Massachusetts General Hospital - Building for the Third Century

Pro-Con Structural Study of Alternate Floor Systems

55 Fruit Street
Boston, MA 02114

The Pennsylvania State University
Department of Architectural Engineering
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EXECUTIVE SUMMARY:

Purpose

This Pro-Con Structural study of alternate floor systems has been developed to compare floor systems which may be considered for use in the Massachusetts General Hospital project "The Building for the Third Century".

Building Description

The B3C hospital facility contains 530,000 square feet total including: 162,300 square feet of patient bed space, 45,900 square feet of mechanical, and 114,900 square feet of procedural space. The façade of the building is mostly glass. The main structural system consists of a steel moment frame with composite metal deck flooring. The columns transfer load through concrete load bearing elements to bedrock. The systems are being constructed in a manner which allows for fast track construction to ensure that the hospital will become operational in a timely manner.

Floor System Comparison Conclusion

Further study of Hollow Core Plank and Non-composite floor systems should be conducted based on the criteria of construction, cost, and architectural influence. One-way and two-way concrete systems present too many obstacles to be considered viable options for construction on this site for the current architectural design. A table is provided at the end of the report summarizing the criteria considered.

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INTRODUCTION

This comparative structural study of alternate flooring systems contains information about the existing floor system as well as three alternate floor systems. Flooring systems will be developed in a schematic manner but include calculations to determine preliminary sizes through checking stress and deflections. Comparisons of the systems will include: fire protection and rating, constructability, weight, deflections, architectural considerations, costs, and system depth.

BACKGROUND

The Building for the Third Century (B3C) project (Cover and Figure 1) is located at 55 Fruit Street in Boston, Massachusetts (Figure 2). The site of the present construction once held three outdated hospital buildings. The Clinics, Tilton and Vincent Burnham Kennedy Buildings were demolished in order for this project to move forward. Logically located at the center of the city, the hospital campus is able to serve over a million patients each year.

Construction of the ten story super structure and four subterranean levels is currently underway through an up-down construction process. B3C was designed for Massachusetts General Hospital (MGH) by NBBJ Architects of New York with the charge of bringing the hospital into its third century of existence. In order to bring this design into a functioning reality NBBJ enlisted the services of several technical firms including: Michael Van Valkenburgh Associates, Inc., Thompson Consultants, Inc., Engineered Solutions Inc., Mcnamara/Salvia, Inc., and Vanasse Hangen Brustlin Inc. Fast tracked construction is being coordinated by the experienced Turner Construction Company to ensure that the facility is operational in a timely manner.

Holding true to its charge the B3C design team has created a unique hospital facility which will improve the patient experience. Functions of the hospital lead to the design of the spaces in an efficiency and comfort driven environment. Lower levels of the building serve as the vehicle access areas such as the loading docks to supply the hospital and the ambulance dock to receive emergency cases. Floors three through four are utilized for procedural space including: general operating rooms (ORs), orthopedic ORs, and neurological ORs. Utilizing the fifth floor of the building for mechanical equipment allows for the roof space to be developed as a green space. Patient beds occupy the top five floors of the building and are a short walk from a large Bamboo filled five-story atrium. The subterranean levels of the building house the sterile processing and radiation oncology departments. Housed in the lowest level of the hospital are eight linear accelerators used for cancer treatment.

Structural System Overview

The structural system overview section of this report will focus on all of the main structural features of the building. Discussion of these systems will provide supporting material for the subsequent discussion of the alternate flooring systems. The features to be discussed include: general floor framing, structural slabs, the lateral force resisting system, foundation system, secondary structural systems, the exterior envelope, and expansion joints. An understanding of the interaction of these building components will allow for deeper study of specific components of the system.

General Floor Framing – The main framing type for this building is a composite steel frame with beams transferring load to girders and girders to columns. The system is constructed of mostly W shapes whose strengths may be found in Appendix C. Most of the connections in the system are simple or shear connections however the main lateral force resisting system consists of a moment frame, which will be discussed later. Beams commonly have 30ft spans in the building but there are spans of up to 42ft. Floor heights vary between 14ft and 30ft. Column splices commonly occur at 4ft above the floor level of the splice. This framing system necessarily holds up the structural slabs of the building which are discussed next.

Structural Slabs – Four levels of this hospital facility are subterranean on the site and play an interesting part in the construction process of the building. The structural slabs of the basement levels are flat slabs supported by the steel columns of the building and drop panels. The slab thickness is 14 inches in most areas and an additional 8 inches is employed for the drop panel areas of the slab. Material strengths of the concrete and the reinforcement utilized in these structural slabs has been documented in Appendix D. The construction of this hospital has been fast tracked, due to its obvious importance, and these structural slabs play an important role in that process which will be described in the foundations discussion.

Main Lateral Force Resisting System – As previously described the main lateral force resisting system is based on a moment frame. The columns are set approximately 10ft inside the perimeter of the slabs, on floors 1-10, makeup the moment frame. This system wraps the building around all sides of the building, as is portrayed in Figure 4. The strengths of this moment frame may also be found on Appendix D. A preliminary analysis of the lateral forces on the building was conducted for both wind and seismic loading. After calculating the lateral forces on the hospital it was determined that the wind loading in the North – South directions would present the largest lateral loads on the building. These calculations and results are discussed eventually in this report. Wind loads are first met by the curtain wall that covers a majority of the building façade. The load is transferred from the glass to the hangers directly into the floor slabs. The metal deck composite floor system aids the lateral force system by distributing the wind forces to the moment frame. The transmission of the lateral load can be seen in Figure 3.

Foundation System – There are several important parts to the foundation system including: a slurry wall, load bearing elements, and caissons. Describing these components in order of construction will be beneficial in describing the construction process being used on this fast track site. The first element of the foundation system is a 30 inch thick slurry wall. The perimeter of the building was excavated to the bearing bed-rock and then reinforcing steel cages were lowered into the slurry filled holes. Concrete is then pumped into the hole while the slurry is removed. These walls will hold back the soil pressure while building. The holes for the Load Bearing Elements (LBEs) were also excavated to proper depths before any of the soil was taken from the slurry wall surrounded site. These LBEs support the majority of the structural load of the building. Thus the columns were imbedded into the concrete of the LBE. Those columns reach from the lowest basement level floor to the first floor when they are placed. This column and slurry wall layout allows for "Up – Down" Construction to take place. This construction method calls for a crew to be working under ground to excavate under the floor slabs and the steel crew to be setting steel going up. This process is presented in Figure 6. Caissons also play an important role in the structural support of the building. The caissons carry the load of the massive shielding walls needed for the use of the Linear Accelerators used to create radiation for cancer treatment. All of the material used in the foundations elements can be found in Appendix D.

Other structural considerations that will need to be made later on are the lateral soil loads that the slurry walls will have to withstand after the lower levels have been excavated. Also the water table is high in this area, due to its proximity to the river, which will necessitate consideration of uplift on the structure.

Secondary Structural Systems – In order to create a more connected atmosphere within the hospital campus bridges are being constructed to a few of the nearby buildings. The Yawkey Center for Outpatient care and the Wang Ambulatory Care Center will be the buildings connected to. This requires creating a structure that will not transfer loads from the new building to the older buildings. These bridges are framed with large W shapes, have concrete on metal deck flooring, and glass facades.

There is also a canopy located at the entrance of the building which will need to be evaluated for wind and snow loads.

Exterior Envelope – The façade of the B3C project is designed to let in maximum amounts of natural light and thus is composed of mostly glass. The curtain wall system is hung from embedded mounts at each floor level. This allows the lateral loads to be transferred directly into the composite floor and eventually to the moment frame serving as the main lateral force resisting system. The curtain wall system also plays an important role in the environmental control in the building but its structural significance is lateral load transmission. Again this transmission is represented in Figure 4. This system is how the building meets the wind, how the building meets other buildings will be discussed next.

Expansion Joints – The building itself does not have any notable expansion joints causing the need for internal load separation but there are important expansion joints between the B3C and other adjacent buildings. Buildings close enough to require expansion joints are Ellison and

White. The materials most commonly used in the expansion joints are large rubber gaskets and aluminum plates. These joints are commonly located where a floor, ceiling, or wall meets a similar feature of the joining buildings. The importance of these joints is to provide transition from one building to another while not transmitting loads from one building to the other. Space is built into these joints to allow for movement of the buildings as well. It is appropriate to end our discussion of the structural system with a discussion about expansion joints because the B3C project is all about expanding into the third century of the hospital's existence.

Existing Floor System Discussion

Steel Composite – A majority of the flooring in the B3C project is composed of steel composite beams utilizing metal deck and shear studs to form the composite system. Figure 6 in Appendix A shows a section of a typical composite system. Figure 3 highlights the area of Basement Level A which will be studied throughout this report. It was chosen because the bay is regular, meaning that all each column connects to two of the other four directly via a wide flange. Commonalities between this area of the structure and the rest of the building make it a viable option for further investigation.

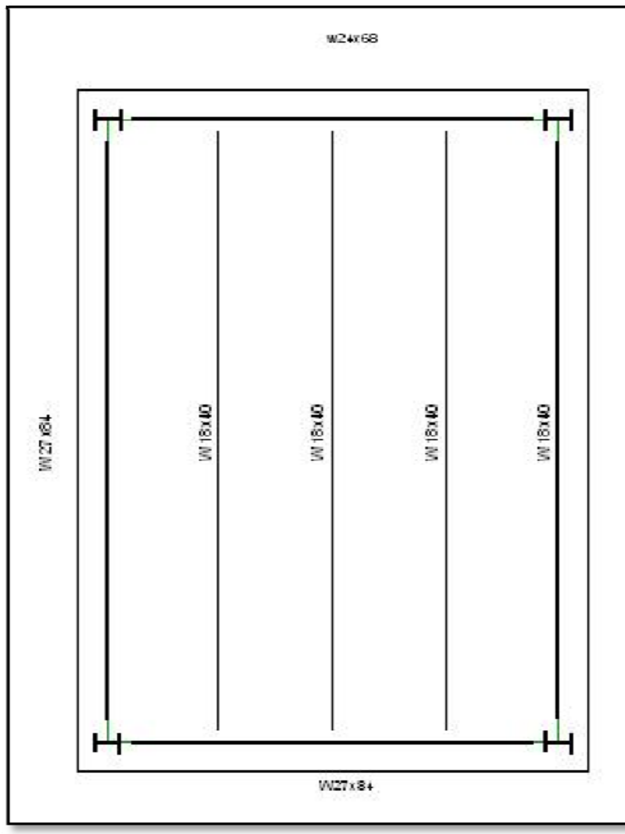


Figure 4 in Appendix A provides the necessary information to analyze the existing floor system, through checking stresses and deflections. The W-shapes used include: W24x68, W27x84, W27x84, and most commonly W18x40. The image presented here represents the basic layout of the floor. Three-dimensional representations of the existing floor system are also presented in Appendix A Figure 7. Loading information employed for the subsequent calculations is available in the load table presented in Appendix B. Nominal moment capacity was determined to be 846 kip ft which sufficiently larger than the design moment of 654.45 kip ft. Construction deflection was determined to be significantly larger than the allowable. As indicated on the figure there are cambers built into each member to combat deflection in the floor.

Important parts of the existing system include the ability for steel to be erected without being slowed by the concrete flooring installation and that the base columns of the building are able to be set on the foundations up to fifty feet below grade. These constructability factors make this system highly efficient for this building design. Part of the building floor system uses Hollow Core Planks to allow access to the lower levels of the building while subterranean excavation continues. Thus a logical flooring option for the building would be to use Hollow

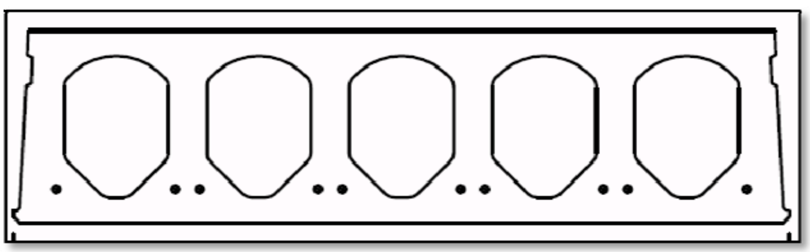
Core planks throughout the building which is the first alternate flooring system considered in this report.

Alternate Flooring Systems

General Considerations – In order to determine a comparable floor system to the existing system a set of control points must be set. One of the most difficult parts of engineering the structure of a building is coordination with the architect to create the desired space. Thus throughout the development of these preliminary floor system designs the column placement will not be changed in order to provide the architect with the desired space. Providing the same columns spacing in each design will cause the larger than normal spans in some of the systems but this control point must be maintained for valid comparisons to be made.

Another important consideration for the floor systems is Up-Down Construction, which is being used to speed up the construction of this hospital facility. Setting the steel columns in footings up to fifty feet below grade before the rest of the site has been excavated is crucial to the construction process. This construction process is most practical when steel is the main structural material because it is able to be set in place without formwork.

Hollow Core Plank – Precast concrete members save money on labor associated with the construction of these systems because they are produced in a controlled factory environment. Labor savings is the initial appeal of this system and the fact that is already located in portions of the building make it a viable option for the first alternate system to be studied. The image to the right is a basic section representation of a Hollow Core Plank. Figure 10 of Appendix A shows how the layout of the 4ft wide planks would look spanning 30ft between girders. The loading used to determine which planks would work is located in Appendix C under the Plank Calculations. Molin a concrete product company produces a member large enough to carry the design loads. Eight pre-stress strands are located near the bottom side of the member where the greatest tension forces exist. Along with determining the correct Hollow Core Planks to use one must also determine the W-shapes to transfer the gravity loads to the columns. Design of these members considering stresses and deflections concludes that W40x167 is adequate. A member has a deflection of 0.893in is present but is within the allowable amount. System depth can be defined by two different connections to the supporting W-shape via embedded plates in the Hollow Core Planks. These attachment options can be compared in Figure 9 of Appendix A. Option number two, where top-of-plank is set equal to top-of-steel; will limit the space wasted by flooring systems in the height of the building.



An important aspect of this floor system to realize is that the construction process will be significantly different if chosen. In order for the building to go up steel must first be laid followed immediately by the Hollow Core Planks. The existing floor system allows for the floors to be finished while the steel erection for the building continues.

One-Way Slab – Changing the primary structural material of the building is another workable option because there is already a fair amount of concrete placed in the existing building's footings. The first system to be explored is a one-way slab. Bay dimensions present a challenging space for the system to work, column layout will not be changed to preserve the control points as architectural changes are not within the scope of this study.

Figure 11 presents the dimensions, beams, columns and direction of reinforcement that will be used in the design of the slab. To avoid excessive deflections the general rule, from ACI, of minimum slab thickness for a slab continuous in both directions was used to calculate the minimum slab depth of 18 inches. There are already slabs in the basement levels of building which are 22 inches thick thus an 18 inch slab is feasible but the support beams must also be considered. If the ℓ value of the one-way slab is considered to be 41′6″ an 18 inch slab is determined as in the calculations in Appendix C. Another case was considered with an ℓ of 20′ 9″ provides a 9″ thickness for the slab. Thus if this design is to be considered in this building a smaller span achieved by more beams will keep the depth of the system down. By including more beams the price per square foot will increase.

Two-Way Slab – Another option for development of a concrete system is the use of a two-way slab which should be a reduced depth from the one-way option. As developed in the calculations in Appendix C labeled Two-Way Slab the minimum slab thickness can conservatively be taken to be 12″ two trials were made once with 24″ square cross section beams and once with 26″ square cross section beams and it was determined that neither of these met the stiffness criteria. Thus bringing one to believe that the depths presented by such a system would not lend it to be used in a building where higher ceiling heights are desirable.

Non-Composite Floor System – The existing composite floor system requires the installation of metal deck and shear studs to create the composite action between the concrete slab and the steel W-shapes. A study of a non-composite slab is valuable because the construction process would be very similar to the system already in use. As seen in the calculations labeled Non-Composite in Appendix C the United Steel Deck Design Manual was used to determine the correct decking to be used for the 7.5 foot spans. The beam layout, see in Figure 11 of Appendix A, was kept the same as the composite system for ease of comparison. After the deck was chosen new dead loads could be calculated to provide information for the calculations of the supporting beams. Beams were determined to be W21x62 size which, as was expected, is significantly larger than those beams used in the composite design. Girders were also determined to be larger than those in the composite design being sized at W40x149. These beams are larger in size due to the increased need to resist compression forces in the top flange of the beam compared to the composite system.

Lack of shear stud installation in the non-composite design is the main cost saving point of this system over a composite system. A trade-off for the lack of shear stud install is larger beam sizes.

Points of Comparison

While considering floor system changes there are several areas which must be considered in order to determine if the floor system is a viable option. Points of comparison include: depth of the system, weight of the system, architectural considerations, construction process considerations, deflections, and fire protection. Using the existing floor system as a control point will allow for a base system

Depth- When considering differing flooring systems it is assumed that the floor to floor heights in the building will remain the same to preserve the height of the building. By keeping the floor levels the same and changing the floor system depths the ceiling heights may change in the building. The portion of Basement Level A being considered in this report calls for 9'6" ceilings. In this portion of the building the floor to floor height is 19 feet meaning that there is sufficient space for any of the floor systems to be utilized. System depth ranging from 42 inches to 26 inches will not affect if the system is feasible to be utilized at this level but other floors with heights of only 14 feet may present problems for integration of structure and HVAC equipment. More often than not larger system depths are concurrent with larger system weights as will be discussed in the next section.

Weight- Foundation considerations are most affected by the weight of the building. A steel structure will be considerably less weight on the foundations compared to a concrete frame building. This comparison is easy to make on Table 1 at the end of this report. Weights included on the table are for the slab, metal deck, support beams and girders. Concrete systems considered in this report are not fully developed and thus the integral beams that would be a part of the one-way or two-way slabs are not included in the weights of the systems, yet they are still comparably large. Most of the foundations of the building are already specified to engage bedrock for bearing however the size of the members may need to be increased. If serious consideration is to be given to the concrete systems then an investigation of the foundations of the building must be conducted.

Architectural Considerations- Most of the floor systems considered in this report would benefit from changing the column layout of the building to reduce the spans of the building. However due to the intricacies of the hospital floor plan movement of the columns is not a permissible option. Thus in order to preserve the architectural spaces of the building the column layout will not be changed. There are still compromises to the spaces in the building due to reduced ceiling heights that would result from the increased depths of some of the systems.

Construction Considerations – System constructability is yet another important consideration when comparing flooring systems. There would be very little change in the construction of the non-composite system for the building but considerable complications would be presented by any of the other systems previously described.

Hollow Core Plank requires that the planks be placed on the steel as the building is erected causing the need for coordination between the steel erector and the contractor placing the precast concrete. This schedule co-dependence could result in massive delays if either contractor were to encounter a setback. A unique solution to this construction sequence would need to be developed in order to make this system a feasible alternate.

One and two-way slabs requiring the conversion of the structural system from steel to concrete would cause significant changes in construction scheduling. Most notably the up-down construction method currently being used would no longer be an option. Steel columns are able to be set below grade because they are comparably slender to the concrete columns that would be required. Form work needed to construct columns requires more space than would be available in the present excavations. Also connections to subsequent subterranean floors would be difficult to construct because significant rebar development lengths would be required and those lengths would have to be achieved with rebar left exposed on the columns. Steel is much easier to set below grade because it can be lowered into a hole and set with a crane in a relatively short amount of time.

Fire Protection – One final consideration for the development of a new floor system is the fire protection requirements involved with each system. The existing system requires that the steel be coated with a cementitious fireproofing material. This material is easily applied through a spray on application. Similar fireproofing would be used on the non-composite system. The concrete systems are inherently fireproofed because the rebar cover called for by ACI provides the steel with necessary protection. Thus money savings is realized because there is no need for additional fireproofing contractors to be used.

Conclusions

After careful consideration of the alternate flooring systems mentioned above it would be worthwhile to further study Hollow Core Plank and Non-Composite flooring systems. One-way and two-way concrete systems present too many obstacles to be considered viable options for construction on this site for the current architectural design. A table is provided on the following page to summarize the criteria considered in this report.

System	Depth (in)	Weight (PSF)	Cost (per ft^2)	Architectural Impact	Construction Impact	Deflection (in)	Fire Protection	More In Depth Analysis
Existing Composite Slab	At Girders 35.5	114	$42.50	NA	NA	1.63	Common	NA
Hollow Core Planks	At Girders 38.6	111.85	$13.46	Greater Floor Depths	Longer Schedule Time	0.893	Common	Yes
One-Way Slab ℓ = 41.5'	42	225	$26.40	Deepest Girders	Longer Schedule Time	1.38	None	No
One-Way Slab ℓ = 20.75'	42	112.5	$13.20	Deepest Girders	Longer Schedule Time	1.05	None	No
Two-Way Slab	26	150	$27.50	NA	Larger Foundations Required	1.38	None	No
Non-Composite	At Girders 38.2	54.69	$34.40	Greater Floor Depths	Shoring During Curing	1.057	Common	Yes

Document and Code Review

Here is provided a list of the Documents and Codes utilized in analysis and discussion of the structural system.

- ASCE/SEI 7-05 *Minimum Design Loads for Buildings and Other Structures* published in 2006 by the American Society of Civil Engineers (ASCE 7)
- *AISC Steel Construction Manual 13th Edition* published December 2005 by the American Institute of Steel Construction, Inc. (AISC 13th ed.)
- ACI 318-08 *Building Code Requirements for Structural Concrete* published August 2008 by the American Concrete Institute (ACI 318)
- Construction Documents S100 – S602 Dated February 29 2008
- *Unified Design of Steel Structures* Published 2008 Author Louis F. Geschwindner

Professional Contacts

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John J. Tracy P.E. LEED AP
Project Manager
McNamara/Salvia Inc. Consulting Engineers

Figure 1 – Birds Eye Views of B3C Project

Figure 2 – Site Map

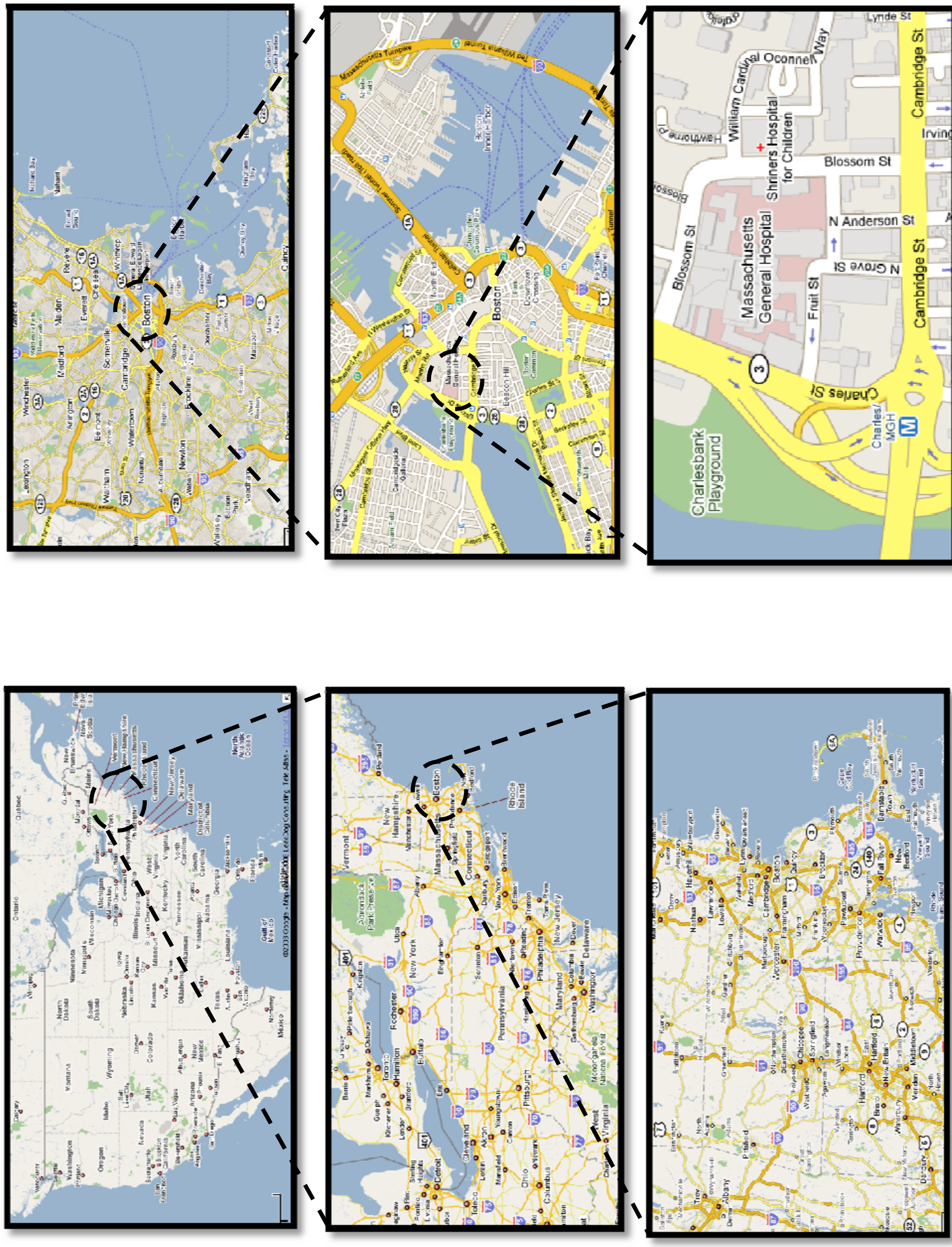


Figure 3: Partial Floor Plan of Basement Level

Figure 4: Existing Framing Plan

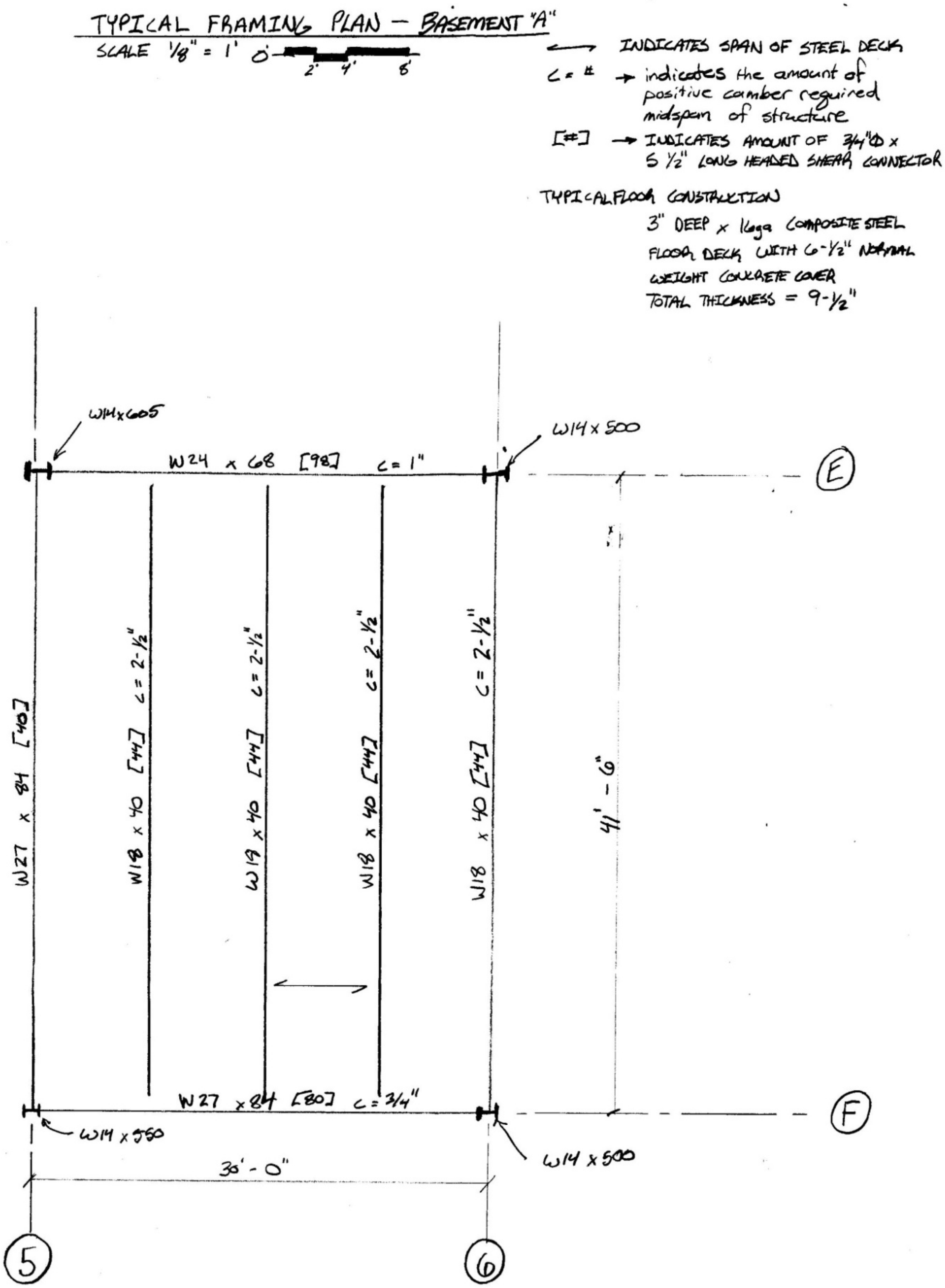


Figure 5: Section of Typical Flooring System

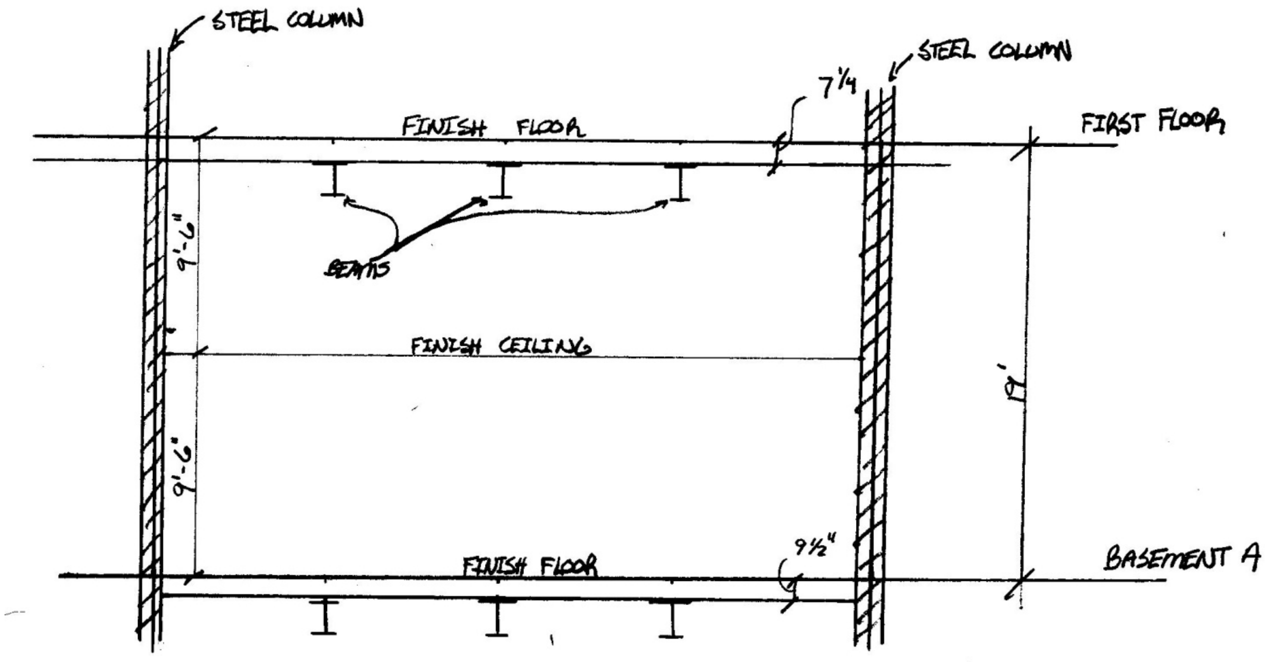


Figure 6: Typical Metal Deck Section

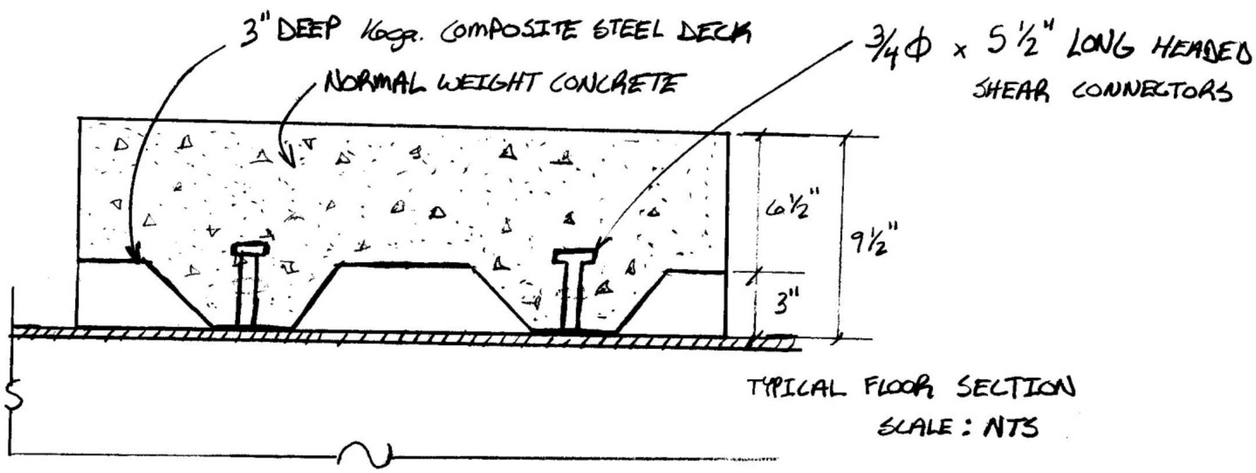
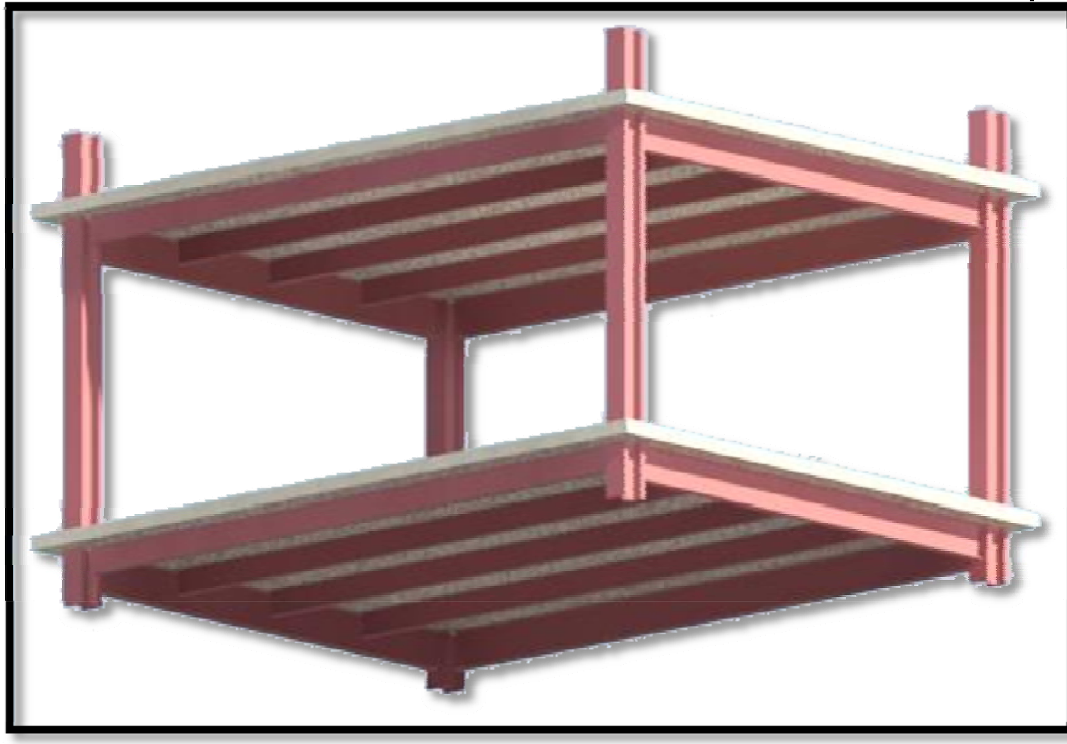


Figure 7: Existing Framing System



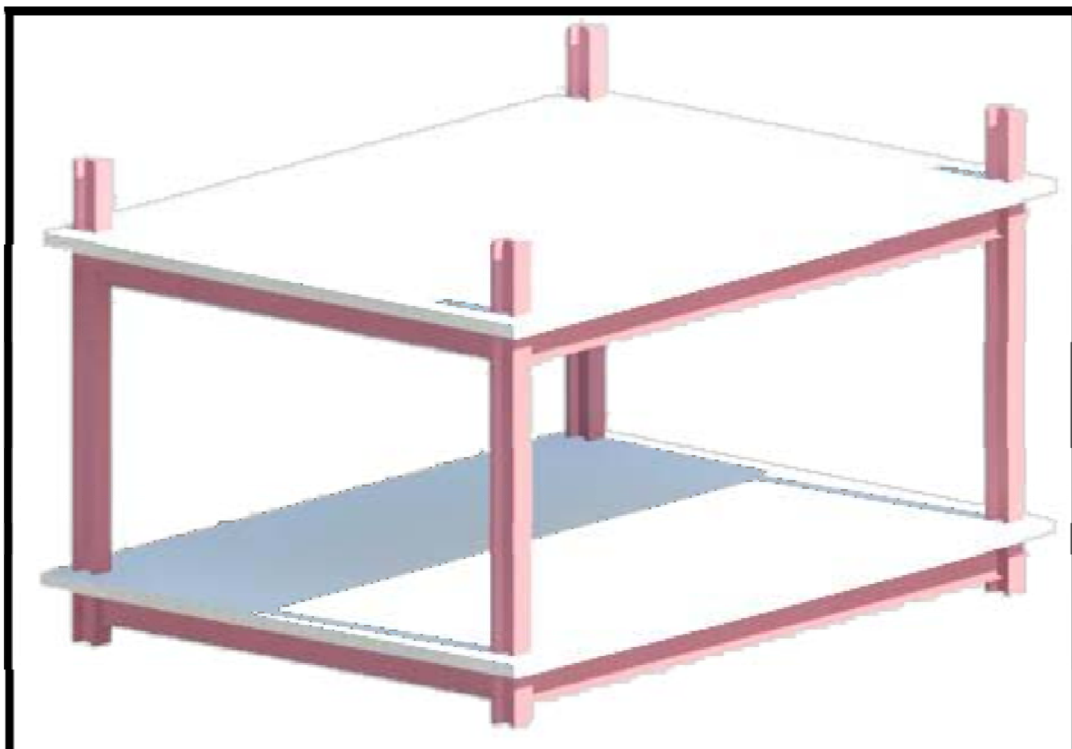


Figure 8: Structural Perspective

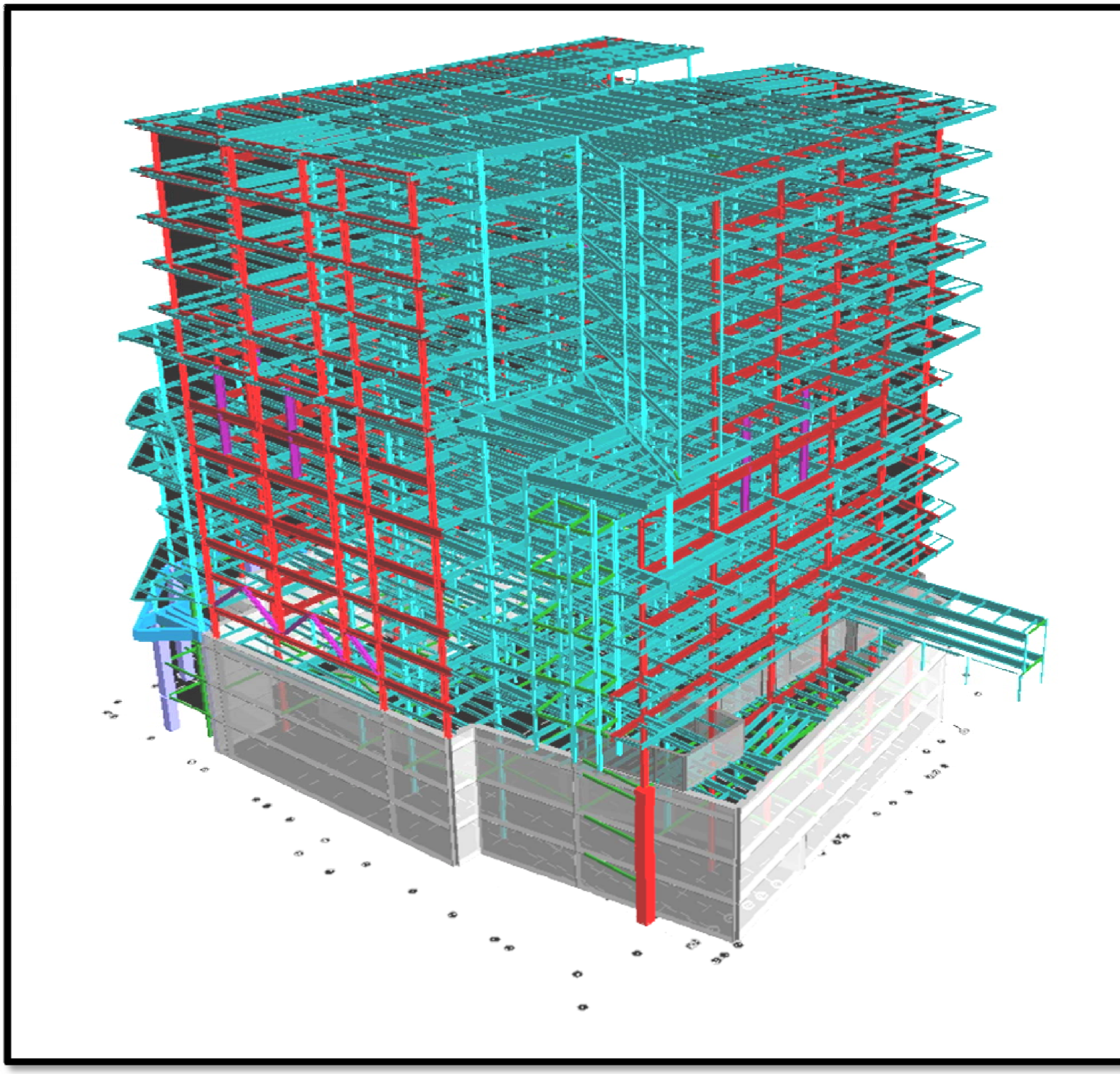
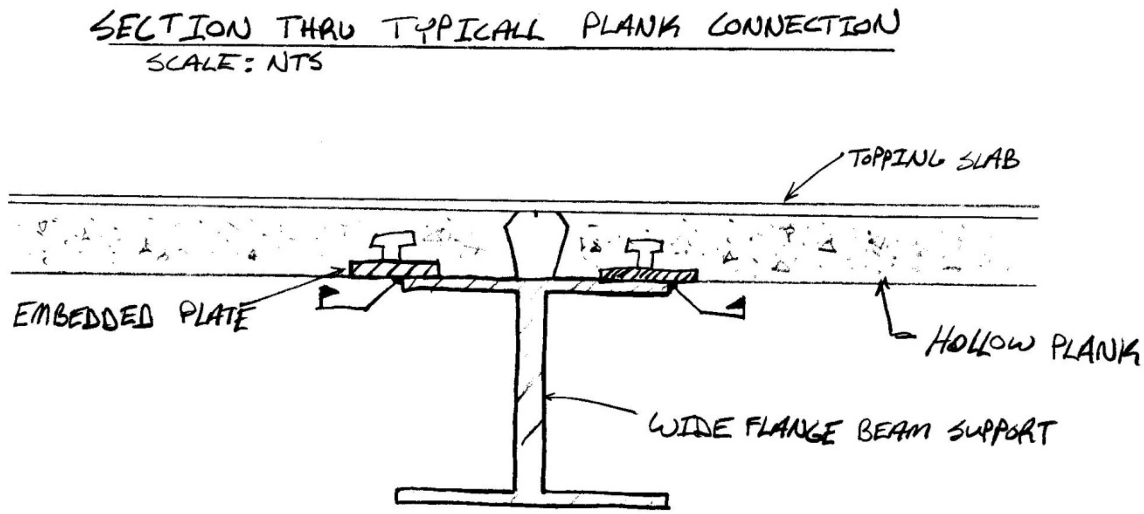


Figure 9: Precast Hollow Concrete Plank Sections



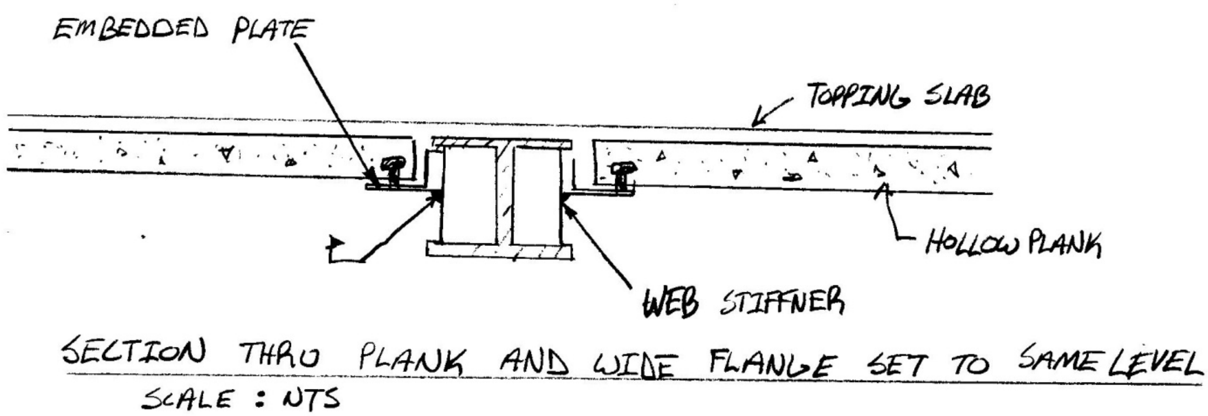


Figure 10: Plank Layout

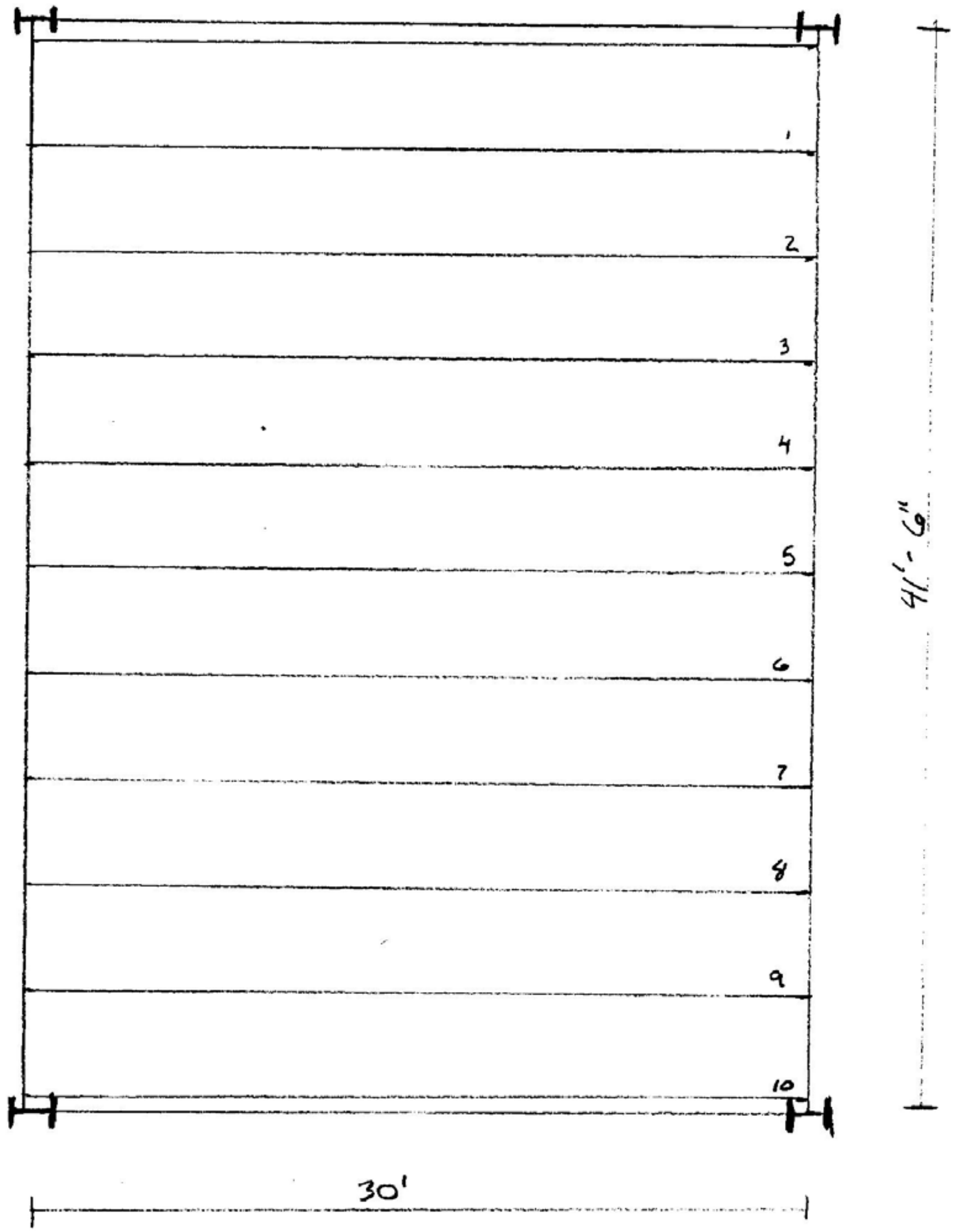


Figure 11: Column and Beam Layout of One-Way Slab

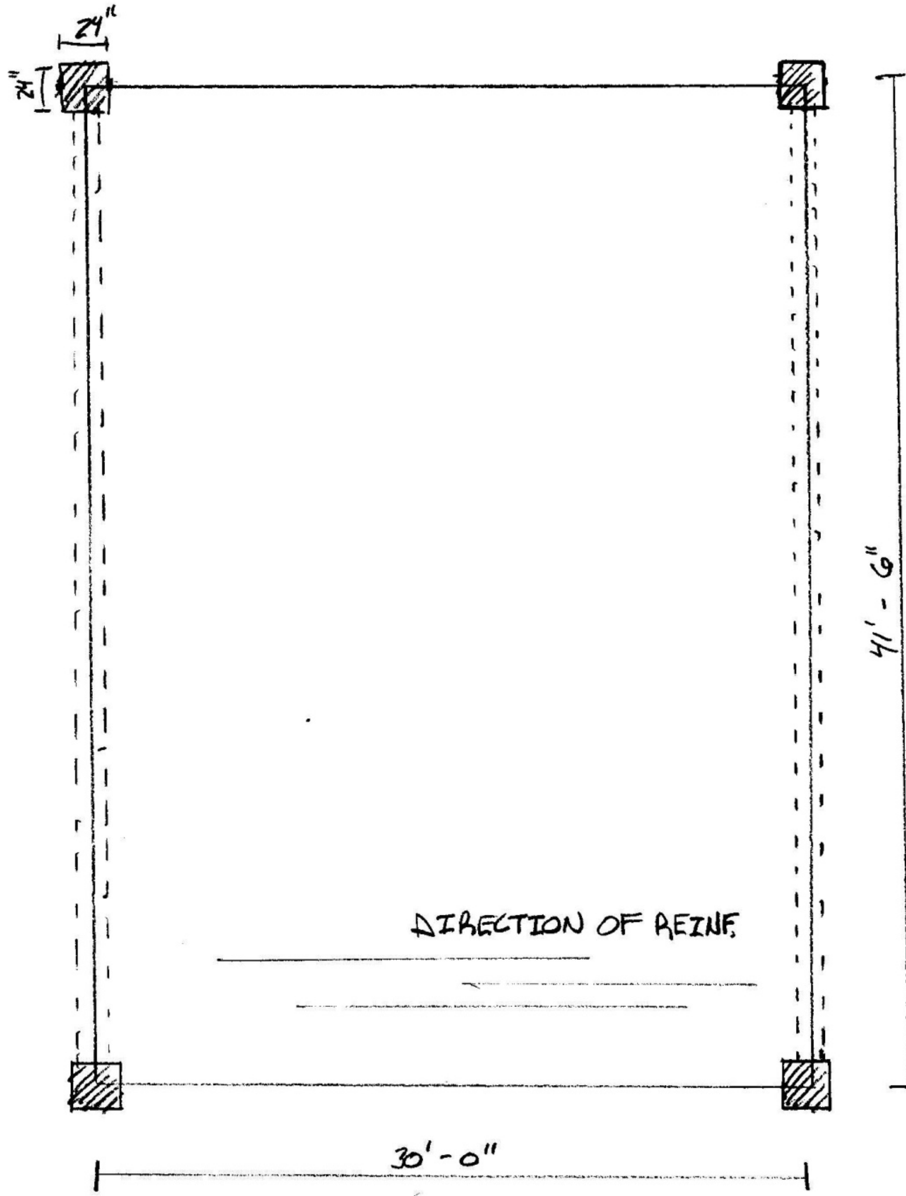
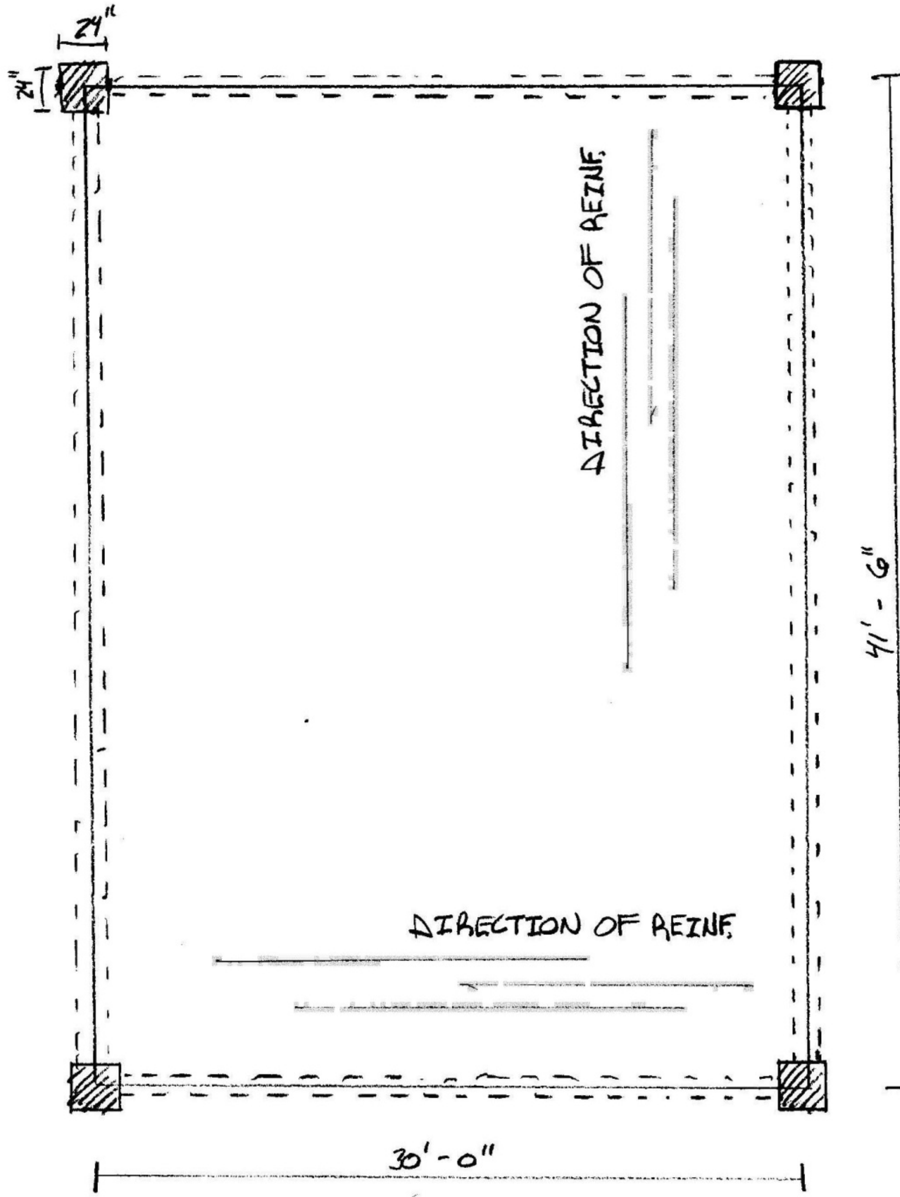


Figure 11: Column and Beam Layout of Two-Way Slab



Appendix B – Calculations

Existing Flooring System

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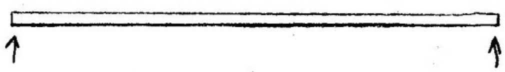
I. Composite Beam → At BASEMENT LEVEL A

Assume simply supported and full composite



Beam Spacing 7.5

F_y = 55 ksi from ASTM
F_u = 65 ksi from ASTM
F'_c = 4,000 psi

Steel W18 x 40 $A_s = 11.8 \text{ in}^2$
$w = 40 \text{ lb/ft} = 5.33 \text{ lb/ft}^2$

Concrete 12.5 lb/ft^2 per inch thicknes
8 in thick
$12.5(8) = 100 \text{ lb/ft}^2$

Metal Deck 2.25 lb/ft^2

Partition Load 10 lb/ft^2

Ceiling Load 20 lb/ft^2

Total DEAD 137.6 lb/ft^2

Total Live 150 lb/ft^2 → from CD S100

$w_u' = 1.2(D) + 1.6(L)$

$w_u' = 1.2(137.6\text{psf}) + 1.6(150\text{psf})$

$w_u' = 405.12$

$w_u = 405.12\text{psf}(7.5')$

$w_u = 3.04 \text{ K/ft}$

$$M_u = \frac{3.04 \text{ k/ft} (41.5')^2}{8} = 654.45 \text{ kip ft}$$

① Determine Required Moment Capacity

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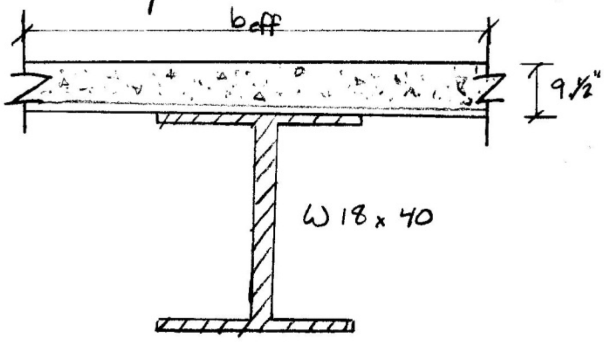
Composite Beam continued



$b_{eff} = b_1' + b_2'$

b_1' is LEAST OF
SPAN/8 = 41.5ft/8 = 5.2ft
OR
1/2 (dist adj beam)
= 1/2 (7.5) = 3.75 ft

$b_2' = b_1'$

$b_{eff} = 2(3.75) = 7.5ft$

$A_s = 11.8\ in^2$
$d = 17.9\ in^2$
$b_f = 6.02\ in$
$t_f = 0.525\ in$
$b_{eff} = 90\ in$

② DETERMINE THE CONTROLLING COMPRESSIVE FORCE

$V'c = 0.85\,(f'c)\ b_{eff}\ t$
$V'c = 0.85\ (4)\ (90in)\ (9.5in)$
$V'c = 2907\ kip$

$V's = A_s F_y = 11.8in^2 (55\ ksi) = 649\ kip$

$V_s' < V'c$ ∴ THE PNA IS IN THE CONCRETE

③ DETERMINE PNA LOCATION USING a

$$a = \frac{V's}{0.85\,f'c\ b_{eff}} = \frac{A_s F_y}{0.85\,f'c\ b_{eff}}$$

$$a = \frac{649\ kip}{0.85\ (4ksi)(90in)} = 2.12$$

④ DETERMINE THE NOMINAL MOMENT STRENGTH USING Eq 9.5

$M_n = T_s\,(d/2) + C_c\,(t - a/2)$
$= 649\ kip\,(17.9in/2) + 649\,(9.5in - 2.12/2)$
$= 11286\ in\ kips$
$= 940.5\ ft\ kips$

$\phi M_n = 0.9 M_n = 0.9(940.5\ ft\ kip) = \underline{846.45}\ kip\,ft$

$\phi M_n = 846.45\ kip\,ft > M_u = 654.45\ kip\,ft$

∴ BEAMS CHECKS OUT OK !!!

AE 481 | Existing Floor System | Matthew J Decker

II. COMPOSITE BEAM → DEFLECTIONS

TOTAL CONSTRUCTION LOAD   NO LOAD FACTORS USED

$$\omega = \omega_D + \omega_{L\text{-}const}$$

$$\omega = (137.6\ lb/ft^2)(7.5') + 0.36\ kip/ft$$

$$\omega = 1.392\ kip/ft$$

DETERMINE THE MOMENT OF INERTIA OF W18x40

$$I_x = 612\ in^4$$

$$\Delta_{CL} = \frac{5\omega L^4}{384\ EI} = \frac{5(1.392\ kip/ft)(41.5')^4(1728)}{384(29000)(612\ in^4)}$$

$$\Delta_{CL} = 5.23\ in > \frac{41.5'(12)}{360} = 1.38\ in$$

DOES NOT MEET DEFLECTION REQUIREMENTS

THUS THE BEAMS ARE CAMBERED THE REQUIRED CAMBER WOULD BE 3 7/8 inch
THE DESIGNED CAMBER IS 2 1/2 inches

LIVE LOAD DEFLECTION
TO FIND LOWER BOUND MOMENT OF INERTIA
USE TABLE 3-20 IN STEEL MANUAL

$$Y2 = 6\tfrac{1}{2}'' - \tfrac{1}{2} = 6\ in$$

$$\Sigma Q_n\ 1140$$

$$I_{LB} = 1590\ in^4$$

$$\Delta_{LL} = \frac{5(.150\ psf \times 7.5')(41.5')^4(1728)}{384(29000)(1590\ in^4)}$$

$$\Delta_{LL} = 1.63\ in > 1.38\ in$$

DIFFERENCE OF 0.25 in

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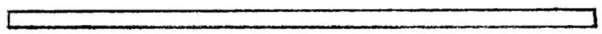
I. PLANK SELECTION



LOADING
DEAD → Partition 10 psf
Ceiling 20 psf
Total 30 psf

LIVE Total 150 psf

$W_u = 1.2(D) + 1.6(\text{LIVE})$

$W_u = 1.2(30\text{ psf}) + 1.6(150\text{ psf})$

$W_u = 276\text{ psf}$ ← SUPERIMPOSED SERVICE LOADS

FROM Molin Concrete products choose Hollow Core:
Designation → T1248-148 4 - 6/10 & 4 - 1/2 STRANDS

Properties: $A_c = 386.81\text{ in}^2$
$f'c = 9000\text{ psi}$

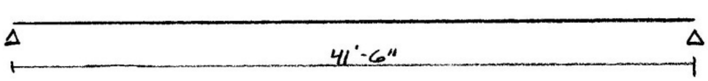
$W_u = 276\text{ psf}\,(4') = 1.104\text{ kpf}$

$M_u = \dfrac{W_u(30')^2}{8} = \dfrac{1.104\text{ kpf}\,(30')^2}{8} = 124.2\text{ kip ft}$

$\phi M_n = 286.8\text{ ft kip} > M_u = 124.2\text{ ft kip}$

∴ PLANK SIZED CORRECTLY FOR LOADING AND MOMENT

II STEEL BEAM DESIGN



LOADING
DEAD: Partition 10 psf
Ceiling 20 psf
Plank 100.725 psf

LIVE: 150 psf

PLANK LOAD
$A_c(L)(150\text{ pcf})$

$386.8\text{ in}^2 / 144\text{ in}^2 = 2.686\text{ ft}^2$

$2.686(30')(150\text{ pcf}) = 12.087\text{k}$
$12.087 / 120\text{ ft}^2 =$

$W_u = 1.2(D) + 1.6(\text{LIVE})$

$W_u = 1.2(130.725\text{ psf}) + 1.6(150\text{ psf})$

$W_u = 396.87\text{ psf}$

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II STEEL BEAM DESIGN CONTINUED

$W_u = 396.87\,psf\,(30')$ ← Trib width
$= 11.906\ k/ft$

Required strength
$M_u = \frac{w_u (l)^2}{8} = \frac{11.906\,k/ft\,(41.5')^2}{8} = 2563.14\ ft\ kip$

$Z_{req} = \frac{M_u}{\phi F_y}$ Assumes compact fully braced section

$Z_{req.} = \frac{2563.14\ ft\ kip}{0.90(65)} = 525.77\ in^3$

CHOOSE W40 x 167 $Z = 693\ in^3 > 525.77\ in^3$ ∴ OK ✓

$\phi M_p = 2600 > 2563 = M_u$ ∴ OK ✓

SHAPE PROPERTIES
$A = 49.2\ in^2$
$d = 38.6\ in$
$bf = 11.8\ in$

DEFLECTION CONSIDERATIONS
PLANKS ASSUMED TO MEET DEFLECTION CRITERIA THROUGH MANUFACTURER'S DESIGN

STEEL DEFLECTION

$\Delta_{max} = \frac{5wL^4}{384EI} = \frac{5(.150\,ksf\,(30'))(41.5')^4(1728)}{384(29000)(11600)}$

$= 0.893\ in$

MAXIMUM ALLOWABLE $= \frac{41.5'(12)}{360} = 1.38\ in$

$\Delta_{max} = 0.893\ in <$ Max ALLOWABLE Deflection 1.38 in

DEFLECTION OK

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I. Interior Slab

Note: Slab designed first so dead load of the slab may be used in beam design.

ASSUMING COLUMNS ARE 24" x 24" & BEAMS ARE 12" WIDE

LOADING

DEAD: PARTITIONS → 10psf
CEILING → 20psf

LIVE: LEVEL 0 → 150psf

SELF WEIGHT TRIAL:
MIN THICKNESS (h) OF NONPRESTRESSED ONEWAY SLABS
both ENDS CONTINUOUS $l/28$

$l = 41'6"$ $\quad 41.5'/28 = 17.8"$ ← NO DEFLECTION CALCS NEEDED

18" → WILL BE USED

$150\ \text{lb/ft}^3 \times 18"/12" = 225\text{psf}$

$w_u = 255\text{psf}(1.2) + 150\text{psf}(1.6) = 546\text{psf}$

FACTORED MOMENTS FROM (NILSON, DARWIN, DOLAN TEXT) TABLE 12.1

$$\frac{1}{11} w_u l_n^2 = \left(\frac{1}{11}\right)(0.546\text{ksf})(41.5)^2 = 85.48 \text{ ft kips}$$

MAXIMUM REINFORCEMENT RATIO

$$\rho_{max} = 0.85 \beta_1 \frac{f'_c}{f_y} \frac{\varepsilon_u}{\varepsilon_u + 0.004} \qquad f'_c = 5000\text{psi} \quad f_y = 60\,000\text{psi}$$

$$\rho_{max} = 0.85(0.80)\frac{5000}{60\,000}\frac{0.003}{0.003+0.004} = 0.0243$$

MINIMUM REQUIRED EFFECTIVE DEPTH

$$d^2 = \frac{M_u}{\phi \rho f_y b(1 - 0.59 \rho f_y / f'_c)}$$

$$= \frac{85.48 \text{ ft kips}\ (12\text{in/ft})}{0.90(0.0243)(60)(12)(1-0.59(0.0243)(60/5))}$$

$$= 78.7 \text{ in}^2$$

$$d = 8.87 \text{ in}$$

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I. Interior Slab Continued

$d = 8.87$ ← effective depth $18'' - 1'' = 17''$

$$A_s = \frac{M_u}{\phi f_y (d - a/2)} = \frac{85.48 \text{ ft kips } (12)}{0.90 \times 60 \times (17'' - 1/2)}$$

$A_s = 1.15 \text{ in}^2$

CHECK ASSUMED a

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{1.15 \text{ in}^2 (60)}{0.85 (5)(12)} = 1.353 \text{ in}$$

SECOND TRIAL $a = 1.353 \text{ in}$

$$A_s = \frac{85.48 \text{ ft kips } (12)}{0.90 \times 60 \times (17 - 1.353/2)}$$

$A_s = 1.16 \text{ in}^2$

USE 1.35 in^2

TEMPERATURE AND SHRINKAGE REINFORCEMENT

$A_{s_{TS}} = 0.0018 \times 12 \times 18'' = 0.389 \text{ in}^2$

$$V_u = 1.15 \times \frac{546 \times 41.5'}{2} - 546 \times \frac{17''}{12} = 17\,568 \text{ lb}$$

$V_n = V_c = 2\sqrt{f'_c}\, bd = 2\sqrt{5000} \times 12 \times 17'' = 28\,849 \text{ lb}$

$V_c = 28\,849 \text{ lb} > V_u = 17\,568 \text{ lb} \therefore$ OK

HOWEVER A SLAB 18" THICK IS NOT EFFECTIVE CONSTRUCTION THUS SHOULD CONSIDER A NEW BEAM LAYOUT TO SHORTEN SPAN

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I. Interior Slab Design Two

MIN THICKNESS $= \ell/28 = 8''$

$\ell = 18.667'$

41.5' FIRST ATTEMPT → TRY 20.75'

$h_{min} = \frac{20.75'}{28} = 0.741' = 8.89 \text{ in} \approx 9 \text{ in}$

NEW DEAD LOAD

SELF WEIGHT $= 150 \text{ lb/ft}^3 \times 9/12 = 112.5 \text{ psf}$

TOTAL $= 142.5 \text{ psf}$

$w_u = 142.5 \text{ psf} (1.2) + 150 \text{ psf} (1.6) = 411 \text{ psf}$

FACTORED MOMENTS: $\frac{1}{11} w_u \ell_n^2 = \frac{1}{11} (.411 \text{ ksf})(20.75')^2$

$= 16.09 \text{ ft kip}$

MINIMUM REQUIRED DEPTH

$$d^2 = \frac{M_u}{\phi \rho f_y b (1 - 0.59 \rho (f_y/f'_c))}$$

$$= \frac{16.09 \text{ ft kip } (12)}{0.9(0.0243)(60)(12)(1 - 0.59(0.0243)\left(\frac{60}{5}\right))}$$

$= 14 \text{ in}^2$

$d = 3.741 \text{ in}$

$d = 3.741 \text{ in} <$ effective depth $9'' - 1'' = 8''$

$$A_s = \frac{M_u}{\phi f_y (d - a/2)} = \frac{16.09 \text{ ft kip } (12)}{0.90(60)(8'' - 1/2'')}$$

$A_s = 0.477 \text{ in}^2$

CHECK ASSUMED a

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.477 \text{ in}^2}{0.85(5)(12)} = 0.561$$

$$A_s = \frac{16.09 \text{ ft kip } (12)}{0.90(60)(8'' - 0.561/2)} = \underline{0.463 \text{ in}^2}$$

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I. Two Way slab system

CHECK IF DIRECT DESIGN METHOD (DDM) CAN BE USED:

- Has 3 spans
- $\frac{\ell_2}{\ell_1} = \frac{41.5'}{30'} = 1.383 < 2$
- $\ell_2 - \ell_1 = 41.5 - 30 = 11.5 < \frac{1}{3}\ell_2 = \frac{1}{3}(41.5) = 13.8$
- No OFFSET

LOADING

DEAD → Partitions 10 psf
Ceiling 20 psf
Self weight $0.5'(150\ \text{lb/ft}^3) = 70$

Total 100 psf

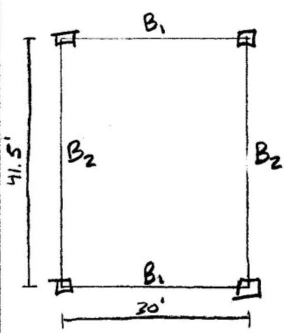
LIVE → 150 psf

$w_L = 150 < 2w_D = 200\text{psf}$

DETERMINE SLAB THICKNESS

Assuming very stiff beams → $t_{min} = \frac{l_n(0.8 + f_y/200000)}{36 + 9\beta}$

MUST BE Greater than 3.5"



$\beta = l_n/s_n = (41.5' - 24/12)/(30 - 24/12)$

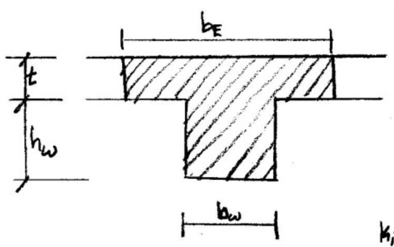
$= \frac{39.5'}{28}$

$= 1.41$

$t_{min} = \frac{(41.5'(12) - 24)(0.8 + 60000/200000)}{36 + 9(1.41)}$

$t_{min} = 10.7"$ conservatively increased to $\boxed{12" = t}$

CHECK THAT $0.2 \le \frac{\ell_1^2/\alpha_1}{\ell_2^2/\alpha_2} \le 5$ STIFFNESS LIMIT



$b_E = b_w + 2h_w$

$b_w = 24\text{in}$

$h_w = 24" - 12" = 12"$

$b_E = 24\text{in} + 2(12\text{in})$

$= 48\text{in}$

$k_{int} = \frac{1 + \left(\frac{48"}{24"} - 1\right)\left(\frac{12}{24}\right)\left[4 - 6\left(\frac{12}{24}\right) + 4\left(\frac{12}{24}\right)^2 + \left(\frac{48}{24} - 1\right)\left(\frac{12}{24}\right)^3\right]}{1 + \left(\frac{48}{24} - 1\right)\left(\frac{12}{24}\right)}$

$k_{int} = \frac{2.0625}{1.5} = 1.375$

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I. Two Way Slab System Continued

Still checking $0.2 \le \dfrac{l_1^2/\alpha_1}{l_2^2/\alpha_2} \le 5$

$\alpha = \dfrac{\cancel{E} I_{beam}}{\cancel{E} I_{slab}}$ E → same for BEAM & SLAB

$I_{beam} = \dfrac{k\, b_w h^3}{12} = \dfrac{1.375\,(24'')(24'')^3}{12} = 38\,016\ in^4$

$I_{slab} = \dfrac{l_2 t^3}{12} = \dfrac{(30 \times 12)(12)^3}{12} = 51\,840\ in^4$

$\alpha_{B_1} = \dfrac{38\,016\ in^4}{51\,840\ in^4} = 0.733$

$I_{beam} = 38\,016\ in^4$

$I_{slab} = \dfrac{l_1 t^3}{12} = \dfrac{41.5' \times 12\ (12)^3}{12} = 71\,712\ in^4$

$\alpha_{B_2} = \dfrac{38\,016\ in^4}{71\,712\ in^4} = 0.53$

$\alpha_m = \dfrac{\alpha_{B_1} + \alpha_{B_2}}{2} = \dfrac{0.733 + 0.53}{2} = 0.6315$

$0.6315 < 2.0$ ∴ Not Very Stiff

Try larger 26" BEAMS

$b_E = b_w + 2 h_w$
$b_w = 26 in$
$h_w = 26 in - 12 in = 14''$
$b_E = 26 in + 2(14'')$
$= 54 in$

$$k_{int} = \frac{1 + \left(\frac{54}{26} - 1\right)\left(\frac{12}{26}\right)\left[4 - 6\left(\frac{12}{26}\right) + 4\left(\frac{12}{26}\right)^2 + \left(\frac{54}{26} - 1\right)\left(\frac{12}{26}\right)^3\right]}{1 + \left(\frac{54}{26} - 1\right)\left(\frac{12}{26}\right)}$$

$k_{int} = \dfrac{2.088}{1.50} = 1.392$

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I. Two-way Slab System Continued

$$\alpha = \frac{E\, I_{beam}}{E\, I_{slab}} \qquad E \text{ same for BEAM \& SLAB}$$

$$I_{beam} = \frac{k\, b_w h^3}{12} = \frac{1.392\,(26'')(26'')^3}{12} = 53009$$

$$I_{slab} = \frac{l_2 t^3}{12} = \frac{(30 \times 12)(12)^3}{12} = 51840$$

$$\alpha_{B_1} = \frac{53009}{51840} = 1.022$$

$I_b = 53009 \text{ in}^4$

$I_s = 71712 \text{ in}^4$ $\qquad \alpha_{B_2} = 0.739$

$$\alpha_m = \frac{1.022 + 0.739}{2} = 0.8805$$

AE 481 | FLOOR SYSTEM ALTERNATE | Matthew J Decker

I. Non – Composite FLOOR SYSTEM

UNITED STEEL DECK DESIGN MANUAL

DECK SPANS 7.5'
DECK SUPPORTS 150 psf Live LOAD

CHOOSE 22 ga DECK WITH 4" SLAB DEPTH

CARRIES 170 psf UNIFORM LIVE SERVICE LOAD

II. STEEL DESIGN
BEAM SPACING AT 7.5'

LOADS
DEAD
CONCRETE 145 pcf (3.25"/12"/') = 39.27 lb/ft²
METAL DECK = 1.5 psf
PARTITIONS = 10 psf
CEILING = 20 psf

TOTAL = 70.77 psf

LIVE = 150 psf

$w_u = [1.2D + 1.6L](7.5')$

$w_u = [1.2(70.77psf) + 1.6(150psf)](7.5')$

$w_u = 2.436\ k/ft$

$M_u = \frac{2.436\ k/ft\ (41.5')^2}{8} = 524.43\ k\,ft$

$Z_{req} = \frac{M_u}{\phi F_y} = \frac{524.43\ ft\ kip}{0.90(65)} = 107.52\ in^3$

CHOOSE W21 x 62 FROM AISC TABLE 3-2

$Z_x = 144\ in^3 > Z_{req.} = 107.5$

$\phi M_p = 540 > M_u = 524.43$ ∴ OK MOMENT CAP

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DEAD LOAD INCLUDING STEEL WILL NOT SIGNIFICANTLY CHANGE THE MOMENT AND WILL NOT CHANGE THE SHAPE

III DEFLECTIONS

THE UNITED STEEL DECK DESIGN MANUAL CONSIDERS THE 1/360 OF THE SPAN DEFLECTION IN ITS TABULATION OF ALLOWABLE UNIFORM LIVE SERVICE LOADS. THUS DEFLECTION OF THE CONCRETE ON METAL DECK NEED NOT BE CONSIDERED

STEEL

$$\Delta_{max} = \frac{5wL^4}{384EI} = \frac{5(.150(7.5'))(30')^4(1728)}{384(29000)(1330)} = 0.5316 \text{ in}$$

MAX ALLOWABLE → $\frac{30'(12)}{360} = 1$ in

$\Delta max = 0.5316$ in < MAX ALLOW 1 in
BEAM WORKS FOR DEFLECTION

IV Girder DESIGN

$Wu = [1.2D + 1.6L](30')$ BEAM WEIGHT
$Wu = [1.2(70.77psf + 2.067psf) + 1.6(150psf)]\ 30'$
$Wu = 9.82$ k/ft

$$Mu = \frac{9.82 \text{ k/ft } (41.5')^2}{8} = 2114 \text{ ft kip}$$

$$Z_{req} = \frac{2114 \text{ k ft}}{0.9\ (65)} = 433 \text{ in}^3$$

← causes missreading TABLE 3-2

CHOOSE W40 x 149

$\phi M_p = 2240 > 2114 = M_u$

DEFLECTION

$$\Delta_{max} = \frac{5wL^4}{384EI} = \frac{5(0.150(30'))(41.5')^4(1728)}{384(29000)(9800 in^4)}$$
$= 1.057$ in

MAX ALLOW $= \frac{41.5'(12)}{360} = 1.38 in > 1.057 in$
DEFLECTION OK ✓

Structural Steel Strengths

ASTM Designation	Governed Elements	F_y Min. Yield Stress (ksi)	F_u Min. Tensile Stress (ksi)	Reference Location
ASTM A-992	All W Shapes	50-65[a]	65[a]	Vol. III Structural General Notes S100 & AISC Table 2-3
ASTM A-36	All other rolled shapes, plates, and bars unless otherwise noted	36	58-80[b]	Vol. III Structural General Notes S100 & AISC Table 2-3
ASTM A-500 Grade B	HSS Sections (Square, Rectangular)	46	58	Vol. III Structural General Notes S100 & AISC Table 2-3
ASTM A-500 Grade C	HSS Sections (Round)	46	62	Vol. III Structural General Notes S100 & AISC Table 2-3
ASTM A-53 Grade B	Pipe	35	60	Vol. III Structural General Notes S100 & AISC Table 2-3
ASTM A-325 Type SC or N	All Bolts for connecting structural members	--	105	Vol. III Structural General Notes S100 & AISC Table 2-5
ASTM F1554 Grade 36	All anchor rods unless otherwise noted	36	58-80	Vol. III Structural General Notes S100 & AISC Table 2-5

Notes: a - A maximum yeild-to-strength ratio of 0.85 and carbon eqivalnet formula are included as mandatory in ASTM- 955

b- For shapes over the 426 lb/ft only the minimum of 5858ksi applies

Concrete Strengths		
Governed Elemets	Minimum Compressive Strenght (f'c) psi	Reference Location
Caissons, LEBs	5,000	Vol. III Structural General Notes S100
Slurry Wall Concrete Diaphram	5,000	Vol. III Structural General Notes S100
Cap Walls	5,000	Vol. III Structural General Notes S100
Two-Way Concrete Slabs	5,000	Vol. III Structural General Notes S100
Formed Walls	4,000	Vol. III Structural General Notes S100
Topping Slabs	4,000	Vol. III Structural General Notes S100
Slabs on Grade	4,000	Vol. III Structural General Notes S100
Fill Concrete Mud Slabs	2,000	Vol. III Structural General Notes S100
LinAcc Sielding	5,000	Vol. III Structural General Notes S100

Reinforcing			
ASTM Designation	Bars	Minimum Yield Strength (psi)	Minimum Tensile Strength (psi)
A 615 Grade 60	Less than #11	60,000	90,000
A 615 Grade 75	#11 and greater	75,000	100,000
A 706	To be welded	60,000	80,000