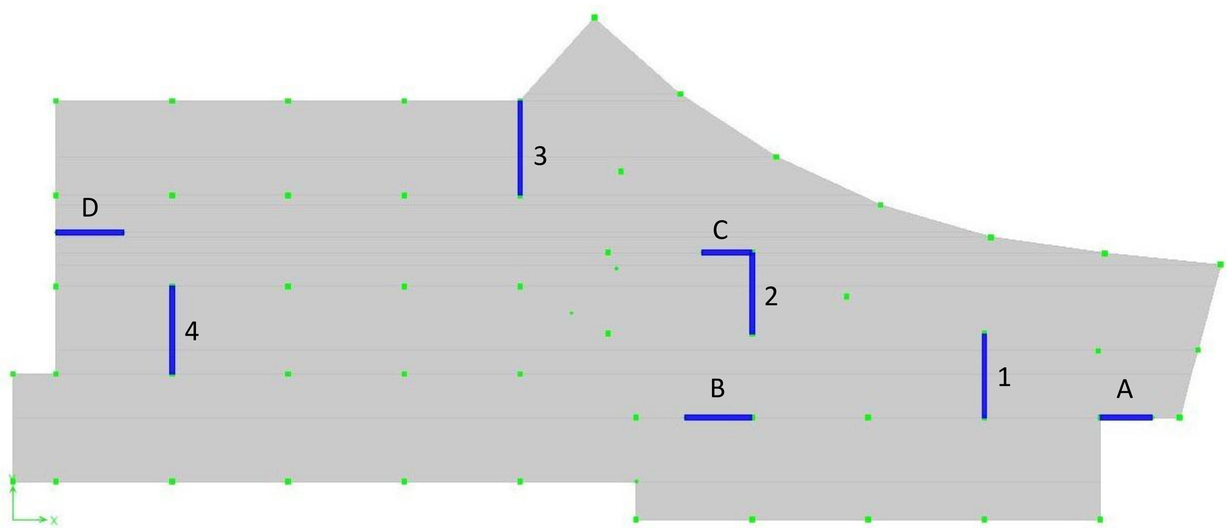


Figure 18 - Shear Wall Layout

Column sizes determined from the column design section were incorporated into the ETABS model. Property modifiers of $0.7I_g$ were applied to all columns to account for cracked sections. Shear walls were assumed to have a thickness of 16 inches. It is assumed that shear walls would take no out-of-plane forces. For this reason, the shear walls were selected to act as a membrane. According to ACI 318-08 Section 8.8.2, "lateral deflections of reinforced concrete building systems shall consider the reduced stiffness of all members under the loading conditions by 10.10.4.1 (a) through (c) or by 50% of stiffness based on gross-section properties." Therefore, a membrane f_{22} modifier of 0.5 was used for the shear walls.

Wind Loading

Wind load analysis was performed using ASCE 7-10 for Main Wind Force Resisting Systems (MWFRS). Using this design procedure, the design wind pressures were determined using a simplified 359 feet by

124 feet rectangle with a building height of 115.5 feet to the top of the parapet. All design assumptions and calculations for the design wind pressures can be found in Appendix A. The base shear and overturning moment were calculated for both principal directions. These values are shown in Tables 5 and 6 below. For reference, the base shear for the original five story structure in the East/West and North/South directions were 492.6 kips and 1549.2 kips respectively. It is therefore reasonable that the proposed structure will be designed using additional lateral loads.

The story forces found in each direction were then applied to the ETABS model. Since deflection due to wind loading is a serviceability issue used to prevent excessive sway, no load factors were used in the analysis of the wind load cases. From the controlling wind case, story drifts were recorded from the analysis and compared to the allowable story drift. This will be discussed further in the Wind Serviceability Check section in this report.

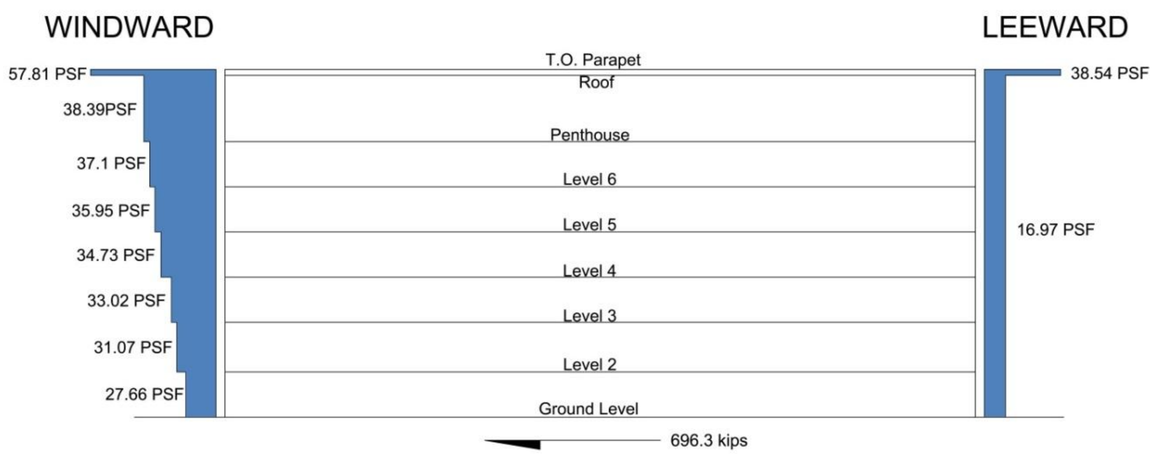


Figure 19 - Wind Pressure East-West Direction

Wind Forces, Story Shears, Overturning Moment For East/West Direction									
Level	Height Above Ground	Floor Height	Tributary Area Below	Tributary Area Above	pz below	pz above	Force	Shear	Moment (Fx*height)
	ft	*ft*	*Sf*	*sf*	*psf*	*psf*	*kips*	*kips*	*kip-ft*
T.O. Parapet	115.5	0	124.25	0	57.81	0	11.97	11.97	1382.71
Roof	113.5	2	1366.75	124.25	38.39	57.81	82.46	94.43	9359.27
Penthouse	91.5	22	931.88	1366.75	37.17	38.39	125.47	219.90	11480.27
6	76.5	15	931.88	931.88	35.95	37.17	99.24	319.14	7592.14
5	61.5	15	931.88	931.88	34.73	35.95	96.97	416.12	5963.78
4	46.5	15	1025.06	931.88	33.02	34.73	98.88	514.99	4597.86
3	30	16.5	931.88	1025.06	31.07	33.02	95.47	610.46	2864.13
2	15	15	931.88	931.88	27.66	31.07	85.84	696.31	1287.61
Ground	0	15	0	931.88	27.66	27.66	0	696.31	44527.76

Table 5 - Wind Forces, Story Shears, Overturning Moment for East-West Direction

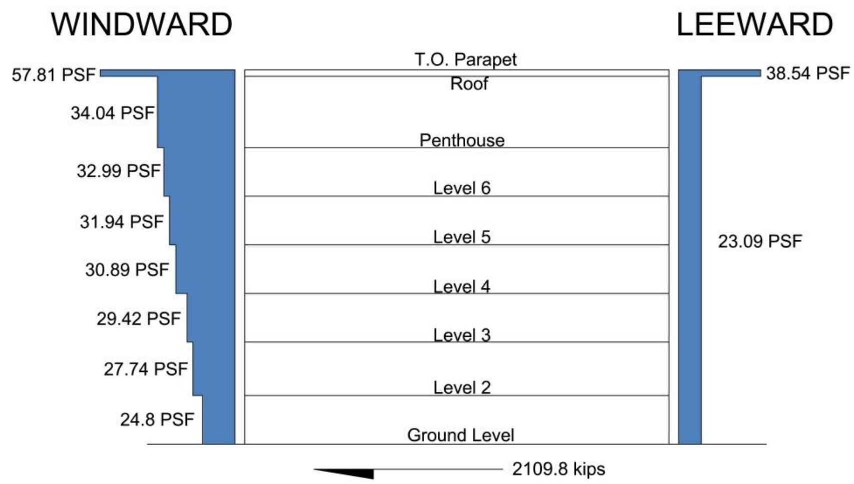


Figure 20 - Wind Pressure North-South Direction

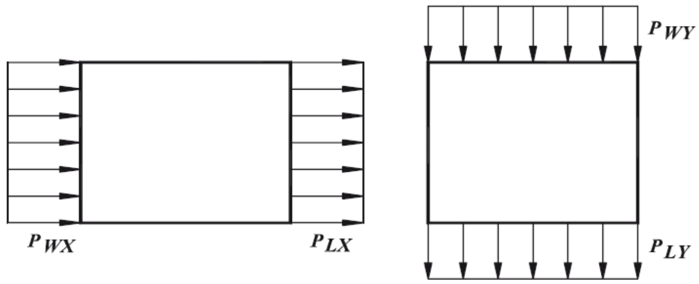
Wind Forces, Story Shears, Overturning Moment For North/South Direction									
Level	**Height Above Ground**	**Floor Height**	**Tributary Area Below**	**Tributary Area Above**	**pz below**	**pz above**	**Force**	**Shear**	**Moment (Fx*height)**
	ft	*ft*	*sf*	*sf*	*psf*	*psf*	*kips*	*kips*	*kip-ft*
T.O. Parapet	115.5	0	359.10	0	57.81	0	34.60	34.60	3996.22
Roof	113.5	2	3950.10	359.10	34.04	57.81	246.42	281.02	27968.99
Penthouse	91.5	22	2693.25	3950.10	32.99	34.04	376.70	657.72	34467.66
6	76.5	15	2693.25	2693.25	31.94	32.99	299.24	956.95	22891.57
5	61.5	15	2693.25	2693.25	30.89	31.94	293.58	1250.53	18055.09
4	46.5	15	2962.58	2693.25	29.42	30.89	300.93	1551.46	13993.29
3	30	16.5	2693.25	2962.58	27.74	29.42	292.44	1843.91	8773.34
2	15	15	2693.25	2693.25	24.80	27.74	265.86	2109.76	3987.85
Ground	0	15	0	2693.25	24.80	24.80	0	2109.76	134134.00

Table 6 - Wind Forces, Story Shears, Overturning Moment for North-South Direction

According to ASCE 7-10, the design wind load cases shall be checked to determine the controlling wind scenario. Using these wind loadings, the following wind cases were considered. Each wind case provides an image of the wind force considered and the resulting forces caused in each shear wall at the ground level.

Wind Case 1:

Wind Case 1 considers the full wind pressures acting perpendicular to the building structure. The pressures are considered separately in each direction as shown below.



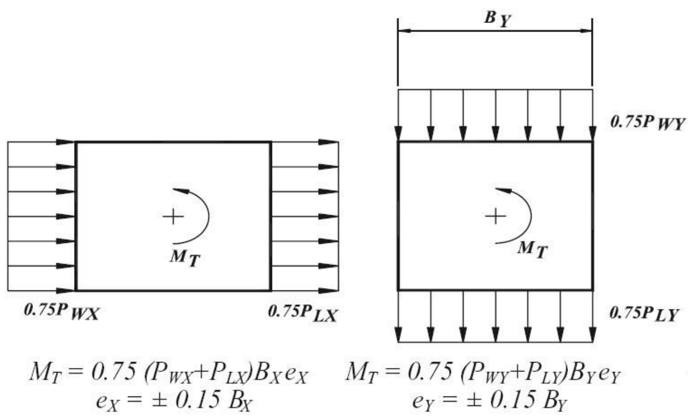
(Courtesy of: ASCE)

	Wall A	Wall B	Wall C	Wall D	Wall 1	Wall 2	Wall 3	Wall 4
Case 1	145.4	223.0	165.1	170.36	521.6	524.9	560.1	481.8

(All values are shown in kips using unfactored loads)

Wind Case2:

Wind Case 2 considers three fourths the design wind pressure acting perpendicular to the building with a torsional moment considered for each principal axis.



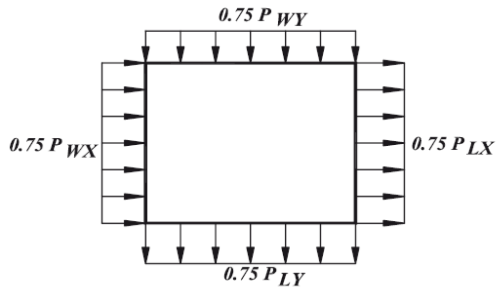
(Courtesy of: ASCE)

	Wall A	Wall B	Wall C	Wall D	Wall 1	Wall 2	Wall 3	Wall 4
Case 2 (+M)	109.3	168.1	124.9	127.5	404.1	404	404.4	329
Case 2 (-M)	109	167.5	126.4	128	358.3	377.1	415.8	379.2

(All values are shown in kips using unfactored loads)

Wind Case 3:

Wind Case 3 considers three fourths of the design wind pressure acting perpendicular to the building in both directions simultaneously.



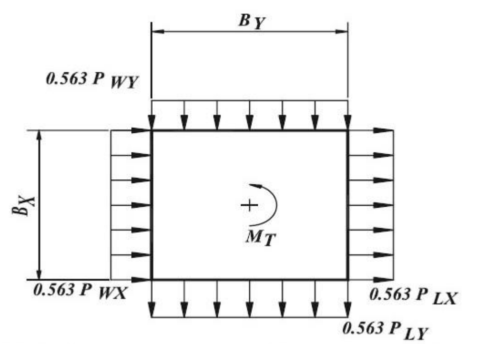
(Courtesy of: ASCE)

	Wall A	Wall B	Wall C	Wall D	Wall 1	Wall 2	Wall 3	Wall 4
Case 3	121.7	187.3	85.8	136.6	363.5	338.3	430	395.4

(All values are shown in kips using unfactored loads)

Wind Case 4:

Wind Case 4 considers combines cases 2 and 3 but considers 56.3% of the design wind pressure.



$$M_T = 0.563\ (P_{WX}+P_{LX})B_X e_X + 0.563\ (P_{WY}+P_{LY})B_Y e_Y$$
$$e_X = \pm\ 0.15\ B_X \qquad e_Y = \pm\ 0.15\ B_Y$$

(Courtesy of: ASCE)

	Wall A	Wall B	Wall C	Wall D	Wall 1	Wall 2	Wall 3	Wall 4
Case 4 (+M)	92.8	143	62	100.8	292	262.3	318	274.2
Case 4 (-M)	89.8	138.6	66.8	104.2	253.7	245.6	331.6	319.5

(All values are shown in kips using unfactored loads)

After analyzing each wind case, the force in each wall was compared to determine which case caused the largest shear force. Table 7 combines the results for each load case below. From the wind analysis of shear forces, it was determined that Case 1 for both directions controlled in all the shear walls. These forces will later be compared with the shear forces caused by seismic forces. From this comparison, the reinforcement can be designed and detailed for each shear wall.

	Wall A	Wall B	Wall C	Wall D	Wall 1	Wall 2	Wall 3	Wall 4
Case 1	145.4	223.0	165.1	170.36	521.6	524.9	560.1	481.8
Case 2 (+M)	109.3	168.1	124.9	127.5	404.1	404	404.4	329
Case 2 (-M)	109	167.5	126.4	128	358.3	377.1	415.8	379.2
Case 3	121.7	187.3	85.8	136.6	363.5	338.3	430	395.4
Case 4 (+M)	92.8	143	62	100.8	292	262.3	318	274.2
Case 4 (-M)	89.8	138.6	66.8	104.2	253.7	245.6	331.6	319.5
Max Shear (kips)	145.4	223.0	165.1	170.36	521.6	524.9	560.1	481.8

Table 7 - Summary of Shear Wall Forces

Wind Serviceability Check

From the ETABS model, story displacements were tabulated to determine if the design met serviceability requirements. A summary of the story drifts can be found in Table 8. The total displacement was calculated from the X and Y displacements as the resultant between the two directions. Story drifts were taken as the change in displacement from floor to floor. This was compared to the drift limit which was limited to H/400 for each story. This drift limit is mainly an assumed standard rather than a code requirement. The function is to prevent excessive sway which may cause discomfort for the occupants of the building. It was determined that all levels were within the drift limits.

Level	Height	X-Disp.	Y-Disp.	Total Disp.	Story Drift	Drift Limit
	ft	*in*	*in*	*in*	*in*	*in*
Roof	22	0.62	0.60	0.86	0.20	0.66
Penthouse	15	0.47	0.45	0.65	0.14	0.45
6	15	0.38	0.35	0.52	0.14	0.45
5	15	0.28	0.26	0.38	0.13	0.45
4	16.5	0.18	0.17	0.25	0.11	0.49
3	15	0.10	0.09	0.13	0.09	0.45
2	15	0.03	0.03	0.04	0.04	0.45

Table 8 - Wind Serviceability Check

Seismic Loading

The seismic analysis was performed using ASCE 7-10 for seismic design criteria. The Equivalent Lateral Force Analysis procedure was used for the seismic calculations. This method involves first calculating the base shear and then distributing it to each floor. To determine the base shear for the structure, the total weight for all floors above grade was calculated and can be found in Appendix C. The total weight of the redesigned structure was determined to be around 37944 kips. The base shear was calculated by finding the seismic response coefficient and multiplying that by the weight of the structure. These calculations can be found in Appendix C. The seismic response coefficient C_s was determined to be 2.2% which is lower than the original steel frame design which was 4.6%. The reason for this can be attributed to the increase in R value from 3 to 5 for switching from a moment frame and concentric braced system to an ordinary concrete reinforced shear wall system. Table 8 shows the calculated seismic lateral forces which were applied to the ETABS model for each level.

Level	Height h_x	Story Weight w_x	w_x*h_x^k	C_{vx}	Lateral Force F_i	Story Shear V_x	Moment
	ft	*kips*	*kip-ft*		*kips*	*kips*	*kip-ft*
Roof	113.5	4359.1	2472243.7	0.254	215.8	215.8	24498.7
Penthouse	91.5	5515.6	2343673.1	0.241	204.6	420.5	18723.0
6	76.5	5432.36	1815912.3	0.187	158.5	579.0	12128.7
5	61.5	5432.95	1355590.0	0.139	118.4	697.4	7278.8
4	46.5	5739.4	984585.5	0.101	86.0	783.3	3997.3
3	30	5740.8	547412.7	0.056	47.8	831.1	1433.8
2	15	5724.1	215610.3	0.022	18.8	849.95	282.4
Total		**37944.31**	**9735027.7**		**849.95**	**849.95**	**68342.6**

Table 9 - Seismic Force, Shear and Overturning Moment

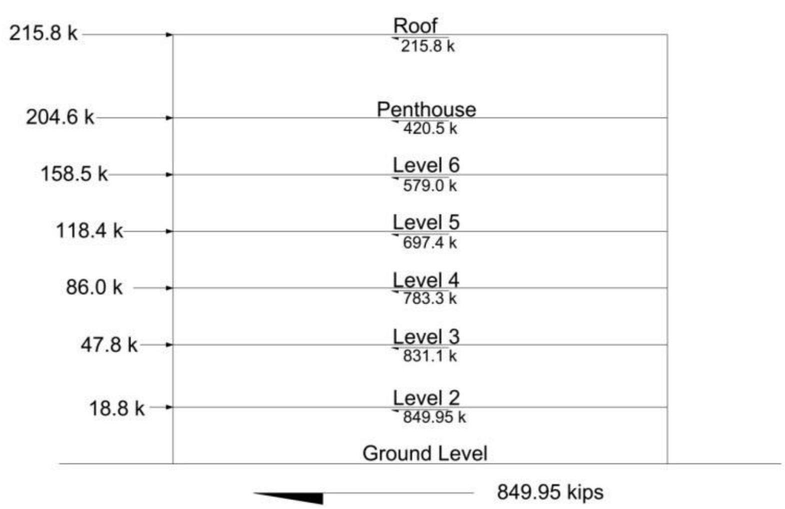


Figure 21 - Seismic Forces and Shear

With the seismic forces calculated, it was necessary to determine the load combination that would generate the maximum shear force in each wall. According to ASCE 7-10 Section 12.5.3 (a), "the requirement for considering the orthogonal combination is deemed satisfied if members and their foundations are designed for 100% of the forces for one direction plus 30% of the forces for the perpendicular direction." Using this criterion, the following seismic cases were considered in the ETABS model:

Case 1: 30%Ex + 100%Ey

Case 2: 30%Ex - 100%Ey

Case 3: 100%Ex + 30%Ex

Case 4: 100%Ey – 30%Ex

These combinations take into account the effect of forces acting on the system simultaneously in both directions. From these load combinations, the maximum shear force was determined for each was due to seismic loading. Table 10 shows the results for each wall due to each case. It was determined that Case 3 controlled for Walls A to D while Case 1 controlled for Walls 1 to 4.

	Wall A	Wall B	Wall C	Wall D	Wall 1	Wall 2	Wall 3	Wall 4
Case 1	60.2	92.5	44.7	71.3	171.1	174.8	245.7	248
Case 2	52.3	79.4	79.5	54.4	165.3	211.4	226.8	231.7
Case 3	188.8	292.5	201.8	212.1	61.2	1.47	96.6	99.2
Case 4	186.3	285.7	209.1	207	41.3	118.7	46.1	44.7
Max Shear (kips)	188.8	292.5	209.1	212.1	171.1	211.4	245.7	248

Table 10 - Shear Wall Forces Due to Seismic Loads

Seismic Drifts

According to ASCE 7-10 Table 12.12-1 for allowable story drifts, the drift limit based on the structure type was determined to be $0.02h_{sx}$, where h_{sx} is the story height below level x. The resulting displacements were tabulated from the ETABS model for the controlling load cases. The resultant displacement was calculated and story drifts determined from floor to floor. Table 11 shows the results of the seismic drifts. It was determined that the structure was within the drift limits for each level.

Level	Height	X-Disp.	Y-Disp.	Total Disp.	Story Drift	Drift Limit
	ft	*in*	*in*	*in*	*in*	*in*
Roof	22	1.2533	0.1996	1.27	0.31	0.44
Penthouse	15	0.9453	0.1483	0.96	0.21	0.3
6	15	0.7373	0.1141	0.75	0.21	0.3
5	15	0.533	0.0811	0.54	0.19	0.3
4	16.5	0.343	0.0515	0.35	0.17	0.33
3	15	0.1796	0.0265	0.18	0.13	0.3
2	15	0.0517	0.0075	0.05	0.05	0.3

Table 11 - Seismic Drifts

Results and Discussion

Through the wind and seismic analysis, the shear force experience in each wall was compared under the controlling load cases. The following table summarizes the shear forces in each wall. From the results, shear walls A through D in-plane with the East-West direction were found to control under seismic loads. Shear walls 1 through 4 in-plane with the North-South direction were controlled under wind loads.

	Wall A	Wall B	Wall C	Wall D	Wall 1	Wall 2	Wall 3	Wall 4
Wind	145.4	223.0	165.1	170.36	521.6	524.9	560.1	481.8
Seismic	188.8	292.5	209.1	212.1	171.1	211.4	245.7	248
Controlling	188.8	292.5	209.1	212.1	521.6	524.9	560.1	481.8

Table 12 - Overall Controlling Load Case

The importance of this comparison is the determinations of the overall controlling shear force that could be experience in each wall. From this comparison, the shear walls can be designed and detailed further to verify that each wall is satisfied under strength design.

Shear Wall Design

From the results of the overall controlling load cases, the cross section and reinforcement can be checked and designed for each shear wall. It was assumed that the width of each shear wall was 16" as used in the ETABS model for the lateral load analysis. The reinforcement for the shear walls were designed according to ACI Code Section 11.9 – "Provision for Walls." The capacity of the concrete section was calculated to include minimum vertical and horizontal reinforcement. For minimum reinforcement throughout the shear wall, #5 reinforcing bars were selected to satisfy the minimum reinforcement ratio. To satisfy flexural requirements of the shear wall, the reinforcement at the ends of the shear wall were upsized as necessary.

Hand calculations can be found in Appendix G for shear wall design. From the lateral analysis, it was determined that Wall 3 experiences the greatest shear force. This wall was selected to be designed by hand to show the maximum reinforcement necessary to resist lateral forces. This process should be repeated for other wall sections with lesser shear forces to optimize each shear wall design. The horizontal reinforcement was determined to be (2) #5 bars spaced at 12" on center. The minimum vertical reinforcement was also (2) #5 bars spaced at 12" on center. To provide sufficient reinforcement for flexure, (8) #9 bars spaced at 2" on center for the tension and compression zones.

Depth Summary

The goals of this depth study were to investigate the redesign of the Children's Hospital using reinforced concrete members. From this report the appropriate sizes of all structural elements were calculated under the applicable gravity and lateral loads. The second part of the proposal was to include the additional two floors in the structural design to simulate the future expansion of the Children's Hospital.

This goal was met through the use of computer aided design tools and hand calculations. As a result it was determined that a 9" flat-slab floor system utilizing 5000 psi reinforced concrete would be adequate for the floor design. The penthouse level would be modified to an 11" flat-slab with 6000 psi concrete. Shear caps with a depth of 4.5" would be necessary around each column face. The columns for the bottom two levels were designed using 24" x 24" square columns. The columns on floors three and four were designed using 20"x20" square columns. The fifth story, penthouse level, and roof were then designed using 18"x18" columns. The primary lateral resisting system was designed using 16" shear wall. From this depth study, it would be possible to perform a cost analysis between the existing structural design with the proposed concrete system. This would allow for a better comparison between the feasibility of the proposed design.

Construction Management Breadth

The alternative design utilizing a concrete system would have a significant impact on the overall cost of the project. To quantify this impact, a detailed cost estimate was constructed for the structural elements in both the existing and proposed framing systems. In addition a simplified construction schedule was developed to compare the estimated time of completion for the structure for both designs. As a result of this study, a more in depth comparison can be drawn toward the feasibility of the proposed design compared with the existing design.

Cost Estimate

A rough estimate for both the steel and concrete structures was compiled using CostWorks®, an online version of the catalogue data provided through R.S. Means. Since the foundation was not included as part of the structural redesign, only above grade work was considered in the cost breakdown. For the existing structure, only steel framing members, metal decking, concrete for the slabs, and finishing for the floor were considered in the cost analysis. Both the total cost and total cost with overhead and profit (O&P) were reported and can be found in Appendix H.

The cost of the redesigned structure includes concrete for all cast-in-place members, formwork, reinforcement, and finishing for the floors. These quantities were taken into account with CostWorks® to develop the project cost for the proposed structure. The redesigned concrete structure has two more floors to account for the future expansion of the Children's Hospital. Therefore, an estimate was performed for both an equivalent five story concrete structure to compare with the existing structure cost. The cost for all seven stories of the proposed structure was also tabulated. A breakdown of all materials costs can be found in Appendix H for all three considerations.

Through this analysis, it was determined that the equivalent five story concrete structure was $631,000 more expensive than the existing steel structure. When considering overhead and profit, the difference was $1.91 million compared with the existing design. With the addition of two stories, the overall project total cost was determined to be $8.14 million without overhead and profit. Table 13 shows the cost comparison between the different structural designs.

		Total	Total with O & P
Existing Structure	**5 Stories**	$5,468,247.73	$6,326,951.99
Equivalent Concrete Structure	**5 Stories**	$6,099,261.80	$8,241,833.34
Final Concrete Structure	**7 Stories**	$8,137,696.81	$11,008,342.66

Table 13 - Cost Estimates

Project Schedule

The alterations of the structural design were found to have a significant impact on the completion time of the project. Since the tasks associated with the concrete structure have different daily outputs from the steel structure, schedules were compared between both designs. The advantage of having a steel designed structure is construction on the adjacent floor can begin directly after the erection of the previous floor. The composite decking can be poured while other tasks are being performed. In comparison, a concrete designed structure must allow time for the concrete to develop significant strength before construction of the adjacent floor can begin.

To develop the schedules, the quantity for each component was divided by the daily output for one crew as cited by RS Means. Since the daily output can vary greatly between tasks, a various number of crews were assumed based on the task to produce a reasonable time frame. For the proposed design, it was assumed that a seven day lag time would be necessary to allow enough concrete strength to develop before framing could begin for the floor above. The existing structure schedule can be found in Appendix I while the proposed redesign schedule can be found in Appendix J

Construction for both designs was scheduled to start in August 2010. It was estimated that construction would take 155 days for the erection of the steel framing and placement of the concrete floors for the existing structure. In comparison, the projected time for completion of the proposed structure was estimated to be 289 days. To compare the redesigned structure with the existing structure, it would take about 212 days. This is an increase of about 2 months to construct an equivalent structure using reinforced concrete. These times are simulated for the ideal construction process. It is understood that a high amount of variability is involved along with coordination of various trades. Unforeseen issues can most certainly be expected, increasing the completion time for both projects.

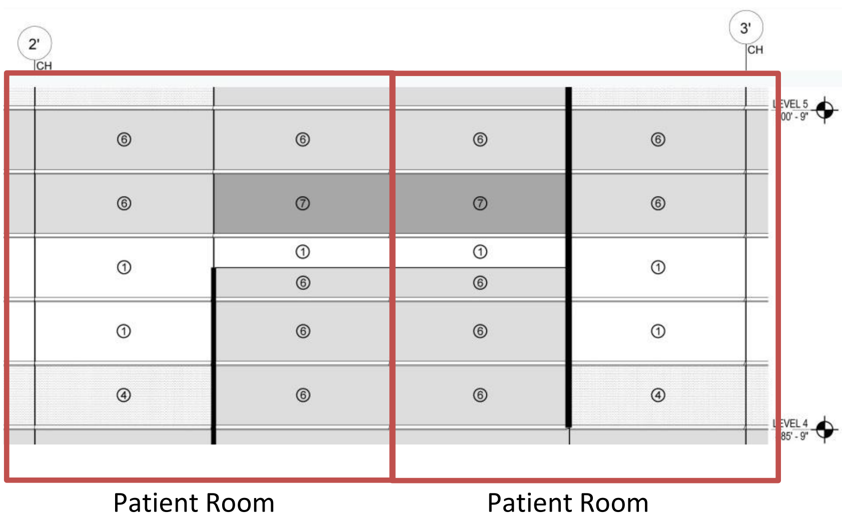
Conclusions

As a result of this study, it was determined that the existing structure was $559,000 less expensive to construct. It is important to note that these numbers are a rough estimate that was used as a gauge in determining the cost of the redesigned structure. Costs were also an average from RS Means to price the system and are unrelated to regional material costs and labor costs based on the project location. Therefore, converting to a concrete structure would be possible in terms the estimated cost.

In terms of the schedule it was determined that the existing project would be completed earlier than the redesigned structure. The project schedule would be extended by about four and a half months to allow for constructability and for the curing of concrete. Since it is unknown whether there were any particular time constraints for the project, the additional time needed for completion may not be a great concern to the owner. If time constraints were an issue then the existing steel design was certainly the preferred method of construction.

Building Enclosure Breadth

The building enclosure surrounding much of the structure consists of 3" insulated metal panels. This changes for levels 3 and 4 around the front entrance to the building. For an architectural effect, a curtain wall system composed of clear vision and warm grey spandrel insulating glass units was used. This system is located on the North facing elevation, where direct solar effects will not affect the occupied space as it would on the South elevation. With this in mind a heat transfer analysis was selected to determine the flow rate through the enclosure between the outside environment and the conditioned space. Based on these results an alternative design was investigated to attempt to reduce the heat transfer through the system. From here a cost analysis was conducted to evaluate the potential savings of the proposed building enclosure.



Curtain Wall Panel Schedule		
Type Mark	Type Comments	Description
	IG-#4	WHITE FRIT VISION INSULATING GLASS
	IG-#5	BEIGE FRIT VISION INSULATING GLASS
1	IG-#1	CLEAR VISION INSULATING GLASS
2	IG-#2	BLUE TINTED VISION INSULATING GLASS
3	IG-#3	GREEN TINTED VISION INSULATING GLASS
4	IG-#4	WHITE FRIT VISION INSULATING GLASS
5	IG-#5	BEIGE FRIT VISION INSULATING GLASS
6	IG-#6	WARM GRAY SPANDREL INSULATING GLASS
7	IG-#7	SUBDUED GRAY SPANDREL INSULATING GLASS
9	IG-#9	BLUE TINTED SPANDREL INSULATING GLASS
10	IG-#10	GREEN TINTED SPANDREL INSULATING GLASS
11	IG-#11	CLEAR LOW IRON STRUCTURAL INSULATING GLASS UNIT

Figure 22 - Curtain Wall Elevation

Existing and Proposed Building Enclosure Designs

The existing curtain wall façade with the panel schedule can be seen in Figure 23. Between the two floors there are 24 patient rooms with the same curtain wall design. This analysis will take into account the effects of heat transfer for one patient room, which will be extrapolated for all other patient rooms. The design of the existing curtain wall system was specified as being an "Oldcastle PPG Solarban 60" insulating glass unit (IGU) product. From the manufacturer's website, specifications and performance characteristics were obtained for the vision IGU and spandrel IGU. These specifications can be seen in Appendix K for both the vision glass and spandrel glass.

For the proposed building enclosure, the vision glass will be kept the same while the spandrel glass will be modified to utilize a "shadow box" design. The "shadow box" consists of a monolithic clear glass layer and 2" rigid insulation. These components are separated by a 2" air cavity to prevent contaminants and moisture from damaging the insulation. Within the trim cover, setting blocks and weep holes will allow any penetrating water to be directed toward the exterior to prevent damage to units below. Thermal breaks and pressure bars will help to separate the exterior environment from the air cavity. Figure 24 shows a simplified diagram of the "shadow box" design for one panel.

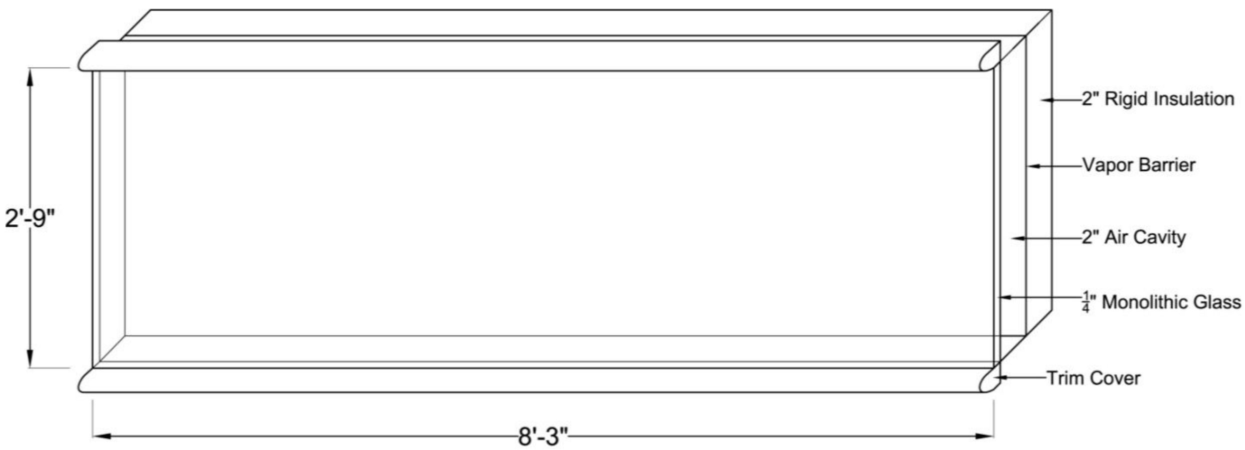


Figure 23 - Proposed Building Enclosure

Since temperature effects for Pennsylvania can vary greatly between seasons, climate conditions were assumed for Hershey, PA. The following table shows the established temperatures based on the location for both summer and winter seasons. With these assumptions, a temperature gradient was established between the indoor and outdoor environments. For the summer condition it was assumed that there were clear skies and an average wind velocity of 6.28 MPH. Additionally, the temperature for the surface of the façade was estimated to be elevated 30°F above the temperature of the surrounding air. Similarly for the winter condition, skies were assumed to be cloudy with an average wind velocity of 11.83 MPH. Due to winter conditions, the temperature of the surface was taken to be equal to the temperature of the air. All calculations for determining the heat flow rate can be found in Appendix K.

	Winter		Summer	
	Temp (°F)	**RH %**	**Temp (°F)**	**RH %**
Indoor	70	25	75	50
Outdoor	9	79	91	50

Table 14 - Climate Conditions for Hershey, PA

Results and Findings

It was determined that the proposed design of exchanging the spandrel glass sections with a "shadow box" design decreases the heat flow rate through the enclosure. The results are shows in Table 15 for both the summer and winter season. It was determined that the amount of heat flow during the summer was decreased slightly. During the winter season there was a greater decrease in heat flow for the proposed enclosure. As a result, by decreasing the heat flow through the system, the load demand on the building heating and cooling systems could be decreased.

	Winter Season (BTU/hr)	Summer Season (BTU/hr)
Existing Enclosure	32488.6	23626.6
Redesigned Enclosure	21411.4	20099.5
Difference	11077.2	3527.1

Table 15 - Heat Flow Rate Comparison

This difference in heat flow was then quantified into an average energy savings cost per year. From the Department of Energy, it was assumed that the average cost of electricity in Pennsylvania was 10.1¢/kW*hr. Assuming that both winter and summer conditions would be prevalent about half a year each, the annual energy savings for one patient room was $6460.65. Therefore, for all 24 patient rooms with this type of building enclosure, the total savings would be $155,055.60 per year.

It is important to note that this cost analysis is purely based on the heat flow rate. It would also be necessary to factor in the preliminary manufacturing costs to produce and install the "shadow box" design compared with the costs to produce and install the spandrel insulating glass units. Experimental mock up designs of each curtain wall system would need to be constructed to determine the structural integrity of each design. Unforeseen issues may arise during testing which would add to the cost of detailing each unit. Periodical maintenance during the life cycle of both curtain walls would also need to be addressed. Once all these factors can be compared, a more realistic image of the feasibility of the proposed enclosure can be concluded. Therefore, from an isolated energy cost analysis, it was concluded that the proposed building enclosure for the partial North elevation would produce a substantial decrease in heat transfer.

Conclusions

The overall focus of this report was to redesign the structure using reinforced concrete and determine if the alternative design is feasible in terms of overall design and cost. Since the existing design includes the capability of two additional floors, the proposed design was analyzed with the consideration of two additional floors. It would then be capable to quantify the structural cost of the existing and proposed systems.

The main goal for the depth of the proposal was met through the design of the reinforced concrete structure. It was determined that a 9" flat-slab floor system utilizing 5000 psi reinforced concrete would be adequate for the floor design. Shear caps with a depth of 4.5" would be necessary around each column face. The columns for all levels were designed using 24" x 24", 20"x20", and 18"x18" square columns. Shear walls would provide sufficient lateral reinforcement for the lateral forces experienced by the expanded structure.

The effects of these changes were then quantified by performing a cost analysis for the Construction Management breadth. It was determined that the proposed design was slightly more expensive than the existing structure when taking into account five stories of the proposed design. With the additional two floors, the total project cost when considering only labor, materials, and equipment was determined to be $8,137,696.81. The proposed schedule also shows that the project length would increase to 213 days for the completion of the structural elements. It is therefore concluded that using the proposed reinforced concrete system would be feasible. The selection of using structural steel by the design team is unconfirmed. Other constraining factors such as time frames and proposed budgets at the time may have influenced the selection of the five story steel design rather than a 7 story reinforced concrete design.

The curtain wall on the north elevation was also redesigned as part of the building enclosure breadth. The existing curtain wall system consists of vision and spandrel insulating glass units. The heat flow rate was calculated to determine the energy transmitted through the system. An alternative "shadow box" design was proposed which consists of a monolithic glass unit, a 2" air cavity, and 2" rigid insulation. The estimated heat flow rate was determined to be less than the heat flow rate through the existing system. This difference in heat flow was quantified into energy savings of $155,055.60 for the entire curtain wall section. Please note that these savings only reflect the heat transfer analysis. Other factors such as manufacturing costs, structural integrity through testing, and the cost due to building life maintenance must be taken into account.

References

American Concrete Institute, *Building Code Requirements for Structural Concrete* (ACI 318-08)

American Society of Civil Engineers, *Minimum Design Loads for Buildings and Other Structures* (ASCE 7-10)

International Building Code (IBC), 2006

MacGregor, James and James Wright. Reinforced Concrete: Mechanics and Design, *fifth ed.* Prentice Hall. 2009

RS Means Construction Publishers and Consultants, Building Construction Cost Data 2008

APPENDIX

Appendix A: Wind Calculations

General Requirements	
Occupancy Category	IV
Exposure Category	C
V (MPH)	120
K_d	0.85
K_{zt}	1.0
Enclosure Classification	Enclosed

Gust Effect Factor	N-S	E-W
B (ft)	359.1	124.25
L (ft)	124.25	359.1
h (ft)	115.5	115.5
n_1	0.65	0.65
β (assumed 1%)	0.01	0.01
Structure (η_1 < 1 Hz)	Flexible	Flexible
g_Q	3.4	3.4
g_v	3.4	3.4
g_R	4.09	4.09
z	69.3	69.3
L_z	579.98	579.98
I_z	0.177	0.177
Q	0.802	0.857
V_z	128.23	128.23
N_1	2.94	2.94
R_n	0.071	0.071
η for R_h	2.69	2.69
R_h	0.303	0.303
η for R_B	8.37	2.90
R_B	0.11	0.29
η for R_L	9.70	28.03
R_L	0.098	0.035
R	0.372	0.578
G_f	**0.887**	**0.973**

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GENERAL REQUIREMENTS

OCCUPANCY CATEGORY: IV
BASIC WIND SPEED (V): 120 MPH
WIND DIRECTIONALITY FACTOR (K_d): 0.85
EXPOSURE CATEGORY: C
TOPOGRAPHIC FACTOR (K_{zt}): 1.0
GUST FACTOR:

FOR N/S WIND DIRECTION — B = 359.1 FT, L = 124.25 FT, H = 115.5 FT
FOR E/W WIND DIRECTION — B = 124.25 FT, L = 359.1 FT, H = 115.5 FT

ASCE 7-10 – 26.9.2.1 — LIMITATIONS FOR APPROXIMATE NATURAL FREQUENCY
① BUILDING HEIGHT = 115.5 FT < 300 FT ∴ OK
② BUILDING HEIGHT = 115.5 FT < 4 L_{eff}

$$L_{eff} = \frac{\Sigma h_i L_i}{\Sigma h_i}$$ WHERE h_i = HEIGHT ABOVE GROUND LEVEL i
L_i = BUILDING LENGTH AT LEVEL i PARALLEL TO WIND

$$(N/S)\ L_{eff} = \frac{124.5\,(15+31.5+46.5+61.5+76.5+91.5+115.5)}{124.5} = 438\ \text{FT}$$

115.5 FT < 4(438 FT) = 1752 FT ∴ OK

$(E/W)\ L_{eff}$ = 438 FT

115.5 FT < 4(438) ∴ OK

n_a SHALL BE CALCULATED BY 26.9.3

ASCE 7-10 – 26.9.3 — APPROXIMATE NATURAL FREQUENCY
FOR CONCRETE BUILDINGS W/ OTHER LATERAL-FORCE-RESISTING SYSTEMS:

$$n_a = 75/h = 75/115.5\ \text{FT} = 0.65\ \text{Hz} < 1.0\ \text{Hz}$$ ∴ FLEXIBLE

ASCE 7-10 – 26.9.5 — FLEXIBLE BUILDINGS

$$G_f = 0.925\left(\frac{1+1.7 I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1+1.7 g_v I_{\bar{z}}}\right)$$

SEE FOLLOWING PAGES FOR ALL VALUES
(N/S) G_f = 0.838
(E/W) G_f = 0.973

ENCLOSURE CLASSIFICATION: ENCLOSED
INTERNAL PRESSURE COEFFICIENT (GC_{pi}) = ±0.18
VELOCITY PRESSURE EXPOSURE COEFFICIENTS (K_z OR K_h): SEE FOLLOWING PAGES
VELOCITY PRESSURE (q_z OR q_h); $q_z = 0.00256\,K_z K_{zt} K_d V^2$ (PSF)

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EXTERNAL PRESSURE COEFFICIENT (C_p OR C_N):

(N/S)

SURFACE	L/B	C_P	USE WITH
WINDWARD	ALL	0.8	q_z
LEEWARD	0.35	−0.5	q_h
SIDE	ALL	−0.7	q_h

(E/W)

SURFACE	L/B	C_P	USE WITH
WINDWARD	ALL	0.8	q_z
LEEWARD	2.89	−0.26	q_h
SIDE	ALL	−0.7	q_h

WIND PRESSURE FOR ENCLOSED FLEXIBLE BUILDINGS:

$p = q G_f C_p - q_i (GC_{pi})$ (PSF)

FOR PARAPET — $P_p = q_p (GC_{pn})$ WHERE $GC_{pn} = +0.15$, -1.0

$q_p = 0.00256(1.23)(1.0)(0.85)(120)^2 = 38.54$

WINDWARD: $P_p = (38.54)(1.5) = 57.81$ PSF

LEEWARD: $P_p = (38.54)(-1.0) = -38.54$ PSF

DESIGN WIND PRESSURES:

(N/S) WINDWARD: $P = q_z(0.838)(0.8) - 38.54(\pm 0.18)$

$= 0.6704\, q_z + 6.94$

LEEWARD: $P = (38.54)(0.838)(-0.5) - (38.54)(\pm 0.18)$

$= -23.09$ PSF

(E/W) WINDWARD: $P = q_z(0.973)(0.8) - (38.54)(\pm 0.18)$

$= 0.778\, q_z + 6.94$

LEEWARD: $p = (38.54)(0.973)(-0.26) - (38.54)(\pm 0.18)$

$= -16.69$ PSF

Wind Pressure East/West Direction

	Level	Height (ft)	K_z	q_z	p_z
Windward	T.O. Parapet	115.5	1.3	40.73	57.81
	Roof	113.5	1.29	40.42	38.39
	Penthouse	91.5	1.24	38.85	37.17
	6	76.5	1.19	37.29	35.95
	5	61.5	1.14	35.72	34.73
	4	46.5	1.07	33.53	33.02
	3	30	0.99	31.02	31.07
	2	15	0.85	26.63	27.66
	Ground	0	0.85	26.63	27.66
Leeward	T.O. Parapet	115.5	1.3	40.73	38.54
	Ground to Roof	113.5	1.29	40.42	16.69

Wind Pressure North/South Direction

	Level	Height (ft)	K_z	q_z	p_z
Windward	T.O. Parapet	115.5	1.3	40.73	57.81
	Roof	113.5	1.29	40.42	34.04
	Penthouse	91.5	1.24	38.85	32.99
	6	76.5	1.19	37.29	31.94
	5	61.5	1.14	35.72	30.89
	4	46.5	1.07	33.53	29.42
	3	30	0.99	31.02	27.74
	2	15	0.85	26.63	24.80
	Ground	0	0.85	26.63	24.80
Leeward	T.O. Parapet	115.5	1.3	40.73	38.54
	Ground to Roof	113.5	1.29	40.42	23.09

Appendix B: Story Weights

Floor	Component	Weight	Loading	Thickness	Length	Perimeter	Area	Total Weight	Story Weight
		pcf	*psf*	*in.*	*ft*	*ft*	*ft* ^{2}	*lbs*	*kips*
2nd Floor	Slab	150		11			36662.6	5041107.5	
2nd Floor	Drop Panels	150		11			2320	319000	
2nd Floor	Columns Below	150			7.5		4	4500	5724.1
2nd Floor	Columns Above	150			7.5		4	4500	
2nd Floor	Façade		25		15	946.6		354975	
3rd Floor	Slab	150		11			36662.6	5041107.5	
3rd Floor	Drop Panels	150		11			2320	319000	
3rd Floor	Columns Below	150			7.5		4	4500	5740.8
3rd Floor	Columns Above	150			8.25		2.78	3440.25	
3rd Floor	Façade		25		15.75	946.6		372723.75	
4th Floor	Slab	150		11			36662.6	5041107.5	
4th Floor	Drop Panels	150		11			2320	319000	
4th Floor	Columns Below	150			8.25		2.78	3440.25	5739.4
4th Floor	Columns Above	150			7.5		2.78	3127.5	
4th Floor	Façade		25		15.75	946.6		372723.75	
5th Floor	Slab	150		11			34569.6	4753320	
5th Floor	Drop Panels	150		11			2320	319000	
5th Floor	Columns Below	150			7.5		2.78	3127.5	5432.95375
5th Floor	Columns Above	150			7.5		2.25	2531.25	
5th Floor	Façade		25		15	946.6		354975	
6th Floor	Slab	150		11			34569.6	4753320	
6th Floor	Drop Panels	150		11			2320	319000	
6th Floor	Columns Below	150			7.5		2.25	2531.25	5432.3575
6th Floor	Columns Above	150			7.5		2.25	2531.25	
6th Floor	Façade		25		15	946.6		354975	
Penthouse	Slab	150		11			34569.6	4753320	
Penthouse	Drop Panels	150		11			2320	319000	
Penthouse	Columns Below	150			7.5		2.25	2531.25	5515.6
Penthouse	Columns Above	150			11		1.78	2937	
Penthouse	Façade		25		18.5	946.6		437802.5	
Roof	Slab	150		9			36151.6	4067055	
Roof	Drop Panels	150		9			256	28800	
Roof	Columns Below	150			11		1.78	2937	4359.1
Roof	Columns Above	150			0		0	0	
Roof	Façade		25		11	946.6		260315	

Appendix C: Seismic Calculations

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SEISMIC USE GROUP: IV
SITE CLASS: D

SPECTRAL RESPONSE ACCEL. SHORT (S_S): 0.207g (USGS)
SPECTRAL RESPONSE ACCEL. LONG (S_1): 0.055g (USGS)
SITE COEFFICIENT (F_a): 1.6
SITE COEFFICIENT (F_v): 2.4

SOIL MODIFIED ACCEL (S_{MS}) $= F_a S_S = (1.6)(0.207) = 0.3312$ g
SOIL MODIFIED ACCEL (S_{M1}) $= F_v S_1 = (2.4)(0.055) = 0.132$ g
DESIGN SPECTRAL RESPONSE SHORT (S_{DS}) $= 2/3\, S_{MS} = 2/3(0.3312) = 0.221$ g
DESIGN SPECTRAL RESPONSE 1 SEC (S_{D1}) $= 2/3\, S_{M1} = 2/3(0.132) = 0.088$ g

RESPONSE MODIFICATION FACTOR (R) = 5 (TABLE 12.2-1: ORDINARY REINFORCED CONCRETE SHEAR WALLS)

IMPORTANCE FACTOR (I_e) = 1.50
SEISMIC DESIGN CATEGORY: C

FOR RISK CATEGORY IV:
$0.167 < S_{DS} < 0.33 \longrightarrow C$
$0.067 < S_{D1} < 0.133 \longrightarrow C$

APPROXIMATE PERIOD PERAMETER: $C_T = 0.02$
$x = 0.75$

STRUCTURAL HEIGHT: 113.5 FT
APPROXIMATE FUNDAMENTAL PERIOD: $T_a = C_T h_n^x = 0.02(113.5)^{0.75} = 0.695$

COEFFICIENT FOR UPPER LIMIT ON CALCULATED PERIOD: $C_u = 1.7$ (TABLE 12.8-1)

PERIOD DETERMINATION: $T = C_u T_a = (1.7)(0.695) = 1.18$

LONG PERIOD TRANSITION PERIOD: $T_L = 6$ sec

SEISMIC RESPONSE COEFFICIENT:

$$C_s = \min \begin{cases} S_{DS}/(R/I) = 0.221/(5/1.5) = 0.0663 \\ \text{FOR } T < T_L:\ S_{D1}/T(R/I) = 0.088/1.18(5/1.5) = \boxed{0.0224} \end{cases}$$

WEIGHT OF STRUCTURE: 45439.7 K
BASE SHEAR: $V_b = C_s W = 0.0224(37944.31) = \boxed{849.95\text{ K}}$

STRUCTURAL PERIOD EXPONENT (k):
$0.5 < T = 1.18 < 2.5$ ∴ INTERPOLATE FOR k BETWEEN 1 AND 2
$k = 1.34$

Appendix D: Snow Calculations

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ASCE 7-10 : 7.3 - FLAT ROOF SNOW LOADS, P_f

$P_f = 0.7 C_e C_t I_s P_g$

WHERE: $C_e = 0.9$ FOR TERRAIN CATEGORY C AND FULLY EXPOSED ROOF

$C_t = 1.0$

$I_s = 1.2$ FOR RISK CATEGORY IV, FROM TABLE 1.5-2

$P_g = 30$ PSF FROM FIGURE 7-1

$P_f = 0.7(0.9)(1.0)(1.2)(30\text{ PSF}) = 22.68$ PSF

ASCE 7-10 : 7.8 — SNOW DRIFT LOAD ACCUMULATING AGAINST PARAPET

$h_d = 0.75\left[0.43(l_u)^{1/3}(P_g+10)^{1/4} - 1.5\right]$ WHERE $l_u = 129.3$ ft

$= 0.75\left[0.43(129.3)^{1/3}(30+10)^{1/4} - 1.5\right]$

$= 2.98\text{ ft} > h_c = 2\text{ ft}$

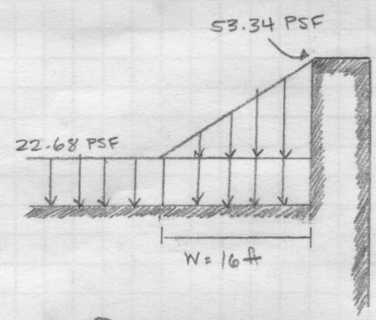
$\therefore w = 4h_d^2/h_c = 4(2.98)^2/2 = 17.8$ ft

HOWEVER, w SHALL NOT EXCEED $8h_c = 8(2) = \underline{16\text{ ft}}$ (DRIFT WIDTH)

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

MAXIMUM INTENSITY OF DRIFT SURCHARGE LOAD:

$P_d = h_d\gamma = (2.98\text{ ft})(17.9\text{ PCF}) = 53.34$ PSF



PARAPET ELEVATION

Appendix E: Slab Design

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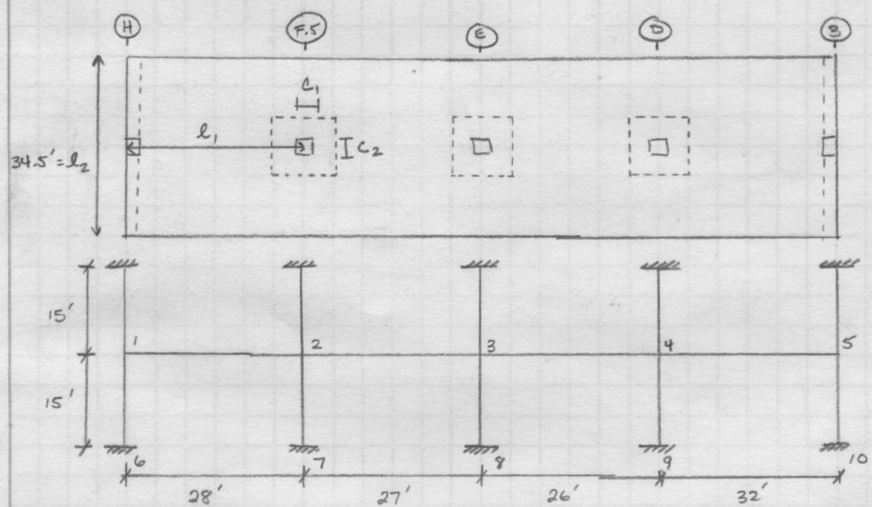
EQUIVALENT FRAME METHOD — FRAME 9



ASSUMPTIONS: $f'_c = 5000$ PSI
$t_{SLAB} = 9$ in
COLUMNS = 24" x 24"
$t_{DROP\ PANEL} = 4.5$ in

LOADING: SELF WT = $9''/12$ (150 PCF) = 112.5 PSF
SDL = 30 PSF
LL = 100 PSF
$w_u = 1.2(142.5)(34.5') + 1.6(100)(34.5)$
$= 11.4$ klf

CALCULATE MOMENT DISTRIBUTION COEFFICIENTS FOR SLAB BEAM:
SINCE DROP PANELS ARE $1.5 \times t_{SLAB}$, USE TABLE A-16 IN MACGREGOR 2009

FEM (UNIFORM LOAD w) = $M w \ell_2 \ell_1^2$
K (STIFFNESS) = $k E \ell_2 t^3 / 12 \ell_1$
CARRYOVER FACTOR = COF

1-2 $\quad \frac{c_1}{\ell_1} = \frac{24''/12}{28'} = 0.0714 \qquad \frac{c_2}{\ell_2} = \frac{24''/12}{34.5'} = 0.058$

INTERPOLATION: M = 0.09307
k = 5.9219
COF = 0.5925

2-3 $\quad \frac{c_1}{\ell_1} = \frac{24''/12}{27'} = 0.0741 \qquad \frac{c_2}{\ell_2} = 0.058$

INTERPOLATION: M = 0.09308
k = 5.9249
COF = 0.5926

3-4 $\quad \frac{c_1}{\ell_1} = \frac{24''/12}{26'} = 0.0769 \qquad \frac{c_2}{\ell_2} = 0.058$

INTERPOLATION: M = 0.09309
k = 5.9279
COF = 0.5927

4-5 $\quad \frac{c_1}{\ell_1} = \frac{24''/12}{32'} = 0.0625 \qquad \frac{c_2}{\ell_2} = 0.058$

INTERPOLATION: M = 0.09304
k = 5.9121
COF = 0.5921

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CALCULATE MOMENT DISTRIBUTION COEFFICIENTS FOR EQUIVALENT COLUMNS:

USE TABLE A-17 IN MACGREGOR 2009

$$K_c = k\frac{EI_c}{l_c}$$

CARRYOVER FACTOR = COF

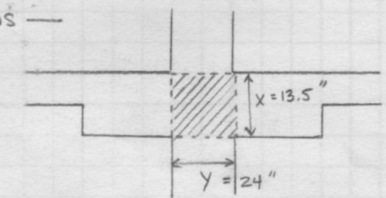
$t_a/t_b = 4.5/9.5 = 0.474$ $\qquad l_c/l_u = 1.08$

INTERPOLATION: $k = 4.66$

$COF = 0.578$

CALCULATE STIFFNESS OF TORSIONAL MEMBERS:

DROP PANELS AT INTERIOR COLUMNS —



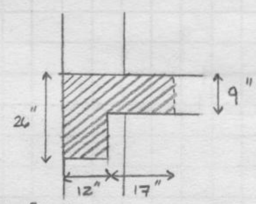
$$K_t = \sum \frac{9E_{cs}C}{l_2(1 - c_2/l_2)^3}$$

$$C = \sum\left(1 - 0.63\frac{x}{y}\right)\frac{x^3y}{3}$$

$$C = \left[1 - 0.63\left(\frac{13.5}{24}\right)\right]\frac{(13.5)^3(24)}{3} = 12707.8\ in^4$$

$$K_t = 2\left[\frac{9E_{cs}(12707.8)}{(34.5')(12)(1 - 24/34.5\times12)^3}\right] = 660.9\ E_{cs}$$

EDGE COLUMN TORSIONAL MEMBER:



$$K_t = \sum \frac{9E_{cs}C}{l_2(1 - c_2/l_2)^3}$$

$$C = \sum\left(1 - 0.63\frac{x}{y}\right)\frac{x^3y}{3}$$

$$C = \left(1 - 0.63\frac{12}{26}\right)\frac{(12)^3(26)}{3} + \left(1 - 0.63\frac{9}{17}\right)\frac{(9)^3(17)}{3}$$

$$C = 10621.44 + 2753.19 = 13374.63\ in^4$$

$$K_t = 2\left[\frac{9E_{cs}(13374.63)}{(34.5)(12)(1 - 24/34.5\times12)^3}\right] = 695.6\ E_{cs}$$

EQUIVALENT SLAB STIFFNESS:

1-2 $\qquad K = \frac{kEl_2t^3}{12l_1} = \frac{5.9219E(34.5')(12)(9in)^3}{12(28')(12)} = 443.3\ E$

2-3 $\qquad K = \frac{5.9249E(34.5')(12)(9in)^3}{12(27')(12)} = 459.9\ E$

3-4 $\qquad K = \frac{5.9279E(34.5')(12)(9in)^3}{12(26')(12)} = 477.9\ E$

4-5 $\qquad K = \frac{5.9099E(34.5')(12)(9in)^3}{12(32')(12)} = 387.1\ E$

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EQUIVALENT COLUMN STIFFNESS:

EXTERIOR

$$I_c = \frac{bh^3}{12} = \frac{(24\text{ in})^4}{12} = 27648 \text{ in}^4$$

$$K_c = \left[\frac{4.66\,(27648 \text{ in}^4)\,E}{(15')(12)}\right](2) = 1431.6E$$

$$K_t = 695.6\,E$$

$$K_{ec} = \left(\frac{1}{K_c} + \frac{1}{K_t}\right)^{-1} = \left(\frac{1}{1431.6} + \frac{1}{695.6}\right)^{-1} = 468.1\,E$$

INTERIOR

$$K_c = 1431.6\,E$$

$$K_t = 660.9\,E$$

$$K_{ec} = \left(\frac{1}{1431.6} + \frac{1}{660.9}\right)^{-1} = 452.2\,E$$

FIXED END MOMENTS:

1-2 $\quad FEM = Mw\ell_1^2 = 0.09307\,(11.4 \text{ klf})(28')^2 = 831.82 \text{ K-FT}$

2-3 $\quad FEM = 0.09308\,(11.4 \text{ klf})(27')^2 = 773.55 \text{ K-FT}$

3-4 $\quad FEM = 0.09309\,(11.4 \text{ klf})(26')^2 = 717.39 \text{ K-FT}$

4-5 $\quad FEM = 0.09304\,(11.4 \text{ klf})(32')^2 = 1086.11 \text{ K-FT}$

PERFORM MOMENT DISTRIBUTION:

SIGN CONVENTION: CLOCKWISE MOMENT ON JOINTS IS POSITIVE

COF		0.5925			0.5926			0.5927			0.5921		
	Joint 1		**Joint 2**			**Joint 3**			**Joint 4**			**Joint 5**	
SPAN	*1-6*	*1-2*	*2-1*	*2-7*	*2-3*	*3-2*	*3-8*	*3-4*	*4-3*	*4-9*	*4-5*	*5-4*	*5-10*
k	468.1	443.3	443.3	452.2	459.9	459.9	452.2	477.9	477.9	452.2	387.1	387.1	468.1
DF	0.514	0.486	0.327	0.334	0.339	0.331	0.325	0.344	0.363	0.343	0.294	0.453	0.547
FEM		831.82	-831.82		773.55	-773.55		717.39	-717.39		1086.11	-1086.11	
DIST 1	-427.23	-404.59	19.06	19.44	19.77	18.58	18.27	19.31	-133.78	-126.58	-108.36	491.62	594.49
CO		11.29	-239.72		11.01	11.72		-79.29	11.44		291.09	-64.16	
DIST 2	-5.80	-5.49	74.80	76.30	77.60	22.36	21.98	23.23	-109.76	-103.86	-88.91	29.04	35.12
CO		44.32	-3.25		13.25	45.99		-65.06	13.77		17.20	-52.64	
DIST 3	-22.76	-21.56	-3.27	-3.33	-3.39	6.31	6.20	6.56	-11.23	-10.63	-9.10	23.83	28.81
CO		-1.94	-12.77		3.74	-2.01		-6.66	3.89		14.11	-5.39	
DIST 4	0.99	0.94	2.95	3.01	3.07	2.87	2.82	2.98	-6.53	-6.18	-5.29	2.44	2.95
CO		1.75	0.56		1.70	1.82		-3.87	1.77		1.44	-3.13	
DIST 5	-0.90	-0.85	-0.74	-0.75	-0.77	0.68	0.67	0.71	-1.16	-1.10	-0.94	1.42	1.71
SUM	**-455.69**	**455.69**	**-994.20**	**94.67**	**899.53**	**-665.24**	**49.94**	**615.30**	**-948.99**	**-248.35**	**1197.35**	**-663.09**	**663.09**
SUM JT	**0.00**		**0.00**			**0.00**			**0.00**			**0.00**	

Moment Distribution (joint labels can be found on page 50)

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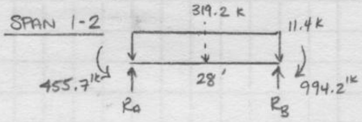
SHEAR AND MOMENT DIAGRAMS:

SPAN 1-2



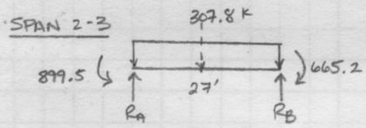
$\circlearrowright + \Sigma M_A = 994.2 + 319.2(14') - 455.7 - R_B(28')$

$R_B = 178.8\ K$

$\Sigma F_y = 0 = R_A + 178.8 - 319.2$

$R_A = 140.4\ K$

SPAN 2-3



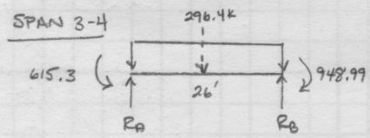
$\circlearrowright + \Sigma M_A = 665.2 + 307.8(13.5') - 899.5 - R_B(27')$

$R_B = 145.2\ K$

$\Sigma F_y = 0 = R_A + 145.2 - 307.8$

$R_A = 162.6\ K$

SPAN 3-4



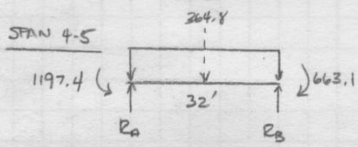
$\circlearrowright + \Sigma M_A = 948.99 + 296.4(13') - 615.3 - R_B(26')$

$R_B = 161.03\ K$

$\Sigma F_y = 0 = R_A + 161.03 - 296.4$

$R_A = 135.4\ K$

SPAN 4-5

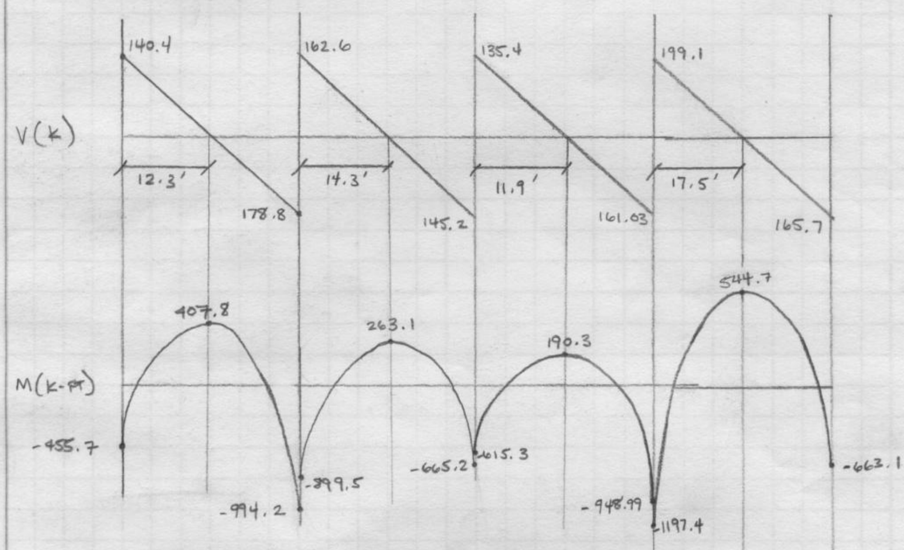


$\circlearrowright + \Sigma M_A = 663.1 + 364.8(16') - 1197.4 - R_B(32')$

$R_B = 165.7\ K$

$\Sigma F_y = 0 = R_A + 165.7 - 364.8$

$R_A = 199.1\ K$



MAX POSITIVE MOMENT: $\frac{319.2}{28'} = \frac{140.4}{x} \longrightarrow x = 12.3\ FT$

$\frac{307.8}{27'} = \frac{162.6}{x} \longrightarrow x = 14.3\ FT$

$\frac{296.43}{26'} = \frac{135.4}{x} \longrightarrow x = 11.9\ FT$

$\frac{364.8}{32'} = \frac{199.1}{x} \longrightarrow x = 17.5\ FT$

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DISTRIBUTION OF MOMENTS TO COLUMN STRIPS:

FOR INTERIOR NEGATIVE MOMENT (ACI 13.6.4.1)

$$\alpha_f \, l_2/l_1 = 0 \longrightarrow 75\% \text{ OF MOMENT}$$

FOR INTERIOR POSITIVE MOMENT (ACI 13.6.4.4)

$$\alpha_f \, l_2/l_1 = 0 \longrightarrow 60\% \text{ OF MOMENT}$$

FOR EXTERIOR NEGATIVE MOMENT (ACI 13.6.4.2)

$$\beta_t = \frac{\cancel{E_{cb}}\, C}{2\cancel{E_s}\, I_s} \qquad C = 13374.63 \text{ in}$$

$$I_{SLAB @ JT 1} = \frac{(168 \text{ in})(9 \text{ in})^3}{12} = 10206 \text{ in}^4$$

$$I_{SLAB @ JT 5} = \frac{(192 \text{ in})(9 \text{ in})^3}{12} = 11664 \text{ in}^4$$

$$\beta_{t_1} = \frac{13374.63 \text{ in}}{2(10206 \text{ in}^4)} = 0.66 \longrightarrow 93.4\% \text{ OF MOMENT}$$

$$\beta_{t_5} = \frac{13374.63}{2(11664 \text{ in}^4)} = 0.57 \longrightarrow 94.3\% \text{ OF MOMENT}$$

DESIGN REINFORCEMENT FOR STRIPS:

- SEE SPREADSHEET FOR RESULTS
- SAMPLE CALCULATION

FOR COLUMN STRIP MAX NEG. MOMENT — $M_U = -1197.4$ K-FT

% TO COLUMN STRIP: $0.75(-1197.4) = 898.05$ K-FT

$$A_{S\,REQ} = \frac{M_U}{\phi f_y \, jd} \qquad \text{USE } \phi = 0.9,\ j = 0.9,\ \text{COVER} = 1.25 \text{ in},\ f_y = 60 \text{ ksi}$$

$$jd = 0.9(9'' - 1.25'') = 6.975 \text{ in}$$

$$A_{S\,REQ.} = \frac{898.05 \text{ k-FT}\,(12)}{0.9(60 \text{ ksi})(6.975 \text{ in})}$$

$$A_{S\,REQ} = 28.6 \text{ in}^2$$

CHECK TENSION CONTROLLED:

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(28.6 \text{ in}^2)(60 \text{ ksi})}{0.85(5 \text{ ksi})(17.25 \text{ ft})(12)} = 1.95 \text{ in}$$

$$c = a/\beta = 1.95/0.85 = 2.29 \text{ in}$$

$$\varepsilon_t = \varepsilon_u\left(\frac{d_t - c}{c}\right) = 0.003\left(\frac{7.75 - 2.29}{2.29}\right) = 0.007 > 0.005$$

∴ TENSION CONTROLLED, $\phi = 0.9$

FOR A CONSTANT a:

$$jd = d - \frac{a}{2} = 7.75 - \frac{1.95}{2} = 6.78 \text{ in}$$

$$A_s = \frac{M_u(\text{k-FT})\,(12)}{0.9(60 \text{ ksi})(6.78 \text{ in})} =$$

$$A_s = 0.0328\, M_u(\text{k-FT})$$ ← USE TO DETERMINE A_s REQ.

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FOR COLUMN STRIP:

MINIMUM A_s ACCORDING TO ACI 13.3.1

$$A_{s\,min} = 0.0018\,bh = 0.0018(9")(17.25')(12) = 3.35\ in^2$$

MINIMUM SPACING:

$$s \le \min \begin{cases} 2h = 2(9") = 18" \\ 18" \end{cases} \quad \therefore \text{ USE } 18"$$

$$\text{MINIMUM \# OF BARS} = \frac{17.25'(12)}{18"} = 11.5 \rightarrow 12 \text{ BARS}$$

FOR MIDDLE STRIP:

$$A_{s\,MIN} = 0.0018\,bh = 0.0018(9")(8.625')(12) = 1.68\ in^2$$

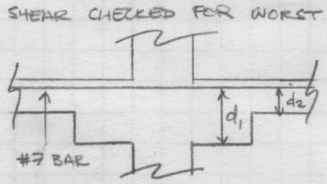
$$\text{MINIMUM \# OF BARS} = \frac{(8.625')(12)}{18"} = 5.75 \rightarrow 6 \text{ BARS}$$

SEE SPREADSHEET FOR FINAL DESIGN OF REINFORCEMENT

NOTE ALL REINFORCEMENT FOR INTERIOR NEGATIVE MOMENT AND POSITIVE MOMENT WERE DESIGNED FOR THE WORST CASE MOMENT (1197.4 K-FT AND 793.1 K-FT).

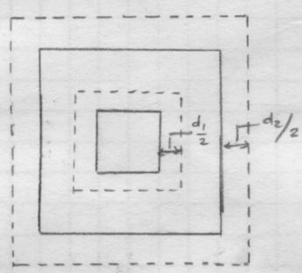
CHECK SHEAR AT COLUMN FACE:

SHEAR CHECKED FOR WORST CASE SHEAR — COLUMN LINE D (JOINT 4)



$$d_2 = 9" - 3/4" - 0.875" = 7.375"$$

$$d_1 = 13.5" - 3/4" - 0.875" = 11.875"$$



$$b_o = 4(24" + 11.875") = 143.5"$$

$$\phi V_c = \phi 4\sqrt{f'_c}\, b_o d$$

$$\phi V_c = 0.75(4)\sqrt{5000}\,(143.5")(11.875")$$

$$\phi V_c = 361.5\ K$$

FROM SHEAR DIAGRAM: $V_u = 199.1 + 161.03 = 360.13\ K$

$\phi V_c > V_u$

$361.5K > 360.13K \quad \therefore$ OK IN PUNCHING SHEAR

Joint 1 Reinforcement	***Middle Strip***	***Column Strip***	***Middle Strip***
Strip Width, ft	8.625	17.25	8.625
Exterior Negative Moment (kip-ft)		-455.7	
Moment Coefficient	0.033	0.934	0.033
Distributed Moments	-15.0381	-425.6238	-15.0381
Required A_s (in^2)	0.49	13.96	0.49
Minimum A_s (in^2)	1.68	3.35	1.68
Selected Steel	6 #5 bars	24 #7 bars	6 #5 bars
Provided A_s (in^2)	1.86	14.4	1.86

Mid Span of 1-2 Reinforcement	***Middle Strip***	***Column Strip***	***Middle Strip***
Strip Width, ft	8.625	17.25	8.625
End Span Positive Moment (kip-ft)		407.8	
Moment Coefficient	0.2	0.6	0.2
Distributed Moments	81.56	244.68	81.56
Required A_s (in^2)	2.68	8.03	2.68
Minimum A_s (in^2)	1.68	3.35	1.68
Selected Steel	10 #5 bars	14 #7 bars	10 #5 bars
Provided A_s (in^2)	3.1	8.4	3.1

Joint 2 Reinforcement	***Middle Strip***	***Column Strip***	***Middle Strip***
Strip Width, ft	8.625	17.25	8.625
Interior Negative Moment (kip-ft)		-994.2	
Moment Coefficient	0.125	0.75	0.125
Distributed Moments	-124.275	-745.65	-124.275
Required A_s (in^2)	4.08	24.46	4.08
Minimum A_s (in^2)	1.68	3.35	1.68
Selected Steel	14 #5 bars	31 #8 bars	14 #5 bars
Provided A_s (in^2)	4.34	24.49	4.34

Mid Span of 2-3 Reinforcement	*Middle Strip*	*Column Strip*	*Middle Strip*
Strip Width, ft	8.625	17.25	8.625
Interior Positive Moment (kip-ft)		263.1	
Moment Coefficient	0.2	0.6	0.2
Distributed Moments	52.62	157.86	52.62
Required A_s (in^2)	1.73	5.18	1.73
Minimum A_s (in^2)	1.68	3.35	1.68
Selected Steel	6 #5 bars	12 #7 bars	6 #5 bars
Provided A_s (in^2)	1.86	7.2	1.86

Joint 3 Reinforcement	*Middle Strip*	*Column Strip*	*Middle Strip*
Strip Width, ft	8.625	17.25	8.625
Interior Negative Moment (kip-ft)		-665.2	
Moment Coefficient	0.125	0.75	0.125
Distributed Moments	-83.15	-498.9	-83.15
Required A_s (in^2)	2.73	16.36	2.73
Minimum A_s (in^2)	1.68	3.35	1.68
Selected Steel	9 #5 bars	28 #7 bars	9 #5 bars
Provided A_s (in^2)	2.79	16.8	2.79

Mid Span of 3-4 Reinforcement	*Middle Strip*	*Column Strip*	*Middle Strip*
Strip Width, ft	8.625	17.25	8.625
Interior Positive Moment (kip-ft)		190.3	
Moment Coefficient	0.2	0.6	0.2
Distributed Moments	38.06	114.18	38.06
Required A_s (in^2)	1.25	3.75	1.25
Minimum A_s (in^2)	1.68	3.35	1.68
Selected Steel	6 #5 bars	13 #5 bars	6 #5 bars
Provided A_s (in^2)	1.86	4.03	1.86

Joint 4 Reinforcement	*Middle Strip*	*Column Strip*	*Middle Strip*
Strip Width, ft	8.625	17.25	8.625
Interior Negative Moment (kip-ft)		-1197.4	
Moment Coefficient	0.125	0.75	0.125
Distributed Moments	-149.675	-898.05	-149.675
Required A_s (in^2)	4.91	29.46	4.91
Minimum A_s (in^2)	1.68	3.35	1.68
Selected Steel	16 #5 bars	38 #8 bars	16 #5 bars
Provided A_s (in^2)	4.96	30.02	4.96

Mid Span 4-5 Reinforcement	*Middle Strip*	*Column Strip*	*Middle Strip*
Strip Width, ft	8.625	17.25	8.625
End Span Positive Moment (kip-ft)		544.7	
Moment Coefficient	0.2	0.6	0.2
Distributed Moments	108.94	326.82	108.94
Required A_s (in^2)	3.57	10.72	3.57
Minimum A_s (in^2)	1.68	3.35	1.68
Selected Steel	12 #5 bars	18 #7 bars	12 #5 bars
Provided A_s (in^2)	3.72	10.8	3.72

Joint 5 Reinforcement	*Middle Strip*	*Column Strip*	*Middle Strip*
Strip Width, ft	8.625	17.25	8.625
Interior Negative Moment (kip-ft)		-663.1	
Moment Coefficient	0.0285	0.943	0.0285
Distributed Moments	-18.89835	-625.3033	-18.89835
Required A_s (in^2)	0.62	20.51	0.62
Minimum A_s (in^2)	1.68	3.35	1.68
Selected Steel	6 #5 bars	26 #8 bars	6 #5 bars
Provided A_s (in^2)	1.86	20.54	1.86

Appendix F: Column Design

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COLUMN D-10 : STRENGTH DESIGN

FROM GRAVITY : $P_u = 1635$ K (FACTORED)

$M_u = 18.7$ K-FT

FROM LATERAL : $P_u = 379$ K

$M_u = 585.9$ K-FT (UNFACTORED)

LOAD COMBINATION: 1.2D + 1.6W + 1.0L

$P_u = 1635 + 1.6(379) = 2241.4$ K

MAGNIFIED MOMENTS DUE TO SWAY:

$M_2 = M_{2ns} + \delta_s M_{2s}$ [ACI 10.10.7]

$$\delta_s = \frac{1}{1 - Q} \geq 1$$

$$Q = \frac{\Sigma P_u \Delta_o}{V_{us} \ell_c}$$

$\Sigma P_u = 46347.9$ K

$\Delta_o = 0.04$ in

$V_{us} = 2109.8$ K

$\ell_c = (15 \text{ FT})(12) = 180$ in

$$Q = \frac{(46347.9 \text{ K})(0.04)}{(2109.8 \text{ K})(180 \text{ in})} = 0.005 < 0.05$$

SINCE $Q < 0.05$, NEGLECTED SWAY FRAME AND WILL CONSIDER GRAVITY LOADED COLUMN ONLY.

$P_u = 1635$ K $\quad M_u = 18.7$ K-FT

$$e = \frac{M_u}{P_u} = \frac{(18.7 \text{ K FT})(12)}{1635 \text{ K}} = 0.14 \text{ in}$$

ASSUME $d' = 2.5$ in

DETERMINE COLUMN DIMENSIONS

h	γ	e/h
20	6.75	0.007
22	0.77	0.0064
24	0.79	0.0058

$$\gamma = \frac{h - 2d'}{h}$$

USING APPENDIX A DESIGN AIDS FROM MACGREGOR 2009:

TRY 22 × 22 ASSUMING 4 FACES

$$\frac{\phi M_n}{bh^2} = \frac{(18.9 \text{ K FT})(12)}{(22)(22)^2} = 0.0213$$

$$\frac{\phi P_n}{bh} = \frac{1635 \text{ K}}{(22)(22)} = 3.38$$

FROM R-4-60-0.75 : $\rho > 0.05$ ∴ NG

TRY 24" × 24"

$$\frac{\phi M_n}{bh^2} = \frac{(18.9)(12)}{(24)(24)^2} = 0.0164$$

$$\frac{\phi P_n}{bh} = \frac{1635 \text{ K}}{(24)(24)} = 2.84$$

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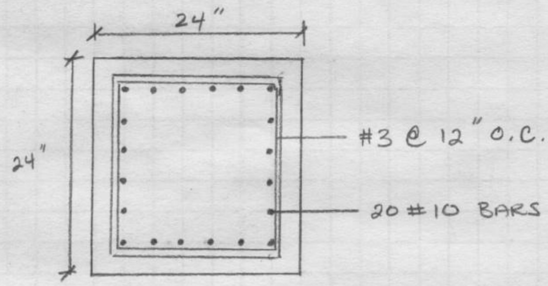
From R-4-60-0.75 : $\rho = 0.037$
From R-4-60-0.9 : $\rho = 0.035$
$\therefore$ INTERPOLATING $\rho = 0.036$ FOR $\gamma = 0.79$

$A_{s\ REQUIRED} = \rho_g bh = 0.036(24)(24) = 20.74\ in^2$
TRY 20 #10 — $A_s = 25.4\ in^2 > 20.74\ in^2$ $\therefore$ OK



CHECK b_{min}:

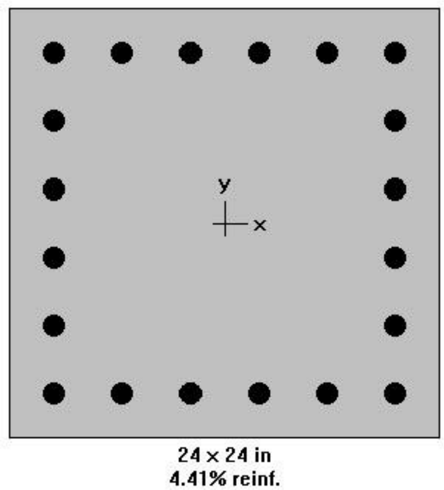
$$b_{min} = 2(\text{COVER}) + 2(\phi\ \text{TIE}) + 6(\phi\ \text{BAR}) + 5(1.5)(\phi\ \text{BAR})$$
$$= 2(1.5) + 2(0.375) + 6(1.27) + 5(1.5)(1.27) =$$
$$= 20.9\ in < 24\ in\ \therefore \text{OK}$$

TIE SPACING:

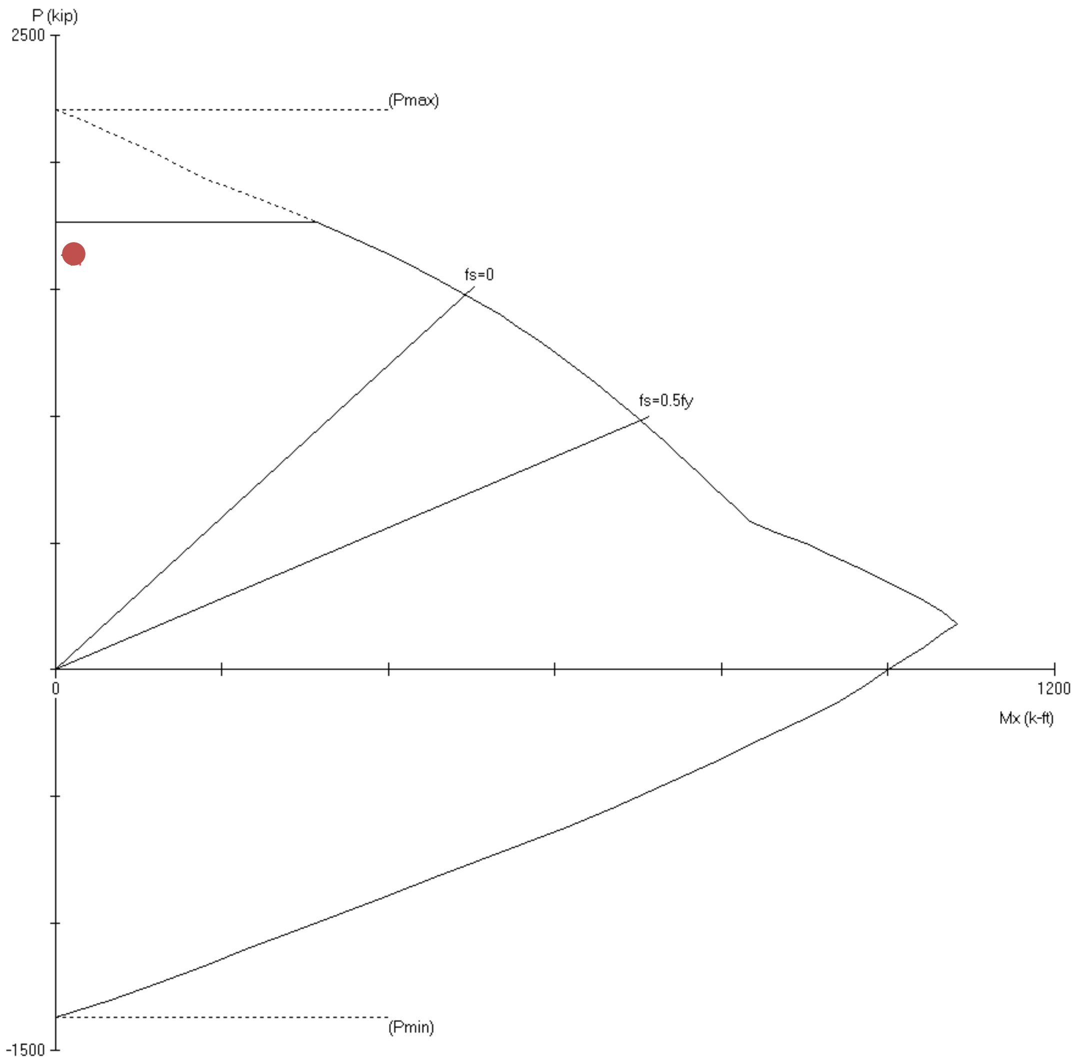
$$s \le \min \begin{cases} 16 \times \phi\ \text{LONGITUDINAL BARS} = 16(1.27) = 20.32\ in \\ 48 \times \phi\ \text{TIE} = 48(0.375) = \boxed{18\ in}\ \text{CONTROLS} \\ \text{LEAST COLUMN DIMENSION} = 24\ in \end{cases}$$

$\therefore$ USE TIES SPACED AT 12" O.C. VERTICALLY.

[FOR INTERACTION DIAGRAM SEE FOLLOWING PAGE]



Column Material and Section Properties	
f'_c	4 ksi
E_c	3605 ksi
f_y	60 ksi
E_s	29000 ksi
A_g	576 in^2
I_g	27648 in^4
Reinforcement	20 #10 bars
Confinement	Tied
Clear Cover	1.88 in



COLUMN B-10 (EDGE) : STRENGTH DESIGN

CONSIDERING GRAVITY LOADS ONLY :

$P_u = 1045.7\text{ k} \qquad M_u = 27.16\text{ k-FT}$

$$e = \frac{(27.16\text{ k}\cdot\text{FT})(12)}{1045.7\text{ k}} = 0.312\text{ in}$$

ASSUME $d' = 2.5$ in

DETERMINE COLUMN DIMENSIONS

h	γ	e/h
20	0.75	0.0156
22	0.77	0.0142
24	0.79	0.013

$$\gamma = \frac{h - 2d'}{h}$$

USING APPENDIX A DESIGN AIDS FROM MACGREGOR 2009:

TRY 20 × 20 ASSUMING 4 FACES

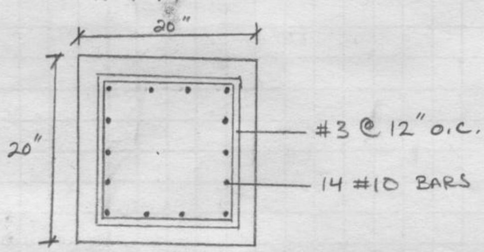
$$\frac{\phi M_n}{bh^2} = \frac{(27.16\text{ k-FT})(12)}{(20)(20)^2} = 0.041$$

$$\frac{\phi P_n}{bh} = \frac{1045.7\text{ k}}{(20)(20)} = 2.61$$

FROM R-4-60-0.75 : $\rho = 0.029$ FOR $\gamma = 0.75$

A_s REQUIRED $= \rho_g bh = 0.029(20)(20) = 11.6\text{ in}^2$

TRY 14 #10 $- A_s = 17.78\text{ in}^2 > 11.6\text{ in}^2 \therefore$ OK



CHECK b_{min} :

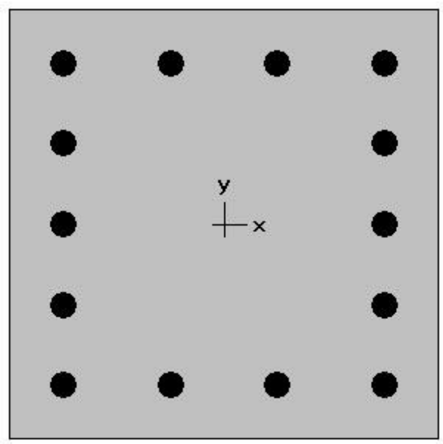
$b_{min} = 2(\text{cover}) + 2(\phi\text{ TIE}) + 4(\phi\text{ BAR}) + 3(1.5)(\phi\text{ BAR})$
$= 2(1.5) + 2(0.375) + 4(1.27) + 3(1.5)(1.27)$
$= 14.5\text{ in} < 20\text{ in} \therefore$ OK

TIE SPACING :

$S \le$ MIN
- 16 × ϕ LONGITUDINAL BARS = 16(1.27) = 20.32 in
- 48 × ϕ TIE = 48(0.375) = $\boxed{18\text{ in}}$ CONTROLS
- LEAST COLUMN DIMENSION = 20 in

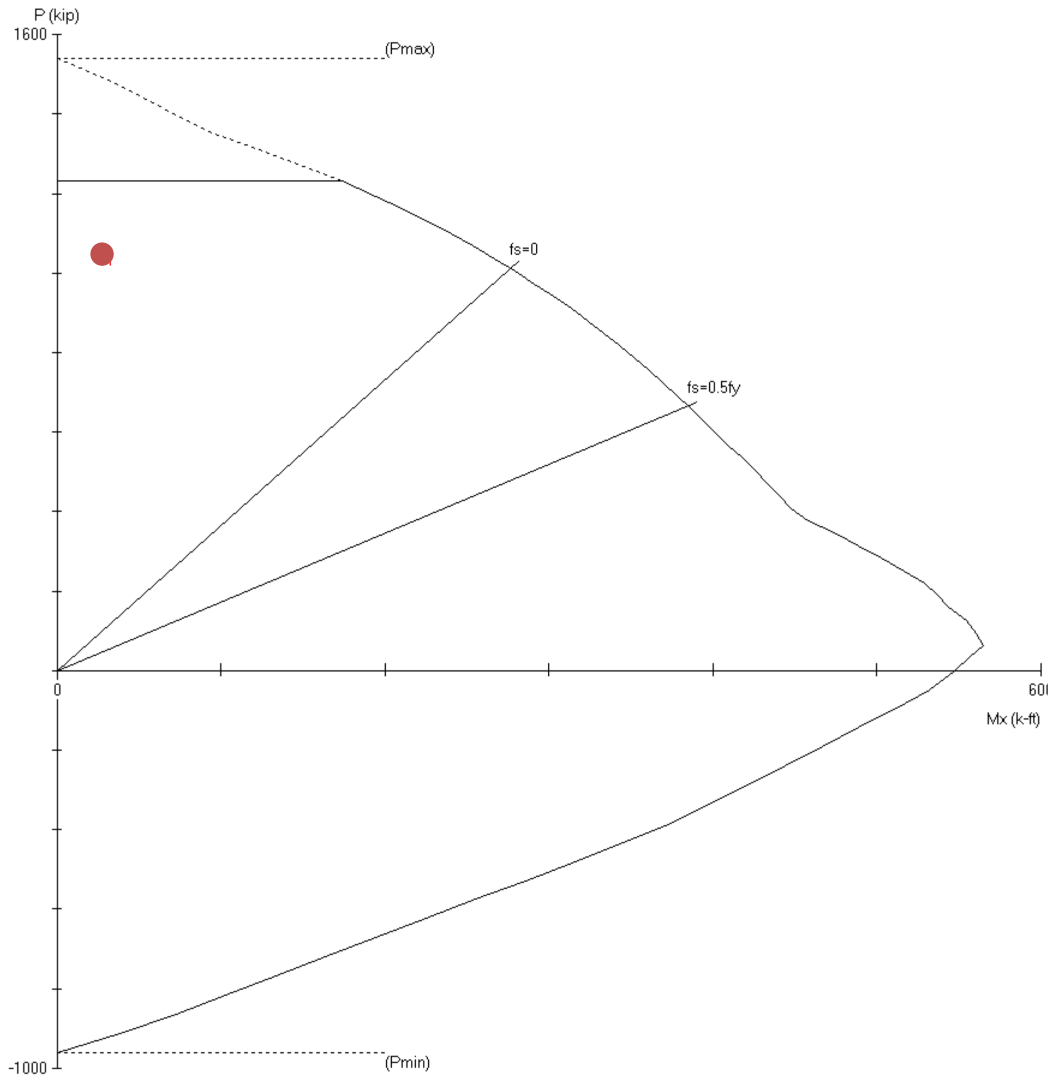
$\therefore$ SPACE VERTICAL TIES AT 12" O.C.

[FOR INTERACTION DIAGRAM SEE FOLLOWING PAGE]



20 x 20 in
4.45% reinf.

Column Material and Section Properties	
f'_c	4 ksi
E_c	3605 ksi
f_y	60 ksi
E_s	29000 ksi
A_g	400 in^2
I_g	13333.4 in^4
Reinforcement	14 #10 bars
Confinement	Tied
Clear Cover	1.87 in



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AMPAD

COLUMN H-11 (CORNER) : STRENGTH DESIGN
CONSIDERING GRAVITY LOADS ONLY
BIAXIALLY LOADED COLUMN EQUIVALENT ECCENTRICITY METHOD:

$P_u = 308.52\text{ k} \quad M_{ux} = 8.08\text{ k-ft} \quad M_{uy} = 10.32\text{ k-ft}$

$$A_{g(TRIAL)} \geq \frac{P_u}{0.4(f'_c + f_y \rho_g)} \qquad \text{ASSUME } \rho_g = 0.015$$

$$\geq \frac{308.52\text{ k}}{0.4(4 + 60(0.015))} = 157.4\text{ in}^2 \text{ OR } 12.6\text{ in SQUARE}$$

TRY 20 in SQUARE COLUMN w/ #8 BARS
ASSUME $d' = 2.5''$

$$\gamma = \frac{h - 2d'}{h} = \frac{20 - 2(2.5)}{20} = 0.75$$

$$e_x = \frac{M_{uy}}{P_u} = \frac{(10.32\text{ k-ft})(12)}{308.52} = 0.401\text{ in}$$

$$e_y = \frac{M_{ux}}{P_u} = \frac{(8.08\text{ k-ft})(12)}{308.52} = 0.314\text{ in}$$

SINCE $\frac{e_x}{l_x} > \frac{e_y}{l_y}$, USE P_u AND $M_{oy} = P_u e_{ox}$

FOR $\frac{P_u}{f'_c A_g} = \frac{308.52}{(4\text{ ksi})(20 \times 20)} = 0.2 < 0.4$ THEN:

$$\alpha = \left(0.5 + \frac{P_u}{f'_c A_g}\right)\frac{f_y + 40{,}000}{100{,}000} \geq 0.6$$

$$\alpha = (0.5 + 0.2)\frac{60000 + 40000}{100{,}000} = 0.7$$

$$e_{ox} = e_x + \frac{\alpha e_y l_x}{l_y}$$

$$e_{ox} = 0.401 + \frac{(0.7)(0.314)(20\text{ in})}{20\text{ in}} = 0.621$$

$$M_{oy} = P_u e_{ox} = 308.52(0.621) = 191.6\text{ k-in}$$

DESIGN COLUMN FOR UNIAXIAL BENDING FOR $P_u = 308.52$ k AND EQUIVALENT MOMENT OF 191.6 k·in

$$\frac{\phi M_n}{bh^2} = \frac{191.6\text{ k}\cdot\text{in}}{(20)(20)^2} = 0.024\text{ ksi}$$

$$\frac{\phi P_n}{bh} = \frac{308.52\text{ k}}{(20)(20)} = 0.77\text{ ksi}$$

FROM R-4-60-0.75 : $\rho_g = 0.01$ FOR $\gamma = 0.75$

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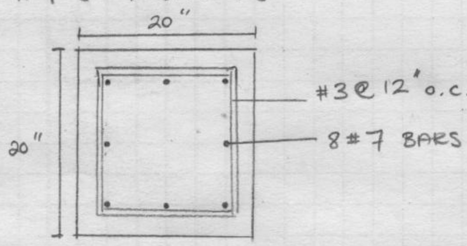
$A_{st} = \rho A_g = 0.01(20)(20) = 4 \text{ in}^2$

TRY 8 #7 BARS $A_s = 4.8 \text{ in}^2 > 4 \text{ in}^2 \therefore$ OK



CHECK b_{min}:

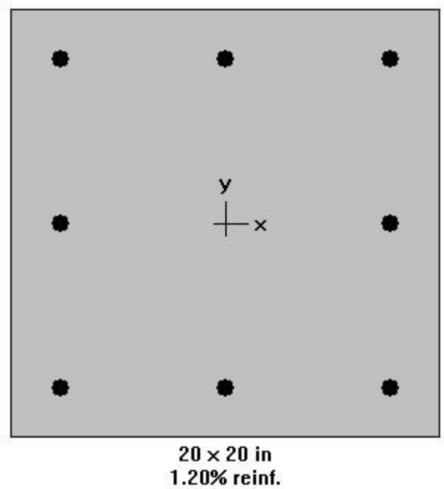
$$b_{min} = 2(\text{cover}) + 2(\phi\ \text{TIE}) + 3(\phi\ \text{BAR}) + 2(1.5)(\phi\ \text{BAR})$$
$$= 2(1.5) + 2(0.375) + 3(0.875) + 2(1.5)(0.875)$$
$$= 9 \text{ in} < 20 \text{ in} \ \therefore \text{OK}$$

TIE SPACING:

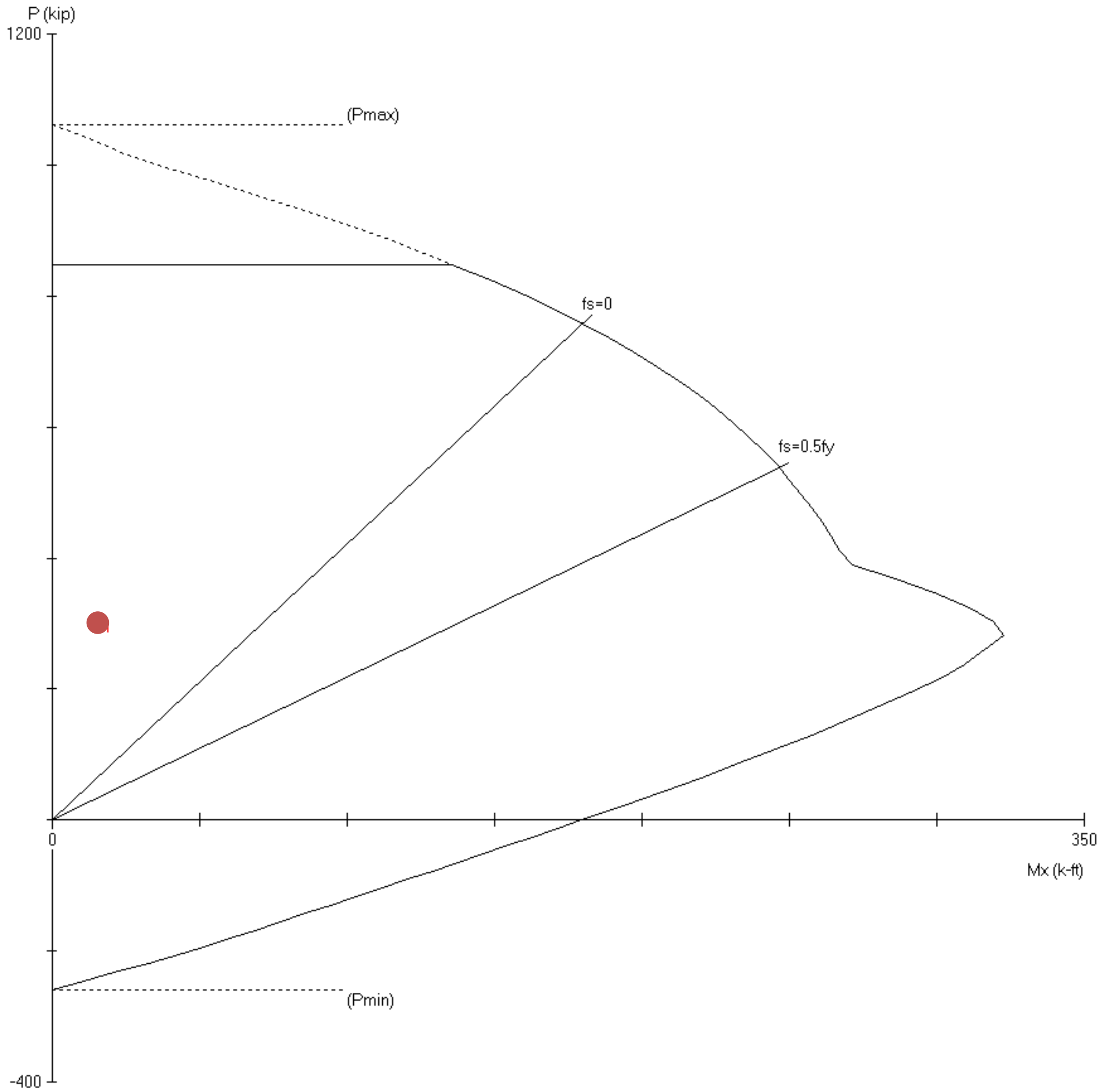
$$s \le \min \begin{cases} 16 \times \phi \text{ LONGITUDINAL BARS} = 16(0.875) = \boxed{14 \text{ in}} \text{ CONTROLS} \\ 48 \times \phi \text{ TIE} = 48(0.375) = 18 \text{ in} \\ \text{LEAST COLUMN DIMENSION} = 20 \text{ in} \end{cases}$$

$\therefore$ SPACE VERTICAL TIES AT 12" O.C.

[FOR INTERACTION DIAGRAM SEE FOLLOWING PAGE]



Column Material and Section Properties	
f'_c	4 ksi
E_c	3605 ksi
f_y	60 ksi
E_s	29000 ksi
A_g	400 in^2
I_g	13333.4 in^4
Reinforcement	8 #7 bars
Confinement	Tied
Clear Cover	1.88 in



Appendix G: Shear Wall Design

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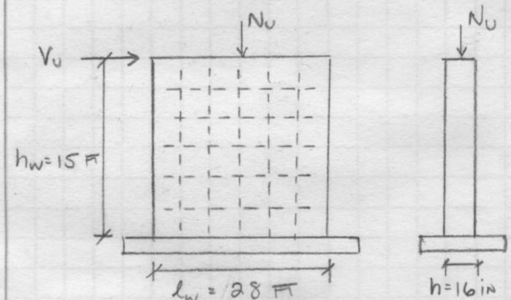
WALL 3: $V_u = 560.1$ kips
$N_u = 1162.1$ kips

ASSUMED 4000 PSI CONCRETE



SIMPLIFIED METHOD: $V_c \le 2\sqrt{f'_c}\,hd$ WHERE $d = 0.8\,l_w$

$$V_c = 2\sqrt{4000}(16)(0.8)(28\text{ FT})(12\text{ IN/FT})/1000 = 544.01\text{ kips}$$

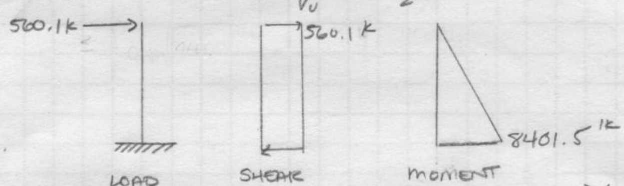
OR MINIMUM OF:

① $V_c \le 3.3\sqrt{f'_c}\,hd + \dfrac{N_u d}{4\,l_w}$

$$\le 3.3\sqrt{4000}(16\text{ in})(0.8)(28\text{ FT})(12\text{ in/FT})/1000 + \frac{(1162.1\text{ k})(268.8\text{ in})}{4(336\text{ in})}$$

$$\le 1130.04\text{ k}$$

② $V_c \le \left[0.6\sqrt{f'_c} + \dfrac{l_w\left(1.25\sqrt{f'_c} + 0.2\frac{N_u}{l_w h}\right)}{\frac{M_u}{V_u} - \frac{l_w}{2}}\right] hd$



$$V_c \le \left[0.6\sqrt{4000} + \frac{(336\text{ in})\left(1.25\sqrt{4000} + 0.2\frac{(1162.1)(1000)}{(336)(16)}\right)}{\frac{(8401.5\text{ FTk})(12)}{560.1\text{ k}} - \frac{336\text{ in}}{2}}\right](16\text{ in})(268.8\text{ in})$$

$$V_c \le \left[37.9 + \frac{(336)(79.06 + 43.23)}{12}\right](4300.8\text{ in}^2)$$

$$V_c \le 14889.5\text{ k}$$

∴ USE $V_c \le 544.01$ kips

AMPAD

HORIZONTAL REINFORCEMENT

$\frac{1}{2}\phi V_c = \frac{1}{2}(0.75)(544.01\,k) = 204\,k < V_u = 560.1\,k \therefore$ REBAR REQ'D

$V_u \leq \phi V_n = \phi(V_c + V_s)$

$560.1\,k \leq 0.75(544.01\,k + V_s) \longrightarrow V_s \geq 202.8\,k$

$$s \leq \begin{cases} \frac{l_w}{5} = \frac{(28ft)(12)}{5} = 67.2\text{ in} \\ 3h = 3(16\text{ in}) = 48\text{ in} \\ 18\text{ in} \leftarrow \text{CONTROLS} \quad (\text{ASSUME SPACED } 12\text{ in O.C.}) \end{cases}$$

$V_s < \frac{A_v f_y d}{s} \longrightarrow A_v > \frac{(202.8\,k)(12\,in)}{0.8(28ft)(12''/ft)(60\,ksi)} = 0.15\,in^2$

TRY $(2)\#3 - A_s = 0.22\,in^2$

MIN. REINF.: $\rho_t = \frac{A_v}{hs} = \frac{0.22}{(16in)(12in)} = 0.0011 < 0.0025 \therefore$ NG

TO SATISFY ρ_t: $\frac{A_v}{hs} > 0.0025$

$A_v > 0.0025(16)(12) = 0.48\,in^2$

$\boxed{\text{USE } (2)\#5\ (A_s = 0.62\,in^2) \text{ SPACED AT } 12'' \text{ O.C.}}$

VERTICAL REINFORCEMENT

$\rho_l \geq 0.0025 + 0.5\left(2.5 - \frac{h_w}{l_w}\right)(\rho_t - 0.0025)$ WHERE $\rho_t = \frac{0.62\,in^2}{(12in)(16in)} = 0.0032$

$\rho_l \geq 0.0025 + 0.5\left(2.5 - \frac{15}{28}\right)(0.0032 - 0.0025)$

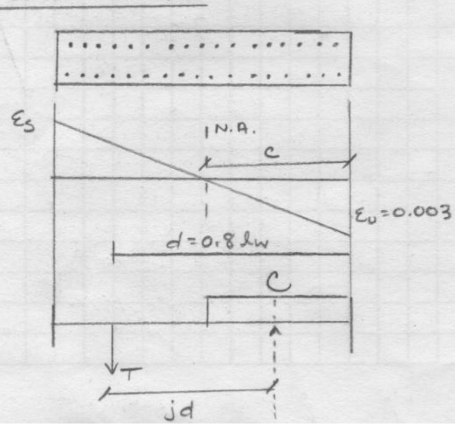
$\rho_l \geq 0.00319$

$\rho_l = \frac{A_v}{sh} \geq 0.00319$ (ASSUME SPACED 12" O.C.)

$A_v \geq 0.00319(12'')(16'') = 0.612\,in^2$ $(2\#5 = .62\,in^2)$

$\boxed{\text{USE } (2)\#5 \text{ SPACED AT } 12'' \text{ O.C.}}$

FLEXURAL DESIGN



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$d = 268.8$ in

$jd = 0.9(268.8\text{ in}) = 241.9$ in

$M_u = (560.1\text{ k})(15\text{ ft}) = 8401.5$ k-ft

$M_u = \phi M_n = \phi A_s f_y (jd)$

$(8401.5\text{ k ft})(12\text{ in/ft}) = 0.9 A_s (60\text{ ksi})(241.9\text{ in})$

$A_s = 7.718\text{ in}^2$

CHECK C=T:

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(7.718\text{ in}^2)(60\text{ ksi})}{0.85(4\text{ ksi})(16\text{ in})} = 8.51\text{ in}$$

$jd = d - a/2 = 268.8 - 8.51/2 = 264.5$ in

$$A_s = \frac{M_u}{\phi f_y jd} = \frac{(8401.5)(12)}{0.9(60)(264.5)} = 7.06\text{ in}^2$$

TRY (8) #9 $A_s = 9\text{ in}^2$

CHECK TENSION CONTROLLED SECTION:

$d_t = (28\text{ ft})(12\text{ in/ft}) - 3\text{ in} = 333$ in

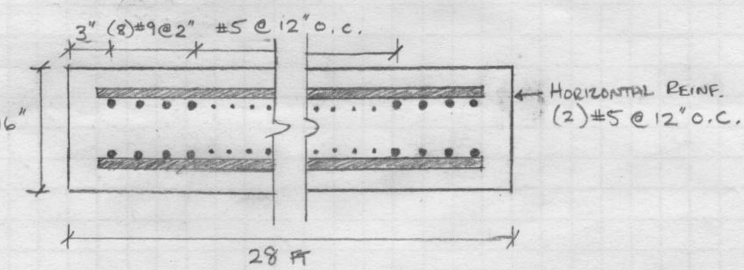
$$C = T \rightarrow a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(9.0\text{ in}^2)(60\text{ ksi})}{0.85(4\text{ ksi})(16\text{ in})} = 9.93\text{ in}$$

$c = a/\beta_1 = 9.93\text{ in}/0.85 = 11.68$ in

$$\varepsilon_t = \varepsilon_u\left(\frac{d_t - c}{c}\right) = 0.003\left(\frac{333 - 11.68}{11.68}\right) =$$

$\varepsilon_t = 0.083 > 0.005$ ∴ TENSION CONTROLLED

FINAL DESIGN



Appendix H: Detailed Cost Estimate

Existing 5 Story Steel Structure Cost Estimate

Line Number	Quantity	Unit	Description	Crew	Daily Output	Labor Hours	Material	Labor	Equipment	Unit Cost	Total	O&P	Total (w/O&P)
33105350300	2460	C.Y.	Structural Concrete, ready mix, n.w.c, 4000 PSI				$90.33	-	-	$90.33	$222,211.80	$99.10	$243,786.00
33105701400	2460	C.Y.	Structural Concrete, placing, elevated slab, pumped, < 6"	C20	140	0.457	-	$15.68	$6.59	$22.27	$54,784.20	$31.00	$76,260.00
33529300200	177034	S.F.	Concrete finishing, floors, basic finishing for unspecified	C10	1265	0.019	-	$0.69	-	$0.69	$122,153.46	$1.03	$182,345.02
53113505300	177034	S.F.	Metal floor decking, composite, galvanized, 2", 20 gauge	E4	3600	0.009	$1.47	$0.52	$0.05	$2.04	$361,149.36	$2.58	$456,747.72
32205500200	2700	C.S.F.	Welded wire fabric, 6x6 - W2.1xW2.1 (8x8)	2 Rodm	31	0.516	$18.98	$25.30	-	$44.28	$119,556.00	$61.82	$166,914.00
51223174550	155	Ea.	Column, structural tubing, 6" x 6" x 1/4" x 12'	E2	54	1.037	$277.40	$57.33	$34.68	$369.41	$57,258.55	$437.68	$67,840.40
51223174600	54	Ea.	Column, structural tubing, 8" x 8" x 3/8" x 14'	E2	50	1.12	$602.43	$61.56	$36.99	$700.98	$37,852.92	$808.59	$43,663.86
51223174600	2	Ea.	Column, structural tubing 12" x 8" x 1/2" x 16'	E2	48	1.167	$1,120.80	$63.97	$38.73	$1,223.50	$2,447.00	$1,365.60	$2,731.20
51223177000	331	L.F.	Column, structural, 2-tier, W10x45	E2	1032	0.054	$50.90	$2.98	$1.80	$55.68	$18,430.08	$63.07	$20,876.17
51223177150	96	L.F.	Column, structural, 2-tier W12x50	E2	1032	0.054	$56.51	$2.98	$1.80	$61.29	$5,883.84	$69.14	$6,637.44
51223177350	2458	L.F.	Column, structural, 2-tier, W14x74	E2	984	0.057	$83.59	$3.14	$1.88	$88.61	$217,803.38	$99.38	$244,276.04
51223177400	1601	L.F.	Column, structural, 2-tier, W14x120	E2	960	0.058	$135.43	$3.21	$1.93	$140.57	$225,052.57	$157.00	$251,357.00
51223177450	2554	L.F.	Column, structural, 2-tier, W14x176	E2	912	0.061	$198.94	$3.38	$2.03	$204.35	$521,909.90	$226.51	$578,506.54
51223750140	52	L.F.	Structural steel member, W6x20	E2	600	0.093	$22.42	$5.14	$3.10	$30.66	$1,594.32	$36.85	$1,916.20
51223750300	305	L.F.	Structural steel member, W8x10	E2	600	0.093	$11.30	$5.14	$3.10	$19.54	$5,959.70	$24.52	$7,478.60
51223750350	111	L.F.	Structural steel member, W8x21	E2	600	0.093	$23.82	$5.14	$3.10	$32.06	$3,558.66	$38.25	$4,245.75
51223750360	12	L.F.	Structural steel member, W8x24	E2	550	0.102	$27.09	$5.60	$3.38	$36.07	$432.84	$43.07	$516.84
51223750500	54	L.F.	Structural steel member, W8x31	E2	550	0.102	$35.03	$5.60	$3.38	$44.01	$2,376.54	$51.94	$2,804.76
51223750600	185	L.F.	Structural steel member, W10x12	E2	600	0.093	$13.54	$5.14	$3.10	$21.78	$4,029.30	$27.00	$4,995.00
51223750620	26	L.F.	Structural steel member, W10x15	E2	600	0.093	$16.95	$5.14	$3.10	$25.19	$654.94	$30.73	$798.98
51223750700	3010	L.F.	Structural steel member, W10x22	E2	600	0.093	$24.75	$5.14	$3.10	$32.99	$99,299.90	$39.65	$119,346.50
51223750740	106	L.F.	Structural steel member, W10x33	E2	550	0.102	$37.36	$5.60	$3.38	$46.34	$4,912.04	$54.28	$5,753.68
51223750900	57	L.F.	Structural steel member, W10x49	E2	550	0.102	$55.57	$5.60	$3.38	$64.55	$3,679.35	$73.89	$4,211.73
51223751100	1621	L.F.	Structural steel member, W12x16	E2	880	0.064	$18.07	$3.50	$2.12	$23.69	$38,401.49	$28.33	$45,922.93
51223751300	133	L.F.	Structural steel member, W12x22	E2	880	0.064	$24.75	$3.35	$2.12	$30.37	$4,039.21	$35.80	$4,761.40
51223751500	705	L.F.	Structural steel member, W12x26	E2	880	0.064	$29.42	$3.50	$2.12	$35.04	$24,703.20	$40.47	$28,531.35
51223751520	233	L.F.	Structural steel member, W12x35	E2	810	0.069	$39.70	$3.80	$2.29	$45.79	$10,669.07	$52.41	$12,211.53
51223751560	32	L.F.	Structural steel member, W12x50	E2	750	0.075	$56.51	$4.12	$2.47	$63.10	$2,019.20	$71.78	$2,296.96
51223751700	17	L.F.	Structural steel member, W12x72	E2	640	0.088	$81.26	$4.82	$2.90	$88.98	$1,512.66	$101.00	$1,717.00
51223751900	3580	L.F.	Structural steel member, W14x26	E2	990	0.057	$29.42	$3.11	$1.87	$34.40	$123,152.00	$39.55	$141,589.00
51223752100	133	L.F.	Structural steel member, W14x30	E2	900	0.062	$34.09	$3.43	$2.07	$39.59	$5,265.47	$45.42	$6,040.86
51223752300	48	L.F.	Structural steel member, W14x34	E2	810	0.069	$38.29	$3.80	$2.29	$44.38	$2,130.24	$51.48	$2,471.04
51223752320	40	L.F.	Structural steel member, W14x43	E2	810	0.069	$48.57	$3.80	$2.29	$54.66	$2,186.40	$62.22	$2,488.80
51223752340	139	L.F.	Structural steel member, W14x53	E2	800	0.07	$59.78	$3.85	$2.32	$65.95	$9,167.05	$74.92	$10,413.88
51223752700	13466	L.F.	Structural steel member, W16x26	E2	1000	0.056	$29.42	$3.08	$1.86	$34.36	$462,691.76	$39.48	$531,637.68
51223752900	6517	L.F.	Structural steel member, W16x36	E2	900	0.062	$35.03	$3.43	$2.07	$40.53	$264,134.01	$46.82	$305,125.94
51223753100	1040	L.F.	Structural steel member, W16x40	E2	800	0.07	$45.30	$3.85	$2.32	$51.47	$53,528.80	$58.57	$60,912.80
51223753120	102	L.F.	Structural steel member, W16x50	E2	800	0.07	$56.51	$3.85	$2.32	$62.68	$6,393.36	$71.18	$7,260.36
51223753140	7	L.F.	Structural steel member, W16x67	E2	760	0.074	$75.65	$4.06	$2.44	$82.15	$575.05	$92.70	$648.90
51223753300	3142	L.F.	Structural steel member, W18x35	E2	960	0.083	$39.70	$4.65	$2.12	$46.47	$146,008.74	$53.72	$168,788.24
51223753500	1455	L.F.	Structural steel member, W18x40	E2	960	0.083	$45.30	$4.65	$2.12	$52.07	$75,761.85	$59.79	$86,994.45
51223753520	1859	L.F.	Structural steel member, W18x46	E2	960	0.083	$51.84	$4.65	$2.12	$58.61	$108,955.99	$67.26	$125,036.34
51223753700	2609	L.F.	Structural steel member, W18x50	E2	912	0.088	$56.51	$4.89	$2.22	$63.62	$165,984.58	$72.94	$190,300.46
51223753900	1973	L.F.	Structural steel member, W18x55	E2	912	0.088	$62.11	$4.89	$2.22	$69.22	$136,571.06	$79.01	$155,886.73
51223753920	1453	L.F.	Structural steel member, W18x65	E2	900	0.089	$73.32	$4.96	$2.25	$80.53	$117,010.09	$91.77	$133,341.81
51223753940	2676	L.F.	Structural steel member, W18x76	E2	900	0.089	$85.93	$4.96	$2.25	$93.14	$249,242.64	$105.31	$281,809.56
51223753960	850	L.F.	Structural steel member, W18x86	E2	900	0.089	$97.17	$4.96	$2.25	$104.35	$88,697.50	$117.46	$99,841.00
51223753980	3154	L.F.	Structural steel member, W18x106	E2	900	0.089	$119.55	$4.96	$2.25	$126.76	$399,801.04	$142.67	$449,981.18
51223754780	170	L.F.	Structural steel member, W21x122	E2	1000	0.08	$138.23	$4.47	$2.02	$144.72	$24,602.40	$161.14	$27,393.80
51223755780	95	L.F.	Structural steel member, W24x146	E2	1050	0.076	$165.32	$4.25	$1.93	$171.50	$16,292.50	$190.57	$18,104.15
51223755940	43	L.F.	Structural steel member, W27x146	E2	1150	0.07	$165.32	$3.87	$1.76	$170.95	$7,350.85	$189.78	$8,160.54
51223756520	116	L.F.	Structural steel member, W30x132	E2	1160	0.069	$149.44	$3.85	$1.75	$155.04	$17,984.64	$172.88	$20,054.08
51223756560	1388	L.F.	Structural steel member, W30x173	E2	1120	0.071	$195.21	$3.98	$1.81	$201.00	$278,988.00	$223.63	$310,398.44
51223756700	151	L.F.	Structural steel member, W33x118	E2	1176	0.068	$133.56	$3.79	$1.72	$139.07	$20,999.57	$155.06	$23,414.06
51223756900	152	L.F.	Structural steel member, W33x130	E2	1134	0.071	$146.64	$3.93	$1.79	$152.36	$23,158.72	$170.31	$25,887.12
51223757100	30	L.F.	Structural steel member, W33x141	E2	1134	0.071	$159.71	$3.93	$1.79	$165.43	$4,962.90	$184.32	$5,529.60
51223757120	30	L.F.	Structural steel member, W33x169	E2	1100	0.073	$190.54	$4.06	$1.84	$196.44	$5,893.20	$219.11	$6,573.30
51223757140	43	L.F.	Structural steel member, W33x201	E2	1100	0.073	$226.96	$4.06	$1.84	$232.86	$10,012.98	$259.27	$11,148.61
51223757300	28	L.F.	Structural steel member, W36x135	E2	1170	0.068	$152.24	$3.81	$1.73	$157.78	$4,417.84	$176.55	$4,943.40
51223757500	74	L.F.	Structural steel member, W36x150	E2	1170	0.068	$169.99	$3.81	$1.73	$175.53	$12,989.22	$195.23	$14,447.02
51223757700	82	L.F.	Structural steel member, W36x194	E2	1125	0.071	$219.49	$3.96	$1.80	$225.25	$18,470.50	$249.71	$20,476.22
51223757900	1346	L.F.	Structural steel member, W36x231	E2	1125	0.071	$261.52	$3.96	$1.80	$267.28	$359,758.88	$293.61	$395,199.06
51223175550	19	Ea.	Column, structural tubing, 6" x 4" x 5/16" x 12'	E2	54	1.037	$256.85	$57.33	$34.68	$348.86	$6,628.34	$419.00	$7,961.00
51223175600	31	Ea.	Column, structural tubing, 8" x 4" x 3/8" x 12'	E2	54	1.037	$373.60	$57.33	$34.68	$465.61	$14,433.91	$545.09	$16,897.79
51223175700	21	Ea.	Column, structural tubing, 12" x 8" x 1/2" x 16'	E4	48	1.167	$1,120.80	$63.97	$38.73	$1,223.50	$25,693.50	$1,365.60	$28,677.60
51223201200	54	L.F.	Curb edging, 6" channel, 8.2 plf	E4	255	0.125	$10.69	$7.18	$0.66	$18.53	$1,000.62	$25.11	$1,355.94
51223201500	569	L.F.	Curb edging, 12" channgel, 20.7 plf	E4	140	0.229	$26.15	$131.00	$1.20	$40.45	$23,016.05	$53.27	$30,310.63
										TOTAL	$5,468,247.73		$6,326,951.99

Equivalent 5 Story Concrete Structure Cost Estimate

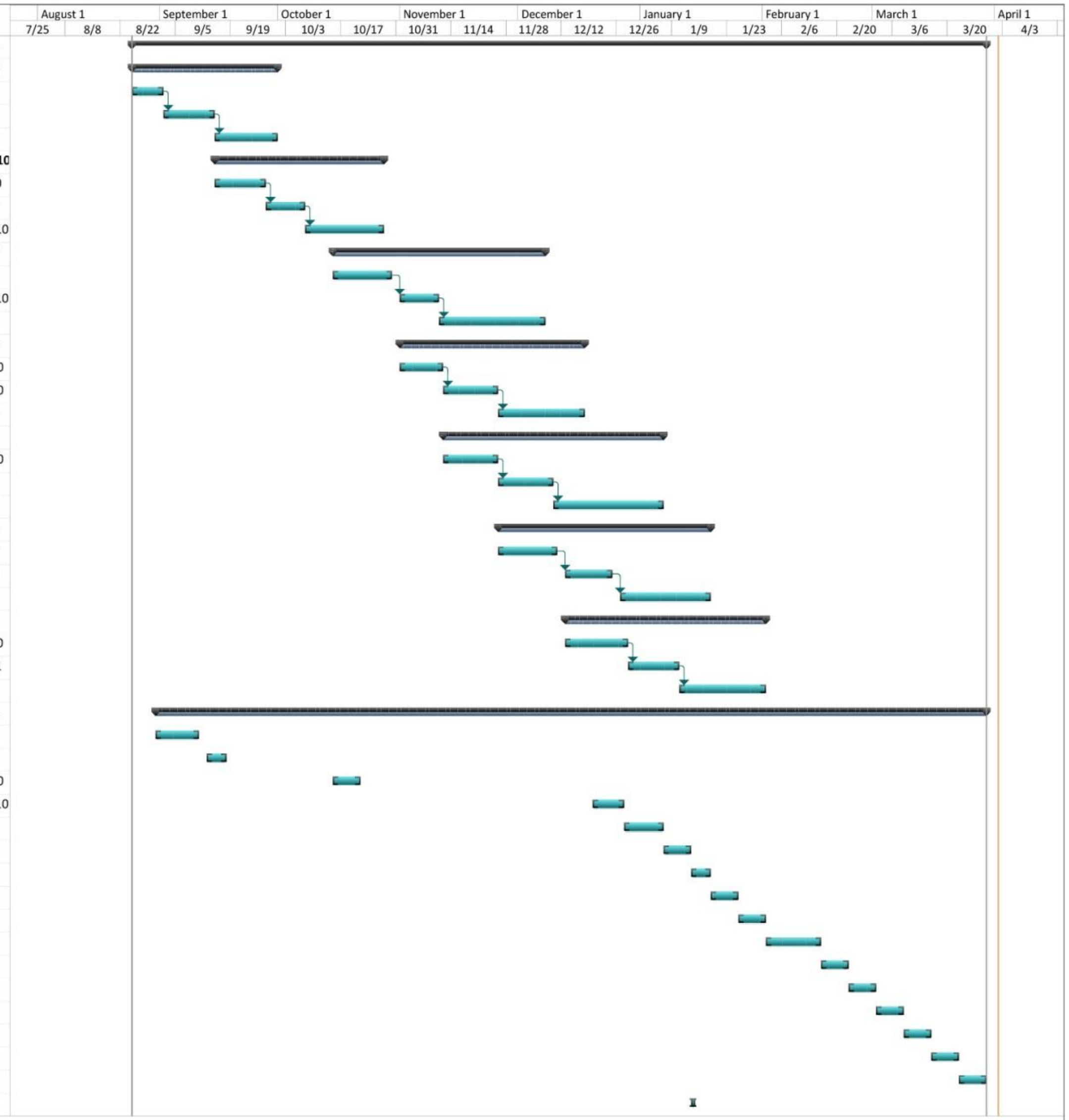
Line Number	Quantity	Unit	Description	Crew	Daily Output	Labor Hours	Material	Labor	Equipment	Unit Cost	Total	O&P	Total (w/O&P)
33105350400	5871.2	C.Y.	Structural concrete, ready mix, n.w.c, 5000 PSI				$109.00	$0.00	$0.00	$109.00	$639,960.80	$120.00	$704,544.00
33105701500	5356.2	C.Y.	Structural concrete, placing, elevated slab, pumped, 6"-10"	C20	160	0.4	$0.00	$13.00	$4.86	$17.86	$95,661.73	$25.35	$135,779.67
33105700600	101	C.Y.	Structural concrete, placing, column, pumped, 18" thick	C20	90	0.711	$ -	$ 23.00	$ 8.65	$ 31.65	$3,196.65	$ 45.00	$4,545.00
interpolated	170	C.Y.	Structural concrete, placing, column, pumped, 20" thick	C20	91		$ -	$ 22.75	$ 8.55	$ 31.30	$5,321.00	$ 44.65	$7,590.50
33105700800	244	C.Y.	Structural concrete, placing, column, pumped, 24" thick	C20	92	0.696	$ -	$ 22.50	$ 8.45	$ 30.95	$7,551.80	$ 44.30	$10,809.20
33105350411	1296	C.Y.	Structural concrete, ready mix, n.w.c, 6000 PSI				$124.00	$0.00	$0.00	$124.00	$160,704.00	$137.00	$177,552.00
33105701600	1296	C.Y.	Structural concrete, placing, elevated slab, pumped, >10"	C20	180	0.356	$0.00	$11.55	$4.32	$15.87	$20,567.52	$22.50	$29,160.00
33529300200	215278.6	S.F.	Concrete finishing, floors, basic finishing for unspecified	C10	1265	0.019	-	$0.69	-	$0.69	$148,542.23	$1.03	$221,736.96
33105350300	715.4	C.Y.	Structural concrete, ready mix, n.w.c, 4000 PSI				$106.00	$0.00	$0.00	$106.00	$75,832.40	$117.00	$83,701.80
33105705300	715.4	C.Y.	Structural concrete, placing, walls, direct chute, 16" thick	C6	105	0.457	$0.00	$14.50	$0.47	$14.97	$10,709.54	$23.02	$16,468.51
32110600400	519.3	Ton	Reinforcing steel, in place, elevated slabs, #4 to #7, grade 60	4 Rodm	2.9	11.034	$990.00	$475.00	$0.00	$1,465.00	$760,774.50	$1,880.00	$976,284.00
32110600200	53.7	Ton	Reinforcing steel, in place, columns, #3 to #7, grade 60	4 Rodm	1.5	21.333	$935.00	$915.00	$0.00	$1,850.00	$99,345.00	$2,525.00	$135,592.50
32110600250	45.5	Ton	Reinforcing steel, in place, columns, #8 to #18, grade 60	4 Rodm	2.3	13.913	$935.00	$600.00	$0.00	$1,535.00	$69,842.50	$2,005.00	$91,227.50
32110600700	21.7	Ton	Reinforcing steel, in place, walls, #3 to #7, grade 60	4 Rodm	3	10.667	$890.00	$460.00	$0.00	$1,350.00	$29,295.00	$1,730.00	$37,541.00
32110600750	14.1	Ton	Reinforcing steel, in place, walls, #8 to #18, grade 60	4 Rodm	4	8	$890.00	$345.00	$0.00	$1,235.00	$17,413.50	$1,540.00	$21,714.00
31113352250	214807.8	S.F.	C.I.P. concrete forms, elevated slab, flat slab with drop panels, 15' to 20' high ceilings, 4 use	C2	480	0.1	$4.30	$3.71	$0.00	$8.01	$1,720,610.48	$10.53	$2,261,926.13
31113256150	77440	SFCA	C.I.P. concrete forms, column, square, 16" x 16", 4 use	C1	235	0.136	$0.76	$4.92	$0.00	$5.68	$439,859.20	$8.48	$656,691.20
interpolated	59940	SFCA	C.I.P. concrete forms, column, square, 18" x 18", 4 use	C1	235	0.135	$0.79	$4.91	$0.00	$5.69	$341,058.60	$8.49	$508,590.90
interpolated	69300	SFCA	C.I.P. concrete forms, column, square, 20" x 20", 4 use	C1	235	0.135	$0.81	$4.89	$0.00	$5.70	$395,010.00	$8.49	$588,357.00
31113256650	156576	SFCA	C.I.P. concrete forms, column, square, 24" x 24", 4 use	C1	238	0.134	$0.86	$4.86	$0.00	$5.72	$895,614.72	$8.50	$1,330,896.00
31113852550	30755.8	SFCA	C.I.P. concrete forms, wall, job built, 8 to 16' high, 4 use	C2	395	0.122	$0.77	$4.51	$0.00	$5.28	$162,390.62	$7.84	$241,125.47
										Total	**$6,099,261.80**		**$8,241,833.34**

Complete Concrete Structure Cost Estimate

Line Number	Quantity	Unit	Description	Crew	Daily Output	Labor Hours	Material	Labor	Equipment	Unit Cost	Total	O&P	Total (w/O&P)
33105350400	8104.16	C.Y.	Structural concrete, ready mix, n.w.c, 5000 PSI				$109.00	$0.00	$0.00	$109.00	$883,353.44	$120.00	$972,499.20
33105701500	7452.16	C.Y.	Structural concrete, placing, elevated slab, pumped, 6"-10"	C20	160	0.4	$0.00	$13.00	$4.86	$17.86	$133,095.58	$25.35	$188,912.26
33105700600	238	C.Y.	Structural concrete, placing, column, pumped, 18" thick	C20	90	0.711	$ -	$ 23.00	$ 8.65	$ 31.65	$7,532.70	$ 45.00	$10,710.00
interpolated	170	C.Y.	Structural concrete, placing, column, pumped, 20" thick	C20	91		$ -	$ 22.75	$ 8.55	$ 31.30	$5,321.00	$ 44.65	$7,590.50
33105700800	244	C.Y.	Structural concrete, placing, column, pumped, 24" thick	C20	92	0.696	$ -	$ 22.50	$ 8.45	$ 30.95	$7,551.80	$ 44.30	$10,809.20
33105350411	1296	C.Y.	Structural concrete, ready mix, n.w.c, 6000 PSI				$124.00	$0.00	$0.00	$124.00	$160,704.00	$137.00	$177,552.00
33105701600	1296	C.Y.	Structural concrete, placing, elevated slab, pumped, >10"	C20	180	0.356	$0.00	$11.55	$4.32	$15.87	$20,567.52	$22.50	$29,160.00
33529300200	286510.8	S.F.	Concrete finishing, floors, basic finishing for unspecified	C10	1265	0.019	-	$0.69	-	$0.69	$197,692.45	$1.03	$295,106.12
33105350300	972.5	C.Y.	Structural concrete, ready mix, n.w.c, 4000 PSI				$106.00	$0.00	$0.00	$106.00	$103,085.00	$117.00	$113,782.50
33105705300	972.5	C.Y.	Structural concrete, placing, walls, direct chute, 16" thick	C6	105	0.457	$0.00	$14.50	$0.47	$14.97	$14,558.33	$23.02	$22,386.95
32110600400	675.81	Ton	Reinforcing steel, in place, elevated slabs, #4 to #7, grade 60	4 Rodm	2.9	11.034	$990.00	$475.00	$0.00	$1,465.00	$990,061.65	$1,880.00	$1,270,522.80
32110600200	72.07	Ton	Reinforcing steel, in place, columns, #3 to #7, grade 60	4 Rodm	1.5	21.333	$935.00	$915.00	$0.00	$1,850.00	$133,329.50	$2,525.00	$181,976.75
32110600250	61.8	Ton	Reinforcing steel, in place, columns, #8 to #18, grade 60	4 Rodm	2.3	13.913	$935.00	$600.00	$0.00	$1,535.00	$94,863.00	$2,005.00	$123,909.00
32110600700	29.5	Ton	Reinforcing steel, in place, walls, #3 to #7, grade 60	4 Rodm	3	10.667	$890.00	$460.00	$0.00	$1,350.00	$39,825.00	$1,730.00	$51,035.00
32110600750	19.4	Ton	Reinforcing steel, in place, walls, #8 to #18, grade 60	4 Rodm	4	8	$890.00	$345.00	$0.00	$1,235.00	$23,959.00	$1,540.00	$29,876.00
31113352250	286030.4	S.F.	C.I.P. concrete forms, elevated slab, flat slab with drop panels, 15' to 20' high ceilings, 4 use	C2	480	0.1	$4.30	$3.71	$0.00	$8.01	$2,291,103.50	$10.53	$3,011,900.11
interpolated	197320	SFCA	C.I.P. concrete forms, column, square, 18" x 18", 4 use	C1	235	0.135	$0.79	$4.91	$0.00	$5.70	$1,124,724.00	$8.49	$1,675,246.80
interpolated	138600	SFCA	C.I.P. concrete forms, column, square, 20" x 20", 4 use	C1	235	0.135	$0.81	$4.89	$0.00	$5.70	$790,020.00	$8.49	$1,176,714.00
31113256650	156576	SFCA	C.I.P. concrete forms, column, square, 24" x 24", 4 use	C1	238	0.134	$0.86	$4.86	$0.00	$5.72	$895,614.72	$8.50	$1,330,896.00
31113852550	41805.8	SFCA	C.I.P. concrete forms, wall, job built, 8 to 16' high, 4 use	C2	395	0.122	$0.77	$4.51	$0.00	$5.28	$220,734.62	$7.84	$327,757.47
										Total	**$8,137,696.81**		**$11,008,342.66**

Appendix I: Existing Construction Schedule

ID	Task Name	Duration	Start	Finish
1	**Superstructure**	**155 days**	**Wed 8/25/10**	**Tue 3/29/11**
2	**Structural Steel: 7-11 West (G to 1)**	**27 days**	**Wed 8/25/10**	**Thu 9/30/10**
3	Set Structural Steel	6 days	Wed 8/25/10	Wed 9/1/10
4	Detail Steel	9 days	Thu 9/2/10	Tue 9/14/10
5	Install Decking	12 days	Wed 9/15/10	Thu 9/30/10
6	**Structural Steel: 11-7 Center (G to 1)**	**31 days**	**Wed 9/15/10**	**Wed 10/27/10**
7	Set Structural Steel	9 days	Wed 9/15/10	Mon 9/27/10
8	Detail Steel	8 days	Tue 9/28/10	Thu 10/7/10
9	Install Decking	14 days	Fri 10/8/10	Wed 10/27/10
10	**Structural Steel: 7-1 East (G to 1)**	**38 days**	**Fri 10/15/10**	**Tue 12/7/10**
11	Set Structural Steel	11 days	Fri 10/15/10	Fri 10/29/10
12	Detail Steel	8 days	Mon 11/1/10	Wed 11/10/10
13	Install Decking	19 days	Thu 11/11/10	Tue 12/7/10
14	**Structural Steel: 11-7 Center (1 to 3)**	**35 days**	**Mon 11/1/10**	**Fri 12/17/10**
15	Set Structural Steel	9 days	Mon 11/1/10	Thu 11/11/10
16	Detail Steel	10 days	Fri 11/12/10	Thu 11/25/10
17	Install Decking	16 days	Fri 11/26/10	Fri 12/17/10
18	**Structural Steel: 7-1 East (1 to 3)**	**40 days**	**Fri 11/12/10**	**Thu 1/6/11**
19	Set Structural Steel	10 days	Fri 11/12/10	Thu 11/25/10
20	Detail Steel	10 days	Fri 11/26/10	Thu 12/9/10
21	Install Decking	20 days	Fri 12/10/10	Thu 1/6/11
22	**Structural Steel: 11-7 Center (3 to Roof)**	**38 days**	**Fri 11/26/10**	**Tue 1/18/11**
23	Set Structural Steel	11 days	Fri 11/26/10	Fri 12/10/10
24	Detail Steel	10 days	Mon 12/13/10	Fri 12/24/10
25	Install Decking	17 days	Mon 12/27/10	Tue 1/18/11
26	**Structural Steel: 7-1 East (3 to Roof)**	**37 days**	**Mon 12/13/10**	**Tue 2/1/11**
27	Set Structural Steel	12 days	Mon 12/13/10	Tue 12/28/10
28	Detail Steel	9 days	Wed 12/29/10	Mon 1/10/11
29	Install Decking	16 days	Tue 1/11/11	Tue 2/1/11
30	**Deck Pours**	**151 days**	**Tue 8/31/10**	**Tue 3/29/11**
31	Pour SOG (West)	9 days	Tue 8/31/10	Fri 9/10/10
32	Pour SOG (Center)	5 days	Mon 9/13/10	Fri 9/17/10
33	Pour SOG (East)	5 days	Fri 10/15/10	Thu 10/21/10
34	Pour 1st Floor Slab (West)	6 days	Mon 12/20/10	Mon 12/27/10
35	Pour 1st Floor Slab (Center)	8 days	Tue 12/28/10	Thu 1/6/11
36	Pour 1st Floor Slab (East)	5 days	Fri 1/7/11	Thu 1/13/11
37	Pour 2nd Floor Slab (West)	3 days	Fri 1/14/11	Tue 1/18/11
38	Pour 2nd Floor Slab (Center)	5 days	Wed 1/19/11	Tue 1/25/11
39	Pour 2nd Floor Slab (East)	5 days	Wed 1/26/11	Tue 2/1/11
40	Pour Roof Slab	10 days	Wed 2/2/11	Tue 2/15/11
41	Pour 5th Floor Slab (Center)	5 days	Wed 2/16/11	Tue 2/22/11
42	Pour 5th Floor Slab (East)	5 days	Wed 2/23/11	Tue 3/1/11
43	Pour 4th Floor Slab (Center)	5 days	Wed 3/2/11	Tue 3/8/11
44	Pour 4th Floor Slab (East)	5 days	Wed 3/9/11	Tue 3/15/11
45	Pour 3rd Floor Slab (Center)	5 days	Wed 3/16/11	Tue 3/22/11
46	Pour 3rd Floor Slab (East)	5 days	Wed 3/23/11	Tue 3/29/11
47	Spray Fire Proofing All Levels	1 day	Fri 1/14/11	Fri 1/14/11



Project: Steel Construction Sched Date: Sat 4/2/11	Task		Summary		External Milestone		Inactive Summary		Manual Summary Rollup		Finish-only	
	Split		Project Summary		Inactive Task		Manual Task		Manual Summary		Deadline	
	Milestone		External Tasks		Inactive Milestone		Duration-only		Start-only		Progress	

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Appendix J: Proposed Schedule

ID	Task Name	Duration	Start
1	**Concrete Superstructure**	**289 days**	**Wed 8/25/10**
2	**Ground Floor**	**17 days**	**Wed 8/25/10**
3	Frame Columns	15 days	Wed 8/25/10
4	Set Rebar in Columns	5 days	Wed 9/8/10
5	Frame Walls	5 days	Wed 9/8/10
6	Set Rebar in Walls	2 days	Mon 9/13/10
7	Place Concrete in Columns	2 days	Wed 9/15/10
8	Place Concrete in Walls	2 days	Wed 9/15/10
9	**2nd Floor**	**38 days**	**Thu 9/23/10**
10	Frame Slab	12 days	Thu 9/23/10
11	Set Rebar in Slab	7 days	Thu 9/30/10
12	Place Slab	7 days	Sat 10/9/10
13	Frame Columns	15 days	Mon 10/25/10
14	Set Rebar in Columns	5 days	Mon 11/8/10
15	Frame Walls	5 days	Mon 11/8/10
16	Set Rebar in Walls	2 days	Thu 11/11/10
17	Place Concrete in Columns	2 days	Sat 11/13/10
18	Place Concrete in Walls	2 days	Sat 11/13/10
19	**3rd Floor**	**39 days**	**Mon 11/22/10**
20	Frame Slab	12 days	Mon 11/22/10
21	Set Rebar in Slab	7 days	Mon 11/29/10
22	Place Slab	7 days	Wed 12/8/10
23	Frame Columns	14 days	Thu 12/23/10
24	Set Rebar in Columns	5 days	Wed 1/5/11
25	Frame Walls	5 days	Wed 1/5/11
26	Set Rebar in Walls	2 days	Mon 1/10/11
27	Place Concrete in Columns	1 day	Thu 1/13/11
28	Place Concrete in Walls	2 days	Wed 1/12/11
29	**4th Floor**	**37 days**	**Thu 1/20/11**
30	Frame Slab	12 days	Thu 1/20/11
31	Set Rebar in Slab	7 days	Thu 1/27/11
32	Place Slab	7 days	Sat 2/5/11
33	Frame Columns	14 days	Mon 2/21/11
34	Set Rebar in Columns	5 days	Fri 3/4/11
35	Frame Walls	5 days	Fri 3/4/11
36	Set Rebar in Walls	2 days	Wed 3/9/11
37	Place Concrete in Columns	1 day	Sat 3/12/11
38	Place Concrete in Walls	2 days	Fri 3/11/11
39	**5th Floor**	**35 days**	**Sat 3/19/11**
40	Frame Slab	12 days	Sat 3/19/11
41	Set Rebar in Slab	7 days	Fri 3/25/11
42	Place Slab	7 days	Tue 4/5/11
43	Frame Columns	13 days	Wed 4/20/11
44	Set Rebar in Columns	5 days	Mon 5/2/11
45	Frame Walls	5 days	Mon 5/2/11
46	Set Rebar in Walls	2 days	Thu 5/5/11
47	Place Concrete in Columns	1 day	Sun 5/8/11
48	Place Concrete in Walls	2 days	Sat 5/7/11
49	**6th Floor**	**36 days**	**Sun 5/15/11**
50	Frame Slab	12 days	Sun 5/15/11
51	Set Rebar in Slab	7 days	Fri 5/20/11
52	Place Slab	7 days	Tue 5/31/11
53	Frame Columns	13 days	Wed 6/15/11
54	Set Rebar in Columns	5 days	Mon 6/27/11
55	Frame Walls	5 days	Mon 6/27/11
56	Set Rebar in Walls	2 days	Thu 6/30/11
57	Place Concrete in Columns	1 day	Mon 7/4/11
58	Place Concrete in Walls	2 days	Sat 7/2/11
59	**Penthouse**	**38 days**	**Mon 7/11/11**
60	Frame Slab	12 days	Mon 7/11/11
61	Set Rebar in Slab	7 days	Mon 7/18/11
62	Place Slab	7 days	Wed 7/27/11
63	Frame Columns	13 days	Thu 8/11/11
64	Set Rebar in Columns	5 days	Tue 8/23/11
65	Frame Walls	5 days	Tue 8/23/11
66	Set Rebar in Walls	2 days	Fri 8/26/11
67	Place Concrete in Columns	1 day	Wed 8/31/11
68	Place Concrete in Walls	2 days	Tue 8/30/11
69	**Roof**	**19 days**	**Wed 9/7/11**
70	Frame Slab	12 days	Wed 9/7/11
71	Set Rebar in Slab	7 days	Wed 9/14/11
72	Place Slab	7 days	Fri 9/23/11

Appendix K: Building Enclosure

MATT VANDERSALL | THESIS REPORT | BUILDING ENCLOSURE | 1

CLIMATE CONDITIONS FOR HERSHEY PA:

	WINTER			SUMMER		
	TEMP (°F)	TEMP (K)	RH %	TEMP (°F)	TEMP (K)	RH %
INDOOR	70	294.3	25	75	297.	50
OUTDOOR	9	260.2	79	91	306	50

FROM SPECIFICATIONS:

IGU #1 : BASIS OF DESIGN PRODUCT : OLDCASTLE PPG SOLARBAN 60
OVERALL THICKNESS AND THICKNESS OF EACH LITE : 25mm AND 6mm
INTERSPACE CONTENT : AIR
OUTDOOR LITE : CLEAR FLOAT GLASS
LOW-E COATING
INDOOR LITE : CLEAR FLOAT GLASS

IGU #6 : BASIS OF DESIGN PRODUCT : OLDCASTLE PPG SOLARBAN 60
OVERALL THICKNESS AND THICKNESS OF EACH LITE : 25mm AND 60mm
INTERSPACE CONTENT : AIR
OUTDOOR LITE : CLEAR FLOAT GLASS
LOW-E COATING
INDOOR LITE : CLEAR FLOAT GLASS
CERAMIC FRIT COATING – COLOR : "WARM GREY"

SEE ATTACHED OLDCASTLE SPEC SHEETS FOR PERFORMANCE CHARACTERISTICS

GLAZING AREA FOR 1 PATIENT ROOM:

AREA OF IGU #1 = 3.5 (8.25 FT)(2.75 FT) = 79.41 FT² (7.38 m²)
AREA OF IGU #1 = 6.5 (8.25 FT)(2.75 FT) = 147.47 FT² (13.7 m²)

RADIATION INDUCED HEAT CALCULATION:

$$q = \alpha I_{SW} + \alpha I_{LW} - F_A F_E \sigma T_s^4 + h_c (T_a - T_s) + U (T_s - T_{rn})$$

FOR SUMMER CONDITION:

$\alpha = 1.0 - \rho - \tau \longrightarrow$ IGU #1 : $\alpha = 1 - 0.29 - 0.33 = 0.38$
IGU #6 : $\alpha = 1 - 0.31 - 0.17 = 0.52$

$I_{SW} = 1000\ W/m^2$

$F_A = 0.5$ FOR WALLS

$F_E = \varepsilon = \alpha$

$\sigma = 5.67 \times 10^{-8}\ W/m^2K^4$

$h_c = 15\ W/m^2K$ (ASSUMING AIR VELOCITY = 2.81 m/s)

$U = 1.53\ W/m^2K$

T_a (AIR) = 306 K

T_s (SURFACE) = 322.6 K (FOR SUMMER CONDITIONS, ASSUMED $T_s = T_a + 30°F$)

$T_{sky} = 1.2\,T_a - 14$ (°C)
$= 1.2(32.8\ °C) - 14 = 25.4\ °C + 273 = 298.4\ K$

$I_{LW} = \varepsilon \sigma T_{sky}^4$ WHERE $\varepsilon = 1.0$ FOR CLEAR SKY
$= (1.0)(5.67 \times 10^{-8})(298.4\ K)^4$
$= 449.6\ \frac{W}{m^2}$

FOR IGU #1:

$$q = (0.38)(1000\,\tfrac{W}{m^2}) + (0.38)(449.6\,\tfrac{W}{m^2}) - (0.5)(0.38)(5.67\times10^{-8}\,\tfrac{W}{m^2K^4})(322.6\,K)^4 + (15\,\tfrac{W}{m^2K})(306 - 322.6) + (1.53\,\tfrac{W}{m^2K})(322.6 - 297\,K)$$

$$q = 380 + 170.8 - 116.8 - 249 + 39.2$$
$$= 224.2\ W/m^2$$

FOR IGU #6:

$$q = (0.52)(1000\,\tfrac{W}{m^2}) + (0.52)(449.6\,\tfrac{W}{m^2}) - (0.5)(0.52)(5.67\times10^{-8}\,\tfrac{W}{m^2K^4})(322.6\,K)^4 + (15\,\tfrac{W}{m^2K})(306 - 322.6) + (1.53\,\tfrac{W}{m^2K})(322.6 - 297\,K)$$

$$q = 520 + 233.8 - 159.7 - 249 + 39.2$$
$$= 384.3\ W/m^2$$

HEAT FLOW RATE $(Q) = (224.2\,\tfrac{W}{m^2})(7.38\,m^2) + (384.3\,\tfrac{W}{m^2})(13.7\,m^2)$

$= \underline{6919.5\ W}$ (23626.6 BTU/HR)

FOR WINTER CONDITION:

α: IGU #1 = 0.38
IGU #6 = 0.52
$I_{sw} = 1000\ W/m^2$
$F_A = 0.5$ FOR WALLS
$F_E = \varepsilon = \alpha$
$\sigma = 5.67\times10^{-8}\ W/m^2K^4$
$h_c = 25\ W/m^2K$ (ASSUMING AIR VELOCITY = 5.29 m/s)
$U = 1.65\ W/m^2K$
T_a (AIR) = 260.2 K
T_s (SURFACE) = 260.2 K (ASSUMED $T_s = T_a$)
$T_{sky} = 260.2\ K$
$I_{LW} = \varepsilon\sigma T_{sky}^4$ WHERE $\varepsilon = 0.8$ FOR CLOUDY CONDITION
$= (0.8)(5.67\times10^{-8})(260.2)^4$
$= 207.9\ W/m^2$

FOR IGU #1:

$$q = (0.38)(1000\,\tfrac{W}{m^2}) + (0.38)(207.9\,\tfrac{W}{m^2}) - (0.5)(0.38)(5.67\times10^{-8}\,\tfrac{W}{m^2K^4})(260.2\,K)^4 + \cancel{(25\,\tfrac{W}{m^2K})(260.2 - 260.2)}^{\,0} + (1.65\,\tfrac{W}{m^2K})(260.2 - 294.3\,K)$$

$$q = 380 + 79 - 49.4 - 56.3$$
$$= 353.3\ W/m^2$$

FOR IGU #6:

$$q = (0.52)(1000\,\tfrac{W}{m^2}) + (0.52)(207.9\,\tfrac{W}{m^2}) - (0.5)(0.52)(5.67\times10^{-8}\,\tfrac{W}{m^2K^4})(260.2\,K)^4 + (1.65\,\tfrac{W}{m^2K})(260.2 - 294.3\,K)$$

$$q = 520 + 108.1 - 67.6 - 56.3$$
$$= 504.2\ \tfrac{W}{m^2}$$

HEAT FLOW RATE $(Q) = (353.3\,\tfrac{W}{m^2})(7.38\,m^2) + (504.2\,\tfrac{W}{m^2})(13.7\,m^2)$

$= \underline{9514.9\ W}$ (32488.6 BTU/HR)

MATT VANDERSALL	THESIS REPORT	BUILDING ENCLOSURES	3

REDESIGN OF SPANDREL IGU #6 UNIT FOR GREATER R-VALUE
SHADOW BOX DESIGN:

MONOLITHIC GLASS (OLDCASTLE): $U = 2.78 \frac{W}{m^2K}$ SUMMER

$U = 3.64 \frac{W}{m^2K}$ WINTER

2" AIR SPACE : $R = \frac{l}{C} = \frac{0.0508\ m}{0.03 \frac{W}{mK}} = 1.69 \frac{m^2K}{W}$

$U = \frac{1}{R} = \frac{1}{1.69} = 0.59\ W/m^2K$

2" RIGID INSULATION : $R = 10 \frac{ft^2 h°F}{BTU}$ (DOW BUILDING – EXTRUDED POLYSTYRENE)

$R = 1.76 \frac{m^2K}{W}$

$U = \frac{1}{R} = \frac{1}{1.76} = 0.57$

SYSTEM U-VALUE :

SUMMER : $U = 3.94 \frac{W}{m^2K}$

WINTER : $U = 4.8 \frac{W}{m^2K}$

SUMMER HEAT CALCULATION :

$\alpha = 0.4$ (ASSUMING GREY MATERIAL ON FACE OF INSULATION

$$q = (0.4)(1000 \tfrac{W}{m^2}) + (0.4)(449.6 \tfrac{W}{m^2}) - (0.5)(0.4)(5.67 \times 10^{-8} \tfrac{W}{m^2K^4})(322.6K)^4 + (15 \tfrac{W}{m^2K})(306 - 322.6) + (3.94 \tfrac{W}{m^2K})(322.6 - 297\ K)$$

$$q = 400 + 179.84 - 122.8 - 249 + 100.9$$

$$= 308.9 \tfrac{W}{m^2}$$

HEAT FLOW RATE $(\dot{Q}) = (224.2 \frac{W}{m^2})(7.38\ m^2) + (308.9 \frac{W}{m^2})(13.7\ m^2)$

$= 5886.5\ W$ (20099.5 BTU/HR)

WINTER HEAT CALCULATION :

$$q = (0.4)(1000 \tfrac{W}{m^2}) + (0.4)(207.9 \tfrac{W}{m^2}) - (0.5)(0.4)(5.67 \times 10^{-8} \tfrac{W}{m^2K^4})(260.2K)^4 + (4.8 \tfrac{W}{m^2K})(260.2 - 294.3\ K)$$

$$q = 400 + 83.16 - 52 - 163.8$$

$$= 267.4$$

HEAT FLOW RATE $(\dot{Q}) = (353.3 \frac{W}{m^2})(7.38\ m^2) + (267.4 \frac{W}{m^2})(13.7\ m^2)$

$= 6270.7\ W$ (21411.4 BTU/HR)

MATT VANDERSALL | THESIS REPORT | BUILDING ENCLOSURE | 4

COST COMPARISON: ASSUMED 10.1 ¢/KW·HR (DATA TAKEN FROM EIA.DOE.GOV)

	WINTER (BTU/HR)	SUMMER (BTU/HR)
EXISTING SYSTEM	32488.6	23626.6
REDESIGNED SYSTEM	21411.4	20099.5
DIFFERENCE	11077.2	3527.1

ASSUMING WINTER CONDITION AND SUMMER CONDITION LAST HALF A YEAR EACH.

$$\frac{11077.2\ \text{BTU/HR}\ (182.5\ \text{DAYS})(24\ \text{HRS})}{1000}\left(\$0.101/_{\text{KW HR}}\right) = \$4900.33$$

$$\frac{3527.1\ \text{BTU/HR}\ (182.5\ \text{DAYS})(24\ \text{HRS})}{1000}\left(\$0.101/_{\text{KW HR}}\right) = \$1560.32$$

THEREFORE FOR 1 PATIENT ROOM, ANNUAL ELECTRICAL COST SAVINGS OF $6460.65. WITH 12 EQUIVALENT SPACES AND 2 FLOORS OF ROOMS, TOTAL COST SAVINGS ARE AROUND $155055.60/YEAR.

Oldcastle Spec Sheet: IGU #1 – Clear Vision Insulating Glass

PRODUCTS

Approved Glass Fabricator Oldcastle BuildingEnvelope™

Glass Description FLOAT GLASS

1. USA - Annealed float glass shall comply with ASTM C1036, Type I, Class 1 (clear), Class 2 (tinted), Quality-Q3. Canada - Annealed float glass shall comply with CAN/CGSB-12.3-M, Quality-Glazing.

2. USA- Heat-strengthened float glass shall comply with ASTM C1048, Type I, Class 1 (clear), Class 2 (tinted), Quality Q3, Kind HS. Canada - Heat-strengthened float glass shall comply with CAN/CGSB-12.9-M, Type 2-Heat-Strengthened Glass, Class A-Float Glass.

3. USA - Tempered float glass shall comply with ASTM C1048, Type I, Class 1 (clear), Class 2 (tinted), Quality Q3, Kind FT. Canada - Tempered float glass shall comply with CAN/CGSB-12.1-M, Type 2-Tempered Glass, Class B-Float Glass.

4. USA - Laminated glass to comply with ASTM C1172. Canada - Laminated glass to comply with CAN/CGSB-12.1-M, Type 1-Laminated Glass, Class B-Float Glass.

5. Glass shall be annealed, heat-strengthened or tempered as required by codes, or as required to meet thermal stress and wind loads.

Sealed Insulating Glass (IG) Vision Glass (vertical) GENERAL

1. IG units consist of glass lites separated by a dehydrated airspace that is hermetically dual sealed with a primary seal of polyisobutylene (PIB), or thermo plastic spacer (TPS) and a secondary seal of silicone or an organic sealant depending on the application.

2. USA - Insulating glass units are certified through the Insulating Glass Certification Council (IGCC) to ASTM E2190. Canada - Insulating Glass units are certified through the Insulating Glass Manufacturers Alliance (IGMA) to either the IGMAC certification program to CAN/CGSB-12.8, or through the IGMA program to ASTM E2190..

IG VISION UNIT PERFORMANCE CHARACTERISTICS

1. Exterior Lite 1/4" PPG Solarban® 60 on Clear Low-E #2

2. Interior Lite 1/4" Clear

3. 1/2" Cavity Air (Standard)

4. Performance Characteristics

Winter U-factor/U-Value (Btu/hr-ft²-F°):	0.29	Visible Light Transmittance:	70%
Summer U-factor/U-Value (Btu/hr-ft²-F°):	0.27	Visible Light Reflectance (outside):	11%
Solar Heat Gain Coefficient:	0.38	Visible Light Reflectance (inside):	12%
Shading Coefficient:	0.44	Total Solar Transmittance:	33%
Relative Heat Gain:	92	Total Solar Reflectance (outside):	29%
Light to Solar Gain:	1.84	Ultraviolet Transmittance:	19%

Contact Oldcastle BuildingEnvelope™ at 866-Oldcastle (653-2278) for samples or additional information concerning performance, strength, deflection, thermal stress or application guidelines. GlasSelect® calculates center of glass performance data using the Lawrence Berkeley National Laboratory (LBNL) Window 5.2 program (version 5.2.17) with Environmental Conditions set at NFRC 100-2001. Gas Library ID#1 (Air) is used for Insulating Glass units with air. Gas Library ID#9 (10% Air/90% Argon) is used for Insulating Glass units with argon. Monolithic glass data is from the following sources: 1. LBNL International Glazing Database (IGDB) version 17.3; 2. Vendor supplied spectral data files. Laminated glass data is from the following sources: 1. LBNL International Glazing Database (IGDB) version 17.3; 2. LBNL Optics 5 (version 5.1 Maintenance Pack 2); 3. Vendor supplied spectral data files; 4. Vendor supplied data.

Oldcastle Spec Sheet: IGU #6 – Warm Grey Spandrel Insulating Glass

PRODUCTS

Approved Glass Fabricator Oldcastle BuildingEnvelope™

Glass Description FLOAT GLASS

1. USA - Annealed float glass shall comply with ASTM C1036, Type I, Class 1 (clear), Class 2 (tinted), Quality-Q3. Canada - Annealed float glass shall comply with CAN/CGSB-12.3-M, Quality-Glazing.

2. USA- Heat-strengthened float glass shall comply with ASTM C1048, Type I, Class 1 (clear), Class 2 (tinted), Quality Q3, Kind HS. Canada - Heat-strengthened float glass shall comply with CAN/CGSB-12.9-M, Type 2-Heat-Strengthened Glass, Class A-Float Glass.

3. USA - Tempered float glass shall comply with ASTM C1048, Type I, Class 1 (clear), Class 2 (tinted), Quality Q3, Kind FT. Canada - Tempered float glass shall comply with CAN/CGSB-12.1-M, Type 2-Tempered Glass, Class B-Float Glass.

4. USA - Laminated glass to comply with ASTM C1172. Canada - Laminated glass to comply with CAN/CGSB-12.1-M, Type 1-Laminated Glass, Class B-Float Glass.

5. Glass shall be annealed, heat-strengthened or tempered as required by codes, or as required to meet thermal stress and wind loads.

Sealed Insulating Glass (IG) Vision Glass (vertical) GENERAL

1. IG units consist of glass lites separated by a dehydrated airspace that is hermetically dual sealed with a primary seal of polyisobutylene (PIB), or thermo plastic spacer (TPS) and a secondary seal of silicone or an organic sealant depending on the application.

2. USA - Insulating glass units are certified through the Insulating Glass Certification Council (IGCC) to ASTM E2190. Canada - Insulating Glass units are certified through the Insulating Glass Manufacturers Alliance (IGMA) to either the IGMAC certification program to CAN/CGSB-12.8, or through the IGMA program to ASTM E2190..

IG VISION UNIT PERFORMANCE CHARACTERISTICS

1. Exterior Lite 1/4" PPG Solarban® 60 on Clear Low-E #2

2. Interior Lite 1/4" Clear with Warm Gray Ceramic Frit Silk-screened #3 Standard Hole Pattern 60% Coverage

3. 1/2" Cavity Air (Standard)

4. Performance Characteristics

Winter U-factor/U-Value (Btu/hr-ft²-F°):	0.29	Visible Light Transmittance:	38%
Summer U-factor/U-Value (Btu/hr-ft²-F°):	0.27	Visible Light Reflectance (outside):	17%
Solar Heat Gain Coefficient:	0.33	Visible Light Reflectance (inside):	19%
Shading Coefficient:	0.38	Total Solar Transmittance:	17%
Relative Heat Gain:	80	Total Solar Reflectance (outside):	31%
Light to Solar Gain:	1.15	Ultraviolet Transmittance:	NA

Contact Oldcastle BuildingEnvelope™ at 866-Oldcastle (653-2278) for samples or additional information concerning performance, strength, deflection, thermal stress or application guidelines. GlasSelect® calculates center of glass performance data using the Lawrence Berkeley National Laboratory (LBNL) Window 5.2 program (version 5.2.17) with Environmental Conditions set at NFRC 100-2001. Gas Library ID#1 (Air) is used for Insulating Glass units with air. Gas Library ID#9 (10% Air/90% Argon) is used for Insulating Glass units with argon. Monolithic glass data is from the following sources: 1. LBNL International Glazing Database (IGDB) version 17.3; 2. Vendor supplied spectral data files. Laminated glass data is from the following sources: 1. LBNL International Glazing Database (IGDB) version 17.3; 2. LBNL Optics 5 (version 5.1 Maintenance Pack 2); 3. Vendor supplied spectral data files; 4. Vendor supplied data.

Proposed Spandrel Unit Components:

PRODUCTS

Approved Glass Fabricator Oldcastle BuildingEnvelope™

Glass Description FLOAT GLASS

1. USA - Annealed float glass shall comply with ASTM C1036, Type I, Class 1 (clear), Class 2 (tinted), Quality-Q3. Canada - Annealed float glass shall comply with CAN/CGSB-12.3-M, Quality-Glazing.

2. USA- Heat-strengthened float glass shall comply with ASTM C1048, Type I, Class 1 (clear), Class 2 (tinted), Quality Q3, Kind HS. Canada - Heat-strengthened float glass shall comply with CAN/CGSB-12.9-M, Type 2-Heat-Strengthened Glass, Class A-Float Glass.

3. USA - Tempered float glass shall comply with ASTM C1048, Type I, Class 1 (clear), Class 2 (tinted), Quality Q3, Kind FT. Canada - Tempered float glass shall comply with CAN/CGSB-12.1-M, Type 2-Tempered Glass, Class B-Float Glass.

4. USA - Laminated glass to comply with ASTM C1172. Canada - Laminated glass to comply with CAN/CGSB-12.1-M, Type 1-Laminated Glass, Class B-Float Glass.

5. Glass shall be annealed, heat-strengthened or tempered as required by codes, or as required to meet thermal stress and wind loads.

Monolithic Vision Glass (vertical) GENERAL

1.Glass heat-treated by the horizontal (roller hearth) process shall have the inherent roller wave distortion running parallel to the bottom edge of the glass as installed when specified.

MONOLITHIC VISION LITE PERFORMANCE CHARACTERISTICS

1. Monolithic Lite 1/4" Pilkington Energy Advantage™ Low-E #2

2. Performance Characteristics

Thermal		Optical	
Winter U-factor/U-Value (Btu/hr-ft²-F°):	0.64	Visible Light Transmittance:	82%
Summer U-factor/U-Value (Btu/hr-ft²-F°):	0.49	Visible Light Reflectance (outside):	10%
Solar Heat Gain Coefficient:	0.70	Visible Light Reflectance (inside):	11%
Shading Coefficient:	0.81	Total Solar Transmittance:	66%
Relative Heat Gain:	169	Total Solar Reflectance (outside):	10%
Light to Solar Gain:	1.17	Ultraviolet Transmittance:	49%

Contact Oldcastle BuildingEnvelope™ at 866-Oldcastle (653-2278) for samples or additional information concerning performance, strength, deflection, thermal stress or application guidelines. GlasSelect® calculates center of glass performance data using the Lawrence Berkeley National Laboratory (LBNL) Window 5.2 program (version 5.2.17) with Environmental Conditions set at NFRC 100-2001. Gas Library ID#1 (Air) is used for Insulating Glass units with air. Gas Library ID#9 (10% Air/90% Argon) is used for Insulating Glass units with argon. Monolithic glass data is from the following sources: 1. LBNL International Glazing Database (IGDB) version 17.3; 2. Vendor supplied spectral data files. Laminated glass data is from the following sources: 1. LBNL International Glazing Database (IGDB) version 17.3; 2. LBNL Optics 5 (version 5.1 Maintenance Pack 2); 3. Vendor supplied spectral data files; 4. Vendor supplied data.

DOW Building Solutions

NORTH AMERICA – STYROFOAM™ BRAND CAVITYMATE™ INSULATION

Blue4Green design
Systems and Solutions
Products
Insulation
Rigid Foam
Spray Polyurethane Foam Insulation
Housewrap/Building Wrap
Spray Foam & Sealants
Adhesives
Specialty Products
Applications
Warranty Information
CAD Details
Guide Specifications
Tools
Installation Instructions
Literature and MSDS
American Recovery Investment Act Certification Statement
Answer Center
Energy Tax Credits
Contact Us
Continuing Education

STYROFOAM™ Brand CAVITYMATE™ Insulation is produced in a special 16" (400 mm) width, making it easy to fit between brick ties in cavity wall applications. This Type X extruded polystyrene foam solution is designed for use in wet cavity wall environments, offering high moisture resistance, durability and excellent thermal performance. Available in butt edge and shiplap edge treatments.

Building Code Compliance

Complies with ASTM C578 Type X. Meets IBC/IRC requirements for foam plastic insulation. Meets CAN/ULC S701 Type 3. See ESR-2142, BOCA-ES RR 21-02. UL Classified, see Classification Certificate D369.

3D Model Using STYROFOAM™ CAVITYMATE™ Exterior Wall Insulation (345KB)
This 3D Model uses Google SketchUp. Please visit the Google Web site to download SketchUp.

STYROFOAM™ Brand CAVITYMATE™ CAD Details

Related Information

Increase Energy Efficiency With STYROFOAM™ Brand CAVITYMATE™ Products (Brochure) (870KB PDF)
Isolant De Murs À Cavité – Assemblage De Blocks De Béton Spécification-type abrégée (Unit Masonry Assembly Cavity Insulation Guide Specification – French) (17KB PDF)
STYROFOAM™ CAVITYMATE™ Extruded Polystyrene Insulation Product Information (102KB PDF)
STYROFOAM™ Extruded Polystyrene Foam Insulation – 50 Year Thermal Limited Warranty (206KB PDF)
STYROFOAM™ Extruded Polystyrene Foam Insulation – 30 Year Thermal Limited Warranty (202KB PDF)
Unit Masonry Assembly Cavity Insulation Guide Specification (17KB PDF)
Tech Solution 510.0 STYROFOAM™ Brand CAVITYMATE™ Insulation Products for Steel Stud Cavity Walls (919KB PDF)

U.S. COMMERCIAL | CANADA COMMERCIAL

Specially designed for wet cavity wall environments, STYROFOAM™ Brand CAVITYMATE™ Insulation is produced in a special 16" width that fits easily between brick ties in cavity wall applications.

Nominal Board Thickness(1), in	R-Value(2)	Board Size, ft	Edge Treatment	Min Compressive Strength(3), psi
1.0	5.0	16 x 96	Butt Edge/Shiplap	15
1.5	7.5	16 x 96	Butt Edge/Shiplap	15
2.0	10.0	16 x 96	Butt Edge/Shiplap	15
3.0	15.0	16 x 96	Butt Edge/Shiplap	15

(1) Not all product sizes are available in all regions.

(2) R means resistance to heat flow. The higher the R-value, the greater the insulating power. R-values are expressed in $ft^2 \cdot h \cdot °F/Btu$. R-value determined by ASTM C518.

(3) Vertical compressive strength is measured at 10% deformation (5% for STYROFOAM™ Brand PLAZAMATE™ Insulation and for STYROFOAM™ Brand HIGHLOAD 40, 60 and 100 Insulation products) or at yield, whichever occurs first. Since STYROFOAM™ Brand Extruded Polystyrene and Dow polyisocyanurate insulation products are visco-elastic materials, adequate design safety factors should be used to prevent long-term creep. For static loads, 3:1 is suggested. For dynamic loads, 5:1 is suggested.