33 Harry Agganis Way

Boston, Massachusetts

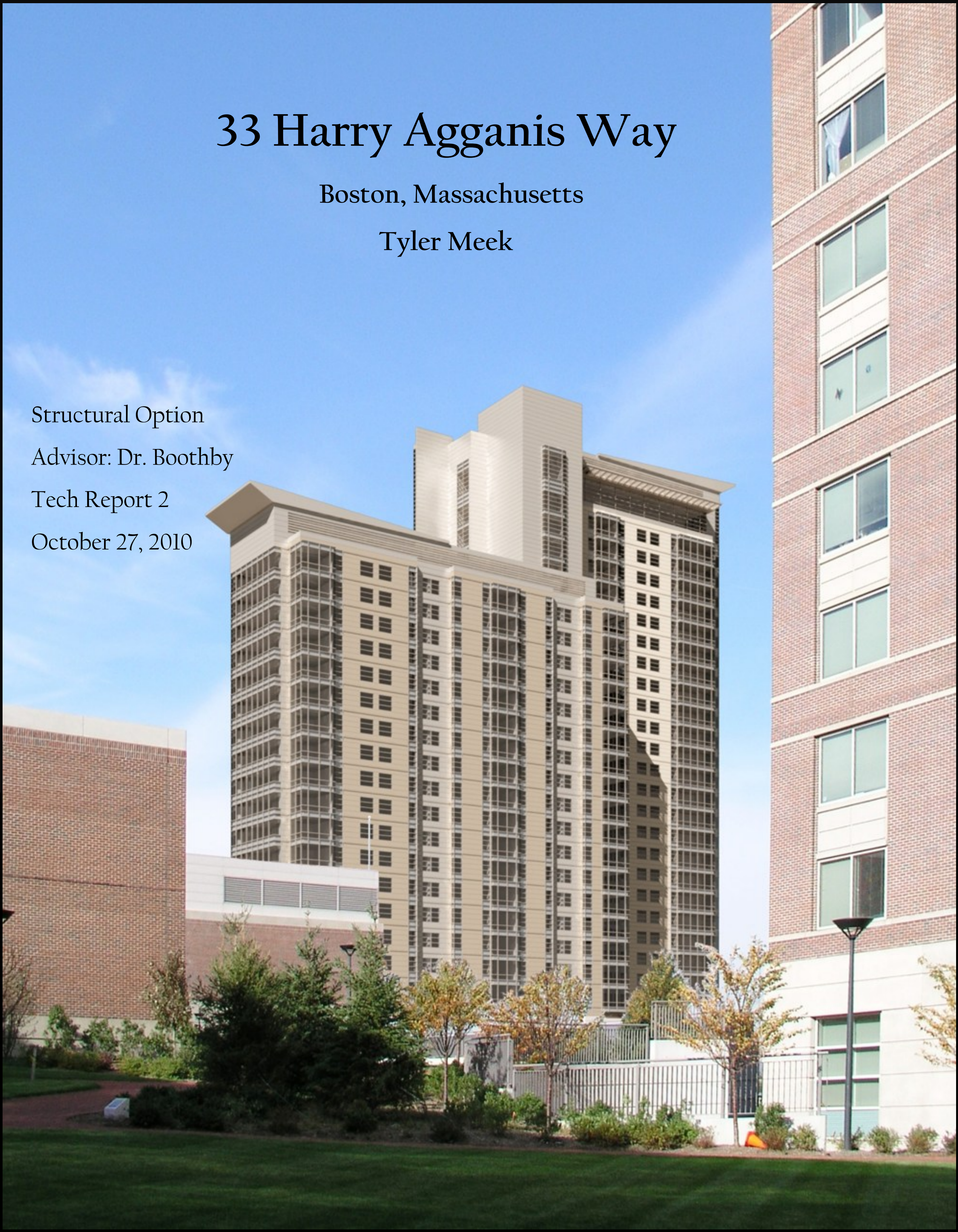
Tyler Meek

Structural Option

Advisor: Dr. Boothby

Tech Report 2

October 27, 2010



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Executive Summary

The goal of this Pro-Con Structural Study of Alternate Floor systems technical report was to examine 3 alternative floor systems and assess the feasibility of each system. In the list that follows are the three floor systems that were researched, analyzed and designed for this study.

- One-way precast planks on steel framing
- One-way precast planks on staggered trusses
- Two-way flat plate with one-way post tensioning

Each system was evaluated using both structural and non-structural criteria; a summary chart of these comparisons is presented near the end of this report. Each system's viability for use in Res Tower II was explored using the results of analysis and comparisons.

Only the two-way flat plate with one-way post tensioning system was determined to not be feasible. This determination was not due to insufficient characteristics but only because inappropriate assumptions and design choices were made. If this system were to be changed to a one-way system with post-tensioned girders, it would become a very viable alternative.

The other two alternative systems were determined to be feasible and viable options. Both these systems use precast planks which come with their own advantages but the framing elements used in the systems are extremely efficient with appropriate design techniques.

Introduction

Located on the Boston University Campus, 33 Harry Agganis Way, which will be referred to as Res Tower II, is a 27 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 to be tall.

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 walkout space, occupies the 27th 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 26, 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; which 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 inviting, yet durable, roof system.

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.

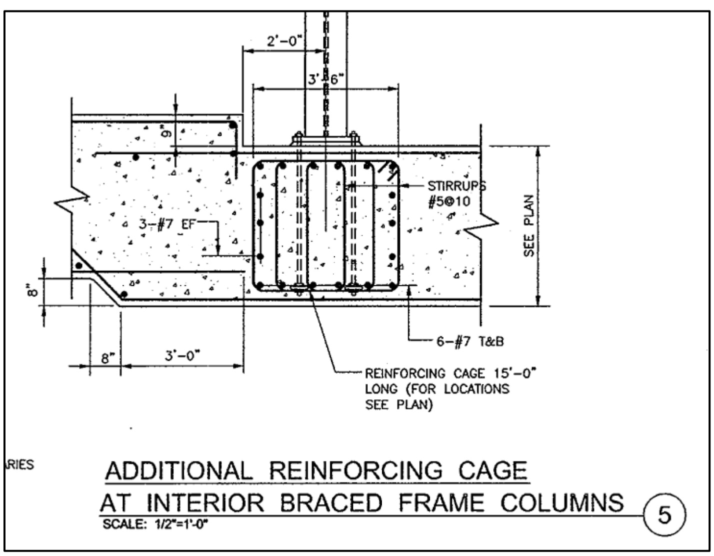


Figure 1: Additional foundation reinforcing

A 9" deep trench runs along the center of each towers foundation, parallel to the length of the building. This trench is filled in with 4000 psi concrete and reinforced with WWF 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 system columns marked in blue.

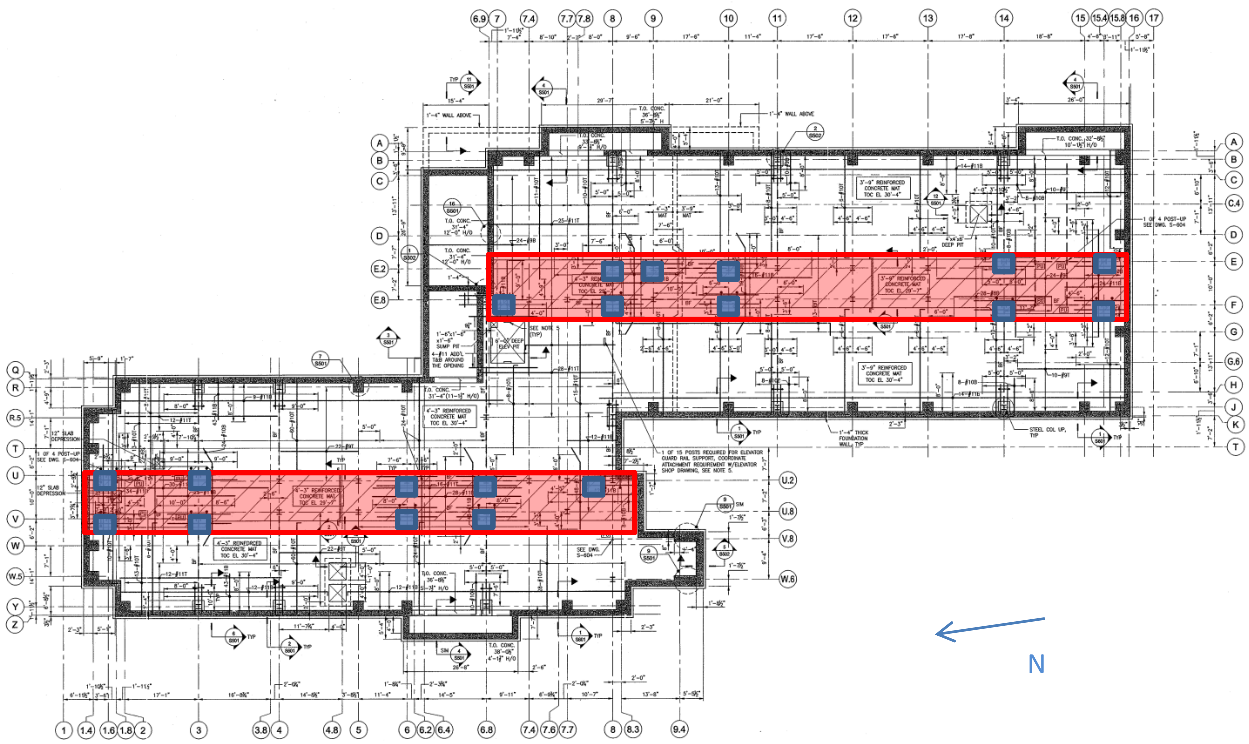


Figure 2: Foundation Trench

Floor Construction

The typical floor construction for Res Tower II is 3" 18 gage galvanized steel deck with 3 ¼" lightweight concrete topping, a total thickness of 6 ¼", and 6x6 WWF 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 acts compositely.

The decking typically spans about 8'-9" supported by beams ranging in size from W14's to W18's. These composite beams then span roughly 23 feet to girders or columns. The girders have the same range in sizes as the beams mentioned previously. These spans create a typical bay size of 17-18' x 24-23'. The actual bay sizes vary but never too far from the typical dimensions. Figure 3 shows a typical floor plan for floors 3-18.

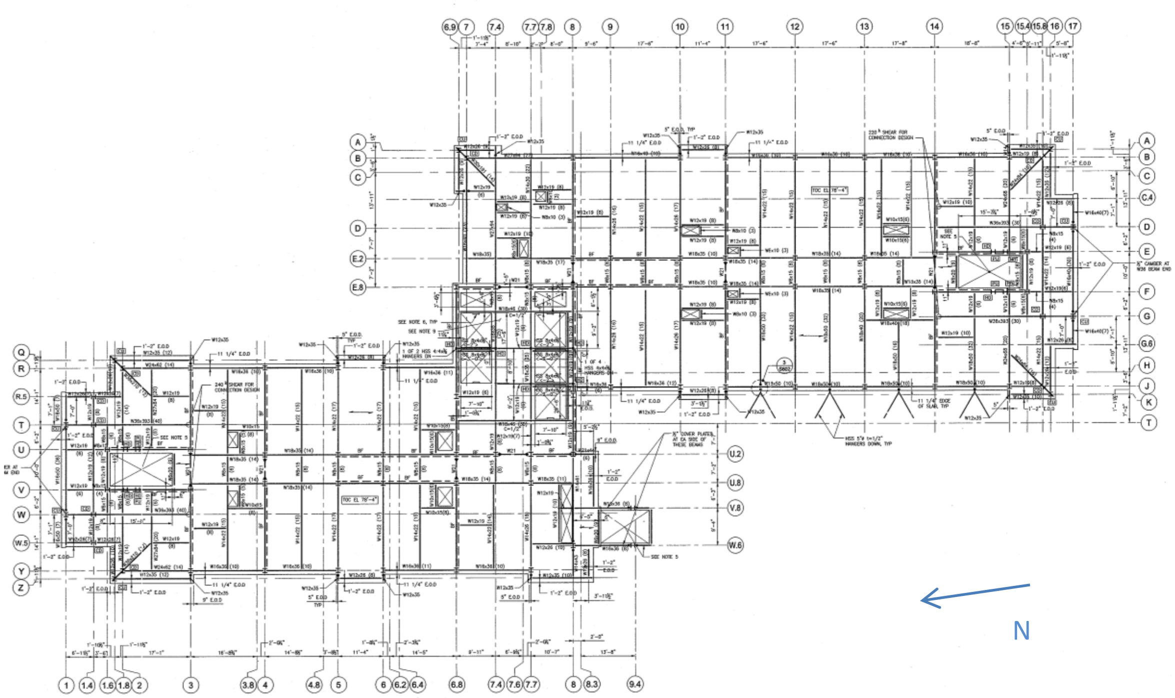
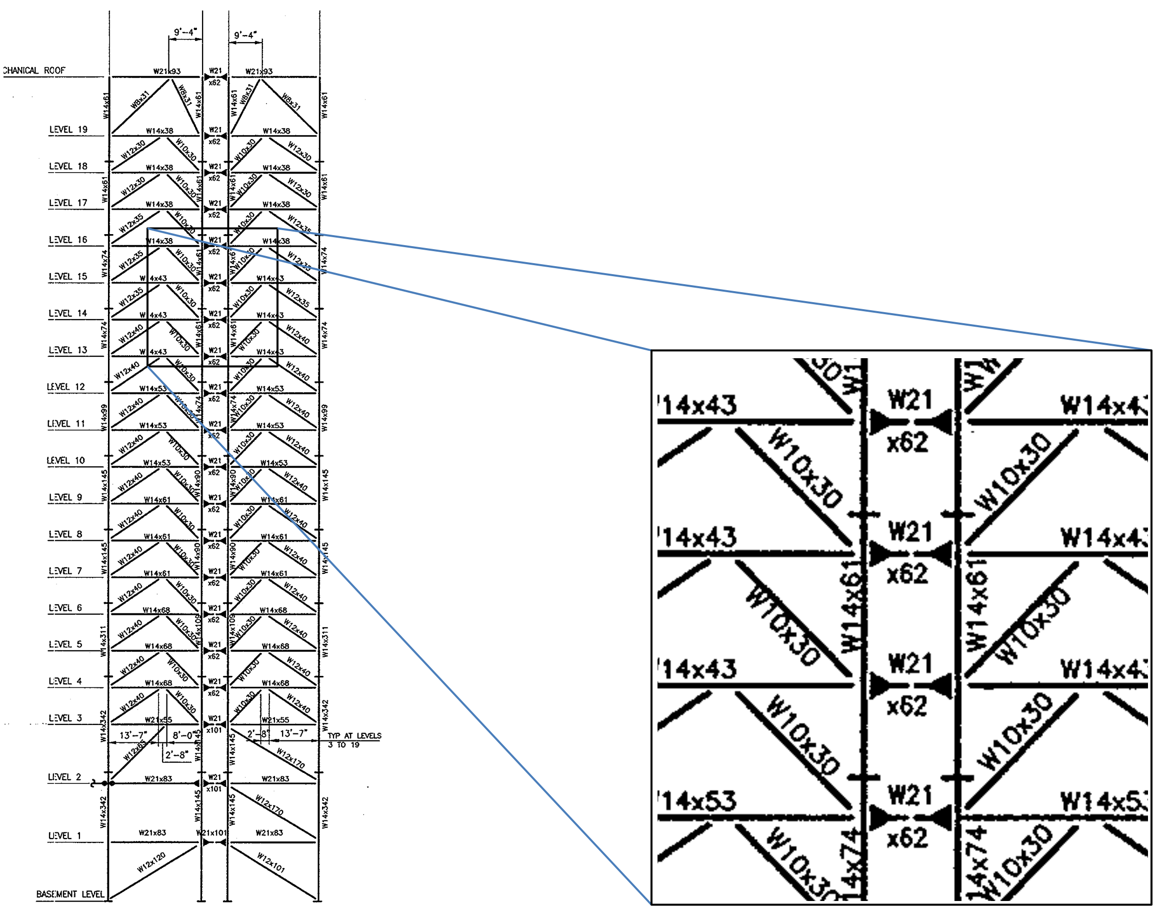


Figure 3: Typical floor plan

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 overturning 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 by ASCE7-10 as a concentrically braced frame that has an R value of 3.25. 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. The braced framed locations are highlighted in figure 5.



BRACED FRAME AT GRIDLINE 11

Figure 4: Braced frame elevation with moment connection

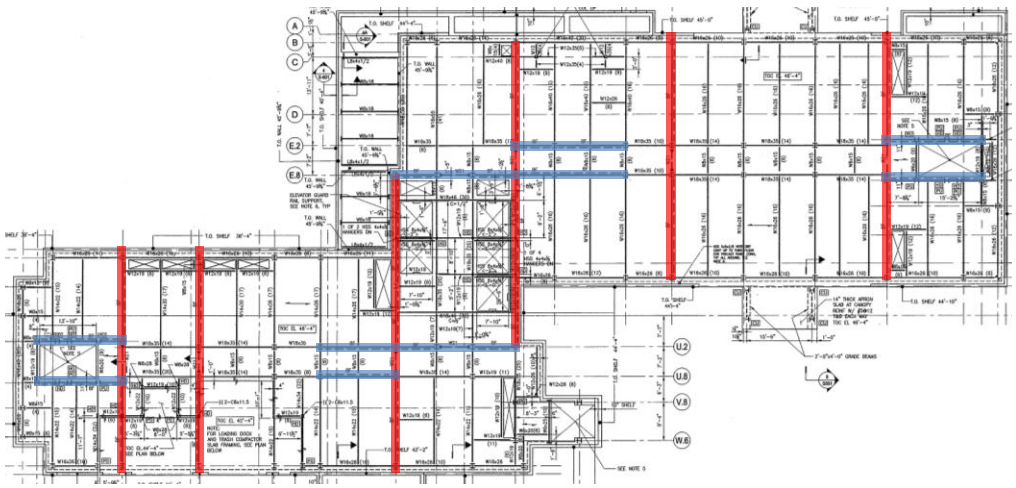


Figure 5: Foundation plan with braced frame locations highlighted

Due to the slender shape of the building in the short direction, the braced frames in this direction (highlighted in red) have wider bases than the braced frames in the longer direction (shown in blue). The wider base provides a more effective geometry for transferring lateral loads to the foundation in the form of vertical loads.

Figure 6: Connection construction photo

Some of the braced frames in perpendicular directions utilize the same columns making for very complicated connection details and erection processes. To successfully portray these connections, 3 dimensional models had to be built, presented and given to the contractors. Because of this, the design phase of the schedule had to be extended and more risk was taken by the structural engineer that designed the connections. A construction photo of these connections is shown in figure 6.

Figure 7 shows one of the further issues encountered due to the connections of the braced frames. Where the columns terminate, some of the foundation had to be cut away to allow for the columns to be placed due to the large connections for the diagonal bracing members. A last minute adjustment of this type is both unnecessary and disruptive. This issue also pushed the steel erection schedule and caused delays in the overall construction schedule.

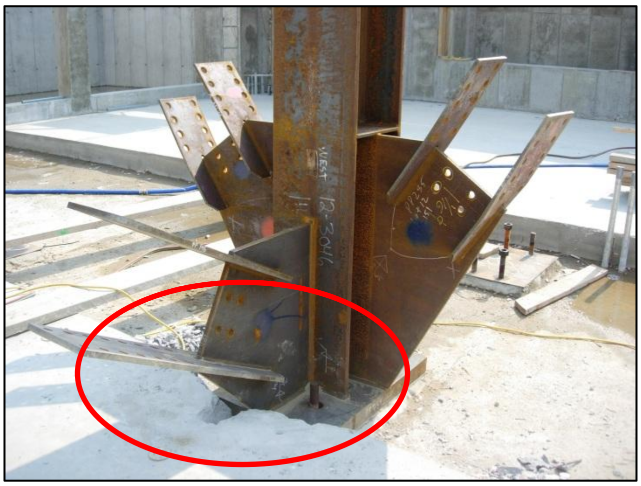


Figure 7:Foundation braced frame connection issues

Design Codes & Standards

Original Design	Thesis Design
Massachusetts Building Code 6th Edition	2009 International Building Code
1993 BOCA National Building Code	American Society of Civil Engineers (ASCE7-10)
American Institute of Steel Construction (2005 Manual)	2005 AISC Steel Manual

Table 1: Design codes vs. Thesis codes

Structural Materials

The materials listed in the chart below are specified in the structural drawings via the General Notes page of the structural drawings (S000) or general notes on the individual framing plans.

Material Properties		
Material		Strength
Steel	**Grade**	**fy = ksi**
Structural Shapes	A992	50
Plates	A36	36
Angles	A36	36
Structural Tubes	A500, B	46
Structural Pipes	A53, B or A501	30
Column Base Plates	A572, 50	50
Concrete	**Weight (lb/ft^3)**	**f'_c = psi**
Mat Foundation	145	4000
Slabs (Dock & Trash)	145	4000
Walls	145	4000
Typ. Slabs	115	3000
Reinforcing Steel		**fy = 60 ksi**
Welding Electrodes	E70 XX	**70 ksi**

Table 2: Material properties

Building Loads

In the tables that follow, the dead and live loads that were used by the designers and that were used for this thesis are listed. The dead loads were looked up in literature, assumed or calculated depending on the type of material they consist of; while the live loads were designated as specified by the codes listed in the tables.

Dead Load

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

Table 3: Dead loads

Live Load

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 4: Live loads

Floor System Analysis

Comparisons were made between the existing floor system and three other alternative systems. Hand calculations combined with computer modeling and reasonable assumptions led to the preliminary design of the alternative systems as well as spot checks of the existing floor system. Listed below are the four floor systems analyzed in this report:

- One-way composite concrete slab
- One-way precast planks on steel framing
- One-way precast planks on staggered trusses
- Two-way flat plate with one-way post tensioning

Costs for the evaluated systems were calculated using RS Means: Square Foot Costs 2010 with the location factor for Boston being 1.17. Appendix G shows the numbers and calculations used for this assignment. Prices for Post-Tensioning and steel trusses were not found in RS Means. Prices for these elements were either estimated or found through a different source.

Existing One-Way Composite Concrete Slab

As part of Tech 1, the existing floor construction was analyzed and evaluated using spot checks of typical framing members. Figure 8 shows the typical detail specified by the structural engineer for the composite deck. Columns F-12, F-13, J-12 and F-13 make up the corners of the bay on floor 5 that was used for these spot checks. Complete hand calculated spot checks can be found in appendix A.

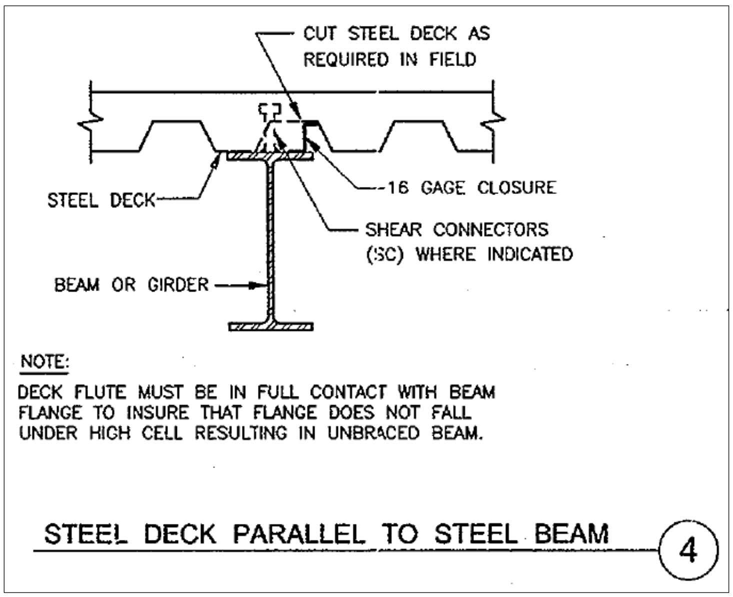


Figure 8: Typical Composite Deck Detail

Decking

The typical floor construction of Res Tower II utilizes a 3" 18 gage steel deck with 3 ¼" light weight concrete. Using the Vulcraft Steel deck catalog, deck type 3VLI18 matches these characteristics. A 3VLI18 works for the unshored length and has almost 4 times the required strength to support the required load. This extra strength was due to the 2 hour fire rating requirement; a slab of light weight concrete must be 3 ¼" thick to receive a 2 hour rating. Hand calculations for decking can be found in appendix A.1.

Beam & Girder

Strength and deflection checks for both the construction and post-construction phases were performed on a typical beam and girder. It appears that the members are slightly over designed but the repetitive nature of the design may be the reason. Also, using repetitive members may have been an emphasis for the original design. Repeating member sizes can lead to using members that have more strength than required in certain locations. This extra strength may also have been designed to allow for variation of use; such that areas could be utilized differently over time and still have sufficient strength. Hand calculations for a typical beam and girder can be found in appendices A.2 and A.3 respectively.

Advantages:

Designing a composite deck exploits the strengths of the materials and allows them to work to their best ability. If designed accordingly, the concrete would be in complete compression while the steel member would be in complete tension and thus creating a very efficient system. By using lightweight concrete as opposed to normal weight concrete, a lighter structure can be considered for strength because there would be less load overall. Lightening the overall load would also positively affect a typical foundation. Large amounts of formwork are not necessary because the concrete can be placed directly on the metal decking. Also depending on the 3 or more unshored span limit, shoring may not be necessary. In the case of Res Tower II, shoring is typically not necessary.

Disadvantages:

Fire proofing of some kind is necessary on the underside of the slab and on the beams and girders because they have exposed steel. This not only drives up the cost of construction but creates an unattractive ceiling that needs to be covered or finished which causes the cost to increase. Shear connectors (shear studs for Res Tower II) are also required for this system to work as it is designed. Making sure that these connectors are placed correctly and effectively can also add to cost through material costs and field inspections. Although the slab and deck combination may not be very deep, some girders can become quite deep and make coordination with the other design disciplines difficult.

One-Way Precast Planks on Steel Framing

Two systems using prestressed hollow core concrete planks were evaluated for this technical assignment. One system supports these planks using a typical steel framing plan and the other utilizes a staggered truss system which is discussed in more detail in a later section.

A preliminary panel size of 6" x 4' (depth x width) with a span of 18' utilizing (4) ½" diameter strands has adequate strength to support the required loads according to the Nitterhouse Concrete specifications for precast hollow core planks; see Appendix B for the calculations that led to this decision. Table 5 provides the maximum service loads specified by Nitterhouse, figure 9 gives the dimensions of the panel selected for Res Tower II and appendix C contains the complete specification.

SAFE SUPERIMPOSED SERVICE LOADS		IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)																		
Strand Pattern		SPAN (FEET)																		
		12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
4 - 1/2"ø	LOAD (PSF)	349	317	290	258	227	197	174	149	127	108	92	78	66	55					
6- 1/2"ø	LOAD (PSF)	524	478	437	377	334	292	269	237	215	188	165	142	122	104	88	73	61	49	39
7 - 1/2"ø	LOAD (PSF)	541	492	451	416	364	331	293	274	242	214	190	167	144	124	107	91	77	64	53

Table 5: Maximum Service Loads for Precast Panels

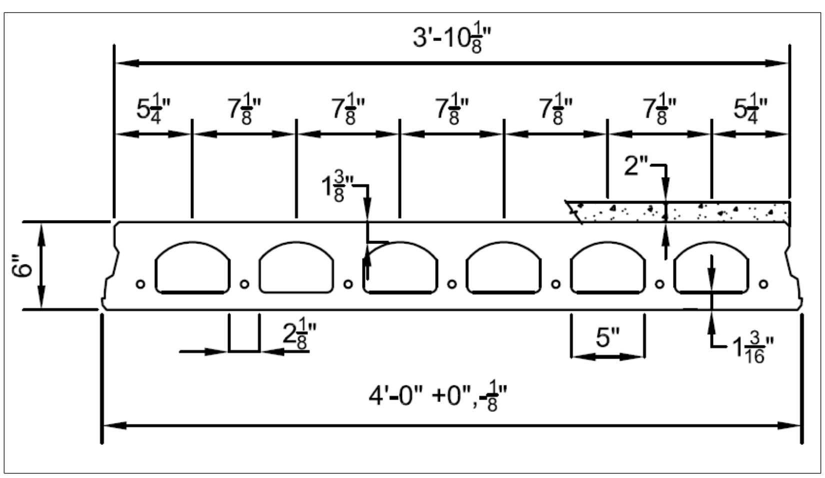


Figure 9: Dimensions of Precast Panel

Adjustments were made to the layout of the columns to make this system work. The exterior spans had to be changed from 23’-7” to 24’ to match the modular precast panels. Making this change decreases the width of the corridor from 10’ to 9’-6” which still exceeds the required width. A girder spanning the 24’ mentioned above was designed as a simply supported beam using the required imposed loads in addition to the self-weight of the panels specified by Nitterhouse. A W12x53 meets all the strength requirements as well as total and live load deflections. Because the planks are not cast in place, no calculations were done using wet concrete or bare beam deflections. Appendix D has the hand calculations and checks for this girder.

Advantages:

By eliminating the need for cast in place concrete, the construction time would decrease because there would be no need to schedule time for curing or concrete finishing. Also, no fireproofing is needed for the underside of the slab and the ceiling finishes can be applied directly to the underside of the panels. No shoring is required to support the planks; therefore construction can be continued near and above these floors allowing the construction schedule to decrease accordingly.

Disadvantages:

Although fire proofing is not necessary for the panels, it is still necessary for the beams and girders supporting these panels. Vibration may be an issue for this system because of all the light weight members that are involved in it. Although the hollow core members require normal weight concrete, the voids make them very light. Supporting these light weight members could be very light framing. This featherweight structure is great for typical structures but for a high rise building, the overturning moment from lateral wind forces would cause uplift forces that wouldn’t be balanced with the compression force of a heavy building. More investigation into the lateral forces would need to be done in order to use this system.

One-Way Precast Planks on Staggered Truss

A staggered truss system utilizes a story deep Vierendeel truss that replaces the need for interior columns by spanning from exterior column to exterior column. Res Tower II has the prescriptive layout for the use of a staggered truss system because it has long outer spans that support private areas and an interior corridor for resident circulation. This is a perfect match to the staggered truss system using a Vierendeel truss because the vertical web members in the center allow space for the corridor while the private spaces of the layout allow for diagonal members towards the ends of the truss. Figure 10 shows the geometry and preliminary member sizes of the Vierendeel truss. Appendix E shows the hand work done to set up the truss model using SAP2000. The corresponding web member sizes are as follows:

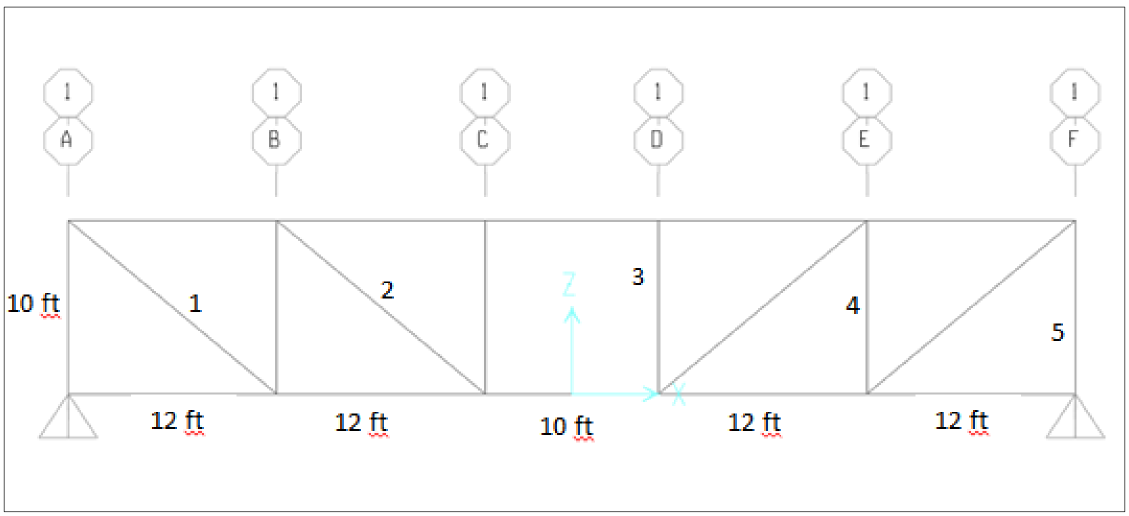


Figure 10: Staggered Truss Member Layout

1.	W8 x 40		ϕP_n = 428 k > P_u = 420.4 k Tension
2.	W8 x 18		ϕP_n = 192 k > P_u = 182.2 k Tension
3.	W8 x 31	Unbraced Length= 10ft	ϕP_n = 317 k > P_u = 58.8 k Compression
4.	W8 x 31	Unbraced Length= 10ft	ϕP_n = 317 k > P_u = 192.7 k Compression
5.	W8 x 31	Unbraced Length= 10ft	ϕP_n = 317 k > P_u = 297.0 k Compression

A line of symmetry exists in the middle of the truss where the origin is located in figure 10 and therefore the mirrored members have the same qualities as listed above. Sizes listed above are strictely preliminary; design for this truss would need to be coordinated with the truss designer, see Considerations. Web members do not need to be W shapes if the fabricator decides on a different shape for constructibility purposes.

Due to the distributed load on the top and bottom continuous truss members from the precast planks, these members will have shear and bending forces as well as axial forces. Force diagrams for the top member are presented in figures 11 and 12 with figure 11 showing the free body and axial diagrams and figure 12 showing shear and moment diagrams. The bottom member forces are diagramed in the same layout using figures 13 and 14. Maximum values of each force and the locations from the left end of the member are given on the right side of the figure. A closer look at the design and interaction of these members is necessary to decide on the best member size.

<u>Top Member</u>

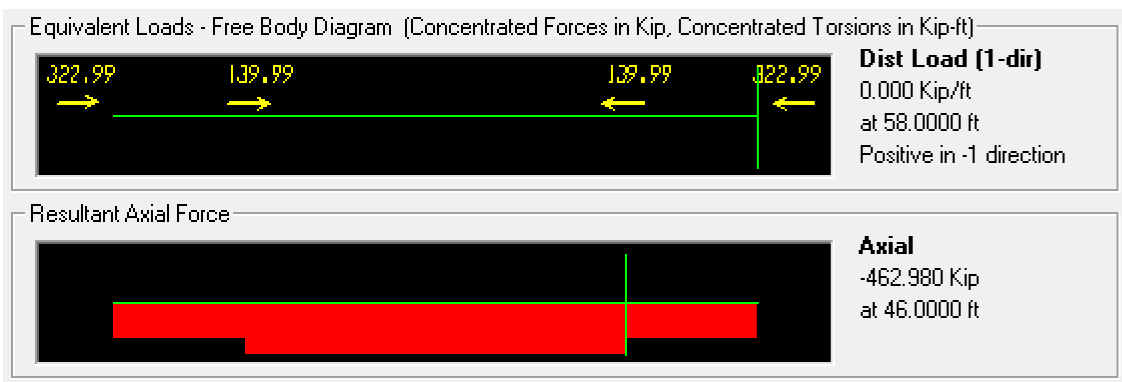


Figure 11: Free Body and Axial Diagrams for Top Member

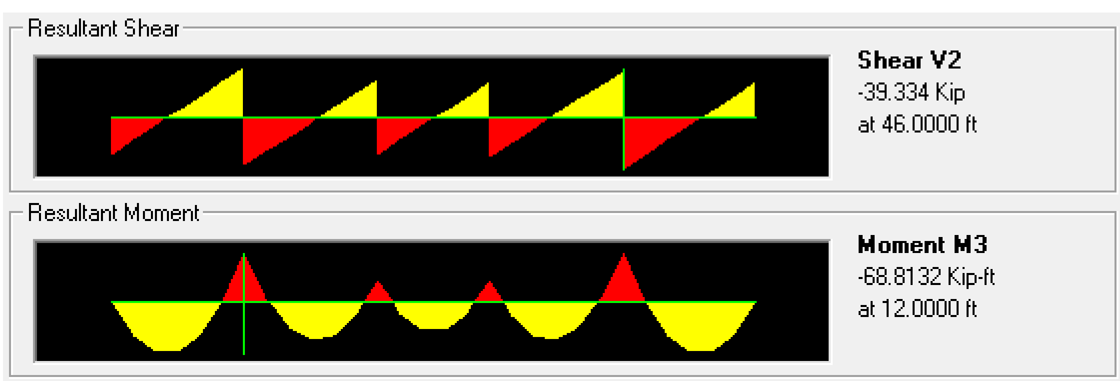


Figure 12: Shear and Moment Diagrams for Top Member

Bottom Member

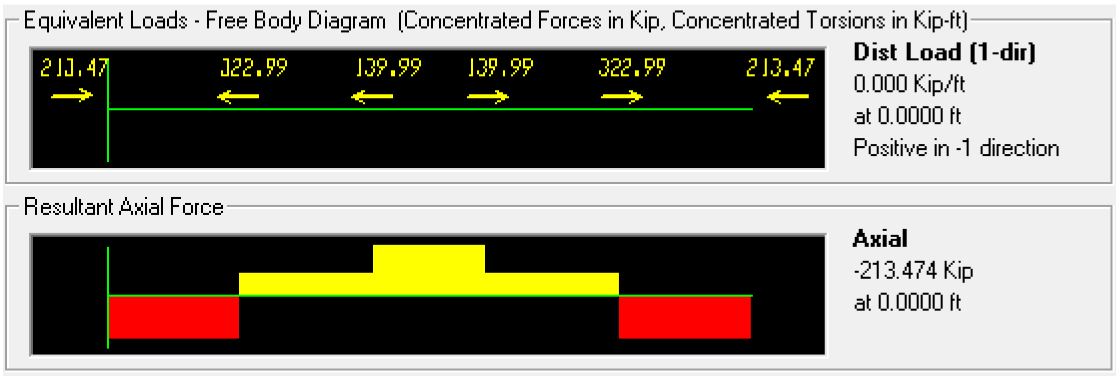


Figure 13: Free Body and Axial Diagrams for Bottom Member

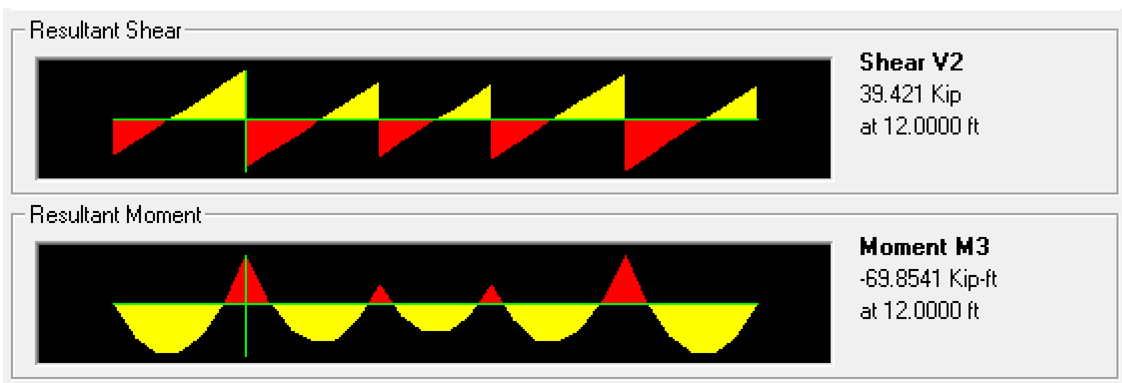


Figure 14: Shear and Moment Diagrams for Bottom Member

It can be seen from the axial diagram for the bottom member that the middle section is in compression but when the member meets the support the forces switch to compression. Further examination into the design of the top and bottom members needs to be done if this system is to be employed in the future.

Advantages:

Using a staggered truss system provides many advantages. Eliminating the need for interior columns greatly improves layout flexibility and allows for large, uninterrupted lobbies and open spaces at the base of the building. Faster erection and a cleaner site is made possible because the trusses are fabricated then brought to the site. One advantage, noted by Aine Brazil in the September 2000 issue of Modern Steel Construction, is the all-dry system speeds up winter construction. This plays an important role in the construction schedule for Res Tower II because during the winter temperatures in Boston can be below freezing for the majority of the season and admixtures may have been added to the slab concrete to decrease the amount of water in the slab and the necessary curing time in low temperatures.

Combining the prefabricated trusses with prefabricated hollow concrete planks provides additional advantages. With the combination of these two elements, the construction process is much quicker than assembling a composite deck with a standard steel frame. Once a plank is in place, no shoring is required to continue construction above that level. Because the planks have voids in them, they greatly reduce the weight of the slabs when compared to composite deck. These planks also reduce the amount of sound and heat transmission.

Disadvantages:

Unfortunately, a few disadvantages come along with the use of staggered trusses. A lead time would have to be planned for in the construction schedule to allow for prefabrication of the trusses. The diagonal web members of the truss limit the locations of corridors and circulation space, both vertical and horizontal. An obvious hindrance is placed on exterior window layouts due to the diagonal members and connections to the exterior columns at corners. Differential camber is an issue when designing with precast planks; as well as curved or angled edges.

Considerations:

To take full advantage of the potential for fast construction for this system close cooperation and coordination is necessary between all project teams. The structural engineer and the fabricator must work closely to design repetitive members to maximize the economy of this system.

Using the precast planks with staggered trusses would allow for an adjustment to the floor to floor height of Res Tower II if desired. To allow for a 2 hour fire rating, 2” of topping concrete must be added to the planks. Combining the 2” of concrete with the 6” plank, the ceiling to floor height is only 8”. Smooth finished or “carpet-ready” (*Faraone*) planks can be purchased to suit the need of the client and further increase construction ease towards the end of the process.

Changes would need to be made to the exterior skin and façade of Res Tower II but the scale of these changes could be minimal depending on decisions made by the client and architect. A choice between exposing the structure and hanging the façade from the trusses will need to be further considered for this system if it is to be pursued.

A cost analysis for this system is difficult to perform at this stage of the design because the combination loaded members are yet to be designed. A cost has been associated with the precast planks and an additional allowance will be made for the trusses.

Two-Way Flat Plate with One-Way Post-Tensioning

Post-tensioning allows greater cracking and deflection control; it allows thinner slabs and longer spans. Normal slab reinforcement is required in a post-tensioned system because the PT tendons are either sheathed or greased to prevent concrete bonding to the strands. Tendons are distributed according to a layout profile that is dictated by the locations of positive and negative moments in the slab. Post-tensioned tendons need to be in the tension face of the concrete to impose compression and control cracking.

Using the calculations shown in Appendix F, a preliminary slab depth of 7" using lightweight concrete was determined to be sufficient for the required loads of Res Tower II. Calculations were only done for one direction of the span due to the preliminary nature of the design. Ten ½" 7-wire PT strands with a jacking force of 266 kips is all that is needed in one bay with a width of 20 ft. The strands are placed according to the tendon profile shown in red in figure 15 which is not drawn to scale. Strands are placed above the neutral axis at mid-span of the interior span because the shorter span length causes a negative moment to still exist.

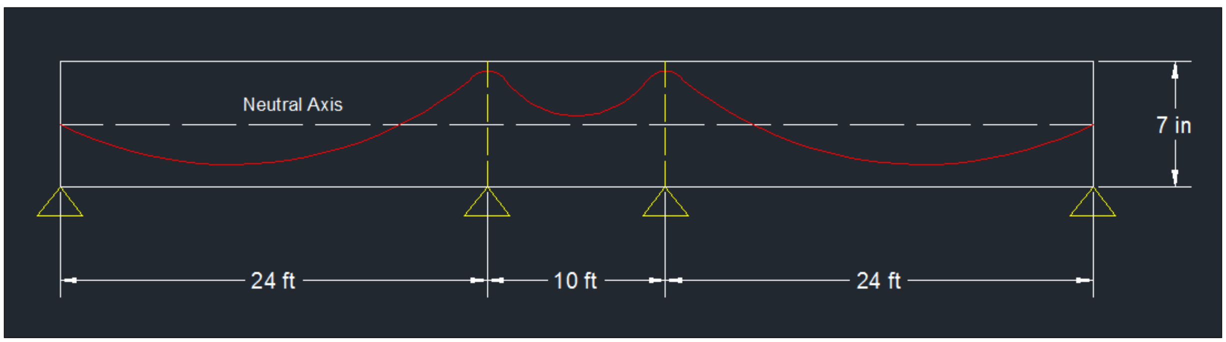


Figure 15: Post-Tension Strand Profile

Normal, bonded reinforcement is still necessary in a post-tensioned slab because the PT tendons are unbonded to the concrete. All bonded reinforcement was chosen to be #5's spaced at 12" O.C. to make the construction process more repetitive and less complicated. The appropriate number bars are given on the last page of the calculations in Appendix F.

Advantages:

Post-tensioning allows for an overall slab thickness of only 7". Combining a thin slab with lightweight concrete creates an extremely light floor system. Very simple formwork is needed to construct a flat plate system because no drop panels are required. Because no drop panels are required the result is a uniform, flat ceiling that already has a 2 hour fire rating. This makes finishes for the ceiling very fast and inexpensive.

Disadvantages:

Although the formwork is simple and reusable, it is still needed unlike the precast or composite systems described above. Anchoring devices and grouting equipment is required to tighten the post-tension tendons which will add to the cost and lead time of the project. As discussed above, curing time in the cold winters of Boston can prove to be issues that need to be planned for either by effective scheduling or adding appropriate admixtures to the concrete. Some issues that are associated with flat plate systems are deflection control, punching shear and future slab cutting. Deflection control and punching shear can be taken care of with careful design but future slab cutting can prove to be troublesome due to the flat plate and the PT tendons.

Considerations:

After designing and inspecting the flat plate system that was designed for this technical report, new considerations and design principles will be adapted to future use of this system. A decision will need to be made between using a flat plate system with two-way post-tensioning and a one-way slab using post-tensioned girders. The one-way system with PT girders seems to be the most reasonable design to use due to the geometry of Res Tower II. A minimum column size of 22” x 22” was used for this design but due to the decision to switch from this system, no other calculations were done for column sizing.

Floor System Summary

	Existing	Alternatives		
	One-way Composite	Precast Planks with Steel Framing	Precast Planks with Staggered Trusses	Two-way Flat Plate with One-way PT
Architectural Alteration (Bay Size)	NO	YES	YES	YES
Architectural Alteration (Facade)	NO	NO	YES	POSSIBLE
Lateral System Alterations	NO	YES	YES	YES
Slab Depth	6 1/4"	6"	6"	7"
System Cost (per square foot)	18.84	30.59	15.20 + Truss Costs	16.50 + an estimated 16.00 for PT = 32.50
Added Fire Protection (slab)	YES	NO	NO	NO
Added Fire Protection (other members in system)	YES	YES	YES	NO
Formwork	Minimal	NO	No	YES
Constructability	Moderate	Easy	Moderate	Difficult
Lead Time	Medium	Medium	Long	Short
Vaible Option	YES	YES	YES	NO

Table 6: Overall System Comparisons

Foundation:

Because the foundation for Res Tower II is a mat foundation, it is hard to say how each system will affect the foundation design. The foundation was designed to fight the uplift forces caused by lateral forces and hold down sections of the building. It is incorrect to say that the lighter the building the better because the foundation relies on the weight of the building to counteract some of the uplift forces. It is also incorrect to say that the heavier the building the better because a heavy building might cause the foundation system to change completely not just moderate adjustments to the existing system. Due to this complication, the foundation is associated with the lateral system and will need to be evaluated as part of tech 3.

Conclusion

As a result of this study, it has been determined whether or not the alternative systems are feasible for Res Tower II. By designing these systems using the existing loading conditions and assessing them with structural and non-structural criteria, the alternative systems can be directly compared with the existing floor construction.

Both the typical steel framing system and steel truss system proved to be viable alternatives for Res Tower II. The typical steel framing supporting precast panels would have a minimal effect on the overall appearance of the building where as the truss system supporting precast panels could have a great effect on the appearance. In order to take full advantage of the precast nature of these two systems, most of the bay dimensions would need to be changed to multiples of 4 ft. For Res Tower II, a change like this could be very inconvenient due to its highly restricted footprint. The cost of these two systems is much higher than the existing system but the time of construction would be much shorter because the precast panels do not require curing time as the composite slab does. To further investigate the feasibility of these two options, especially the truss system, a lateral evaluation will need to be done as part of tech 3.

Due to inappropriate design decisions and assumptions, the flat plate system had to be deemed unfeasible. If this system is changed to a one-way slab with post-tensioned girders, it would be extremely viable. Using the flat plate post-tensioned system would require changing the entire structure of Res Tower II to concrete which could potentially be a thesis proposal depending on the research and outcome of technical report 3.

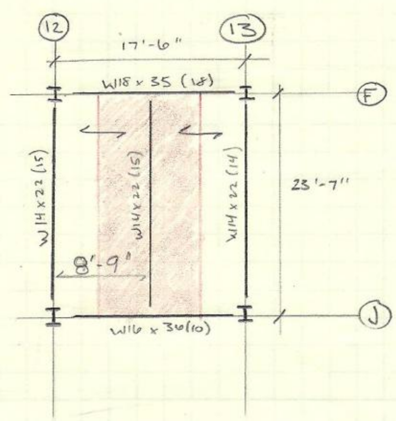
Appendix A: Existing Floor Calculations

A.1: Decking Check

Tyler Meek | AE Senior Thesis | Decking Spot Checks | 1

Spot checks performed on typical bay of typical floor

Floor 5



Floor Construction:

3" - 18g Steel Deck

$3\frac{1}{4}$" LWT Concrete

Total $t = 6\frac{1}{4}$"

$f'_c = 3000$ psi

Loads:

LL = 40 psf

SDL = 30 psf

Total = 70 psf

Vulcraft Decking Catalog

3VLI18: Check unshored length (3 span)

15' > 8'-9" ∴ ok for unshored length

Check superimposed live load

9'-0" clear span:

275 psf >> 70 psf ∴ ok for loading

The loading check shows that this floor construction is very over designed but this is not the controlling load factor

Fire Resistance:

2HR rating ⇒ $3\frac{1}{4}$" LWT Concrete

2HR rating ok

$\boxed{\text{Deck 3VLI18 checks out}}$

A.2: Beam Check

TYLER MEEK | AE SENIOR THESIS | BEAM SPOT CHECKS | 2

COMPOSITE BEAM: W14 x 22 (15)

TRIBUTARY WIDTH = 8'-9"
SPAN = 23'-7"

W14 x 22:
$A_g = 6.49 \text{ in}^2$
$I_x = 199 \text{ in}^4$
$F_y = 50$ ksi

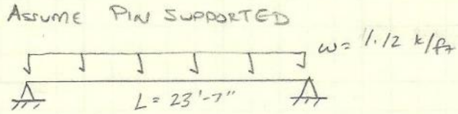
LL = 40 psf (DO NOT REDUCE)
SDL = 30 psf
DL = 47 psf (DECKING CATALOG + ALLOWANCE FOR REINFORCEMENT)
SELF WEIGHT = 22 plf

$w_u = 1.2D + 1.6L$

$D = (30 + 47)(8.75) + 22 = 463.25$ plf
$L = 40(8.75) = 350$ plf

$w_u = 1.2(463.25) + 1.6(350)$
$w_u = 1.12$ k/ft

ASSUME PIN SUPPORTED



$V_u = \frac{(1.12)(23.583)}{2} \Rightarrow V_u = 13.2^k$

$M_u = \frac{wl^2}{8} = \frac{(1.12)(23.583)^2}{8}$
$M_u = 77.86$ ft-k

b_{eff}: (EQUAL ON BOTH SIDES)

$b_{eff} = \min \begin{cases} \text{SPAN}/4 = 5.896 \leftarrow \text{CONTROLS} \\ \text{SPACING} = 8.75 \end{cases}$

(TABLE 3-19) STEEL MANUAL

W14 x 22, PNA = 7 $\Rightarrow \Sigma Q_n = 81.2$

$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{81.2}{0.85(3)(5.896)(12)} \Rightarrow a = 0.45 < 1.0$

USE $a = 1.0$

$Y_2 = t_{SLAB} - \frac{a}{2} = 6.25 - \frac{1}{2} = 5.75$

$\phi M_n = 190.5$ ft-k $> M_u = 77.86 \therefore$ OK

$Q_n = \frac{81.2}{17.2} = 4.72 \sim 5 \therefore 10$ STUDS REQUIRED < 15 STUDS USED $\therefore$ OK

(TABLE 3-2) $\phi V_n = 94.8^k > V_u = 13.2^k \therefore$ OK

TYLER MEEK | AE SENIOR THESIS | BEAM SPOT CHECKS | 3

CHECK Δ_{LL}:

$$\Delta_{LL} = \frac{l}{360} = \frac{(23.583)(12)}{360} = 0.7861 \text{ in}$$ MAXIMUM DEFLECTION

(TABLE 3-20) $I_{LB} = 406 \text{ in}^4$ $\quad w_{LL} = (40 \text{ psf})(8.75)/1000$

$$\Delta_{LL} = \frac{5 w_{LL} l^4}{384 E I_{LB}} = \frac{(5)(0.35)(23.583)^4(1728)}{384(29000)(406)}$$

$$\Delta_{LL} = 0.207 \text{ in} < \Delta_{LL\,max} = 0.7861 \quad \therefore \text{ ok}$$

CHECK BEAM DEFLECTIONS UNDER WET CONCRETE

$$\Delta_{max} = \frac{l}{240} = \frac{(23.583)(12)}{240} = 1.18 \text{ in}$$

$$I_{REQ} = \frac{5 w l^4}{384 E \Delta_{max}} \qquad w = \left[(47)(8.75) + 22\right]/1000$$

$$w = 0.414$$

$$I_{REQ} = \frac{(5)(0.414)(23.583)^4(1728)}{384(29000)(1.18)}$$

$$I_{REQ} = 84.20 \text{ in}^4 < I_{BB} = 199 \text{ in}^4 \quad \therefore \text{ ok}$$

$$\boxed{\text{W14 x 22 WORKS}}$$

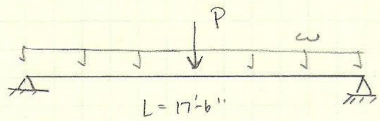
A.3: Girder Check

TYLER MEEK | AE SENIOR THESIS | GIRDER SPOT CHECKS | 4

$I_x = 510\ in^4$

COMPOSITE GIRDER: W18 x 35 (18)

ASSUMING PIN IS CONSERVATIVE FOR BEAM DESIGN



$P_u = 2(13.2^k) = 26.2^k$

$w_u = 35/1000 = 0.035\ k/ft$

$V_u = 26.2/2 + \frac{(0.035)(17.5)}{2} \Rightarrow V_u = 13.41^k$

$M_u = \frac{(0.035)(17.5)^2}{8} + \frac{(26.2)(17.5)}{4}$

$M_u = 116\ ft\text{-}k$

(TABLE 3-19) PNA=7

$\Sigma Q_n = 129$ $\quad a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{129}{(0.85)(3)(4.375)(12)}$

$b_{eff} = \min \begin{cases} SPAN/4 = 4.375 \leftarrow \text{CONTROLS} \\ SPACING = 23 \end{cases}$

$a = 0.96 < 1.0$ USE $a = 1.0$

$Y_2 = t_{SLAB} - \frac{a}{2} = 6.25 - \frac{1}{2} = 5.75$

(TABLE 3-19)

$\phi M_n = 369.5\ ft\text{-}k > M_u = 116\ ft\text{-}k$ ∴ OK

$\phi V_n = 159^k \gg V_u = 13.41^k$ ∴ OK

STRENGTH GOOD

DEFLECTION

Δ_{LL}: $P_L = \frac{(350)(23.583)}{1000} = 8.25^k$

$\Delta_{LL} \le \frac{l}{360} = \frac{(17.5)(12)}{360} = 0.5833\ in$

(TABLE 3-20) $I_{LB} = 950\ in^4$

$\Delta_{LL} = \frac{Pl^3}{48EI} + \frac{5wl^4}{384EI} = \frac{(8.25)(17.5)^3(1728)}{48(29000)(950)} + \frac{5(0.035)(17.5)^4(1728)}{384(29000)(950)}$

$= 0.06\ in$

$\Delta_{LL} < 0.5833\ in$ ∴ OK

W/ CONCRETE:

$\Delta_{MAX} = \frac{l}{240} = 0.875\ in$ $\quad P = 9.76^k$

$I_{REQ} = \frac{Pl^3}{48E\Delta} + \frac{5wl^4}{384E\Delta} = \frac{(9.76)(17.5)^3(1728)}{48(29000)(.875)} + \frac{5(0.035)(17.5)^4(1728)}{384(29000)(0.875)}$

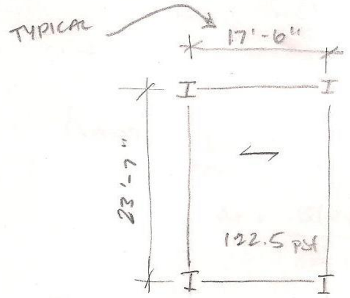
$I_{REQ} = 77.12\ in^4 < 510\ in^4$ ∴ OK

$\boxed{\text{W18 x 35 WORKS}}$

Appendix B: Hollow core plank calculations

HOLLOW CORE PLANK:

6" x 4' HOLLOW CORE (2" TOPPING) 2 HR FIRE RATING



SDL = 30 psf

DL = 48.75 psf (FROM NITTERHOUSE)

LL = 40 psf

SELF WEIGHT INCLUDED IN CHARTS

$w_u = 1.2D + 1.6L = 100$ psf

USING DESIGN SPAN OF 18 ft, USE

$\boxed{\text{6"x4' HOLLOW CORE PLANK W/ 2" TOPPING \& 4 1/2" ⌀ STRANDS}}$ ⇒ SUPPORTS 174 psf > w_u

174 psf > 100 psf ∴ OK TO USE

PROBABLY NOT VIABLE DUE TO 4 ft WIDTH OF PLANKS, BAY SIZE WOULD CHANGE → DECREASE HALLWAY BE 3" EACH SIDE

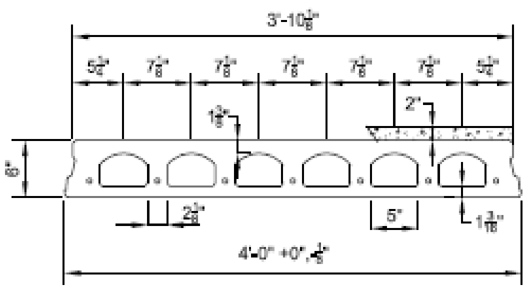
Appendix C Nitterhouse precast plank specification

Prestressed Concrete 6"x4'-0" Hollow Core Plank

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES Composite Section	
$A_c = 253\ in.^2$	Precast $b_w = 16.13$ in.
$I_c = 1519\ in.^4$	Precast $S_{bcp} = 370\ in.^3$
$Y_{bcp} = 4.10$ in.	Topping $S_{tt} = 551\ in.^3$
$Y_{tcp} = 1.90$ in.	Precast $S_{tcp} = 799\ in.^3$
$Y_{tt} = 3.90$ in.	Precast Wt. = 195 PLF
	Precast Wt. = 48.75 PSF

DESIGN DATA



1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
   4-1/2"Ø, 270K = 67.4 k-ft at 60% jacking force
   6-1/2"Ø, 270K = 92.6 k-ft at 60% jacking force
   7-1/2"Ø, 270K = 95.3 k-ft at 60% jacking force
7. Maximum bottom tensile stress is $10\sqrt{f'c} = 775$ PSI
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. Topping Strength @ 28 days = 3000 PSI. Topping Weight = 25 PSF.
12. These tables are based upon the topping having a uniform 2" thickness over the entire span. A lesser thickness might occur if camber is not taken into account during design, thus reducing the load capacity.
13. Load values to the left of the solid line are controlled by ultimate shear strength.
14. Load values to the right are controlled by ultimate flexural strength or fire endurance limits.
15. Load values may be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
16. Camber is inherent in all prestressed hollow core slabs and is a function of the amount of eccentric prestressing force needed to carry the superimposed design loads along with a number of other variables. Because prediction of camber is based on empirical formulas it is at best an estimate, with the actual camber usually higher than calculated values.

SAFE SUPERIMPOSED SERVICE LOADS — IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)

Strand Pattern		SPAN (FEET) 12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
4 - 1/2"ø	LOAD (PSF)	349	317	290	258	227	197	174	149	127	108	92	78	66	55					
6- 1/2"ø	LOAD (PSF)	524	478	437	377	334	292	269	237	215	188	165	142	122	104	88	73	61	49	39
7 - 1/2"ø	LOAD (PSF)	541	492	451	416	364	331	293	274	242	214	190	167	144	124	107	91	77	64	53

This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths. The allowable loads shown in this table reflect a 2 Hour & 0 Minute fire resistance rating.

NITTERHOUSE CONCRETE PRODUCTS
2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17202-9203
717-267-4505 Fax 717-267-4518

11/03/08

6F2.0T

Appendix D: Steel framing calculations

HOLLOW CORE PLANK W/ STEEL BEAMS:

DESIGN GIRDER TO SUPPORT PLANKS: TRIB WIDTH = 17'-6"

SDL = 30 psf

DL = 48.75 psf

DO NOT REDUCE TO ALLOW FOR DIRECT COMPARISON → LL = 40 psf

$w_{u_s} = 1.2D + 1.6L = 158.5$ psf

$w_{u_\ell} = 2.8$ k/ft

2.8 k/ft

24'

$V_u = 33.6$ k

$M_u = \frac{w\ell^2}{8} = \frac{(2.8)(24)^2}{8} = 201.6$ ft-k

$M_u = 201.6$ ft-k

TABLE 3-10 (STEEL MANUAL): BEAM CURVES

USE UNBRACED LENGTH = 24 ft

USE W12x53 ⇒ $\phi M_n = 208.5$ ft-k

DEFLECTION LIMITS: W12x53 : $I_x = 425$ in⁴

$\Delta_{LL} \le \frac{\ell}{360} = 0.8$ in

$\Delta_{LL} = \frac{5w\ell^4}{384EI} = \frac{5(0.04 \times 17.5)(20)^4(1728)}{384(29000)(425)} = 0.2 < \frac{\ell}{360}$ ∴ $\underline{OK}$

$\Delta_{TOTAL} \le \frac{\ell}{360}$

$\Delta_T = \frac{5(2.8)(20)^4(1728)}{384(29000)(425)} = 0.82 > \frac{\ell}{360}$ ∴ NOT GOOD

TRY W12x58

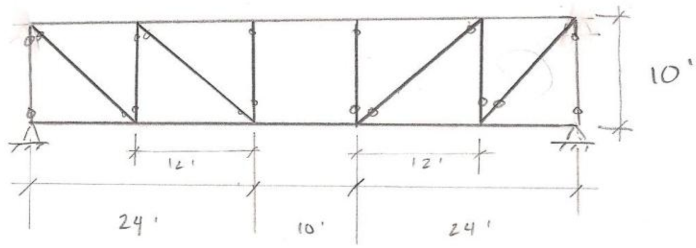
$I_x = 475$ in⁴

$\Delta_T = \frac{5w\ell^4}{384EI}$ $\Delta_T = 0.732 < \frac{\ell}{360}$ ∴ $\underline{OK}$

$\boxed{\text{USE W12x58 FOR GIRDER}}$

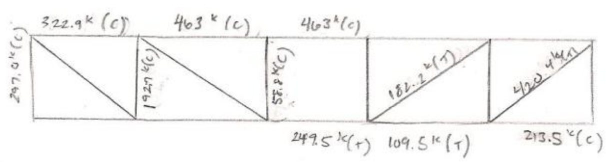
Appendix E: Staggered truss calculations

STAGGARED TRUSS (ON PRECAST PLANKS)



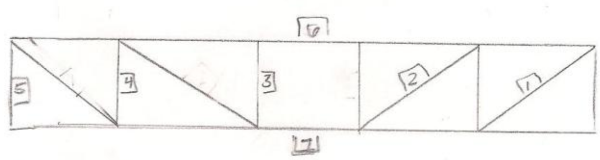
W_u = 5.6 k/ft ON TOP & BOTTOM CHORDS

MODELED IN SAP 2000



AXIAL

	TOP CHORD	BOTTOM CHORD
MAX V	39.93k	40.46k
MAX M	75.98 ft-k	82.33 ft-k



1 W8×40 ϕP_n = 428k > 420.4k (T)

2 W8×18 ϕP_n = 192k > 182.2k (T)

3 UNBRACED LENGTH = 10 ft W8×31 ϕP_n = 317k > 58.8k (C)

4 " " W8×31 ϕP_n = 317k > 192.7k (C)

5 " " W8×31 ϕP_n = 317k > 297.0k (C)

Appendix F: Post-tensioned slab calculations

1/4

POST-TENSIONED ONE WAY SLAB :

PCA ⇒ TIME SAVING DESIGN AIDS

LOADS: FRAMING DEAD LOAD = SELF WEIGHT

SDL = 30 psf

LL = 40 psf

2 HR FIRE RATING

USE LIGHTWEIGHT CONCRETE (120 pcf):

$f'_c = 3000$ psi

ASSUME → $f'_{ci} = 1500$ psi

REBAR : $f_y = 60$ ksi

PT: (ASSUME) UNBONDED TENDONS

1/2" ø 7-WIRE STRANDS , $A = 0.153$ in^2

$f_{pu} = 270$ ksi

ESTIMATED PRESTRESS LOSSES = 15 ksi (ACI)

$f_{se} = 0.7(270) - 15 = 174$ ksi (ACI)

$P_{eff} = A f_{se} = 26.6$ k/TENDON

DETERMINE PRELIMINARY SLAB THICKNESS

START W/ $L/H = 45$

LONGEST SPAN = 24 ft $\quad h = \dfrac{(24\text{ft})(12)}{45} = 6.4$ in PRELIMINARY SLAB THICKNESS

USE 7"

LOADING:

$DL = \left(\dfrac{7\text{ in}}{12}\right)(120\text{ pcf}) = 70$ psf

SDL = 30 psf

LL = 40 psf

EQUIVALENT FRAME METHOD (ACI 13.7)

• CALCULATE SECTION PROPERTIES

$A = bh \quad b = 20 \text{ ft} \times 12 = 240 \text{ in}$

$h = 7 \text{ in}$

$A = 1680 \text{ in}^2$

$S = \frac{bh^2}{6} = 1960 \text{ in}^3$

• SET DESIGN PARAMETERS

ALLOWABLE STRESSES : CLASS U → UNCRACKED (ACI 18.4.1)

AT TIME OF JACKING:

$f'_{ci} = 1500 \text{ psi}$

COMPRESSION $= 0.6 f'_{ci} = 900 \text{ psi}$

TENSION $= 3\sqrt{f'_{ci}} = 116.2 \text{ psi}$

AT SERVICE LOADS:

$f'_c = 3000 \text{ psi}$

COMPRESSION $= 0.45 f'_c = 1350 \text{ psi}$

TENSION $= 6\sqrt{f'_c} = 328.6 \text{ psi}$

AVERAGE PRECOMPRESSION LIMITS:

$P/A =$ 125 psi MIN

300 psi MAX

TARGET LOAD BALANCES:

60-80% OF DL (SELFWEIGHT) FOR SLABS

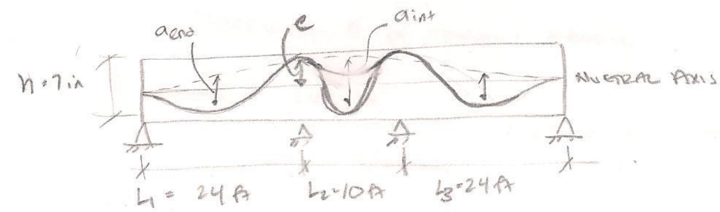
$0.75 w_{DL} = 0.75(100 \text{ psf}) = 75 \text{ psf}$

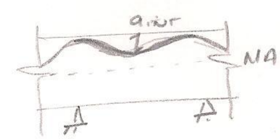
COVER REQUIREMENTS (2 HR RATING)

$\left.\begin{array}{l}\text{RESTRAINED SLABS} = 3/4\text{" BOTTOM} \\ \text{UNRESTRAINED} = 1\tfrac{1}{2}\text{" BOTTOM}\end{array}\right\}$ & 3/4" TOP

TENDON PROFILE

NEW LAYOUT OF INTERIOR SPAN





CONTINUOUS POST-TENSIONED BEAM

TENDON ORDINATE	TENDON LOCATION (CG) ⇒ CENTER OF GRAVITY	
EXTERIOR SUPPORT - ANCHOR	3.5 in	
INTERIOR SUPPORT - TOP	6.0 in	
INTERIOR SPAN - BOTTOM	1.0 in	⟹ NEW VALUE = 5 in
END SPAN - BOTTOM	1.5 in	

$a_{int} = 6.0 - 1.0 = 5.0$ in $\Rightarrow$ NEW VALUE = 1.0 in

$a_{end} = (3.5 + 6)/2 = 4.75$ in

PRESTRESS FORCE REQUIRED TO BALANCE 75% OF SELFWEIGHT DL

$$w_b = 0.75 w_{DL} = 0.75(100 \text{ psf})(18 \text{ ft}) = 1350 \text{ plf} = 1.35 \text{ k/ft}$$

FORCE NEEDED IN TENDONS TO COUNTERACT THE LOAD IN THE END BAY

$$P = \frac{w_b L^2}{8 a_{end}} = \frac{(1.35 \text{ k/ft})(24)^2}{8\left(\frac{4.75}{12}\right)} = 245.6 \text{ k}$$

$$P = 245.6 \text{ k}$$

1/11

CHECK PRECOMPRESSION ALLOWANCE

- DETERMINE # OF TENDONS NEEDED

$$\# \text{ TENDONS} = \frac{245.6^k}{26.6 \ ^k/\text{TENDON}} = 9.23 \Rightarrow 10 \text{ TENDONS}$$

- ACTUAL FORCE FOR BONDED TENDONS

$$P_{ACTUAL} = (26.6)(10) = 266^k$$

- THE BALANCED LOAD FOR THE END SPAN IS SLIGHTLY ADJUSTED

$$w_b = \left(\frac{266}{245.6}\right)(1.35 \ ^k/ft) = 1.46 \ ^k/ft$$

- DETERMINE ACTUAL PRECOMPRESSION STRESS

$$P_{ACTUAL}/A = 266(1000)/1680 \text{ in}^2 = 158.\bar{3} \text{ psi} > 125 \text{ psi Min } \underline{ok}$$

$$< 300 \text{ psi Min } \underline{ok}$$

CHECK INTERIOR SPAN FORCE

$a_{INT} = 5.0$ in

NEW: $a_{INT} = 1.0$

$$P = \frac{(1.35 \ ^k/ft)(10 ft)^2}{8\left(\frac{5.0}{12}\right)} = 40.5^k << 266^k$$

LESS FORCE IS REQUIRED IN CENTER BAY

NEW VALUE: $P = 202.5^k < 266^k \therefore$ SAME APPLIES

CHECK INTERIOR SPAN BALANCE:

$$w_b = (266^k)(7)(5/12)/10^2 = 1.55 \ ^k/ft$$

$$\frac{w_b}{w_{DL}} = \frac{1.55}{1.80} = 0.86 < 1.0 \ \underline{ok}$$

$$\boxed{\text{EFFECTIVE PRESTRESS FORCE} = 266^k}$$

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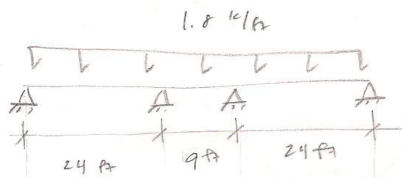
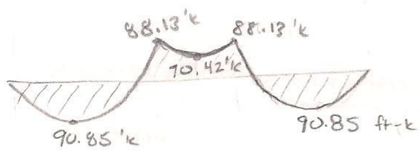
CHECK SLAB STRESSES

DEAD LOAD MOMENTS

$W_{DL} = 1.80$ k/ft



MODELED IN SAP



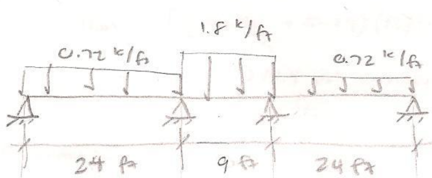
★ BECAUSE MID SPAN OF CENTER SPAN IS NEGATIVE MOMENT, NEED TO PLACE TENDONS ABOVE NEUTRAL AXIS.

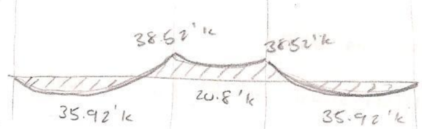
∴ NEW C.G.NT

LIVE LOAD MOMENTS

(DWELLING UNIT) $W_{LL}(END) = (40\ psf)(18\ ft) = 720\ lb/ft = 0.72\ k/ft$

(CORRIDOR) $W_{LL}(INT) = (100\ psf)(18\ ft) = 1.8\ k/ft$

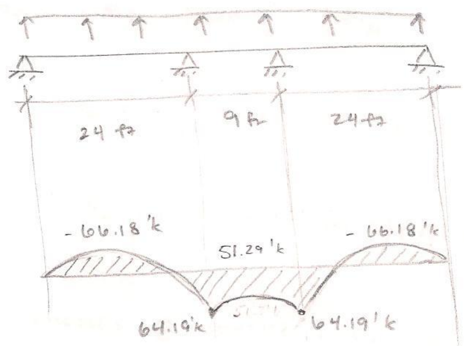




6/11

TOTAL BALANCING MOMENTS

$w_b = 1.35$ k/ft



STAGE 1: STRESSES IMMEDIATELY AFTER JACKING (DL+PT)

MIDSPAN:

$$f_{TOP} = (-M_{DL} + M_{BAL})/S - P/A$$

$$f_{BOT} = (+M_{DL} - M_{BAL})/S - P/A$$

INTERIOR SPAN:

$$f_{TOP} = [(70.42 - 51.29)(12)(1000)] / 1960 - 158.3 = -38.8 \text{ psi}$$

38.8 psi COMPRESSION $\leq 0.6f'_{ci} = 900$ psi $\therefore$ ok

$$f_{BOT} = [(-70.42 + 51.29)(12)(1000)] / 1960 - 158.3 = -275.4 \text{ psi}$$

275.4 psi COMPRESSION $\leq 0.6f'_{ci} = 900$ psi $\therefore$ ok

END SPAN:

$$f_{TOP} = [(90.85 - 66.18)(12)(1000)] / 1960 - 158.3 = -7.26 \text{ psi}$$

7.26 psi COMPRESSION $\leq 0.6f'_{ci} = 900$ $\therefore$ ok

$$f_{BOT} = [(-90.85 + 66.18)(12)(1000)] / 1960 - 158.3 = -309.3 \text{ psi}$$

309.3 psi COMPRESSION $\leq 0.6f'_{ci} = 900$ $\therefore$ ok

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SUPPORT STRESSES

$f_{TOP} = (+M_{DL} - M_{BAL})/S - P/A$

$f_{BOT} = (-M_{DL} + M_{BAL})/S - P/A$

$f_{TOP} = [(88.13 - 64.19)(12)(1000)]/1960 - 158.3 = -11.73$ psi ∴ ok

$f_{BOT} = [(-88.13 + 64.19)(12)(1000)]/1960 - 158.3 = -304.9$ psi ∴ ok

STAGE 2: STRESSES @ SERVICE LOAD (LL + DL + PT)

MIDSPAN STRESSES

$f_{TOP} = (-M_{DL} - M_{LL} + M_{BAL})/S - P/A$

$f_{BOT} = (M_{DL} + M_{LL} - M_{BAL})/S - P/A$

INTERIOR SPAN:

$f_{TOP} = [(70.42 + 20.8 - 51.29)(12)(1000)]/1960 - 158.3 = 86.17$ psi

86.17 psi TENSION $\leq$ 328.63 psi $= 6\sqrt{f'_c}$ ∴ ok

$f_{BOT} = [(-70.42 - 20.8 + 51.29)(12)(1000)]/1960 - 158.3 = -402.8$ psi

402.8 psi COMPRESSION $\leq 0.45 f'_c = 675$ psi ∴ ok

END SPAN

$f_{TOP} = [(90.85 + 35.92 - 66.18)(12)(1000)]/1960 - 158.3 = 212.66$ psi

212.66 psi TENSION $\leq 6\sqrt{f'_c} = 328.6$ psi ∴ ok

$f_{BOT} = [(-90.8 - 35.92 + 66.18)(12)(1000)]/1960 - 158.3 = -528.2$ psi

528.2 psi COMPRESSION $\leq 0.45 f'_c = 675$ psi ∴ ok

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SUPPORT STRESSES

$$f_{TOP} = (+M_{DL} + M_{LL} - M_{BAL})/S - P/A$$

$$f_{BOT} = (-M_{DL} - M_{LL} + M_{BAL})/S - P/A$$

$$f_{TOP} = [(88.13 + 38.52 - 64.19)(12)(1000)]/1960 - 158.3 = 224.12 \text{ psi}$$

224.12 psi TENSION $< 6\sqrt{f'_c} = 328.6$ psi $\therefore$ ok

$$f_{BOT} = [(-88.13 - 38.52 + 64.19)(12)(1000)]/1960 - 158.3 = -640.7 \text{ psi}$$

540.7 psi COMPRESSION $< 0.45 f'_c = 675$ psi $\therefore$ ok

ALL STRESSES ARE WITHIN THE PERMISSIBLE CODE LIMITS $\therefore$ GOOD

ULTIMATE STRENGTH

DETERMINE FACTORED MOMENTS

PRIMARY POST-TENSIONING MOMENTS, M_1, VARY ALONG LENGTH

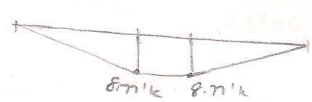
$M_1 = P \cdot e$ $\quad e = 0$ in @ EXTERIOR SUPPORT

$e = 2.5$ in @ INTERIOR SUPPORT

$$M_1 = (266)(2.5)/12 = 55.42 \text{ ft-k}$$

SECONDARY POST-TENSIONING MOMENTS, M_{SEC}, VARY LINEARLY BETWEEN SUPPORTS

$M_{SEC} = M_{BAL} - M_1 = 64.19 - 55.42 = 8.77$ ft-k @ INTERIOR SUPPORTS



1/u

$M_U = 1.2 M_{DL} + 1.6 M_{LL} + 1.0 M_{SEC}$

INTERIOR SPAN:

AT MIDSPAN: $M_u = 1.2(-70.42) + 1.6(-20.8) + 1.0(8.77) = -102.0$ ft-k

EXTERIOR SPAN:

AT MIDSPAN: $M_u = 1.2(90.85) + 1.6(35.92) + 1.0(4.39) = 170.9$ ft-k

AT SUPPORT: $M_u = 1.2(-88.13) + 1.6(-38.52) + 1.0(8.77) = -158.6$ ft-k

DETERMINE MINIMUM BONDED REINFORCEMENT

POSITIVE MOMENT REGION

INTERIOR SPAN: $f_t = 86.17$ psi $< 2\sqrt{f'_c} = 109.5$ $\therefore$ NO POSITIVE REINFORCEMENT REQUIRED

EXTERIOR SPAN: $f_t = 212.66$ psi $> 2\sqrt{f'_c}$ $\therefore$ POSITIVE REINFORCEMENT REQUIRED

$$y = \frac{f_t}{(f_c + f_t)} h = \frac{212.66\,(7)}{(212.66 + 528.2)} = 2.0 \text{ in}$$

$$N_c = M_{DL+LL}/S \cdot 0.5 \cdot y \cdot l_2 = [(90.85 + 35.92)(12)/1960](0.5)(2.0)(18)(12)$$

$$N_c = 167.6^k$$

$$A_{s,min} = \frac{N_c}{0.5 f_y} = \frac{167.6^k}{0.5(60)} = 5.59 \text{ in}^2$$

$$A_{s,min} = \frac{5.59 \text{ in}^2}{18 \text{ ft}} = 0.31 \text{ in}^2/\text{ft}$$

USE #5 @ 12" OC BOTTOM = 0.31 in²/ft

?/11

NEGATIVE MOMENT REGION:

$A_{s,min} = 0.00075 A_{cf}$

INT & EXT SUPPORTS & INT SPAN

$A_{cf} = (18 ft)(8)(12) = 1728 \text{ in}^2$

$A_{s,min} = 1.296 \text{ in}^2$

$\Rightarrow$ USE (5) #5 TOP ($A_s = 1.55 \text{ in}^2$)

CHECK MINIMUM REINFORCEMENT IF IT IS SUFFICIENT FOR ULTIMATE STRENGTH:

AT I. SUPPORTS

$d = 6''$

$A_{ps} = 0.153 \text{ in}^2 (10) = 1.53 \text{ in}^2$

$f_{ps} = 174{,}000 + 10{,}000 + \frac{(3000)(18)d}{300(1.53)} \Rightarrow f_{ps} = 174{,}000 + 117.65d$

$f_{ps} = 174{,}706$ psi

$a = \frac{A_s f_y + A_{ps} f_{ps}}{0.85 f'_c b} = \frac{(1.55)(60000) + (1.53)(174{,}706)}{(0.85)(3000)(18 \times 12)} = 0.65$ in

$\phi M_n = 0.9[(1.55)(60) + 1.53(174.7)][6 - \frac{0.65}{2}]/12 \Rightarrow \phi M_n = 153$ ft-k

$\phi M_n = 153 < 158.6$ ft-k $\therefore$ ULTIMATE STRENGTH GOVERNS @ SUPPORTS & @ MIDSPAN EXTERIOR

11/11

EXTERIOR MIDSPAN

TRY $A_s \Rightarrow$ (8) #5 $A_s = 2.48\ in^2$

$a = 0.76$

$\phi M_n = 175.4$ ft-k > 170.9 ft-k $\therefore$ ok

SUPPORT

TRY (6) #5 $A_s = 1.86\ in^2$

$a = 0.69$

$\phi M_n = 160.7$ ft-k > 158.6 ft-k $\therefore$ ok

REINFORCEMENT

(8) #5 TOP @ EXTERIOR MIDSPAN

(6) #5 TOP @ SUPPORTS

(5) #5 TOP @ INTERIOR MIDSPAN

#5 @ 12" O.C. BOTTOM END SPANS

(10) PT TENDONS → JACKING FORCE = 266^k

Appendix G: Cost analysis

COST ANALYSIS (RSMEANS SQUARE FOOT COSTS 2010)

BAY SIZE: 18 ft x 24 ft ≈ 20 ft x 25 ft

LOCATION FACTOR: BOSTON = 1.17

CAST IN PLACE FLAT PLATE W/ PT: (P. 262)

($14.10)(1.17) = $16.50 / ft²

PRECAST PLANK W/ 2" CONCRETE TOPPING: (P. 264) (P. 268)

(PLANK) ($12.99)(1.17) = $15.20 / ft²

(BEAMS & GIRDER) ($13.15)(1.17) = $15.39 / ft²

$30.59 / ft²

COMPOSITE DECK: (P. 277)

($16.10)(1.17) = $18.84 / ft²

Appendix H: References

1. Brazil, Aine. "Staggered Truss System Proves Economical For Hotels." *Modern Steel Construction* September 2000

2. Faraone, Tom. "Real-life Adventures in Staggered Truss Framing." *Modern Steel Construction* July 2003