Weill Cornell Medical Research Building
413 E. 69th Street
New York, NY

Jonathan Coan

Structural Option

Advisor: Dr. Boothby

Final Thesis Report

Submitted: 4/10/12

Weill Cornell Medical Research

413 East 69th Street

New York, NY

Occupant: Weill Cornell Medical College
Type: Laboratory/Research Facility
Size: 455,000 ft^2
Number of Stories: Below Grade – 3
Above Grade – 18 + Penthouse
Cost: $650 Million
Dates of Construction: 2010-2014
Delivery Method: Design-Bid-Build

Project Team

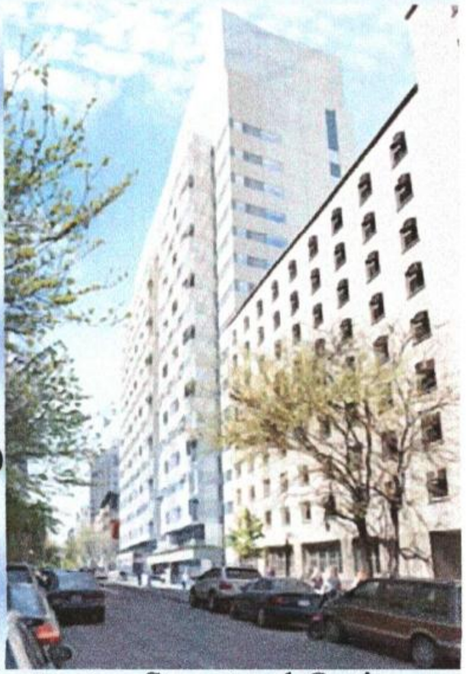
Architect:	Ennead Architects
Structural Engineer:	Severud Associates
Mechanical Engineer:	Jaros Baum & Bolles
Laboratory Consultant:	Jacobs Consultancy/GPR
Construction Manager:	Tishman Construction

Architecture:

- Undulating glass sunshade curtain wall
- 13 floors of laboratory space
- 2 floor underground animal facility
- 2nd floor terrace connects to neighboring Lasdon House
- Doubles Weill Cornell Medical College's existing research space
- LEED Silver Rating upon completion

Structure:

- Foundation: Spread footing with rock anchors to resist water table uplift
- Floor System: Two way flat plate concrete slabs
- Lateral System: Reinforced Concrete Shear walls
- Concrete columns in various sizes with typical bays of 21'-0" (East-West) x 27'-7," 25'-0," and 16'-3" (North-South)
- Concrete beams in various sizes located as needed



MEP Systems:

- (13) Variable and control volume air handling units totaling 675,000 cfm; (4) Cooling towers on the roof totaling 1,228,000 cfm
- 265/460V electrical service brought to the building via four 4000A switchboards
- Fluorescent lighting throughout the building mostly at 277V

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Executive Summary

The Weill Cornell Medical Research Building is a 19 story, 455,000 square foot, 294'-6" tall building located on East 69th Street in New York City. The building features three stories below grade and eighteen, plus a penthouse and an interstitial floor, above grade.

The purpose of this thesis was to design a pre-stressed concrete alternative to the existing reinforced concrete two-way flat plate floor slab. Two such systems were investigated. A banded beam system and a two-way post-tensioned flat plate slab were designed. It was determined that the two-way PT slab would be the better of the two alternatives. The criteria for the viability of these alternatives was the elimination of the need to camber the concrete slab for the front cantilever while still meeting deflection requirements and limiting floor-to-floor heights. This was accomplished. The slab was decreased in thickness from 12 ½ inches to 10 inches. This has the added benefit of allowing more flexibility for MEP equipment and reducing the amount of concrete needed for the structure.

Following that, investigations were made into the possibility of altering the size of the massive 14 x 72 columns from which the cantilever extends, and into the removal of the columns in Row B. It was determined that the 14 x 72 columns are necessarily large and surprisingly well utilized. The investigation into the removal of the Row B columns showed that deflections would be much too severe and the idea was deemed not possible.

Finally, mechanical and architectural studies were conducted on the enclosure system resulting in a redesign of the system from a brick cavity wall to an EIFS wall system. The goals of the design of the new exterior wall system were to reduce the chance of condensation in the air space (a danger posed by the existing system), reduce the amount of heat loss and gain of the wall, and to create a thinner and lighter system. The new EIFS system performs better in preventing condensation and reducing heat transfer through the wall and was also deemed to be architecturally more versatile and more becoming of the Weill Cornell Medical Research Building.

Acknowledgements

I would like to thank the following people for their help throughout the year on this project:

Severud Associates

Steve Reichwein

Janice Clear

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Dr. Linda Hanagan – Academic Advisor

I was also like to thank my friends and family, without whom I wouldn't be where I am or who I am today.

Introduction

The Weill Cornell Medical Research Building is the newest addition to the campus of the Weill Cornell Medical College on the upper east side of Manhattan. Located at 413 East 69th Street in New York City, the Medical Research Building is adjacent to other Weill Cornell buildings. The Weill Greenberg Center on its northeast side is an educational facility designed by the same architects as the Medical Research Building. Olin Hall to the east, and the Lasdon House to the north are residential buildings that house students of the medical college. 69th Street slopes down to the east across the site of the Medical Research Building and the utilities run under it. The Con. Edison power vaults are also located under 69th Street and the sidewalk in front of the building.

The $650 million Medical Research Building is approximately 455,000 square feet with three stories below grade and eighteen, plus a penthouse and an interstitial floor, above grade. The total height of the building above grade is 294'-6." Floors 4-16 are dedicated to laboratory space. The first basement level, as well as the interstitial floor between floors 16 and 17, and the 17th and 18th floors are designated as mechanical floors. The bottom two levels of the basement contain the MRB's animal facility. Service and freight elevators and vertical circulation are located on the west side of the building next to the loading docks on the 69th Street side. Passenger elevators and vertical circulation are nearer the center of the building where the two story lobby atrium welcomes people into this hub of scientific exploration.

In the rear of the building, adjoining the second floor, there is a terrace that bridges the gap between the rear façade of the MRB and the Lasdon House. A grand staircase leads from the lobby on the ground floor up to the enclosed lounge on the second floor that opens onto the terrace. There are two entryways from the Lasdon House to the terrace so anyone living in that building and working in the Medical Research Building would have easy access. The terrace also wraps around the side of the Lasdon House and connects to a stairway leading down to the sidewalk on 70th street.

The building is defined visually by the undulating glass sunshade curtain wall across the front of the building. This curtain wall is attached to the floor slabs that are cantilevered out approximately 9’-8” from the exterior row of columns to meet it. The curtain wall itself has two layers. The outer layer features the glass sunshade wall with aluminum mullions. This wall is tied to the inner layer of insulated glass (also with aluminum mullions) by aluminum struts. The inner layer is anchored to the slab either directly through the mullion or with a steel outrigger.

Structural Systems

Foundation System

The foundation system consists of spread footings bearing on undisturbed bedrock. Strap beams are provided as necessary around the perimeter. This undisturbed bedrock is expected to support 40 tons per square foot. According to the geotechnical report, there are two types of bedrock encountered on the site. One type supports 40 tsf and the other 60 tsf, but it is recommended by Langan Engineering and Environmental Services that the footings be designed to rest on 40 tsf bedrock. The slab on grade is a 6” concrete slab resting on a 3” mud slab on 24” of crushed stone. The perimeter concrete walls of the basement are 20” thick with strip footings. Below, Figure 1 is an image of the foundation plan.

The geotechnical report also states that the water table is approximately 50 feet above the foundation level. This poses the problem of seepage through the rock and also uplift on the foundation. A few different design solutions are presented in the geotechnical report. The resolution of this problem comes in the form of 4-50 ton rock anchors located at the bottom of Stairwell B on the East side of the building to resist the uplift.

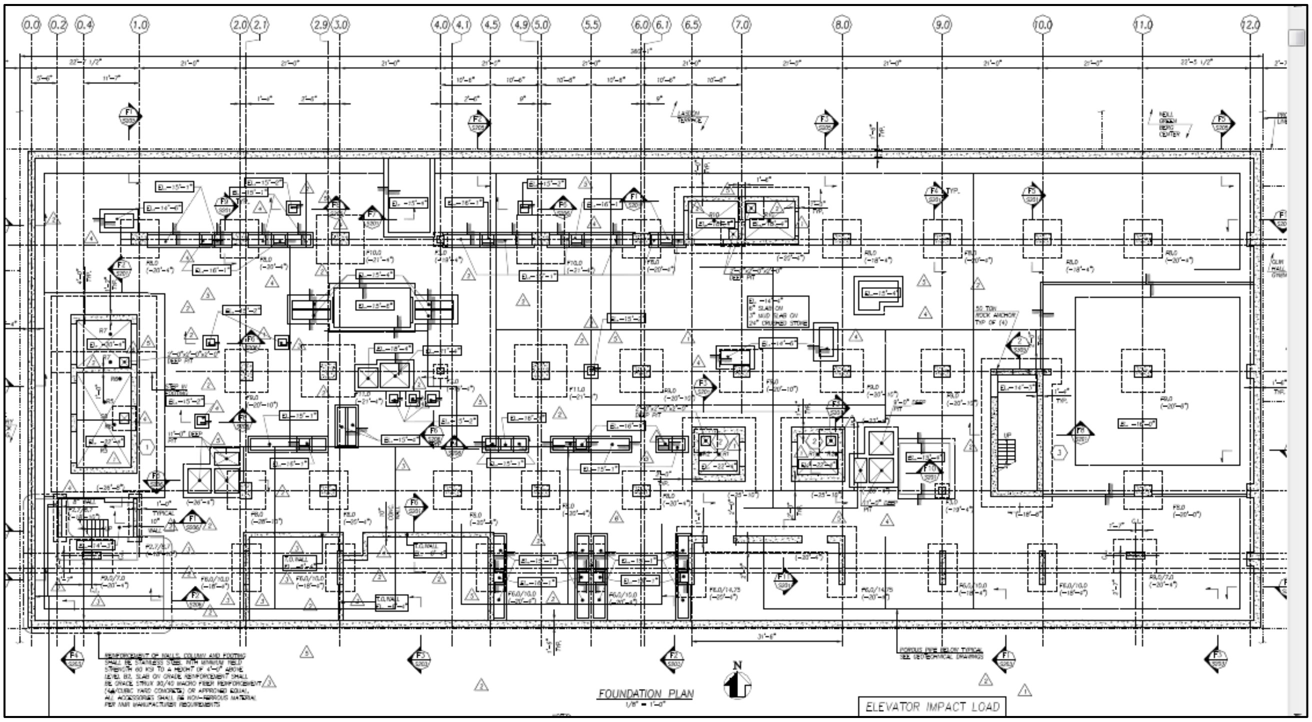


Figure 1: Basement Level 3 – Foundation Plan

Floor System

The floor system in the Medical Research Building is 2 way flat plate concrete slabs. These slabs vary in depth from floor to floor (see Figure 2 below). The bottom reinforcement is typically #5 bars at 12." Top reinforcement and additional bottom reinforcement varies as needed throughout the building. The slabs are especially thick in this building because much of the design was constrained by strict vibration requirements of the medical and research equipment in the building. Laboratory floors were designed to limit vibration velocities to 2000 micro-inches per second. Walking paces were assumed to be moderate (75 footfalls per minute) in the labs and corridors and fast (100 footfalls per minute) only in public areas such as the lobby. There are also vertical HSS members at alternate floors through the middle of the building where the laboratories are located. These members serve no structural load bearing purpose, they are simply meant to tie each floor to another floor to further limit vibrations by forcing any impact to excite vibrations in two floors instead of just one.

Floor	Slab Depth (in)
B3	6
B2	12.5
B1	12.5
1	11
2	12
3	12.5
4	12.5
5	12.5
6	12.5
7	12.5
8	12.5
9	12.5
10	12.5
11	12.5
12	12.5
13	12.5
14	12.5
15	12.5
16	12.5
Interstitial	10.5
17	10.5
18	12.5
19	10.5

Figure 2: Slab Depth per Floor

The front of the building features a cantilever slab extending approximately 9'-8" from the center of column line D. The glass sunshade curtain wall is connected to the edge of the slab. The slab is the same thickness as the rest of the floor, but is cambered up to reduce deflections caused by the curtain wall load. On the second floor, the slab is cambered 1" upward. For the third through the interstitial floors, the slab is cambered 5/8" upward.

Lateral System

Lateral loads, such as seismic and wind loads, are primarily resisted by 14"-16" reinforced concrete shear walls located around the stairwells and elevator cores. A couple of these shear walls step in at the second floor. Extra precautions were taken to make sure that the lateral moment still has a viable path to travel through that step in. Severud, the structural engineers for the project, desired to transfer lateral loads toward the perimeter of the building. In the front of the building there are massive 14 x 72 inch columns from which the slabs cantilever out and the glass sunshade curtain wall is hung. These columns also take some of the lateral loads. See the sketch in Appendix E for the location of lateral system elements on a typical floor.

Beams and Columns

There is a very wide variety of beam and column sizes in this building. There are almost forty different sizes of columns with dimensions ranging from 12” to 84,” with the most common column being 24 x 36. There are also approximately fifty five different sizes of beams ranging from 8 x24 to 84 x 48. Except on the laboratory floors, which are quite uniform, the column sizes tend to change from floor to floor. Reinforcement was provided to ensure the continuity of the load path through these column transfers.

Columns are located on the specified grid of 4 major rows in the East-West direction for the majority of the floors—except the first floor and below grade, which have a fifth row in the back of the building. Bay sizes are 27’-7,” 25’-0,” and 16’-3” in the North-South direction and the typical bay in the East-West direction is 21’-0” with end spans approximately 22’-6.” Beams, however, are only placed where they are needed. They are rarely in the same place from floor to floor and each floor has a different number of beams. The fourth floor has the fewest with 6, and the second floor has the most with 33. Below in Figure 3 is a typical framing plan for the 5th-15th floors.

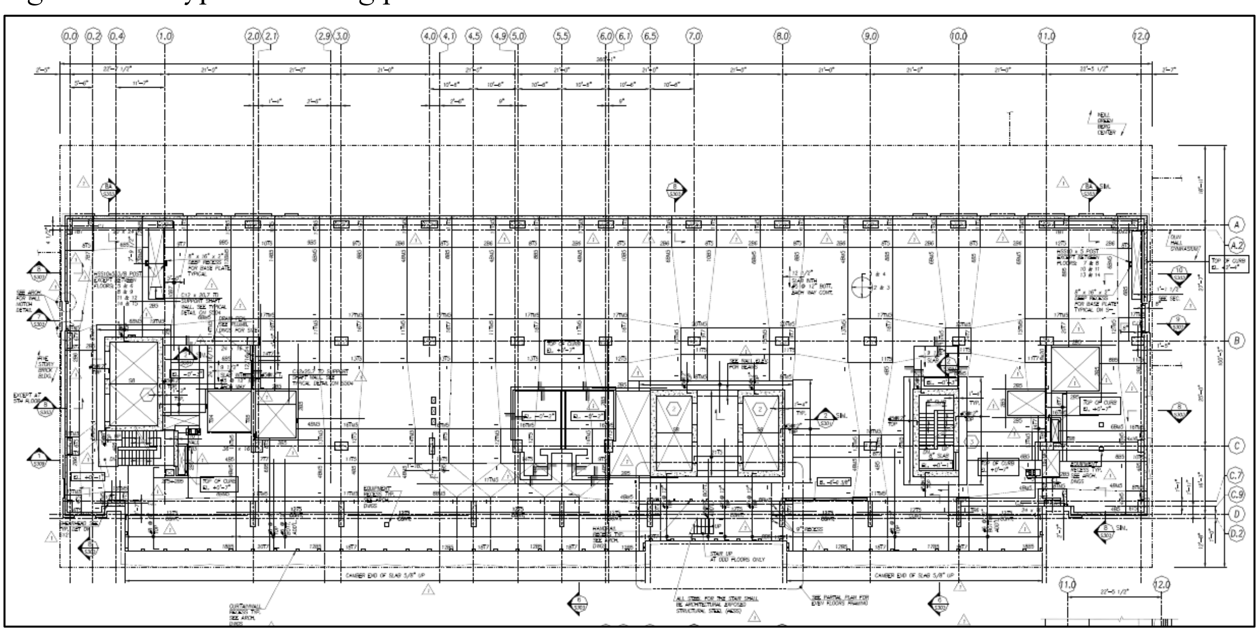


Figure 3: Typical Framing Plan – 5th-15th Floors

Design Codes and Standards

The Weill Cornell Medical Research Building was designed according to the 1968 New York City Building Code based on the UBC. In 2008 New York City updated their building code, which is now based on the IBC. For this report, the new 2008 code for analysis and design is being used; which references ASCE 7-02, ACI 318-02, etc. For relevance, ASCE 7-05, ACI 318-08, and the AISC Steel Construction Manual 14th ed. will be referenced in this report. The design for the Medical Research Building was submitted in 2008 and the project team decided to file under the old code. The MRB is located in New York City's zoning district R8, the use group is 3 (college), the construction class is I-C, and the occupancy group is D-2.

Proposal Objectives

Problem Statement

Technical Reports one, two, and three showed that the structural systems of the Weill Cornell Medical Research Building are adequate for both strength and serviceability requirements. However, there is one portion of the design that, while not an unsuccessful solution, has sufficient potential flaws to require a further investigation into alternative solutions.

In the existing design of the floor system, a two-way flat plate slab is used to minimize floor to floor heights. The nearly ten foot cantilever on the front of the building, from which the curtain wall is hung, presents a challenging problem of resolving deflection issues. The existing design solution calls for a 5/8" camber of the concrete slab for the cantilever portion. This solution has multiple potential hazards. First, cambering concrete is not an exact science. One can only really design the camber to balance deflection from dead loads however the slab is subjected to live loads from the offices and conference rooms located on the cantilever as well. There is also the issue of constructability. It is difficult to execute a cambered cantilever slab on a job site and inspections would have to be made that it was done well enough, and uniform enough throughout the floors. Any errors in construction or too much deflection in the cantilever could have disastrous effects on the glass sunshade curtain wall that is tied to the edge of the cantilevered slabs at every floor.

Lastly, this solution is very vulnerable to creep. Concrete is infamous for creep under sustained load. In this design situation, the load of the curtain wall exacerbates the creep problem. If another floor system could be designed to eliminate the need for manually cambering a concrete slab, while still meeting deflection requirements for the cantilever section and keeping floor to floor heights at a minimum, this system would be a viable alternative.

Problem Solution

In the second technical report, multiple floor systems were assessed based on structural and non-structural criteria and it was deemed that a post-tensioned system was the best alternative to the two-way flat plate slab system. Two types of post-tensioned floor systems will be investigated: a banded beam system and a post-tensioned slab system. The inherent deflection-reducing characteristics of post-tensioned concrete make it a good solution for the cantilever for multiple reasons. A post-tensioned floor system would reduce the deflections and reduce the significance of creep. It is also easier from a constructability point of view than a cambered two-way slab. These post-tensioned systems will be designed using methods learned in AE 431 as well as design guides and computer programs.

Due to the post-tensioned system's ability to cover large spans, after designing the new floor system, a couple of options will be assessed regarding the column layout. First, it will be determined if the 14x72 columns that support the cantilever can be reduced in size while still maintaining deflection requirements. Since these columns take lateral loads as well, if they change in size, the building's lateral system will also need to be reevaluated. Second, the impact of removing column row B in the laboratory spaces will be considered. This will create an elongated span adjacent to the cantilever span and could help balance deflections. Eliminating a row of columns will also potentially have effects on the sizes of the remaining columns. The magnitude of these effects will also be investigated.

Breadth Topic: Building Enclosure

The breadth investigation will be a redesign of the brick cavity curtain wall system that encloses three of the four sides of the building (the fourth side being the front with its undulating glass curtain wall). The new curtain wall system will be designed according to structural, mechanical, and architectural criteria. Methods of design and analysis learned in AE 542 will be essential to the completion of this breadth study. First the new system's fenestration will be designed to withstand lateral loads due to wind, seismic, impact, and blast events. Next, the moisture and heat transfer characteristics of the curtain wall will be examined using H.A.M. Toolbox (this will constitute the mechanical portion of the breadth). After the new curtain wall design is completed, a Revit model of both systems will be produced in order to compare the architectural features of the new system with the old (this will constitute the architectural portion of the breadth).

MAE Course Related Study

Information acquired in AE 597A will be applied in order to model the building in either ETABS, SAP, or RAM. Understanding the methods by which the computer program arrives at its solution allows for a better analysis and scrutiny of results. Also, methods and tools learned in AE 542 will be used in order to complete the redesign of the building enclosure and analyze it.

Structural Depth

For the redesign of the floor system, there were three floors chosen to design as representative of the entire structure. The Typical Floor refers to floors 3 through 16, which are, structurally, essentially identical. The 17th and 18th floors, which serve as two of the mechanical floors at the top of the building, were chosen to be specifically designed for loading conditions that were characteristic of the other floors. The dead and live loads used in the design of these floors are summarized in the chart below (Figure 4). All concrete used in the design was chosen to have $f'_c = 4000$ psi.

Level	Dead Load (psf)	Live Load (psf)
Typical Floor	27, 47	60, 150
17th Floor	97	150
18th Floor	107	400

Figure 4: Design Loads for Floor System Redesign

Banded Beam System

The first investigation conducted for the redesign of the floor system was the banded beam. This system consists of a uniform slab with thickened portions along column lines (usually in the long direction). These thickened portions are typically post-tensioned and called "band-beams." All reinforcement used in this floor system is Grade 250 Seven-wire strands.

The one-way slabs were designed first using the method taught in AE 431 for the design of one-way pre-stressed slabs. The maximum compressive stress in the concrete was assumed to be $.45*f'_c = 1800$ psi. For the reinforcement, 18 - .196" strands were used with $f_{pu} = 250$ ksi and $f_{pi} = .7*f_{pu} = 175$ ksi. Pre-stress losses were assumed to be 15%. Initial thicknesses of the slabs were determined using the rule of thumb $t \leq L/45$, and then adjusted as necessary. There were 5 span conditions assessed in order to represent the various loading conditions on the design floors. These include the Typical Span of the Typical Floor, the longer End Span of the Typical Floor, Higher Load Areas of the Typical Floor, and the typical spans of the 17^{th} and 18^{th} Floors. An excel spreadsheet was developed to carry out the various calculations. The slabs were checked for stresses at transfer, stresses after losses, and ultimate load in flexure. The results of this design and analysis are summarized in the chart below (Figure 5). For the details of the calculations, see the spreadsheets in Appendix A.

Location	Superimposed Dead Load (psf)	Live Load (psf)	Thickness (in)	Prestressing	Spacing (in)
Typical Span	27	60	8	18 - .196"	17
Typical Floor (End Span)	27	60	8	18 - .196"	15
Higher Load Areas	47	150	8	18 - .196"	15
17th Floor	97	150	10	18 - .196"	20
18th Floor	107	400	14	18 - .196"	15.50

Figure 5: Summary of One-Way Pre-stressed Slab Design

Next the band-beams were designed using the method taught in AE 431 for the design of pre-stressed beams. The typical horizontal column-to-column distance is 21 feet and it was decided that the beam (or thickened portion of the slab) would be six feet wide, which is just under a third of the of the beam's tributary distance. All of the beams were designed with bundles of 12 – 3/8" strands spaced evenly along the width of the member. The design span conditions for the beams consist of the Typical, Edge Beam, Higher Load, and Cantilever spans of the Typical Floor, as well as the typical spans of the 17th and 18th Floors. Again, an excel spreadsheet was developed to carry out the calculations for the design of the beams. A summary of the design is featured in the chart below (Figure 6). For the details of the calculations see the spreadsheets in Appendix A.

Location	Superimposed Dead Load (psf)	Live Load (psf)	Beam Height (in)	Reinforcement Depth (in)	A_p (in^2)	Tendon Spacing (in O.C.)	M_u (kip-ft)	ϕM_n (kip -ft)
Typical	27	60	14	11.5	5.76	12	351	787
Edge Beam	27	60	14	11.5	2.88	24	232	393
Cantilever	27	60	14	11.5	5.76	12	294	787
Higher Load Areas	47	150	14	11.5	5.76	12	574	787
17th Floor	97	150	14	11.5	5.76	12	654	787
18th Floor	107	400	16	13.5	11.52	6	1214	1360

Figure 6: Summary of Band-Beam Design

Two-Way PT Flat Plate Slab

The second system investigated for the floor system redesign was a two-way post-tensioned flat plate slab. In order to design and model the post-tensioned slab, RAM Conce*pt* was used. Produced by Bentley Systems, Inc., RAM Conce*pt* is a wonderful tool for both reinforced and pre-stressed concrete analysis and design. Conce*pt* uses the finite element method for elevated concrete floor systems. While other structural analysis problems reduce the structure to a frame model in order to be solved, Conce*pt*'s use of the finite element method produces more precise results, especially when the structure is irregular.

The two-way post-tensioned slab was modeled following the tutorial provided in the RAM Conce*pt* Manual for modeling of two-way PT flat plate slabs using ACI 318-05. First, the floor plans were drawn in AutoCAD and imported into Conce*pt* (see Figure 7 for the Typical Floor plan drawn in AutoCAD). The structural elements, i.e. slab area, slab openings, columns, and shear walls, were defined by tracing the elements on the CAD drawing. A thickness of 10 inches was chosen for all two-way slabs. An initial mesh was

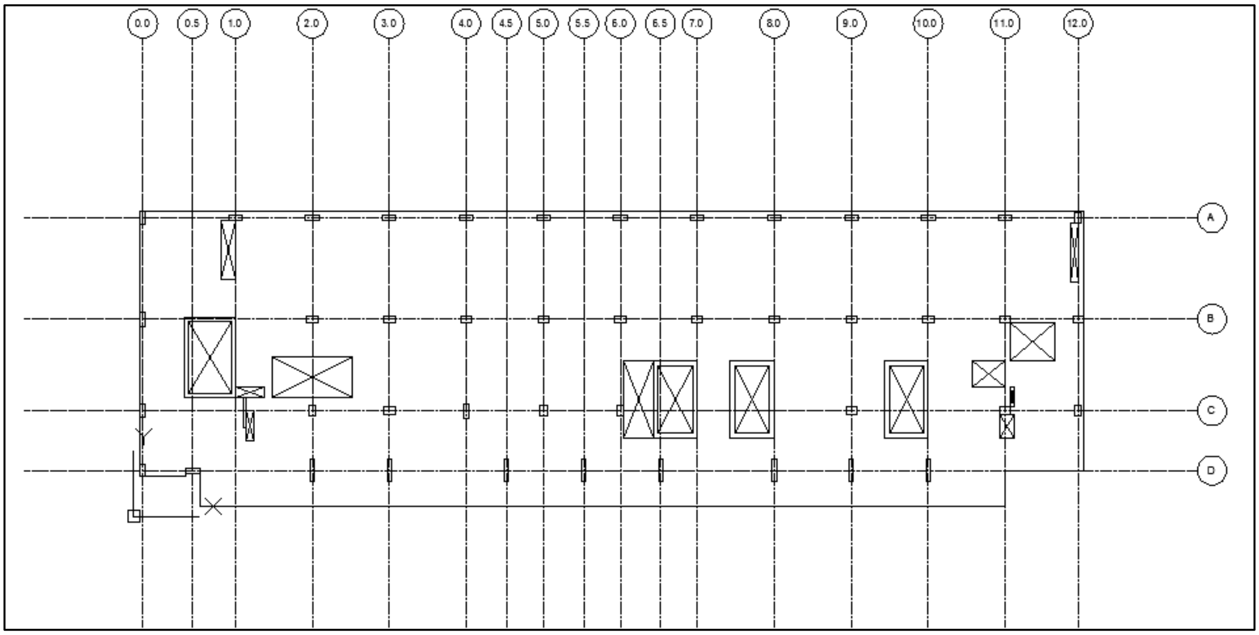


Figure 7: Typical Floor Plan Drawn In CAD

created and then the superimposed dead and live loads were placed.

Next the post-tensioning was designed. For all of the two-way slabs, bundles of 12 strands of ½" diameter unbonded tendons were used. Minimum clear cover top and bottom was assumed to be 1 ½ inches. RAM Conce*pt* designates the two perpendicular directions of reinforcement in a two-way slab as latitude and longitude directions. First the latitude pre-stressing was laid out along the horizontal column lines, and then the longitude pre-stressing was placed manually with a typical spacing. These pre-stressing tendons were then manually manipulated to avoid the openings in the floor slab. Conce*pt* then generated latitude and longitude design strips and column locations were noted for punching shear checks. The mesh was then regenerated and results calculated.

RAM Conce*pt* calculates results using all of the above inputs. It then designs the necessary non-prestressed reinforcement and checks all of the results against the chosen code, in this case ACI 318-05 (the most up-to-date version of the ACI code available in the version of RAM Conce*pt* used).

A simplified summary of the typical longitudinal reinforcement spacing, approximate maximum deflection, and approximate maximum moment is presented in the chart below (Figure 8). Also, diagrams of the mesh elements (slab, openings, columns, and shear walls), as well as of the distribution of deflection and moment are shown below (Figures 9-17). For the layout of both directions of reinforcement, see Appendix B.

Level	Long. Spacing	Max Deflection (in)	Max Moment (kip-ft)
Typical Floor	5-7 ft	0.225	500
17th Floor	5-6 ft	0.24	850
18th Floor	5-7 ft	0.24	1000

Figure 8: Simplified Summary of Two-Way PT Flat Plate Slab Design

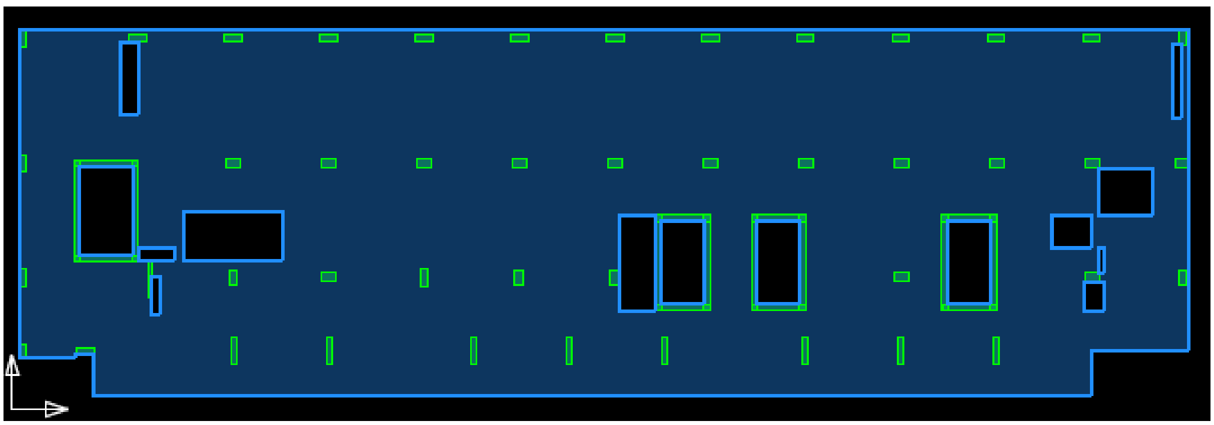


Figure 9: Typical Floor Plan

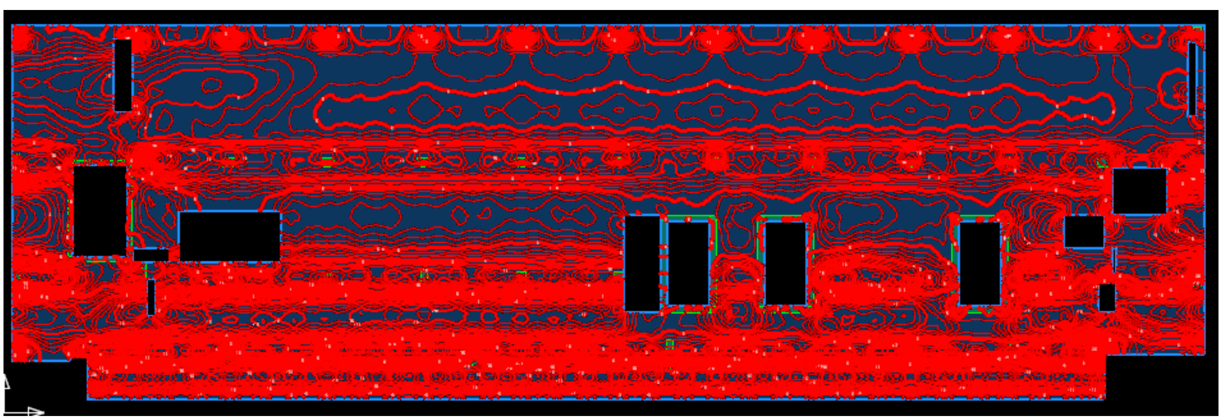


Figure 10: Typical Floor Moments

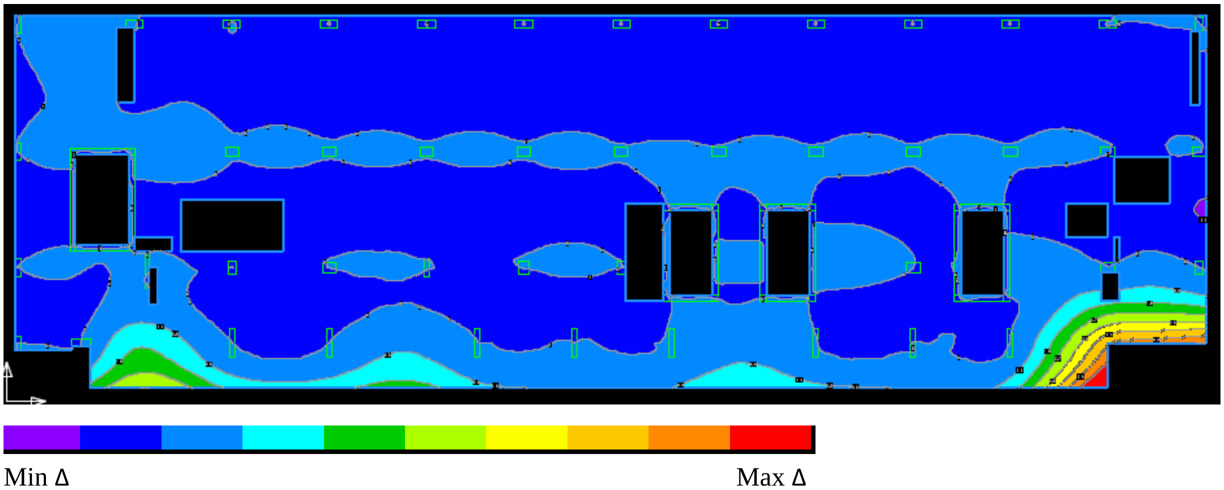


Figure 11: Typical Floor Deflections

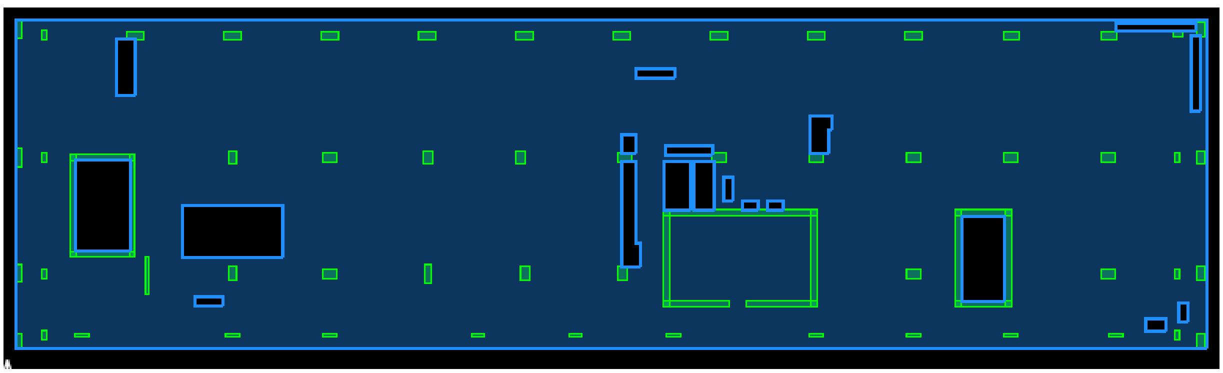


Figure 12: 17th Floor Plan

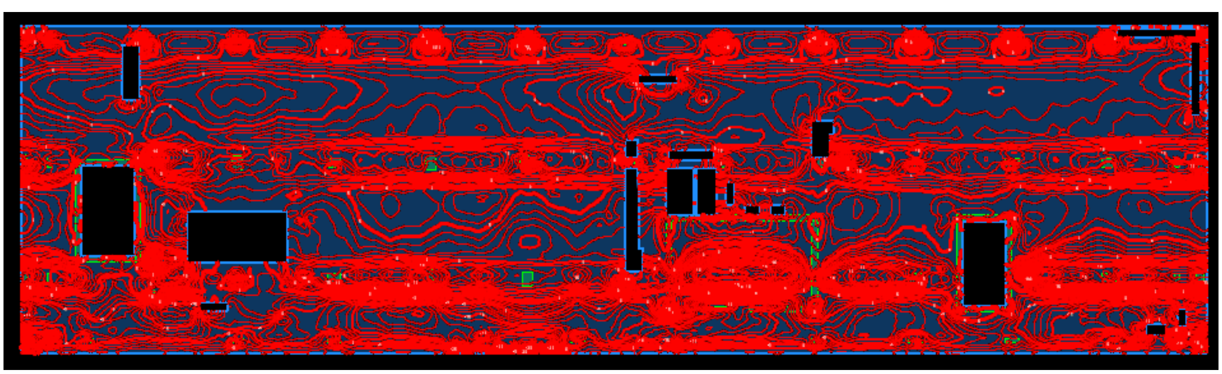


Figure 13: 17th Floor Moments

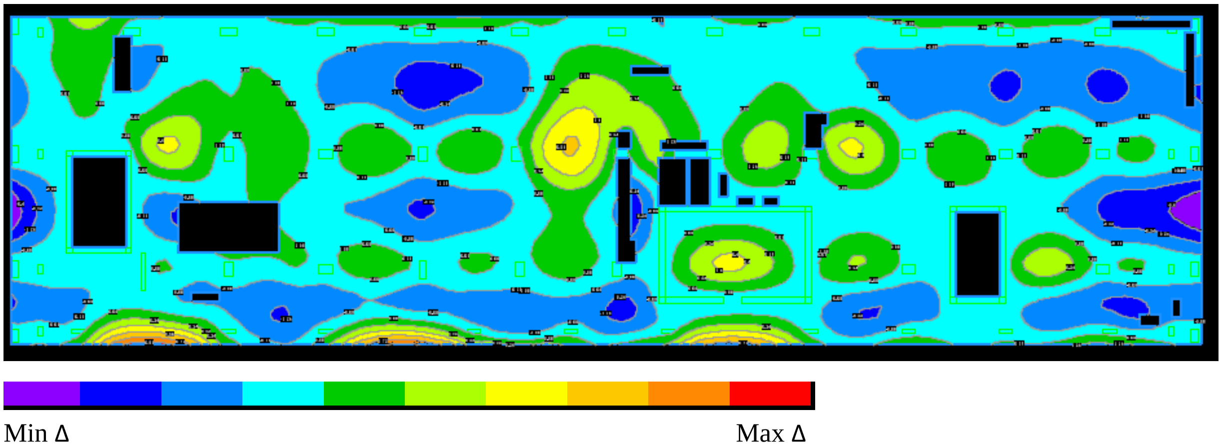


Figure 14: 17th Floor Deflections

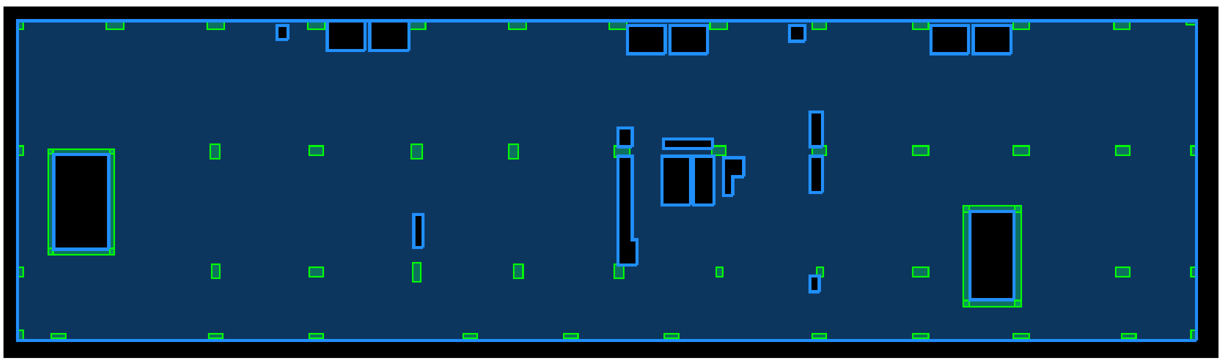


Figure 15: 18th Floor Plan

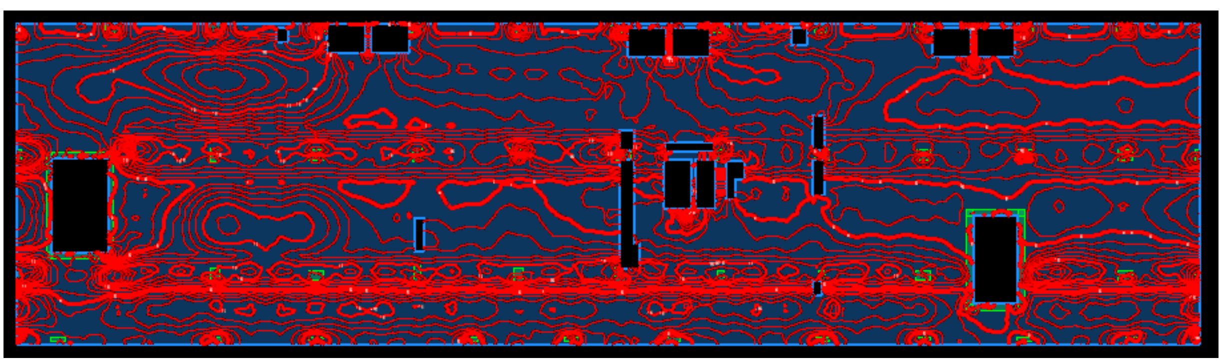


Figure 16: 18th Floor Moments

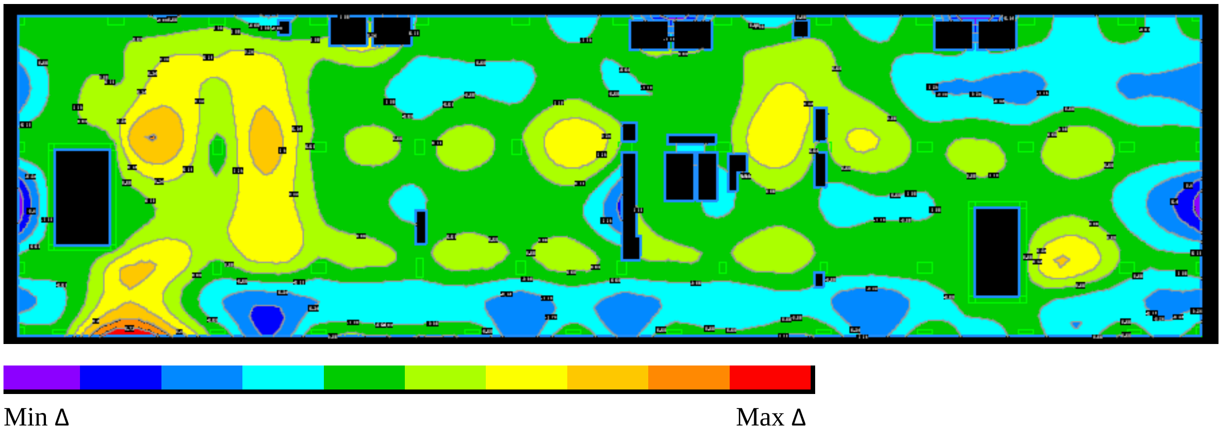


Figure 17: 18th Floor Deflections

Results and Comparison

The use of post-tensioning in the floor system proved to be a good alternative to the existing two-way flat plate slab design. Of the two post-tensioned systems investigated, the two-way flat plate slab worked best; echoing the best non-prestressed floor system option.

The banded beam system was successful in strength, showed negligible deflections (most of which were in the hundredths or thousandths of an inch), and fulfilled the necessary structural design criteria for this investigation. There would be no need to camber the concrete in the cantilever and, except on the 18th floor, less overall concrete would be needed. However, due to the complicated nature of the mechanical equipment in the laboratory spaces and the potential need for flexibility in MEP placement, it was decided that having beams along the column lines would restrict that MEP freedom and create the potential for overcrowding in the regions under the one-way slab, or force the floor-to-floor heights to increase.

For that reason, the post-tensioned two-way flat plat slab was chosen as the most viable alternative to the existing design. On the Typical Floors, there would be no need to camber the concrete. According to the results from RAM Concept, the cantilever would see the greatest deflections of any other region of the slab, but even those deflections were limited to approximately 0.225 inches, which is just less than L/480. The 17th and 18th floors showed their maximum deflections along the front of the building as well, but most of the deflections throughout all three slabs were in the hundredths and thousandths of an inch.

Along with eliminating the need to camber the slab, the PT flat plate slab has the advantage of significantly reducing the amount of concrete used in the building. The post-tensioned design features a ten inch slab which is a full 2.5 inches less than the typical depth of the existing slab design. Over 18 stories, that equates to 3.75 feet. According to the *Prestressed Concrete Designer's Handbook* (Abeles and Bardhan-Roy), "there is little difference between the initial costs of reinforced and prestressed members." However, with almost the entirety of the structure (including columns and shear walls) being concrete, that is a substantial decrease in the amount of concrete needed. Overall building height might not be an issue since the building is located in New York City, but at the very least, this PT flat plate slab provides more space and flexibility for MEP equipment.

According to the project structural engineer at Severud Assoc., prestressed concrete is not typically employed in New York City. This might be due to unions or perhaps to the extra labor involved in the jacking of the tendons and the potential for more job site inspections to ensure the post-tensioning process is carried out properly. Whatever the case may be, if post-tensioning were a more common practice in New York City, then a post-tensioned two-way flat plate slab would be the best choice of floor system.

Column Investigations

14 x 72 Columns:

At first, the idea of a six foot long, essentially (and literally in some spots) one foot wide, column makes one think, "that's not a column, that's a wall." Would these columns be eyesores, or simply an unnecessary amount of concrete? Also with its proximity to the edge of the cantilevered slab (9'-8" from the column centerline, would the space feel awkward?

Upon visiting the site in February of 2012, it was observed that, visually, the extremely long and awkwardly skinny columns did not feel out of place. Despite some concrete consolidation issues, which appear minor enough to avoid worries of spalling and that need some finishing touches, these columns actually look good. They are thin enough to blend with what will be the divisions between offices located on the cantilever. As can be seen in the partial floor plan below (Figure 18), the architect made full use of the columns to lay out the offices and provide as much potential window space and view as possible.

Upon investigating the structure and the post-tensioned slab system, it was determined that deflection was a critical criterion for the design of the cantilever and decreasing the size of the massive columns anchoring the cantilever would only exacerbate that situation. Therefore, it was decided that these columns should be left their original design size.

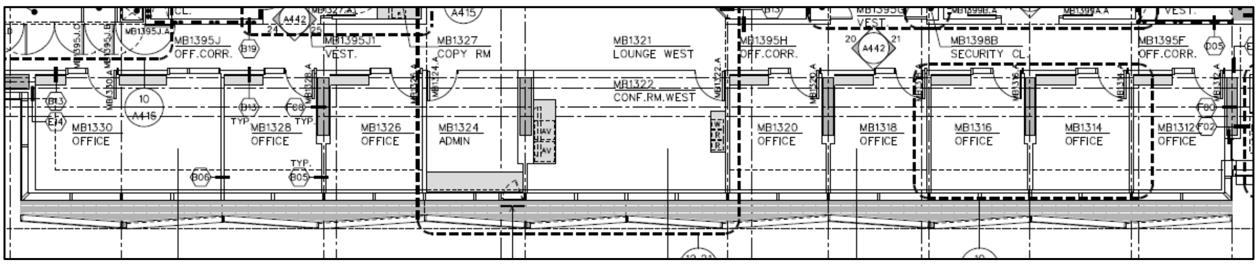


Figure 18: Partial Typical Floor Plan

Removing Row B Columns:

New Column Sizes and Designs

If the columns in row B were to be removed, this would invariably place a larger load on the columns in rows A and C. A typical column in the 3rd vertical row was chosen to analyze and redesign as a representative of the magnitude of the increase in load caused by removal of row B columns. Hand calculations with the use of design aids from *Reinforced Concrete Mechanics and Design* (Wight and MacGregor) were performed to size the new column and design the reinforcement (see Appendix C). The columns were assumed to only take axial loads and a reinforcement ratio of 2% was used.

For Column A3, the original size was 44 x 20 with an axial load of 1555 kips and 16 #9 bars. The new axial load was determined to be 2518 kips and the new Column A3 was designed to be 48 x 24 with 16 #11 bars (ϕP_n = 3464 kips). The original Column C3 was 36 x24, with an axial load of 1520 kips and 16 #7 bars. The new Column C3 was found to have an axial load of 2493 kips and was designed to be 42 x 28 with 16 #11 bars (ϕP_n = 3517 kips). The new column details were drawn in AutoCAD and can be seen below

(Figure 19). The columns were then modeled in SP Column in order to obtain the interaction diagrams. These results can also be viewed below (Figures 20 and 21).

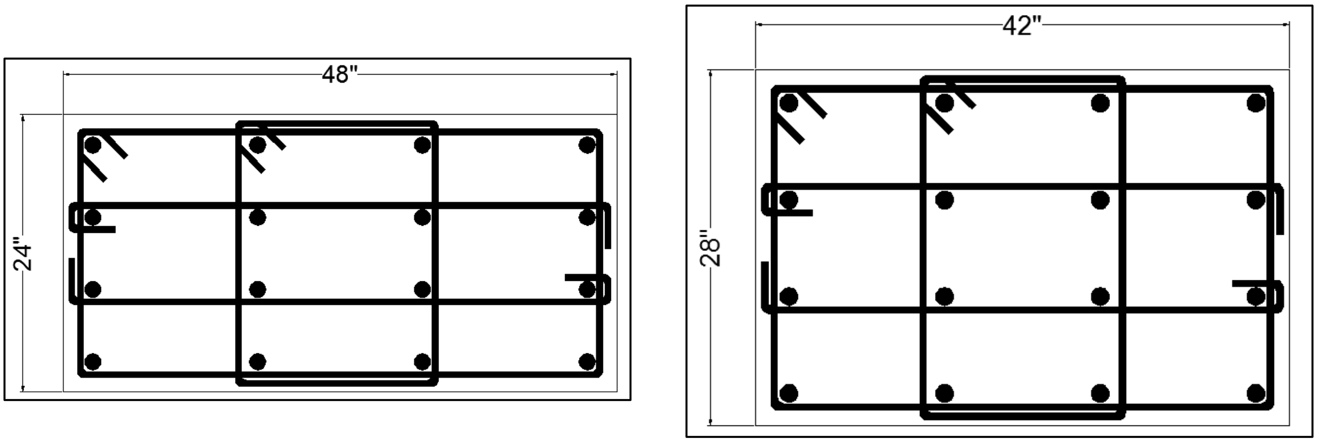


Figure 19: (Left) New Column A3, (Right) New Column C3

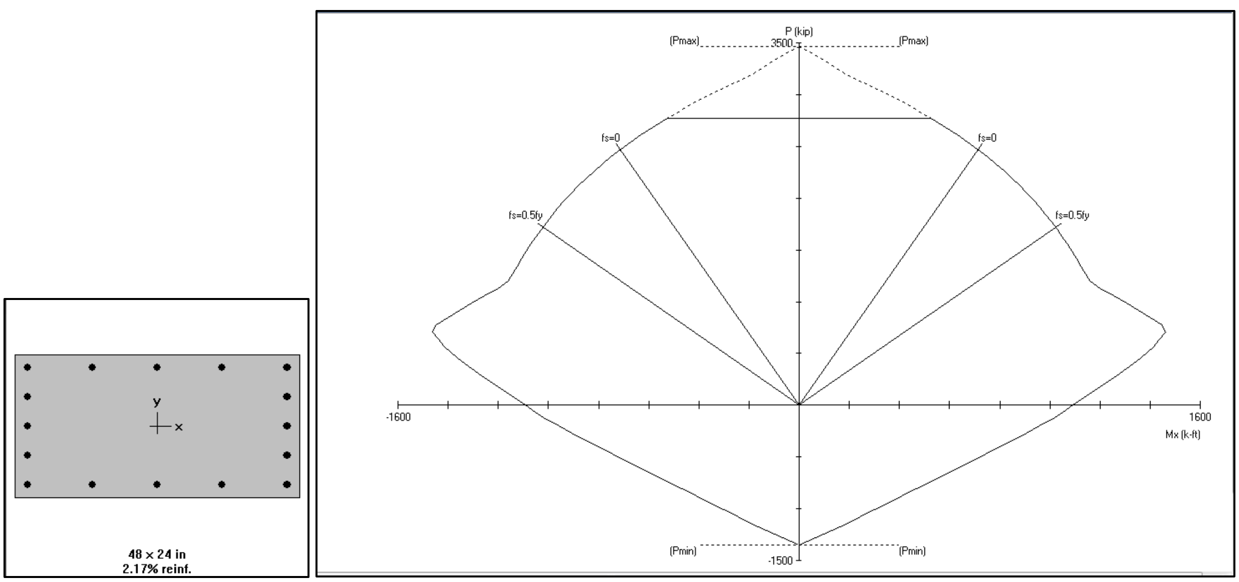


Figure 20: New Column A3 in SP Column

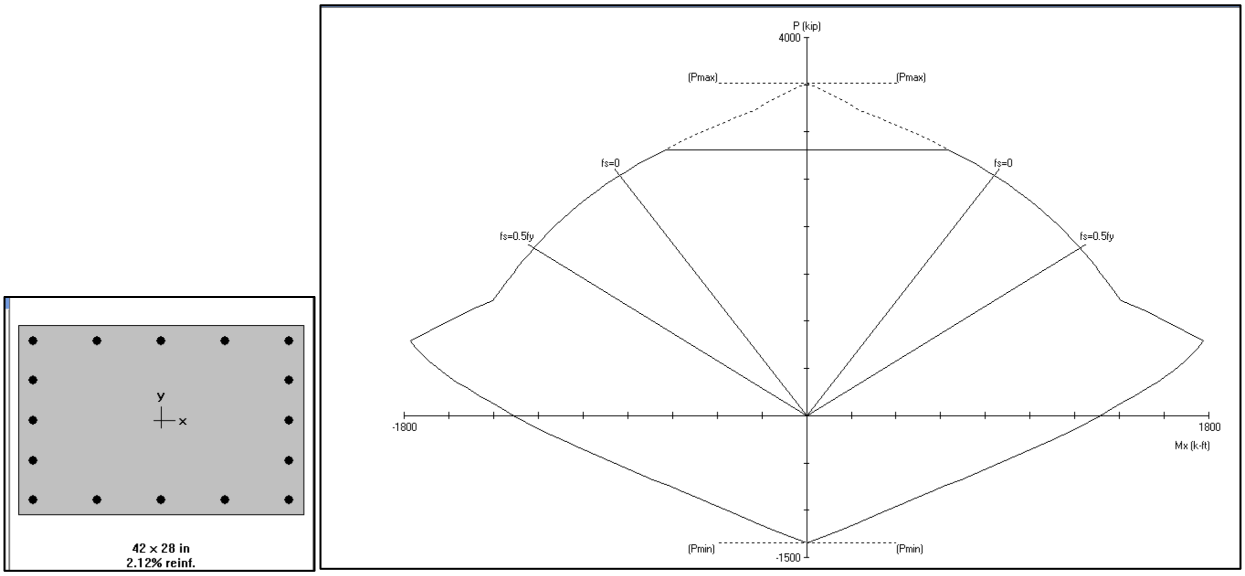


Figure 21: New Column C3 in SP Column

Effects on Floor Systems

The span between column rows A and B is 27'-7" and the span between rows B and C is 25,' so removing the columns in row B increases the longest span of the floor system to 52'-7." The effect of this on both post-tensioned floor system alternatives was studied.

For the banded beam system, the same excel spreadsheets were employed to design the band-beams. Once again, the beams were chosen to be 6 feet wide and bundles of 12 strands were used. However, instead of the 3/8" strands used in initial band-beam design, 1/2" tendons were necessary for this longer span. The results of this design are summarized in the chart below (Figure 22). For the details of the calculations see the spreadsheets in Appendix D.

Location	Superimposed Dead Load (psf)	Live Load (psf)	Beam Height (in)	Reinforcement Depth (in)	A_p (in^2)	Tendon Spacing (in O.C.)	M_u (kip-ft)	ϕM_n (kip -ft)
Typical	27	60	14	11.5	11.52	6	1275	1352
Higher Load Areas	47	150	16	13.5	31.10	6	2129	2052
17th Floor	97	150	18	15.5	20.74	6	2461	2705
18th Floor	107	400	24	21.5	20.74	6	4579	4650

Figure 22: Summary of Band-Beam Design, Column Investigation

To measure the effect of removing the Row B columns on the two-way flat plate slab system, RAM Concept was used. All of the inputs were the same, except the Row B columns were removed from the mesh elements. A simplified summary of the results of this design can be seen in the chart below (Figure 23). Diagrams of the mesh elements (slab, openings, columns, and shear walls), as well as of the distribution of deflection and moment are shown below (Figures 24-32). For the layout of both directions of reinforcement, see Appendix D.

Level	Long. Spacing	Max Deflection (in)	Max Moment (kip-ft)
Typ Floor	3-4 ft	2	1500
17th Floor	4-6 ft	2.5	1700
18th Floor	3-6 ft	2.5	2000

Figure 23: Simplified Summary of Two-Way PT Flat Plate Slab Design, Column Investigation

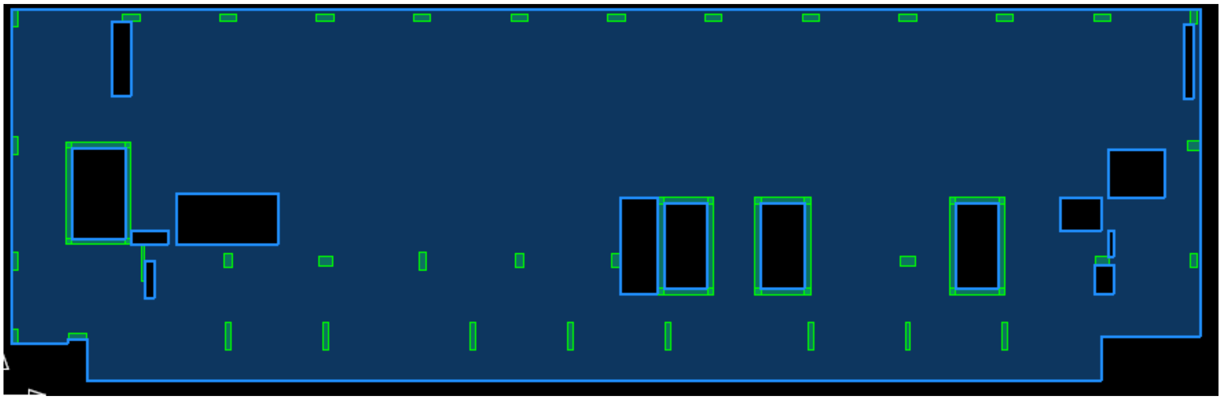


Figure 24: Typical Floor Plan, Row B Columns Removed

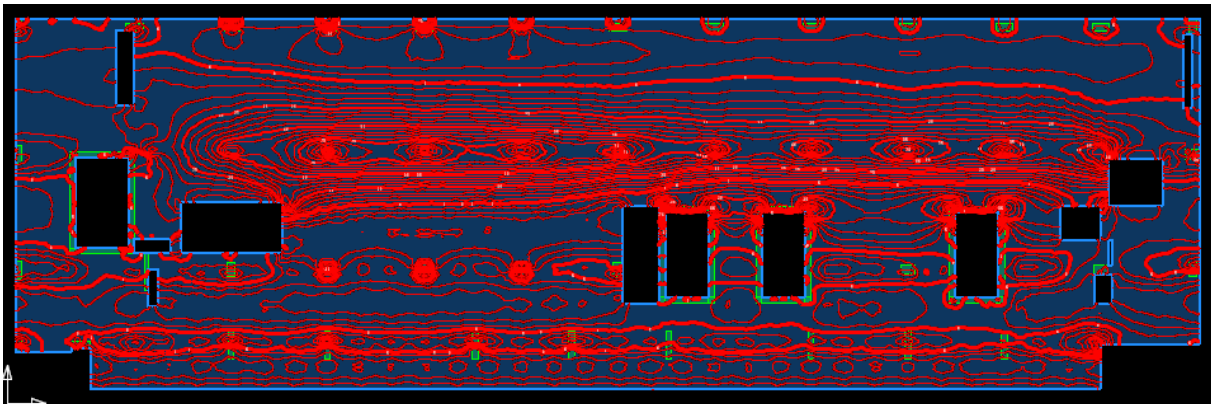


Figure 25: Typical Floor Moments

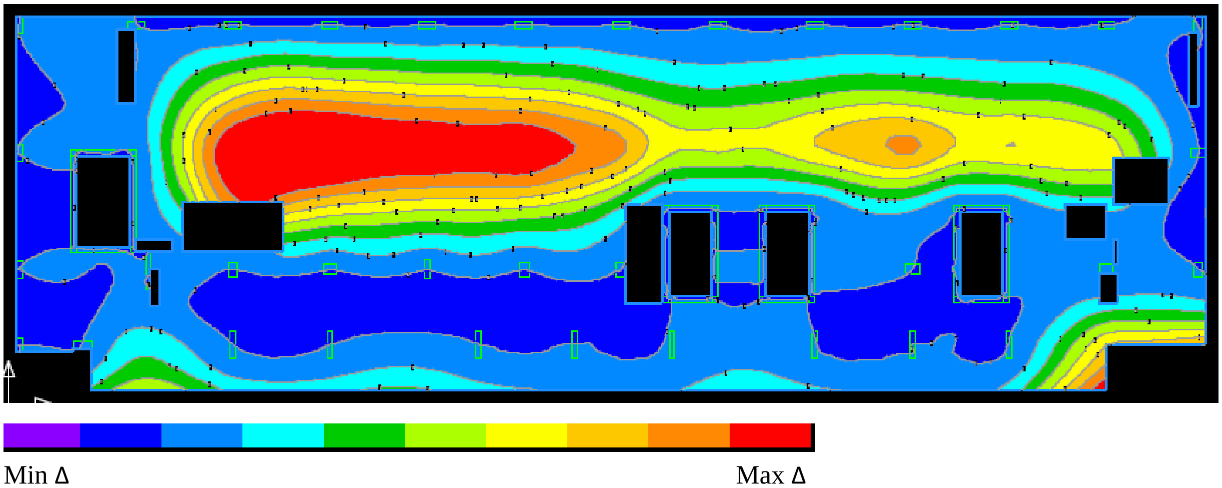


Figure 26: Typical Floor Deflections

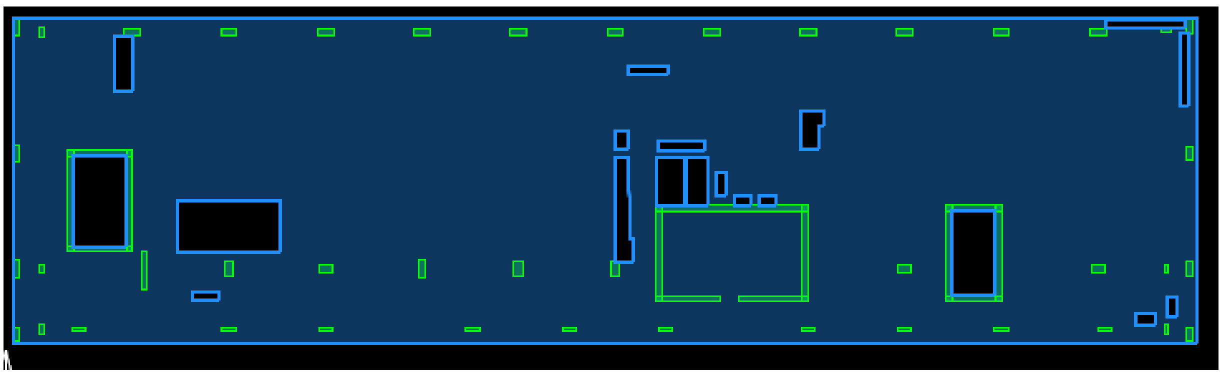


Figure 27: 17th Floor Plan, Row B Columns Removed

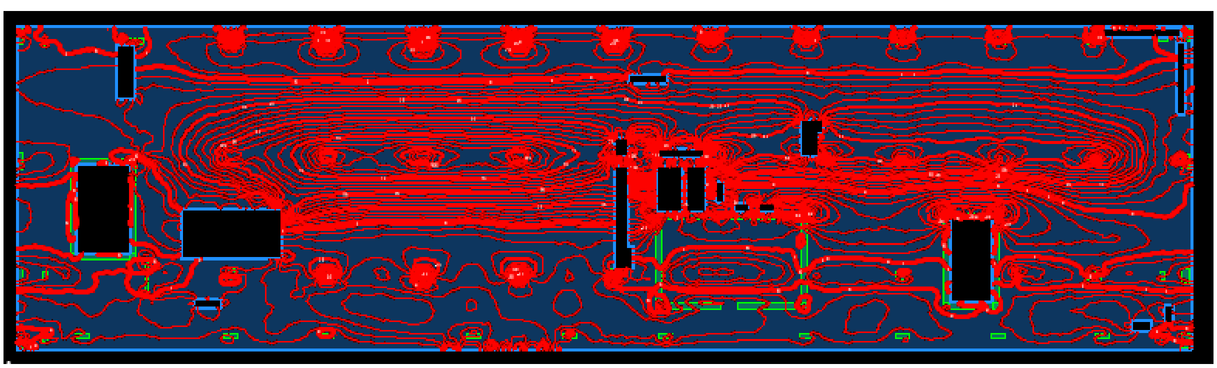


Figure 28: 17th Floor Moments

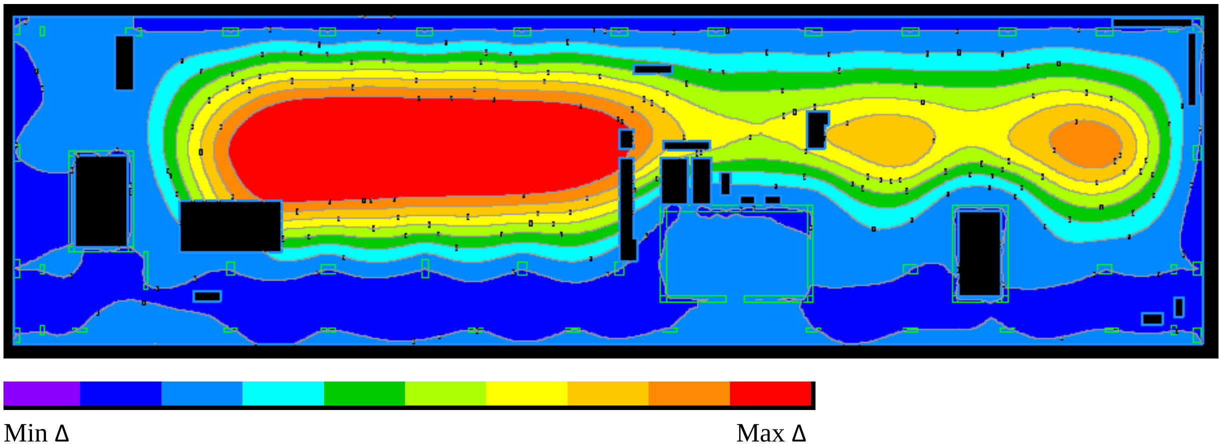


Figure 29: 17th Floor Deflections

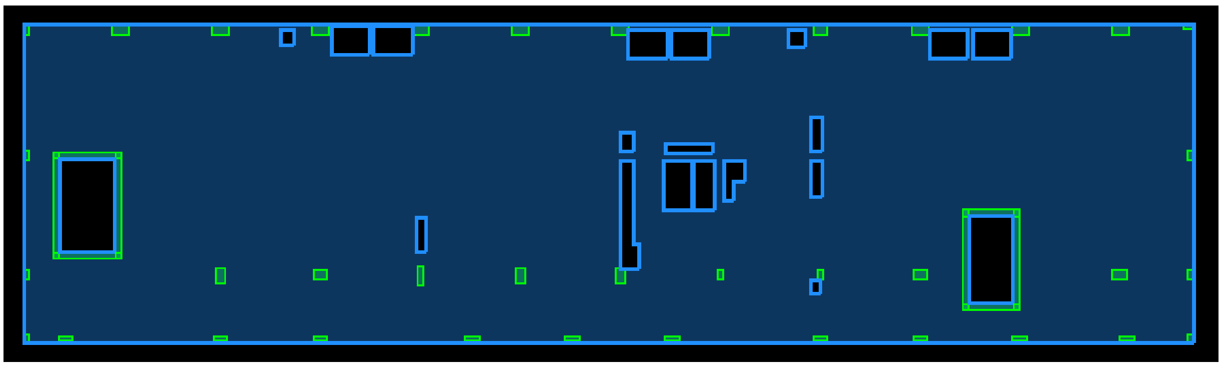


Figure 30: 18th Floor Plan, Row B Columns Removed

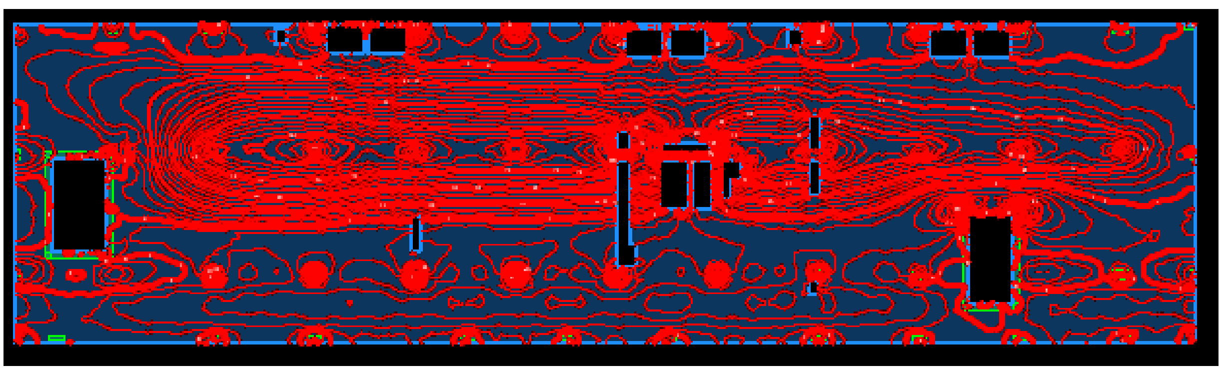


Figure 31: 18th Floor Moments

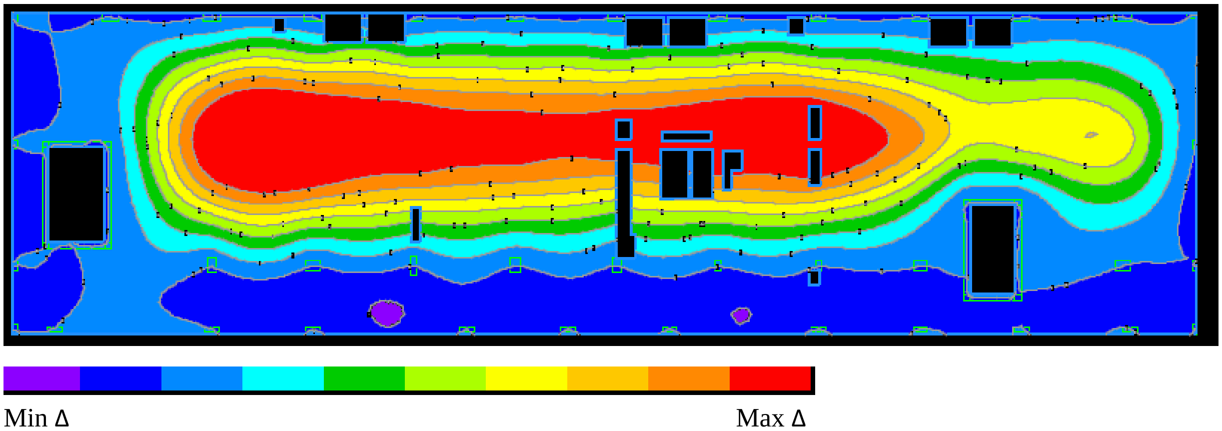


Figure 32: 18th Floor Deflections

Results and Column Conclusions:

As is obvious, neither post-tensioned floor system fared well when the columns in Row B were removed. For the banded beam system, the height of the beam was forced to increase dramatically as a result of the increased loads. This would further limit the freedom of the MEP layout and further reinforces the exclusion of the banded beam system as a viable alternative to the existing design.

For the two-way post-tensioned flat plate slab system, as can be seen in the diagrams from RAM Concept, there were extreme deflections in the areas where the columns in Row B originally were. Despite efforts to increase the amount of post-tensioned tendons, thicken the slab, as well as change the spans of the individual tendons, a solution could not be found to alleviate the severe deflections caused by removing the Row B columns. Therefore, removing columns is deemed not feasible even with the added benefits of a post-tensioned concrete system. The column design both in the layout and sizes, as confirmed by this investigation, was very well executed by the original architects and engineers.

Enclosure Breadths

Glass Sunshade Wall

The spectacular, striking, shining feature of the Weill Cornell Medical Research Building is the undulating glass sunshade curtain wall that unfolds itself across the front façade of the building. This is a double layered curtain wall, which serves to naturally ventilate itself as well as filter light and keep the solar heat from entering the building, as shown in the diagram below (Figure 33). This curtain wall's visually engaging design reflects the innovative and stunning research that will hopefully be going on inside. A rendering of the curtain wall is also shown below looking up from the front entrance to the building (Figure34).

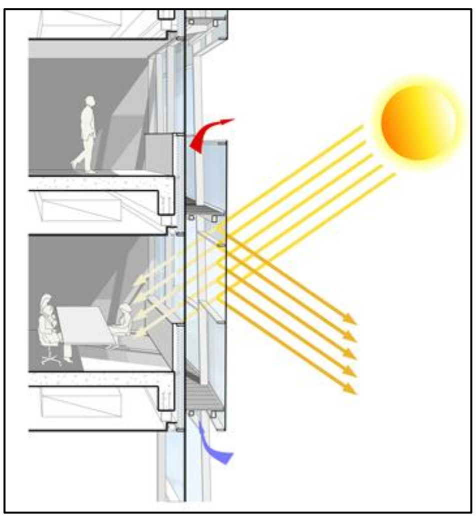


Figure 33: Glass Sunshade Wall Double Layer Detail (Image courtesy of Ennead Architects)

Figure 34: Glass Sunshade Wall Rendering (Image courtesy of Ennead Architects)

Enclosure Redesign (Mechanical)

Brick Cavity Wall:

The enclosure system that was chosen to be investigated and redesigned was the brick cavity wall, which is featured on the other three sides of the building. This system is comprised of beige four inch roman style bricks on the exterior, followed by a three inch air space, three inches of rigid insulation, an air/vapor barrier, and an eight inch concrete wall on the interior. On the north face of the building, the exact opposite of the side featuring the glass sunshade wall, there are horizontal ribbon windows across the length of the façade (see the partial elevation in Figure 35).

Figure 35: Partial Elevation – Ribbon Windows

First, the system was modeled using H.A.M. Toolbox, made by Quirouette Building Science Software. The Heat, Air, and Moisture analysis done by the program is based on inputting the materials that make up the wall and the city in which the structure is located. The first tool used was the R Value Analysis. This yielded the R Value, or thermal resistance, for each layer and a diagram which shows the location of the dew point within the wall system. The printout of the R Value Analysis results can be found in Appendix E.

Next, a heat transfer analysis was performed manually using an excel spreadsheet. First, the R Values given by H.A.M. Toolbox were converted to U Values. The sum of R Values was used to determine the overall heat transfer coefficient (U) of the wall system.

The design indoor and outdoor temperature conditions, as specified for New York City by H.A.M. Toolbox, were listed and used to calculate the total heat loss or gain for the summer and winter conditions. For this calculation a sample area of 30 square meters was used. It was determined that the existing brick cavity system gained 20.16 W/m^2*K in the summer and lost 63.84 W/m^2*K in the winter. To view the complete excel spreadsheet, see Appendix F.

The final mechanical investigation carried out on the existing enclosure system was a moisture analysis. First H.A.M. Toolbox's Condensation Analysis tool was used. The wall system was analyzed for both summer and winter conditions. The output included the R_{vap} Values and a diagram showing that there was no condensation for either the winter or summer condition (Appendix E). There were a couple of areas where the amount of vapor in the wall system was close to the saturated vapor level.

For the purpose of studying this further, a manual moisture analysis was carried out using an excel spreadsheet. This spreadsheet used the material thickness, conductivity (K) taken as the inverse of the R Value given by H.A.M. Toolbox, vapor permeance (M) taken as the inverse of the R_{vap} Values given by H.A.M., as well as the design indoor and outdoor temperature and humidity conditions to compute the temperature, pressure, and relative humidity at the interface of each layer of the wall system. The change in temperature at each layer was found by taking the total change in temperature across the system and multiplying by the ratio of the R Value of the layer to that of the entire wall. The saturated pressure values for each layer interface were obtained using the formula $P_{sat} = 1000*e^{[52.58-6790.5/T-5.028(\ln T)]}$, where T is the temperature in Kelvin. Pressure values for each layer interface were determined in a similar fashion to the temperatures, except that the total change in pressure was multiplied by the corresponding ratio of R_{vap} Values. The relative humidity at each layer is the ratio of the pressure at a given interface to the saturated pressure at that interface. For the complete spreadsheets for the H.A.M. Toolbox design conditions for New York City see Appendix F.

The results of this analysis showed that the layer interface with the highest relative humidity, and therefore, the interface that would first see condensation was the air/vapor barrier in the summer and the interior of the brick in the winter. This is fine for the summer because the vapor barrier would be able to protect the other layers from condensation forming there, however, the winter condition could pose greater problems. If condensation were to form on the interior of the brick, there would need to be sufficient mechanisms for drainage within the air space as well as the necessary weep holes at the base of the wall.

New Wall System:

It was decided that another exterior wall system should be designed to try to decrease the potential for condensation forming in the air space. Also, this new system would hopefully decrease the amount of heat loss in the winter and heat gain in the summer. The ideal system would also be thinner and lighter weight than the existing brick cavity wall.

Changes were made to every layer except the air/vapor barrier in order to accomplish these goals. Starting on the interior of the wall, the solid eight inch concrete wall was replaced by a six inch CMU, the three inches of expanded rigid insulation was replaced by two and a half inches of extruded rigid insulation, the air space was decreased from three inches to two inches, and finally, the four inch brick was replaced by inch and a quarter exterior insulation finishing system (EIFS). By observation this new EIFS system was lighter weight and thinner, but would it perform better than the brick cavity wall?

The same analyses were carried out for the EIFS system that had been for the brick cavity system. The R Value Analysis results for the EIFS system can be seen in Appendix E. The total R Value of this new system was 19.23; versus that of the brick cavity wall, which was 18.05. The heat transfer analysis yielded a heat gain of 15.6 W/m^2*K in the summer and a heat loss of 49.4 W/m^2*K in the winter. Both values were less than those of the brick cavity wall. Full spreadsheets of the heat transfer analysis can be found in Appendix F. Next the moisture analysis was carried out using H.A.M. Toolbox and the excel spreadsheet developed earlier. The results of the Condensation Analysis in H.A.M. showed a visible increase in the distance between the vapor content of the wall and the saturation level of vapor once inside the wall system. The spreadsheet moisture analysis showed a dramatic decrease in the relative humidity levels within the system. The highest relative humidity would be kept outside the wall system entirely with condensation only occurring on the exterior face of the EIFS when the air outside was saturated. The H.A.M. Condensation Analysis output and the moisture analysis spreadsheets can be found in Appendices E and F, respectively.

Overall, mechanically, this new EIFS system performed much better than the brick cavity wall. The heat losses and gains were diminished, the potential for condensation within the air space was mitigated, and the design was lighter and thinner. Thus, the EIFS wall system accomplished all of the design goals and is deemed a better choice mechanically for the Weill Cornell Medical Research Building.

Enclosure Redesign (Architecture)

Revit Models:

In an attempt to get a better idea of what the original brick cavity wall system design and the new EIFS system would look like, a Revit model of a mock-up size portion of the wall was created. By using Revit, a more realistic visual could be created than CAD drawings or even the wall cross-section diagram produced by H.A.M. (see output in Appendix E) could provide. Revit has the capability to model the wall system with specific details that are able to be customized for each layer. A quasi-realistic view of the wall systems in section is shown below (Figure 36). Also, renderings of the mock-ups were created. These can be seen below as well (Figure 36). The color of the EIFS was chosen to match that of the brick of the existing wall system, but the design is more flexible than that (as will be discussed later).

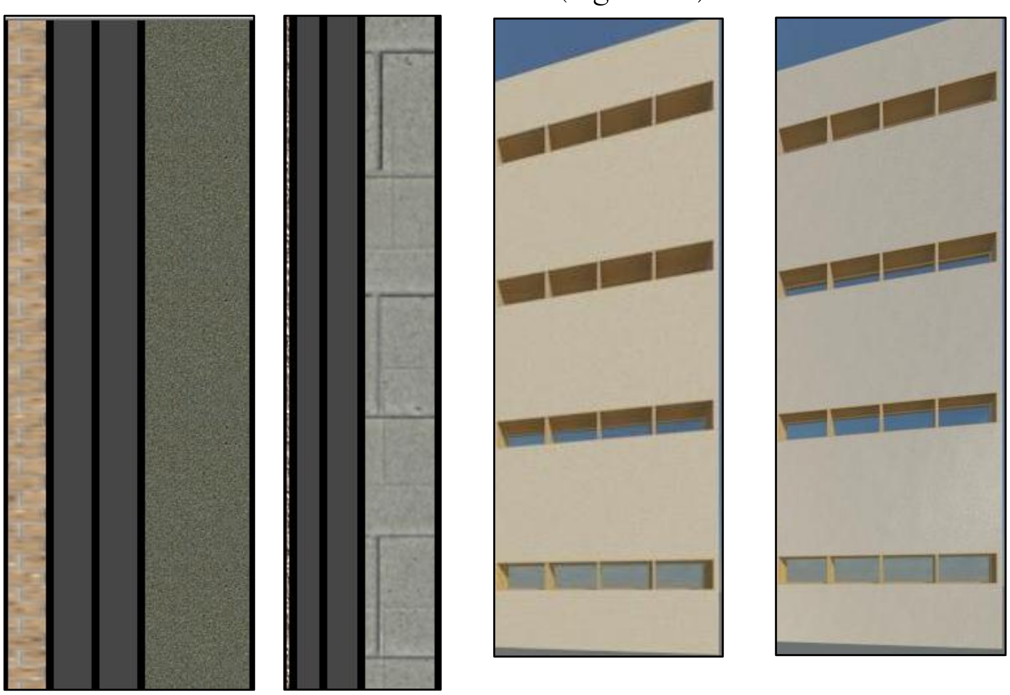


Figure 36: (Far Left) Brick Cavity Wall in Section, (Left) EIFS Wall in Section, (Right) Brick Cavity Wall Rendering, (Far Right) EIFS Wall Rendering

Exterior Insulating Finishing System (EIFS):

EIFS is an incredible versatile exterior finish for wall systems. It works to insulate and protect while being flexible enough to accommodate essentially any kind of aesthetic style. Though sometimes referred to as "synthetic stucco," it is, in fact, nothing like stucco.

EIFS has three main components: insulation board, base coat, and finish coat. The first part of the system applied is an insulation board typically made of polystyrene (or similar materials). Then a base coat is applied which is reinforced with a fiberglass mesh. Finally a finish coat is applied. It is the finish coat that often looks like stucco and causes people to make the comparison. Typical layering of an EIFS system is shown in the diagram below (Figure 37).

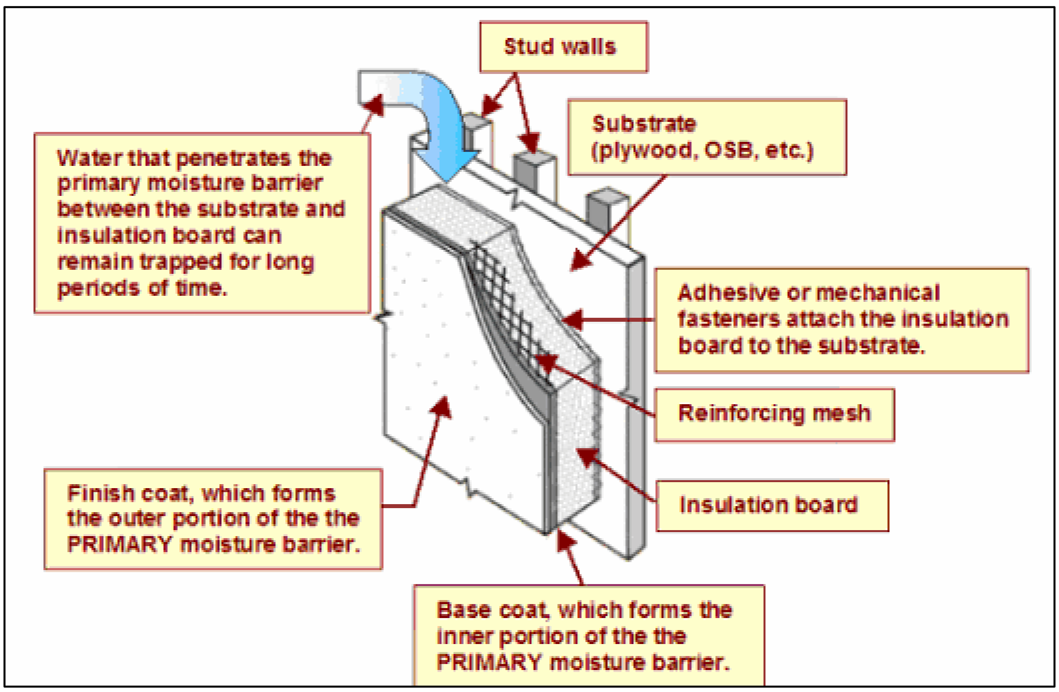


Figure 37: Typical EIFS Construction

There are many advantages to using EIFS. The visual appearance is perhaps more flexible than any other type of cladding, except maybe precast concrete. EIFS can be produced in any color and be made to look like any other type of material. Also, as shown in the mechanical analyses, EIFS out-performs brick in insulation and moisture protection. Also, studies have shown EIFS to decrease air infiltration as well by up to 55%.

For the Weill Cornell Medical Research Building, the EIFS system would represent the new wave of cutting edge research going on inside and continue the bold exterior expression of the building began by the glass sunshade wall. The original design of beige brick was to match existing buildings adjacent to the Medical Research Building. Due to the flexibility of EIFS, that same color could be used and the exterior could also even be made to look like the same beige roman bricks that were originally designed. Overall, the EIFS system makes a viable, and perhaps better, alternative to the brick cavity wall.

Structural Considerations:

For the new wall system, the ribbon windows were designed for wind, seismic, impact, and blast criteria. Each window panel is 5'-3" wide and 2'-9" high. First it was decided that the window would be an insulating glass unit (IGU) with a laminated exterior lite and monolithic interior lite. Also, annealed glass was chosen at the outset of design. ASTM E1300 was used for the glazing design and ASCE 7-05 was used to calculate wind loads. Blast loads ended up controlling the design and requiring a 5mm thick laminated glass unit (LGU) with heat-strengthened glass for the exterior lite and a 3mm fully tempered monolithic interior lite. The details of the calculations and the graphs from ASTM E1300 used can be found in Appendix G.

Conclusion

For the Weill Cornell Medical Research Building, two pre-stressed alternatives to the existing reinforced concrete two-way flat plate floor slab were investigated. A banded beam system and a two-way post-tensioned flat plate slab were designed. It was determined that the two-way PT slab would be the better of the two alternatives. The criteria for the viability of these alternatives was the elimination of the need to camber the concrete slab for the front cantilever while still meeting deflection requirements and limiting floor-to-floor heights. This was accomplished. The slab was decreased in thickness from 12 ½ inches to 10 inches. This has the added benefit of allowing more flexibility for MEP equipment and reducing the amount of concrete needed for the structure.

Following that, studies were conducted into the possibility of altering the size of the massive 14 x 72 columns from which the cantilever extends and into the removal of the columns in Row B. It was determined that the 14 x 72 columns are necessarily large and surprisingly well utilized. The investigation into the removal of the Row B columns showed that deflections would be much too extreme and the idea was deemed not possible.

Finally, mechanical and architectural studies were conducted on the enclosure system resulting in a redesign of the system from a brick cavity wall to an EIFS wall system. The new system performs better in preventing condensation and reducing heat transfer through the wall and was also deemed to be architecturally more versatile and more becoming of the Weill Cornell Medical Research Building.

Appendix A: Banded Beam System

Pre-stressed One-Way Slab Spreadsheets

Typical Span

Typical Span						
Assumptions:	f'_c =	4000	psi			
	f_{cmax} = .45*f'_c =	1800	psi			
	f_{pu} =	250000	psi			
	f_{pi} = .7*f_{pu} =	175000	psi			
	C_{cmin} =	2	in			
	Prestress Losses =	15	%			
Design for 1' wide strip, b =	12	in				
Span of One-Way Slab =	21	ft				
Slab Thickness						
$t \le L/45$ =	5.6	in				
Try: t =	8	in				
Loads						
Dead Load:	Self-weight =	100	psf			
	Superimposed =	27	psf			
Live Load:	Typical floor =	60	psf			
Prestressing Force						
w_o = Dead Load =	127	psf				
$w_o = 8Ph/L^2$ =>	$P_eh = w_oL^2/8$ =	84010.5	in-lb/ft width			
Try: h =	1.5	in				
P_e =	56007	lb/ft width				
$P_i = P_e/.85$	65890.58824	lb/ft width	w_i =	149.4117647	psf	
Check Stresses						
A =	96	in^2				
S =	128	in^3				
Stresses at Transfer:						
$w_{net} = w_i - w_o$ =	22.41176471	psf	upward			
$M_{(midspan)} = -w_{net}L^2/16$ =	-7412.691176	in-lb/ft width				
$M_{(support)} = +w_{net}L^2/8$ =	14825.38235	in-lb/ft width				
Stresses: $f = P_i/A \pm M/S$						
Midspan: f =	686.3602941	±	57.91164982			
f =	628.4486443	or	744.2719439	psi	< 1800 psi	OK
Center Support: f =	686.3602941	±	115.8232996			
f =	570.5369945	or	802.1835938	psi	< 1800 psi	OK
End Support: f =	686.3602941	psi	< 1800 psi	OK		
Stresses After Losses:						
w_{net} =	87	psf	downward			
$M_{(midspan)} = -w_{net}L^2/16$ =	28775.25	in-lb/ft width				
$M_{(support)} = +w_{net}L^2/8$ =	-57550.5	in-lb/ft width				
Stresses: $f = P_i/A \pm M/S$						
Midspan: f =	583.40625	±	224.8066406			
f =	358.5996094	or	808.2128906	psi	< 1800 psi	OK
Center Support: f =	583.40625	±	449.6132813			
f =	133.7929688	or	1033.019531	psi	< 1800 psi	OK
End Support: f =	583.40625	psi	< 1800 psi	OK		
Prestressing Tendons						
$A_p = P_i/f_{pi}$ =	0.376517647	in^2/ft width				
Try:	18	- .196" wire	A_p =	0.54	in^2	
	Spacing =	17.21034871	in spacing			
Ultimate Load in Flexure	d_p =	6	in			
At midspan and middle support	$\rho_p = A_p/b*d_p$ =	0.0075				
Unbonded tendons, span/depth	> 35					
$f_{pe} = P_e/A_p$ =	103716.6667	psi	or f_{pe} = .6*f_{pu} =	150000	psi	
$f_{ps} = f_{pe} + 10000 + f'_c/300\rho_p$ =	161777.7778	psi	< f_{pe} +30000 =	180000	psi	
Max. Reinf. :						
$w_p \le .36*\beta_1$ =	0.306					
$w_p = \rho_p*(f_{ps}/f'_c)$ =	0.303333333					
$a = A_p*f_{ps}/.85*f'_c*b$ =	2.141176471	in				
$M_n = A_p*f_{ps}*(d_p-a/2)$ =	430633.4118	in-lb/ft	ϕ =	0.9		
ϕM_n =	387570.0706	in-lb/ft				
Plastic Hinge Analysis						
Load to cause P.H. @ support:						
$w = 8M_u/L^2$ =	585.8957983	psf				
Load to cause P.H. @ midspan:						
$w = 12M_u/L^2$ =	878.8436975	psf				
Factored Applied Load:						
w_u =	248.4	psf				
$M_u = w_uL^2/8$ =	13693.05	in-lb				
Shear Capacity	ϕ_v =	0.75				
$\phi V_c = \phi 2*f'^{1/2}_c*b*d_p$ =	6830.519746	lb/ft width				
$V_u = 5/8*w_u*L$ =	3260.25	lb/ft width				

Typical Floor (End Span)

Typical Floor (End Span)						
Assumptions:	f'_c =	4000	psi			
	f_{cmax} = .45*f'_c =	1800	psi			
	f_{pu} =	250000	psi			
	f_{pi} = .7*f_{pu} =	175000	psi			
	C_{cmin} =	2	in			
	Prestress Losses =	15	%			
Design for 1' wide strip, b =	12	in				
Span of One-Way Slab =	22.625	ft				
Slab Thickness						
$t \le L/45$ =	6.033333333	in				
Try: t =	8	in				
Loads						
Dead Load:	Self-weight =	100	psf			
	Superimposed =	27	psf			
Live Load:	Typical floor =	60	psf			
Prestressing Force						
w_o = Dead Load =	127	psf				
$w_o = 8Ph/L^2$ =>	$P_eh = w_oL^2/8$ =	97515.16406	in-lb/ft width			
Try: h =	1.5	in				
P_e =	65010.10938	lb/ft width				
$P_i = P_e/.85$	76482.48162	lb/ft width	w_i =	149.4117647	psf	
Check Stresses						
A =	96	in^2				
S =	128	in^3				
Stresses at Transfer:						
$w_{net} = w_i - w_o$ =	22.41176471	psf	upward			
$M_{(midspan)} = -w_{net}L^2/16$ =	-8604.279182	in-lb/ft width				
$M_{(support)} = +w_{net}L^2/8$ =	17208.55836	in-lb/ft width				
Stresses: $f = P_i/A \pm M/S$						
Midspan: f =	796.6925169	±	67.22093111			
f =	729.4715857	or	863.913448	psi	< 1800 psi	OK
Center Support: f =	796.6925169	±	134.4418622			
f =	662.2506546	or	931.1343791	psi	< 1800 psi	OK
End Support: f =	796.6925169	psi	< 1800 psi	OK		
Stresses After Losses:						
w_{net} =	87	psf	downward			
$M_{(midspan)} = -w_{net}L^2/16$ =	33400.86328	in-lb/ft width				
$M_{(support)} = +w_{net}L^2/8$ =	-66801.72656	in-lb/ft width				
Stresses: $f = P_i/A \pm M/S$						
Midspan: f =	677.1886393	±	260.9442444			
f =	416.2443949	or	938.1328837	psi	< 1800 psi	OK
Center Support: f =	677.1886393	±	521.8884888			
f =	155.3001506	or	1199.077128	psi	< 1800 psi	OK
End Support: f =	677.1886393	psi	< 1800 psi	OK		
Prestressing Tendons						
$A_p = P_i/f_{pi}$ =	0.437042752	in^2/ft width				
Try:	18	- .196" wire	A_p =	0.54	in^2	
	Spacing =	14.82692475	in spacing			
Ultimate Load in Flexure	d_p =	6	in			
At midspan and middle support	$\rho_p = A_p/b*d_p$ =	0.0075				
Unbonded tendons, span/depth	> 35					
$f_{pe} = P_e/A_p$ =	120389.0914	psi	or f_{pe} = .6*f_{pu} =	150000	psi	
$f_{ps} = f_{pe} + 10000 + f'_c/300\rho_p$ =	161777.7778	psi	< f_{pe} +30000 =	180000	psi	
Max. Reinf. :						
$w_p \le .36*\beta_1$ =	0.306					
$w_p = \rho_p*(f_{ps}/f'_c)$ =	0.303333333					
$a = A_p*f_{ps}/.85*f'_c*b$ =	2.141176471	in				
$M_n = A_p*f_{ps}*(d_p-a/2)$ =	430633.4118	in-lb/ft	ϕ =	0.9		
ϕM_n =	387570.0706	in-lb/ft				
Plastic Hinge Analysis						
Load to cause P.H. @ support:						
$w = 8M_u/L^2$ =	504.756357	psf				
Load to cause P.H. @ midspan:						
$w = 12M_u/L^2$ =	757.1345355	psf				
Factored Applied Load:						
w_u =	248.4	psf				
$M_u = w_uL^2/8$ =	15894.20391	in-lb				
Shear Capacity	ϕ_v =	0.75				
$\phi V_c = \phi 2*f'^{1/2}_c*b*d_p$ =	6830.519746	lb/ft width				
$V_u = 5/8*w_u*L$ =	3512.53125	lb/ft width				

Higher Load Areas

Higher Load Typical Floor						
Assumptions:	f'_c =	4000	psi			
	f_{cmax} = .45*f'_c =	1800	psi			
	f_{pu} =	250000	psi			
	f_{pi} = .7*f_{pu} =	175000	psi			
	C_{cmin} =	2	in			
	Prestress Losses =	15	%			
Design for 1' wide strip, b =	12	in				
Span of One-Way Slab =	21	ft				
Slab Thickness						
t ≤ L/45 =	5.6	in				
Try: t =	8	in				
Loads						
Dead Load:	Self-weight =	100	psf			
	Superimposed =	47	psf			
Live Load:	Typical floor =	150	psf			
Prestressing Force						
w_o = Dead Load =	147	psf				
$w_o = 8Ph/L^2$ =>	$P_eh = w_oL^2/8$ =	97240.5	in-lb/ft width			
Try: h =	1.5	in				
P_e =	64827	lb/ft width				
$P_i = P_e/.85$	76267.05882	lb/ft width	w_i =	172.9411765	psf	
Check Stresses						
A =	96	in^2				
S =	128	in^3				
Stresses at Transfer:						
$w_{net} = w_i - w_o$ =	25.94117647	psf	upward			
$M_{(midspan)} = -w_{net}L^2/16$ =	-8580.044118	in-lb/ft width				
$M_{(support)} = +w_{net}L^2/8$ =	17160.08824	in-lb/ft width				
Stresses: $f = P_i/A \pm M/S$						
Midspan: f =	794.4485294	±	67.03159467			
f =	727.4169347	or	861.4801241	psi	< 1800 psi	OK
Center Support: f =	794.4485294	±	134.0631893			
f =	660.3853401	or	928.5117188	psi	< 1800 psi	OK
End Support: f =	794.4485294	psi	< 1800 psi	OK		
Stresses After Losses:						
w_{net} =	197	psf	downward			
$M_{(midspan)} = -w_{net}L^2/16$ =	65157.75	in-lb/ft width				
$M_{(support)} = +w_{net}L^2/8$ =	-130315.5	in-lb/ft width				
Stresses: $f = P_e/A \pm M/S$						
Midspan: f =	675.28125	±	509.0449219			
f =	166.2363281	or	1184.326172	psi	< 1800 psi	OK
Center Support: f =	675.28125	±	1018.089844			
f =	-342.8085938	or	1693.371094	psi	< 1800 psi	OK
End Support: f =	675.28125	psi	< 1800 psi	OK		
Prestressing Tendons						
$A_p = P_i/f_{pi}$ =	0.435811765	in^2/ft width				
Try:	18	- .196" wire	A_p =	0.54	in^2	
	Spacing =	14.86880466	in spacing			
Ultimate Load in Flexure	d_p =	6	in			
At midspan and middle support	$\rho_p = A_p/b*d_p$ =	0.0075				
Unbonded tendons, span/depth	> 35					
$f_{pe} = P_e/A_p$ =	120050	psi	or f_{pe} = .6*f_{pu} =	150000	psi	
$f_{ps} = f_{pe}$ +10000 + $f'_c/300\rho_p$ =	161777.7778	psi	< f_{pe} +30000 =	180000	psi	
Max. Reinf. :						
$w_p \leq .36*\beta_1$ =	0.306					
$w_p = \rho_p*(f_{ps}/f'_c)$ =	0.303333333					
$a = A_p*f_{ps}/.85*f'_c*b$ =	2.141176471	in				
$M_n = A_p*f_{ps}*(d_p-a/2)$ =	430633.4118	in-lb/ft	ϕ =	0.9		
ϕM_n =	387570.0706	in-lb/ft				
Plastic Hinge Analysis						
Load to cause P.H. @ support:						
$w = 8M_u/L^2$ =	585.8957983	psf				
Load to cause P.H. @ midspan:						
$w=12M_u/L^2$ =	878.8436975	psf				
Factored Applied Load:						
w_u =	416.4	psf				
$M_u = w_uL^2/8$ =	22954.05	in-lb				
Shear Capacity	ϕ_v =	0.75				
$\phi V_c = \phi 2*f'_c{}^{1/2}*b*d_p$ =	6830.519746	lb/ft width				
$V_u = 5/8*w_u*L$ =	5465.25	lb/ft width				

17^{th} Floor

17^{th} Floor						
Assumptions:	f'_c =	4000	psi			
	f_{cmax} = .45*f'_c =	1800	psi			
	f_{pu} =	250000	psi			
	f_{pi} = .7*f_{pu} =	175000	psi			
	C_{cmin} =	2	in			
	Prestress Losses =	15	%			
Design for 1' wide strip, b =	12	in				
Span of One-Way Slab =	21	ft				
Slab Thickness						
t ≤ L/45 =	5.6	in				
Try: t =	10	in				
Loads						
Dead Load:	Self-weight =	125	psf			
	Superimposed =	97	psf			
Live Load:	Typical floor =	150	psf			
Prestressing Force						
w_o = Dead Load =	222	psf				
$w_o = 8Ph/L^2$ =>	$P_eh = w_oL^2/8$ =	146853	in-lb/ft width			
Try: h =	3	in				
P_e =	48951	lb/ft width				
$P_i = P_e/.85$	57589.41176	lb/ft width	w_i =	261.1764706	psf	
Check Stresses						
A =	120	in^2				
S =	200	in^3				
Stresses at Transfer:						
$w_{net} = w_i - w_o$ =	39.17647059	psf	upward			
$M_{(midspan)} = -w_{net}L^2/16$ =	-12957.61765	in-lb/ft width				
$M_{(support)} = +w_{net}L^2/8$ =	25915.23529	in-lb/ft width				
Stresses: $f = P_i/A \pm M/S$						
Midspan: f =	479.9117647	±	64.78808824			
f =	415.1236765	or	544.6998529	psi	< 1800 psi	OK
Center Support: f =	479.9117647	±	129.5761765			
f =	350.3355882	or	609.4879412	psi	< 1800 psi	OK
End Support: f =	479.9117647	psi	< 1800 psi	OK		
Stresses After Losses:						
w_{net} =	272	psf	downward			
$M_{(midspan)} = -w_{net}L^2/16$ =	89964	in-lb/ft width				
$M_{(support)} = +w_{net}L^2/8$ =	-179928	in-lb/ft width				
Stresses: $f = P_e/A \pm M/S$						
Midspan: f =	407.925	±	449.82			
f =	-41.895	or	857.745	psi	< 1800 psi	OK
Center Support: f =	407.925	±	899.64			
f =	-491.715	or	1307.565	psi	< 1800 psi	OK
End Support: f =	407.925	psi	< 1800 psi	OK		
Prestressing Tendons						
$A_p = P_i/f_{pi}$ =	0.329082353	in^2/ft width				
Try:	18	- .196" wire	A_p =	0.54	in^2	
	Spacing =	19.69111969	in spacing			
Ultimate Load in Flexure	d_p =	8	in			
At midspan and middle support	$\rho_p = A_p/b*d_p$ =	0.005625				
Unbonded tendons, span/depth	> 35					
$f_{pe} = P_e/A_p$ =	90650	psi	or f_{pe} = .6*f_{pu} =	150000	psi	
$f_{ps} = f_{pe}$ +10000 + $f'_c/300\rho_p$ =	162370.3704	psi	< f_{pe} +30000 =	180000	psi	
Max. Reinf. :						
$w_p \leq .36*\beta_1$ =	0.306					
$w_p = \rho_p*(f_{ps}/f'_c)$ =	0.228333333					
$a = A_p*f_{ps}/.85*f'_c*b$ =	2.149019608	in				
$M_n = A_p*f_{ps}*(d_p-a/2)$ =	607226.9804	in-lb/ft	ϕ =	0.9		
ϕM_n =	546504.2824	in-lb/ft				
Plastic Hinge Analysis						
Load to cause P.H. @ support:						
$w = 8M_u/L^2$ =	826.159157	psf				
Load to cause P.H. @ midspan:						
$w=12M_u/L^2$ =	1239.238735	psf				
Factored Applied Load:						
w_u =	506.4	psf				
$M_u = w_uL^2/8$ =	27915.3	in-lb				
Shear Capacity	ϕ_v =	0.75				
$\phi V_c = \phi 2*f'_c{}^{1/2}*b*d_p$ =	9107.359661	lb/ft width				
$V_u = 5/8*w_u*L$ =	6646.5	lb/ft width				

18^{th} Floor

18^{th} Floor						
Assumptions:	f'_c =	4000	psi			
	$f_{cmax} = .45*f'_c$ =	1800	psi			
	f_{pu} =	250000	psi			
	$f_{pi} = .7*f_{pu}$ =	175000	psi			
	C_{cmin} =	2	in			
	Prestress Losses =	15	%			
Design for 1' wide strip, b =	12	in				
Span of One-Way Slab =	21	ft				
Slab Thickness						
$t \leq L/45$ =	5.6	in				
Try: t =	14	in				
Loads						
Dead Load:	Self-weight =	175	psf			
	Superimposed =	107	psf			
Live Load:	Typical floor =	400	psf			
Prestressing Force						
w_o = Dead Load =	282	psf				
$w_o = 8Ph/L^2$ =>	$P_eh = w_oL^2/8$ =	186543	in-lb/ft width			
Try: h =	3	in				
P_e =	62181	lb/ft width				
$P_i = P_e/.85$	73154.11765	lb/ft width		w_i =	331.7647059	psf
Check Stresses						
A =	168	in^2				
S =	392	in^3				
Stresses at Transfer:						
$w_{net} = w_i - w_o$ =	49.76470588	psf	upward			
$M_{(midspan)} = -w_{net}L^2/16$ =	-16459.67647	in-lb/ft width				
$M_{(support)} = +w_{net}L^2/8$ =	32919.35294	in-lb/ft width				
Stresses: $f = P_i/A \pm M/S$						
Midspan: f =	435.4411765	±	41.98897059			
f =	393.4522059	or	477.4301471	psi	< 1800 psi	OK
Center Support: f =	435.4411765	±	83.97794118			
f =	351.4632353	or	519.4191176	psi	< 1800 psi	OK
End Support: f =	435.4411765	psi	< 1800 psi	OK		
Stresses After Losses:						
w_{net} =	582	psf	downward			
$M_{(midspan)} = -w_{net}L^2/16$ =	192496.5	in-lb/ft width				
$M_{(support)} = +w_{net}L^2/8$ =	-384993	in-lb/ft width				
Stresses: $f = P_i/A \pm M/S$						
Midspan: f =	370.125	±	491.0625			
f =	-120.9375	or	861.1875	psi	< 1800 psi	OK
Center Support: f =	370.125	±	982.125			
f =	-612	or	1352.25	psi	< 1800 psi	OK
End Support: f =	370.125	psi	< 1800 psi	OK		
Prestressing Tendons						
$A_p = P_i/f_{pi}$ =	0.418023529	in^2/ft width				
Try:	18	- .196" wire	A_p =	0.54	in^2	
	Spacing =	15.50151976	in spacing			
Ultimate Load in Flexure	d_p =	12	in			
At midspan and middle support	$\rho_p = A_p/b*d_p$ =	0.00375				
Unbonded tendons, span/depth	> 35					
$f_{pe} = P_e/A_p$ =	115150	psi	or $f_{pe} = .6*f_{pu}$ =	150000	psi	
$f_{ps} = f_{pe} + 10000 + f'_c/300\rho_p$ =	163555.5556	psi	< f_{pe} +30000 =	180000	psi	
Max. Reinf. :						
$w_p \leq .36*\beta_1$ =	0.306					
$w_p = \rho_p*(f_{ps}/f'_c)$ =	0.153333333					
$a = A_p*f_{ps}/.85*f'_c*b$ =	2.164705882	in				
$M_n = A_p*f_{ps}*(d_p-a/2)$ =	964246.5882	in-lb/ft		ϕ =	0.9	
ϕM_n =	867821.9294	in-lb/ft				
Plastic Hinge Analysis						
Load to cause P.H. @ support:						
$w = 8M_u/L^2$ =	1311.90012	psf				
Load to cause P.H. @ midspan:						
$w=12M_u/L^2$ =	1967.85018	psf				
Factored Applied Load:						
w_u =	978.4	psf				
$M_u = w_uL^2/8$ =	53934.3	in-lb				
Shear Capacity	ϕ_v =	0.75				
$\phi V_c = \phi 2*f'_c{}^{1/2}*b*d_p$ =	13661.03949	lb/ft width				
$V_u = 5/8*w_u*L$ =	12841.5	lb/ft width				

Band-Beam Spreadsheets

Typical Floor

Bay AB							
Span:	27.58333333	ft					
Dimensions (Assumed)							
b =	72	in					
h =	14	in					
d=	11.5	in					
Loads							
Dead:	Superimposed:	567	plf				
	Slab self-wt:	1312.5	plf				
	Beam self-wt:	1050	plf				
Live:							
	Typ. Live Load:	1260	plf				
w_u =	1.2D+1.6L =	5.5314	klf				
Assume: fixed ends							
M_u =	350.709326	kip-ft					
Beam Design							
Assume: bonded tendons							
Gr 250 STL							
f_{pu} =	250000	psi	f_{py}/f_{pu} =	0.86	> .85	γ_p =	0.4
f_{py} =	215000	psi					
f_{pc} =	150000	psi					
f'_c =	4000	psi					
Try: 3/8" diameter strands	at 12" O.C.						
d_p =	0.375	in					
n =	12	strands					
A_p =	5.76	in^2					
ρ_p =	0.006956522						
f_{ps} =	f_{pu} [1-(γ_p/β_1)*(ρ_p*f_{pu}/f'$_c$)] =		198849.1049	psi	< f_{py}, OK		
a =	4.678802467	in	c =	5.504473491	in		
Mn =	A_p*f_{ps}*(d_p-a/2)=	874.356895	kip-ft				
ε_s =	0.003267629		ϕ =	0.9			
ϕM_n =	786.9212055	kip-ft	> M_u, OK				
$A_{s\ min}$ =	.004*A_{ct} =	2.446711635	in^2	< A_p, OK			
Deflections							
$\bar{y}$ =	7.125	in					
I_{ut} =	17031	in^4					
f_r =	474.341649	psi					
M_{cr} =	112.2852821	kip-ft					
w_{D+L} =	4189.5	plf					
M_a =	265.628362	kip-ft					
I_g =	16464	in^4					
ρ_g =	0.005714286						
k =	0.229810743						
kd =	2.642823545	in					
I_{cr} =	3154.229562	in^4					
I_e =	4159.575982	in^4					
k =	0.6	(Fixed-end beams)					
Δ_i =	0.000842352	in					
l/360 =	0.919444444	in					

Typical Floor (Edge Beam)

Bay AB (Edge Beam)							
Span:	27.58333333	ft					
Dimensions (Assumed)							
b =	36	in					
h =	14	in					
d=	11.5	in					
Loads							
Dead:	Superimposed:	305.4375	plf				
	Slab self-wt:	1312.5	plf				
	Beam self-wt:	525	plf				
Live:							
	Typ. Live Load:	678.75	plf				
w_u =	1.2D+1.6L =	3.657525	klf				
Assume: fixed ends							
M_u =	231.8993614	kip-ft					
Beam Design							
Assume: bonded tendons							
Gr 250 STL							
f_{pu} =	250000	psi	f_{py}/f_{pu} =	0.86	> .85	γ_p =	0.4
f_{py} =	215000	psi					
f_{pc} =	150000	psi					
f'_c =	4000	psi					
Try: 3/8" diameter strands	at 24" O.C.						
d_p =	0.375	in					
n =	12	strands					
A_p =	2.88	in^2					
ρ_p =	0.006956522						
f_{ps} =	f_{pu} [1-(γ_p/β_1)*(ρ_p*f_{pu}/f'_c)] =		198849.1049	psi	< f_{py}, OK		
a =	4.678802467	in	c =	5.504473491	in		
Mn =	A_p*f_{ps}*(d_p-a/2)=	437.1784475	kip-ft				
ε_s =	0.003267629		ϕ =	0.9			
ϕM_n =	393.4606028	kip-ft	> M_u, OK				
$A_{s\,min}$ =	.004*A_{ct} =	1.223355817	in^2	< A_p, OK			
Deflections							
$\bar{y}$ =	7.125	in					
I_{ut} =	8515.5	in^4					
f_r =	474.341649	psi					
M_{cr} =	56.14264105	kip-ft					
w_{D+L} =	2821.6875	plf					
M_a =	178.9044584	kip-ft					
I_g =	8232	in^4					
ρ_g =	0.005714286						
k =	0.229810743						
kd =	2.642823545	in					
I_{cr} =	1577.114781	in^4					
I_e =	1782.77751	in^4					
k =	0.6	(Fixed-end beams)					
Δ_i =	0.001323708	in					
l/360 =	0.919444444	in					

Cantilever

Cantilever							
Span:	9.666666667	ft					
Dimensions (Assumed)							
b =	72	in					
h =	14	in					
d=	11.5	in					
Loads							
Dead:	Superimposed:	1197	plf				
	Slab self-wt:	1312.5	plf				
	Beam self-wt:	1050	plf				
Live:							
	Typ. Live Load:	1260	plf				
w_u =	1.2D+1.6L =	6.2874	klf				
Assume: fixed ends							
M_u =	293.7613	kip-ft					
Beam Design							
Assume: bonded tendons							
Gr 250 STL							
f_{pu} =	250000	psi	f_{py}/f_{pu} =	0.86	> .85	γ_p =	0.4
f_{py} =	215000	psi					
f_{pc} =	150000	psi					
f'_c =	4000	psi					
Try: 3/8" diameter strands	at 12" O.C.						
d_p =	0.375	in					
n =	12	strands					
A_p =	5.76	in^2					
ρ_p =	0.006956522						
f_{ps} =	$f_{pu}\,[1-(\gamma_p/\beta_1)*(\rho_p*f_{pu}/f'_c)]$ =		198849.1049	psi	< f_{py}, OK		
a =	4.678802467	in	c =	5.504473491	in		
Mn =	$A_p*f_{ps}*(d_p-a/2)$=	874.356895	kip-ft				
ε_s =	0.003267629		ϕ =	0.9			
ϕM_n =	786.9212055	kip-ft	> M_u, OK				
$A_{s\ min}$ =	.004*A_{ct} =	2.446711635	in^2	< A_p, OK			
Deflections							
$\bar{y}$ =	7.125	in					
I_{ut} =	17031	in^4					
f_r =	474.341649	psi					
M_{cr} =	112.2852821	kip-ft					
w_{D+L} =	4819.5	plf					
M_a =	225.17775	kip-ft					
I_g =	16464	in^4					
ρ_g =	0.005714286						
k =	0.229810743						
kd =	2.642823545	in					
I_{cr} =	3154.229562	in^4					
I_e =	4804.528584	in^4					
k =	2.4	(Cantilever)					
Δ_i =	0.000303713	in					
l/360 =	0.322222222	in					

Higher Load Areas

Higher Load Areas Typical Floors							
Span:	27.58333333	ft					
Dimensions (Assumed)							
b =	72	in					
h =	14	in					
d=	11.5	in					
Loads							
Dead:	Superimposed:	987	plf				
	Slab self-wt:	1312.5	plf				
	Beam self-wt:	1050	plf				
Live:							
	Typ. Live Load:	3150	plf				
w_u =	1.2D+1.6L =	9.0594	klf				
Assume: fixed ends							
M_u =	574.3963677	kip-ft					
Beam Design							
Assume: bonded tendons							
Gr 250 STL							
f_{pu} =	250000	psi	f_{py}/f_{pu} =	0.86	> .85	γ_p =	0.4
f_{py} =	215000	psi					
f_{pc} =	150000	psi					
f'_c =	4000	psi					
Try: 3/8" diameter strands	at 12" O.C.						
d_p =	0.375	in					
n =	12	strands					
A_p =	5.76	in^2					
ρ_p =	0.006956522						
f_{ps} =	f_{pu} [1-(γ_p/β_1)*(ρ_p*f_{pu}/f'_c)] =		198849.1049	psi	< f_{py}, OK		
a =	4.678802467	in	c =	5.504473491	in		
Mn =	A_p*f_{ps}*(d_p-a/2)=	874.356895	kip-ft				
ε_s =	0.003267629		ϕ =	0.9			
ϕM_n =	786.9212055	kip-ft	> M_u, OK				
$A_{s\ min}$ =	.004*A_{ct} =	2.446711635	in^2	< A_p, OK			
Deflections							
$\bar{y}$ =	7.125	in					
I_{ut} =	17031	in^4					
f_r =	474.341649	psi					
M_{cr} =	112.2852821	kip-ft					
w_{D+L} =	6499.5	plf					
M_a =	412.0901155	kip-ft					
I_g =	16464	in^4					
ρ_g =	0.005714286						
k =	0.229810743						
kd =	2.642823545	in					
I_{cr} =	3154.229562	in^4					
I_e =	3423.483755	in^4					
k =	0.6	(Fixed-end beams)					
Δ_i =	0.001587787	in					
l/360 =	0.919444444	in					

17^{th} Floor

17^{th} Floor							
Span:	27.58333333	ft					
Dimensions (Assumed)							
b =	72	in					
h =	14	in					
d=	11.5	in					
Loads							
Dead:	Superimposed:	2037	plf				
	Slab self-wt:	1312.5	plf				
	Beam self-wt:	1050	plf				
Live:							
	Typ. Live Load:	3150	plf				
w_u =	1.2D+1.6L =	10.3194	klf				
Assume: fixed ends							
M_u =	654.2845969	kip-ft					
Beam Design							
Assume: bonded tendons							
Gr 250 STL							
f_{pu} =	250000	psi	f_{py}/f_{pu} =	0.86	> .85	γ_p =	0.4
f_{py} =	215000	psi					
f_{pc} =	150000	psi					
f'_c =	4000	psi					
Try: 3/8" diameter strands	at 12" O.C.						
d_p =	0.375	in					
n =	12	strands					
A_p =	5.76	in^2					
ρ_p =	0.006956522						
f_{ps} =	f_{pu} [1-(γ_p/β_1)*(ρ_p*f_{pu}/f'_c)] =		198849.1049	psi	< f_{py}, OK		
a =	4.678802467	in	c =	5.504473491	in		
Mn =	A_p*f_{ps}*(d_p-a/2)=	874.356895	kip-ft				
ε_s =	0.003267629		ϕ =	0.9			
ϕM_n =	786.9212055	kip-ft	> M_u, OK				
$A_{s\,min}$ =	.004*A_{ct} =	2.446711635	in^2	< A_p, OK			
Deflections							
$\bar{y}$ =	7.125	in					
I_{ut} =	17031	in^4					
f_r =	474.341649	psi					
M_{cr} =	112.2852821	kip-ft					
w_{D+L} =	7549.5	plf					
M_a =	478.6636398	kip-ft					
I_g =	16464	in^4					
ρ_g =	0.005714286						
k =	0.229810743						
kd =	2.642823545	in					
I_{cr} =	3154.229562	in^4					
I_e =	3326.039291	in^4					
k =	0.6	(Fixed-end beams)					
Δ_i =	0.001898328	in					
l/360 =	0.919444444	in					

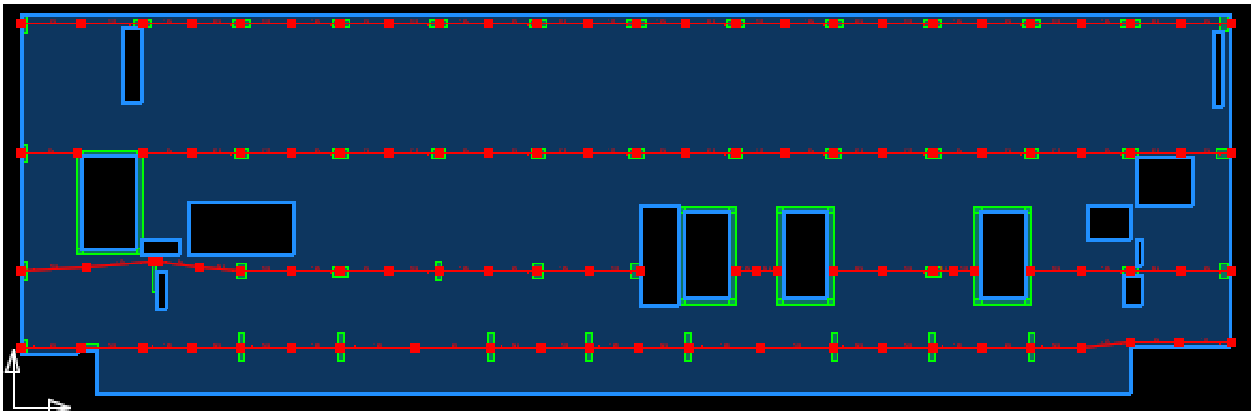
18th Floor

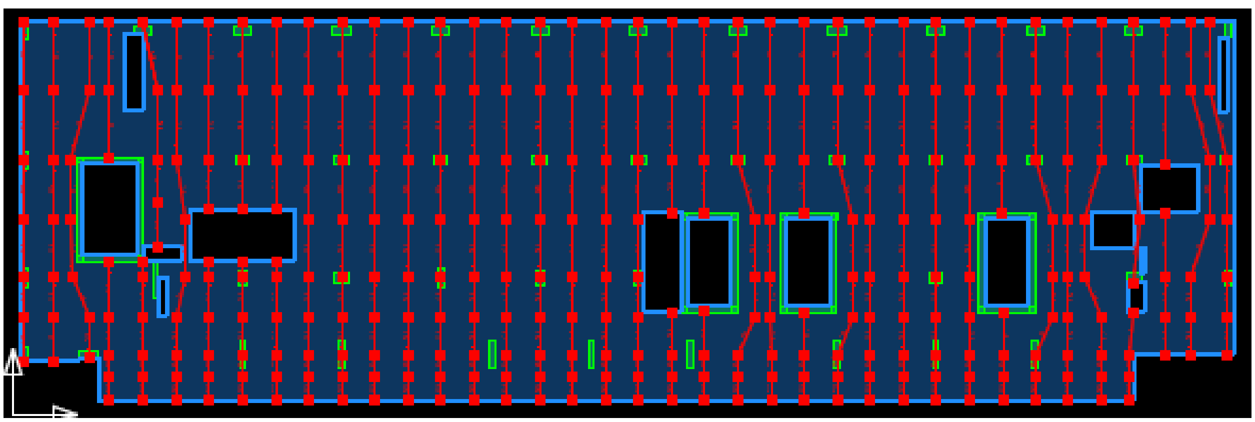
18th Floor							
Span:	27.58333333	ft					
Dimensions (Assumed)							
b =	72	in					
h =	16	in					
d=	13.5	in					
Loads							
Dead:	Superimposed:	2247	plf				
	Slab self-wt:	1312.5	plf				
	Beam self-wt:	1200	plf				
Live:							
	Typ. Live Load:	8400	plf				
w_u =	1.2D+1.6L =	19.1514	klf				
Assume: fixed ends							
M_u =	1214.263041	kip-ft					
Beam Design							
Assume: bonded tendons							
Gr 250 STL							
f_{pu} =	250000	psi	f_{py}/f_{pu} =	0.86	> .85	γ_p =	0.4
f_{py} =	215000	psi					
f_{pc} =	150000	psi					
f'_c =	4000	psi					
Try: 3/8" diameter strands	at 6" O.C.						
d_p =	0.375	in					
n =	12	strands					
A_p =	11.52	in^2					
ρ_p =	0.011851852						
f_{ps} =	f_{pu} [1-(γ_p/β_1)*(ρ_p*f_{pu}/f'_c)] =		162854.0305	psi	< f_{py}, OK		
a =	7.663719082	in	c =	9.016140097	in		
Mn =	A_p*f_{ps}*(d_p-a/2)=	1511.515816	kip-ft				
ε_s =	0.001491944		ϕ =	0.9			
ϕM_n =	1360.364234	kip-ft	> M_u, OK				
$A_{s\,min}$ =	.004*A_{ct} =	2.011351652	in^2	< A_p, OK			
Deflections							
$\bar{y}$ =	8.261904762	in					
I_{ut} =	26235.42857	in^4					
f_r =	474.341649	psi					
M_{cr} =	231.2842938	kip-ft					
w_{D+L} =	13159.5	plf					
M_a =	834.3564696	kip-ft					
I_g =	24576	in^4					
ρ_g =	0.01						
k =	0.291567917						
kd =	3.936166874	in					
I_{cr} =	7785.82391	in^4					
I_e =	8143.457878	in^4					
k =	0.6	(Fixed-end beams)					
Δ_i =	0.001351484	in					
l/360 =	0.919444444	in					

Appendix B: Two-Way PT Flat Plate Slab

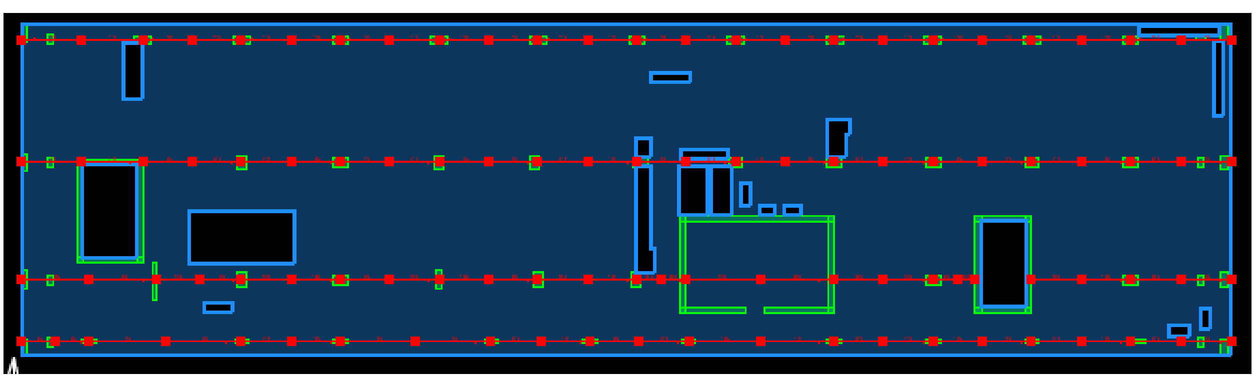
Reinforcement Plans

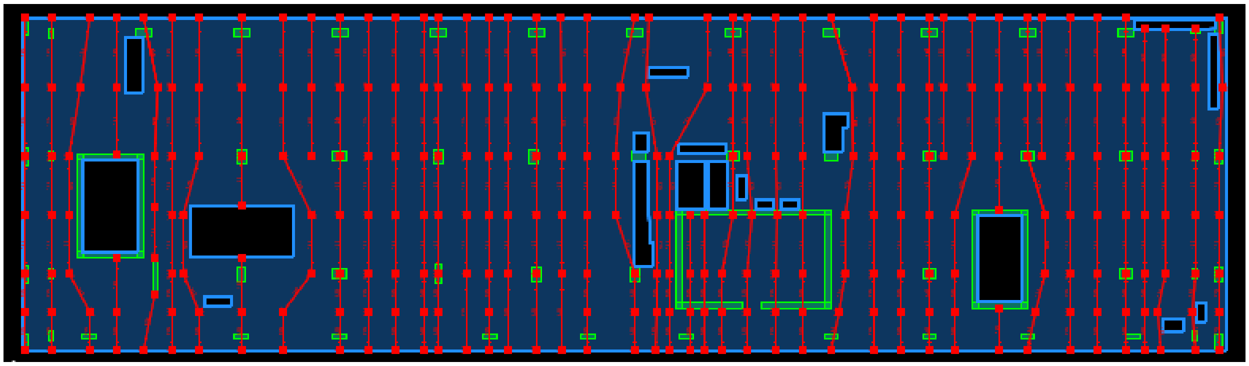
Typical Floor



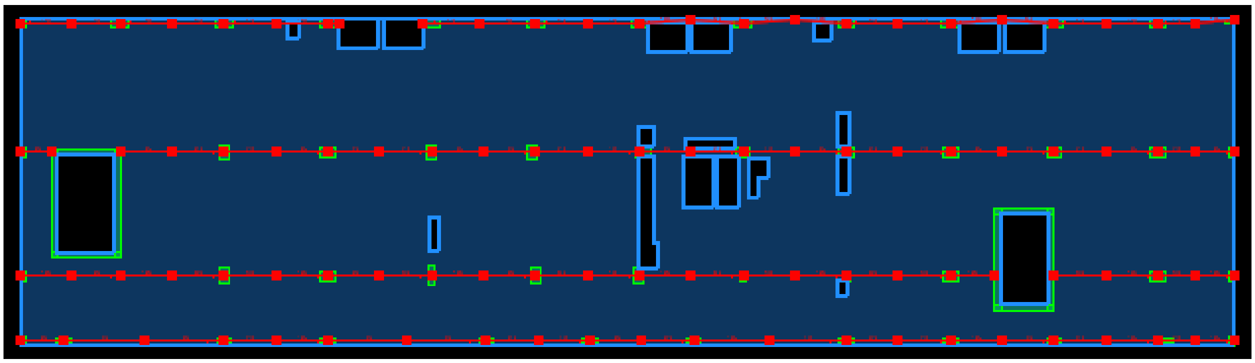


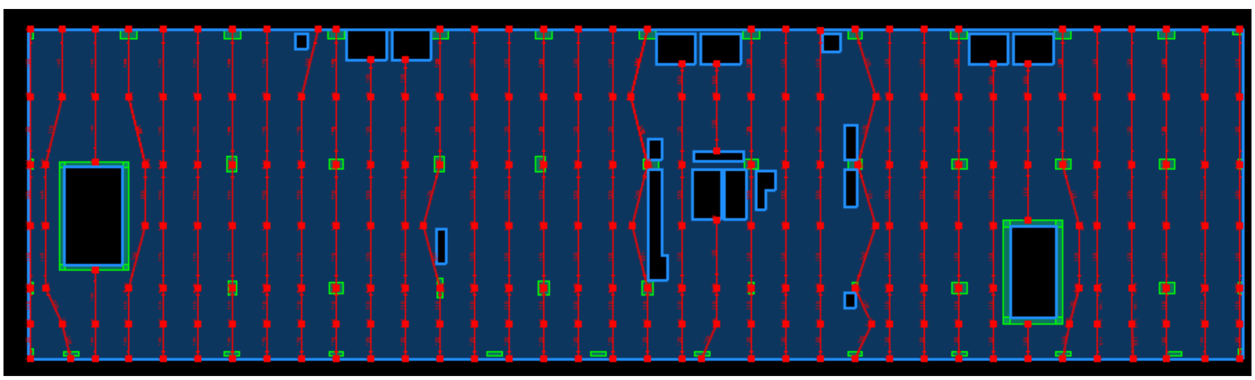
17th Floor





18th Floor





Appendix C: Column Design Hand Calculations

Jonathan Coan	AE Senior Thesis	Column Design	1

Column A3

11th Floor, Original Column: 44x20, P_u=1555 Kips, 16#9's

P_u = 2528 kips (Assume: Pure Axial Load)

f'_c = 4000psi

f_y = 60 ksi

Using Design Aid R-4-60-0.6 (γ irrelevant, axial load)

Assume: $\rho_g = .02$

$\frac{\phi P_n}{bh} = 2.32$ ksi Assume: $b = \frac{1}{2}h$

$\phi P_n = 2.32(\frac{1}{2}h^2)$

$2528 = \frac{2.32}{2}h^2$

$\Rightarrow h = 46.7$ in

Try: 24x48 column

$A_{s_{req}} = .02A_g = .02(24)(48) = 23.04\text{ in}^2$

Try: 16-#11's ($A_{st} = 24.96\text{ in}^2$)

$P_o = .85f'_c A_c + A_s f_y$

$= .85(4)(24(48) - 24.96) + 24.96(60)$

$= 5329$ kips

$\phi P_n = 5329(.65) = 3464$ kips > 2528k ok

ACI 318-08 §10.10.1

$h_{min} = 24 \geq \frac{l_u}{12} = \frac{(13+\frac{48}{12})(12)}{12} = 13.833$ in ok

Not Slender

§7.6.3.

Clr distance between bars $> 1.5d_b = 1.5(1.41) = 2.115$ in

min spacing = 6.25 in > 2.115 in ok

Column C3

11th Floor, Original Column: 36x24, P_u=1520 kips, 16-#7's

P_u = 2493 kips

Design Aid R-4-60-0.6

Assume: $\rho_g = .02$

$\frac{\phi P_n}{bh} = 2.32$ ksi Assume: $b = \frac{2}{3}h$

$\phi P_n = 2.32(\frac{2}{3}h^2)$

$2493 = 2.32(\frac{2}{3})h^2$

$\Rightarrow h = 40.15$ in

Try: 28x42

$A_{s_{req}} = .02(28)(42) = 23.52\text{ in}^2$

Try: 16-#11's ($A_{st} = 24.96\text{ in}^2$)

$P_o = .85(4)(28(42) - 24.96) + 24.96(60) = 5411$ kips

$\phi P_n = 5411(.65) = 3517$ kips > 2493 kips

ACI 318-08 §10.10.1.

$h_{min} = 28 > \frac{l_u}{12} = 13.833$ ok Not Slender

§7.6.3. $1.5d_b = 2.115$ in

min spacing = 7.6 in > 2.115 in ok

Appendix D: Column Investigations

Band-Beam Spreadsheets

Typical Floor

Bay AB							
Span:	52.58333333	ft					
Dimensions (Assumed)							
b =	72	in					
h =	14	in					
d=	11.5	in					
Loads							
Dead:	Superimposed:	567	plf				
	Slab self-wt:	1312.5	plf				
	Beam self-wt:	1050	plf				
Live:							
	Typ. Live Load:	1260	plf				
w_u =	1.2D+1.6L =	5.5314	klf				
Assume: fixed ends							
M_u =	1274.529951	kip-ft					
Beam Design							
Assume: bonded tendons							
Gr 250 STL							
f_{pu} =	250000	psi	f_{py}/f_{pu} =	0.86	> .85	γ_p =	0.4
f_{py} =	215000	psi					
f_{pc} =	150000	psi					
f'_c =	5950	psi					
Try: 3/8" diameter strands	at 6" O.C.						
d_p =	0.375	in					
n =	12	strands					
A_p =	11.52	in^2					
ρ_p =	0.013913043						
f_{ps} =	$f_{pu}\,[1-(\gamma_p/\beta_1)*(\rho_p*f_{pu}/f'_c)]$ =		181225.6872	psi	< f_{py}, OK		
a =	5.733289165	in	c =	6.745046077	in		
Mn =	$A_p*f_{ps}*(d_p-a/2)$=	1502.002338	kip-ft				
ε_s =	0.002114865		ϕ =	0.9			
ϕM_n =	1351.802104	kip-ft	> M_u, OK				
$A_{s\ min}$ =	.004*A_{ct} =	2.08942673	in^2	< A_p, OK			
Deflections							
$\bar{y}$ =	7.243243243	in					
I_{ut} =	17567.35135	in^4					
f_r =	578.5218233	psi					
M_{cr} =	178.1141713	kip-ft					
w_{D+L} =	4189.5	plf					
M_a =	965.3330495	kip-ft					
I_g =	16464	in^4					
ρ_g =	0.011428571						
k =	0.30805159						
kd =	3.542593283	in					
I_{cr} =	5443.726939	in^4					
I_e =	5512.950869	in^4					
k =	0.6	(Fixed-end beams)					
Δ_i =	0.008393905	in					
l/360 =	1.752777778	in					

Higher Load Areas

Higher Load Areas Typical Floors							
Span:	52.58333333	ft					
Dimensions (Assumed)							
b =	72	in					
h =	16	in					
d=	13.5	in					
Loads							
Dead:	Superimposed:	987	plf				
	Slab self-wt:	1312.5	plf				
	Beam self-wt:	1200	plf				
Live:							
	Typ. Live Load:	3150	plf				
w_u =	1.2D+1.6L =	9.2394	klf				
Assume: fixed ends							
M_u =	2128.917097	kip-ft					
Beam Design							
Assume: bonded tendons							
Gr 250 STL							
f_{pu} =	250000	psi	f_{py}/f_{pu} =	0.86	> .85	γ_p =	0.4
f_{py} =	215000	psi					
f_{pc} =	150000	psi					
f'_c =	5950	psi					
Try: 1/2" diameter strands	at 6" O.C.						
d_p =	0.375	in					
n =	18	strands					
A_p =	31.104	in^2					
ρ_p =	0.032						
f_{ps} =	f_{pu} [1-(γ_p/β_1)*(ρ_p*f_{pu}/f'_c)] =		91819.08057	psi	< f_{py}, OK		
a =	7.842974356	in	c =	9.227028655	in		
Mn =	A_p*f_{ps}*(d_p-a/2)=	2279.638704	kip-ft				
ε_s =	0.001389279		ϕ =	0.9			
ϕM_n =	2051.674833	kip-ft	> M_u, OK				
$A_{s\ min}$ =	.004*A_{ct} =	1.950615747	in^2	< A_p, OK			
Deflections							
$\bar{y}$ =	8.654185022	in					
I_{ut} =	28720.9163	in^4					
f_r =	578.5218233	psi					
M_{cr} =	324.0461091	kip-ft					
w_{D+L} =	6649.5	plf					
M_a =	1532.159473	kip-ft					
I_g =	24576	in^4					
ρ_g =	0.027						
k =	0.42981416						
kd =	5.80249116	in					
I_{cr} =	15746.50288	in^4					
I_e =	15830.03327	in^4					
k =	0.6	(Fixed-end beams)					
Δ_i =	0.004639734	in					
l/360 =	1.752777778	in					

17^{th} Floor

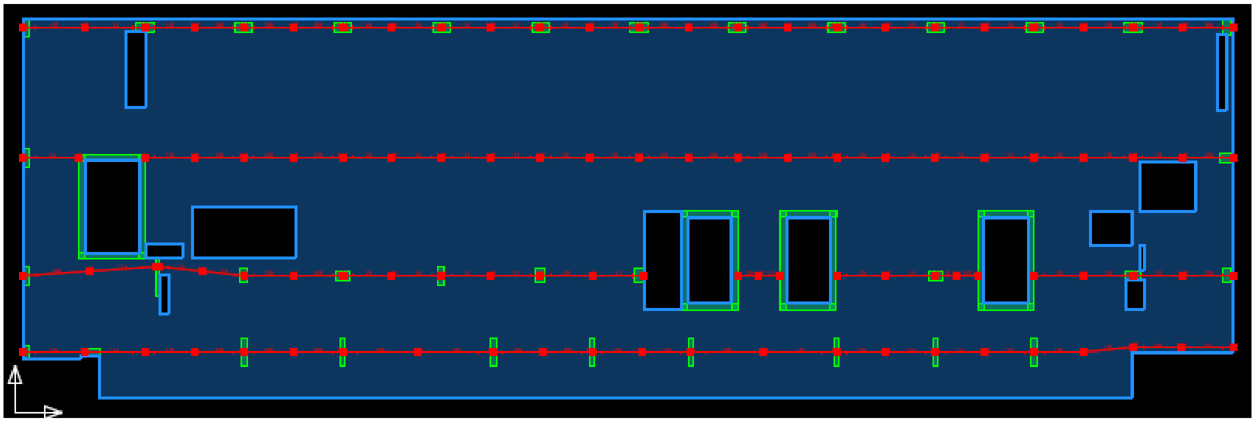
17^{th} Floor							
Span:	52.58333333	ft					
Dimensions (Assumed)							
b =	72	in					
h =	18	in					
d=	15.5	in					
Loads							
Dead:	Superimposed:	2037	plf				
	Slab self-wt:	1312.5	plf				
	Beam self-wt:	1350	plf				
Live:							
	Typ. Live Load:	3150	plf				
w_u =	1.2D+1.6L =	10.6794	klf				
Assume: fixed ends							
M_u =	2460.71793	kip-ft					
Beam Design							
Assume: bonded tendons							
Gr 250 STL							
f_{pu} =	250000	psi	f_{py}/f_{pu} =	0.86	> .85	γ_p =	0.4
f_{py} =	215000	psi					
f_{pc} =	150000	psi					
f'_c =	5950	psi					
Try: 1/2" diameter strands	at 6" O.C.						
d_p =	0.375	in					
n =	12	strands					
A_p =	20.736	in^2					
ρ_p =	0.018580645						
f_{ps} =	f_{pu} [1-(γ_p/β_1)*(ρ_p*f_{pu}/f'_c)] =		158153.0145	psi	< f_{py}, OK		
a =	9.006044129	in	c =	10.59534603	in		
Mn =	A_p*f_{ps}*(d_p-a/2)=	3005.346605	kip-ft				
ε_s =	0.001388719		ϕ =	0.9			
ϕM_n =	2704.811944	kip-ft	> M_u, OK				
$A_{s\ min}$ =	.004*A_{ct} =	2.132540342	in^2	< A_p, OK			
Deflections							
$\bar{y}$ =	9.481481481	in					
I_{ut} =	39048	in^4					
f_r =	578.5218233	psi					
M_{cr} =	383.8211678	kip-ft					
w_{D+L} =	7849.5	plf					
M_a =	1808.660168	kip-ft					
I_g =	34992	in^4					
ρ_g =	0.016						
k =	0.352571065						
kd =	5.464851501	in					
I_{cr} =	16446.14917	in^4					
I_e =	16623.38952	in^4					
k =	0.6	(Fixed-end beams)					
Δ_i =	0.005215649	in					
l/360 =	1.752777778	in					

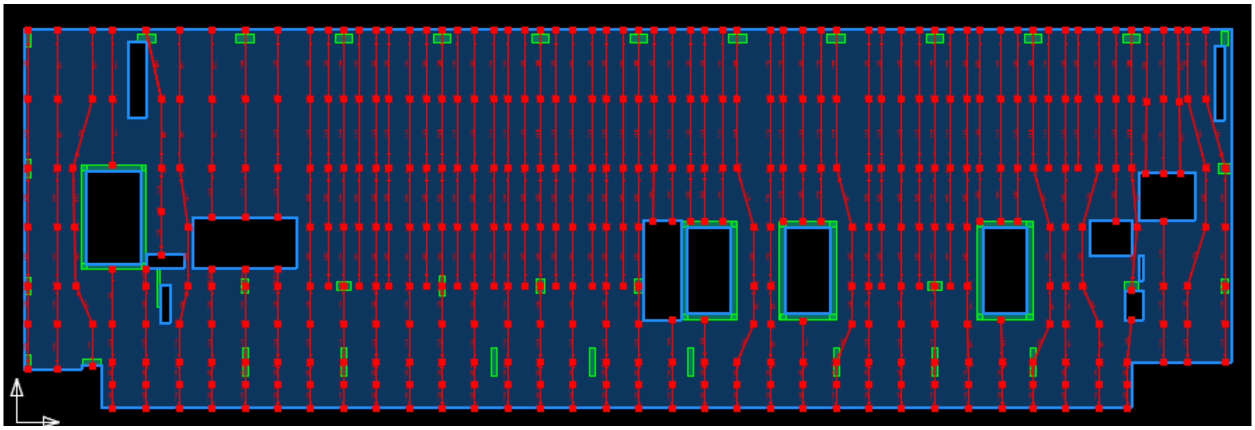
18^{th} Floor

18^{th} Floor							
Span:	52.58333333	ft					
Dimensions (Assumed)							
b =	72	in					
h =	24	in					
d=	21.5	in					
Loads							
Dead:	Superimposed:	2247	plf				
	Slab self-wt:	1312.5	plf				
	Beam self-wt:	1800	plf				
Live:							
	Typ. Live Load:	8400	plf				
w_u =	1.2D+1.6L =	19.8714	klf				
Assume: fixed ends							
M_u =	4578.71325	kip-ft					
Beam Design							
Assume: bonded tendons							
Gr 250 STL							
f_{pu} =	250000	psi	f_{py}/f_{pu} =	0.86	> .85	γ_p =	0.4
f_{py} =	215000	psi					
f_{pc} =	150000	psi					
f'_c =	5950	psi					
Try: 1/2" diameter strands	at 6" O.C.						
d_p =	0.375	in					
n =	12	strands					
A_p =	20.736	in^2					
ρ_p =	0.013395349						
f_{ps} =	f_{pu} [1-(γ_p/β_1)*(ρ_p*f_{pu}/f'_c)] =		183784.7314	psi	< f_{py}, OK		
a =	10.4656456	in		c =	12.31252424	in	
Mn =	A_p*f_{ps}*(d_p-a/2)=	5166.130393	kip-ft				
ε_s =	0.002238568		ϕ =	0.9			
ϕM_n =	4649.517353	kip-ft	> M_u, OK				
$A_{s\ min}$ =	.004*A_{ct} =	3.365993019	in^2	< A_p, OK			
Deflections							
$\bar{y}$ =	12.53773585	in					
I_{ut} =	91771.4717	in^4					
f_r =	578.5218233	psi					
M_{cr} =	481.5595393	kip-ft					
w_{D+L} =	13759.5	plf					
M_a =	3170.426088	kip-ft					
I_g =	82944	in^4					
ρ_g =	0.012						
k =	0.314243447						
kd =	6.756234101	in					
I_{cr} =	34446.97485	in^4					
I_e =	34616.92133	in^4					
k =	0.6	(Fixed-end beams)					
Δ_i =	0.00439036	in					
l/360 =	1.752777778	in					

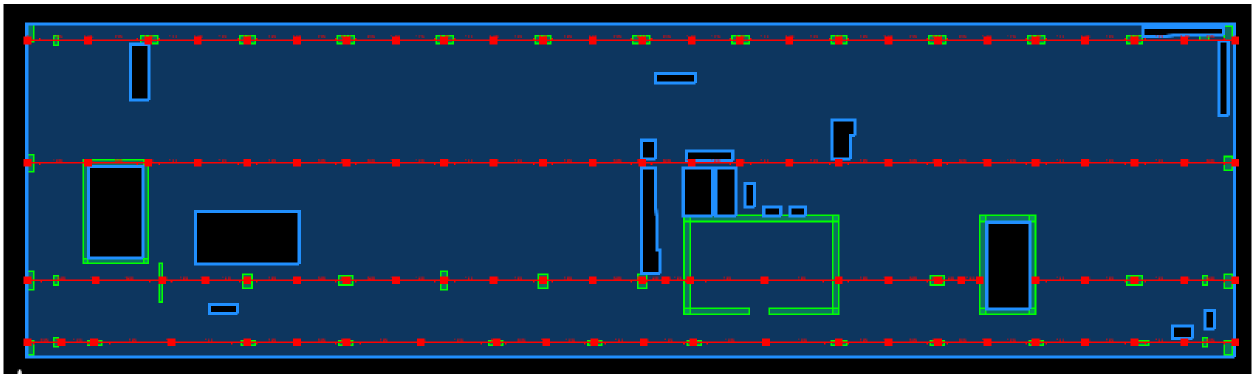
Two-Way PT Flat Plate Slab Reinforcement Plans

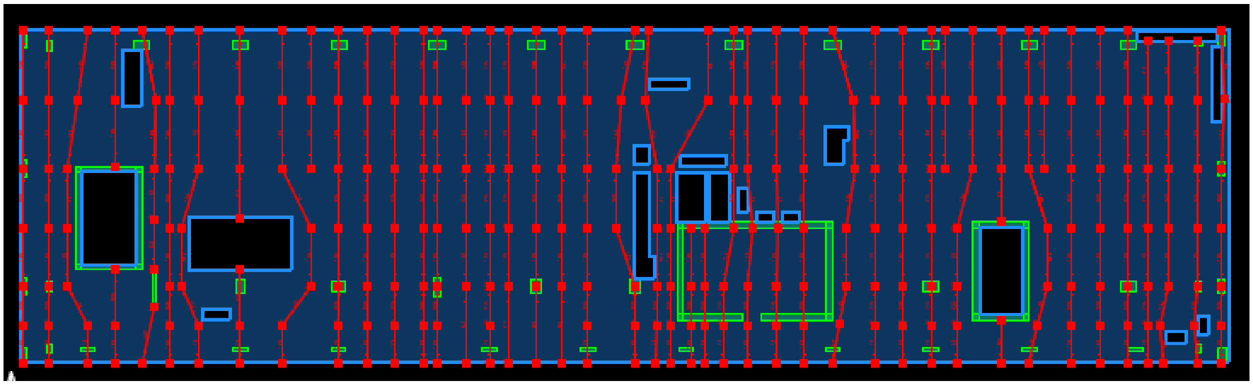
Typical Floor



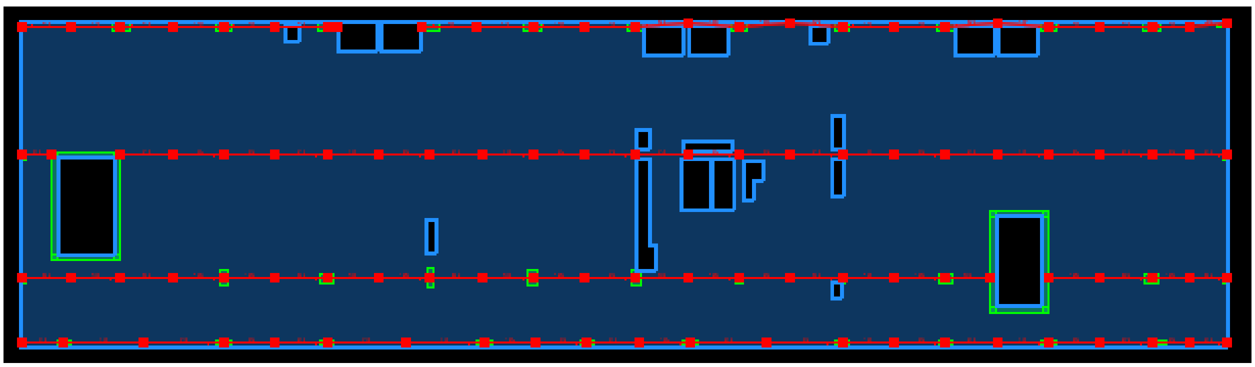


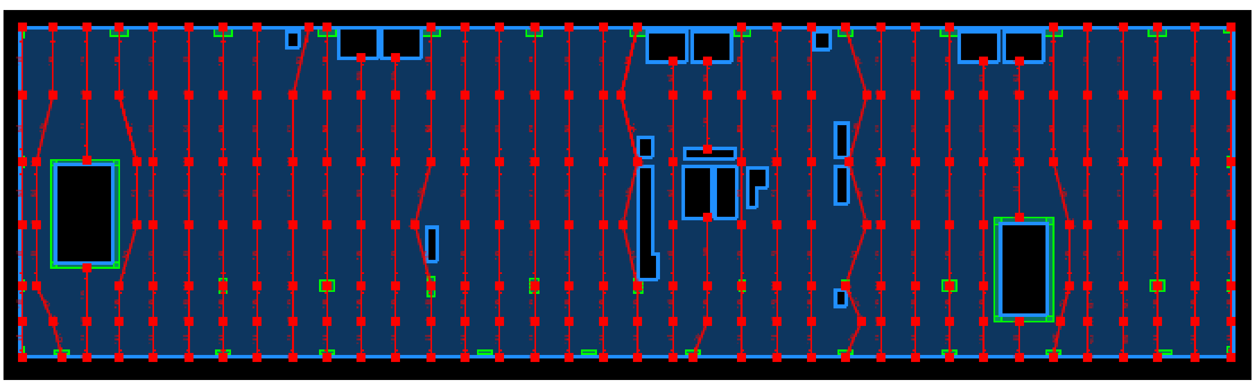
17th Floor





18th Floor



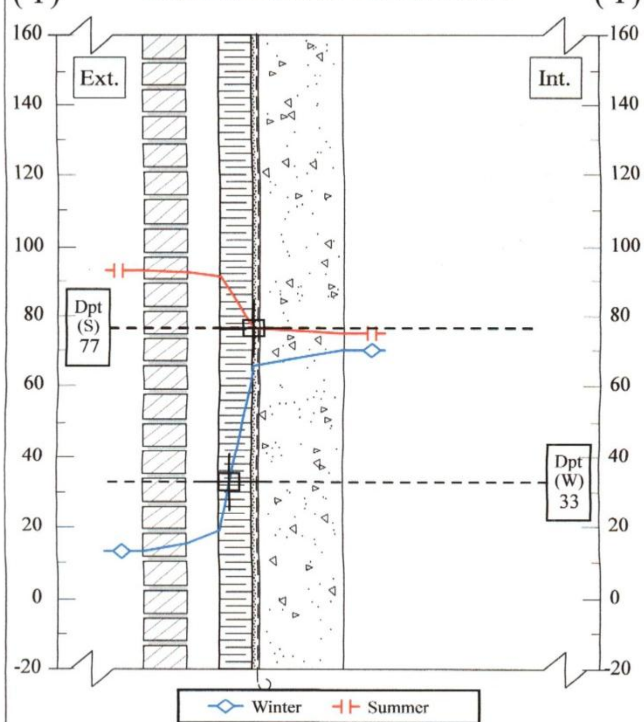


Appendix E: H.A.M. Toolbox Output

R VALUE ANALYSIS

The Heat, Air and Moisture Building Science Toolbox - V.1B-E/U (11)

WALL SECTION AND TEMPERATURE GRADIENTS



PROJECT

Name	WCMC-Existing
Number	
City	New York, NY
Date	4/5/2012
Analysis by:	Jonathan Coan
Wall Type	
Option	

CLIMATIC CONDITIONS

	Winter		Summer	
	Int.	Ext.	Int.	Ext.
Temp (°F)	70	13	75	93
RH (%)	25	80	50	57
DPT (°F)	33	8	56	77

PENNSYLVANIA STATE UNIVERSITY

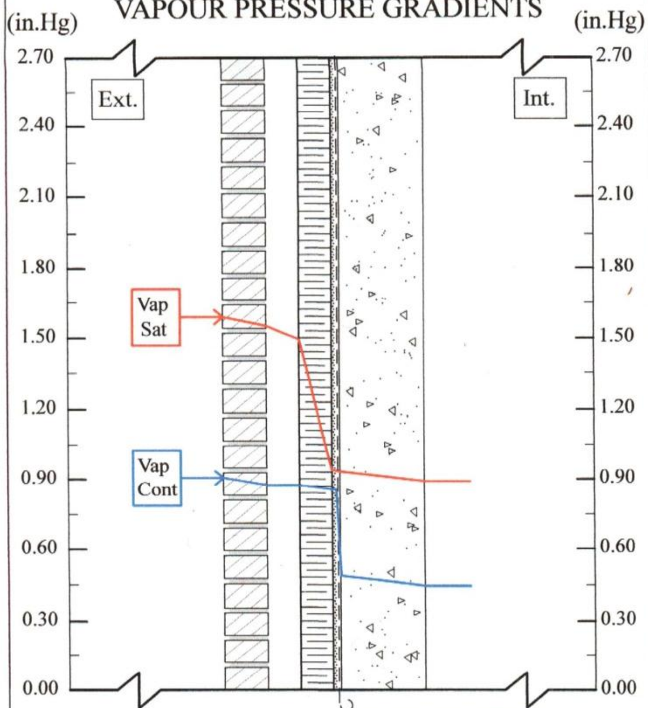
104 ENGINEERING, UNIT A
UNIVERSITY PARK, PA, USA, 16802

	Generic Material	Manufacturer	Model No.	Thick (in.)	RVal (R)	W.Temp. (°F)	S.Temp. (°F)
1	brick (TTW), 4 in.	No Recor...	Generic...	4.00	0.64	15.2	92.4
2	cavity, 3 in.	No Recor...	Generic...	3.00	0.98	19.0	91.2
3	rigid ins.,(expand.), 3 in.	No Recor...	Generic...	3.00	11.86	64.7	76.7
4	paper, stand., (8mil)	No Recor...	Generic...	0.01	0.12	65.1	76.6
5	poly film, (6mil)	No Recor...	Generic...	0.01	0.12	65.5	76.4
6	concrete wall, 8 in.	No Recor...	Generic...	8.03	1.16	70.0	75.0
	Total or (Layer 0)			18.05	14.86	(12.7)	(93.2)

CONDENSATION ANALYSIS

The Heat, Air and Moisture Building Science Toolbox - V.1B-E/U (11a)

WALL SECTION AND VAPOUR PRESSURE GRADIENTS



** NO CONDENSATION **

PROJECT

Name	WCMC-Existing
Number	
City	New York, NY
Date	4/5/2012
Analysis by:	Jonathan Coan
Wall Type	
Option	

CLIMATIC CONDITIONS

	Winter		Summer	
	Int.	Ext.	Int.	Ext.
Temp (°F)	—	—	75	93
RH (%)	—	—	50	57
DPT (°F)	—	—	56	77

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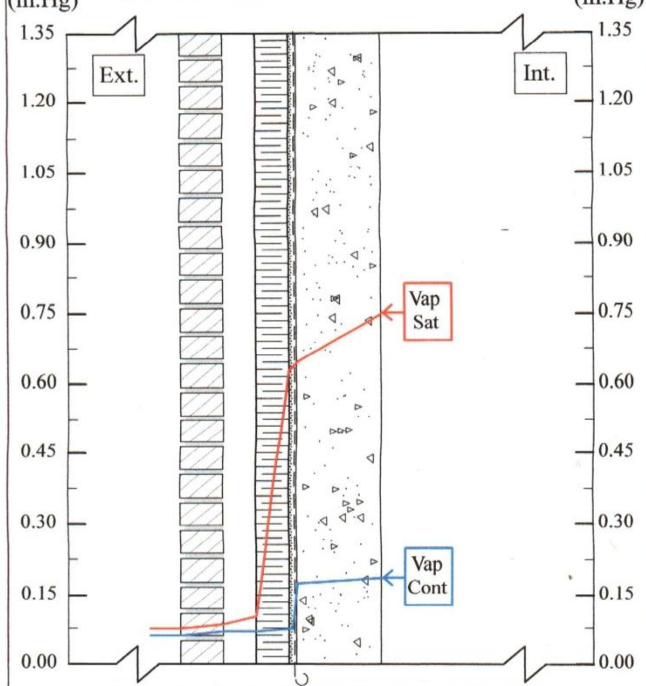
	Material	Manufacturer	Model No.	Rvap (1/M)	Temp (°F)	VapSat (in.Hg)	VapCont (in.Hg)
1	brick (TTW), 4 in.	No Recor...	Generic...	1.430	92.2	1.526	0.860
2	cavity, 3 in.	No Recor...	Generic...	0.025	91.0	1.470	0.859
3	rigid ins.,(expand.), 3 in.	No Recor...	Generic...	0.773	76.7	0.926	0.843
4	paper, stand., (8mil)	No Recor...	Generic...	0.023	76.5	0.922	0.842
5	poly film, (6mil)	No Recor...	Generic...	16.827	76.4	0.918	0.479
6	concrete wall, 8 in.	No Recor...	Generic...	1.907	75.0	0.876	0.438
7							
8							
9							
10							
11							
12							
	TOTAL or (Layer 0)			21.072	(93.0)	(1.563)	(0.891)

CONDENSATION ANALYSIS

The Heat, Air and Moisture Building Science Toolbox - V.1B-E/U (11a)

WALL SECTION AND VAPOUR PRESSURE GRADIENTS



** NO CONDENSATION **

PROJECT

Name	WCMC-Existing
Number	
City	New York, NY
Date	4/5/2012
Analysis by:	Jonathan Coan
Wall Type	
Option	

CLIMATIC CONDITIONS

	Winter		Summer	
	Int.	Ext.	Int.	Ext.
Temp (°F)	70	13	—	—
RH (%)	25	80	—	—
DPT (°F)	33	8	—	—

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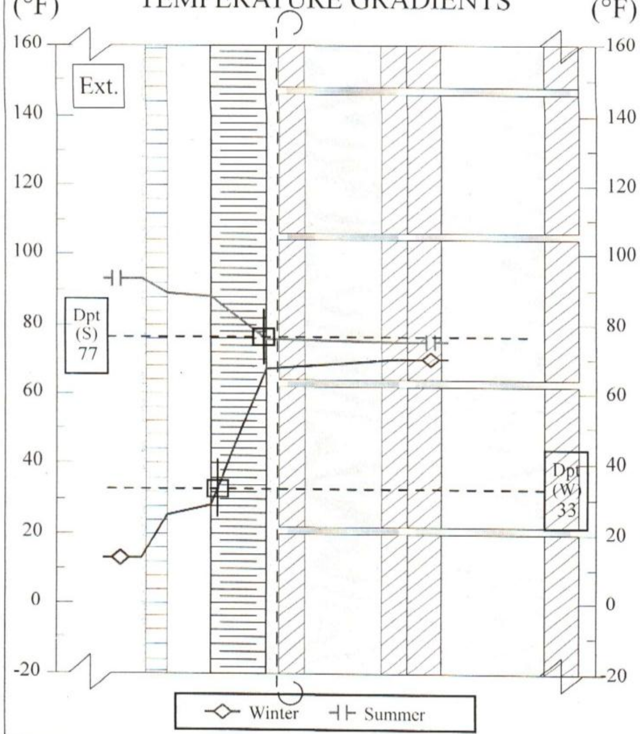
	Material	Manufacturer	Model No.	Rvap (1/M)	Temp (°F)	VapSat (in.Hg)	VapCont (in.Hg)
1	brick (TTW), 4 in.	No Recor...	Generic...	1.430	15.4	0.082	0.067
2	cavity, 3 in.	No Recor...	Generic...	0.025	19.2	0.099	0.067
3	rigid ins.,(expand.), 3 in.	No Recor...	Generic...	0.773	64.7	0.616	0.072
4	paper, stand., (8mil)	No Recor...	Generic...	0.023	65.1	0.625	0.072
5	poly film, (6mil)	No Recor...	Generic...	16.827	65.6	0.635	0.173
6	concrete wall, 8 in.	No Recor...	Generic...	1.907	70.0	0.740	0.185
7							
8							
9							
10							
11							
12							
	TOTAL or (Layer 0)			21.072	(13.0)	(0.073)	(0.058)

R VALUE ANALYSIS

The Heat, Air and Moisture Building Science Toolbox - V.1B-E/U (11)

WALL SECTION AND TEMPERATURE GRADIENTS



PROJECT

Name	WCMC - New
Number	
City	New York, NY
Date	4/5/2012
Analysis by:	Jonathan Coan
Wall Type	
Option	

CLIMATIC CONDITIONS

	Winter		Summer	
	Int.	Ext.	Int.	Ext.
Temp (°F)	70	13	75	93
RH (%)	25	80	50	57
DPT (°F)	33	8	56	77

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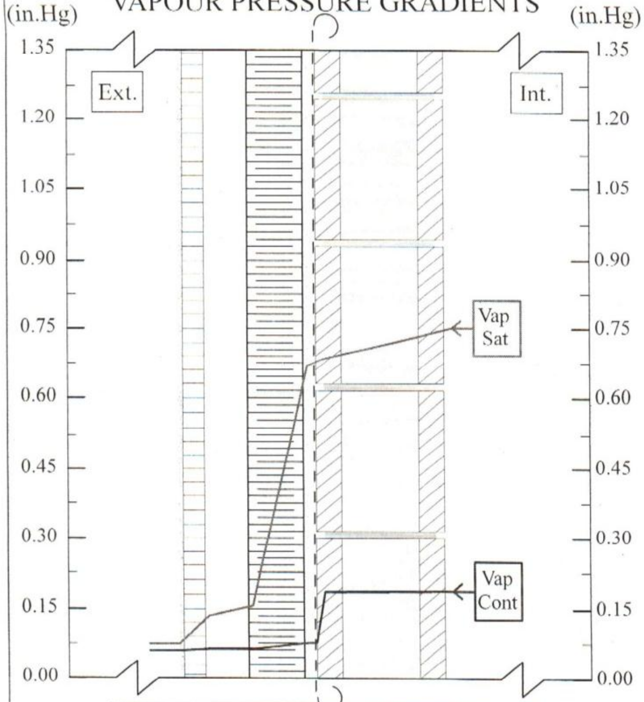
	Generic Material	Manufacturer	Model No.	Thick (in.)	RVal (R)	W.Temp. (°F)	S.Temp. (°F)
1	EIFS, 1-1/4 in.	No Recor...	Generic...	1.18	4.25	25.4	89.2
2	cavity, 2 in.	No Recor...	Generic...	2.00	0.98	28.3	88.3
3	rigid ins.,(extru.), 2-1/2 in.	No Recor...	Generic...	2.50	12.84	66.6	76.1
4	paper, stand., (8mil)	No Recor...	Generic...	0.01	0.12	66.9	76.0
5	poly film, (6mil)	No Recor...	Generic...	0.01	0.12	67.2	75.9
6	block, 6 in.	No Recor...	Generic...	6.02	0.92	70.0	75.0
				11.72	19.23	(12.7)	(93.2)

CONDENSATION ANALYSIS

The Heat, Air and Moisture Building Science Toolbox - V.1B-E/U (11a)

WALL SECTION AND VAPOUR PRESSURE GRADIENTS



** NO CONDENSATION **

PROJECT

Name	WCMC - New
Number	
City	New York, NY
Date	4/5/2012
Analysis by:	Jonathan Coan
Wall Type	
Option	

CLIMATIC CONDITIONS

	Winter		Summer	
	Int.	Ext.	Int.	Ext.
Temp (°F)	70	13	—	—
RH (%)	25	80	—	—
DPT (°F)	33	8	—	—

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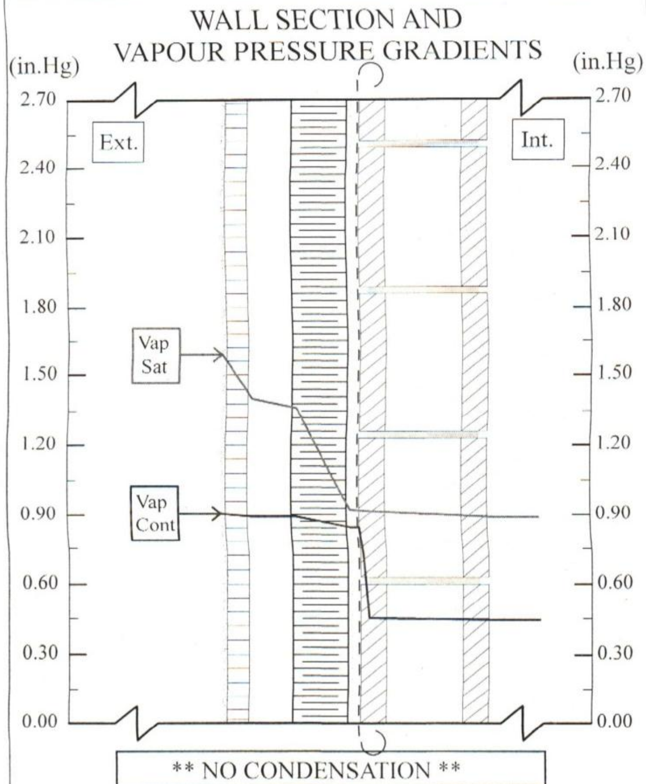
	Material	Manufacturer	Model No.	Rvap (1/M)	Temp (°F)	VapSat (in.Hg)	VapCont (in.Hg)
1	EIFS, 1-1/4 in.	No Recor...	Generic...	0.502	25.6	0.134	0.062
2	cavity, 2 in.	No Recor...	Generic...	0.016	28.5	0.154	0.062
3	rigid ins.,(extru.), 2-1/2 in.	No Recor...	Generic...	2.119	66.6	0.658	0.075
4	paper, stand., (8mil)	No Recor...	Generic...	0.023	66.9	0.666	0.075
5	poly film, (6mil)	No Recor...	Generic...	16.827	67.3	0.674	0.183
6	block, 6 in.	No Recor...	Generic...	0.313	70.0	0.740	0.185
7							
8							
9							
10							
11							
12							
	TOTAL or (Layer 0)			19.882	(13.0)	(0.073)	(0.058)

CONDENSATION ANALYSIS

The Heat, Air and Moisture Building Science Toolbox - V.1B-E/U (11a)

WALL SECTION AND VAPOUR PRESSURE GRADIENTS



PROJECT

Name	WCMC - New
Number	
City	New York, NY
Date	4/5/2012
Analysis by:	Jonathan Coan
Wall Type	Option

CLIMATIC CONDITIONS

	Winter		Summer	
	Int.	Ext.	Int.	Ext.
Temp (°F)	—	—	75	93
RH (%)	—	—	50	57
DPT (°F)	—	—	56	77

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UNIVERSITY PARK, PA, USA, 16802

	Material	Manufacturer	Model No.	Rvap (1/M)	Temp (°F)	VapSat (in.Hg)	VapCont (in.Hg)
1	EIFS, 1-1/4 in.	No Recor...	Generic...	0.502	89.0	1.380	0.879
2	cavity, 2 in.	No Recor...	Generic...	0.016	88.1	1.341	0.879
3	rigid ins.,(extru.), 2-1/2 in.	No Recor...	Generic...	2.119	76.1	0.908	0.830
4	paper, stand., (8mil)	No Recor...	Generic...	0.023	76.0	0.905	0.830
5	poly film, (6mil)	No Recor...	Generic...	16.827	75.9	0.902	0.445
6	block, 6 in.	No Recor...	Generic...	0.313	75.0	0.876	0.438
7							
8							
9							
10							
11							
12							
	TOTAL or (Layer 0)			19.882	(93.0)	(1.563)	(0.891)

Appendix F: Heat Transfer and Moisture Analysis

Brick Cavity Wall

Heat Transfer

Wall Materials	R-Value (from H.A.M. Toolbox)	U-Value (1/R)
4" Brick	0.64	1.563
3" Air Space	0.98	1.020
3" Rigid Insulation	11.86	0.084
Building Paper (8mil)	0.12	8.333
Poly Film (6mil)	0.12	8.333
8" Concrete Wall	1.16	0.862

Total R = ΣR =	14.88
Total U = 1/ΣR =	**0.0672**

Wall area =	30	m^2

Condition	Temperature (°C)
Outdoor (Summer)	34
Outdoor (Winter)	-11
Indoor (Summer)	24
Indoor (Winter)	21
ΔT_{summer}	10
ΔT_{winter}	-32

Q = A*U*ΔT	Q (w/m^2*K)
Summer:	**20.16**
Winter:	**-63.84**

$\Delta T_{layer} = \Delta T_{total} * R_{layer} / \Sigma R$

Wall Materials	$\Delta T_{layer, summer}$ (°C)	$T_{layer, summer}$ (°C)	$\Delta T_{layer, winter}$ (°C)	$T_{layer, winter}$ (°C)
4 in Brick	**0.430**	33.46	**-1.362**	-9.194
3" Air Space	**0.659**	32.80	**-2.086**	-7.108
3" Rigid Insulation	**7.970**	24.83	**-25.240**	18.132
Building Paper (8mil)	**0.081**	24.75	**-0.255**	18.387
Poly Film (6mil)	**0.081**	24.67	**-0.255**	18.642
8" Concrete Wall	**0.780**	23.89	**-2.469**	21.111

Moisture Analysis	Winter							
Layer	Material	Thickness (t) (mm)	Material Conductivity (K) (W/(m^2*K))	Vapor Permeance (M) (ng/(s*Pa*m^2))	Thermal Resistance (R) (m^2*K/W)	ΔT_i (K)	R_{vi}	ΔP_i (Pa)
Layer 1 (Exterior)	Brick	101.6	1.56	0.699	0.640	1.362	1.430	27.590
Layer 2	Air Space	76.2	1.02	40	0.980	2.086	0.025	0.482
Layer 3	Rigid Insulaion	76.2	0.08	1.29	11.860	25.240	0.773	14.914
Layer 4	Building Paper	0.254	8.33	43.5	0.120	0.255	0.023	0.444
Layer 5	Poly Film	0.254	8.33	0.059	0.120	0.255	16.827	324.650
Layer 6	Concrete Wall	203.2	0.86	0.524	1.160	2.469	1.907	36.792
		ΣM =	86.06	ΣR =	14.880	ΣR_{vi} =	20.985	
Winter Outside Air Temperature:	-10.6	°C =	262.4	K				
Inside Air Temperature:	21.1	°C =	294.1	K				
Outside RH:	80	%	P_{int} =	622.852				
Inside RH:	25	%	P_{ext} =	217.981				
Surface	Temperature (K)	Pressure (Pa)	RH (%)					
Outside	262.444	217.981	80.00					
1,2	263.806	245.570	80.93					
2,3	265.892	246.052	68.94					
3,4	291.132	260.966	12.60					
4,5	291.387	261.410	12.42					
5,6	291.642	586.060	27.41					
Inside	294.111	622.852	25.00					
$P_{sat, ext}$ =	272.476							
$P_{sat, 1,2}$ =	303.417							
$P_{sat, 2,3}$ =	356.886							
$P_{sat, 3,4}$ =	2070.441							
$P_{sat, 4,5}$ =	2103.904							
$P_{sat, 5,6}$ =	2137.841							
$P_{sat, Int}$ =	2491.408							

	Summer							
Layer	Material	Thickness (t) (mm)	Material Conductivity (K) (W/(m^2*K))	Vapor Permeance (M) (ng/(s*Pa*m^2))	Thermal Resistance (R) (m^2*K/W)	ΔT_i (K)	R_{vi}	ΔP_i (Pa)
Layer 1 (Exterior)	Brick	101.6	1.56	0.699	0.640	-0.430	1.430	-103.858
Layer 2	Air Space	76.2	1.02	40	0.980	-0.659	0.025	-1.816
Layer 3	Rigid Insulaion	76.2	0.08	1.29	11.860	-7.970	0.773	-56.141
Layer 4	Building Paper	0.254	8.33	43.5	0.120	-0.081	0.023	-1.670
Layer 5	Poly Film	0.254	8.33	0.059	0.120	-0.081	16.827	-1222.105
Layer 6 (Interior)	Concrete Wall	203.2	0.86	0.524	1.160	-0.780	1.907	-138.501
		ΣM =	86.05503478	ΣR =	14.880	ΣR_{vi} =	20.985	
Summer Outside Air Temperature:	33.88888889	°C =	306.8888889	K				
Inside Air Temperature:	23.88888889	°C =	296.8888889	K				
Outside RH:	57	%	P_{int} =	1474.702				
Inside RH:	50	%	P_{ext} =	2998.793				
Surface	Temperature (K)	Pressure (Pa)	RH (%)					
Outside	306.889	2998.793	57.00					
1,2	306.459	2894.936	56.36					
2,3	305.800	2893.120	58.44					
3,4	297.830	2836.979	90.92					
4,5	297.749	2835.308	91.30					
5,6	297.668	1613.203	52.20					
Inside	296.889	1474.702	50.00					
$P_{sat, ext}$ =	5261.040							
$P_{sat, 1,2}$ =	5136.264							
$P_{sat, 2,3}$ =	4950.168							
$P_{sat, 3,4}$ =	3120.361							
$P_{sat, 4,5}$ =	3105.377							
$P_{sat, 5,6}$ =	3090.455							
$P_{sat, Int}$ =	2949.404							

EIFS Wall

Wall Materials	R-Value (from H.A.M. Toolbox)	U-Value (1/R)
1-1/4" EIFS	4.25	0.235
2" Air Space	0.98	1.020
2-1/2" Rigid Insulation	12.84	0.078
Building Paper (8mil)	0.12	8.333
Poly Film (6mil)	0.12	8.333
6" Concrete Block	0.92	1.087

Total R = ΣR =	19.23	
Total U = 1/ΣR =	**0.0520**	
Wall area =	30	m^2

Condition	Temperature (°C)
Outdoor (Summer)	34
Outdoor (Winter)	-11
Indoor (Summer)	24
Indoor (Winter)	21
ΔT_{summer}	10
ΔT_{winter}	-32

Q = A*U*ΔT	Q (w/m^2*K)
Summer:	**15.60**
Winter:	**-49.40**

$\Delta T_{layer} = \Delta T_{total} * R_{layer} / \Sigma R$

Wall Materials	$\Delta T_{layer, summer}$ (°C)	$T_{layer, summer}$ (°C)	$\Delta T_{layer, winter}$ (°C)	$T_{layer, winter}$ (°C)
4 in Brick	**2.210**	31.68	**-6.999**	-3.557
3" Air Space	**0.510**	31.17	**-1.614**	-1.943
3" Rigid Insulation	**6.677**	24.49	**-21.144**	19.201
Building Paper (8mil)	**0.062**	24.43	**-0.198**	19.399
Poly Film (6mil)	**0.062**	24.37	**-0.198**	19.596
8" Concrete Wall	**0.478**	23.89	**-1.515**	21.111

	Winter							
Layer	Material	Thickness (t) (mm)	Material Conductivity (K) (W/(m^2*K))	Vapor Permeance (M) (ng/(s*Pa*m^2))	Thermal Resistance (R) (m^2*K/W)	ΔT_i (K)	R_{vi}	ΔP_i (Pa)
Layer 1 (Exterior)	EIFS	31.75	0.24	1.99	4.250	6.999	0.502	10.265
Layer 2	Air Space	50.8	1.02	63	0.980	1.614	0.016	0.327
Layer 3	Rigid Insulaion	63.5	0.08	0.472	12.840	21.144	2.119	43.331
Layer 4	Building Paper	0.254	8.33	43.5	0.120	0.198	0.023	0.470
Layer 5	Poly Film	0.254	8.33	0.06	0.120	0.198	16.826	344.067
Layer 6	Concrete Block	152.4	1.09	3.2	0.920	1.515	0.313	6.410
		ΣM =	111.6916465	ΣR =	19.230	ΣR_{vi} =	19.799	
Winter Outside Air Temperature:	-10.55555556	°C =	262.4444444	K				
Inside Air Temperature:	21.11111111	°C =	294.1111111	K				
Outside RH:	80	%	P_{int} =	622.852				
Inside RH:	25	%	P_{ext} =	217.981				

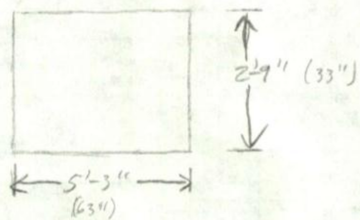
Surface	Temperature (K)	Pressure (Pa)	RH (%)
Outside	262.444	217.981	80.00
1,2	269.443	228.246	48.83
2,3	271.057	228.573	43.37
3,4	292.201	271.904	12.28
4,5	292.399	272.375	12.15
5,6	292.596	616.442	27.17
Inside	294.111	622.852	25.00
$P_{sat, ext}$ =	272.476		
$P_{sat, 1,2}$ =	467.447		
$P_{sat, 2,3}$ =	527.056		
$P_{sat, 3,4}$ =	2213.723		
$P_{sat, 4,5}$ =	2241.134		
$P_{sat, 5,6}$ =	2268.841		
$P_{sat, Int}$ =	2491.408		

	Summer							
Layer	Material	Thickness (t) (mm)	Material Conductivity (K) (W/(m^2*K))	Vapor Permeance (M) (ng/(s*Pa*m^2))	Thermal Resistance (R) (m^2*K/W)	ΔT_i (K)	R_{vi}	ΔP_i (Pa)
Layer 1 (Exterior)	Brick	101.6	0.24	0.699	4.250	-2.210	1.430	-103.858
Layer 2	Air Space	76.2	1.02	40	0.980	-0.510	0.025	-1.816
Layer 3	Rigid Insulaion	76.2	0.08	1.29	12.840	-6.677	0.773	-56.141
Layer 4	Building Paper	0.254	8.33	43.5	0.120	-0.062	0.023	-1.670
Layer 5	Poly Film	0.254	8.33	0.059	0.120	-0.062	16.827	-1222.105
Layer 6 (Interior)	Concrete Wall	203.2	1.09	0.524	0.920	-0.478	1.907	-138.501
		ΣM =	86.05503478	ΣR =	19.230	ΣR_{vi} =	20.985	
Summer Outside Air Temperature:	33.88888889	°C =	306.8888889	K				
Inside Air Temperature:	23.88888889	°C =	296.8888889	K				
Outside RH:	57	%	P_{int} =	1474.702				
Inside RH:	50	%	P_{ext} =	2998.793				

Surface	Temperature (K)	Pressure (Pa)	RH (%)
Outside	306.889	2998.793	57.00
1,2	304.679	2894.936	62.30
2,3	304.169	2893.120	64.09
3,4	297.492	2836.979	92.77
4,5	297.430	2835.308	93.06
5,6	297.367	1613.203	53.15
Inside	296.889	1474.702	50.00
$P_{sat, ext}$ =	5261.040		
$P_{sat, 1,2}$ =	4646.734		
$P_{sat, 2,3}$ =	4514.256		
$P_{sat, 3,4}$ =	3058.044		
$P_{sat, 4,5}$ =	3046.646		
$P_{sat, 5,6}$ =	3035.285		
$P_{sat, Int}$ =	2949.404		

Appendix G: Enclosure Structural Considerations

Structural Criteria for Windows



Use: IGU w/ laminated ext. lite and monolithic int. lite

Impact Criteria

Exterior lite $< \frac{8}{1000}$ probability of inner glass ply breakage from 2g steel ball impact at 39.6 m/s (130 ft/s)

Assume: - Outer lite is a sacrificial ply
- LGU is Annealed (AN) glass, PVB interlayer

Use: $\boxed{\text{3 mm outer ply } (\tfrac{1}{8}") \quad \text{5.5 mm inner ply } (\tfrac{3}{16}")}$

Wind Criteria (using ASCE 7-05)

Assume: - $\alpha = .33$ for center city exposure
- Top story controls for wind design
- Design wind velocity = 110 mph
- Category IV $\Rightarrow I = 1.15$
- $K_z = K_{zt} = K_d = 1.0$
- $G = .85$, $GC_{pi} = \pm .18$, $C_p = .8$ (Assume windward wall controls)
- Ignore IGU load share factor from ASTM E1300

ASCE 7-05

Components + Cladding, $h > 60$ ft

$p = qGC_p - q_i(GC_{pi})$

windward walls

$q = q_z = .00256(1)(1)(1)(110)^2(1.15)$

$q = 35.6$ psf

$q_i = q_h$

$p = 35.6(.85)(.8) + 35.6(.18)$

$p = 30.6$ psf

$LR_{req} = p = 30.6$ psf

$30.6 \text{ psf} \times \frac{1 \text{ kPa}}{20.9 \text{ psf}} = 1.46$ kPa

ASTM E1300

Ext. Lite: Assume AN glass

(Table 1) GTF = 1.0

$LR = NFL(GTF) \Rightarrow NFL = \frac{1.46}{1.0} = 1.46$ kPa

(Fig A1.27) 5 mm LGU $\Rightarrow$ NFL = 2.4 kPa > 1.46 kPa ok
($\frac{3}{16}"$)

Int. Lite: Assume: AN glass

(Table 1) GTF = 1.0

$NFL = \frac{1.46}{1.0} = 1.46$ kPa

(Fig A1.4) 4 mm Monolithic $\Rightarrow$ NFL = 1.8 kPa > 1.46 kPa ok
($\frac{5}{32}"$)

Wind Design: $\boxed{\text{Exterior Lite: 5 mm LGU AN} \quad \text{Interior Lite: 4 mm Mono AN}}$

2

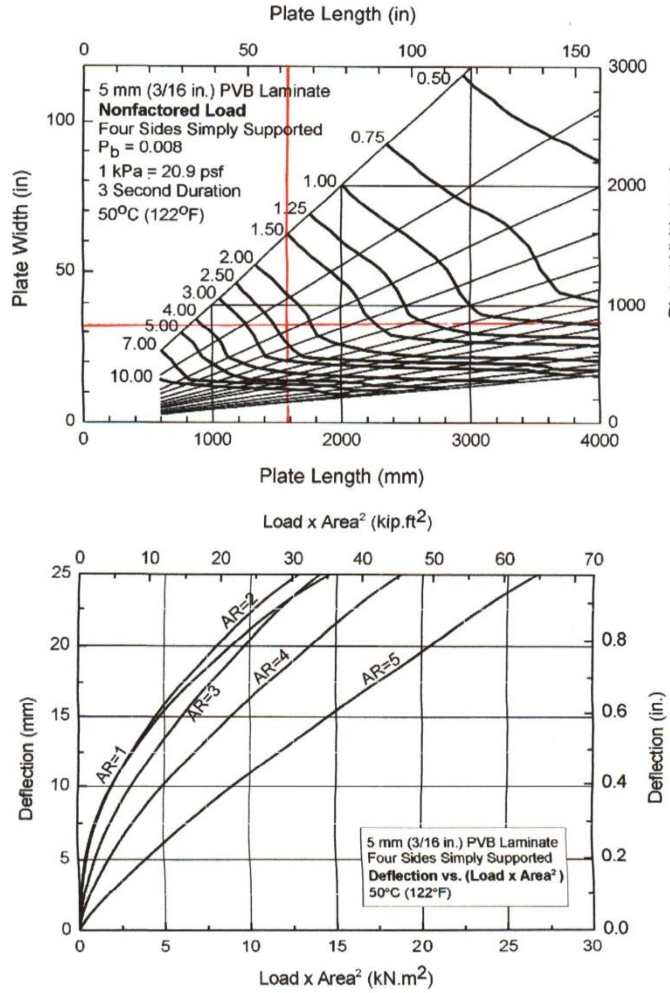
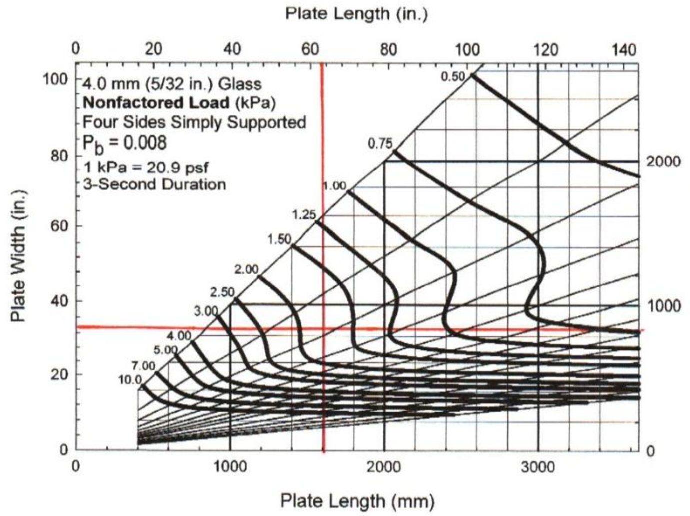


FIG. A1.27 (upper chart) Nonfactored Load Chart for 5.0 mm (3/16 in.) Laminated Glass with Four Sides Simply Supported
(lower chart) Deflection Chart for 5.0 mm (3/16 in.) Laminated Glass with Four Sides Simply Supported

33

3

E 1300 – 04^ε1



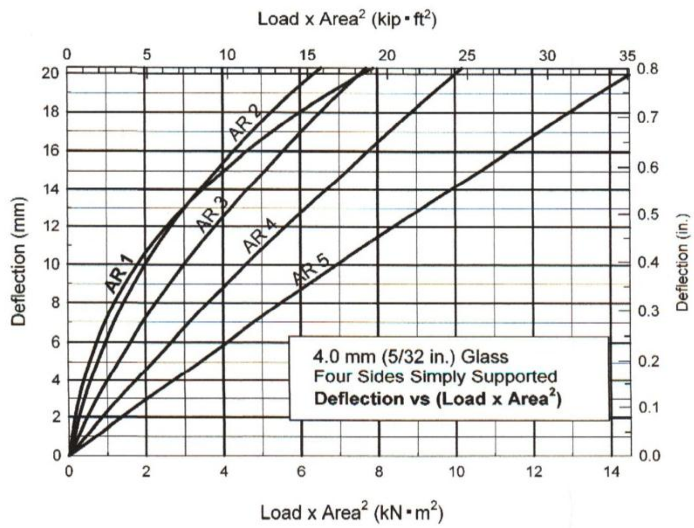


FIG. A1.4 (upper chart) Nonfactored Load Chart for 4.0 mm (5/32 in.) Glass with Four Sides Simply Supported
(lower chart) Deflection Chart for 4.0 mm (5/32 in.) Glass with Four Sides Simply Supported

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Blast Criteria
100 lb charge at 100 ft standoff distance
ASTM F2248-03 (Table 3): $P_{3d} = 3.48$ kPa

ASTM E1300
Ext. Lite: Assume AN glass (same as wind)
(Table 1) GTF = 1.0 $NFL = \frac{P_{3d}}{GTF} = 3.48$ kPa
(Fig. A1.30) 10 mm LGU => NFL = 5 kPa > 3.48 kPa ok
($\frac{3}{8}$")
Try: HS glass
GTF = 2.0 NFL = 1.74 kPa
(Fig A1.27) 5 mm LGU => NFL = 2.3 kPa > 1.74 kPa ok
($\frac{3}{16}$")
Int. Lite:
Try: FT glass
GTF = 4.0 NFL = .87 kPa
(Fig A1.3) 3mm FT Mono => NFL = 1.24 > .87 ok
($\frac{1}{8}$")

$\boxed{\text{Ext. Lite: } \frac{3}{16}\text{" (5 mm) HS LGU; Int. Lite: } \frac{1}{8}\text{" (3 mm) FT Mono}}$

Seismic Criteria: - glass-to-frame must meet exception so that laboratory testing is not required.
- Max allowable interstory drift = 1.5%
Story height = 13'-10" = 166 in
Drift Capacity = 166 (.015) = 2.49 in
$\Delta_{clear} \geq 1.25 D_p$

$D_p = \frac{\text{Panel height}}{\text{Story height}}(\text{Drift Capacity}) = \frac{33}{166}(2.49) = .495$ in

h_p = glass height = 33 in
b_p = glass width = 63 in
Assume: $c_1 = c_2$

$$D_{clear} = 2c_1\left(1+\frac{h_p c_2}{b_p c_1}\right) = 2c\left(1+\frac{h_p}{b_p}\right) \geq 1.25(.495)$$

$$\Rightarrow c \geq \frac{1.25(.495)}{2(1+\frac{33}{63})} = .203 \text{ in}$$

$\frac{1}{4}$ in = .25 in > .203 in ok

$\boxed{\text{Use glass-to-frame clearance of } \frac{1}{4}\text{" (6 mm) for prevention of glass-to-frame contact}}$

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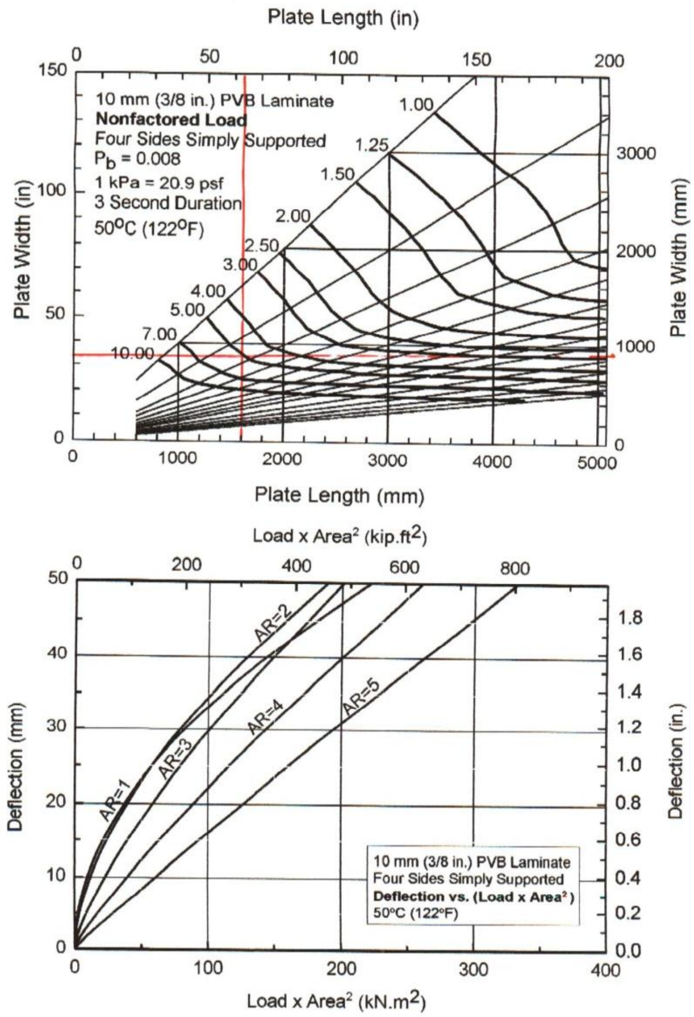

FIG. A1.30 (upper chart) Nonfactored Load Chart for 10.0 mm (3/8 in.) Laminated Glass with Four Sides Simply Supported
(lower chart) Deflection Chart for 10.0 mm (3/8 in.) Laminated Glass with Four Sides Simply Supported

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E 1300 – 04ε1

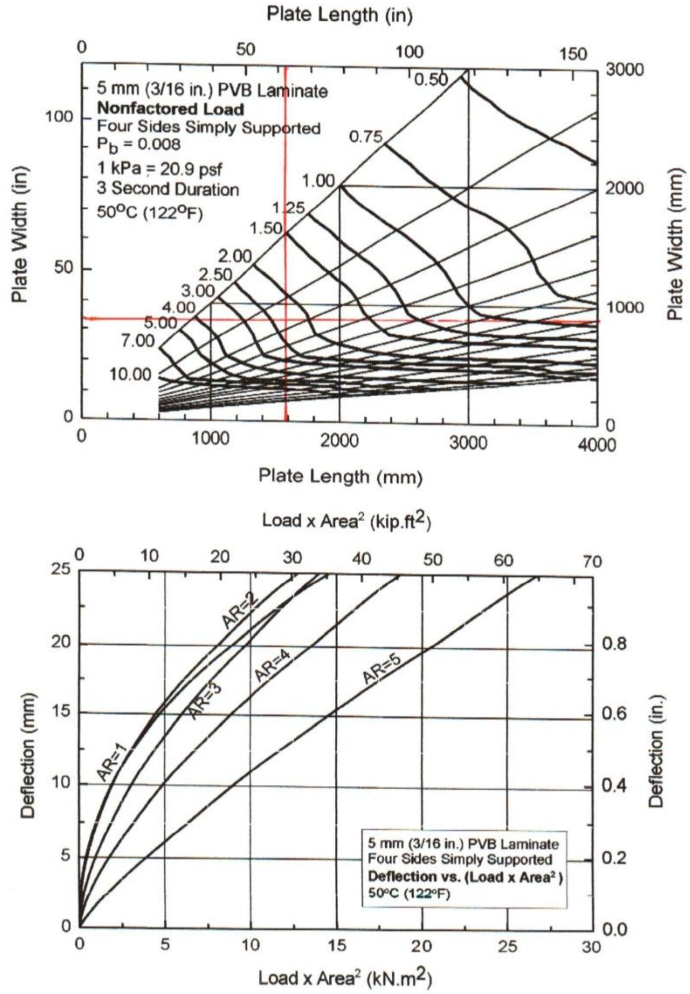

FIG. A1.27 (upper chart) Nonfactored Load Chart for 5.0 mm (3/16 in.) Laminated Glass with Four Sides Simply Supported
(lower chart) Deflection Chart for 5.0 mm (3/16 in.) Laminated Glass with Four Sides Simply Supported

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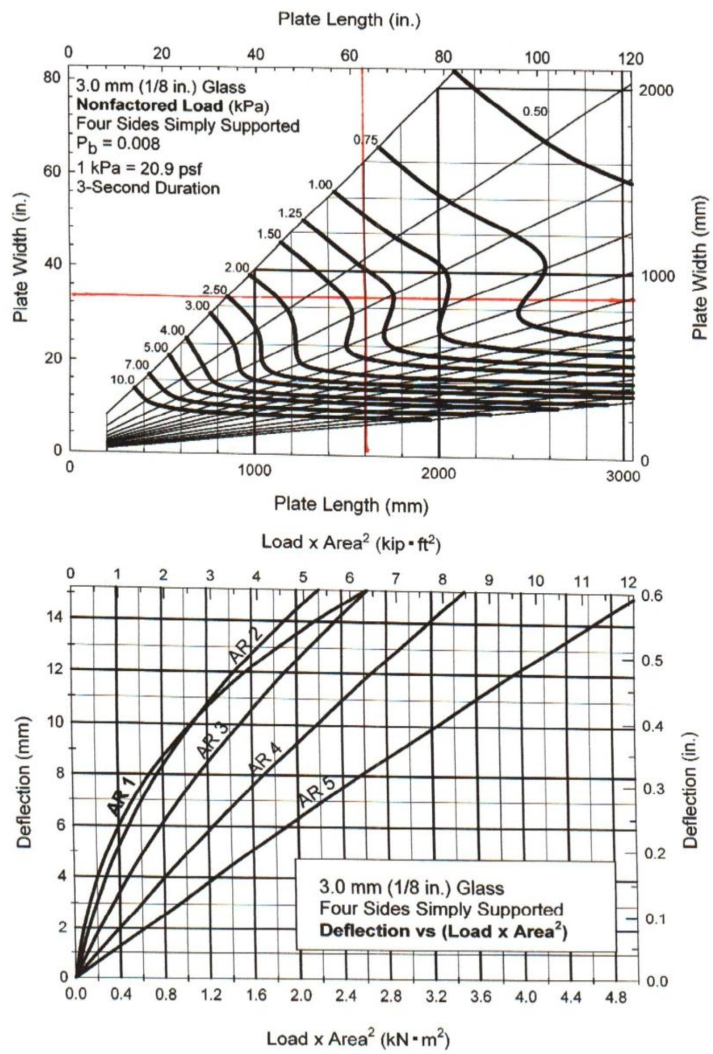

FIG. A1.3 (upper chart) Nonfactored Load Chart for 3.0 mm (⅛ in.) Glass with Four Sides Simply Supported
(lower chart) Deflection Chart for 3.0 mm (⅛ in.) Glass with Four Sides Simply Supported

A1.5.3 Enter the vertical axis of the chart at the point corresponding to the short dimension of the glass and project a horizontal line until it intersects the vertical line of A1.5.2.

A1.5.4 Draw a line of constant aspect ratio from the point of zero length and width through the intersection point in A1.5.3.

A1.5.5 Determine the NFL by interpolating between the load contours along the diagonal line of constant aspect ratio drawn in A1.5.4.

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