33 Harry Agganis Way

Boston, Massachusetts

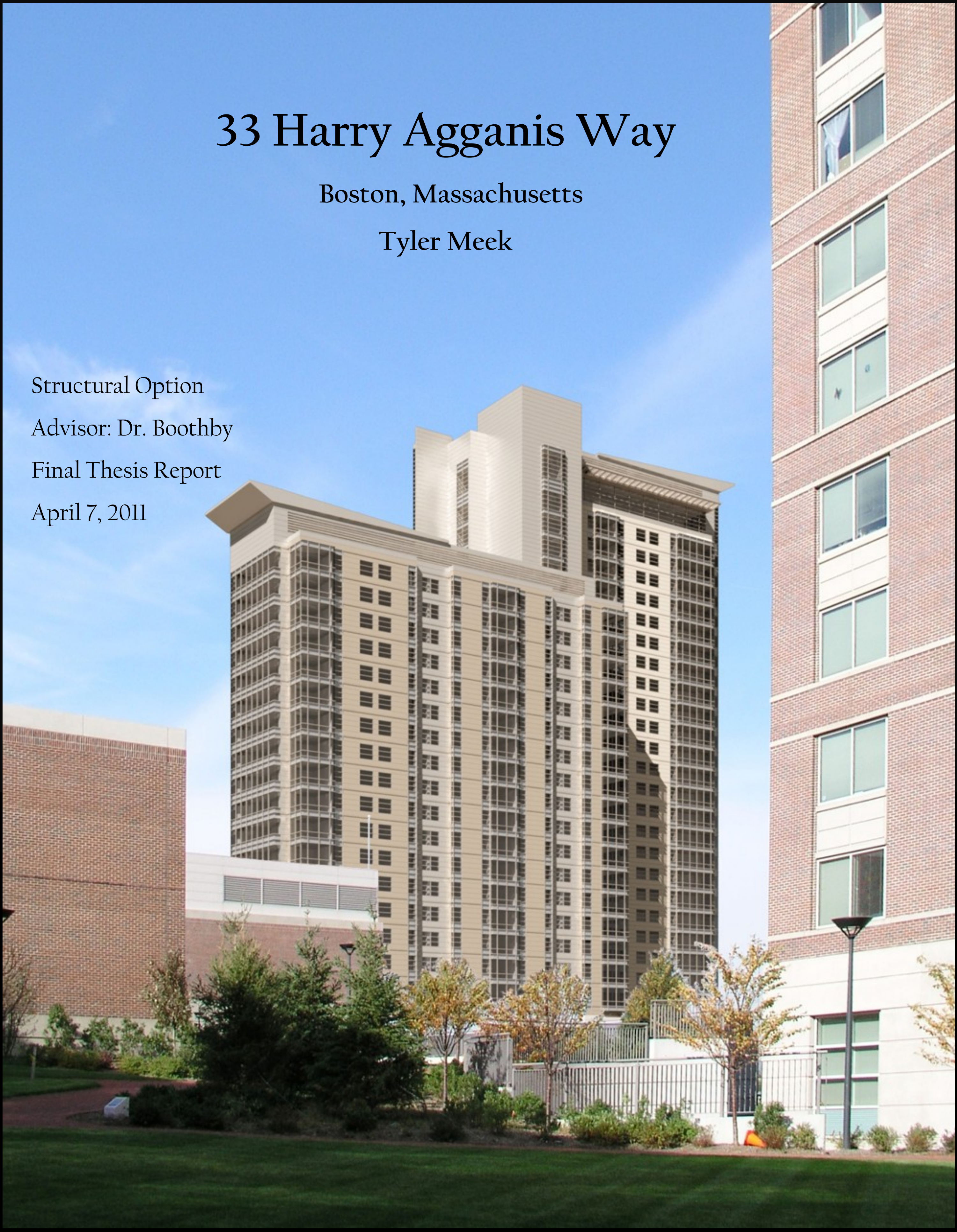
Tyler Meek

Structural Option

Advisor: Dr. Boothby

Final Thesis Report

April 7, 2011



BOSTON UNIVERSITY

33 Harry Agganis Way: Res Tower II

Owner:	Boston University
Site:	John Hancock Student Village
Occupant:	BU Student Housing
Type:	Residential
Size:	26 stories
	396,000 total sf

Project Team:

CM & GC:	Walsh Brothers
Architect:	Cannon Design
MEP:	Cannon Design
Structure:	Weidlinger Associates
Construction:	January '07 - April '09
Cost:	$291 Million

Architecture:

- Two-tower configuration sharing a common core and lower entry levels.
- South Tower is 19 stories and the north tower is 26 stories.
- Panelized terracotta and metal panel rainscreen exterior skin system.

Structure:

- Reinforced Concrete MAT foundations are 3'-9" for the shorter tower and 4'-3" for the taller tower
- Steel structure utilizes a braced framing system to transfer lateral loads to foundation
- Lightweight concrete slab on metal decking for composite floor construction

Mechanical:

- A desiccant wheel energy recovery ventilation system for all suites and apartments.
- Evaporative coolers on the ventilation units to supplement the air-cooled DX cooling system.
- ECM motors and a variable flow fan coil system for each HVAC unit serving each suite and apartment.

Electrical:

- Medium Voltage (13.8 kV) Service will be to tied to the existing Student Housing Phase 1 BU switchgear loop extension.
- Total Demand with Growth Factor (1.5) is 5,226 kVA
- Secondary distribution voltage will be 480 V (3 phase) to provide service to equipment loads and 208 V (3 phase) for dwelling demand loads.

Lighting:

- High efficiency lighting systems have been provided throughout the building. The average lighting power density is approximately 0.78W/sf, compared with the code allowed 1.5 w/sf. This has been achieved using high efficiency ballasts and luminaires

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Executive Summary

Res Tower II is a 26 story, 296 foot tall, dormitory located in Boston, Massachusetts. There are three levels of public space with 23 levels of private study and living spaces. A steel framing system supports the lightweight concrete composite floor system and lateral loads are resisted by moment connected steel braced frames pinned to a mat foundation.

The goal of this thesis was to design a staggered truss system for Res Tower II and investigate the most efficient use of the trusses. Investigations were made into using the staggered truss system to resist 100% of both gravity and lateral loads or using it to support the gravity loads only and designing a new appropriate lateral system. AISC Design Guide 14: *Staggered Truss Framing Systems* was followed closely in the design of truss members and connections.

An acceptable shear wall design was completed but the wall thickness was larger than desired. For this reason, a moment frame was implemented into the structure and wall thicknesses decreased. To design the most efficient structural system, an investigation was completed to find an appropriate height to stop the moment frames and allow the shear walls to continue for the remainder of the building height.

Recognizing that changing the structure of the building will impact all parts of its design, studies were completed for the architectural and construction impacts a staggered truss system would have on Res Tower II.

There were three main areas of concern for the architectural study. In each of these spaces, a rendering was done to analyze how an exposed truss would affect the interior architectural dynamic. In some cases, the truss had to be removed to avoid negatively affecting the architecture but in one case, it was decided to keep the truss in the system and keep it exposed because it added excitement to a mundane space.

A new site logistics plan and construction schedule were created to adjust for the new structural system. This involved studying the surrounding buildings, deciding on a proper site layout and determining construction durations for five main steps of the construction process.

Two highly repetitive truss connections were designed to meet the MAE requirements for this thesis. To allow for construction ease and by following typical practice, connections were designed using bolts and welds depending on the type of connection.

Acknowledgements

Through this project I have learned many things both professional and personal. A few pieces of knowledge gained are:

1. Check thrice, design twice
2. In most cases, pencil and paper are better than a computer model and an analyze button
3. No matter what happens, the sun will always come up the next day

By no means could I have done this year long project on my own. I would like to thank the following people for the professional support they have given during this project, as well as the mentoring role some of them have taken. Thanks also to Cannon Design for providing me with my thesis building and the required drawing set.

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Dr. Hanagan
Dr. Boothby
Dr. Geschwindner
Prof. Bob Holland
Ryan Solnosky

<u>Family:</u>

Thank you for all the support you have given me over the years, even though you may not understand exactly what I have been working on, you have always given more than I could have asked for. I can't thank you enough for allowing me to make my own decisions and mistakes along the way to becoming my own person.

My AE guys:

Thank you for putting up with me over the past five years. I cannot believe how lucky I was to find such a great group of people to spend almost every hour of my time at Penn State with. Best of luck in everything you do and I can't wait to start our own super firm one day.

I'd also like to thank God. I have been blessed in many aspects of my life and am very thankful for it. I hope that I can use my talents for His glory.

Introduction

Located on the Boston University Campus, 33 Harry Agganis Way, which will be referred to as Res Tower II, is a 26 story, steel framed dormitory. It is located on the northwest corner of the John Hancock Student Village, bordered by the Charles River and Commonwealth Ave. Because two more dormitories are planned for the JH Student Village and the cost of developing in Boston is so high, the footprint of Res Tower II had to be as small as possible, thus forcing the structure upwards.

The south tower is 19 stories tall with a fan room and mechanical penthouse on the top level. A student activity space, with large windows and a terracotta surfaced walkout space, occupies the 26th story of the north tower. The roof of the north tower supports a fan room, large air handling units and other large service equipment. Floors 3 through 25, aside from the spaces mentioned above, are all private residential areas with some study rooms and computer labs mixed in. The first two levels of Res Tower II serve as the public and service offices for the rest of the building.

The façade of Res Tower II is a panelized skin comprised of terracotta and a metal panel rainscreen. This façade is a curtain wall system with its self-weight being supported by the floor above it; this can be assumed to be a continuous load due the small spacing of hung supports.

Res Tower II utilizes four main roof systems, all of which include gypsum under-laminate board, a vapor retarder and an adhered roofing membrane; the prior three aspects will be referred to as the typical roof assembly. Where mechanical equipment is being supported the typical roof assembly is placed on concrete deck while on the outer edges of the building, a metal deck is used. On the 26th story, to support the walkout space mentioned above, terracotta pavers on concrete deck are combined with the typical roof assembly to create an attractive and durable roof system.

Existing Structural Systems

Foundation

Haley & Aldrich performed the geotechnical studies for the JH Student Village area and provided the report in which H&A explain site and below-grade conditions along with recommendations for the structure. A net allowable soil bearing pressure of 6 kips per square foot (ksf) was recommended for the design of foundations on the natural, undisturbed glacial deposits below the site. A recommended design groundwater level was also given which is on average 10-12' below the bottom of the existing foundation.

Res Tower II utilizes a mat foundation system with two main thicknesses, 4'-3"and 3'-9". Logically, the taller tower is supported using the deeper mat foundation to resist the higher loads transferred by the braced frames. The foundation step occurs between grid lines 9 and 10. The typical reinforcement in the east-west direction is #10's spaced at 10" on center, top and bottom while in the north-south direction, the reinforcement is #9's spaced at 10" on center, top and bottom. Additional reinforcing cages are placed under the braced frame columns with the anchor bolts of these columns being tied to the bottom of the cage to increase the resistance to uplift. A detail of this connection is shown below in figure 1.

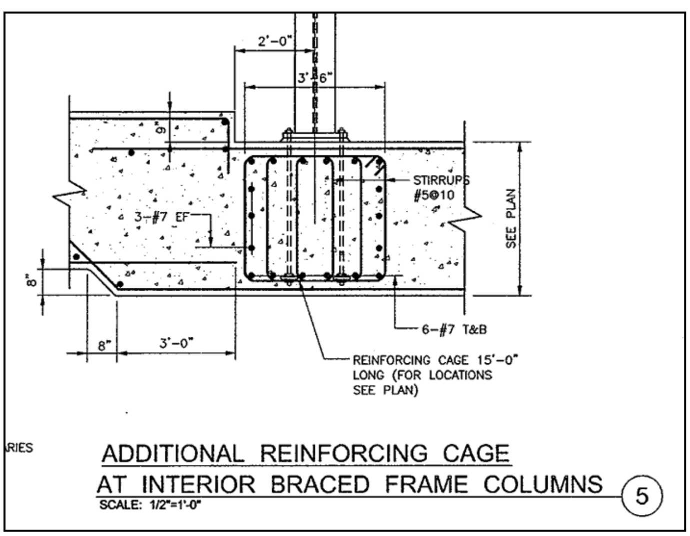


Figure 1: Additional foundation reinforcing

A 9” deep trench runs along the center of each tower’s foundation, parallel to the length of the building. This trench is filled in with 4000 psi concrete and reinforced with welded wire fabric after the erection of the interior columns in this area. In figure 2 below, the trench is shaded and outlined in red with the lateral force resisting columns marked in blue.

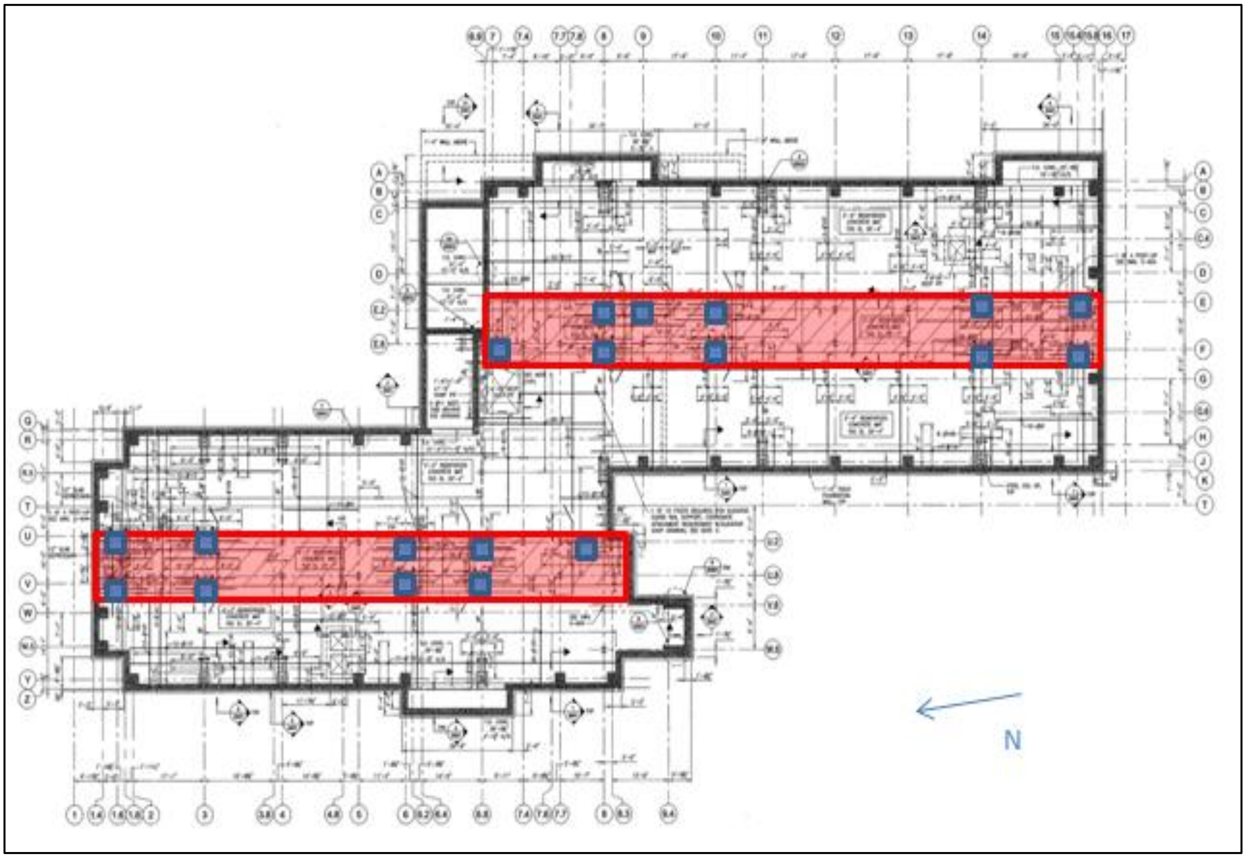


Figure 2: Foundation Trench

Floor Construction

The typical floor construction for Res Tower II is 3” 18 gage galvanized steel deck with 3-¼” lightweight concrete topping and welded wire fabric reinforcement. This is used everywhere except the loading dock and trash compactor area on the first floor. The floor system for these areas is comprised of 3” 16 gage steel deck with 6” normal weight concrete topping, a total thickness of 9”, and epoxy coated reinforcement of #7’s spaced at 12” on center in the bottom of the flutes and #5’s spaced at 12” on center in the top running each way. All deck is designed to act compositely with beams.

Decking typically spans about 8’-9” supported by beams ranging in size from W14s to W18s. These composite beams span roughly 23 feet to girders or columns. The girders have the same range in size as the beams. These spans create a typical bay size of 17-18’ by 24’-23’. The actual bay sizes vary moderately from typical dimensions. Figure 3 shows a typical framing plan for floors 3-18.

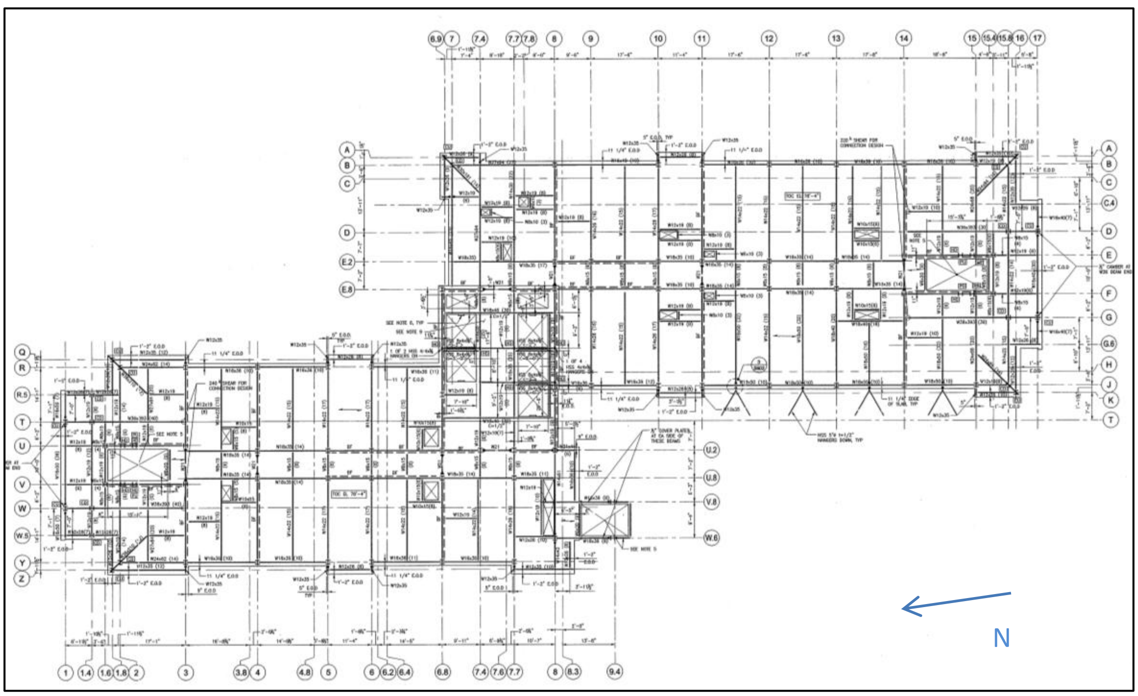


Figure 3: Typical Framing Plan

Lateral System

Steel braced frames are used to resist the lateral loads placed on the structure. At the termination of these columns, extra reinforcement is added to better tie the columns to the foundation and resist uplift forces. All columns in these braced frames are W14's ranging in size from W14x61 near the top of the structure to W14x398 for the bottom columns. The diagonal bracing members are W12's ranging in size from W12x152 to W12x45. This braced frame construction is categorized as a concentrically braced frame in ASCE7-10 for which an R value of 3.25 is prescribed but due to the moment connections, an R value of 5 was used by the engineer of record. To allow for corridors to pass through the center of these braced frames, moment connections were made. Figure 4 shows an elevation of a braced frame with the moment connections clearly shown.

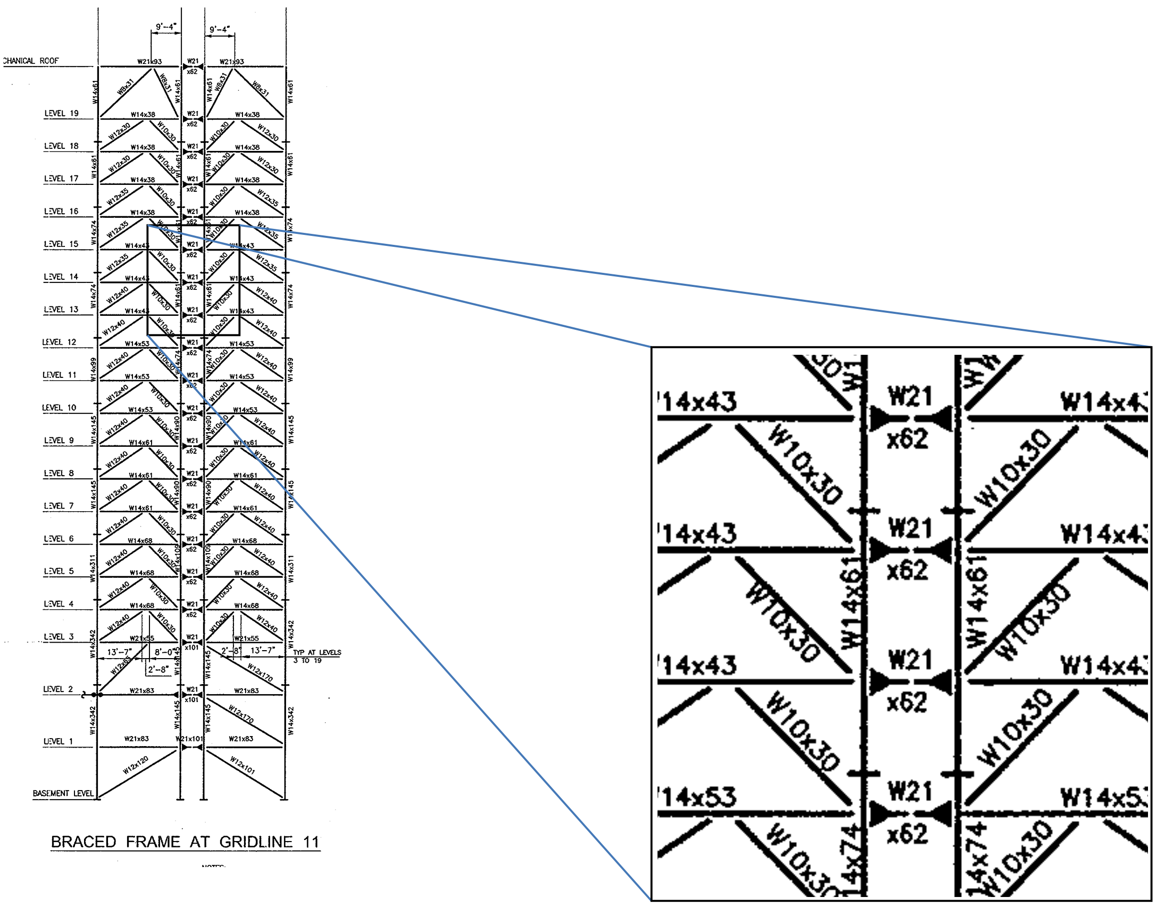


Figure 4: Braced frame elevation with moment connection

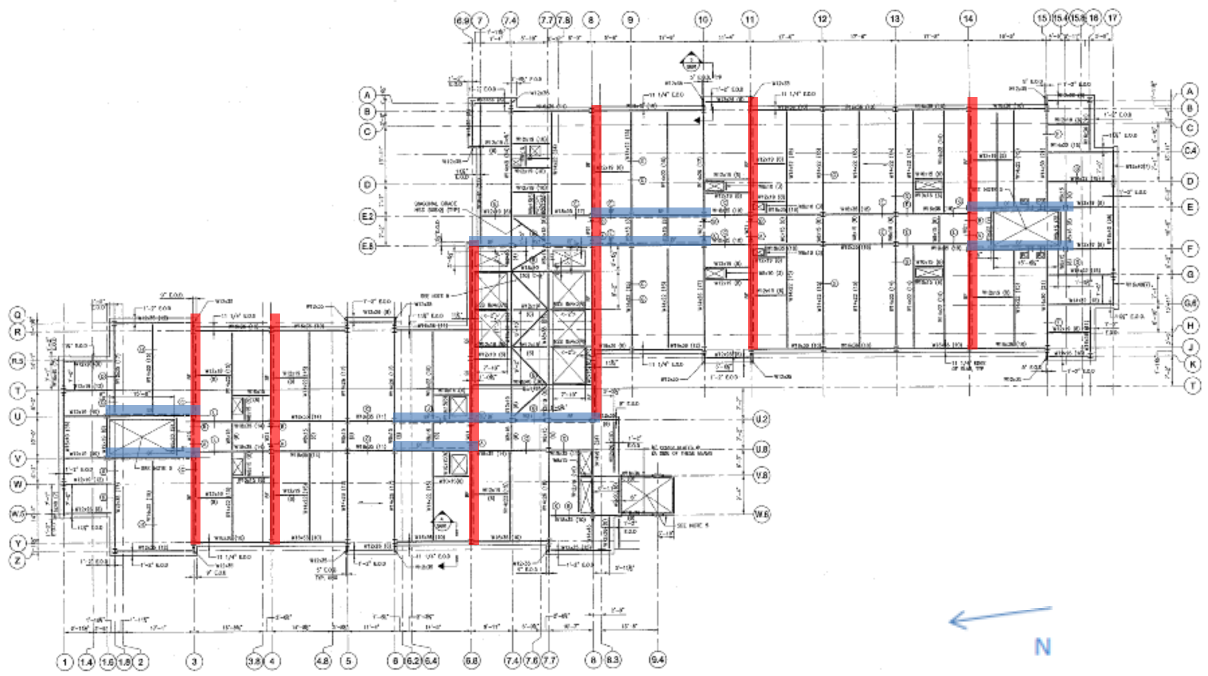


Figure 5: Typical plan with braced frame locations highlighted

Figure 5 shows the location of the braced frames in plan. The braced frames supporting loads from the short side of the building are highlighted in blue and the braced frames supporting loads from the long side are highlighted in red. Frames running parallel to the long direction are on average shorter than frames running in the perpendicular direction; this is permissible because the loads from the short side are much smaller in magnitude than the loads from the long direction.

Proposal Objective

Structural Depth:

As part of Technical Report 2:*Pro-Con Structural Study of Alternate Floor Systems* an investigation was made into using a staggered truss system to support the gravity loads of Res Tower II. This system was found to meet all strength requirements and proved to be a viable option that would not only allow for open space but also work well with the existing floor plan. Only a gravity analysis was performed for the staggered truss system and therefore more studies have been done to fully understand how well this system can be implemented into Res Tower II. The main concern was how the trusses react when subjected to lateral loads. Multiple options have been evaluated to determine the best use of the trusses. These options are:

- Designing the trusses as the main lateral load resisting system.
- Using the staggered truss system as a strictly gravity system and using the existing lateral system
- Designing a new lateral system that works well with the existing floor plan and using the trusses to only resist gravity loads.

To fully understand how effective a staggered truss system would be if implemented into Res Tower II, disciplines other than structural design needed to be considered. As part of this report, the architecture discipline and construction management discipline will be investigated.

Architectural Study:

Locations of trusses had to be carefully planned with respect to the architectural floor plans to avoid negatively affecting open spaces. In most cases, the trusses can be enclosed by walls in the existing floor plan but in three spaces the trusses would need to be exposed if they were to be kept in the structural system. These spaces will be discussed in more detail in a later section.

Construction Management Study:

A staggered truss system uses a large quantity of prefabricated members and therefore the construction schedule and site logistics for this system will be different than what was used for the existing “stick built” system. A new site logistics plan and construction schedule were designed based on assumptions made during the design and typical management practice.

MAE study; Connection Design:

The staggered truss system is comprised of shop welded members that are bolted to the column web. This particular system uses a large amount of repetitive members and therefore repetitive connections. The following two connections were designed:

1. Diagonal and vertical web members to the bottom chord member (welded)
2. Top chord member and diagonal web member to column web (bolted)

Structural Redesign

Background

A staggered truss system utilizes a story deep Vierendeel truss that replaces the need for interior columns by spanning the entire width of a structure. Res Tower II has a favorable layout for the use of a staggered truss system because it has long outer spans that support private areas with no intermediate doorways and a short interior span for a central corridor. This is a good match to the layout of the staggered truss system because the vertical web members framing the center span allow space for the corridor while the private living spaces allow for diagonal members to run in the outer two spans.

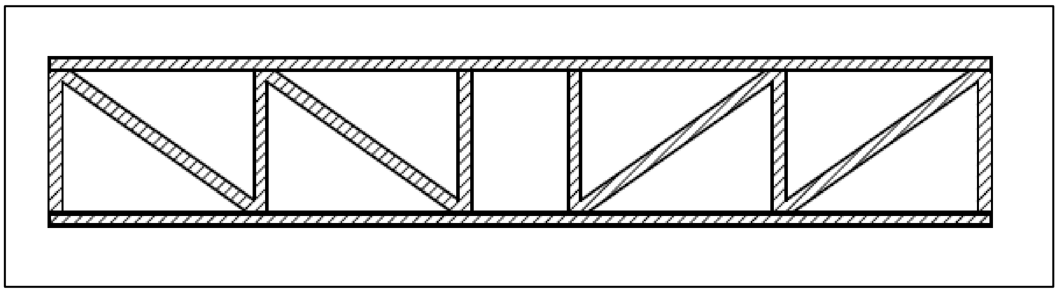


Figure 6: Typical elevation of truss

The image above gives the elevation of a typical truss that will span the entire width of the structure. The top and bottom members are continuous with all members connected using gusset plates. *AISC Design Guide 14: Staggered Truss Framing Systems* provides a summary of the systems history, descriptions of the typical materials, design equations, and a design example of how to use the equations and what assumptions can be made during the design process. Design Guide 14 recommends using W10 shapes for the top and bottom members and HSS shapes for the vertical and diagonal web members.

To maintain a 10 ft floor-to-floor height, the trusses were designed to be 9'-6" tall. A 6" concrete composite deck will be used for the floor system to match the existing conditions. Supporting the concrete deck will be metal joists that span from truss to truss. This system is unique compared to most truss systems because both the top and bottom flange are loaded vertically.

The figure below shows a schematic view of the structural system with both the top and bottom chord members being loaded. Figure 8 also gives a closer look at an individual truss used in this system. It is clear why this system is given the name “staggered;” trusses skip a bay on each level and locations are staggered from level to level.

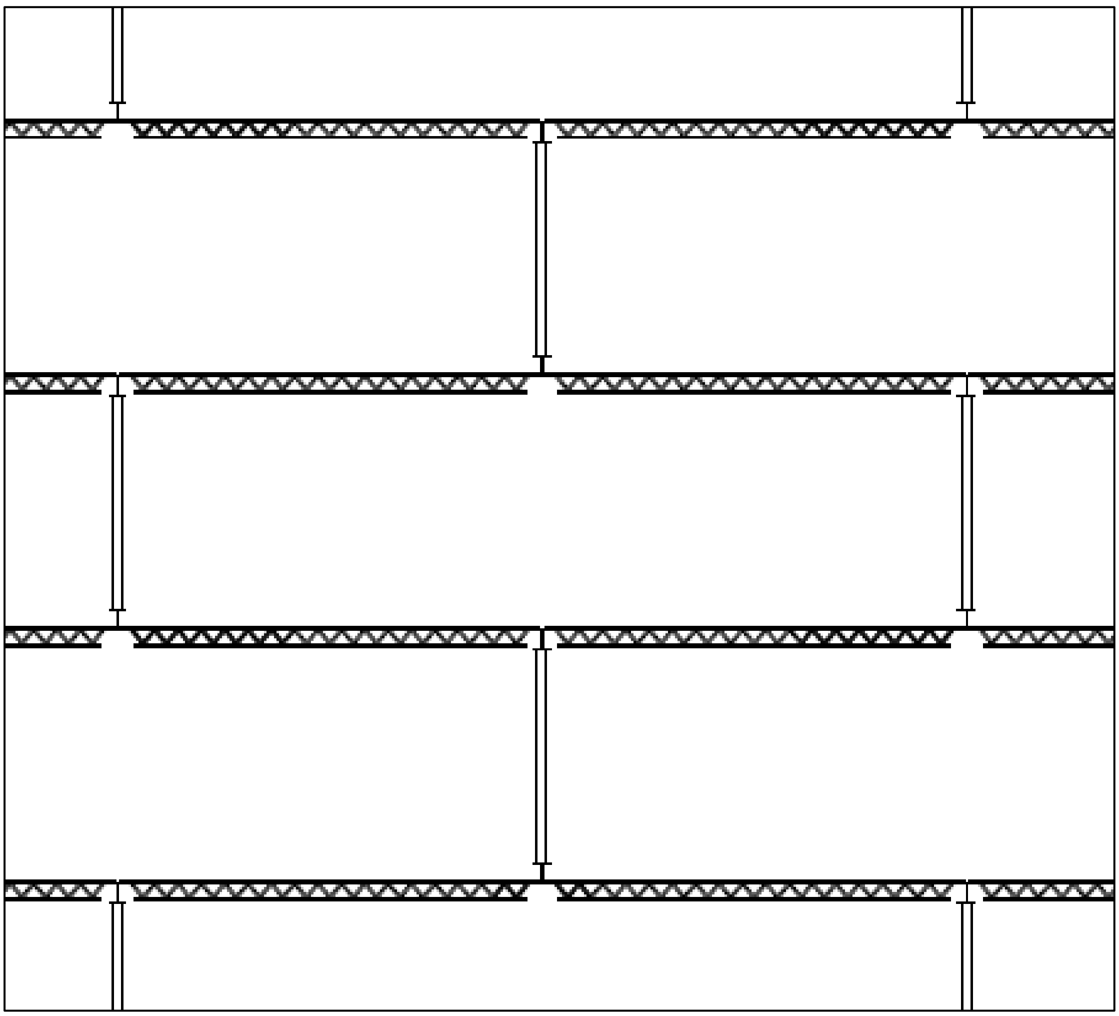


Figure 7: Schematic representation of structural system

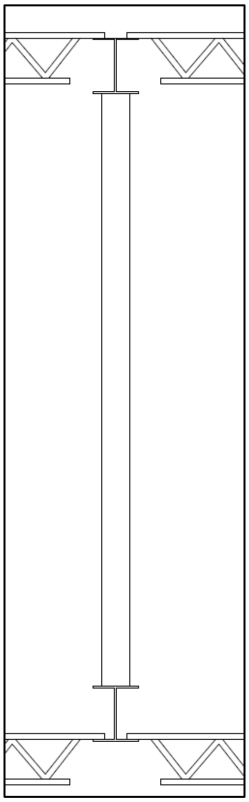


Figure 8: Truss loading scheme

A minimum width requirement is placed on the flanges of the chord members to ensure the flange has enough bearing area and will not sag under the load. A visual representation of this limit is provided in figure 9. One requirement that is not mentioned in the design guide is on the bottom chord member a designer must confirm that the member loading the truss (a joist in this case) is short enough to not contact the web of the W shape.

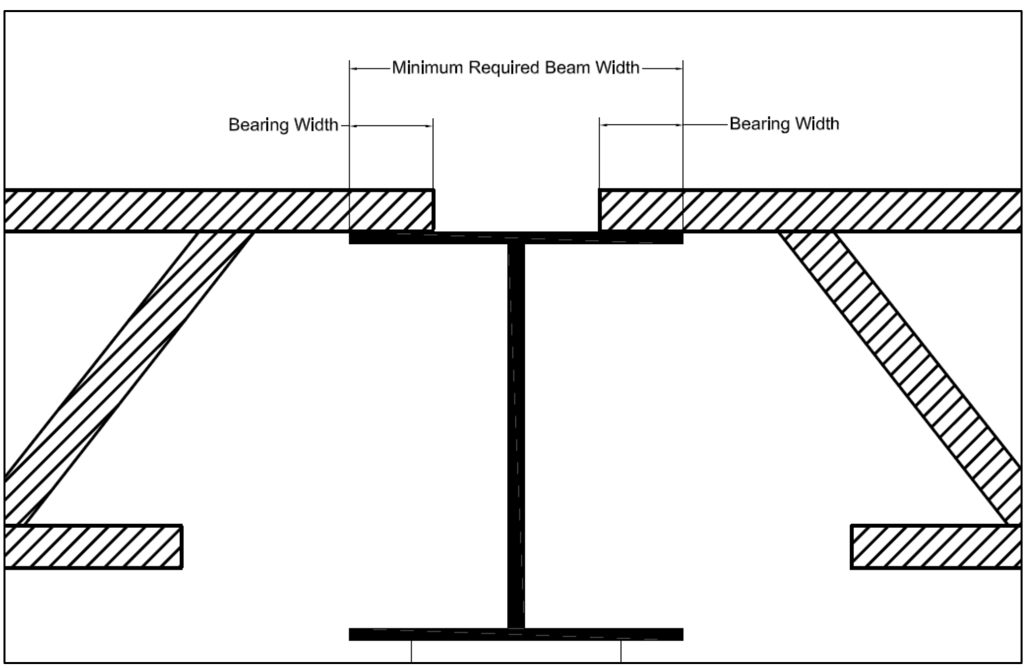


Figure 9: Width requirements

Truss Locations

Trusses were typcially spaced as close to 17 ft as possible to create a reasonable tributary width for each truss and an efficient span for the joists that will run from truss to truss. Preliminary truss locations were determined with only gravity loads in mind and with a goal of creating prime load paths. The image below shows these preliminary locations with each color representing the two levels.

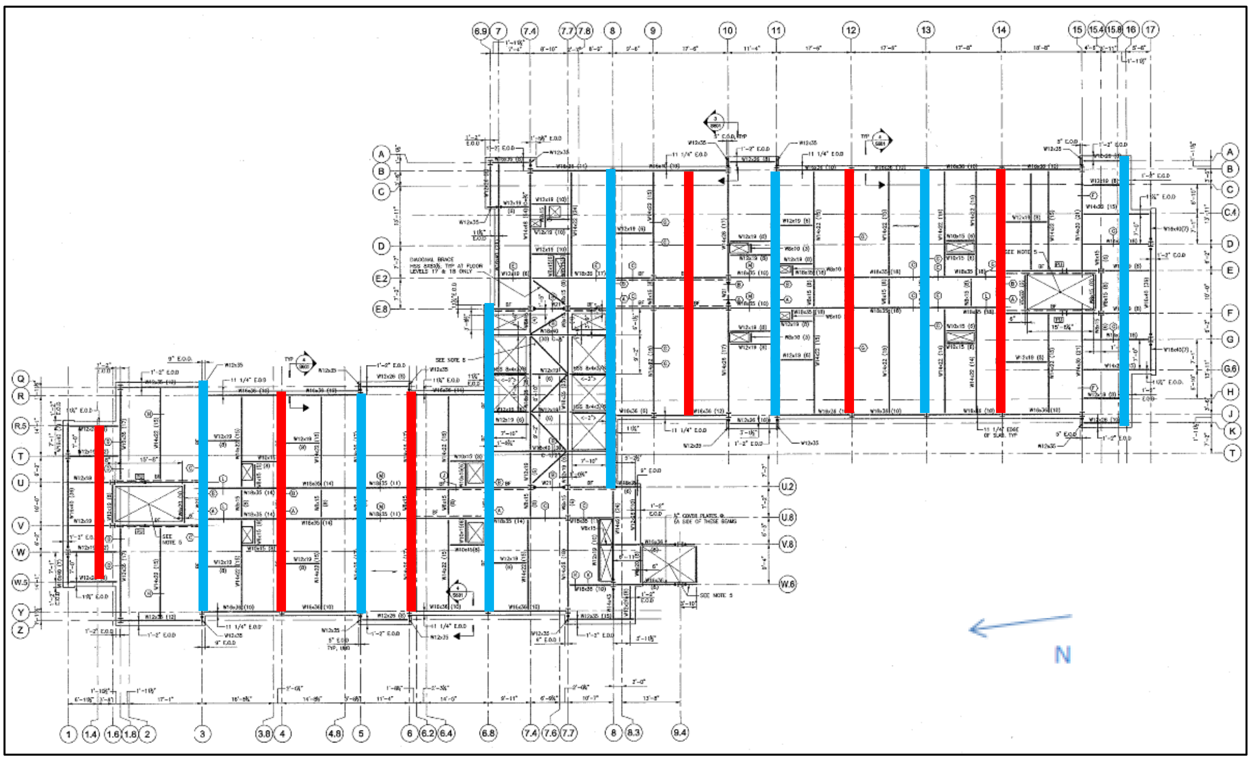


Figure 10: Preliminary truss locations

After intial locations were chosen, they had to be adjusted with regard to the architectural organization of the building. Figure 11 shows which trusses had to be removed at this point of the design. In order to accommodate the existing floor plan and maintain programmed square footage, the two trusses on each end of the structure had to be removed. Figure 12 shows that if these trusses were not removed, they would run through the middle of an existing bedroom. The central truss was removed to avoid large transfer forces from the altered floor plan of levels 19 through 26.

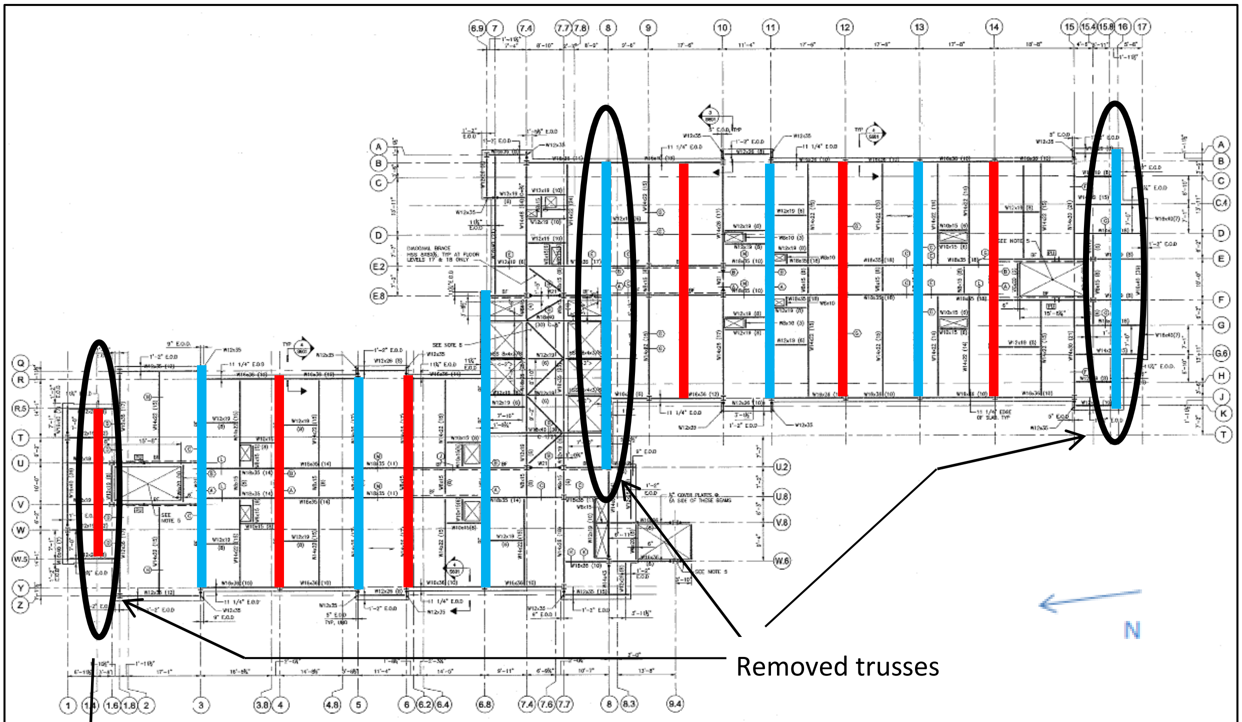


Figure 11: Removed trusses

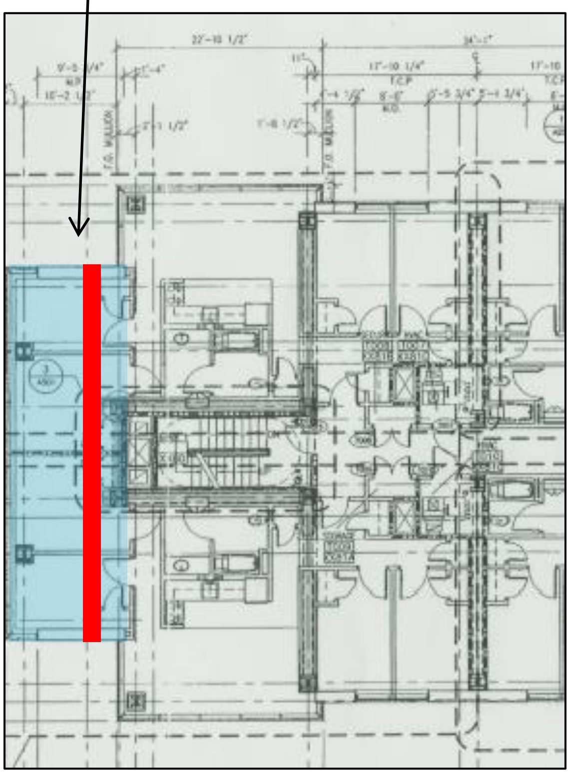


Figure 12: Architectural plans with truss

As stated above, figure 12 shows the typical architectural plan for the residential levels with the problem truss highlighted in red and the bedrooms that would be interrupted in blue.

Because trusses could not be used in these locations, the existing framing plan can be used in the vertical circulation areas in the middle of the building and at each end. Another problem area is the main lobby on the first floor.

The rendered images of figures 13 and 15 represent the conditions of the existing lobby. It has a large, open area with four very large columns in the middle that create a central lounge and waiting area. This area will be framed in the same manner as the existing design to allow for this architectural feature, which was originally designed to satisfy the owner's request, to be maintained. The matching images in figures 14 and 16 show what this area would look like if a truss were to be kept in the current location. This subject will be discussed in more detail in the Architectural Study section later in this report.

Figure 13: View from desk of existing lobby

Figure 14: View from desk of lobby with truss

Figure 15: View from main entrance of existing lobby

Figure 16: View from main entrance of lobby with truss

Truss Member Design

Following chapter 3 of *AISC Design Guide 14: Staggered Truss Framing Systems*, hand calculations were completed for unfactored live, dead and lateral loads on one typical truss. It was assumed that gravity forces only acted at panel points as per the recommendation from the design guide. The trusses were solved using an adapted method of joints. Because the chord members are continuous, they will carry some moment and the typical method of joints technique could not be used. After gaining a firm understanding of this process by completing it by hand, an Excel spreadsheet was used to find the factored controlling load case. It was determined that 1.2D +1.6L caused the greatest forces in all members of the truss. The spreadsheet used to find the controlling load case is presented in appendix A. The gravity loads used in this analysis are presented in tables 1 and 2.

It was determined that 3” 18 gage composite steel decking supported by 14K4 joists will provide the necessary strength to support gravity loads and transfer them to the trusses. Special detailing or tack welding may need to be specified to ensure lateral forces are transferred from the diaphragm to the trusses.

Live Loads		
	Design Load (psf)	Thesis Load (psf)
Occupancy Type	Mass. State Building Code	IBC 2009 & ASCE7-10
Public Area	100	100
Corridor	80	100
Dwelling Unit	40	40
Loading Dock	250	250
Mechanical Penthouse	150	125
Roof	30	20

Table 1: Live Loads for Res Tower II

Dead Loads	
Material	Load (psf)
Slab	
-Roof Deck	56
-Floor Deck	46
Façade	18
Superimposed	30

Table 2: Dead loads for Res Tower II

Member sizes were selected for typical top and bottom chord members as well as the web members. W10x33 members were chosen for the chord members and HSS10x5x5/16 members were used for the vertical and diagonal web members.

See appendix A for the hand calculations used to determine member sizes.

To ensure that no errors were made and that deflection would not control the design, a RISA-2D model was made of one truss. The original RISA model is shown in figure 17 and had a deflected shape presented in pink in figure 18. As can be seen from the images, only the bottom chord is pinned where it would meet an exterior column and the top chord is allowed to deflect freely. Because the top chord was free to rotate around the pinned support, the deflection values at midspan were approaching the code specified limit of $\ell/240$ which equals 2.96 inches. Although the deflection was still below code values, the complete structure needs to be considered. If the trusses deflect two inches and the members spanning from truss-to-truss deflect one inch; that gives a total deflection of 3 inches, which is not acceptable.

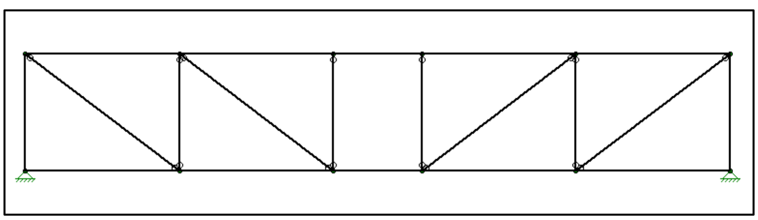


Figure 17: Original RISA model

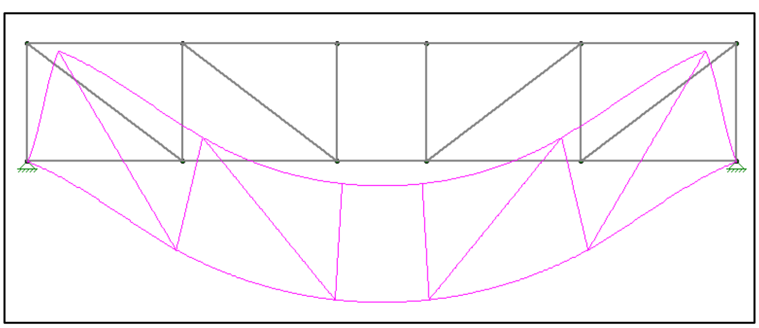


Figure 18: Original RISA model with deflected shape

After further consideration of the real structure, the RISA model was changed to what is shown in figure 19. Instead of placing supports on the truss itself, exterior columns were modeled and pinned at floor heights above and below the truss. Figure 20 shows the deflected shape of the updated model. This is a more realistic model because the columns will prevent the top chord from rotating and translating freely. It was important to model the column with weak axis bending because a moment frame may run in the perpendicular direction of the truss. Deflections of this model were well below code limits, $\delta = 1.28$ inches. This is an acceptable value that will not greatly affect the overall system.

Loads were then changed from point loads to linearly distributed loads along the length of the truss. The assumption of using point loads in the hand calculations was proven to be an accurate and conservative assumption because changes in forces and deflections were marginally less than what was previously calculated.

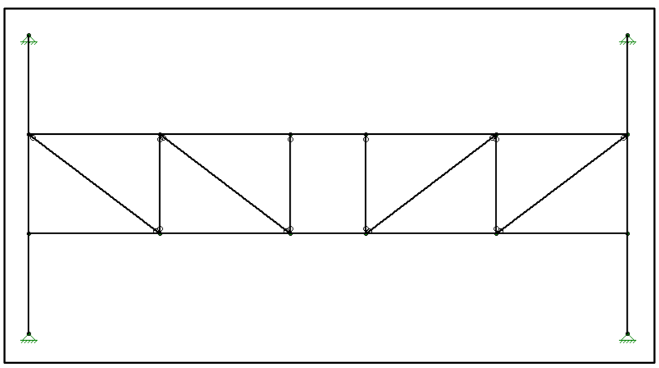


Figure 19: Updated RISA model

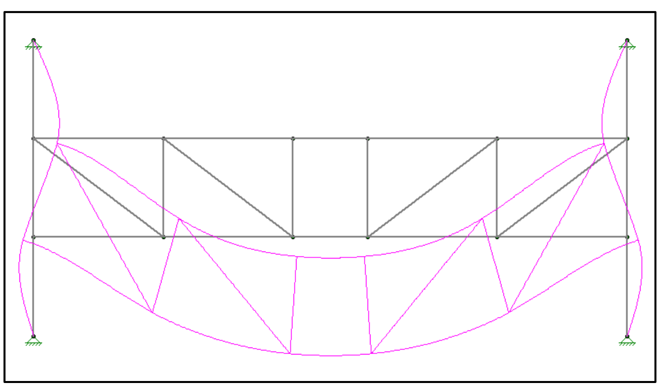


Figure 20: Deflected shape of updated RISA model

Gravity and Lateral System

A controlling load case of 1.2D +1.0L + 1.6W was used to combine gravity and lateral loads. After reading sections of AISC7, it was determined that it was a reasonable assumption to use wind as the contolling lateral load parallel to the short direction and to use seismic parallel to the long direction. The wind loads on Res Tower II were calculated as part of Tech 3:*Lateral System Analysis and Confirmation Design* and are presented in table 3.

East West	
Floor	Force (k)
1	53.26
2	106.87
3	90.06
4	74.18
5	76.54
6	78.50
7	80.27
8	82.01
9	83.48
10	84.57
11	85.59
12	86.70
13	87.82
14	88.88
15	89.73
16	90.42
17	91.22
18	92.09
19	92.84
20	70.88
21	49.29
22	56.08
23	62.33
24	62.67
25	75.63
26	95.06

Table 3: Wind loads at center of pressure

Forces presented in this table were calculated as acting at each floors center of pressure. Relative stiffness of each frame was used to determine the percentage of the load to be assigned to each frame. Because each truss has the same profile, the length of the truss was used as the absolute stiffness value. Once a center of rigidity was established using a similar calculation, additional force was added to each frame from torsion caused by the difference in center of gravity and center of pressure locations. The spreadsheet used for this calculation can be found in appendix B.

Once the members of each truss were designed for strength and checked for serviceability, it was important to understand how multiple bays of full height truss systems react under both gravity and lateral loads. A multi-bay model is also helpful because drift values may be more extreme in single bay model due to the staggered truss layout. Every other level has zero stiffness when there is no truss at that level, therefore modeling three trusses will ensure that each level has atleast one truss and therefore stiffness.

Figure 23 to the left shows the preliminary model used to investigate how efficiently a staggered truss system will resist lateral forces parallel to a trusses length. A view of one individual bay is shown in figure 24; this will be useful when discussing the deflected shape in figure 25. Design Guide 14 recommends bracing the bottom truss to the column below. This is done to strengthen the bottom of a structure for gravity loads and to prevent first story mechanism failure or pancaking.

Maximum deflection at the 19th floor was 8 ½" inches which is far above allowable code limits and therefore unacceptable. By visually investigating the deflected shape, it is clear that the chord members of the bottom truss are over stressed and need to be reevaluated.

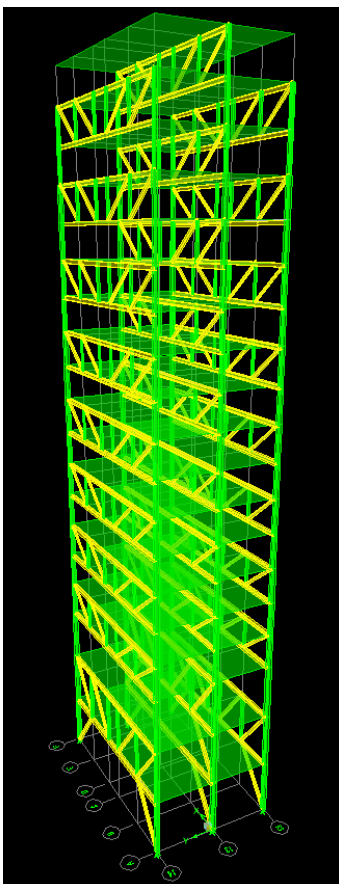


Figure 23: Three bay model

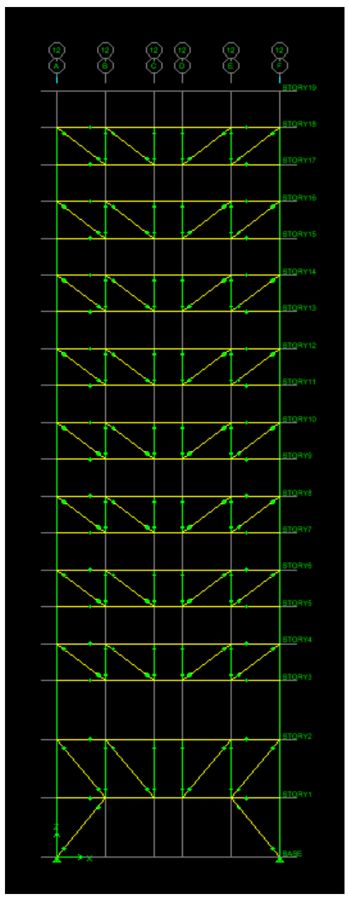


Figure 22: Individual bay of three bay model

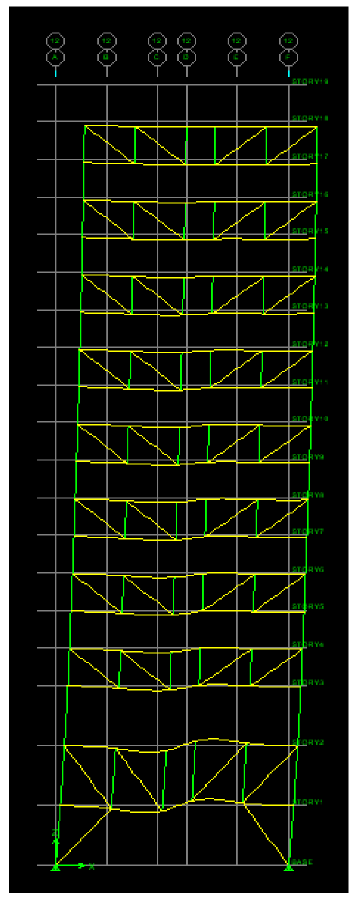


Figure 21: Deflected shape of individual frame

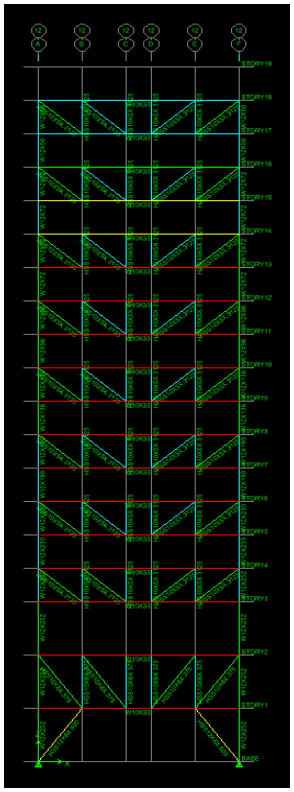


Figure 24: Representation of member stresses

The steel design feature of ETABS provides a visual representation of member stresses with color. Any member colored red is over stressed and therefore is straining under the assigned load. Figure 24 presents the output of a steel design check for this bay. As stated above, the chord members of most of the trusses need to be redesign with lateral loads in mind.

By building a preliminary three bay model instead of constructing a model for the entire structure, an issue was found while there was still a manageable amount of members to check.

Using the information from the preliminary model of three bays, the model in figure 25 was created for the entire structure. After placing all the necessary lateral loads on the trusses, it was determined that the staggered truss system will not efficiently provide the required stiffness to resist the loads. In the process of trying to size chord members, only unreasonably large W shapes had the capacity to limit the lateral deflections. Changing from W10 chords to a deeper member caused almost 25% less deflection, but the value was still not acceptable. Again, an increase in member depth decreased deflections by roughly another 20% but like the first time, deflection values did not meet the required criteria. After multiple attempts to obtain a permissible deflection value by increasing member sizes, the conclusion was made that for this particular building a staggered truss system will not work to carry 100% of the lateral loads. The graph below (figure 26) presents the findings of this investigation. It shows that an exponential curve can be approximated to represent the relationship of lateral deflection to member size. Arbitrary, unitless values were used to obtain the desired shape of the curve. It shows that an increase in member size can greatly affect the deflection initially but it is not an efficient method of reaching an acceptable deflection.

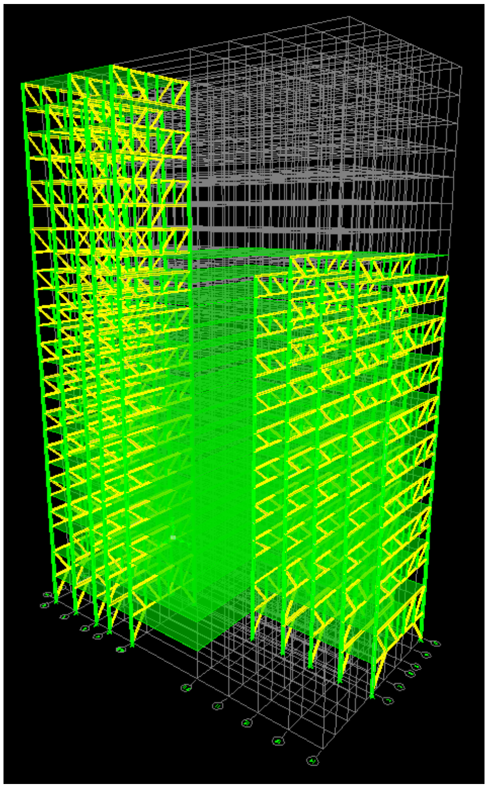


Figure 25: Model of entire structure

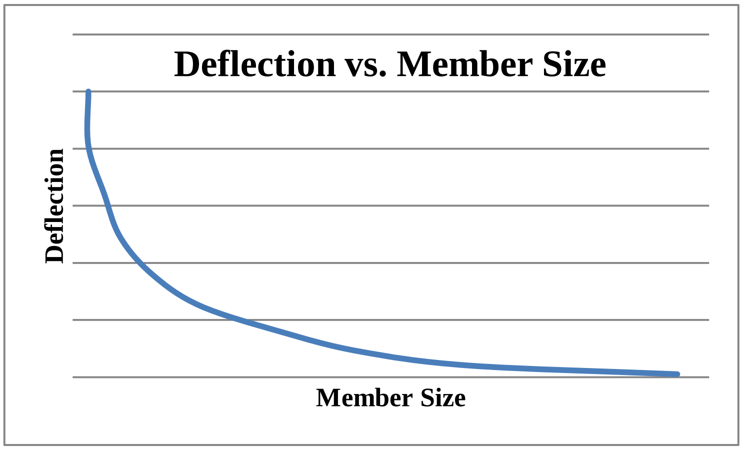


Figure 26: Graphical representation of deflection vs. member size

Using the staggered truss to resist 100% of the lateral loads is not an efficient design for Res Tower II. This system may function better under lateral loads for shorter structures or for buildings with different demands but for Res Tower II, a separate lateral system must be designed.

Originally, the existing lateral system which consists of braced frames was to be combined with the staggered truss system to resist lateral loads. Implementing braced frames into the staggered truss system and requiring interior columns would be counterproductive. The main selling point of a staggered truss system is it removes the need for interior columns and allows for more open space at each level. Therefore this option was not investigated further due to its obvious negative impact on the overall system.

New Lateral System

The staggered truss system is very efficient for supporting gravity loads but under lateral loads, the column-truss interaction will not provide sufficient stiffness with reasonable member sizes. Therefore a new lateral system had to be designed that works well with the existing floor plan and does not physically contact the trusses. If the new lateral system physically contacts or ties to the trusses an undesired interaction may occur causing unforeseen errors.

The decision was made to use concrete shear walls to create a center core with another set of walls at the ends. The central core is continuous through the full height of the building, along with shears walls in the north end. Compared to concentrically braced frames, a concrete shear wall system will not require interior column. Also, the floor plan of Res Tower II lends itself well to accommodating shear walls.

As an initial design, a trial size of 24 inches was chosen for the thickness of the shear walls to match the existing wall thickness of the elevator shafts. Locations of the shear walls are denoted in figures 27 and 28 as thick red lines.

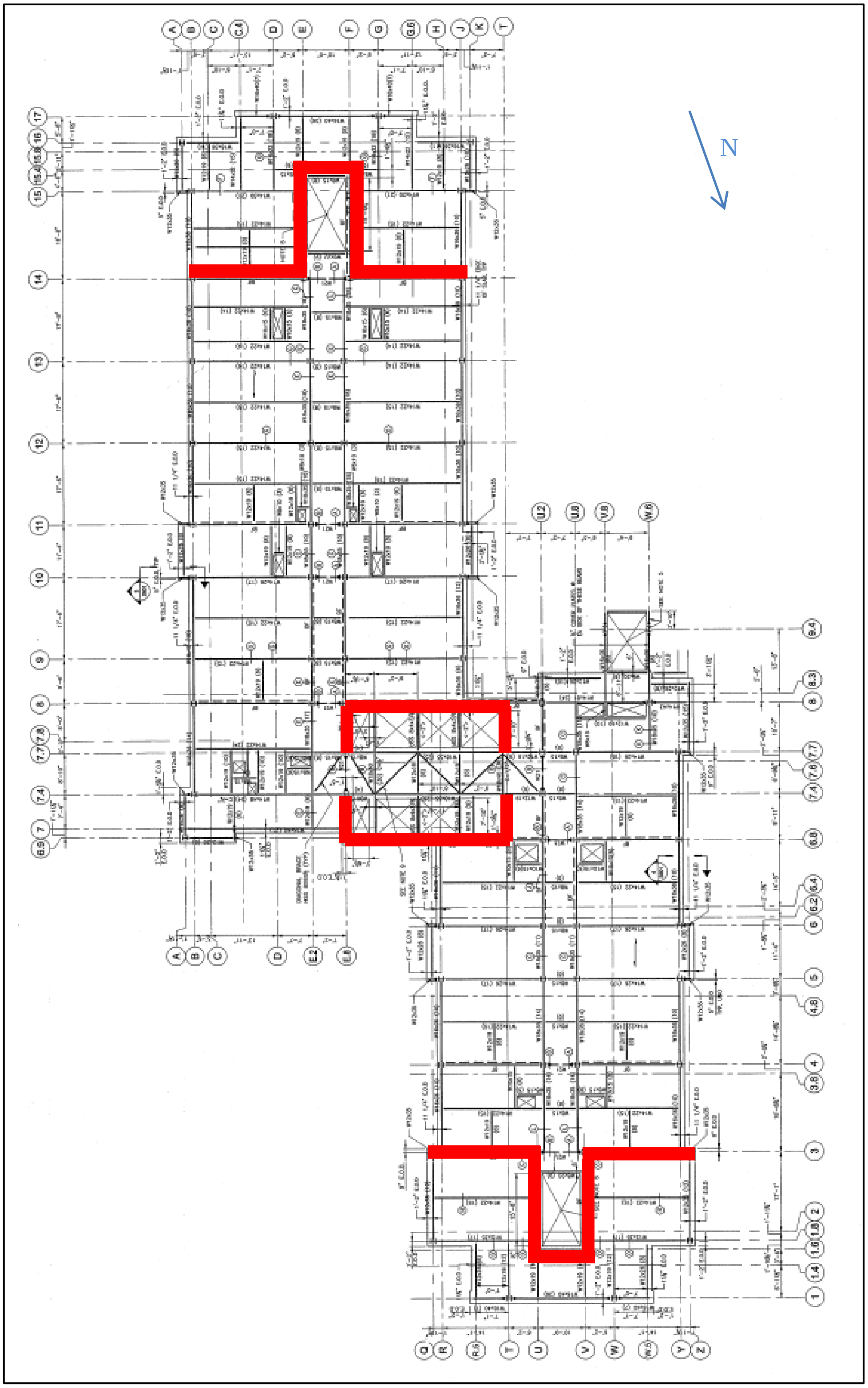


Figure 27: Initial shear wall layout for lower floors

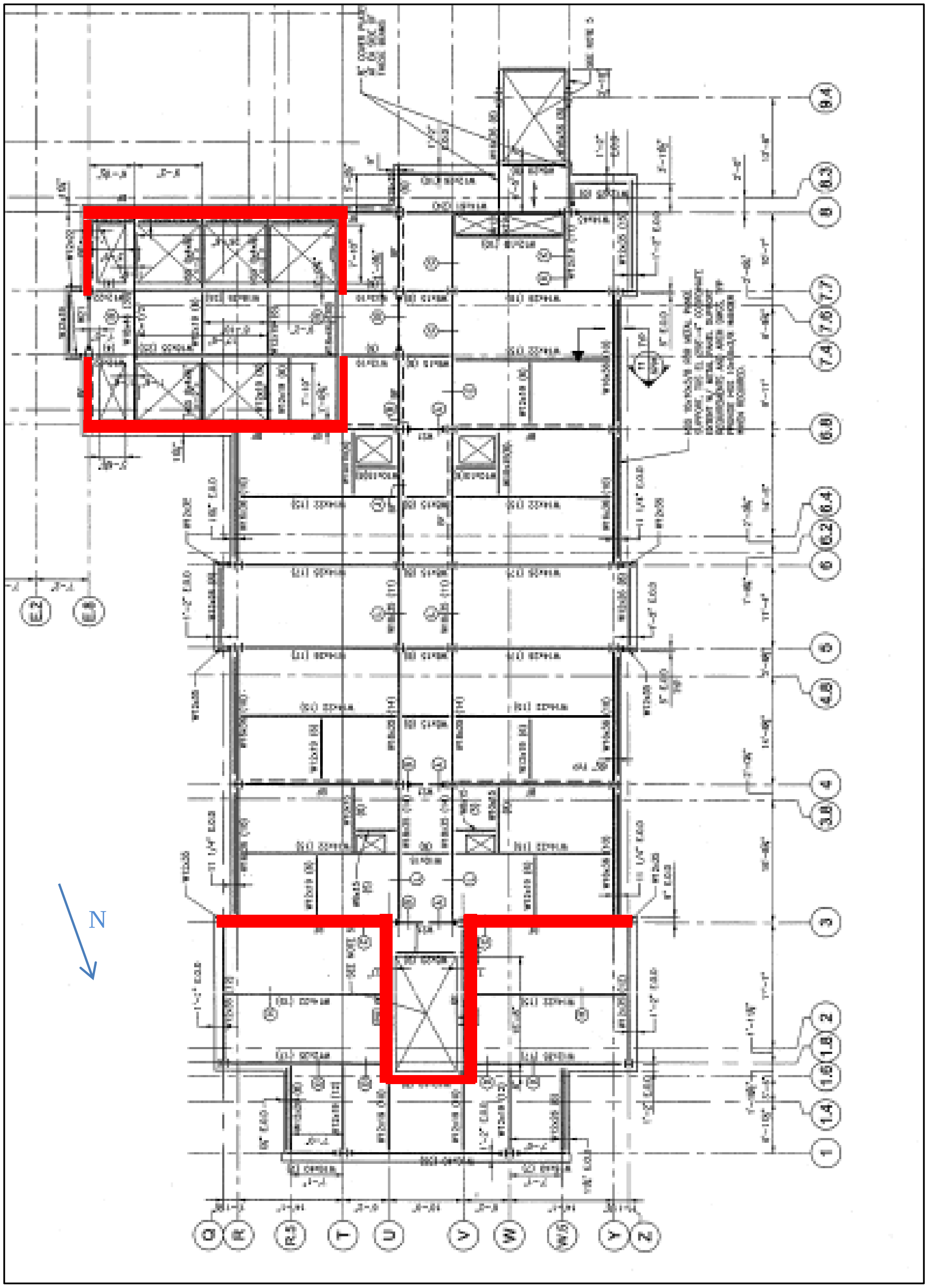


Figure 28: Initial shear wall layout for upper floors

This initial design yielded maximum deflections of 6.17 inches at the top floor and 3.81 inches at floor 19 where the floor plan changes. Because these deflections are within the code limit of H/400 this shear wall design and layout can be considered acceptable. Changes had to be made to this system because the layout would cause problems with the existing architectural floor plan. In figure 29, the circled shear walls block existing doorways between private living spaces. Also, the overall stiffness of the structure in the direction parallel to the circled walls is very high. Deflections in this direction are in the magnitude of one inch which make them far below code compliance and the structure may be considered overdesigned in that direction. The circled shear walls were removed from the system to avoid architectural complications and to create a more efficient system.

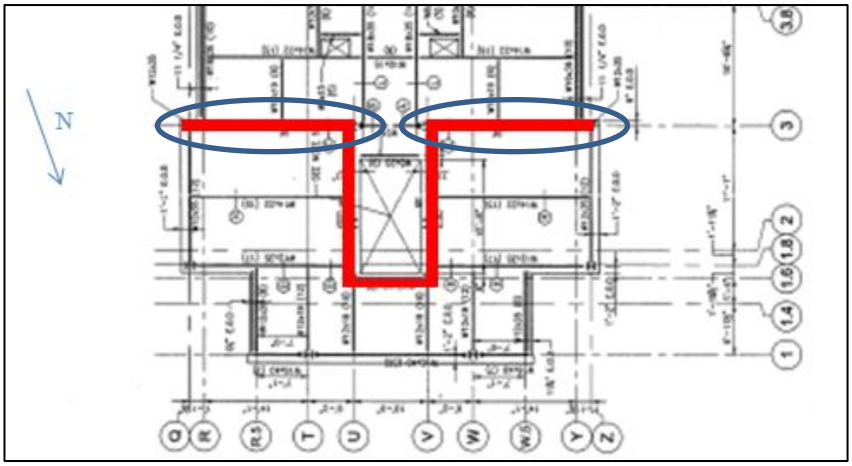


Figure 29: Architectural issue of shear walls

Once these shear walls were removed, all deflections were within allowable code limits. Design Guide 14 suggests using moment frames perpendicular to trusses to resist lateral forces. Following the suggestion of an advisor, a study was done to combine moment frames with shear walls to create a lateral system that would decrease the thickness of the shear walls and suit the geometry of Res Tower II more effectively.

Because this system uses two building materials in systems with different R values, a reasonable assumption had to be made. Detailed concrete shear walls have an R value of 2 and a steel moment frame uses an R value of 3.5. An R value of 2 was used for Res Tower II because it is the lesser of the two values. See appendix C for the new seismic design criteria and loads.

Mulitple attempts were made to find the best combination of moment frames and shear walls. The image in figure 30 provides a visual of the first attempt. In this system both the shear walls and moment frames ran from the foundation to the top of the respective towers. The thickness of the shear walls was decreased by a third to 16 inches. Deflections for this model jumped to values far greater than code limitations. This severe increase proves that the shear walls provide most of the lateral support. The moment frames may not have been effective due to large height-to-width ratio.

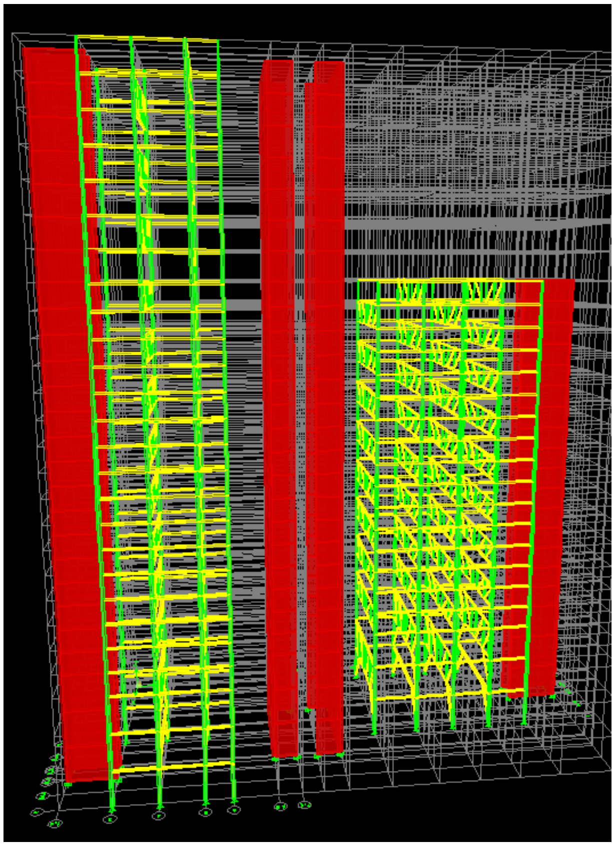


Figure 30: First combination of moment frames and shear walls

Following the study of decreasing wall thickness and adding moment frames, an investigation was done to find a way to increase the stiffness of the central core of the structure. In figure 31, the central core is examined. C-shaped shear walls, shown in red, enclose the elevator core but cannot be continuous because of a corridor between the two. To increase the stiffness of this area, 16” x 24” coupling beams (shown in blue) were added to span across the corridor from shear wall to shear wall. The dimensions of the coupling beams are not arbitrary. The thickness was chosen to match the shear walls and the 24 inch depth was selected to ensure the beams would remain in the ceiling to floor space.

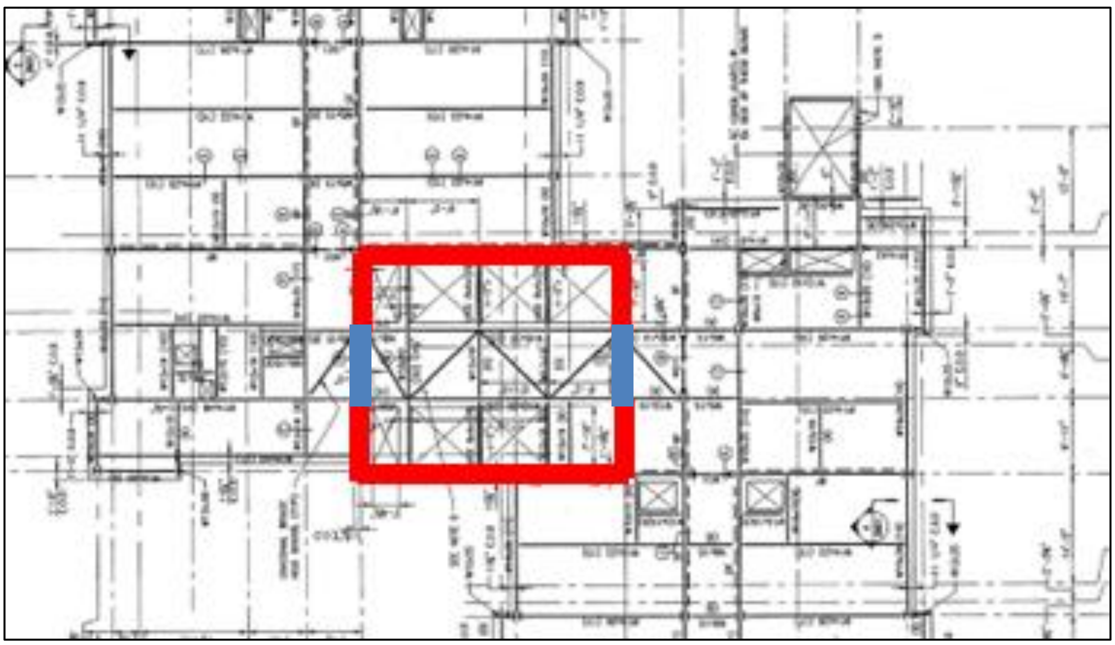


Figure 31: Central core of structure

The results of this model prove how much of a difference in strength the coupling beams created. Deflections dropped below code limits and are comparative to the results of a system using no moment frames and 24 inch shear walls. An increase in strength by creating an enclosed square makes logical sense because a square has a far greater moment of inertia (I value) than a C shape does.

Once a plausible solution was found for strengthening the shear walls, a modification to the moment frames had to be made to allow them to contribute to the overall stiffness of the building. An elementary solution of adding two bays to the width proved affective. Figure 32 shows the previous moment frames in red and the additional bays in blue. The width of each frame was increased by a substantial amount creating a much more favorable height-to-width ratio for each frame. A model with this change yielded decreased deflections but a smaller difference than anticipated.

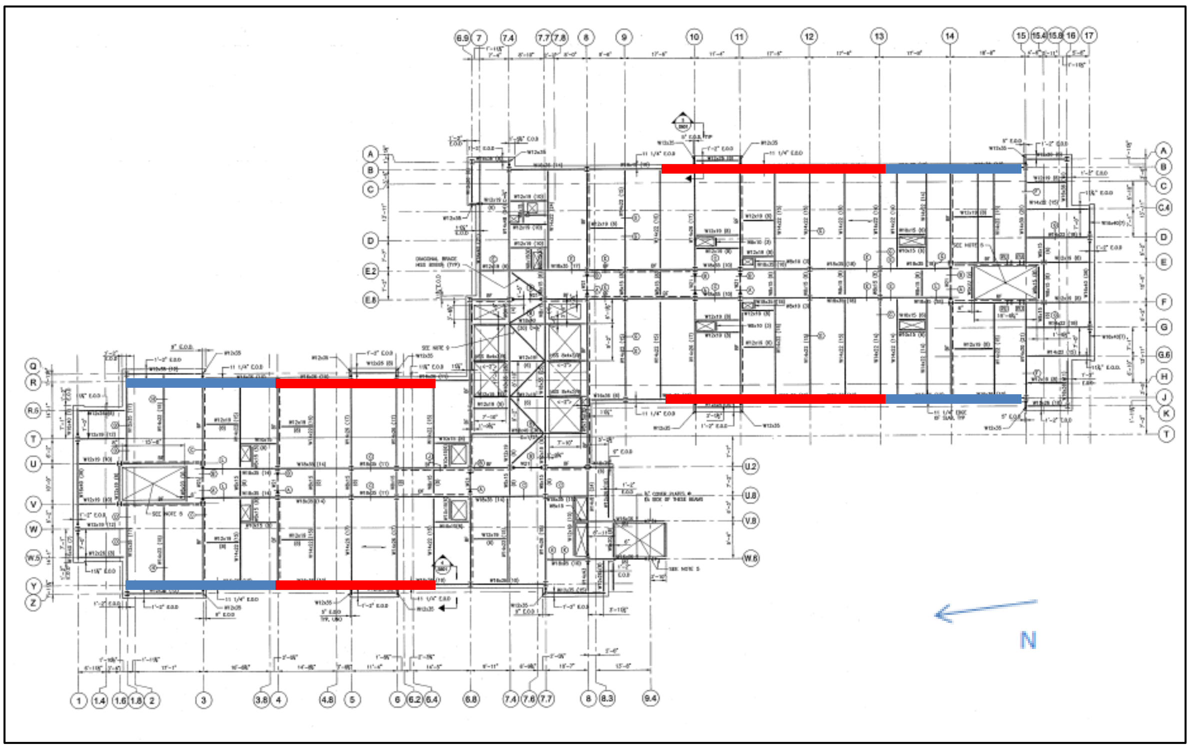


Figure 32: Expanded moment frames

A final model was created to locate an effective height to stop the moment frames. A logical place to stop the frames is the 19^{th} floor where the floor plan steps back. Results of this investigation proved that the moment frames are most effective on the lower stories. This would be caused by an accumulation of story shear at the lower floors. At the higher floors, the story shear is within the strength of the shear walls but for the lower floors, the moment frames support whatever loads are left once the shear walls reach strength capacity. Deflections were within 10% of the previous two investigations.

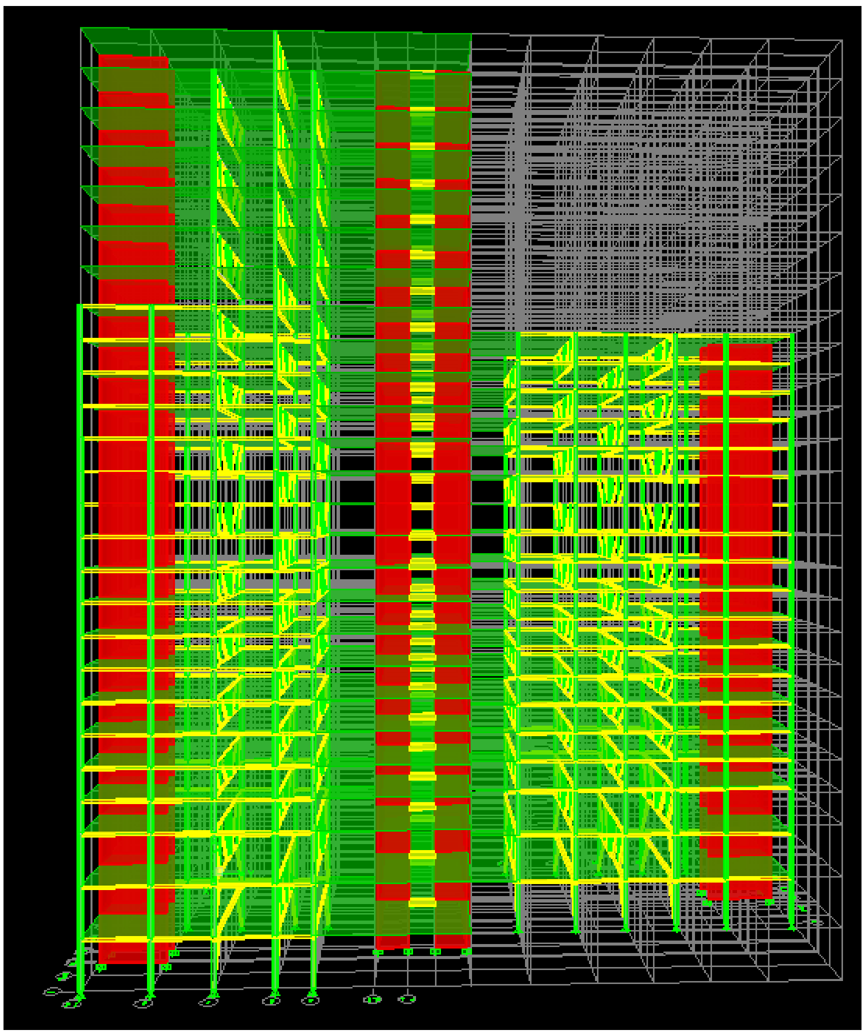


Figure 33: Model with moment frames stopping at 19th floor

Summary of lateral systems

To conclude this investigation of an efficient combination of moment frames and shear walls, table 4 provides deflections at the 19th and 26th floor for each of the lateral systems discussed.

Model	Deflections (in)	
	Floor 19	Floor 26
Code Limit	5.94	8.88
A	9.50	15.49
B	5.80	8.25
C	5.80	8.64
D	5.79	8.74

Table 4: Summary of deflections for different systems

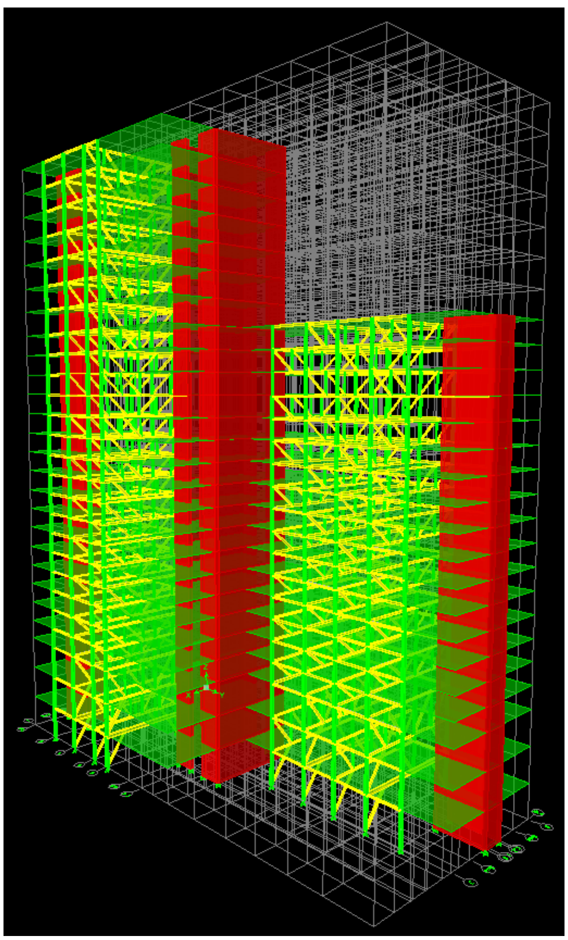


Figure 35: Model A

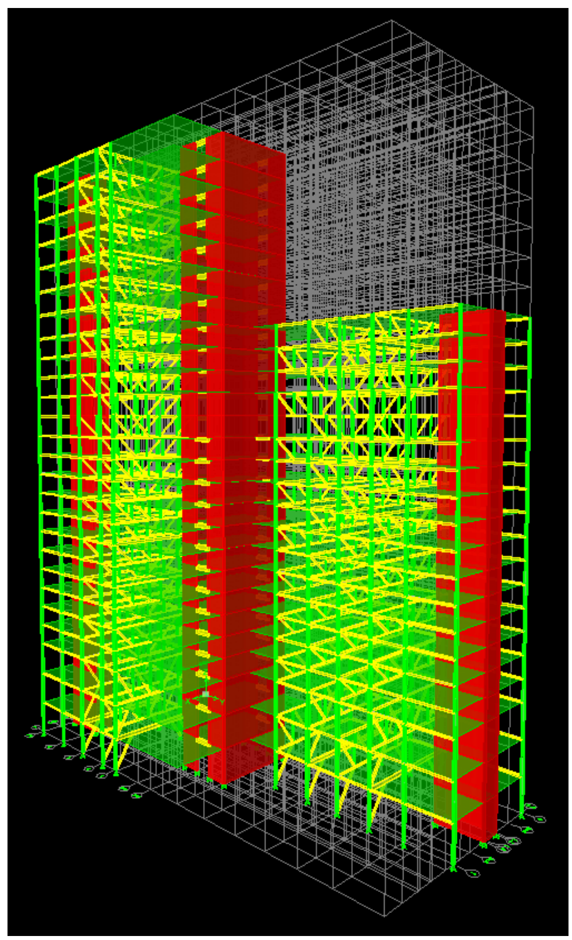


Figure 34: Model B & C

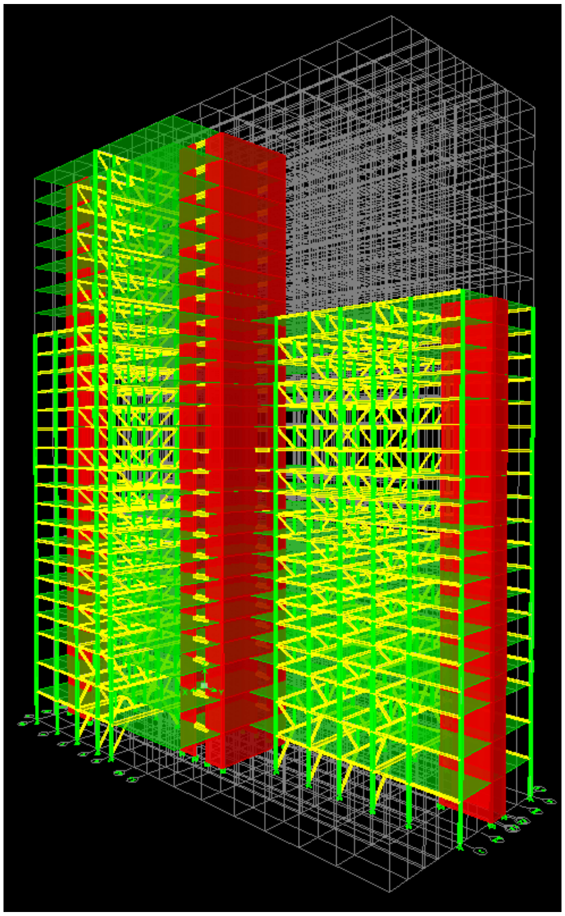


Figure 36: Model D

The models are titled A through D depending on the order they were discussed in this paper. Model A was discussed first; it has 16 inch shear walls and moment frames through the entire height of the building but the central core has no coupling beams.

Models B & C have the same appearance but have slight differences. Both have moment frames that have additional bays to increase their effectiveness and coupled shear walls but model B has 16 inch shear walls while model C has 18 inch shear walls.

Model D, is the final model that was discussed. It has coupled shear walls through the entire height of the building and extended moment frames until the 19th floor. It seems that model D is the most efficient use of materials and therefore may be the best system depending on labor and material costs.

Shear Wall and Coupling Beam Design

It is extremely important to not use a computer modeling program as a black box. There are multiple dangers of accepting computer results with no scrutiny; the worst being that a user may not fully understand the output or misinterpret the information. For this reason, hand calculations were completed for two shear walls and one coupling beam to understand what changes would need to be made to each element under different forces. With the understanding gained from completing hand calculations, spreadsheets were created so that when more of the same elements needed designed, the process would be completed efficiently.

Shear Wall Design

Calculations for the design of a shear wall were done for two sample walls. The first calculation was completed for a 19 story, 24 foot wide wall at the southern wing of the building circled in blue in figure 37. Please see appendix D for hand calculations. For this particular shear wall, only the minimum area of steel required by code was necessary to resist the lateral loads. By treating the wall as a cantilevered beam, the base shear, story shears and overturning moment were found using the simple shear and moment diagrams shown in the hand calculations.

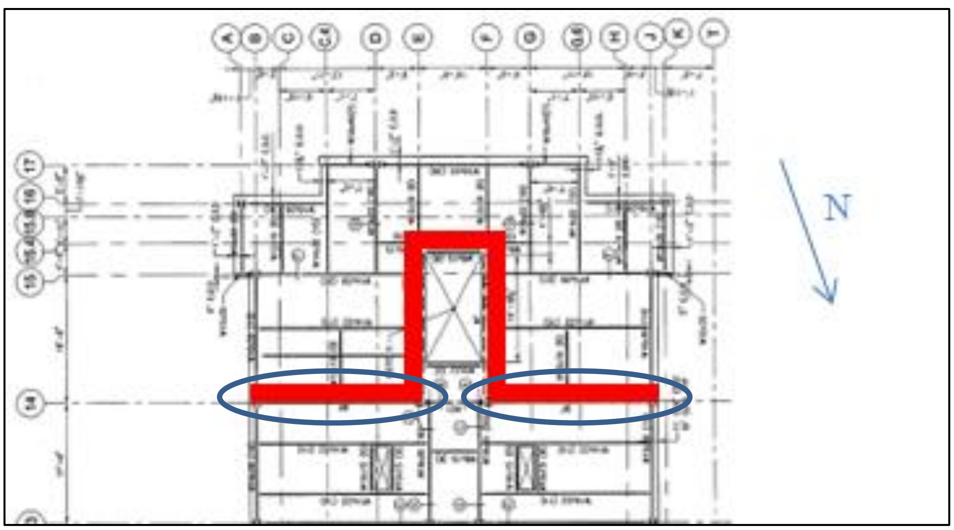


Figure 37: Southern shear walls

Another shear wall was designed for two reasons. First, the wall designed above was removed in most of the models and secondly, it was important to design a wall of the central core to understand the interaction between the walls and coupling beams. The 10 foot wide, 16 inch thick wall circled in black below was chosen as the sample wall to design. Instead of using the same technique to find lateral loads as above, vertical and horizontal reaction forces were taken from ETABS and designed for because these would be the controlling forces. To ensure the accuracy of these forces, the summation of support reactions were added to the summation of the lateral loads and checked to equal zero.

Because this wall is much thinner than the last one designed, it required more than the minimum amount of steel. It was determined that vertical #3's spaced at 14 inches on center along each side of the wall and horizontal #3's at 12 inches on center will provide the necessary shear reinforcement. Flexural reinforcement of (13) #10 bars spaced at 2 inches for the first 2 ½ feet from each side and #3's spaced at 14 inches in the space in between. The detail in figure 39 provides a visual representation of this description.

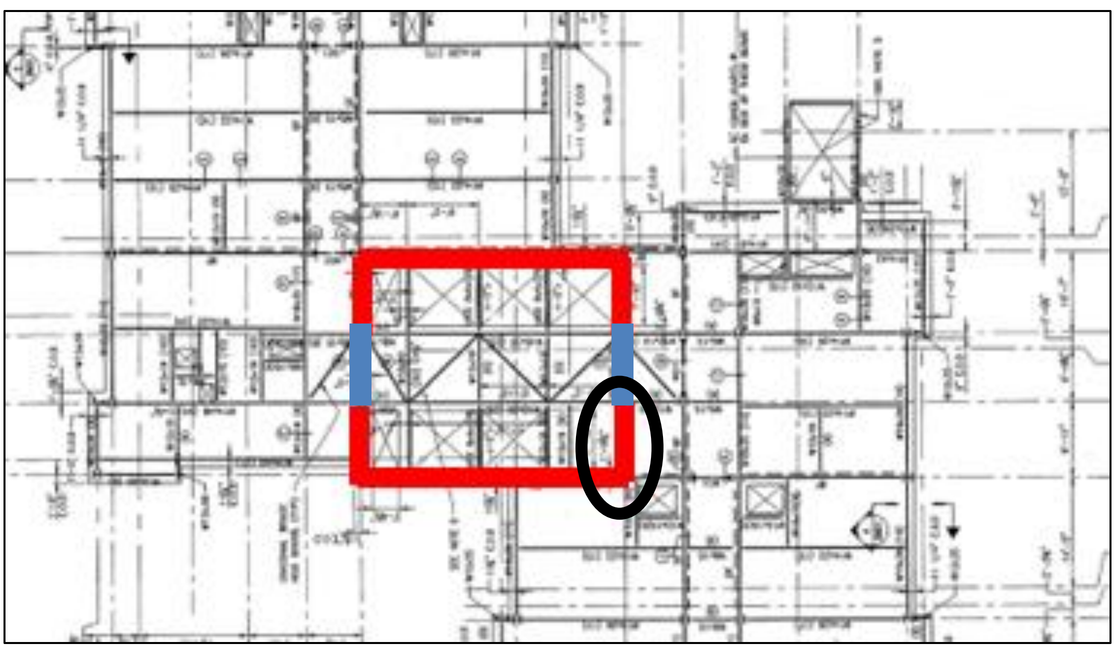


Figure 38: Central core shear wall

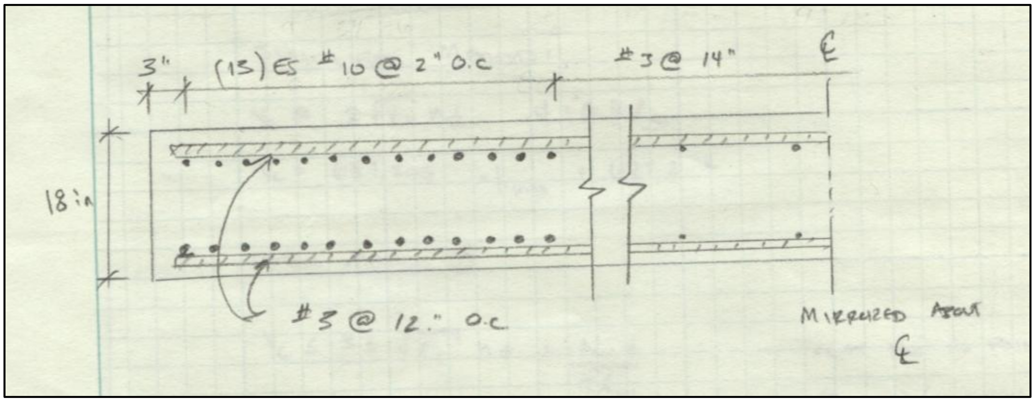


Figure 39: Detail of shear wall design

Coupling Beam Design

As a sample calculation, the coupling beam at the top floor was designed under the loads provided by ETABS which are displayed in figure 40. Appendix E provides the hand calculations required for this design.

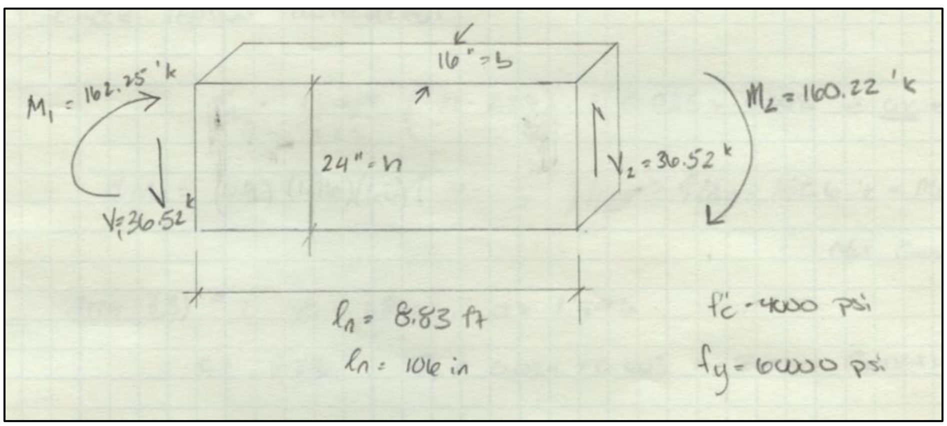


Figure 40: Forces on coupling beam

It was found that the coupling beam has sufficient strength with (4) #7 bars on both the top and bottom of the beam to resist flexure and #3 stirrups spaced at 10 inches to resist shear forces. The moments on each end of the beam are in opposite direction because lateral forces controlled the design; therefore flexural reinforcement is required in both the top and bottom of the beam. Because the maximum moment occurs at the interface between the coupling beam and the edge of the wall, a check for development length had to be done. It was determined that the reinforcement requires 27 ½ inches to develop its strength if a hook is not used. If a hook (figure 42) is used, the length of the hooked end must be greater than 22 inches. As long as either requirement is met, the contractor is free to decide which layout to use when placing the steel.

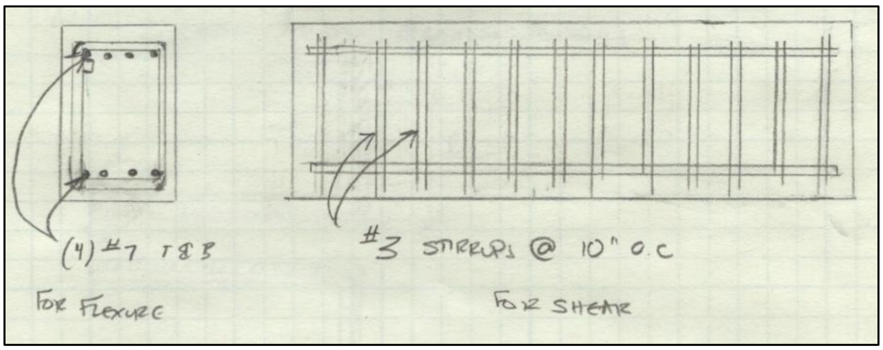


Figure 41: Detail of coupling beam reinforcement

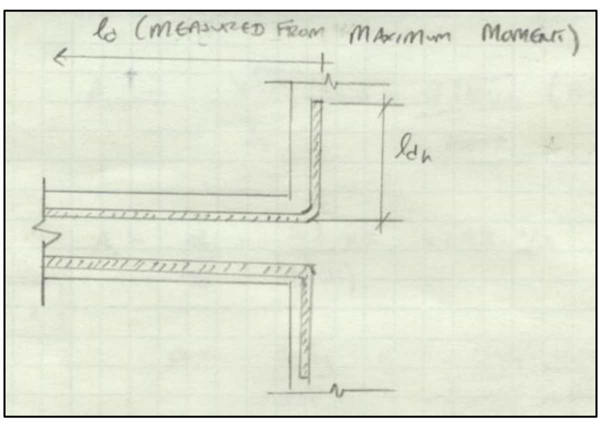


Figure 42: Detail of developed length

Impact on Foundation

The existing mat foundation was designed due to poor soil conditions and uplift forces in the columns of the lateral system. Because of the large gravity loads in each of the exterior columns, the original assumption was that there would be no uplift forces on the foundation. After analysis, the contrary was proven. The columns of the moment frame have uplift forces caused by seismic forces. Therefore, the recommendation would be to keep the mat foundation but depending on the detailing of the additional rebar cages, their size may be reduced or they may not be needed.

Because there are only columns on the exterior of the foundation, the trench in the original design is not necessary. The shear walls will require details to ensure rebar is continuous from the walls to the foundation. In this case, the recommendation would be to use hooked bars to guarantee a strong connection at the interface of these elements. If possible, it may be beneficial to pour a segment of the shear walls with the foundation to create a monolithic connection.

Breadth Topic I: Architectural Study

Using a staggered truss system has a direct impact on the interior spaces of Res Tower II. In some cases, the story deep trusses will run through open spaces and affect the architectural dynamic of the building. This study will follow the decisions to exclude some trusses and allow others to interact with the existing architectural scheme. In each case, an image of the existing appearance, a rendered image of the truss in the existing conditions and the corresponding floor plans will be provided. The following three spaces will be addressed:

- Main lobby on first floor
- 26th floor conference room
- Large study area on second floor

Main Lobby on First Floor

The main lobby of Res Tower II is a large open space with four large columns in the middle that define a seating area with no walls. Figures 44 and 46 are renderings of the existing conditions for the lobby and show the seating area that was just discussed. Figure 43 is a plan of the first floor and shows the view point of each rendering, along with the corresponding view angles. This lobby layout was requested by the client and therefore is an important space to maintain. In the original truss layout for the structural system a truss ran through the middle of the lobby. Rendered images of how this truss would fit into the lobby are shown in figures 45 and 47. Having a truss through the middle of the lobby has an obvious negative impact on the space. It is clear that if this truss were to remain in the lobby it would interfere with the flow of the space and conflict with the client's request of an open lobby area. For the reasons discussed, the truss in the space was not used in the final structural design.

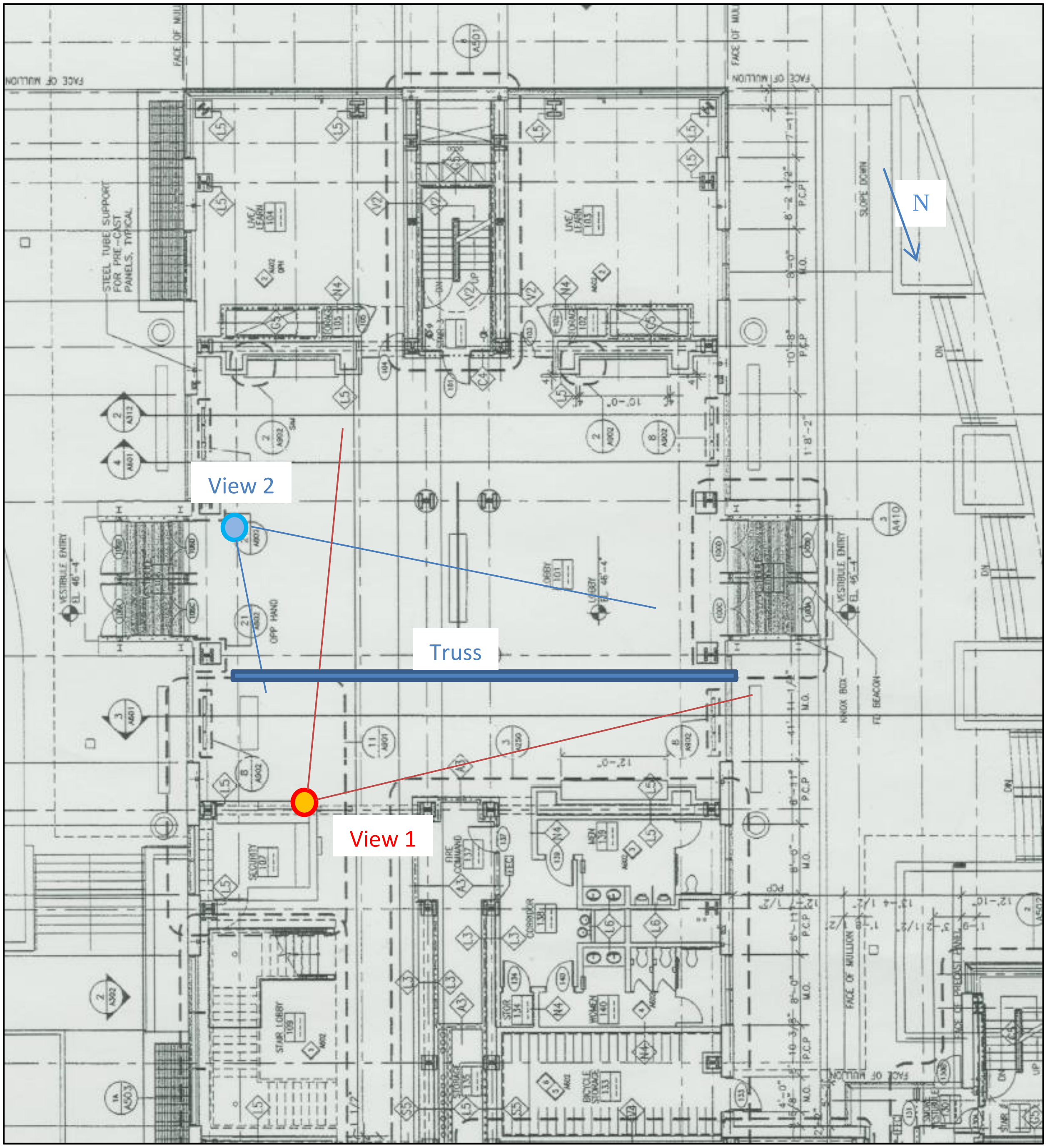


Figure 43: Floor plan of lobby with views denoted

View from reception desk (view 1)

Figure 44: View of lobby from reception desk

Figure 45: View of lobby with truss from reception desk

View from main entrance (view 2)

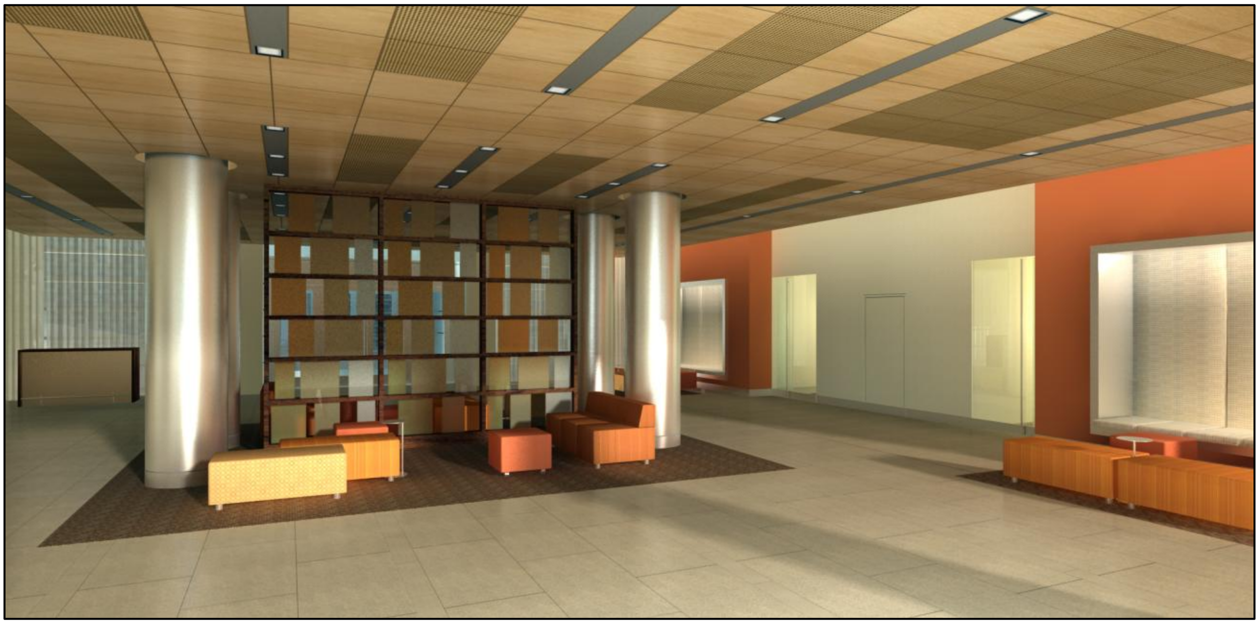


Figure 46: View of lobby from main entrance

Figure 47: View of lobby with truss from main entrance

26th Floor Conference Room

The conference room on the top floor of Res Tower II is another large open area and is completely enclosed in glass. Round columns line the exterior of the building but aside from these and the stairwell walls, there is no structure exposed at this level. In the original structural redesign, a truss ran through this space to maintain a lateral tie between exterior columns. Figure 48 presents the 26th story floor plan with an angle denoting the view point of the rendering in figure 49. An architectural study (figure 50) of this space with a truss running along the wall of the stairwell shows that a truss in this location is plausible but not desirable due to safety hazards. This truss would need to be fireproofed and most likely wrapped in some kind of padding to meet safety codes. The fireproofing could be done using an attractive paint but the safety padding may cause the truss to become bulky and block the corridor along the outside of the space. Due to the negative impact a truss in this location would have, it was decided to use the existing framing plan for this area.

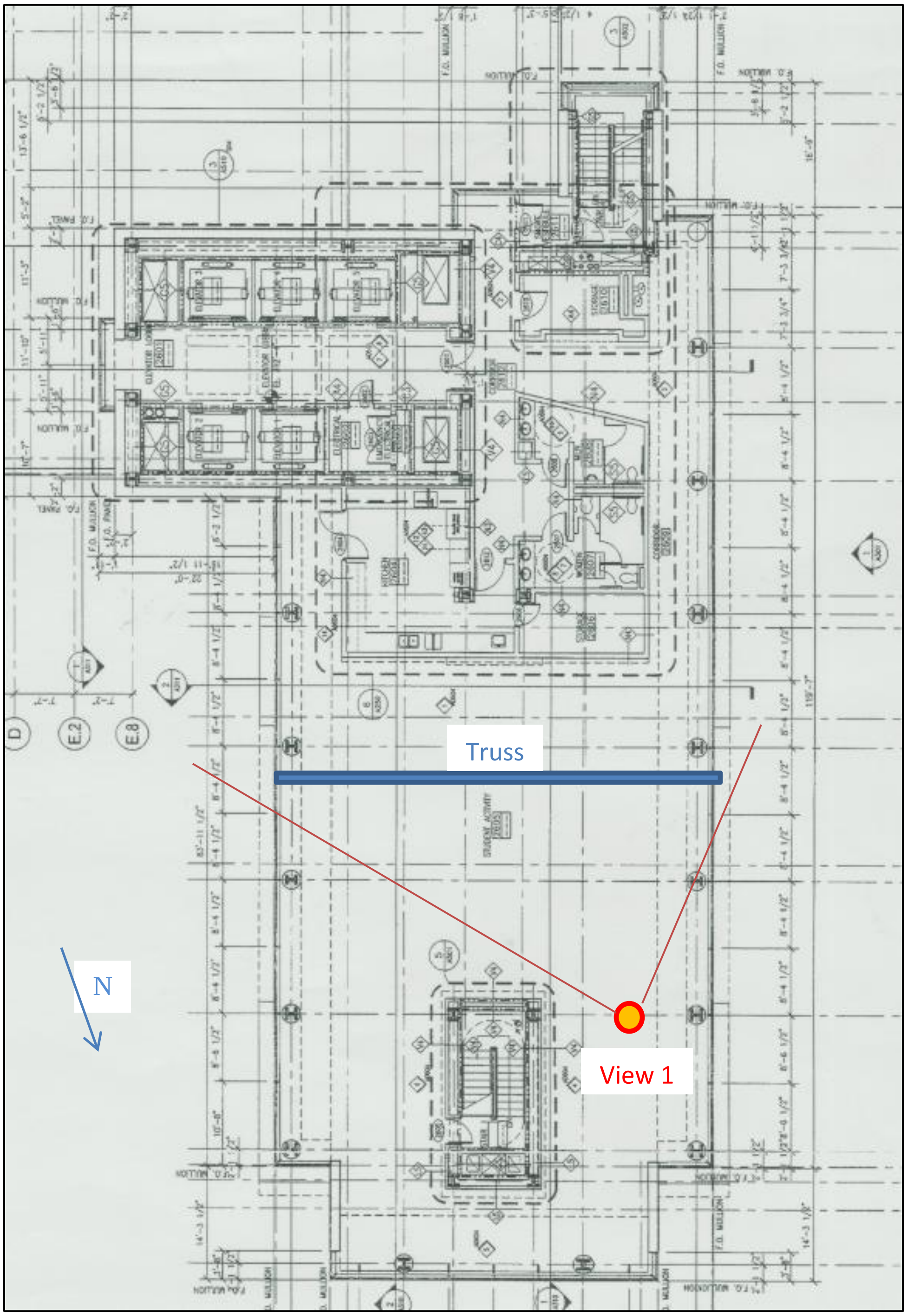


Figure 48: Plan of 26th floor with view point

Figure 49: View of 26th story area

Figure 50: View of 26th story area with truss

Large Study Area on Second Floor

The second floor of Res Tower II is the support floor for the residents. It consists of study lounges, computer labs and res-life coordination offices. In all but one case, the trusses can be hidden in existing walls. An open space on the western side of the building presents an opportunity to allow a truss to interact positively with the architectural dynamic. Figure 51 shows where this area is located on the floor plan as well as denoting the point from which the photo in figure 52 was taken. Figure 53 represents how this space would look with a truss exposed. Of course this truss could be hidden by adding a division wall from the column to the exterior wall but the original space gains an attractive feature if the truss remains exposed. Not only does the truss make the space more attractive but it could also become a utilitarian aspect of the study lounge. Plexiglass panels or dry-erase boards could be hung from the HSS members and could be used to work out problems as a group. A truss at the 2nd floor would be difficult to remove because a change in this area would greatly affect the design of the rest of the structure. A truss in this space also provides a positive, exciting dynamic that was not present in this space before. For the reasons presented above, this truss was kept in the structural system.

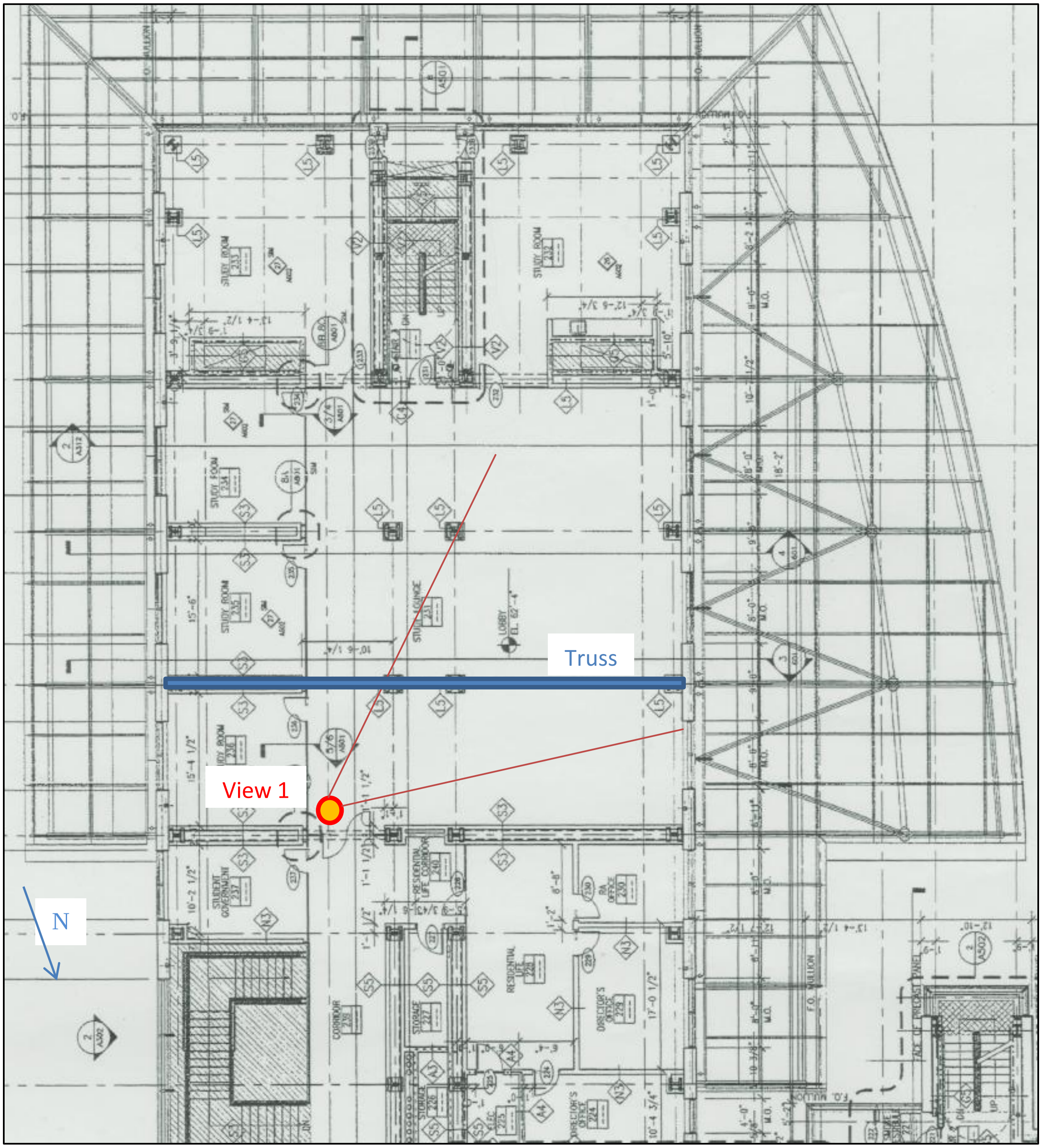


Figure 51: 2nd floor plan with view point denoted

Figure 52: View of study lounge

Figure 53: View of study lounge with truss

Breadth Topic II: Site Logistics and Schedule

The second breadth study focuses on the creation of an efficient site logistics plan based on the new structural steel design. Additionally, a new estimated schedule will be developed to plan how the structural materials will be delivered and stored on site when necessary.

To start, a schematic plan was created of the site before construction started (shown in figure 54) to evaluate what impacts on the surrounding environment needed to be avoided. Res Tower II is shown in blue with access roads outlined in red. See the key on the image for further explanation.

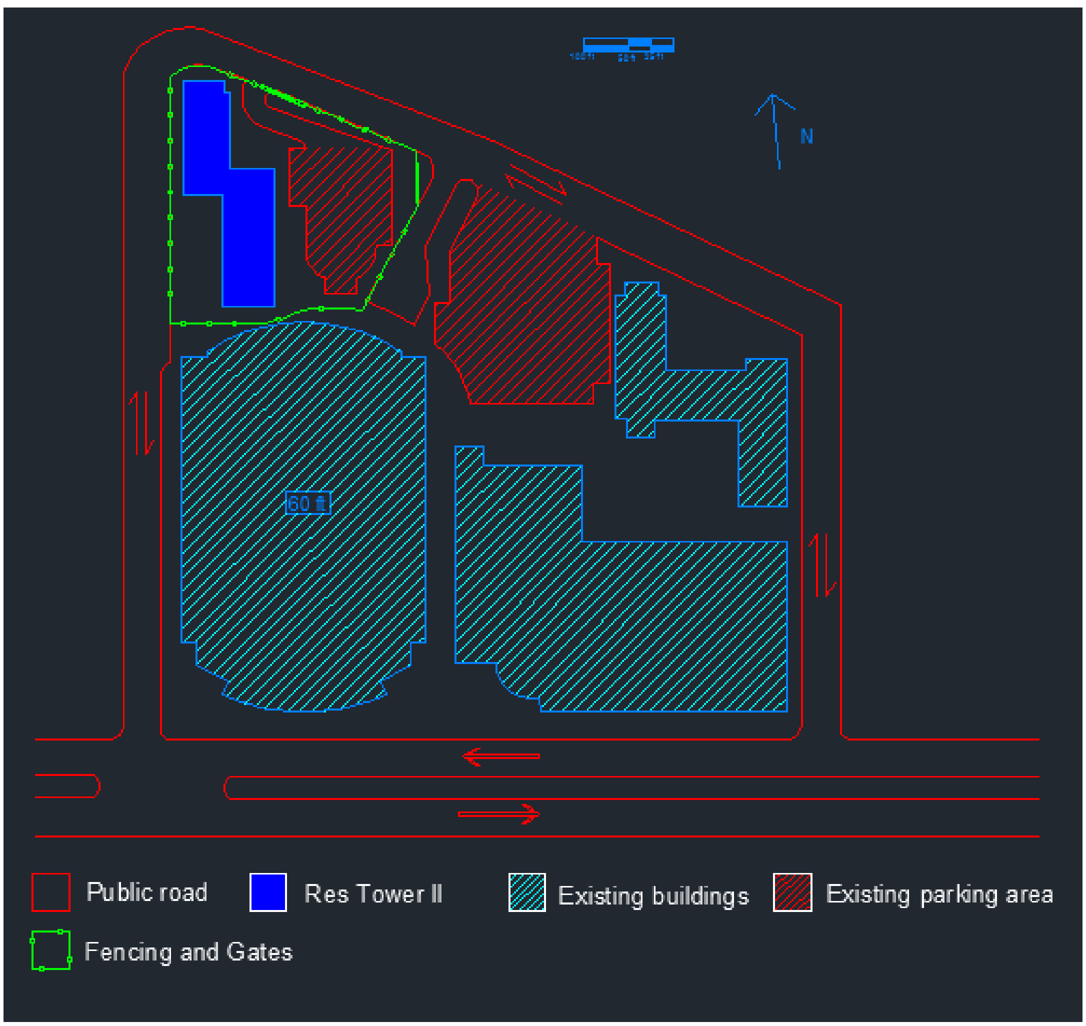


Figure 54: Preconstruction site plan

To allow for full operation of the surrounding buildings and athletic facilities, the construction site was confined to the area enclosed by the fencing in figure 54. The access road along the south east side of the fence provides service vehicles and buses access to Harry Agganis Arena. The two other roads defining the shape of the site are public roads which are only used by BU employees who park in the lots shown.

Once the boundaries of the site were established, site storage and delivery routes had to be determined for the structural materials. Due to the restricted site and the size of the prefabricated trusses, the decision was made to have them delivered and taken directly from the truck to their final position instead of storing them on site. The public roads surrounding the site are large enough for an 18 wheel truck and so are the entrances onto them from the main road. This quality of the site allows delivery trucks to enter and exit the site easily without blocking off any roads for an extended period of time. The delivery path is shown as yellow arrows in the figure on the following page. The delivery location and storage areas, denoted by a yellow hatch, were carefully planned based on distance from the building and distance from the delivery route. A temporary delivery road was added from the existing public road on the north side of the site to the eastern gate so that no traffic will be blocked while the trusses are being delivered.

Crane location was dictated by the length of the building and the delivery locations. The most efficient location was determined to be at the north corner where the two towers meet because it allowed the crane to reach all sides of the building and be closest to the tallest tower. It was important to keep the crane closest to the tallest part of the building to allow for a lateral tie back all the way up the crane. Of course, this location could not have been used if there wasn't a crane that could carry the weight of the trusses at the farthest required distance. Each truss weighs about 3000 to 3500 lbs and 160 ft is the greatest horizontal distance a truss is located from the center of the crane. Terex Cranes makes a flat top crane that has a max jib length of 180 ft and a capacity at max length of 1.93 tons or 3860 lbs (model number CTT 121/A-5 TS16). This crane location ensures that it will not swing over any adjacent buildings, especially Harry Agganis Arena.

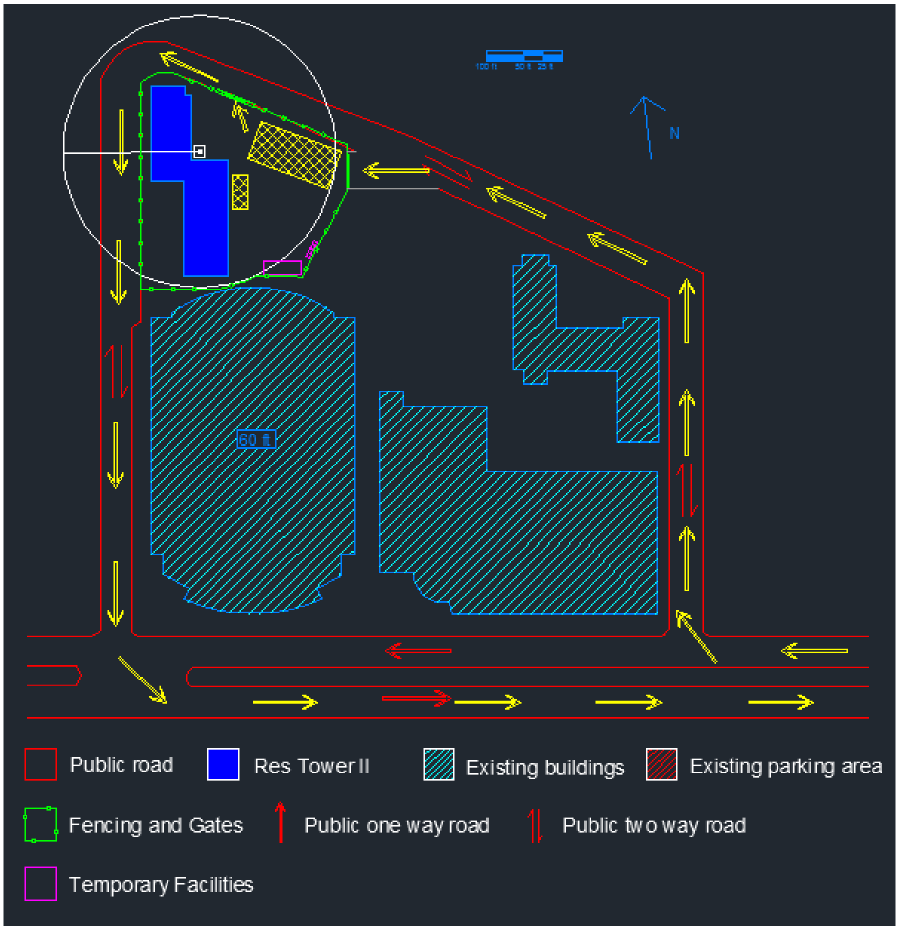


Figure 55: Site logistics plan

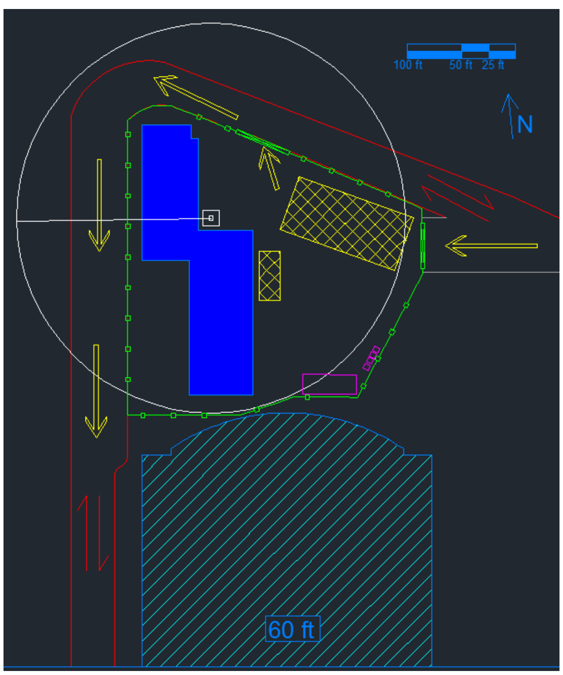


Figure 56: Site logistics plan (scaled up)

Temporary facilities such as office trailers and porta-johns were located on the edge of the site to avoid taking up valuable space for material and personnel circulation. If more space is necessary, space can be designated on the ground floor that can be temporarily occupied by contractors.

Figure 56 provides a closer look at the construction site using the same key as figure 55.

The second part of this study was to create a schedule for the new structural design which is presented in appendix F. An arbitrary date of April 1, 2011 was chosen but the start date does not affect the schedule itself. To create this schedule, the construction process had to be broken down into five main, chronological steps:

1. Shear walls
2. Truss and Columns
3. Joists
4. Decking
5. Slab

As can be seen from the schedule, the construction of the shear walls will have the longest duration due to the curing time of concrete. Because all other structural elements frame into the shear walls, they need to be completed to a certain floor before other disciplines can start construction. The decision was made to break the building up into groups of 4 levels. The shear walls are constructed for four levels and allowed to cure then the steel erection (column and trusses) complete the same four floors. This sequence continues up the building until complete. Joist, decking and slab construction are done relatively quickly but must wait until each contractor can work continuously without being slowed by the shear wall construction and steel framing erection. Limiting the contractors time on site will decrease cost of the project and prevent site congestion.

This division of disciplines also allows efficient use of the crane. If the crane is not being used for multiple disciplines, a more precise lifting schedule could be created and confusion could be avoided by limiting the crane operator to one task at a time.

Table 5 provides the daily output values from RSMEANS to create the schedule. These values are per crew. Two decking crews were used to increase productivity and to allow the schedule to continue efficiently.

Element	Crew	Daily Output	(units)
Slab	C-20	160	C.Y
Decking	E-4	2850	sq. ft
Joists	E-7	1500	Linear ft

Table 5: Daily output values

MAE Course Related Study: Connections Design

Utilizing the knowledge gained from AE 534: Steel Connections and by following the discussion presented in *Design Guide 14: Chapter 4*, typical connections were designed for the trusses. The following two connection types were chosen to design because they are highly repetitive in the structure:

- Diagonal and vertical web members to the bottom chord member
- Truss to the column web

All connections are done using gusset plates to allow for multiple members to frame into the same location with no eccentricity. *Design Guide 14* suggests using welded-welded connections for members of the truss and bolted-bolted connections for truss to column interfaces. A welded-welded connection between truss members is done because these trusses will be shop fabricated and transported to the site in one piece. Steel erection practices dictate bolting the truss to the column. The truss will be connected to the web of the pre-erected columns and therefore using a bolted-bolted connection allows for larger construction tolerances than a bolted-welded connection would. Also, it would be difficult to align the truss if the angles of the connection were already a part of the truss. Please see Appendix G for the hand calculations of this design.

Web Members to Bottom Chord Member:

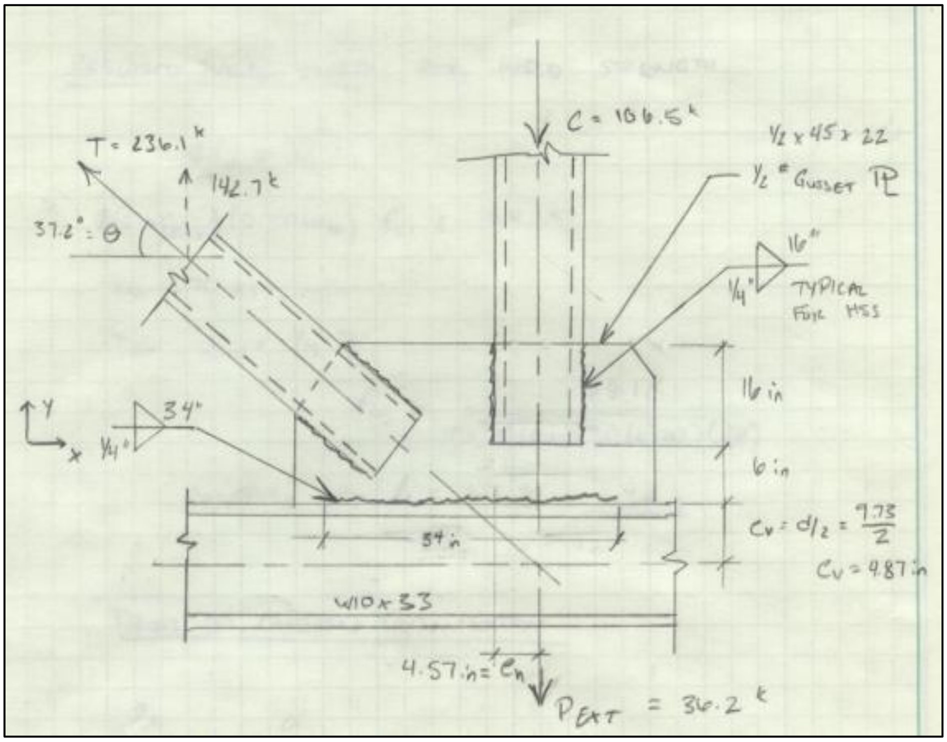


Figure 57: Detail of welded connection

Figure 57 shows the geometries and loads used in this design. As stated above, *Design Guide 14* recommends using welded-welded connections for truss members because the trusses will be assembled in a controlled environment. This controlled environment allows for more control of construction tolerances and therefore welds can be used to an advantage. All aspects of this connection were designed including gusset plate size, weld sizes and weld lengths.

The design guide gives four limit states to check for members in tension and a fifth for compression members. They are as follows:

1. Shear Lag Fracture Strength in the HSS
2. Shear Strength of the HSS at Welds
3. Strength of the Weld Connecting the Gusset Plate to the HSS
4. Shear Strength of the Gusset Plate
5. Strength Based on Buckling of the Gusset Plate

<u>Truss to Column Web:</u>

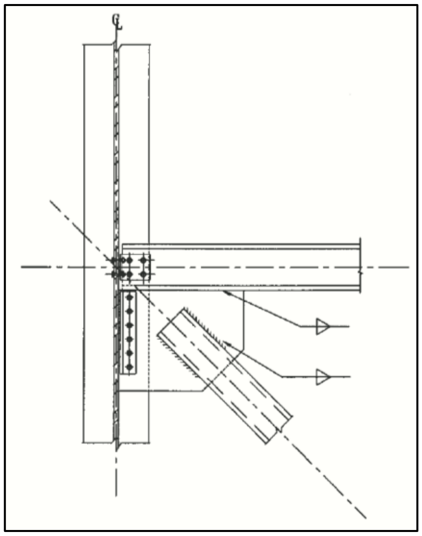


Figure 58: AISC connection detail

Figure 58 is a detail taken directly from *Design Guide 14* of the "truss to column connection." As stated above, the design guide recommends using a bolted-bolted connection for easier construction. The weld connecting the truss members to the gusset plate was not designed in this section; it was assumed that by designing the weld in the previous section the forces will be transferred completely to the gusset plate. The design guide discusses a technique to design this connection that is slightly ambiguous and therefore this design was done completely using lecture notes and knowledge from AE 534: Steel Connections.

Figure 59 shows the members, loads and geometries used in the design of this connection. Using the geometry of the connection, it was determined how the tension force in the bracing member is distributed to the column in terms of shear and axial forces at the beam and gusset locations. It was determined that the beam to column connection needs to transfer 49.24 ^{k} in shear and 64.1 ^{k} in compression and the gusset to column connection needs to transfer 93.53 ^{k} in shear and 64.2 ^{k} in tension. Only the number of bolts and angle sizes were calculated for this section of the connection design.

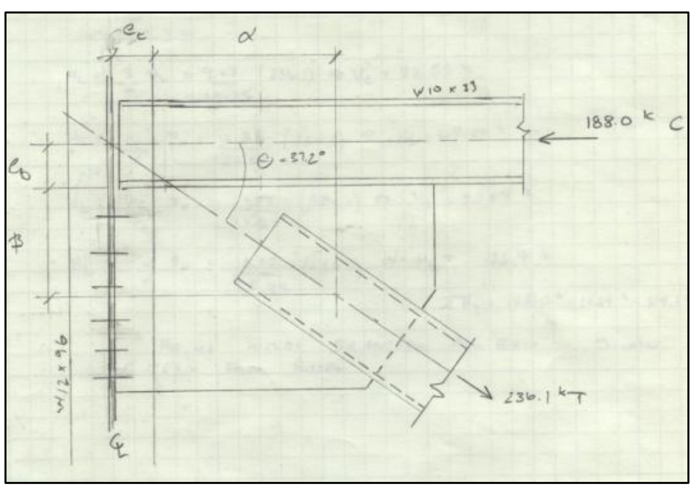


Figure 59: Bolted connection detail

Gusset-to-Column Connection:

Table 6 shows which limit states were checked for this connection as well as the corresponding max shear (ϕV_n) and tension (ϕT_n) values. It is clear to see that all values are greater than the forces of the connection (V_u) and therefore the connection has adequate strength.

Gusset-to-Column Connection $V_u = 93.53^k$ $T_u = 64.21^k$		
Bolt Limit States	ϕV_n (k)	ϕT_n (k)
Shear	96.76	
Tension		90.09
Bearing & Tearout	232	
Angle Limit States	ϕV_n (k)	
Shear Yield	177.7	
Shear Rupture	355.5	
Block Shear	110.7	

Table 6: Connection design strength design strength

Gusset-to-Column Connection:

Table 7 shows the limit states that were checked for this connection. For the "Bolt Limit States" section, it is important that the force on each indivudual bolt (R_u) is less than the capacity of the bolt (ϕV_n). The "Bearing & Tearout" value for ϕV_n must be greater than both the shear and compression forces because both of these can cause bearing failure. In the "Angle Limit States" section, the capacity (ϕV_n) is greater than required strength (V_u) and therefore the connection is adequate.

Beam-to-Column Connection $V_u = 49.24^k$ $C_u = 64.1^k$		
Bolt Limit States	ϕV_n (k)	R_u (k)
Shear	31.8	30.62
Bearing & Tearout	100.05	
Angle Limit States	ϕV_n (k)	
Shear Yield	108	
Shear Rupture	74	
Block Shear	107.8	

Table 7: Gusset-to-Column design strength

Conclusion

A staggered truss system was successfully designed and efficiently implemented into Res Tower II. As a result of investigating the best use of a staggered truss system to resist gravity and lateral loads, it was determined that for Res Tower II, it is not practical to design the truss system to resist lateral loads. The staggered truss system efficiently supports the gravity loads but to resist lateral loads, member sizes would need to be unreasonably large.

To allow the trusses to only support gravity loads, a lateral system was designed using concrete shear walls and steel moment frames. To increase the moment of inertia and stiffness of the central core shear walls, coupling beams were added to connect the two C shaped walls surrounding the elevators. The steel moment frames were stopped at the 19th floor to increase the efficiency of the system. Max deflections of 5.79 inches at the 19th floor and 8.74 inches at the 26th floor are both within code limitations. This is an efficient design and the designer feels confident that this system could be used in the construction of a Boston high rise.

A goal for this design was to avoid negatively affecting the interior appearance of Res Tower II. The trusses are only exposed in a study lounge on the second floor. Exposing the structure in this area positively changes the space by adding a landmark feature to an ordinary space.

A logical site logistics plan was created that provides a delivery route and site layout that avoids blocking any traffic and does not influence the infrastructure of the surrounding buildings. The construction schedule provides a rational process that allows for an uncongested site and efficient construction duration.

Appendix

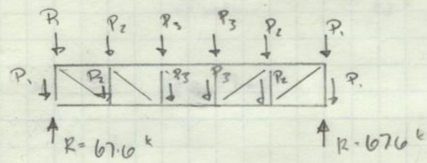
Appendix A: Method of Joints and Truss member design

<u>Method of Joints</u>

TYLER MEEK | THESIS CALCS | DEAD LOAD ON TRUSS & LIVE LOAD PARAMETERS | 1/2

UNFACTORED ⇒ ω_{UN} = 76 psf (17.5 ft)

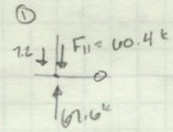
ω = 1.33 klf



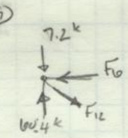
P_1 = 7.20 k
P_2 = 14.41 k
P_3 = 12.19 k

JOINT & MEMBER LOCATIONS ON PRIOR CALCULATIONS

Joint ①



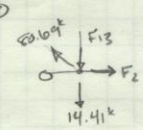
Joint ⑦



$F_{12} = \dfrac{(60.4 - 7.2)(14.4087)}{9.5} = 80.69^k$

$F_6 = \dfrac{(80.69)(10.833)}{(14.4087)} = 60.67^k$

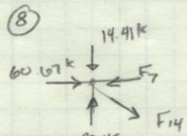
Joint ②



$F_2 = \dfrac{(80.69)(10.833)}{(14.4087)} = 60.67^k$

$F_{13} = \dfrac{(80.69^k)(9.5)}{(14.4087)} - 14.41 = 39.45^k$

Joint ⑧

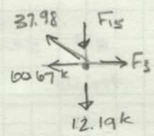


$F_{14} = \dfrac{(39.45 - 14.41)(14.4087)}{9.5} = 37.98^k$

$F_7 = \dfrac{(37.98)(10.833)}{(14.4087)} + 60.67 = 89.22^k$

TYLER MEEK | THESIS CALCS | DEAD LOAD ON TRUSS & LIVE LOAD PARAMETERS | 2/2

JOINT ③



$$F_{15} = \frac{(37.98)(9.5)}{(14.4087)} - 16.19 = 12.85^k$$

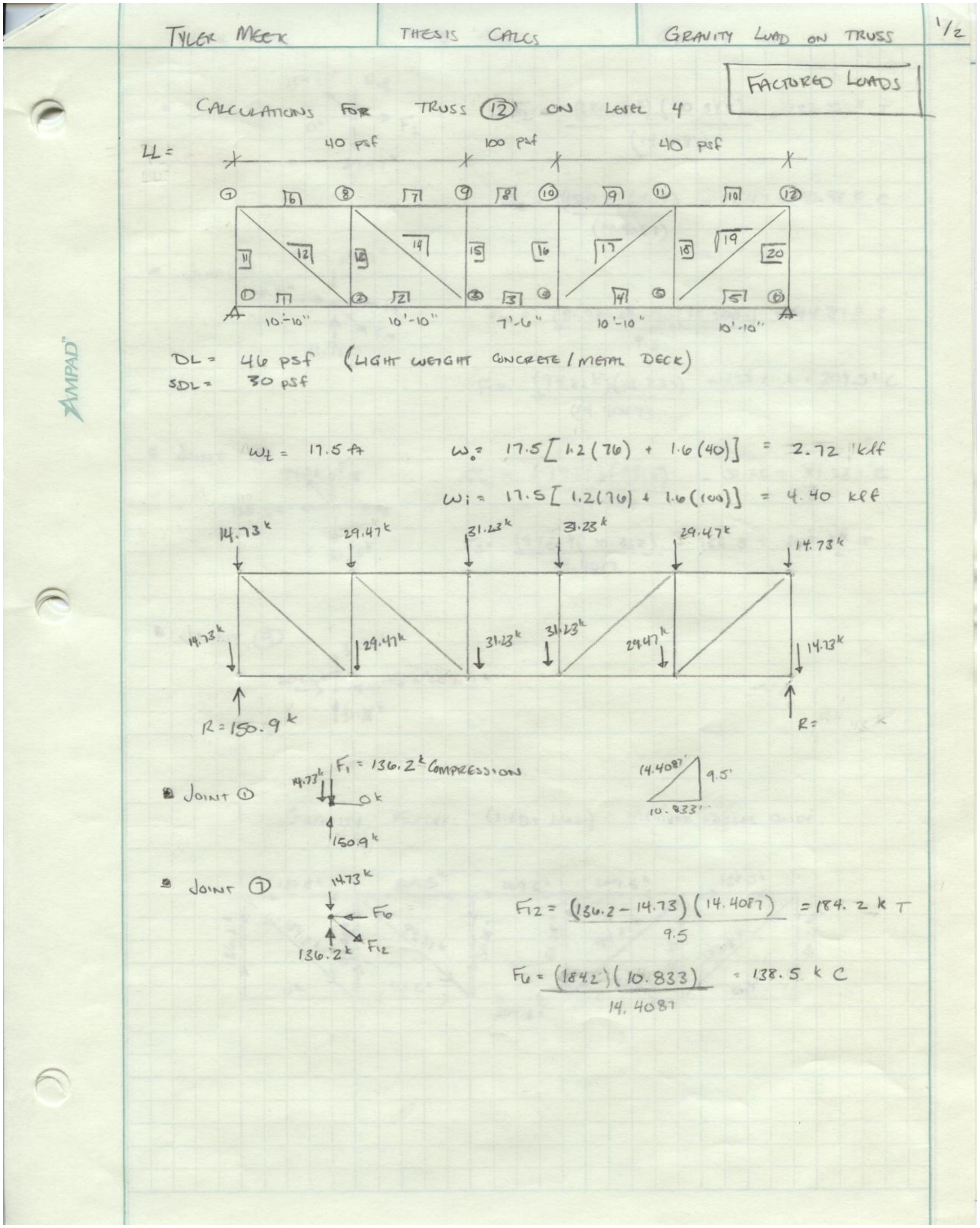
$$F_3 = \frac{(37.98^k)(10.833)}{(14.4087)} + 60.67 = 89.22^k$$

JOINT ⑨ $F_8 = 89.22^k$

LIVE LOAD CALCULATIONS DONE USING EXCEL SPREADSHEET

$\omega_{UN_i} = 100$ psf

$\omega_{UN_o} = 40$ psf



Tyler Meek | Thesis Calcs | Gravity Load on Truss | 1/2

FACTORED LOADS

Calculations for Truss 12 on Level 4

LL = 40 psf | 100 psf | 40 psf

10'-10" | 10'-10" | 7'-6" | 10'-10" | 10'-10"

DL = 46 psf (Light weight concrete / metal deck)

SDL = 30 psf

$w_t = 17.5$ ft

$w_o = 17.5[1.2(76) + 1.6(40)] = 2.72$ klf

$w_i = 17.5[1.2(76) + 1.6(100)] = 4.40$ klf

14.73^k | 29.47^k | 31.23^k | 31.23^k | 29.47^k | 14.73^k

$R = 150.9^k$ | R =

Joint 1

14.73^k, $F_1 = 136.2^k$ Compression, 0^k, 150.9^k

14.4087' | 9.5' | 10.833'

Joint 7

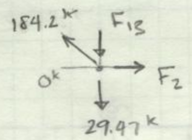
14.73^k, F_6, F_{12}, 136.2^k

$$F_{12} = \frac{(136.2 - 14.73)(14.4087)}{9.5} = 184.2 \text{ k T}$$

$$F_6 = \frac{(184.2)(10.833)}{14.4087} = 138.5 \text{ k C}$$

Tyler Meek | Thesis Calcs | Gravity Load on Truss | 2/2

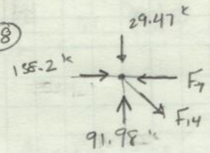
- Joint ②



$$F_2 = \frac{(184.2^k)(10.833)}{(14.4087)} = 138.5^k \ T$$

$$F_{13} = \frac{(184.2)(9.5)}{(14.4087)} - 29.47 = 91.98^k \ C$$

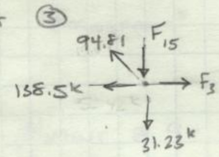
- Joint ⑧



$$F_{14} = \frac{(91.98-29.47)(14.4087)}{9.5} = 94.81^k \ T$$

$$F_7 = \frac{(94.81^k)(10.833)}{(14.4087)} + 138.2^k = 209.5^k \ C$$

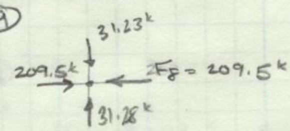
- Joint ③



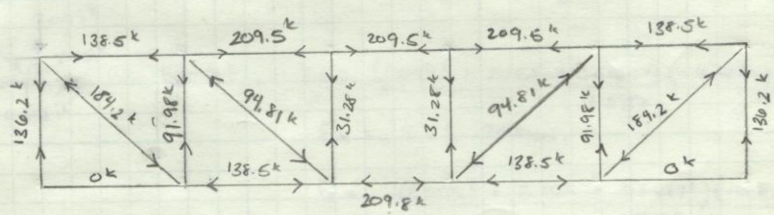
$$F_{15} = \frac{(94.81)(9.5)}{14.4087} - 31.23 = 31.28^k \ C$$

$$F_3 = \frac{(94.81)(10.833)}{14.4087} + 138.5 = 209.8^k \ T$$

- Joint ⑨

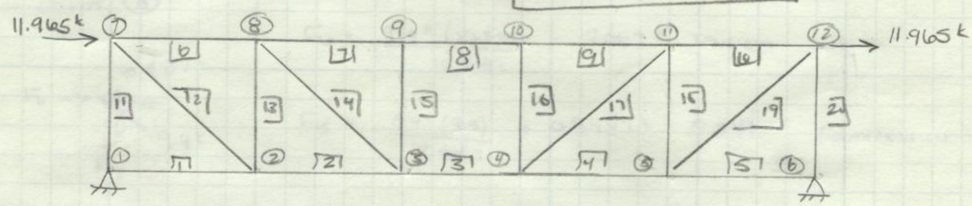


Gravity Forces: (1.2D + 1.6L) - Member Forces Only



TYLER MEEK | THESIS CALCS | LATERAL LOADS ON TRUSS | 1/2

UNFACTORED LOADS



$$A_y = B_y = \frac{(23.93^k)(9.5\,ft)}{50.5} = 4.5^k$$ (P. 17 DESIGN GUIDE)

$$V_{@\,MIDSPAN} = \frac{\frac{1}{2}(11.965^k)(9.5)}{25.25} = 2.25^k$$

$$\text{MOMENT @ 9} = (2.25^k)\left(\frac{7.15\,ft}{2}\right) = 8.0625\ ft\text{-}k$$

$$V_{@\ \text{FIRST ADJACENT PANEL}} = \frac{(8.0625\ ft\text{-}k + 0)}{9.5} = 0.8487^k$$

JOINT 9

$F_{15} = 3.099^k$ COMPRESSION

$F_8 = F_7 = 0$

JOINT 3

$F_3 = 0^k$

$$F_{14} = \frac{(3.099^k + 2.25^k + 0.8487^k)(14.4087)}{9.5\,ft}$$

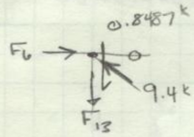
$F_{14} = 9.4^k$ TENSION

$$F_2 = \frac{(3.099^k + 2.25^k + 0.8487^k)(10.833\,ft)}{9.5\,ft}$$

$F_2 = 7.067$ COMPRESSION

TYLER MEEK | THESIS CALCS | LATERAL LOADS ON TRUSS | 2/2

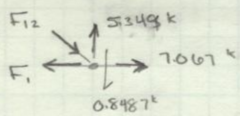
JOINT 8



$$F_6 = \frac{9.4^k(10.833)}{14.4087} = 7.067^k \text{ TENSION}$$

$$F_{15} = \frac{9.4^k(9.5)}{14.4087} - 0.8487^k = 5.349^k \text{ COMPRESSION}$$

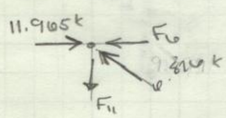
JOINT 2



$$F_{12} = \frac{(5.349 - 0.8487)(14.4087)}{9.5} = 6.826^k \text{ TENSION}$$

$$F_1 = \frac{(6.826^k)(10.833)}{14.4087} + 7.067 = 12.20^k \text{ COMPRESSION}$$

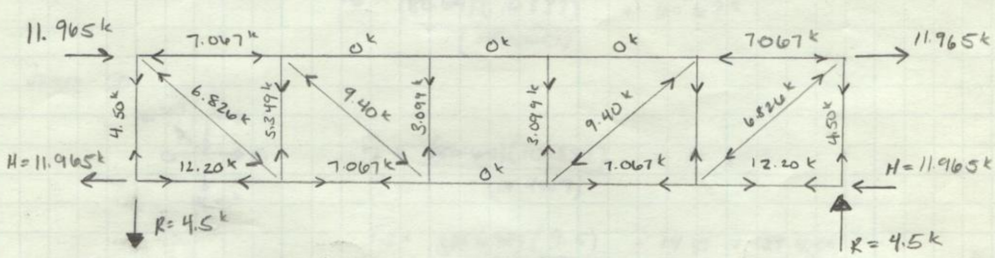
JOINT 7



$$F_{11} = \frac{(6.826^k)(9.5)}{14.4087} = 4.50^k \text{ COMPRESSION}$$

$$F_6 = 11.965^k - \frac{(6.826^k)(10.833)}{14.4087} = 6.833^k$$

LATERAL FORCES :



DESIGN GUIDE CONVENTION :

o←——→o ⇒ COMPRESSION

o→——←o ⇒ TENSION

Geometry of Truss

	Exterior Span=	12.5000	ft
125	Interior Span=	7.16666667	ft
	Diagonal L=	15.7003	ft
	Depth=	9.5	ft
	Trib Width =	18.75	ft
	Length=	57.1667	ft

LATERAL LOAD (Unfactored)

Force at Joint 7 and 12 (total lat/2)

F=	25.00	k
R=	8.31	k
H=	12.50	k
V@mid=	4.15	k
$Moment_g$=	14.89	ft-k
V@Adj=	1.57	k

Member	Force (k)
1	25.99 T
2	15.06 T
3	0.00
4	15.06 T
5	25.99 T
6	15.06 C
7	0.00
8	0.00
9	0.00
10	15.06 C
11	8.31 T
12	13.73 C
13	9.88 T
14	18.91 C
15	5.72 T
16	5.72 T
17	18.91 C
18	9.88 T
19	13.73 C
20	8.31 T

DEAD LOAD (Unfactored)

W_{psf} =	76
W_{klf} =	1.425
P_1=	8.91 k
P_2=	17.81 k
P_3=	14.01 k

Member	Force (k)
1	0.00
2	83.75 T
3	120.63 T
4	83.75 T
5	0.00
6	83.75 C
7	120.63 C
8	120.63 C
9	120.63 C
10	83.75 C
11	72.56 C
12	105.19 T
13	45.84 C
14	46.32 T
15	14.01 C
16	14.01 C
17	46.32 T
18	45.84 C
19	105.19 T
20	72.56 C

LIVE LOAD (Unfactored)

W_{psf} =	40	Outside Span
W_{klf} =	0.75	Outside Span
W_{psf} =	100	Inside Span
W_{klf} =	1.875	Inside Span
P_1=	4.69	k
P_2=	9.38	k
P_3=	11.41	k

Member	Force (k)
1	0.00
2	54.69 T
3	84.70 T
4	54.69 T
5	0.00
6	54.69 C
7	84.70 C
8	84.70 C
9	84.70 C
10	54.69 C
11	46.25 C
12	68.69 T
13	32.19 C
14	37.70 T
15	11.41 C
16	11.41 C
17	37.70 T
18	32.19 C
19	68.69 T
20	46.25 C

Sign Convention

Tension	(+)
Compression	(-)

Load Combinations

	Member	1.4D	1.2D+1.6L	1.2D+0.8W	1.2D+1.6W+L	0.9D+1.6W
Bottom	1	0.00	0.00	20.79	41.58	41.58
Bottom	2	117.25	188.00	112.55	179.28	99.47
Bottom	3	168.88	280.28	144.75	229.45	108.56
Bottom	4	117.25	188.00	112.55	179.28	99.47
Bottom	5	0.00	0.00	20.79	41.58	41.58
Top	6	-117.25	-188.00	-112.55	-179.28	-99.47
Top	7	-168.88	-280.28	-144.75	-229.45	-108.56
Top	8	-168.88	-280.28	-144.75	-229.45	-108.56
Top	9	-168.88	-280.28	-144.75	-229.45	-108.56
Top	10	-117.25	-188.00	-112.55	-179.28	-99.47
COL	11	-101.58	-161.07	-80.42	-120.02	-52.01
DIAG	12	147.27	236.13	115.24	172.95	72.70
VERT	13	-64.17	-106.51	-47.10	-71.39	-25.45
DIAG	14	64.84	115.90	40.45	63.02	11.43
VERT	15	-19.62	-35.07	-12.24	-19.07	-3.46
	16	-19.62	-35.07	-12.24	-19.07	-3.46
	17	64.84	115.90	40.45	63.02	11.43
	18	-64.17	-106.51	-47.10	-71.39	-25.45
	19	147.27	236.13	115.24	172.95	72.70
COL	20	-101.58	-161.07	-80.42	-120.02	-52.01

max values: 236.13 Tension
-161.07 Compression

Truss Member Design

Tyler Meek | Thesis Calcs | Member Design | P. 1/

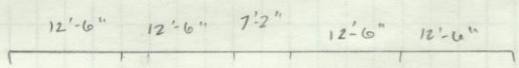
Truss Line 13 @ Level 20

Most common
W10 chords
HSS web

Depth = $9\frac{1}{2}$ ft

Trib width = $17\frac{1}{2}$ ft

Length = 57.167 ft



East - West

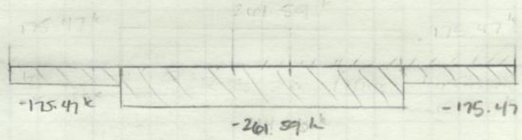
Windward (psf) = 26.75
Leeward (psf) = -11.81

F_{13} = 25k

Top Member

Tension

Compression



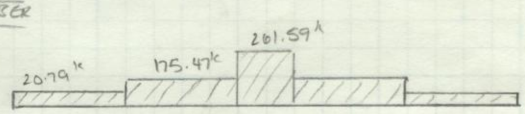
Axial

Try: W10 x 33 $\quad \phi P_{n_t}$ = 355k
$A = 9.71\ in^2 \quad \phi P_{n_c}$ = 272k
$I = 170\ in^4$

Bottom Member

Tension

Comp



Try: W10 x 33

Web Members: Max Comp = -150.33 k $(L = 9.5')$
Max Tens = 220.39 k

ϕP_c = 190k
ϕP_t = 267k

HSS 10x5x5/16
$A = 8.17\ in^2$
$I = 104\ in^4$

Appendix B: Lateral Load Spreadsheet

Typical Story Height = 10 ft

Typical Lower Levels = 19 & 20

Highest lateral forces for lower part

East-West level 19

Windward (psf)	Leeward (psf)
26.75	-11.81
(plf)	(plf)
267.50	118.10

Floors B	
C.P (ft) =	124
C.R (ft) =	121.22
Ecc (ft) =	-2.78
Add. 5% (ft) =	12.40
Total Ecc. (ft) =	9.62

Direct: 93.82932

F_T = 90.6 k

	K α L	FORCE
F_3	1	13.26501 k
F_5	1	13.26501 k
$F_{6.8}$	1.41	18.70366 k
$F_{9.5}$	1.14	15.12211 k
F_{12}	1.14	15.12211 k
F_{14}	1.14	15.12211 k

sign convention = ccw (+)

Eccentric: $F = k*d*\Theta$ $\Theta = P*e/\Sigma(K*d^2)$ = 0.049415 rad

	K	d = distance to C.R	FORCE
F_3	1	-95.01	-4.69466 k
F_5	1	-57.84	-2.85808 k
$F_{6.8}$	1.41	-28.09	-1.95706 k
$F_{9.5}$	1.14	19.50	1.098212 k
F_{12}	1.14	57.08	3.215394 k
F_{14}	1.14	92.25	5.196438 k

$\Sigma(K*d^2)$ = 17630.95855

TOTAL FORCE:	
F_3	13.27 k
F_5	13.27 k
$F_{6.8}$	18.70 k
$F_{9.5}$	16.22 k
F_{12}	18.34 k
F_{14}	20.32 k

East-West level 20

Windward (psf)	Leeward (psf)
26.75	-11.81
(plf)	(plf)
267.49	118.15

Floors A	
C.P (ft) =	124
C.R (ft) =	121.15
Ecc (ft) =	-2.85
Add. 5% (ft) =	12.40
Total Ecc. (ft) =	9.55

Direct: 93.83721

F_T = 92.84 k

	K α L	FORCE
$F_{1.4}$		0 k
F_4	1	21.69159 k
F_6	1	21.69159 k
F_8		0 k
F_{11}	1.14	24.72841 k
F_{13}	1.14	24.72841 k
F_{16}		0 k

sign convention = ccw (+)

Eccentric: $F = k*d*\Theta$ $\Theta = P*e/\Sigma(K*d^2)$ = 0.07752 rad

	K α L	d = distance to C.R	FORCE
$F_{1.4}$		-128.50	0 k
F_4	1	-85.34	-6.61516 k
F_6	1	-55.59	-4.30894 k
F_8		-7.84	0 k
F_{11}	1.14	30.50	2.69522 k
F_{13}	1.14	65.50	5.788264 k
F_{16}		112.21	0 k

$\Sigma(K*d^2)$ = 11432.124

TOTAL FORCE:	
$F_{1.4}$	0.00 k
F_4	21.69 k
F_6	21.69 k
F_8	0.00 k
F_{11}	27.42 k
F_{13}	30.52 k
F_{16}	0.00 k

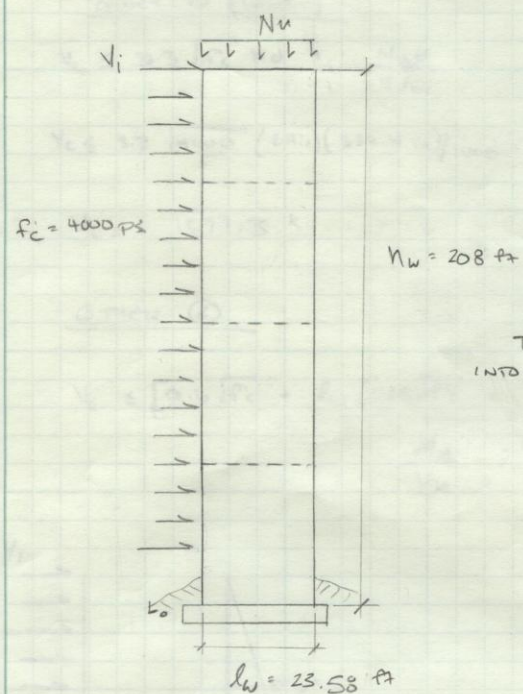
Appendix C: New Seismic Design Criteria and Loads

Seismic Design Criteria			
S_{DS} =	0.40615	$T_{model\text{-}x}$ =	2.1424 s
S_{D1} =	0.2263	$T_{model\text{-}y}$ =	1.7839 s
R =	2	C_u =	1.474
I =	1.25	T_a =	0.701
T =	1.033 s	C_s =	0.1367

Level	Height (ft)	Weight (k)	w*hk	CVX	Fi (k)	Vi (k)	M (ft-k)
26	266	715.65	2532299	0.053	144	477	38398.64
25	252	715.65	2339647	0.049	133	610	33610.13
24	242	715.65	2205030	0.046	126	736	30419.29
23	232	715.65	2072967	0.043	118	854	27415.71
22	222	715.65	1943517	0.041	111	965	24595.77
21	212	715.65	1816742	0.038	104	1069	21955.75
20	202	1424.02	3368200	0.070	192	1261	38785.41
19	192	1424.02	3126997	0.065	178	1439	34225.35
18	182	1424.02	2891549	0.060	165	1604	30000.00
17	172	1424.02	2662024	0.056	152	1755	26101.15
16	162	1424.02	2438604	0.051	139	1894	22520.37
15	152	1424.02	2221489	0.046	127	2021	19248.95
14	142	1424.02	2010898	0.042	115	2136	16277.87
13	132	1424.02	1807074	0.038	103	2239	13597.82
12	122	1424.02	1610289	0.034	92	2331	11199.09
11	112	1424.02	1420845	0.030	81	2412	9071.60
10	102	1424.02	1239089	0.026	71	2482	7204.80
9	92	1424.02	1065417	0.022	61	2543	5587.62
8	82	1424.02	900290	0.019	51	2594	4208.39
7	72	1424.02	744256	0.016	42	2637	3054.74
6	62	1424.02	597973	0.012	34	2671	2113.45
5	52	1424.02	462261	0.010	26	2697	1370.28
4	42	1424.02	338175	0.007	19	2716	809.67
3	32	1424.02	227145	0.005	13	2729	414.35
2	16	1426	82480	0.002	5	2734	75.23
	Σ	32812.46		**Base:**	2734		518470

Appendix D: Shear Wall Design

TYLER MEEK | THESIS CALCS | SHEAR WALL DESIGN P. 1/6



$t = 24$ in

V_i TABULATED IN EXCEL

TO SIMPLIFY DESIGN, BREAK INTO 4 SECTIONS:

3 OF 5 STORIES

1 OF 4 STORIES

150 pcf

SIMPLIFIED METHOD:

$V_c \le 2\sqrt{f'_c}\,hd \qquad d = 0.8\,l_w$

$V_c = 687{,}205 \text{ lb}/1000 = 687.2^k$

OTHER METHOD ①

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

TREAT N_u AS POINT LOAD

$D = (76 \text{ psf})(17.58)(23.58)$

$D = 31.36^k$

$L = (40 \text{ psf})(17.5 \text{ ft})(23.58)$

$L = 16.51^k$

$N_u = 1.2D + 1.6L = 64.0^k/\text{FLOOR}$

TYLER MEEK | THESIS CALCS | SHEAR WALL DESIGN | P. 2/6

OTHER ① (CONT.)

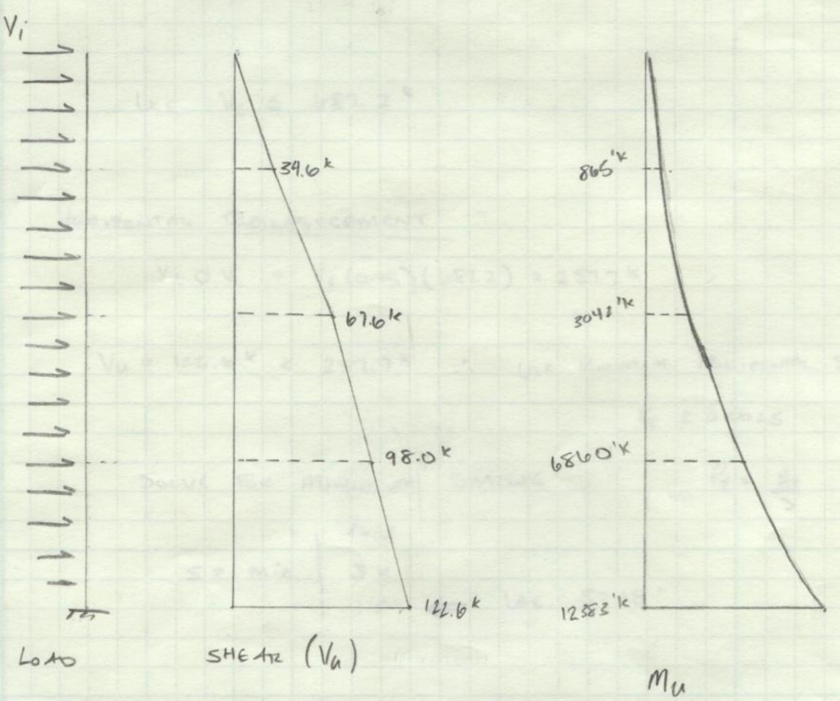
$$V_c \le 3.3\sqrt{f'_c}\,hd + \frac{N_u d}{4 l_w}$$

$$V_c \le 3.3\sqrt{4000}\,(24\text{in})(226.4\text{ in})/1000 + \frac{(64.0^k)(19\text{ Floors})(226.4)}{4\,(282.96\text{in})}$$

$$V_c \le 1377.3^k$$

OTHER ②

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



TYLER MEEK | THESIS CALCS | SHEAR WALL DESIGN p. 3/6

OTHER ② (CONT.)

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

$$V_c \le \left[0.6\sqrt{4000} + \frac{(283\text{ in})\left(1.25\sqrt{4000} + 0.2\frac{64^k(1000)(19)}{(283\text{ in})(24\text{ in})}\right)}{\frac{(12383\text{ ft-k})(12)}{(122.6^k)} - \frac{283}{2}}\right](24\text{ in})(226.4)$$

$$V_c \le \left[37.95\text{ psi} + \frac{(283\text{ in})(79.1\text{ psi} + 35.8\text{ psi})}{1212\text{ in} - 141.5\text{ in}}\right] 5433.6\text{ in}^2$$

$$V_c \le 371.3^k$$

USE $V_c \le 687.2^k$

HORIZONTAL REINFORCEMENT

$$\tfrac{1}{2}\phi V_c = \tfrac{1}{2}(0.75)(687.2) = 257.7^k$$

$V_u = 122.6^k < 257.7^k$ ∴ USE MINIMUM HORIZONTAL STEEL

$\rho_t \ge 0.0025$

SOLVE FOR MINIMUM SPACING $\qquad \rho_t = \frac{A_v}{s}$

$$s = \min \begin{cases} l_w/5 \\ 3h \\ 18'' \end{cases} \Leftarrow \text{USE } s = 18''$$

$A_v = s\rho_t = (18'')(0.0025) = 0.045\text{ in}^2$ VERY SMALL

→ USE #3

$\boxed{(2)\ \#3 \text{ @ } 18''\text{ O.C.}}$

TYLER MEEK | THESIS CALCS | SHEAR WALL DESIGN | P. 4/6

VERTICAL REINFORCEMENT

$\rho_t = 0.0025$

$$\rho_L = \frac{A_v}{S_n} \geq 0.0025 + 0.5\left(2.5 - \frac{h_w}{l_w}\right)\left(\rho_t - 0.0025\right) \rightarrow 0$$

USE $\rho_L = \rho_t$

USE (2) #3 @ 18" O.C

THIS SECTION IS WORST CASE AND ONLY REQUIRES MINIMUM REINFORCEMENT

∴ USE MINIMUM FOR ENTIRE WALL

TYLER MEEK | THESIS CALCS | SHEAR WALL DESIGN | P. 5/6

FLEXURAL DESIGN

$$d = 226.4 \text{ in}$$
$$jd = 0.9d = 203.7 \text{ in}$$

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

$$(12383\,'k)(12) = (0.9) A_s (60)(203.7) \Rightarrow A_s = 13.51 \text{ in}^2$$

C = T

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(13.51)(60000)}{(0.85)(4000)(24)} = 9.93 \text{ in}$$

$$jd = d - \frac{a}{2} = 226.4 - \frac{9.93}{2} = 221.4 \text{ in}$$

NEW A_s

$$A_s = \frac{(12383)(12)}{(0.9)(60)(221.4)} = 12.43 \text{ in}^2$$

TRY (16) #8 $\Rightarrow A_s = 12.64 \text{ in}^2$

TENSION CONTROLLED

$$d_t = (23.58)(12) - 3 = 279 \text{ in}$$

C = T

$$a = \frac{(12.64)(60000)}{0.85(4000)(24)} = 9.29 \text{ in}$$

$$c = \frac{a}{\beta_1} = \frac{9.29}{0.85} = 10.93 \text{ in}$$

$$\varepsilon_t = \varepsilon_u \left(\frac{d_t - c}{c} \right) = 0.003 \left(\frac{279 - 10.93}{10.93} \right) = 0.074$$

$\varepsilon_t = 0.074 > 0.005$ TENSION CONTROLLED

$\Rightarrow$ DESIGN MEETS REQUIREMENTS

TYLER MEEK | THESIS CALCS | SHEAR WALL DESIGN P. 1/

AMPAD

NEW LAYOUT:

$V_u = 144\ k$

$\ell = 119.04\ in$

$d = 95.23\ in$

$V_c = 216.82\ k$ (FROM SPREADSHEET)

HORIZONTAL REINFORCEMENT

$\phi V_n = 1/2 \phi V_c = 1/2(0.75)(216.82^k) = 81.31\ k$

$V_u = \phi V_s + \phi V_c$

$V_{S\,REQ} = \dfrac{V_u - \phi V_c}{\phi} = \dfrac{144 - 81.31}{0.75} \Rightarrow V_s = 83.59\ k$

$\dfrac{A_v}{s} = \dfrac{V_s}{f_y d} = \dfrac{83.59}{(60)(95.23)} = 0.0146$

$(2)\ \#3 = \dfrac{2(0.11)}{0.0146} = 15.06" \Rightarrow$ USE (2) #3 @ 14"

$s \leq 18"$ ∴ ok $\boxed{\text{USE (2) \#3 @ 14"}}$

VERTICAL REINFORCEMENT

$\rho_\ell = 0.0025 + 0.5\left(2.5 - \dfrac{h}{\ell}\right)(\rho_t - 0.0025)$

$\rho_\ell = 0.0025 + 0.5\left(2.5 - \dfrac{18}{119}\right)(0.0157 - 0.0025)$

$\rho_\ell = 0.0180$

TRY (2) #3 $s = \dfrac{2(0.11)}{0.0180} = 12.2\ in \Rightarrow$ $\boxed{\text{USE (2) \#3 @ 12"}}$

Tyler Meek | Thesis Calcs | Shear Wall Design | p. 21

Flexural Design

$d = 95.23$ in

$M_u = 12383$ ft k

$A_s = \frac{M_u}{4d} \Rightarrow A_s = 32.51$ in²

Try (26) #10 ; $A_s = 33.02$ in²

$a = \frac{A_s f_y}{0.85 f'_c b} = 32.37$ in

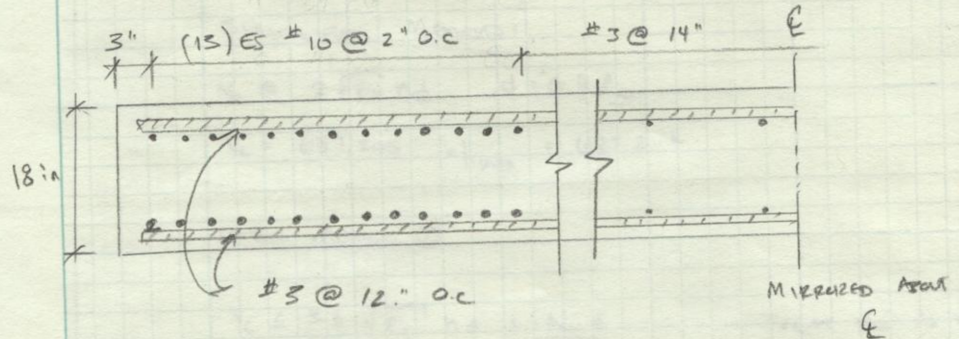
Tension Controlled

$d_t = 92.23$ in

$\varepsilon_t = 0.003\left(\frac{92.23 - 32.4}{32.4}\right) = 0.0055 > 0.005 \therefore$ ok

$\phi M_n = (0.9)(32.51)(60)\left(92.25 - \frac{32.37}{2}\right) \Rightarrow \phi M_n = 13350.0$ ft k

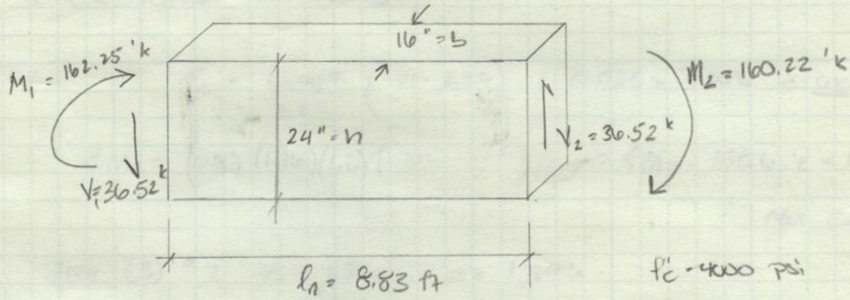
$\phi M_n > M_u \therefore$ ok



Appendix E: Coupling Beam Design

TYLER MEEK | THESIS CALCS | COUPLING BEAM DESIGN P. 1/3

SAMPLE CALC: TOP FLOOR



$\ell_n = 106$ in

$f'_c = 4000$ psi

$f_y = 60000$ psi

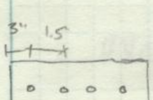
COUPLING BEAMS: (ACI 21.9.7)

$\frac{\ell_n}{h} = \frac{106}{24} = 4.42 \geq 4 \therefore$ SATISFY 21.5 EXCLUDING 21.5.1.3, 21.5.1.4

$b_w \geq \min \left| \begin{array}{l} 0.3h = 7.2 \checkmark_{ok} \\ 10 \text{ in} \end{array} \right.$

DESIGN AS FLEXURAL MEMBER & SHEAR MEMBER SEPARATELY:

$A_{s,min} = \frac{3\sqrt{f'_c}}{f_y} b_w d = \frac{3\sqrt{4000}}{60000}(10)(24-3) \Rightarrow A_{s,min} = 0.664 \text{ in}^2/\text{ft}$



$A_s = \frac{M_u}{4d} = \frac{162.25}{4(21)} = 1.93 \text{ in}^2/\text{ft}$ TRY (4) #6 $A_s = 1.76 \text{ in}^2$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(1.76)(60000)}{(0.85)(4000)(16)} \Rightarrow a = 1.94$ in

$c = \frac{a}{\beta_1} = \frac{1.94}{0.85} = 2.28$ in

TYLER MEEK | THESIS CALCS | COUPLING BEAM DESIGN | P. 2/5

CHECK TENSION CONTROLLED:

$$\varepsilon_t = \frac{0.003}{2.28}\left(21 - 2.28\right) = 0.025 > 0.005 \quad \therefore \text{ OK ✓}$$

$$\phi M_n = (0.9)(1.76)(60)\left(21 - \frac{1.94}{2}\right)/12 \Rightarrow \phi M_n = 158.6\ 'k < M_u$$

$\therefore$ NOT GOOD

TRY (3) #7, $A_s = 1.8\ in^2$: $a = 1.99\ in$

$c = 2.33$, $\varepsilon_t = 0.024 > 0.005 \Rightarrow$ TENSION CONTROLLED

$\phi M_n = 162.0\ 'k < 162.25$ $\therefore$ NOT GOOD

TRY (4) #7, $A_s = 2.4\ in^2$; $a = 2.65\ in$

$c = 3.11\ in$

$\varepsilon_t = 0.017 > 0.005$ $\therefore$ TENSION CONTROLLED

$\phi M_n = 212.5\ 'k > M_u$ $\therefore$ OK FOR FLEXURE

(4) #7

SHEAR DESIGN

$V_u = 36.52\ k$

W/OUT STEEL

$$V_c = 2\sqrt{f'_c}\, b_w d = 2\sqrt{4000}(16)(24)/1000 = 48.57\ k$$

$$\phi V_n = \tfrac{1}{2}\phi V_c = \tfrac{1}{2}(0.75)(48.57) = 18.2\ k$$

SHEAR STRENGTH OF MINIMUM STEEL

ASSUME #3 ($A_s = 0.22\ in^2$) $\quad d/2 = \frac{21}{2} = 10\ in$

$$V_{s,min} = \frac{A_v f_y d}{s} = \frac{(0.22)(60)(21)}{10} = 27.1\ k$$

TYLER MEEK | THESIS CALCS | COUPLING BEAM DESIGN P. 3/3

$$4\sqrt{f'_c}\,bd = 97.15$$

$V_{s\,min} < 97.15 \Rightarrow$ SPACING IS OK

$$A_{v\,min} = \max \begin{cases} \dfrac{50\,b\,s}{f_y} = 0.133\ in^2 \Leftarrow \\ \dfrac{0.75\sqrt{f'_c}\,b\,s}{f_y} = 0.126\ in^2 \end{cases}$$

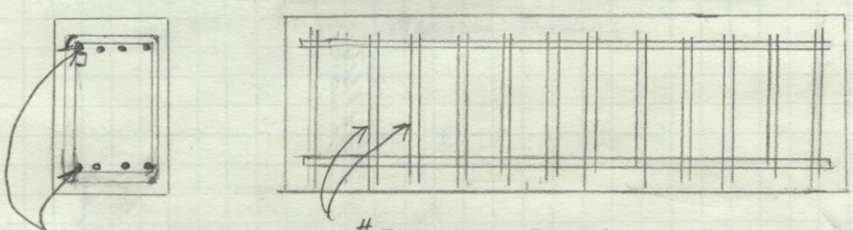
$A_v = 0.22\ in^2 > 0.133\ in^2 \therefore$ OK

$$V_{u\,max} = 36.52 = \phi V_c + \phi V_{s\,max}$$

$$36.52 \le 18.2^k + 27.7^k = 45.9^k$$

$V_u \le \phi V_n \therefore$ OK

#3 STIRRUPS SPACED @ 10"



(4) #7 T&B

FOR FLEXURE

#3 STIRRUPS @ 10" o.c

FOR SHEAR

TYLER MEEK | THESIS CALCS | DEVELOPMENT LENGTH P 1/1

ACI 12.5 – DEVELOPMENT OF STANDARD HOOKS IN TENSION

$$\ell_{dh} = \frac{0.02\, \psi_e f_y}{\lambda \sqrt{f'_c}\, d_b}$$

$\psi_e = 1.0$
$\lambda = 1.0$
$d_b = 0.875$

$$\ell_{dh} = \frac{0.02\,(1.0)(60000)}{(1.0)\left(\sqrt{4000}\right)(0.875)}$$

$$\boxed{\ell_{dh} = 21.68 \text{ in} \sim 22 \text{ in}}$$

ACI 12.2.3 – DEVELOPMENT OF LONGITUDINAL BARS

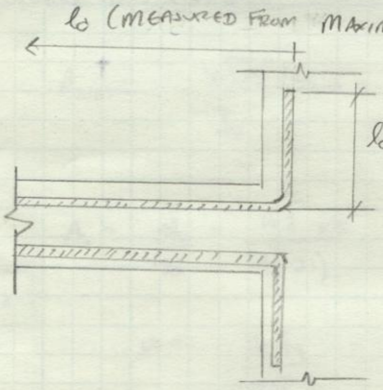
$$\ell_d = \left[\left(\frac{3}{40}\right)\left(\frac{f_y}{\lambda\sqrt{f'_c}}\right)\left(\frac{\psi_t\,\psi_e\,\psi_s}{\frac{c_b + K_{tr}}{d_b}}\right)\right] d_b$$

$\psi_t = \psi_e = \psi_s = 1.0$
$\lambda = 1.0$
$c_b = 2"$
$d_b = 0.875"$

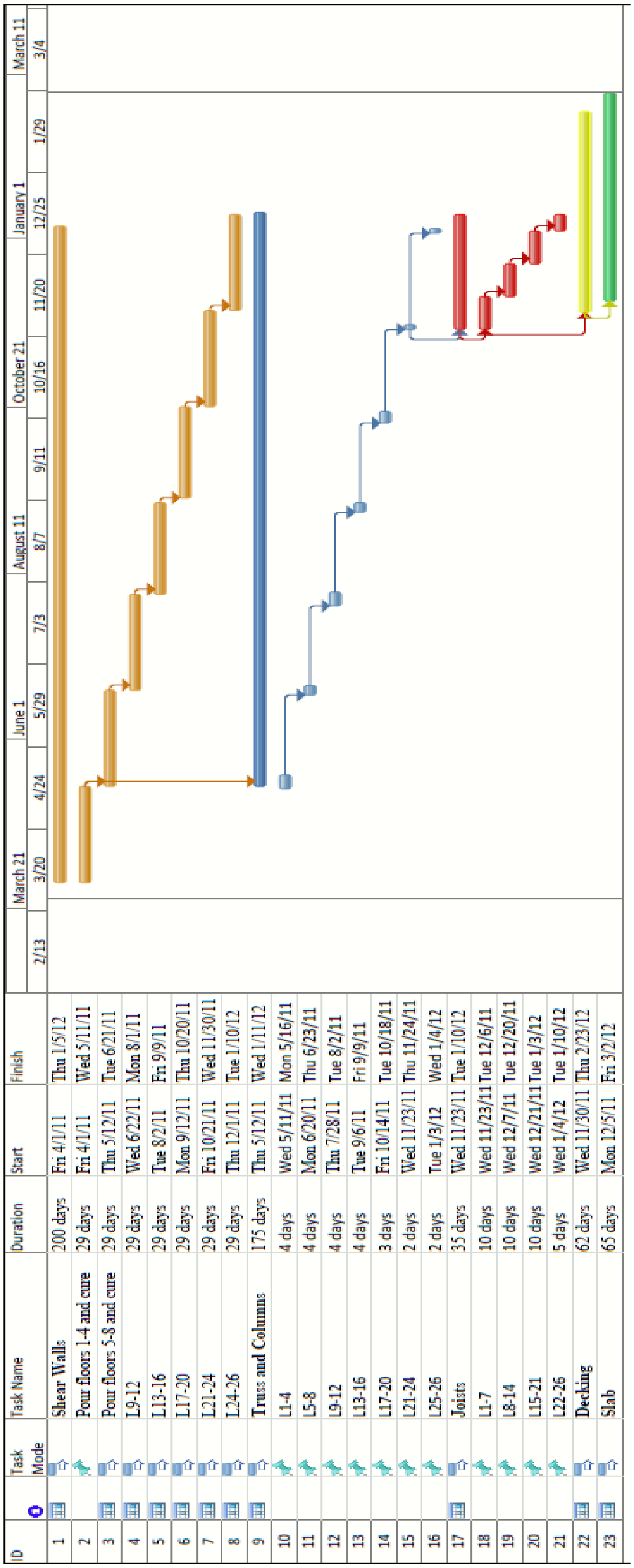
$K_{tr} = 0$ FOR THIS CALC

$$\boxed{\ell_d = 27.24" = 27\tfrac{1}{2}"}$$

ℓ_d (MEASURED FROM MAXIMUM MOMENT)



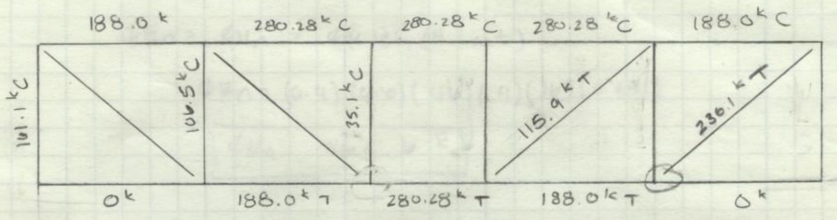
Appendix F: Construction Schedule



ID	Task Mode	Task Name	Duration	Start	Finish
1		Shear Walls	200 days	Fri 4/1/11	Thu 1/5/12
2		Pour floors 1-4 and cure	29 days	Fri 4/1/11	Wed 5/11/11
3		Pour floors 5-8 and cure	29 days	Thu 5/12/11	Tue 6/21/11
4		L9-12	29 days	Wed 6/22/11	Mon 8/1/11
5		L13-16	29 days	Tue 8/2/11	Fri 9/9/11
6		L17-20	29 days	Mon 9/12/11	Thu 10/20/11
7		L21-24	29 days	Fri 10/21/11	Wed 11/30/11
8		L24-26	29 days	Thu 12/1/11	Tue 1/10/12
9		Truss and Columns	175 days	Thu 5/12/11	Wed 1/11/12
10		L1-4	4 days	Wed 5/11/11	Mon 5/16/11
11		L5-8	4 days	Mon 6/20/11	Thu 6/23/11
12		L9-12	4 days	Thu 7/28/11	Tue 8/2/11
13		L13-16	4 days	Tue 9/6/11	Fri 9/9/11
14		L17-20	3 days	Fri 10/14/11	Tue 10/18/11
15		L21-24	2 days	Wed 11/23/11	Thu 11/24/11
16		L25-26	2 days	Tue 1/3/12	Wed 1/4/12
17		Joists	35 days	Wed 11/23/11	Tue 1/10/12
18		L1-7	10 days	Wed 11/23/11	Tue 12/6/11
19		L8-14	10 days	Wed 12/7/11	Tue 12/20/11
20		L15-21	10 days	Wed 12/21/11	Tue 1/3/12
21		L22-26	5 days	Wed 1/4/12	Tue 1/10/12
22		Decking	62 days	Wed 11/30/11	Thu 2/23/12
23		Slab	65 days	Mon 12/5/11	Fri 3/2/12

Appendix G: Connection Design

TYLER MEEK | THESIS CALCS | CONNECTION DESIGN P. 1/10



CHORDS: W10 x 33

WEB: HSS 10 x 5 x 5/16

(1) SHEAR LAG FRACTURE STRENGTH IN HSS:

$\phi R_n = \phi F_u A_e$

$A_n = A_g - 2 t t_1$, $A_g = 8.17 \text{ in}^2$

$t = 0.291 \text{ in}$

$t_1 = 1/2 \text{ in}$

$A_n = 8.17 - 2(0.291)(1/2)$

$A_n = 7.88 \text{ in}^2$

$L_w = 16 \text{ in}$ ⇐ ASSUME

$\bar{x} = \dfrac{B^2 + 2BH}{4(B+H)}$; $B = (b/t)\, t = (14.2)(0.291) = 4.13$

$H = (h/t)\, t = (31.4)(0.291) = 9.14$

$\bar{x} = \dfrac{4.13^2 + 2(4.13)(9.14)}{4(4.13 + 9.14)}$

$\bar{x} = 1.74 \text{ in}$

$U = 1 - \dfrac{1.74}{16} = 0.891 < 0.9$

$\phi R_n = (0.75)(62)(0.891)(7.88) \Rightarrow \boxed{\phi R_n = 326.1 \text{ k}}$

SOLVE FOR U TO FIND EFFICIENT L_w

TYLER MEEK | THESIS CALCS | CONNECTION DESIGN p. 2/11

② SHEAR STRENGTH OF HSS @ WELDS

$\phi R_n = \phi V_n = \phi 0.6 F_y (4 L_w t)$

$\phi R_n = (0.9)(0.6)(46)(4)(16)(0.291)$

$\boxed{\phi R_n = 462.6^k}$

SOLVE FOR L_w FOR MOST EFFICIENT DESIGN

③ STRENGTH OF THE WELD CONNECTING GUSSET PLATE TO HSS:

$\phi R_n = \phi F_w A_w$

$\theta = 0°$ (∴ LOAD PARALLEL TO WELD)

$F_{Exx} = 70$ ksi

$F_w = (0.6)(70)(1.0 + 0) = 42$ ksi

$A_w = (0.707)(3/16 - 1/16)(4)(16) = 5.66 \text{ in}^2$

MIN WELD = 1/8 in

∴ TRY 3/16

$\phi R_n = (0.75)(42)(5.66) = \Rightarrow \phi R_n = 178.3^k < P_u$

∴ NOT GOOD

TRY 1/4" WELD

$A_w = 8.48 \text{ in}^2$

$\phi R_n = (0.75)(42)(8.48) = 267.1^k > P_u$ ∴ OK

$\boxed{\phi R_n = 267.1^k}$

TYLER MEEK | THESIS CALCS | CONNECTION DESIGN | P. 3/11

(4) SHEAR STRENGTH OF GUSSET PLATE

$$\phi R_n = \phi v_n = \phi(0.6 F_{y1})(2 L_w t_1)$$

$$\phi R_n = (0.9)(0.6)(50)(2)(16)(1/2) \Rightarrow \boxed{\phi R_n = 432.0^k}$$

∴ CONNECTION OK, DEPENDING ON RESULTS MAY RESIZE GUSSET PLATE TO LESS THICKNESS

(2), (3), & (4) STILL APPLY

$\phi R_n = 462.6^k$

$\phi R_n = 467.1^k$

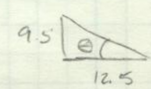
$\phi R_n = 432.0^k$

(5) BUCKLING OF GUSSET PLATE

$T = 236.1^k$

$C = 106.5^k$

$\theta = \tan^{-1}\left(\frac{9.5}{12.5}\right) = 37.2°$



$$\phi_c P_n = \phi_c A_g F_{cr}$$

$$\lambda_c = \frac{kl}{r\pi}\sqrt{\frac{F_{y1}}{E}}$$

$K = 1.2$ TRY $l = 6$ in

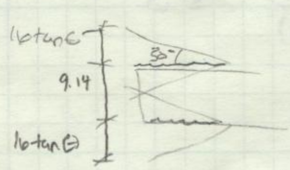
$r = 0.144$

$F_{y1} = 50$ ksi

$E = 29000$ ksi

$\lambda_c = 0.66 < 1.5$

∴ $F_{cr} = 0.658^{0.66^2}(50) \Rightarrow F_{cr} = 41.6$ ksi



$l_w = 27.62$ in

$A_g = (27.62 \text{ in})(1/2 \text{ in})$

$A_g = 13.81 \text{ in}^2$

$\phi P_c = (0.85)(13.81)(41.6)$

$\phi P_c = 488.3^k < P_u$ ∴ OK

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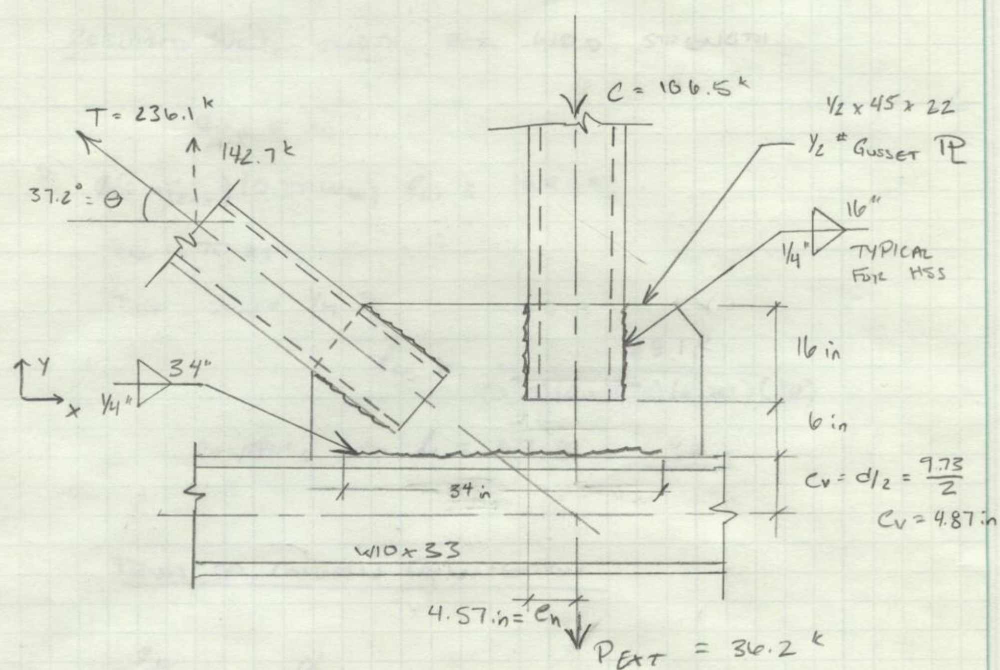

$\Sigma F_y = 0$ ∴ NO TENSION ON WELD

$M = T\cos\theta \times e_v = (236.1^k)\cos(37.2°)(4.87) = 915.86$ in k

REQUIRED WELD LENGTH: FOR DIRECT SHEAR FOR GUSSET PL

$\phi R_n \geq V_u$

$\phi(0.6F_y)\, t\, l_w \geq V_u = 188.1^k$

$l_w \geq \dfrac{188.1}{(0.9)(0.6)(50)(1/2)} = 13.9\text{ in} \sim 14\text{ in}$

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Required Weld Length: Per Weld Strength

$$\phi R_n \geq V_u$$

$$\phi(0.6F_{EXX})(0.707w_w)\,l_w \geq 188.1^k$$

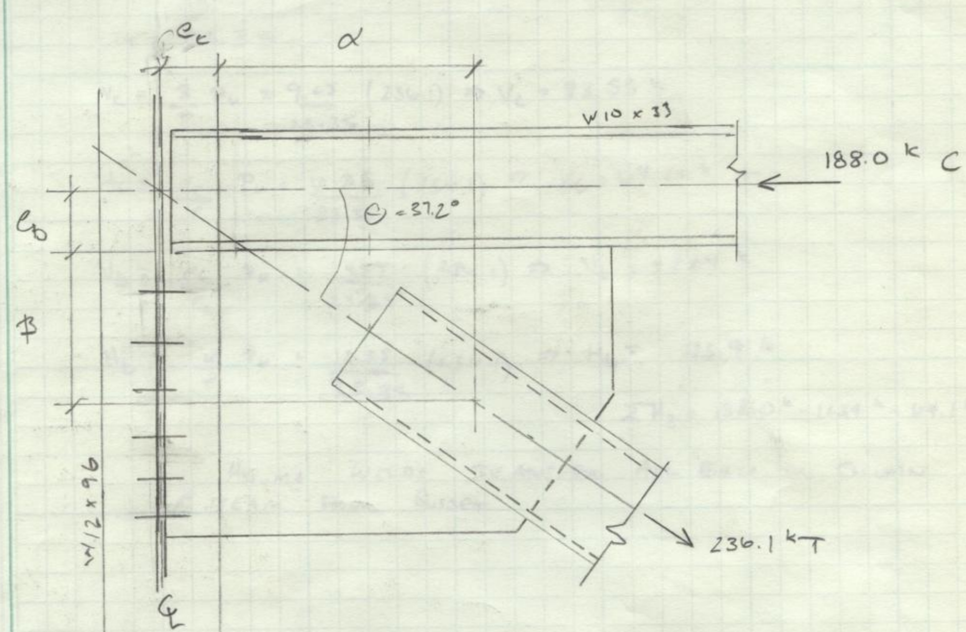
$F_{EXX} = 70$ ksi

Try $w_w = 1/4$"

$$l_w \geq \frac{188.1^k}{(0.75)(0.6)(70)(0.707)(1/4)}$$

Controls $\Rightarrow$ $l_w = 33.8$ in $\approx$ 34 in

Truss to Column Connection



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TRY $B = 9.25"$ ⇐ 6 BOLTS w/ $s = 2"$

TRY 3/4" ⌀ A325 BOLTS

$e_b = 4.87$ in

$e_c = 6.35$ in

SOLVE FOR α

$$\tan\theta = \frac{e_b + B}{e_c + \alpha} \Rightarrow \alpha = \frac{e_b + B}{\tan\theta} - e_c$$

$$\alpha = \frac{4.87 + 9.25}{\tan(37.2)} - 6.35$$

$$\alpha = 12.25 \text{ in}$$

DETERMINE FORCE DISTRIBUTION

$$r = \sqrt{(e_c + \alpha)^2 + (e_b + B)^2} = \sqrt{(6.35 + 12.25)^2 + (4.87 + 9.25)^2}$$

$$r = 23.35$$

$$V_c = \frac{B}{r} P_u = \frac{9.25}{23.35}(236.1) \Rightarrow V_c = 93.53^k$$

$$H_c = \frac{e_c}{r} P_u = \frac{6.35}{23.35}(236.1) \Rightarrow H_c = 64.21^k \; T$$

$$V_b = \frac{e_b}{r} P_u = \frac{4.87}{23.35}(236.1) \Rightarrow V_b = 49.24^k$$

$$H_b = \frac{\alpha}{r} P_u = \frac{12.25}{23.35}(236.1) \Rightarrow H_b = 123.9^k$$

$$\Sigma H_B = 188.0^k - 123.9^k - 64.1^k \; C$$

ASSUME WELDS TRANSFER ALL FORCE TO COLUMN & BEAM FROM GUSSET

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BEAM TO COLUMN: $V = 49.24^k$
$H = 64.11^k$ C

GUSSET TO COLUMN: $V = 93.53^k$
$H = 64.211^k$ T

BOLTED - BOLTED CONNECTION FOR CONSTRUCTION EASE

LIMIT STATES: (GUSSET-TO-COLUMN)

BOLT:

① SHEAR & TENSION

② BEARING & TEAROUT
- PL
- ANGLE
- COLUMN WEB

ANGLES:

③ SHEAR YIELD

④ SHEAR RUPTURE

⑤ BLOCK SHEAR

$\left.\right\}$ ANGLE & PL

(BEAM-TO-COLUMN)

① BOLT SHEAR

② BOLT BEARING & TEAROUT
- BEAM
- COLUMN
- ANGLE

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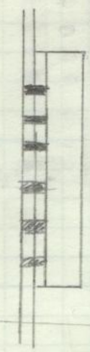
GUSSET - TO - COLUMN:

BOLTS (3/4" Ø A325N) ⇒ $\frac{93.53^k}{15.9^k/\text{BOLT}}$ = 5.9 → 6 BOLTS

SHEAR STRESS = $\frac{93.53^k}{(6)(0.442\ in^2)}$ = 35.27 KSI



AVAILABLE TENSILE STRENGTH ⇒

$F'_t = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v = 1.3(90) - \frac{90}{36}(35.27)$

F'_t = 28.83 KSI < 90 KSI ∴ 28.83 KSI CONTROLS

ϕr_{nt} = (0.75)(28.83 KSI)(0.442 in²) = 9.56 k

$r_{ut} = \frac{64.21^k}{6\ \text{BOLTS}}$ = 10.70 k > ϕr_{nt} ∴ NOT GOOD

TRY 7 BOLTS : β = 10.25 in e_b = 4.87 in

α = 13.57 in e_c = 6.35 in

r = 25.01 in

GUSSET-COLUMN V_c = 96.76 k H_c = 59.95 k T

BEAM-COLUMN V_b = 45.97 k H_b = 128.1 k ΣH_b = 188 k - 128.1 k = 59.9 k C

BOLTS (3/4" Ø A325 N) ⇒ $\frac{96.76^k}{15.9\ ^k/\text{BOLT}}$ = 6.08 → 7 BOLTS

SHEAR STRESS = $\frac{96.76^k}{7(0.442)}$ = 31.27 KSI

AVAILABLE TENSILE STRENGTH ⇒ F'_t = 38.82 KSI ⇐ CONTROLS

ϕr_{nt} = 12.87 k

$r_{ut} = \frac{64.21^k}{7\ \text{BOLTS}}$ = 9.17 k/BOLT < 12.87 k ∴ OK

SHEAR & TENSION OK

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BEARING & TEAROUT:

PL = 1/2" THICK
A36

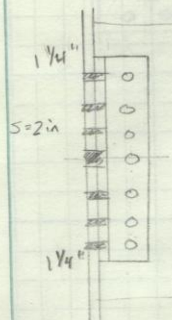
NOTE: NO TEAROUT IN PL

COLUMN WEB
$t = 0.55$ in
A992

ANGLE: TRY
L 5x5x 1/2

WILL NOT CONTROL,
THICKER & STRONGER
MATERIAL

$t = 1/2$"



$BEARING = 70.15^k (1/2) = 35.1^k$

$TEAROUT = 44.0^k (1/2) = 22^k$

$\Sigma = 232^k \gg H_c = 96.76^k \therefore$ OK FOR BEARING & TEAROUT

SHEAR RUPTURE, SHEAR YIELD & BLOCK SHEAR

SHEAR RUPTURE:

$\phi V_n = 0.6 \phi F_u A_n = (0.6)(0.75)(58)(6.81 \text{ in}^2)$

$A_n = \left[19.75 - 7\left(0.75 + \frac{1}{8}\right)\right](1/2) = 6.81 \text{ in}^2$

$\phi V_n = 177.7^k > V_u \therefore$ OK

SHEAR YIELDING:

$\phi V_n = \phi F_y A_g = (1.0)(36)(19.75)(1/2) = 355.5^k \gg V_u$

$\therefore$ OK

BLOCK SHEAR: $L_{en} = 1 1/2$"

TENSION RUPTURE $= 46.2$ k/in $(1/2) = 23.1^k$

SHEAR: YIELD $= (0.6)(36)(13.25)(1/2) = 143.1^k$

RUPTURE $= (0.6)(58)[13.25 - 7.5(3/4 + 1/16)](1/2) = 124.5^k$

$\phi V_n = 0.75(23.1^k + 124.5^k)$

$\phi V_n = 110.7^k \gg V_u \therefore$ OK

GUSSET - TO - COLUMN CONNECTION OK

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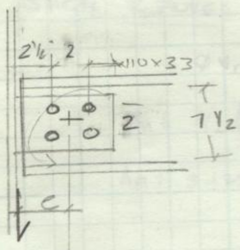
BEAM-TO-COLUMN :



$\# BOLTS = \frac{59.95^k}{15.9^k/BOLT} = 3.8 \to 4$ BOLTS

BEAM WEB = 0.29 in

COLUMN WEB = 0.55 in

L6 x 6 x 1/2 A36

DIRECT SHEAR ⇒ $\frac{59.95^k}{4} = 14.99^k$ /BOLT

MOMENT SHEAR COMPONENTS:

$M_u = (59.95)(3 1/2)$

$M_u = 209.8$ in-k

$R_{imx} = \frac{M_u x_i}{\Sigma(x_j^2 + y_i^2)}$

$R_{imx} = \frac{(209.8)(1.0)}{4(1.0) + 4(1.0)} = 26.2^k$

$R_{imy} = R_{imx} = 26.2^k$

$R_u = \sqrt{26.2^2 + (26.2 + 14.99)^2} = 30.62^k$

NOT GOOD FOR SINGLE SHEAR.

MAKE DOUBLE ANGLE CONNECTION :

$\phi r_n = 31.8^k$/BOLT $> r_u = 30.62^k$

∴ OK

FROM LAST CALCULATION,

SHEAR CONTROLS ∴ CONNECTION WORKS

BEARING & TEAROUT

TEAROUT $= 1.2 L_c F_u t = 1.2(1 1/2 - 1/8)(58)(1/2) = 47.85^k$

BEARING $= 2.4 d_b F_u t = (2.4)(3/4)(58)(1/2) = 52.2^k$

⇒ $100.05^k > V_u$ & H_u ∴ OK

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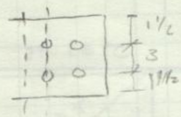
$L\,6 \times 6 \times 1/2$



SHEAR RUPTURE:

$\phi V_n = 0.6 \phi F_u A_n$

$A_n = 1/2 \left[6 - 2\left(3/4 + \frac{1}{8}\right)\right]$

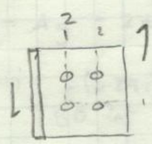
$A_n = 2.125$ $\phi V_n = (0.6)(1.0)(58)(2.125) = \phi V_n = 74.0^k > V_u$

$\therefore$ OK

SHEAR YIELDING:

$\phi V_n = \phi F_y A_g = (1.0)(36)(3\ in^2) = 108^k > V_u \therefore$ OK

BLOCK SHEAR



1-1

TENSION RUPTURE = $F_u A_{nt}$

$= (58)(1/2)\left[4.5 - 1.5\left(3/4 + 1/16\right)\right]$

$= 95.16^k$

SHEAR RUPTURE = $0.6 F_u A_{nv}$

$= (0.6)(58)(1/2)\left[4.5 - 1.5(3/4 + 1/16)\right]$

$= 57.09^k$

SHEAR YIELD = $0.6 F_y A_{nv}$

$= (0.6)(36)(4.5)(1/2)$

$= 48.6$

$\phi V_n = 0.75(95.16^k + 48.6^k)$

$\phi V_n = 107.8^k > V_u \therefore$ OK

CONTROLS

2-2

TENSION RUPTURE = $F_u A_{nt}$ = $(58)(1/2)\left[3 - 1(3/4 + 1/16)\right]$

$= 63.4^k$

SHEAR RUPTURE = $0.6 F_u A_{nv}$ $= (0.6)(58)(2)(1/2)\left[4.5 - 1.5(3/4 + 1/16)\right]$

$= 114.1^k$

SHEAR YIELDING = $0.6 F_y A_{nv} = (0.6)(36)(4.5)(2)(1/2) = 97.2$

$\phi V_n = 120.4^k > V_u \therefore$ OK