Technical Report 2

Kathryn Gromowski | Structural Option

Floor System Exploration

Faculty Advisor: Dr. Andrés Lepage

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University Sciences Building

Northeast USA

Table of Contents

Executive Summary

The purpose of Technical Report 2 is to design three alternative floor systems and compare them to the analysis performed on the existing structural system of the University Sciences Building (USB). This is accomplished through both hand and computer-aided calculations performed on a typical 36'-4"x21'-0" bay spanning in the North-South direction from column lines C to D and in the East-West direction from column lines 1 to 2. The systems were compared on the basis of general conditions (weight, cost per square foot, and structural depth), architectural conditions (fire rating and other impacts), structural conditions (foundation impact and lateral system impact), serviceability conditions (maximum deflection and vibration control) and construction concerns (additional fire protection required, schedule impact, and constructability). The existing floor system is an 8" thick voided filigree slab with 18" deep voided filigree beams. The three systems designed in this report include:

- Composite Steel Framing with Composite Steel Deck
- Post-Tensioned Concrete
- Precast Hollow core Plank on Steel Girders

The design of the composite steel system results in 3 ½" concrete topping on 3" Vulcraft 3VLI19 composite deck. The framing is W10x22 infill beams spanning 21'-0" with W18x60 girders spanning 36'-4". This is nearly half the weight of the existing system, and has a comparable cost. It receives its strongest benefit from its additional constructability as well as the potential to reduce the required foundations, but it also is the only system considered in this report on which the floor system can be cored without significant structural impact. Its largest flaw is the addition of structural depth, and the requirement for fireproofing that would probably necessitate a drop ceiling. However, the flexibility of this system makes it a viable alternative to the filigree system.

A 7" thick slab with a 14"Dx3'-6"W wide-shallow beam resulted from the post-tensioning design. To achieve this, (18) ½" Ø 7-wire unbonded tendons were used in the distributed direction and (15) ½" Ø 7-wire unbonded tendons were used in the banded direction, grouped into bundles of 5. This system weighed less than the filigree system, and cost essentially the same amount per square foot. Its major advantages were the reduction in structure depth and the preservation of architectural elements. The only perceived drawbacks are the increased construction difficulty due to the post-tensioning tendons and the fact that the slab cannot be easily cored in the event of future space renovation. With so much in favor of this system, it is clear that it is a feasible system.

Nitterhouse Concrete Products was the selected manufacturer for the precast hollow core. Using their product information, a 6" thick hollow core with 2" topping was chosen to maintain the required fire rating and provide topping for floor leveling purposes and diaphragm action. These are supported by W18x175 girders. The resulting structural depth is slightly greater than the filigree slab/beam depth, but the weight is less. The largest drawbacks to this system are the cost, which exceeds the cost of the filigree system by nearly 60%, and the extreme fabrication and construction difficulties associated with the support condition chosen for the precast to help reduce structural depth. The only real advantage of the system is that it is less weight, and thus may have a positive impact on the foundations. Due to high costs, poor constructability, and potential conflicts between the lateral and gravity systems, this system was deemed to be an unacceptable choice.

Building Introduction

The University Sciences Building (USB) is a new building located on an urban university campus in the Northeast USA. The site chosen was previously a parking lot serving adjacent campus buildings (See Figure 1). However, the USB provides a much more appealing image on this busy street corner. It is a departure from typical campus architecture in both material usage and architectural style. These differences serve as a visible indication of the university's new commitment to building sustainable, functional buildings.

Figure 1 Aerial map from Google.com showing the location of the building site.

While most other campus buildings have brick facades with narrow, strip-like windows, the USB is clad largely in a prefabricated natural stone panel with aluminum-honeycomb back-up, which enables the façade to be very light. Seemingly in homage to the surrounding buildings, the USB also utilizes tall, narrow windows. However, they are of varying widths and placement on the building, which adds interest to the façade (See Figure 2). An additional feature is the 5 story atrium that forms the core of the building. It provides significant focal points such as a sweeping spiral staircase and a four-story "biowall," the first of its kind on a US university campus (See Figure 3). The biowall is used to help mitigate air quality within the building, and it is just one of many features that will help to earn the building a LEED Silver rating upon completion.

Figure 2 Exterior rendering showing the stone façade and variation of windows on the USB.

The USB is a multi-use building, incorporating four large lecture-hall style classrooms, an auditorium, several teaching and research laboratories, and faculty offices. It locates the large classrooms and administrative functions on the ground floor of the building for easy public access, but removes the laboratories and offices to the upper four stories for additional privacy. Including the mechanical penthouse, the building stands 94'-3" above grade with a partial basement. It provides the university with 138,000 square feet of new space, and has a construction cost of approximately $50 million. Construction began in August of 2009, and has an expected completion date of September 2011.

Figure 3 Interior rendering of the atrium.

Structural Overview

The University Sciences Building rests on drilled concrete caissons ranging in diameter from 36" to 58" capped by caisson caps and then grade beams. The lower five floors utilize a voided filigree slab and beam system with cast-in place concrete columns. The mechanical penthouse, however, uses steel columns and floor framing. The lateral system consists of several shear walls spanning from ground to various heights. Masonry infill walls are used between columns on the lower floors to help dampen sound from the surrounding urban environment. These non-structural walls are used solely as back-up walls to support the cladding, and were not a part of this technical report, but their design is an important consideration.

The importance factors for all calculations were based on Occupancy Category III. This was chosen because the USB fits the description of a "college facility with more than 500 person capacity," which requires Occupancy Category III.

Foundations

Geosystems Consultants, Inc. performed several test borings on the proposed site of the USB in October 2007. They found that the subsurface conditions consisted largely of extremely loose brick and rubble fill, followed by alluvium and finally residual soils with relatively low load-bearing capabilities. However, comparatively intact bedrock was encountered approximately 25 feet to 34 feet below the surface of the site.

In light of these conditions, traditional shallow spread footings would not be acceptable. Both driven steel H-piles and drilled caissons were considered as options for deep foundations, but H-piles were rejected due to vibration concerns within the subway station adjacent to the site, as well as noise concerns for the surrounding academic buildings. Instead, drilled caissons ranging in diameter from 36" to 58" were chosen to carry the loads from grade beams to the bedrock below. It was also recommended that the fill under the slab on grade (SOG) comprising the majority of the first floor be removed to a level of approximately 4 feet below the surface, followed by heavy compaction of subsurface materials, and then backfilled with structural fill to minimize settlement of the SOG due to the extremely poor load-bearing capacity of the brick/rubble fill.

Lastly, groundwater observation wells were installed, and groundwater was found to be present approximately 13 feet to 18 feet below the surface of the site. This is a potential concern, because some of the basement walls are 14 feet underground, and could encounter some loading due to hydrostatic pressure, particularly in seasons where the groundwater table rises due to rain. This was not evaluated in this technical report, but is a consideration for future design.

Framing System

The columns in the lower five stories of the USB are all cast-in-place concrete. The columns closest to the atrium on the ground floor are round columns 2 feet in diameter. Most are changed at the second level to 36"x16" rectangular columns. All other columns are 36"x16" columns from their base to the penthouse, rotated as required to fit into walls. At the penthouse level, the columns change to A572 steel W-shapes. These columns range in size from W8x40 to W8x67.

Lateral System

Shear walls are the main lateral force resisting system in the USB. They are scattered throughout the building to best resist the lateral forces in the building (See Figure 4). All of these walls are 12" thick cast-in-place concrete. Most span from ground level to the roof, but since roof heights vary, they are not necessarily the same height. They are anchored at the base by grade beams that run the full length of the walls. This is a potential overturning concern due to the large forces that can occur on a shear wall. This concern was not investigated in depth in this technical report. Another issue not investigated for this technical report, but that will be of concern later, are the checks for force transfer at the thin, link-like elements to ensure that the lateral forces are able to reach the shear walls.

Roof Systems

There are six different roofs on the USB, due mostly to architectural reasons. Figure 5 shows these roofs and their heights above the ground reference elevation of 0'-0". The Office roof (shown in red) is at the same elevation as the fifth floor. Its structure is a 10" flat plate filigree slab system, similar to the office floors below it. The "Ledge" roof (shown in orange) is at the same level as the Penthouse floor, and is a continuation of the 10" voided filigree slab (V.F.S.)/24" voided filigree beam (V.F.B.) system used in the adjacent AHU Mechanical Room. The atrium roof, 5th Level Mechanical Room roof, and AHU Mechanical Room roof (shown in yellow, green, and purple, respectively) are all 3" P2404 Canam roof deck on steel W-shape framing. The Chiller Mechanical Room roof (shown in blue) is 3" of cast-in-place concrete topping on 3" P2432 Canam composite deck (6" total depth) supported by W-shape framing. This heavier structure is necessary because this roof supports two large cooling towers and a diesel generator. This roof is also the only one with a parapet, which serves as a screen to hide the mechanical equipment and stretches from this roof level to 94'-3".

Figure 4 Typical floor plan taken from Sheet S203. Shear walls are indicated in blue.

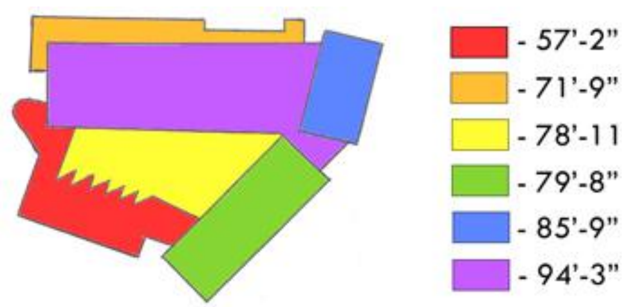


Figure 5 Modified keyplan image from Sheets S205, & S206 showing different roof heights in relation to 0'-0"

Regardless of the underlying structure, all roofs receive the same finish. This consists of sloped rigid insulation under Thermoplastic-Polyolefin (TPO) single-ply membrane.

Design Codes

According to Sheet S001, the original building was designed to comply with:

- 2006 International Building Code (IBC 2006) with Local Amendments
- 2006 International Mechanical Code (IMC 2006) with Local Amendments
- 2006 International Electrical Code (IEC 2006) with Local Amendments
- 2006 International Fuel Gas Code (IFGC 2006) with Local Amendments
- Local Fire Code based on the 2006 International Fire Code (IFC 2006) with Local Amendments.
- Minimum Design Loads for Buildings and Other Structures (ASCE 7-05)
- Building Code Requirements for Structural Concrete (ACI 318-08)
- Masonry Construction for Buildings (ACI 530)
- AISC Manual of Steel Construction, Load Resistance Factor Design (LRFD)

These are also the codes that were used to complete the analyses contained in this technical report, with heavy emphasis on the use of ACI 318-08, AISC Manual, and ASCE 7-05.

Materials Used

Due to the variety of structural types on this project, there are also many different kinds of materials. These are listed in Table 1 below. All information was derived from Sheet S001.

Concrete		
Usage	Weight	Strength (psi)
Caissons	Normal	3000
Caisson Caps	Normal	3500
Footings	Normal	3500
Foundation Walls	Normal	4500
Shear Walls	Normal	4500
Slab-on-Grade	Normal	3500
Columns	Normal	5000
Structural Slabs/Beams	Normal	4500
Precast	Normal	5000
Housekeeping Pads	Normal	3500
Concrete on Steel Deck	Normal	3000

Steel		
Type	Standard	Grade
W-Shaped Structural Steel	ASTM A572	50
Hollow Structural Sections (HSS)	ASTM A500	C
Anchor Rods	ASTM F1554	N/A
Bolts, Washers, and Nuts	ASTM A325	N/A
3/4"x4 1/2" Long Welded Shear Studs	ASTM A496	N/A
Steel Deck	ASTM A653	A or B
Deformed Reinforcement Bars	ASTM A615	60
Welded Wire Fabric	ASTM A185	N/A

Masonry		
Type	Standard	Strength (psi)
Concrete Masonry Units	ACI 530	2175
Mortar	ASTM C270	N/A
Grout	ASTM C475	3000-5000

Miscellaneous	
Type	Strength (psi)
Non-Shrink Grout	10,000

Table 1 Summary of materials used on the USB project with design standards and strengths.

Gravity Loads

As a part of this technical report, dead, live and snow loads were all calculated and compared to loads listed on the structural drawings. Following basic load documentation, several gravity members in the structure were checked to verify their adequacy. Detailed calculations for these gravity member checks can be found in Appendix B.

Dead and Live Loads

Superimposed Dead Loads	
Description	Load
1st Level Ceiling/Mechanical	10 psf
Other Levels Ceiling/Mechanical	15 psf
Electrical Room 4" Housekeeping Pad	55 psf
Mechanical Rooms 6" Housekeeping Pads	80 psf
Roofing	20 psf
Topping on Office Roof	36 psf
Masonry Wall	840 plf

Table 2 Summary of Superimposed Dead Loads.

Weight per Level		
Level	Area (ft^2)	Weight (psf)
Ground	25,459	131.62
2nd	21,135	217.83
3rd	21,135	216.39
4th	21,135	216.39
5th	22,215	234.24
Penthouse	22,602	265.50
Roof	12,780	170.28

Weight of a Typical Floor (3rd Level)			
Description	Weight	Quantity	Total Weight (k)
8" VFS/18" VFB	127 psf	17,200 ft^2	2184.40
10" VFS	100 psf	2,890 ft^2	289.00
12" VFS	120 psf	1,045 ft^2	125.40
Superimposed DL	15 psf	21,135 ft^2	317.03
(43) 36"x16" Columns	600 plf/col	14 ft/col	361.20
Shear Wall	2100 plf	350 ft	735.00
Exterior Wall	840 plf	670 ft	562.80
		Total Weight=	4574.83 k
		Weight per Square Foot=	216.46 psf

Note: Values may differ slightly from values in "Weight per Level" table due to simplifications made in this table to allow for grouping

Table 3 Summary of building weight per level and a typical level.

The structural drawings list superimposed dead loads, summarized in Table 2. Analyses found that these loads are accurate, although conservative in some cases. The ceiling and mechanical load applied is potentially higher than usual, but this can be explained by the large ductwork required to bring 100% outside air into the laboratory spaces. The uniform application of housekeeping pad loads to mechanical and electrical spaces is conservative because these pads are scattered over these spaces. However, these loads seem to be calculated by weight of concrete required for the depth of the pad specified. The masonry walls in the structure are 8" concrete masonry unit (CMU), weighing approximately 60 pounds per square foot (psf). Thus, the masonry wall load corresponds to a 14 foot high 8" CMU wall.

Following the verification of the superimposed dead loads, estimations were made in order to calculate the overall building weight (which was also used in seismic calculations). By looking at typical sections through filigree slabs and beams, it was decided to consider the slabs 80% solid concrete and the beams 90% solid concrete.

Also considered in the building weight calculation were the weights of the columns, shear walls, superimposed dead loads, roofs, and wall loads (both exterior and interior). The exterior walls were considered to be 60 psf, as they are 8" CMU back-up walls with a cladding that weighs approximately 1 psf. The results of this calculation are summarized per level and for a typical level in Table 3. The overall building weight was found to be approximately 30,500 k.

Live loads were also listed on the structural drawings. These were compared to the live loads in Table 4-1 in ASCE 7-05 based on the usage of the spaces, and the results are summarized in Table 4. Although many of these loads matched their ASCE 7-05 counterparts, some exceed the minimum significantly.

The large classrooms on the first floor were all designed for 100 psf, which is the design load for assembly areas with movable seating. These classrooms all have fixed seating, but it is possible that this was not yet decided at the time of the initial structural design, and therefore the more conservative load was used. There is no provision for laboratories in classroom or research facilities, so the provision for "Hospitals – Operating Rooms, Laboratories" was used for comparison. It is possible that this was exceeded because most of these labs are to be teaching facilities, where occupant loads could exceed typical values depending on class sizes. The last major discrepancy was the live load on the Office Roof. This roof was accessible during construction, and was used for materials storage during this phase of the building's life. It is possible this load was increased to account for the loads associated with this, such as workers on the roof to access materials stored there.

It was also noted on the structural drawings that live load reduction was used where allowed by code. Therefore, live load was reduced wherever possible for all gravity calculations in this technical report.

Live Loads			
Space	Design Live Load (psf)	ASCE 7-05 Live Load (psf)	Notes
Atrium	100	100	N/A
Large Classrooms	100	60	Fixed Seating in all
Laboratories	80	60	Based on "Hospitals - Laboratories"
Offices	50+20	50+20	Office Load+Partition Load
Links/Stairs	100	100	N/A
5th Level Lab	80+20	60+20	Based on "Hospitals - Laboratories"+ Partition Load
5th Level Mech. Room	100	N/A	N/A
Electrical Room	150	N/A	N/A
Office Roof	50	20	May be due to construction loading
AHU Mechanical Room	100	N/A	N/A
Chiller Mechanical Room	150	N/A	N/A
Other Roofs	20	20	N/A

Table 4 Summary of design live loads, compared to ASCE 7-05 typical live loads.

Flat Roof Snow Load Calculations	
Variable	Value
Ground Snow Load, p_g (psf)	30
Temperature Factor, C_t	1.0
Exposure Factor, C_e	1.0
Importance Factor, I_s	1.1
Flat Roof Snow Load , p_f (psf)	23.1

Table 5 Summary of roof snow load calculations.

Snow Loads

The roof snow load was calculated using the procedure outlined in Chapter 7 of ASCE 7-05, and the factors required for this calculation are summarized in Table 5. The structural drawings used a C_t of 0.8, but this does not seem to be permissible by code. Therefore, the drawings used a flat roof snow load of 20 psf, whereas 23.1 psf was calculated (and used for all subsequent calculations) in this technical report.

Floor Systems

Although it may not appear so upon first glance at the very irregular shape of the building, the bay sizes are relatively consistent throughout the USB. It simply rotates the bays as necessary to accommodate the different rotations of the wings of the building. Figure 6 shows a typical floor plan with the different bay sizes highlighted with different colors. The legend lists the bay sizes with the span required for the slab first, and then the span required for the girder (if one is present).

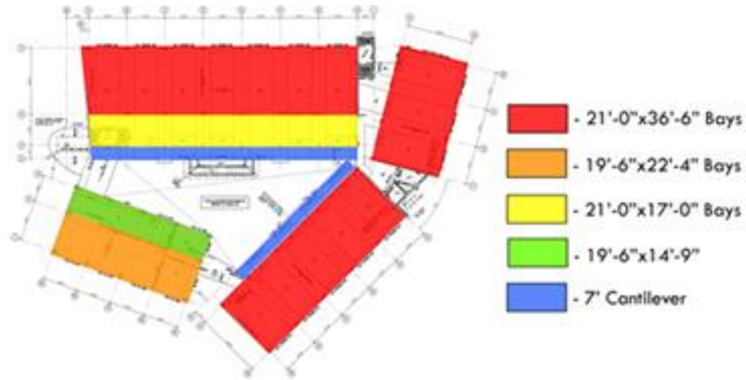


Figure 6 Floor plan from Sheet S203 showing typical bay sizes.

The main objective of this technical report was to analyze the existing floor system, and then design three other floor systems. For ease of comparison, all analysis and design was conducted on a 21'-0"x36'-4" bay spanning North-South between column lines C and D and East-West between column lines 1 and 2 (included as Figure A.6 in Appendix A). All four systems were then compared on the basis of general conditions (weight, cost per square foot, and structural depth), architectural conditions (fire rating and other impacts), structural conditions (foundation impact and lateral system impact), serviceability conditions (maximum deflection and vibration control) and construction concerns (additional fire protection required, schedule impact, and constructability).

Voided Filigree Slab/Beam

The elevated floors of the USB are a voided filigree system. This is a hybrid of precast, prestressed concrete and cast-in-place concrete. In essence, it consists of 2 ¼" of precast, prestressed concrete that functions as leave-in formwork. This is assembled and shored on site, followed by the placement of top and additional bottom reinforcing (if required, placed on rebar chairs on the bottom of the precast), and then further concrete is cast in place to unite the system. To help reduce the weight of the structure, polystyrene voids are incorporated where the concrete is not required for structural strength. Wire joists referred to as "filigree trusses" are used to transfer horizontal shear over the cold joint between precast and cast-in-place concrete.

Different systems were used depending on the required spans and uses. For the area under consideration in this technical report, an 8" voided filigree slab (V.F.S.) was used to span between 18" deep voided filigree beams (V.F.B.). A schematic layout of this type of system, used in the majority of the building, is shown in Figure 7. Spot checks were performed on a V.F.S. panel, the V.F.B. along column line D, and the interior column on column line D (Column D/2). These results can be found in Appendix B.

General

The filigree slab system was found to weigh 127 pounds per square foot (psf), which served as a baseline to compare to the other flooring systems. At approximately $17.50/sf, this is the least expensive system of all systems considered, which may have been the driving force behind its selection for the USB. This cost is an assemblies estimate based on data from RS Means which includes the precast production, transportation, and installation and the cast-in-place concrete materials and placement (including the columns). This estimating method carries an error of approximately 15%. Costs in this technical report do not reflect any schedule effects as a result of altered construction method.

The depth of the structure is 8" in the slab region and 18" under the beams. This is important because almost the entire remaining ceiling cavity is being consumed by the large ductwork required to bring in the required 100% outside air for the laboratories. Therefore, any additional structural depth would either require significant mechanical redesign or additional building height. The building height is not limited by zoning, but by the requirements to remain a non-high-rise building by IBC 2006. This code states in section 403.1 that high-rise provisions only apply to buildings with an occupied floor above 75 feet above the lowest level of fire department access. The highest occupied level at present is the 5th Level, located 57'-2" above grade. Therefore, the building can easily add as much as 17'-10" of height before this becomes truly problematic.

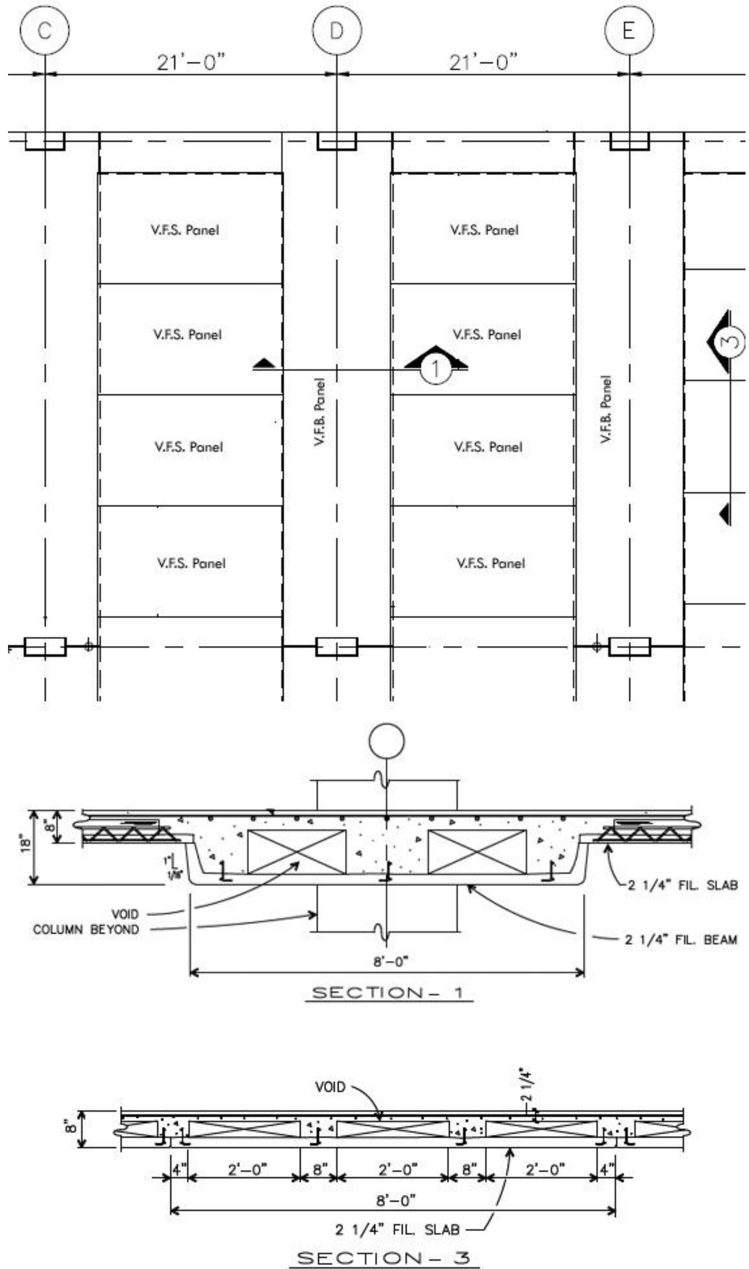


Figure 7 Typical bay with section cuts showing the condition within the beam and the slab. Modified from the filigree slab show drawings and not to scale (NTS).

Architectural

The system has a minimum of the required 2 hour fire rating, and since the building was designed around this system, there are no additional architectural impacts. It should be noted that there are several locations in the building where the bottom of the structure was left exposed, which was made possible by the smooth surface of the precast concrete.

Structural

Drilled concrete caisson foundations and a cast-in-place concrete shear wall lateral system were designed for this system and are unchanged should this system remain.

Serviceability

For the purposes of this report, maximum deflection was calculated by adding maximum slab/infill beam total load deflection to the maximum total load deflection of the girder. This choice was made because the calculated value better represents the overall deflection of the system. For the filigree slab system, this value was found to be 1.07 inches, which is acceptable. It is also the lowest deflection calculated for all of the systems.

Vibrational analyses were not performed for this report, but general research was done on how the system types analyzed/designed typically behave for vibration. The filigree slab system was given a vibrational control rating of "very good."

Construction

This system requires no additional fireproofing to reach the required rating. The construction schedule of over 2 years was developed specifically for this system, and thus was not impacted.

Constructability ratings were given on the basis of difficulty of construction as well as how many different types of crews would have to be involved to successfully install the system. For the filigree slab, it was determined that a crew would be required to place the precast, and then a crew would be required to pour and cast the concrete. Handling of the precast is very difficult because of the thinness of the panels. Therefore, it was assigned a constructability rating of "medium."

System Pro-Con Analysis

Pros:
- Low cost per square foot
- Minimal floor depth allows room for mechanical equipment
- Relatively low deflection

Cons:
- Relatively heavy, which requires large foundations
  - Adverse effect on seismic loads
- Higher construction difficulty
- Difficult to drill through slab due to prestressing strands

Despite the fact that this system is relatively heavy, it performs well in almost every other category analyzed. Therefore, it is easy to see why this system would be chosen over the numerous other options that could have been used for the USB.

Composite Steel

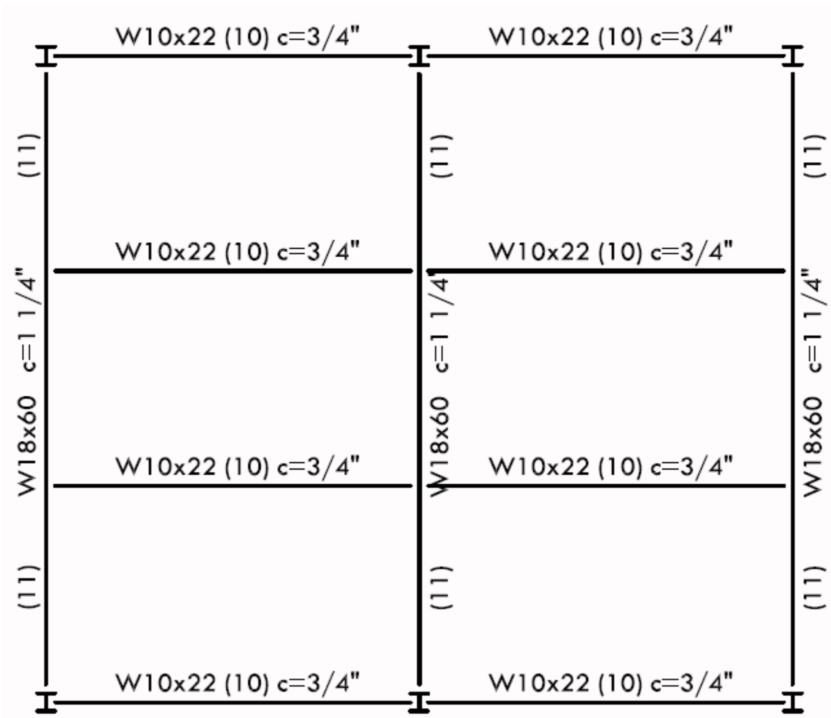


Figure 8 Typical bay (36'-4"x21'-0") showing results of composite steel design.

The first system designed was a composite steel system, which was chosen because it seemed the most practical steel-based system to span the long bays for the relatively heavy loads. Resulting beam and girder sizes with the required camber and shear studs from the hand calculations are shown in Figure 8 (hand calculations can be found in Appendix C). The beams are topped by 3" Vulcraft 3VLI19 composite deck with 3 ½" concrete topping, and composite action is achieved with 4 ½" welded studs. RAM Structural System was used to verify the hand calculations. Values obtained by RAM were almost universally different because the program simply looks for the most economical section without incorporating the need to minimize floor depth.

This layout was the result of several preliminary investigations, the calculations for which are not included in this report but are available upon request. First, beams spanning the long direction with spacing of both 7'-0" and 10'-6" were checked. This resulted in beams and girders of the same size, typically W16's or W18's. Such a layout would have had an overly negative impact on the mechanical plan of the building because it would make running mechanical equipment in the ceiling more difficult. Therefore, the beams were turned to span the short direction to get the recessed "cavity" between the girders in which mechanical equipment could be easily run. Beam spacing of 9'-1" and 12'-1" were checked for this layout, and the 12'-1" spacing was found to be the most economical. All layouts were evaluated for both 4 ½" topping (which automatically achieves a 2 hour fire rating) and spray-on fireproofing the entire bottom of the deck. It was found that neither provided any advantage, and therefore it was chosen to fireproof the entire deck in an attempt to minimize dead load as much as possible.

General
With a 6 ½" total thickness deck and the beams shown above, this system was found to weigh approximately 68 pounds per square foot. This is significantly less than the filigree system. However, it costs $20.40 per square foot (including framing, metal deck, pour stops, concrete for the deck,

fireproofing, columns, and erection, but does not account for impacts on the schedule or foundations). Although this is not an excessive increase, it is certainly a factor to consider. The most important impact is the increase in the overall floor thickness to 17" in the "slab" region and 25" in the girder region. The 9" increase in depth in the slab region is much easier to absorb in the mechanical layout than the 7" increase in depth under the girder.

Architectural
Due to the decision to fireproof the underside of the deck, the system achieves the required 2 hour rating. However, this fireproofing makes the structure impossible to leave exposed, both for aesthetic reasons as well as the concern of pieces of fireproofing flaking off on building inhabitants. A remedy to this would be the use of intumescent paint, but this is usually considered excessively expensive in comparison to simply adding a drop ceiling. The addition of a drop ceiling would provide more mechanical space in which to run equipment, and may actually absorb the effects of the increased structure depth. However, if this is not the case, an additional 7" per floor (2'-4" overall) would have to be added to the 2nd through 5th levels. This is well within the allowable height limits, and thus is not a major concern.

Structural
Since this system is nearly half the weight of the filigree slab/beam system, it was originally hoped that the foundations could be reduced significantly, perhaps even using shallow foundations. However, upon reviewing the geotechnical report, it was found that bearing capacities are not listed for any of the intermediate layers of soil. Even so, these soils are largely described as loose to medium density, composed of largely sand and clay, and are universally declared to have "low load-bearing capacities." Therefore, shallow foundations seem to be impractical. However, the concrete caissons (which are limited to a minimum diameter to achieve sufficient "skin" friction) cannot be reduced in size below 36" in diameter. At this size, the caissons on this project have a service load-bearing capacity of 630 k. RAM gives the service load in Column D/2 with structural steel framing for all floors as slightly less than 490 k, and this is one of the more highly-loaded columns in the structure. Therefore, it seems as though the concrete caissons would be impractical for a structural steel frame due to grossly excessive capacity. To achieve a more reasonable capacity, alternative deep foundation systems could be explored to determine if there is a significant cost-savings in using a smaller foundation system. The most suitable alternate foundation would probably be drilled micropiles. Rammed aggregate piers (Geopiers) and driven piles were also briefly considered as options, but they were rejected due to vibrational and noise concerns for the buildings adjacent to the site. These foundation systems were not designed in this technical report, but would be important to explore further should composite steel be chosen.

The lateral system could be easily transformed into braced frames or moment frames, potentially even placing them in the same locations as the shear walls are currently positioned. This was also not considered in this analysis, but would need to be investigated if composite steel were to be used in the building.

Serviceability
The maximum deflection for the composite steel system was found to be 1.44 inches, approximately 35% larger than that in the filigree slab/beam. It is still well within permissible limits, but it may limit the selection of floor materials more than the filigree slab/beam. Although no vibration analyses were performed, it is known that vibrations are a much larger concern in steel. Should this system be chosen for further investigation, vibrational checks would likely be important to verify the system behaves adequately.

Construction
Structural steel requires spray-on fireproofing to reach the required fire rating, which impacts cost and construction schedule. However, the erection of steel is usually able to be completed more quickly than casting of concrete, and therefore the use of steel may reduce the construction schedule significantly. Because this system is so typical, it was given a constructability rating of "very good."

System Pro-Con Analysis

Pros:
- ❖ Less weight
  - ♦ Potential to reduce required foundations
  - ♦ Positive effect on seismic loads
- ❖ May shorten construction schedule
- ❖ Relatively simple system to construct
- ❖ Ease of drilling through floor

Cons:
- ❖ Higher cost system
- ❖ Leaving structure in the ceilings exposed would be difficult and costly
- ❖ Potential height increase
- ❖ Performs worse for serviceability concerns

Although this system is less effective in comparison to the filigree slab/beam system, none of its comparative flaws render it legitimately inadequate, and therefore it merits further consideration.

Post-Tensioned Concrete

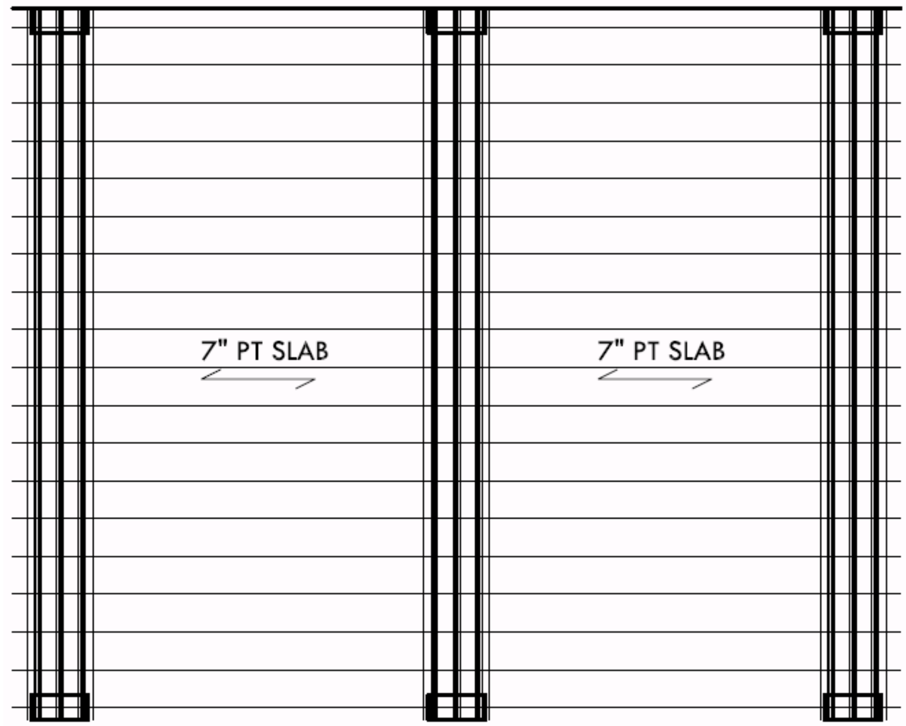


Figure 9 Typical bay showing results of post-tensioned design.

Post-tensioned design is often used to reduce the depth of a traditional concrete system, which was particularly important for the USB. The design was performed by hand calculations (which can be found in Appendix D) based on a design example published by the Portland Cement Association (PCA). Also referenced was the Post-Tensioning Institute's (PTI) Technical Note 3, written by Dr. Bijan Aalami, and an article from the May 2003 issue of Concrete International by Dr. Bijan Aalami and Jennifer Jurgens entitiled "Guidelines for the Design of Post-Tensioned Floors."

These calculations resulted in a 7" deep slab and a 14"Dx3'-6"W wide-shallow beam. The post-tensioning required was (18) ½" Ø 7-wire unbonded tendons in the distributed direction, and (15) ½" Ø 7-wire unbonded tendons in the banded direction. Per recommendations from previous projects, as provided by Dr. Andrés Lepage, the strands in the banded direction are placed into 3 bundles of 5 strands each within the beams, as can be seen in Figure 9.

General
Despite having no voids, the reduction in depth was such that the post-tensioned system only weighs 102 pounds per square foot, which is 20% less than the filigree slab/beam system. The cost of the post-tensioning essentially offsets the cost of the precast for the filigree system, and therefore the post-tensioned costs approximately the same as the filigree at $17.95 per square foot. Since the post-tensioned slab is 7" deep and the beam is 14" deep, this system is actually an improvement on the filigree system (8" V.F.S/18" V.F.B.) in terms of floor depth.

Architectural
This system achieves the required fire rating from cover requirements on the reinforcing, all of which were incorporated into this design, and therefore the required 2 hour fire rating was maintained. The structural depth is actually less than the filigree system, and therefore may create a slightly reduced building

height. This possibility was not explored in this technical report, because any reduction would be minimal (approximately 1'-4"). With careful construction practices, a smooth underside of the structure could be achieved, which would then allow the structure to be left exposed. However, this may be more costly than the basic costs that were evaluated in this report.

Structural

Although the building weight will be reduced if this system were to be used, the impact on the foundations would likely be small, consisting of smaller caissons rather than the possibility of changing foundation systems. This system would also have little or no effect on the lateral system, since concrete shear walls make the most sense for a structure that will be cast-in-place concrete.

Serviceability

Deflections were not directly calculated for this system, but rather were limited by acceptable span-to-depth ratios from industry practice as outlined in PTI's Technical Note 3. It is known that post-tensioned floors also tend to perform very well under vibration loading, and thus serviceability is not likely to be a concern for this system.

Construction

No additional fire proofing is required to achieve the required rating. It is likely that this system will either keep the same construction schedule or potentially lengthen the schedule, depending upon how much concrete will be poured in cold-weather conditions. This system was given a constructability rating of "medium," because although it only involves a concrete crew, this crew must be familiar with post-tensioned construction to complete the project successfully and safely.

System Pro-Con Analysis

Pros:

- ❖ Less weight
    - ♦ Likely minimal impact on foundations/seismic loads
- ❖ Cost approximately comparable to filigree
- ❖ Less floor depth
- ❖ No need for finished ceilings
- ❖ Very good performance under vibration

Cons:

- ❖ Added construction difficulty due to post-tensioning requirements
- ❖ Difficult to drill through slab due to tendons

The benefits of this system far outweigh the negatives, and none of the negatives are such that would preclude the use of post-tensioning. Therefore, this system is a viable alternative.

Precast Hollow core/Steel Girder

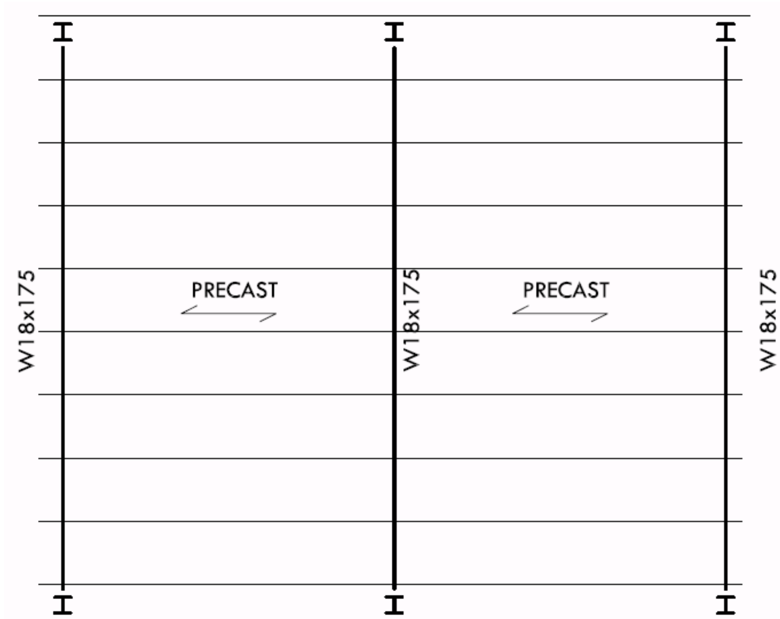


Figure 10 Typical bay showing results of precast hollowcore/steel girder design.

The precast hollow core/steel girder system was chosen after evaluating several other possibilities, the calculations for which are not included in this report but are available upon request. The choice to investigate this system was initiated by the wish to find a steel-based system that would have less impact on the building height than composite steel. The Girder-Slab (www.girder-slab.com) was first considered, but it was found that this would not be suitable for this building because the D-beam support members could not span even the 21 foot dimension under the required loads. Next, a precast hollow core/steel girder system was considered simultaneously with an entirely precast system. It was found that the entirely precast system was impractical because it added weight and structure depth unnecessarily, and thus the final design was performed on the precast hollow core/steel girder system. The system was considered with hollow cores spanning both the long and short directions, but the short direction was chosen because Nitterhouse Concrete Products' design data indicates a 16" deep hollow core would be required to span 36'-4", which defeated the aforementioned purpose of reducing structure depths.

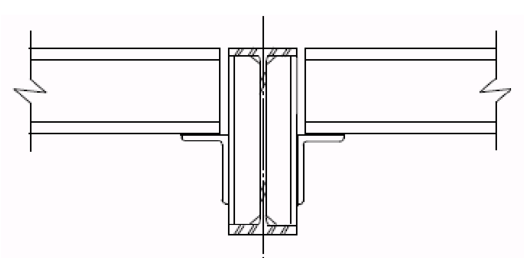


Figure 11 Hollow core to steel beam connection, taken from a September 2007 Modern Steel Construction article by Todd Alwood entitled "Let's Be Plank…"

The final design consists of 6"Dx4'-0"W hollow core planks from Nitterhouse Concrete Products with 2" topping spanning the 21'-0" direction and resting on W18x175 steel beams (see Figure 10) with a connection detail similar to Figure 11. The design was performed by hand calculations, which can be found in Appendix E. The design sheet for the hollow core as provided by Nitterhouse was also included in Appendix E.

General

This system falls in the middle of the weights calculated for the various systems in this report at 82 pounds per square foot. However, it costs by far the most at $27.45 per square foot. This cost includes the precast production, transportation, and installation, the steel framing (including the columns) and erection, the concrete topping, and fireproofing for the steel, but no schedule or foundation impacts. It has a structural depth of 8” in the slab region, which matches the filigree slab/beam system, but is 22” deep at the girder. This 4” increase in depth is in the region that would be very difficult to absorb without increasing building height.

Architectural

The precast portion of the structure achieves the required 2 hour rating for fire protection simply through its design. However, the steel girders would require fire proofing wherever they are left exposed to reach the appropriate rating. Therefore, it would be difficult to leave these beams architecturally exposed. It is possible that it might be easier to build soffits for the beams for this system, but it is likely that the most economical solution to the required fireproofing is to provide a drop ceiling. The addition of a drop ceiling would provide extra mechanical space, and therefore potentially alleviate the issues caused by the increased structural depth. However, if this is not enough, the overall building height would be increased by approximately 1’-4”, which is well below the maximum allowable height increase.

Structural

This system is significantly lighter than the filigree slab/beam system, and may merit a different foundation system, similar to the composite steel system. Since the vertical columns are steel, it is likely the lateral system would have to be either steel moment or braced frames. However, both systems would be complicated significantly by the connection of the precast to the girders. Since fabrication costs for this system are already high (due to the stiffener and shelf angles required to support the precast), unique or difficult moment/braced frame connections would only serve to exacerbate the problem.

Serviceability

The deflection for this system was the worst of all the systems calculated in this report at 1.74 inches (approximately 63% greater than the filigree slab/beam). Although this is within permissible limits, it may limit selection of floor finishes. Most of this deflection comes from the girder, and the only economical way to reduce this deflection is to provide a deeper girder. Since this is undesirable, the deflection presents itself as an unsolvable problem. The behavior of this system under vibration is unknown, and would have to be investigated carefully if this system were to be chosen for further consideration.

Construction

Spray-on fireproofing would be required to ensure the rating of the steel girders. Due to the extremely complicated construction process, which will likely include tack-welding of precast planks during erection, it is possible that the construction schedule for this system would be longer than the one for the filigree slab/beam. Due to the number of trades required to complete this system, as well as how uncommonly used it is (and therefore how little familiarity most contractors will have with the system), it was given a constructability rating of “difficult.”

System Pro-Con Analysis

Pros:
- Less weight
  - Potential to reduce required foundations
  - Positive effect on seismic loads

Cons:
- Very high cost
- Leaving structure in the ceilings exposed would be difficult and costly
- Potential height increase
- Unknown performance in vibration
- Potentially increased construction schedule
- Construction very difficult
- Difficult to drill through slab due to prestressing

The drawbacks to this system are insurmountable by its benefits, which renders this system not a feasible option for the USB.

Summary of Systems

Figure 12 summarizes the results discussed in the preceding sections in a tabular format.

Consideration		System			
		Filigree Slab	Composite Steel	Post-Tensioned Concrete	Precast Hollow core/ Steel Girder
General	Weight (psf)	127	68	102	82
	Cost ($/SF) *	17.50	20.40	17.95	27.45
	Floor Depth (inches)	8 slab/ 18 beam	17 slab/ 25 girder	7 slab/ 14 beam	8 slab/ 22 beam
Architectural	Fire Rating	2 hr	2 hr	2 hr	2 hr
	Other Impacts	Bottom of slab currently left exposed in some locations	Adds 6"-9" of height per floor, cannot be left exposed	Can only be left exposed if underside is well finished	Adds 4" of height per floor, beams cannot be left exposed
Structural	Foundation Impact	Existing cast-in-place caissons & grade beams	May reduce required foundations	May slightly reduce required foundations	May reduce required foundations
	Lateral System Impact	Existing cast-in-place shear walls	Steel braced/moment frames would be considered	None - shear walls would remain	Steel braced/moment frames would be considered
Serviceability	Maximum Deflection (inches)	1.07	1.44	N/A	1.74
	Vibration Control	Very Good	Average	Very Good	Unknown
Construction	Additional Fire Protection Required	None	Spray-on for beams/deck	None	Spray-on for beams
	Schedule Impact	N/A	May reduce construction schedule	Will likely have no effect/potentially increase duration	Will likely have no effect/potentially increase duration
	Constructability	Medium	Easy	Medium	Difficult
Feasible?		N/A	Yes	Yes	No

* - All costs are calculated using RS Means Assemblies Costs (which carries an approximate error of ±15%) for a typical interior bay with dimensions of 36'-4"x21'-0". They include materials (ie steel/concrete/fireproofing), installation, and labor, but do not include impacts on the foundations, construction schedule or architectural elements (ie facade or drop ceilings)

Figure 12 Summary chart of this report's findings.

Conclusion

Technical Report 2 analyzed the existing floor system and compared it to three additional floor systems, all of which were also designed as a part of the technical report. The analysis/design of all systems was performed at a typical bay. Major factors in the comparison of the systems were cost, weight, structural depth, and architectural impact, although several other considerations were also included. It was desirable to reduce the weight of the building without adversely affecting the cost or structural depth.

The existing 8" voided filigree slab and 18" voided filigree beam system remains the least costly system, but by a narrow margin. It is the heaviest system that was considered in this report. It was found in this technical report that the structural depth achieved in this system is very difficult to reduce, or even match. It was verified to be a very reasonable choice as the structural system of the USB.

Composite steel was found to be slightly more expensive but significantly lighter than the voided filigree system. However, it has several negative impacts on the building architecture, such as the potential of increased height (due to higher structural depth) and the inability to leave the structure exposed. Despite these concerns, the system has a great deal of inherent flexibility, and it is possible that with further refinement, these concerns could be resolved. It also can utilize either a braced frame or moment frame lateral system, which provides additional opportunities to adjust the design to suit the building. For these reasons, it was deemed to be a viable alternative.

Of the alternative systems considered, the 7" post-tensioned concrete slab supported by 14" deep wide-shallow beams was by far the most successful. It reduces the weight of the structure by approximately 20%, costs nearly the same, provides a lesser structural depth, and has no architectural impacts whatsoever. The only major drawback of this system is the additional construction difficulty associated with the post-tensioning process. However, this was not deemed to overwhelm the obvious benefits of the system, and it is therefore a feasible option.

The only system of those investigated that was found to be inadequate was the 6" precast hollow core with 2" topping on W18x175 steel girders. Although it reduced the building weight by35% and the structural depth increase was minimal, the system cost was excessive (approximately 60% more than the current filigree slab system) and the architectural impacts would be difficult to accommodate. It also lacks the flexibility of the composite steel system due to the precast-to-steel connection that would make both braced frames and moment frames difficult to design. It was therefore rejected, and will no longer be considered as an alternative.

Appendices

Appendix A: Typical Plans

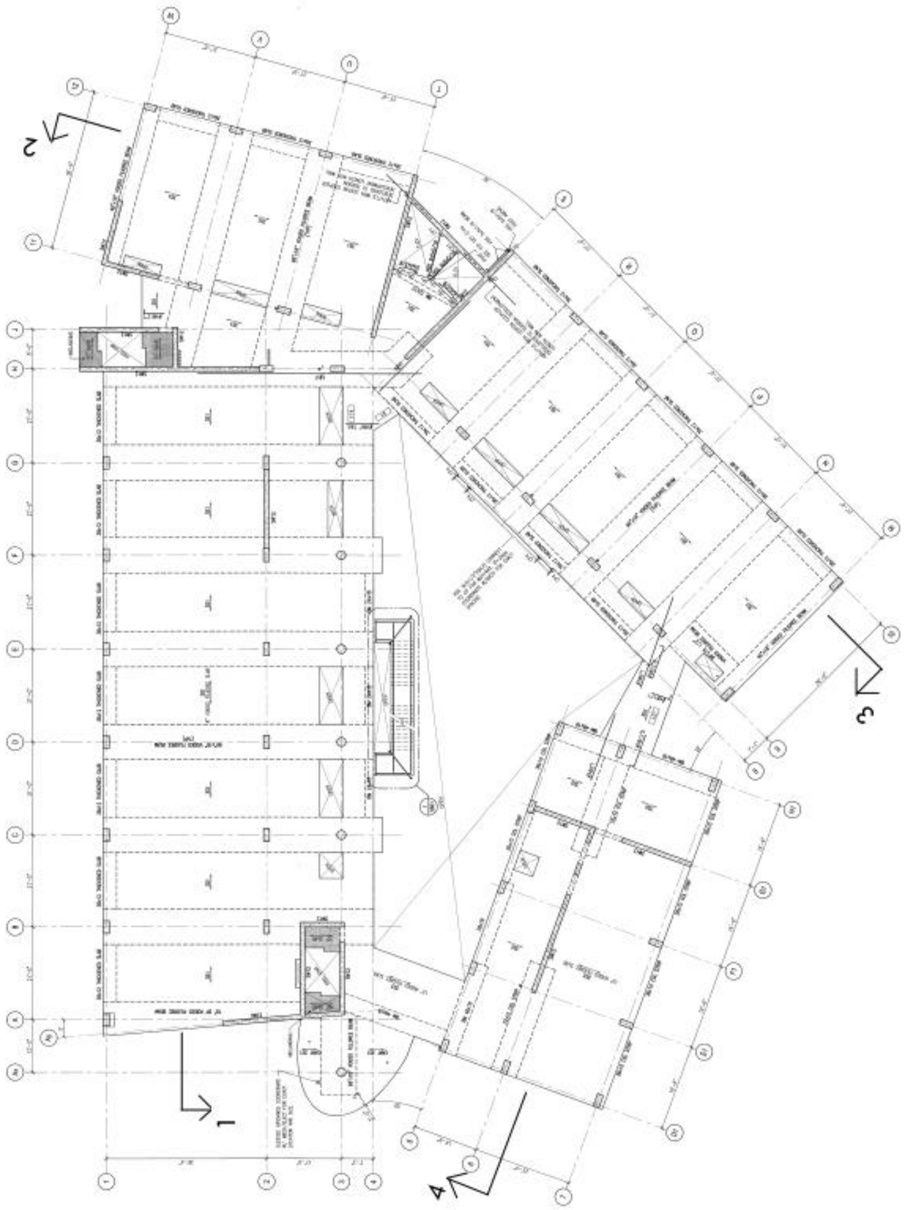


Figure A.1 Typical Floor plan, taken from S202. See following figures for sections indicated on the plan.

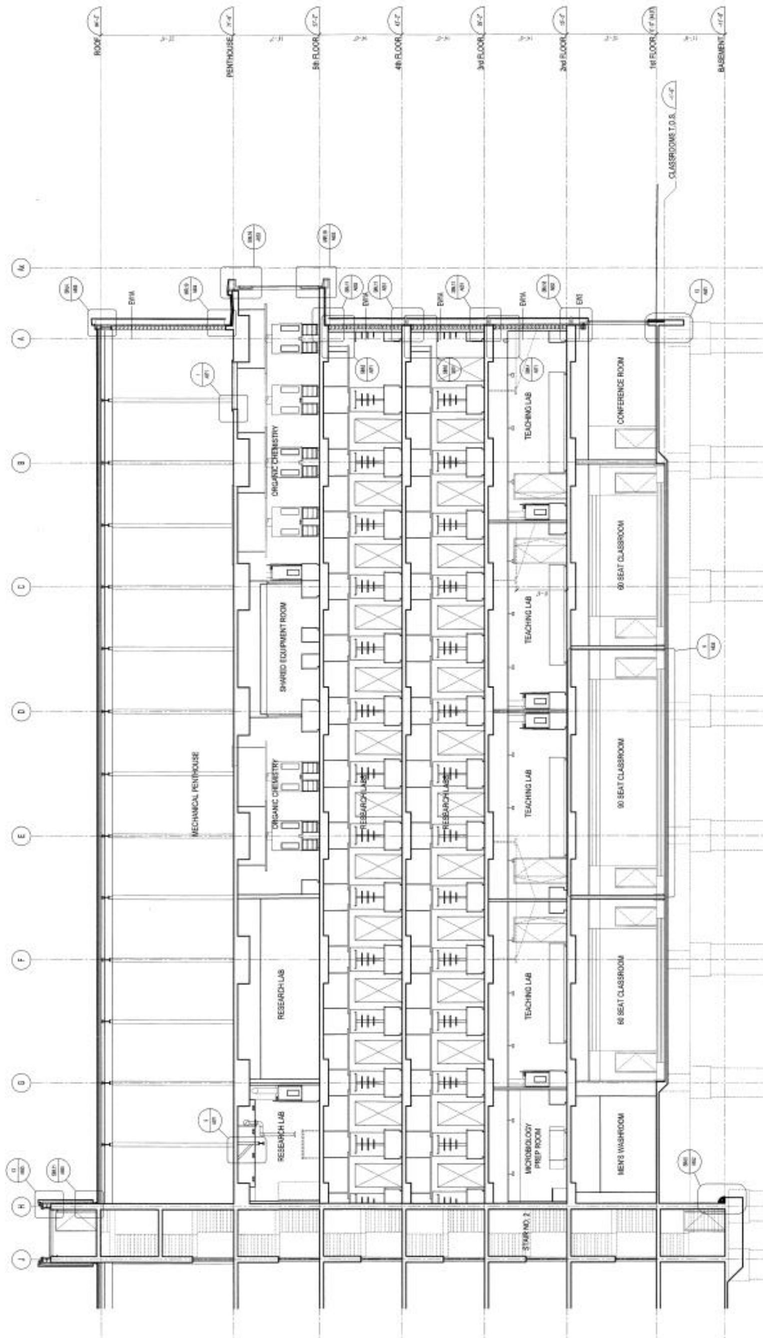


Figure A.2 Section 1 through portion of building at 0° rotation (see Figure 1), taken from 3/A401.

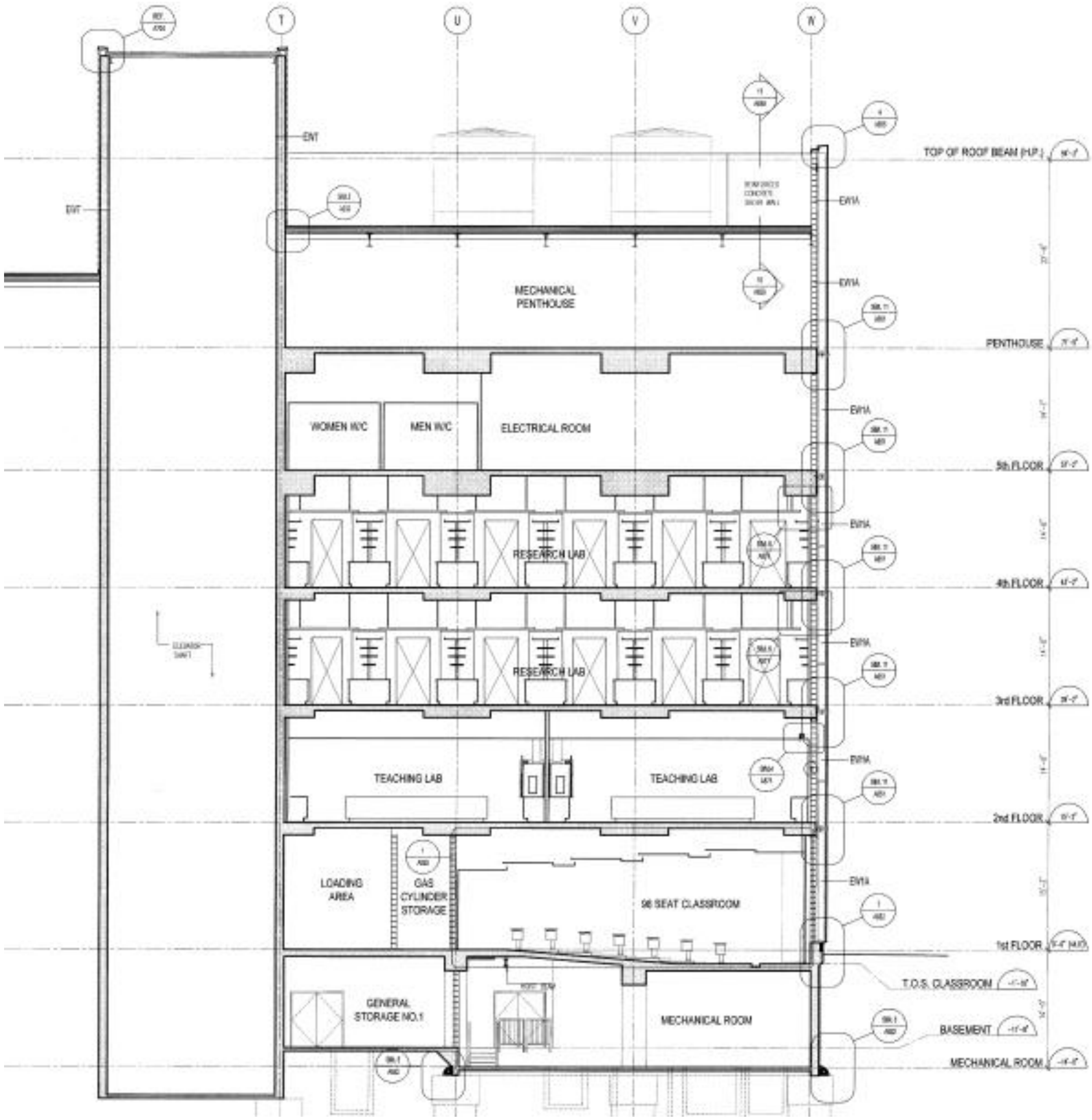


Figure A.3 Section 2 through portion of building at -15° rotation (see Figure 1), taken from 2/A402.

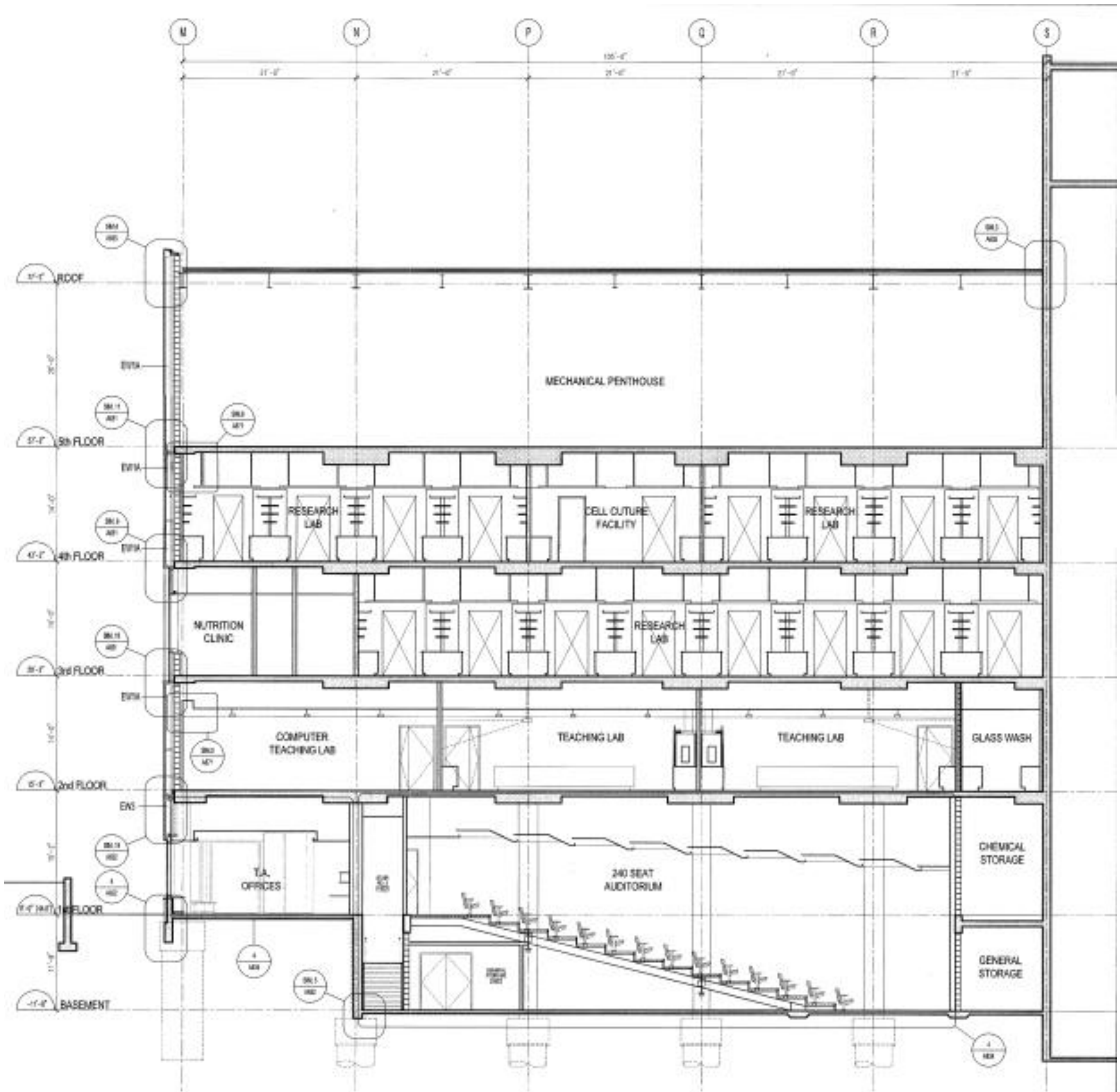


Figure A.4 Section 3 through portion of building at -45° rotation (see Figure 1), taken from 4/A402.

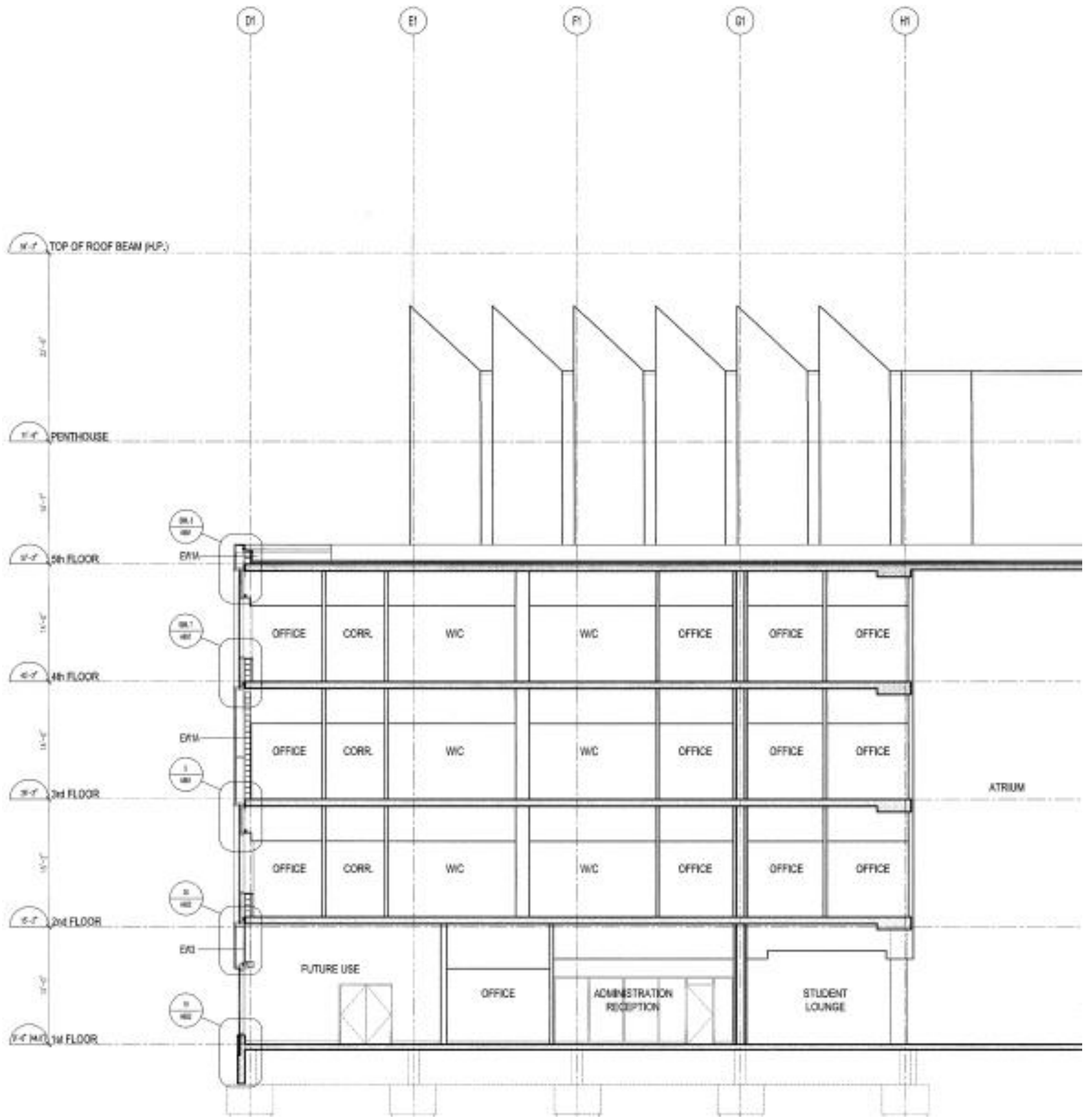


Figure A.5 Section 4 through portion of building at -20° rotation (see Figure 1), taken from 3/A403.

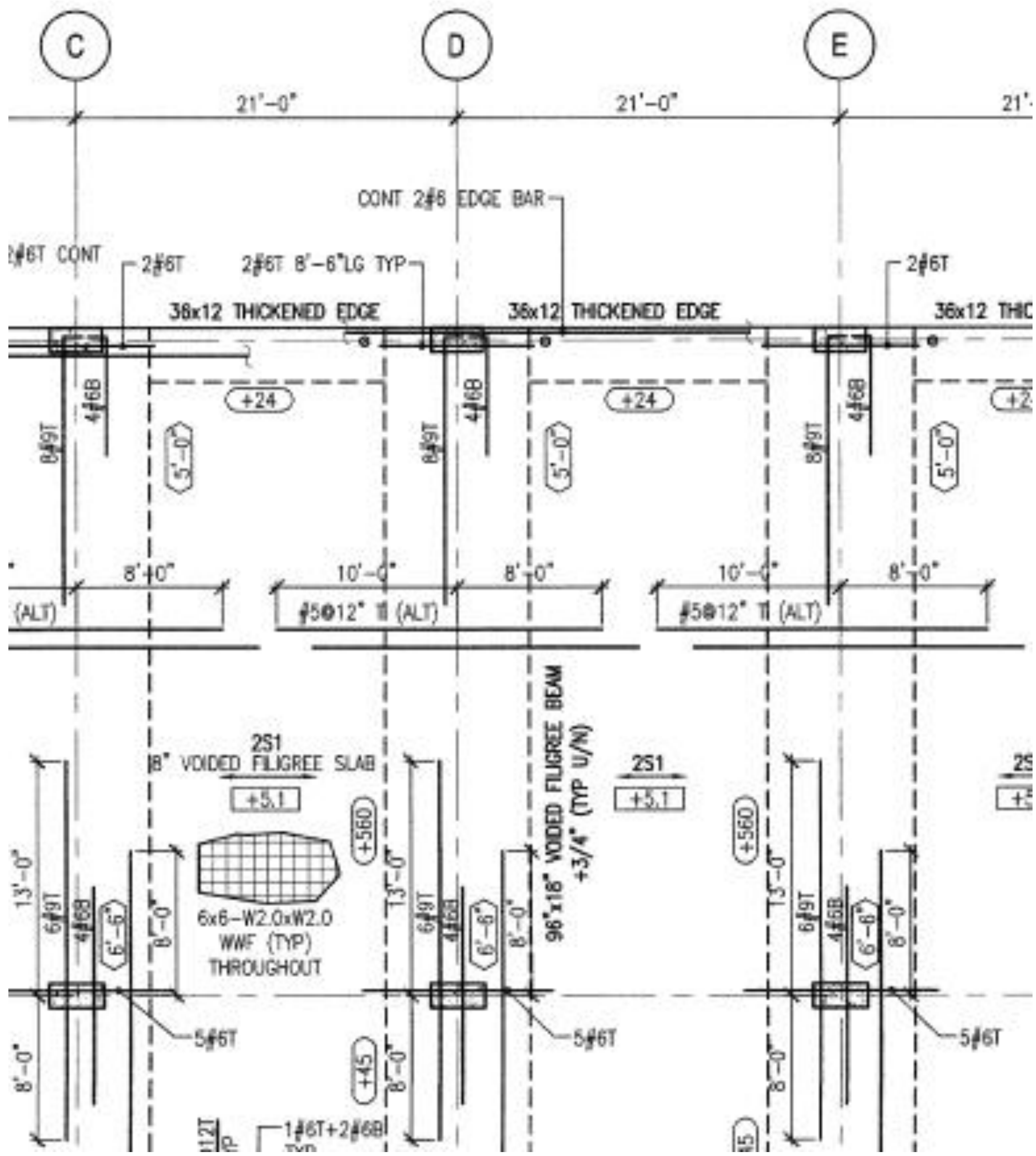
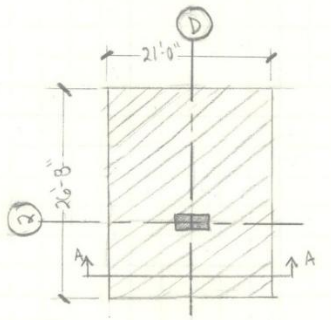


Figure A.6 Enlarged floor plan for the area in which the gravity checks were performed, taken from S202 (levels 2 through 4 are identical, and reinforcing is only displayed on level 2). Slab design moments are boxed (k-ft/ft), beam design moments are enclosed in an oval (k-ft), and the location of the first void in the beams with relation to the face of columns is enclosed in a prism-like shape.

Appendix B: Voided Filigree Slab/Beam Calculations

GRAV. CHECK → COL. D/2 | TECH 1 | Pg 1 OF 2



1 PARTIAL PLAN
NTS

▨ = TRIBUTARY AREA, A_T = 560 SF

ASSUMPTIONS
- VOIDED FILIGREE SLABS (V.F.S) 80% SOLID
- VOIDED FILIGREE BEAMS (V.F.B) 90% SOLID

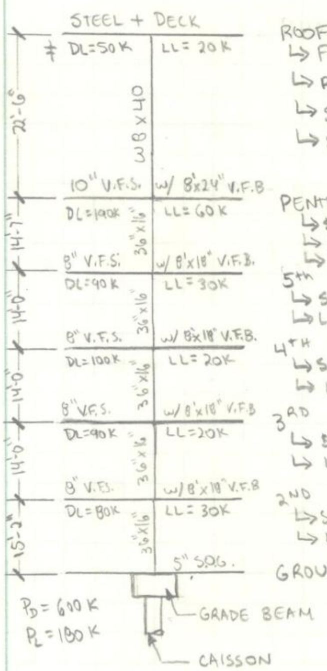
SELF WEIGHTS OF SLABS/BEAMS

10" V.F.S. w/ 8'x24" V.F.B. →

$$\left[\left(\frac{8'}{21'}\right)\left(\frac{24"}{12"}\right)(0.9)+\left(\frac{13'}{21'}\right)\left(\frac{10"}{12"}\right)(0.8)\right]150\text{ pcf} = 165\text{ psf}$$

8" V.F.S. w/ 8'x18" V.F.B →

$$\left[\left(\frac{8'}{21'}\right)\left(\frac{18"}{12"}\right)(0.9)+\left(\frac{13'}{21'}\right)\left(\frac{8"}{12"}\right)(0.8)\right]150\text{ pcf} = 130\text{ psf}$$



ROOF
↳ FRAMING = 15 psf
↳ ROOFING = 20 psf
↳ SDL = 50 psf
↳ SL = 23 psf *

PENTHOUSE
↳ SDL = 90 psf
↳ LL = 100 psf
↳ MECH. EQUIP. = 10 K

5th
↳ SDL = 15 psf
↳ LL = 80 psf

4th
↳ SDL = 15 psf
↳ LL = 80 psf

3rd
↳ SDL = 15 psf
↳ LL = 80 psf

2nd
↳ SDL = 15 psf
↳ LL = 80 psf

GROUND

2 SECTION A-A
NTS

NOTES:
* STRUCTURAL DRAWINGS USE 50 psf
‡ LOADS LISTED ON SECTION ARE SERVICE LOADS FROM THE COLUMN SCHEDULE, SHEET S301

COLUMN LOADS

ROOF
↳ $P_D = 560(15+20+50) = 47.6\text{ K} \rightarrow 50\text{ K}$ ✓
↳ $P_L = 560(23) = 12.9\text{ K} \rightarrow 15\text{ K}$ ✓

PENTHOUSE
↳ $P_D = 560(90+165)+40(22.5')+10{,}000 = 154\text{ K}$
↳ $P_L = 560(100) = 56\text{ K} \rightarrow 60\text{ K}$ ✓

5th
↳ $P_D = 560(15+130)+\left(\frac{36\times16}{144}\right)(150)(14.58') = 90\text{ K}$ ✓
↳ $LL_r = 0.25+\frac{15}{\sqrt{(4)(560)}} = 0.57 \rightarrow 0.6$ OK
$P_L = 560(0.6)(80) = 26.9\text{ K} \rightarrow 30\text{ K}$ ✓

4th
↳ $P_D = 560(15+130)+\left(\frac{36\times16}{144}\right)(150)(14') = 90\text{ K}$
↳ $LL_r = 0.25+\frac{15}{\sqrt{4(2)(560)}} = 0.47 \rightarrow 0.5$ OK
$P_L = 560(0.5)(80) = 22.4\text{ K} \rightarrow 20\text{ K}$ ✓

3rd
↳ $P_D = 90\text{ K}$ (SEE 4th) ✓
↳ $LL_r = 0.25+\frac{15}{\sqrt{4(3)(560)}} = 0.43 \rightarrow 0.4$ OK
$P_L = 560(0.4)(80) = 17.9\text{ K} \rightarrow 20\text{ K}$ ✓

2nd
↳ $P_D = 90\text{ K}$ (SEE 4th)
↳ $P_L = 20\text{ K}$ (SEE 3rd)

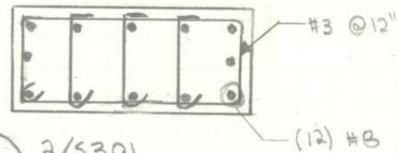
TOTALS
↳ $P_D = 564\text{ K}$
↳ $P_L = 165\text{ K}$
$> P_U = 1.2(564)+1.6(165)$ $\boxed{= 940\text{ K}}$ OK ✓

$P_{U,\,GIVEN} = 1.2(600)+1.6(180) = 1010\text{ K}$ (~7% DIFF)

GRAV. CHECK → COL. D/2 | TECH 1 | PG 2 OF 2

CHECK REINFORCING

$A_s = 12(0.79) = 9.48\ in^2$



3 2/S301
NTS

COLUMN IS CHECKED FOR PURE COMPRESSION BECAUSE IT IS AN INTERIOR COLUMN NOT IN A MOMENT FRAME OR NEAR A SHEAR WALL. THEREFORE, COLUMN ECCENTRICITY (e) IS NEGLIGIBLE.

$$\phi P_o = \phi\left(0.85 f'_c A_c + A_s f_y\right) = 0.65\left[0.85(5\ ksi)(16(36") - 9.48 in^2) + 9.48 in^2 (60 ksi)\right]$$
$$= 1935\ K$$

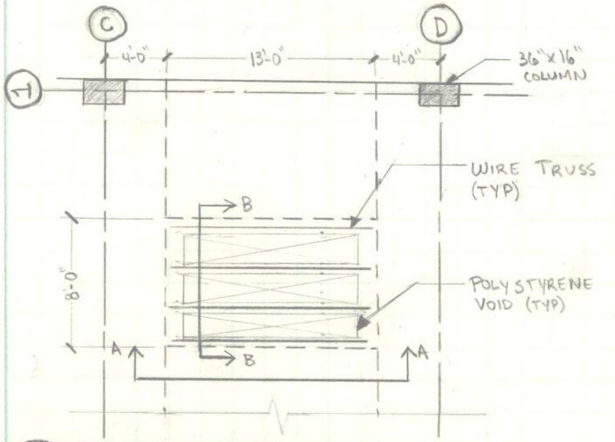
PURE COMPRESSION LIMITED BY α (ACCOUNTS FOR MINIMUM ECCENTRICITY)

$\phi P_n = \alpha \phi P_o = 0.8(1935 K) = 1548\ K >> P_u$ OK ✓

$\rho = \frac{9.48}{36(16)} = 0.016 > 1\%$ (MINIMUM ALLOWABLE ρ, PER ACI 318-08 SECTION 10.9.1)
OK ✓

GRAV. CHECK → V.F.S. | TECH 1 | pg 1 OF 3

CHECK AN INTERIOR VOIDED FILIGREE SLAB (V.F.S.) PANEL SPANNING BETWEEN COLUMN LINES C & D ON THE 4th LEVEL



1 PARTIAL PLAN
NTS

ASSUMPTIONS:
- VOIDED FILIGRE SLAB (V.F.S.) IS 80% SOLID
- VOIDED FILIGREE BEAMS (V.F.B.) ARE 90% SOLID

DESIGN MOMENT GIVEN ON STRUCTURAL DRAWINGS → $5.1\ \frac{k\text{-}ft}{ft}$

FROM COMPARISON WITH MOMENTS ON SLAB AT ADJACENT BAYS, IT SEEMS AS THOUGH ACI MOMENT COEFFICIENTS WERE USED

SPAN A-B $M_{u+} = 8.0\ \frac{k\text{-}ft}{ft} = \frac{w\ell^2}{11}$

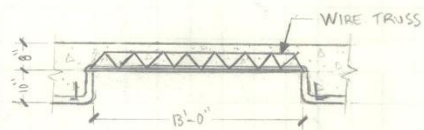
SPAN B-C $M_{u+} = 5.5\ \frac{k\text{-}ft}{ft} = \frac{w\ell^2}{16}$

$\frac{5.5}{8.0} = 0.6875 = \frac{11}{16}$ OK ✓

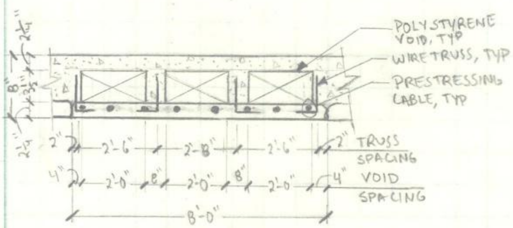
SPAN CD $M_u = 5.1\ \frac{k\text{-}ft}{ft}$

$\frac{5.1}{8} = \frac{11}{x} \Rightarrow x = 17.25$

$M_{u+} = \frac{w\ell^2}{17}$



2 SECTION A-A
NTS



3 SECTION B-B
NTS

LOADS FROM STRUCTURAL DRAWINGS →
SDL = 15 psf
LL = 80 psf

SLAB SELF-WEIGHT → $\left[\left(\frac{8''}{21'}\right)\left(\frac{18''}{12''}\right)(0.9) + \left(\frac{13'}{21'}\right)\left(\frac{8''}{12''}\right)(0.8)\right] 150\ pcf = 130\ psf$

CLEAR LENGTH → $\ell_n = 21' - 2\left(\frac{1}{2}\right)\left(\frac{36''}{12''}\right) = 18\ ft$

$w_D = 130 + 15 = 145\ plf/ft$
$w_L = 80\ plf/ft$
$\rangle\ w_u = 1.2(145) + 1.6(80) = 302\ plf/ft = 0.302\ klf/ft$

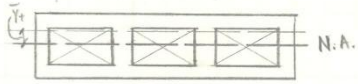
$M_{u+} = \frac{w_u \ell_n^2}{17} = \frac{0.302(18)^2}{17} = 5.76\ k\text{-}ft/ft \rightarrow M_{u,serv.} = \frac{5.76\ k\text{-}ft/ft}{1.5} = 3.84\ k\text{-}ft/ft$

$M_{u-} = \frac{w_u \ell_n^2}{11} = \frac{0.302(18)^2}{11} = -8.9\ k\text{-}ft/ft$

$V_u = 0.302(18')\left(\frac{1}{2}\right) = 2.72\ k/ft$

GRAV. CHECK → V.F.S. TECH 1 pg 2 of 3

POSITIVE MOMENT CHECK



BOTTOM REINFORCING → (8) 3/8" ⌀ P.S. STRANDS

$P_e = 0.085\ in^2/strand\ (8\ strands)(0.6)(270\ ksi) = 110.2\ K$

$$\bar{y}_t = \frac{(4''+4''+8''+8'')(1.75'')(0.875'') + 2.25''(96'')(2.875'')}{(4''+4''+8''+8'')(1.75'') + 2.25''(96'')} = 2.55\ in$$

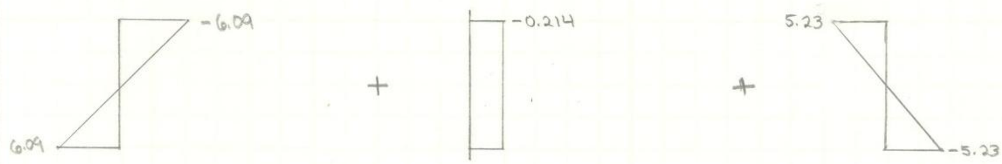
$$I_t = \frac{(4''+4''+8''+8'')(1.75'')^3}{12} + (4''+4''+8''+8'')(1.75'')(2.55''-0.875'')^2 + \frac{96''(2.25'')^3}{12} + 96''(2.25'')(2.55''-2.875'')^2$$

$= 242.5\ in^4$

$$S_t = \frac{I_t}{\bar{y}_t} = \frac{242.5\ in^4}{2.55\ in} = 60.625\ in^3 = S_b$$

$A = [(4''+4''+8''+8'')(1.75'') + (96'')(2.25'')]\ 2 = 516\ in^2$

$M_{SERV} = 3.84\ \frac{k\text{-}ft}{ft}\ (8\ ft)(12\ in/ft) = 369\ k\text{-}in$



$$\frac{M}{S} = \frac{369\ k\text{-}in}{60.625\ in^3} = 6.09\ ksi \quad + \quad \frac{P_e}{A} = \frac{110.2\ k}{516\ in^2} = 0.214\ ksi \quad + \quad \frac{P_e e}{S} = \frac{110.2\ k(6.875''-4'')}{60.625\ in^3} = 5.23\ ksi$$

$f_t = -6.09 - 0.214 + 5.23 = -1.074\ ksi \quad < 0.45 f'_c = 0.45(5) = 2.25\ ksi$ OK ✓

$f_b = 6.09 - 0.214 - 5.23 = 0.646\ ksi \quad < \underbrace{12\sqrt{f'_c}}_{\text{LIKELY DESIGNED AS CLASS T}} = 0.849\ ksi$ OK ✓

$f_{t,u} = 1.5(-1.074) = -1.611\ ksi$ $\boxed{\text{OK ✓}}$

$f_{b,u} = 1.5(0.646) = 0.969\ ksi$

NEGATIVE MOMENT CHECK

REINFORCING PROVIDED → #5 @ 12" WITH WWF (FOR SHRINKAGE & TEMP.)

$A_s = 0.31\ in^2/ft$

$d = 8 - 0.75 - \frac{1}{2}(5/8'') = 6.94\ in$

$$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{0.31\ in^2/ft\ (60\ ksi)}{0.85\ (5\ ksi)(12\ in/ft)} = 0.365\ in$$ ← IN SOLID PORTION OF SLAB ∴ BEHAVES AS A RECTANGULAR SECT.

$\phi M_n = \phi A_s F_y (d - \frac{a}{2}) = 0.9(0.31\ in^2/ft)(60\ ksi)(6.94'' - \frac{0.365''}{2}) = 9.43\ k\text{-}ft/ft > M_u = 8.9\ \frac{k\text{-}ft}{ft}$

$\boxed{\text{OK ✓}}$

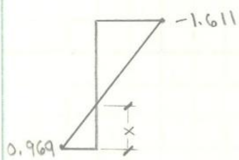
GRAV. CHECK → V.F.S. | TECH. 1 | pg 3 OF 3

VERTICAL SHEAR CHECK

$\phi V_c = \phi 2\sqrt{f'_c}\, b_w d = 0.75(2)\sqrt{5000}\,(12\text{ in/ft})(6.94\text{ in})\left(\frac{1}{1000}\right) = 8.83\ \text{k/ft}$

$\phi V_c > V_u = 2.72\ \text{k/ft}$ $\boxed{\text{OK}✓}$

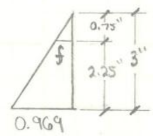
HORIZONTAL SHEAR CHECK



$\frac{0.969}{x} = \frac{1.611}{8-x}$

$0.969(8) - 0.969x = 1.611x$

$x = 3.00\text{ in}$



$\frac{f}{0.75} = \frac{0.969}{3}$

$f = 0.242\text{ ksi}$

$V_{u,h} = \frac{1}{2}(0.242\text{ ksi})(12\text{ in/ft})(0.75\text{ in}) = 1.09\ \text{k/ft}$

FROM ACI 318-08 SECT. 17.5.3.1,

$\phi V_{nc,h} = \phi\, 80\, b_v d = 0.75(80)(12\text{ in/ft})(6.94")\left(\frac{1}{1000}\right) = 5.0\ \text{k/ft} > V_{u,h}$ $\boxed{\text{OK}✓}$

DEFLECTION

PER ACI 318-08 SECT. 9.5.5.1, COMPOSITE MEMBERS CAN BE CONSIDERED EQUIVALENT TO A MONOLITHICALLY CAST MEMBER DUE TO USE OF SHORING DURING CASTING OF ADDITIONAL CONCRETE.

TRY CALCULATING DEFLECTIONS AS IF SIMPLY SUPPORTED
↳ THIS IS CONSERVATIVE, AND IF IT PASSES DEFLECTION REQUIREMENTS, NO MORE SPECIFIC/COMPLICATED CALCULATIONS WILL BE REQUIRED

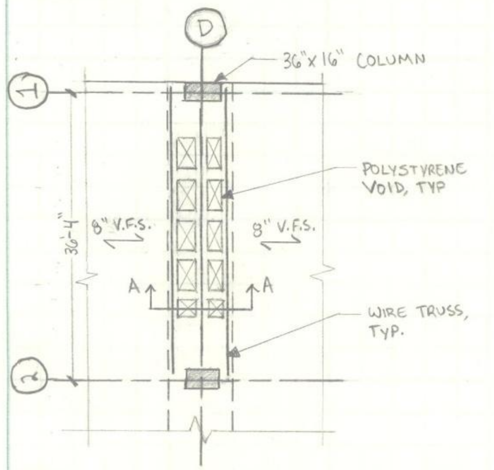
$\Delta_{LL} = \frac{5(1.6)(0.080\ \text{k/ft})(18')^4(1728)}{384(57\sqrt{5000})[2(242.5)]} = 0.155\text{ in} < \Delta_{LL,max} = \frac{18' \times 12\text{ in/ft}}{360} = 0.6\text{ in}$ OK✓

$\Delta_{TL} = \frac{5(0.302\ \text{k/ft})(18')^4(1728)}{384(57\sqrt{5000})[2(242.5)]} = 0.365\text{ in} < \Delta_{TL,max} = \frac{18' \times 12\text{ in/ft}}{240} = 0.9\text{ in}$ OK✓

$\boxed{\text{OK}✓}$

GRAV. CHECK → V.F.B. | TECH 1 | PG 1 OF 5

CHECK 36'-4" BEAM SPANNING BETWEEN COLUMN LINES 1 & 2 ALONG COLUMN LINE D ON THE 4th LEVEL.



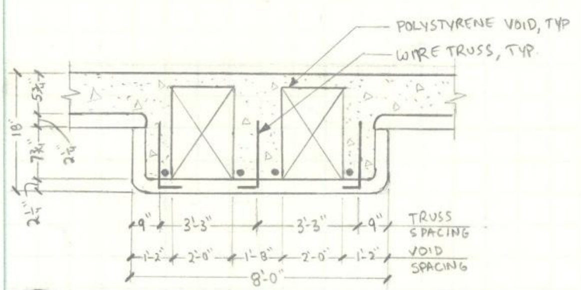
1 PARTIAL PLAN
NTS

ASSUMPTIONS:
- VOIDED FILIGREE SLAB (V.F.S.) IS 80% SOLID
- VOIDED FILIGREE BEAM (V.F.B.) IS 90% SOLID

POSITIVE DESIGN MOMENT GIVEN ON STRUCTURAL DRAWINGS → 560 k-ft

IT SEEMS LIKELY THAT ACI MOMENT COEFFICIENTS WOULD BE USED FOR THE BEAMS, SINCE THEY WERE USED FOR THE SLAB. THEREFORE, THIS IS HOW I CHECKED THE BEAM.

LOADS FROM STRUCTURAL DRAWINGS →
SDL = 15 psf
LL = 80 psf



2 SECTION A-A
NTS

SLAB SELF-WEIGHT → $\left[\left(\frac{9'}{21'}\right)\left(\frac{18"}{12"}\right)(0.9) + \left(\frac{13'}{21'}\right)\left(\frac{8"}{12"}\right)(0.8)\right] 150 \text{ pcf} = 130 \text{ psf}$

CLEAR LENGTH → $l_n = 36.33' - 2\left(\frac{1}{2}\right)\left(\frac{16"}{12"}\right) = 35 \text{ ft}$

$LL_r = 0.25 + \frac{15}{\sqrt{2(36.33)(21)}} = 0.63 > 0.5$ ok ✓

$1526 > 400$ ok ✓

$w_D = 21'(130+15) = 3045 \text{ plf} = 3.045 \text{ klf}$
$w_L = 21'(0.63)(80) = 1058 \text{ plf} = 1.058 \text{ klf}$
$w_u = 1.2(3.045) + 1.6(1.058) = 5.3468 \text{ klf}$

$M_{u,+} = \frac{w_u l_n^2}{11} = \frac{5.3468(35')^2}{11} = 595 \text{ k-ft} \rightarrow M_{u,serv} = \frac{595 \text{ k-ft}}{1.5} = 397 \text{ k-ft}$

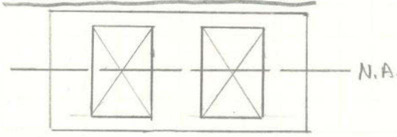
$M_{u,-,1} = \frac{w_u l_n^2}{16} = \frac{5.3468(35)^2}{16} = 409 \text{ k-ft}$

$M_{u,-,2} = \frac{w_u l_n^2}{9} = \frac{5.3468(35)^2}{9} = 728 \text{ k-ft}$

$V_u = 5.3468(35')\left(\frac{1}{2}\right) = 93.6 \text{ k}$

GRAV. CHECK → V.F.B. | TECH 1 | pg 2 of 5

POSITIVE MOMENT CHECK



BOTTOM PRESTRESSING → (26) 3/8" ø STRANDS

$P_e = 0.085 \text{ in}^2/\text{strand}\,(26 \text{ strands})(0.6)(270 \text{ ksi}) = 358 \text{ k}$

BOTTOM REINFORCING → (4) #6

$A_s = 0.44(4) = 1.76 \text{ in}^2$ $\quad d = 18'' - 2.5'' = 15.5''$

$a = \dfrac{A_s F_y}{0.85 f'_c b} = \dfrac{1.76(60)}{0.85(5)(96)} = 0.259 \text{ in}$

$\phi M_n = \phi A_s F_y \left(d - \frac{a}{2}\right) = 0.9(1.76)(60)\left(15.5 - \frac{0.259}{2}\right) = 122 \text{ k-ft}$

$M_u = 397 - 122 = 275 \text{ k-ft} \times 12 \text{ in/ft} = 3300 \text{ k-in}$

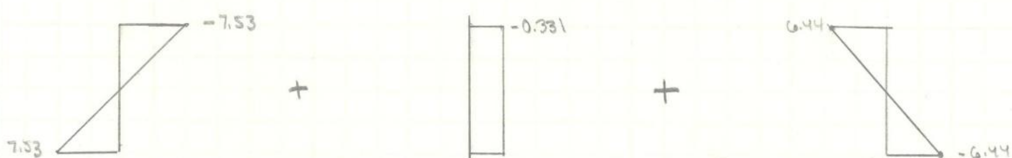
$\bar{y}_+ = \dfrac{(14''+14''+20'')(6.75'')(3.375'') + 96''(2.25'')(7.875'')}{(14''+14''+20'')(6.75'') + 96''(2.25'')} = 5.175 \text{ in}$

$I_+ = \dfrac{(14''+14''+20'')(6.75'')^3}{12} + (14''+14''+20'')(6.75'')(5.175''-3.375'')^2 + \dfrac{96''(2.25'')^3}{12} + 96''(2.25)(7.875''-5.175'')^2$

$= 3946 \text{ in}^4$

$S_+ = \dfrac{I}{y} = \dfrac{3946 \text{ in}^4}{9 \text{ in}} = 438 \text{ in}^3$

$A = \left[(96'')(2.25'') + (14''+14''+20'')(6.75'')\right] 2 = 1080 \text{ in}^2$



$\dfrac{M}{S} = \dfrac{3300 \text{ k-in}}{438 \text{ k-in}} = 7.53 \text{ ksi} \quad + \quad \dfrac{P_e}{A} = \dfrac{358 \text{ k}}{1080 \text{ k}} = 0.331 \text{ ksi} \quad + \quad \dfrac{P_e e}{S} = \dfrac{358 \text{ k}\,(16.875''-9'')}{438 \text{ in}^3} = 6.44 \text{ ksi}$

$f_t = -7.53 - 0.331 + 6.44 = 1.42 \text{ ksi} < 0.45 f'_c = 0.45(5) = 2.25 \text{ ksi}$ OK ✓

$f_b = 7.53 - 0.331 - 6.44 = 0.759 \text{ ksi} < 12\sqrt{f'_c} = 12\sqrt{5000} = 0.849 \text{ ksi}$ OK ✓

$f_{t,u} = 1.5(1.42) = 2.13 \text{ ksi}$ $\quad \boxed{\text{OK ✓}}$

$f_{b,u} = 1.5(0.759) = 1.14 \text{ ksi}$

GRAV. CHECK → V.F.B. | TECH 1 | PG 3 OF 5

NEGATIVE MOMENT CHECK @ 1

REBAR PROVIDED → (8) #9

$A_s = 8(1.0) = 8.0 \text{ in}^2$

$d = 18 - 0.75 - \frac{1}{2}(\frac{9}{8}") = 16.69 \text{ in}$

$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{8(60)}{0.85(5)(96)} = 1.18 \text{ in}$

$\phi M_n = \phi A_s F_y (d - \frac{a}{2}) = 0.9(8 \text{ in}^2)(60 \text{ ksi})(16.69" - \frac{1.18"}{2}) = 579 \text{ k·ft} > M_u = 409 \text{ k·ft}$ $\boxed{\text{OK ✓}}$

NEGATIVE MOMENT CHECK @ 2

REBAR PROVIDED → (12) #9

$A_s = 12(1.0) = 12 \text{ in}^2$

$d = 16.69 \text{ in}$

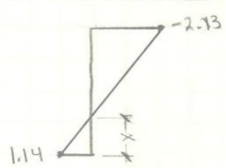
$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{12(60)}{0.85(5)(96)} = 1.76 \text{ in}$

$\phi M_n = \phi A_s F_y (d - \frac{a}{2}) = 0.9(12 \text{ in}^2)(60 \text{ ksi})(16.69" - \frac{1.76"}{2}) = 854 \text{ k·ft} > M_u = 728 \text{ k·ft}$ $\boxed{\text{OK ✓}}$

VERTICAL SHEAR

$\phi V_n = \phi 2\sqrt{f'_c}\, b_w d = 0.75(2)\sqrt{5000}(96")(16.69") = 170 \text{ k} > V_u = 93.6 \text{ k}$ $\boxed{\text{OK ✓}}$

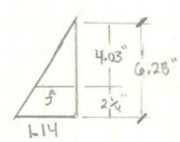
HORIZONTAL SHEAR



$\frac{1.14}{x} = \frac{2.13}{18 - x}$

$1.14(18) - 1.14x = 2.13x$

$x = 6.28 \text{ in}$



$\frac{1.14}{6.28} = \frac{f}{4.03"}$

$f = 0.732 \text{ ksi}$

$V_{u,h} = \frac{1}{2}(0.732 \text{ ksi})(96")(4.03") = 142 \text{ K}$

$\phi V_{nc,h} = \phi 80 b_w d = 0.75(80)(96")(16.69")(\frac{1}{1000}) = 96.1 \text{ K} < V_{u,h}$ ∴ MUST CONSIDER STEEL

FROM ESR 96-14, A TYPICAL FILIGREE TRUSS IS MADE OF 6mm DIAMETER WIRE AND PENETRATES THE HORIZONTAL SHEAR PLANE EVERY 5". THERE ARE THREE TRUSSES.

GRAV. CHECK → V.F.B. | TECH 1 | pg 4 of 5

$0.6\ cm \times \frac{1\ in}{2.54\ cm} = 0.236\ in$

$A = \frac{\pi}{4} d^2 = \frac{\pi}{4}(0.236\ in)^2 = 0.0438\ in^2$ $\quad A_s = 3(0.0438) = 0.1314\ in^2$

$\rho_v = \frac{A_s}{b_v s} = \frac{0.1314}{96(5)} = 0.000274$

$\phi V_{n,h} = \phi(260 + 0.6 \rho_v f_y) \lambda b_v d = 0.75[260 + 0.6(0.000274)(60{,}000)](96")(16.69")(\frac{1}{1000})$ (λ: 1.0 FOR NWC)

$= 324\ k > V_u = 142\ k$ $\boxed{OK ✓}$

DEFLECTION

PER ACI 318-08 SECT. 9.5.5.1, COMPOSITE MEMBERS CAN BE CONSIDERED EQUIVALENT TO A MONOLITHICALLY CAST MEMBER DUE TO USE OF SHORING DURING CASTING OF ADDITIONAL CONCRETE

TRY CALCULATING DEFLECTIONS AS SIMPLY-SUPPORTED
↳ THIS IS CONSERVATIVE, AND IF IT PASSES DEFLECTION REQUIREMENTS, NO MORE SPECIFIC/COMPLICATED CALCULATIONS WILL BE REQUIRED

$\Delta_{LL} = \frac{5(1.6)(1.058\ klf)(35')^4(1728)}{384(57\sqrt{5000}')[2(3946\ in^4)]} = 1.80\ in > \Delta_{LL,max} = \frac{35' \times 12}{360} = 1.17 in$

∴ MORE ACCURATE CALCULATION REQUIRED.

$M_{LL@1} = \frac{1.6(1.058)(35)^2}{16} = 130\ k \cdot ft$ $\quad M_{LL@2} = \frac{1.6(1.058)(35)^2}{9} = 230.4\ k \cdot ft$

FROM AISC TBL 3-23, FIG. 32,

Δ_{max} OCCURS AT $x = \frac{\ell}{2} + \frac{M_1 - M_2}{w\ell} = \frac{35}{2} + \frac{130 - 230.4}{1.6928(35)} = 15.81\ ft$

$\Delta_{LL} = \frac{wx}{24EI}\left[x^3 - \left(2\ell + \frac{4M_1 - 4M_2}{w\ell}\right)x^2 + \frac{12M_1}{w}x + \ell^3 - \frac{8M_1\ell}{w} - \frac{4M_2\ell}{w}\right]$

$= \frac{1.6928(15.81)(1728)}{24(57\sqrt{5000})[2(3946)]}\left[15.81^3 - \left(2(35') + \frac{4(130) - 4(230.4)}{1.6928(35)}\right)15.81^2 + \frac{12(130)(15.81)}{1.6928} + 35^3 - \frac{8(130)(35)}{1.6928} - \frac{4(230.4)(35)}{1.6928}\right]$

$\Delta_{LL} = 0.225\ in < \Delta_{LL,max}$ OK ✓

ASSUME MORE ACCURATE CALCULATION REQUIRED FOR TOTAL LOAD DEFLECTION ALSO

GRAV. CHECK → V.F.B. | TECH 1 | pg 5 OF 5

$M_{TL@1} = 409$ k·ft $\qquad M_{TL@2} = 728$ k·ft

Δ_{max} OCCURS @ $x = \frac{l}{2} + \frac{M_1 - M_2}{wl} = \frac{35}{2} + \frac{409-728}{5.3468(35)} = 15.80$ ft

$$\Delta_{TL} = \frac{5.3468(15.8)(1728)}{24(57\sqrt{5000})[2(3946)]}\left[15.8^3 - \left(2(35') + \frac{4(409) - 4(728)}{5.3468(35)}\right)15.8^2 + \frac{12(409)(15.8)}{5.3468} + 35^3 - \frac{8(409)(35)}{5.3468} - \frac{4(728)(35)}{5.3468}\right]$$

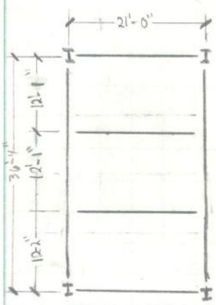
$\Delta_{TL} = 0.701$ in $\underset{\text{OK ✓}}{<}$ $\Delta_{TL,max} = \frac{35' \times 12}{240} = 1.75$ in

$\boxed{\text{OK ✓}}$

Appendix C: Composite Steel Calculations

COMPOSITE STEEL | TECH 2 | Pg 1 OF 4

CHOOSE DECK → USE VULCRAFT MANUAL (CANAM NOT READILY ACCESSIBLE)
↳ USE 1.5" OR 3" DECK (SIZES BEING USED ON PROJECT ALREADY)
↳ 4.5" TOPPING -OR- SPRAY FIREPROOFING ON BOTTOM OF DECK FOR 2 HR. FIRE RATING
↳ SPANS ARE VERY LONG, CHOOSE TO FIREPROOF TO REDUCE WEIGHT
↳ SUPERIMPOSED LOAD
↳ SDL = 15 psf
↳ LL = 80 psf
> 95 psf



→ DESIGN FOR 2-SPAN CONDITION FOR SAFETY
→ USE 3.5" TOPPING

1.5 VLR (pg. 50 IN CATALOG, 56 psf FOR 5" TOTAL THICKNESS)
↳ SPAN NOT ACCEPTABLE

3VLI (pg. 54 IN CATALOG, 63 psf FOR 6.5" TOTAL THICKNESS)
↳ SPAN OK FOR > 3VLI 19
↳ LOAD OK FOR > 3VLI 20

CHOOSE $\boxed{3VLI19}$ w/ TOTAL THICKNESS = $6\frac{1}{2}$" ∴ S.W. = 63 psf

DESIGN INTERIOR BEAM
↳ ASSUME SIMPLY SUPPORTED
↳ ADD 5 psf FOR BEAM SELF-WEIGHT
↳ CONSIDER CAMBER TO REDUCE STRUCTURE DEPTH
↳ $C_{min} = 1/4$"
↳ $C_{max} = 1$"
} PRACTICAL LIMITS

$w_D = (15\text{ psf} + 63\text{ psf} + 5\text{ psf})(12.083\text{ ft}) = 1003\text{ plf} = 1.003\text{ klf}$

$LL_r = 0.25 + \frac{15}{\sqrt{\underbrace{2(12.083)(21)}_{=507 > 400 \text{ OK}}}} = 0.92$

$w_L = 80\text{ psf}(0.92)(12.083\text{ ft}) = 889\text{ plf} = 0.889\text{ klf}$

$w_u = 1.2(1.003) + 1.6(0.889) = 2.63\text{ klf}$

$M_u = \frac{w_u \ell^2}{8} = \frac{2.63\text{ klf}(21\text{ ft})^2}{8} = 145\text{ k-ft}$

$V_u = 2.63\text{ klf}(21\text{ ft})(1/2) = 27.6\text{ k}$

FROM AISC TBL. 3-21, FIND Q_n
↳ DECK ⊥
↳ WEAK POSITION
↳ 1 STUD PER RIB
↳ 3/4" ∅
↳ $f'_c = 3$ ksi
↳ $w_c = 145$ pcf
} $Q_n = 17.2$ K

COMPOSITE STEEL | TECH 2 | pg 2 OF 4

$\Delta_{LL,max} = \frac{l}{360} + 1 = \frac{21' \times 12\ ft/in}{360} + 1 = 1.7\ in$

$\Delta_{LL} = \frac{5 w_L l^4 (1728)}{384\ E\ I_{LB}}$ $\therefore$ $I_{LB,min} = \frac{5 w_L l^4 (1728)}{384\ E\ \Delta_{LL,max}}$

$I_{LB,min} = \frac{5[1.6(0.889\ klf)](21\ ft)^4(1728)}{384\ (29000\ ksi)(1.7 in)} = 126\ in^4$

USE AISC TABLES 3-19 & 3-20 TO FIND OPTIONS
↳ ASSUME $Y2 = 5.5 in$

① W10x17 w/ $\Sigma Q_n = 150$ K $\phi M_n = 156$ k-ft & $I_{LB} = 291\ in^4$
↳ % COMPOSITE = 60%
↳ # OF STUDS = $150/17.2 = 8.7 = 9$ STUDS TOTAL = 18 STUDS < 21 ok✓
↳ ECONOMY ⇒ 17 (21) + 18 (10) = 537

② W10x19 w/ $\Sigma Q_n = 122$ K $\phi M_n = 156$ k-ft, & $I_{LB} = 288\ in^4$
↳ % COMPOSITE = 43.4%
↳ # OF STUDS = $122/17.2 = 7.1 = 8$ STUDS TOTAL = 16 STUDS < 21 ok✓
↳ ECONOMY → 19(21) + 16(10) = 559

③ W10x22 w/ $\Sigma Q_n = 81.1$ K $\phi M_n = 152$ k-ft, & $I_{LB} = 263\ in^4$
↳ % COMPOSITE = 25%
↳ # OF STUDS = $81.1/17.2 = 4.7 = 5$ STUDS TOTAL = 10 STUDS < 21 ok✓
↳ ECONOMY → 22(21) + 10(10) = 562

DO NOT TRY W12's BECAUSE W10's WORK AND SHALLOWER IS BETTER IN THIS CASE. TRY W10x22 (Δ OF BARE BEAM USUALLY CONTROLS)

$\Delta_{LL} = \frac{5[1.6(0.889\ klf)](21\ ft)^4(1728)}{384\ (29000\ ksi)(263\ in^4)} = 0.816\ in - 0.7 in = 0.116 in$
$\therefore\ c = 1/4''$

$\Delta_{TL} = \frac{5(2.63\ klf)(21\ ft)^4(1728)}{384\ (29000\ ksi)(263\ in^4)} = 1.51\ in - \frac{21 \times 12}{240} = 0.46\ in$
$\therefore\ c = 1/2''$

$M_{u,const} = \frac{[1.2(0.068) + 1.6(0.020)](12.083')(21\ ft)^2}{8} = 75.7$ k-ft

$\phi M_{n,BARE\ BEAM} = 97.5$ k-ft $> M_{u,const}$ OK✓

$\Delta_{const} = \frac{5[1.2(0.068)+1.6(0.020)](12.083ft)(21\ ft)^4(1728)}{384\ (29000)(118)} = 1.76\ in - \frac{21 \times 12}{240} = 0.71 in$
$\therefore\ c = 3/4 in$

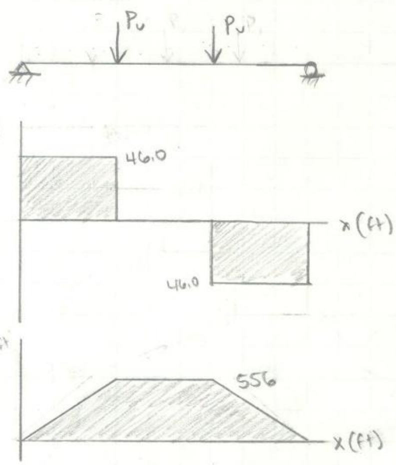
$\phi V_n = 73.2\ K > V_u$ OK✓

CHECK S.W. ASSUMPTION → $\frac{22 plf}{12.083\ ft} = 1.82$ psf < 5 psf OK✓

$\boxed{\text{USE W10x22 w/ 10 STUDS AND } c = 3/4''}$

COMPOSITE STEEL | TECH 2 | pg 3 of 4

DESIGN GIRDER
↳ ASSUME SIMPLY SUPPORTED
↳ ADD 1 K TO EACH POINT LOAD FOR GIRDER SELF-WEIGHT
↳ CONSIDER CAMBER TO REDUCE STRUCTURE DEPTH
↳ $C_{min} = 1/4"$
↳ $C_{max} = 1 1/2"$
} PRACTICAL LIMITS



$P_D = 1.003$ klf (21 ft) = 21.1 K

$LLr = 0.25 + \frac{15}{\sqrt{2(21)(36.33)}} = 0.634 > 0.5$ ok ✓

1526 > 400 ok ✓

$P_L = 0.08$ ksf (12.083 ft)(21 ft)(0.634) = 12.9 K

$P_U = 1.2(21.1) + 1.6(12.9) + 1 = 46.0$ K

$M_U = 556$ K-ft, $V_U = 46.0$ K

FROM AISC TBL 3-21, FIND Q_n
↳ DECK ∥
↳ $\frac{w_r}{h_r} = \frac{4.75}{3} = 1.58 > 1.5$
↳ 3/4" ∅
↳ $f'_c = 3$ ksi
↳ $w_c = 145$ pcf
} $Q_n = 21.0$ K

$\Delta_{LL,max} = \frac{l}{360} + 1 = \frac{36.33 \times 12}{360} + 1.5 = 2.71$ in

FROM AISC TBL 3-22 a WITH n = 4, $\Delta_{max} = \frac{0.036 P l^3}{EI}$ ∴ $I_{LB,min} = \frac{0.036 P l^3}{E \Delta_{LL,max}}$

$I_{LB,min} = \frac{0.036[1.6(12.9\text{ k})](36.33\text{ ft})^3(1728)}{29000\text{ ksi }(2.71\text{ in})} = 783$ in^4

$I_{BARE\ BEAM,MIN} = \frac{0.036[1.2(0.068)+1.6(0.02)](12.083')(21')(36.33\text{ ft})^3(1728)}{29000(3.32\text{ in})} = 893$ in^4

CONTROLS

USE AISC TBLS 3-19 & 3-20 TO FIND OPTIONS
↳ ASSUME Y2 = 5 in

① W18x60 w/ $\Sigma Q_n = 220$ K, $\phi M_n = 650$ K-ft & $I_{LB} = 1690$ in^4
↳ % COMPOSITE = 25%
↳ # OF STUDS = 220/20.1 = 10.9 = 11 STUDS TOTAL = 22 STUDS < 36' OK ✓
↳ ECONOMY = 60(36.33) + 22(10) = 2400

NEXT SIZE LISTED IS A W21. TO KEEP FLOOR DEPTHS TO A MINIMUM, TRY W18x60.

$\Delta_{LL} = \frac{0.036[1.6(12.9\text{ k})](36.33\text{ ft})^3(1728)}{29000\text{ ksi }(1690\text{ in}^4)} = 1.26\text{ in} - \frac{36.33 \times 12}{360} = 0.05$ in

∴ c = 1/4"

COMPOSITE STEEL | TECH 2 | pg 4 OF 4

$$\Delta_{TL} = \frac{0.036(46.0\,k)(36.33)^3(1728)}{29000\,ksi\,(1690)} = 2.8\,in - 1.82\,in = 0.98\,in \quad \therefore c = 1''$$

$$M_{u,const} = 0.33[1.2(0.068) + 1.6(0.02)](21\,ft)(12.083\,ft)(36.33\,ft) = 346\,k\text{-}ft$$

$\phi M_{n,\,BARE\,BEAM} = 461\,k\text{-}ft > M_{u\,const}$ OK ✓

$$\Delta_{CONST} = \frac{0.036[1.2(0.068)+1.6(0.02)](21\,ft)(12.083\,ft)(36.33\,ft)^3(1728)}{29000\,(984\,in^4)} = 3.01\,in$$

$$= 3.01 - 1.82 = 1.19\,in \quad \therefore c = 1\tfrac{1}{4}''$$

$\phi V_n = 227\,k > V_u$ OK ✓

CHECK S.W. ASSUMPTION → 60 plf × 12.083 ft = 725 lbs = 0.73 k < 1 k OK ✓

$\boxed{\text{USE W18×60 w/ 22 STUDS AND } c = 1\tfrac{1}{4}''}$

DESIGN INTERIOR COLUMN

$P_u = 1.5(46.0) + 2(27.6) = 124\,k$ @ 14 ft UNBRACED LENGTH

USE $k = 2.0$ (VERY CONSERVATIVE) $KL = 28\,ft$

FROM AISC TBL 4-1 ON pg 4-18,

$\boxed{\text{USE W12×53}}$ w/ $\phi P_n = 192\,k$

		HAND	RAM
BEAM	SIZE	W10x22	W12x14
	STUDS	10	18
	C	3/4"	1"
GIRDER	SIZE	W18x60	W21x50
	STUDS	22	22
	C	1 1/4"	1"
COL	SIZE	W12x53	W12x65

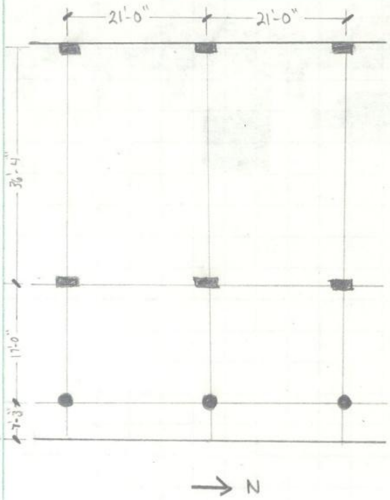
RAM RESULTS DIFFER BECAUSE RAM DEFAULTS TO THE MOST ECONOMICAL SECTION, WHEREAS I WAS SEEKING TO MINIMIZE DEPTHS.

THE COLUMN DIFFERS BECAUSE RAM SIZED IT TO HAVE THE SAME COLUMN ALL THE WAY UP THE BUILDING, DESPITE BEING GIVEN SPLICE LEVELS

CAMBER IN RAM WAS ALSO LIMITED UNIFORMLY TO 1", AS IT DOES NOT APPEAR POSSIBLE TO APPLY DIFFERENT CAMBER ALLOWANCES TO BEAMS VS. GIRDERS

Appendix D: Post-Tensioned Concrete Calculations

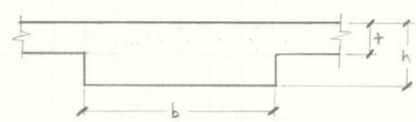
POST - TENSIONING | TECH 2 | Pg 1 OF 12



ASPECT RATIO OF LARGEST BAY → $\frac{36.33'}{21'} = 1.73$

PER TECHNICAL NOTE 3 FROM THE POST-TENSIONING INSTITUTE (PTI), WRITTEN IN OCTOBER 1993 BY DR. BIJAN AALAMI, WIDE-SHALLOW BEAMS SHOULD BE INCORPORATED FOR ASPECT RATIOS BETWEEN 1.5 & 2.5

THE SAME DOCUMENT RECOMMENDS LIMITS ON THE DIMENSIONS OF THE W-S BEAMS



$h \leq 2t$ $\quad$ $b \leq 6t$

MATERIALS

CONCRETE → NWC (150 pcf)
$f'_c = 5000$ psi
$f'_{ci} = 3000$ psi

REBAR → $F_y = 60$ ksi

PT → UNBONDED TENDONS
½" Ø, 7-WIRE STRANDS ($A = 0.153\ in^2$)
$f_{pu} = 270$ ksi
ESTIMATED PRESTRESS LOSS = 15 ksi
$f_{se} = 0.7(270\text{ ksi}) - 15\text{ ksi} = 174\text{ ksi}$
$P_{eff} = 174\text{ ksi}(0.153\ in^2) = 26.6\ ^k/\text{TENDON}$

PRELIMINARY SLAB / BEAM SIZES

PER PTI'S TECHNICAL NOTE 3, INITIAL SLAB THICKNESS SHOULD BE BASED ON THE SHORTER SPAN FOR W-S BEAMS / SLABS

$t = \frac{L}{40} = \frac{21 \times 12}{40} = 6.3\text{ in} \Rightarrow 7"$ SLAB

REASONABLE TO LIMIT DEFLECTION

$\therefore h = 14"$
$b = 42" = 3'-6"$

POST-TENSIONING | TECH 2 | Pg 2 OF 12

LOADS

$DL = \left[\left(\frac{3.5'}{21'}\right)\left(\frac{14''}{12''}\right) + \left(\frac{17.5'}{21'}\right)\left(\frac{7''}{12''}\right)\right] 150\ pcf = 102\ psf$

SDL = 15 psf

LL = 80 psf → REDUCE AS ALLOWED

21' x 36'-4" BAY → $KA_T = 1(21')(36.33') = 763 > 400$ OK
$LL_r = 0.25 + \frac{15}{\sqrt{763}} = 0.79$

21' x 17'-0" BAY → $KA_T = 1(21')(17') = 357 < 400$ ∴ NO REDUCTION

21' x 7'-3" BAY → NO REDUCTION

DESIGN N-S DIRECTION, 36'-4" BAY (DISTRIBUTED PT)

USE EQUIVALENT FRAME METHOD
BAY WIDTH = 36'-4"
IGNORE COLUMN STIFFNESS FOR SIMPLICITY.

$\frac{LL}{DL} = \frac{0.79(80)}{102} = 0.62 < 0.75$ ∴ NO PATTERN LOADING REQUIRED (ACI 318-08 §13.7.6.2)

SECTION PROPERTIES

$A = (36.33' \times 12\ in/ft)(7\ in) = 3,050\ in^2$
$S = \frac{bh^2}{6} = \frac{(36.33' \times 12\ in/ft)(7\ in)^2}{6} = 3,560\ in^3$

CLASS U DESIGN PERMITS USE OF GROSS SECTION

SET DESIGN PARAMETERS

ALLOWABLE STRESSES:
@ JACKING → $f'_{ci} = 3000$ psi
COMPRESSION → $0.6 f'_{ci} = 1800$ psi
TENSION → $3\sqrt{f'_{ci}} = 164$ psi

@ SERVICE → $f'_c = 5000$ psi
COMPRESSION → $0.45 f'_c = 2,250$ psi
TENSION → $6\sqrt{f'_c} = 424$ psi

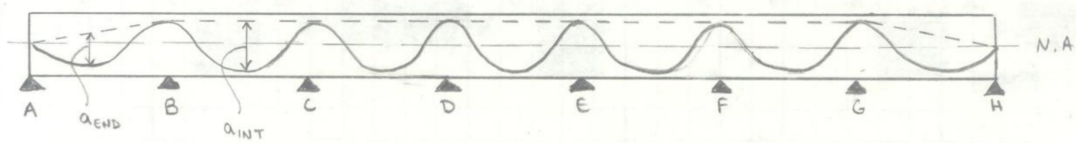
PRECOMPRESSION LIMITS:
MIN = 125 psi
MAX = 300 psi

TARGET LOAD BALANCE: 60-80% DL → USE 70%

COVER REQUIREMENTS: TOP = 3/4"
BOTTOM → 1½" UNRESTRAINED
3/4" RESTRAINED

POST - TENSIONING | TECH 2 | PG 3 OF 12

TENDON PROFILE



DESCRIPTION	HEIGHT *
ANCHOR	3.5"
END - BOTTOM	1.75"
INT. - TOP	6.0"
INT. - BOTTOM	1.0"

* = MEASURED FROM BOTTOM OF SLAB

$a_{END} = \frac{(3.5" + 6.0")}{2} - 1.75" = 3 \text{ in}$

$a_{INT} = 6.0" - 1.0" = 5.0"$

REQUIRED PRESTRESS

$w_b = 0.7(102 \text{ psf})(36.33 \text{ ft}) = 2594 \text{ plf} = 2.594 \text{ klf}$

$P_{END} = \frac{w_b L^2}{8 a_{END}} = \frac{2.594(21 \text{ ft})^2}{8(\frac{3"}{12"})} = 572 \text{ K}$

OF TENDONS → $\frac{572 \text{ K}}{26.6 \text{ K/TENDON}} = 21.5$ TENDONS → USE 20 TENDONS

$P_{ACT} = 20 \times 26.6 = 532 \text{ K}$

$\frac{P}{A} = \frac{532}{3050} \times 1000 = 174 \text{ psi}$ > 125 OK ✓ < 300 OK ✓

$P_{INT} = \frac{w_b L^2}{8 a_{INT}} = \frac{2.594(21)^2}{8(5/12")} = 343 \text{ K}$ (LESS PRESTRESS REQUIRED)

$w_{b,INT} = \frac{532(8)(5"/12")}{21^2} = 4.02 \text{ klf}$

$w_b / w_D = 108\%$ NOT GOOD → USE 18 TENDONS ($P_{ACT} = 479 \text{ K}$)

$\frac{P}{A} = \frac{479}{3050} \times 1000 = 157 \text{ psi}$ OK ✓

$w_{b,END} = \frac{479(8)(3"/12")}{21^2} = 2.17 \text{ klf}$ (59% w_D) OK ✓

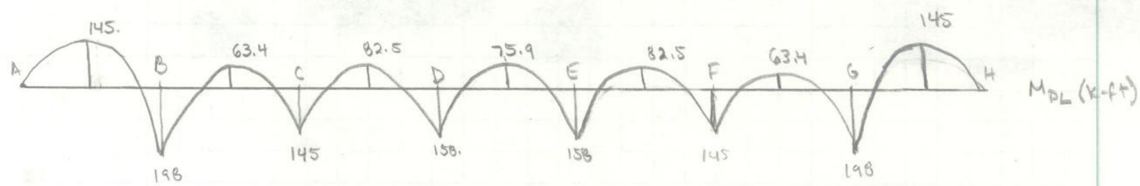
$w_{b,INT} = \frac{479(8)(5"/12")}{21^2} = 3.62 \text{ klf}$ (98% w_D) OK ✓

$P_{eff} = 479 \text{ K}$

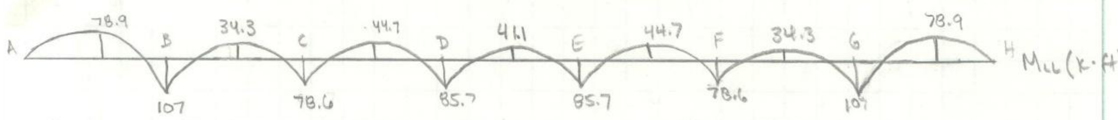
POST-TENSIONING | TECH 2 | pg 4 of 12

CHECK SLAB STRESSES

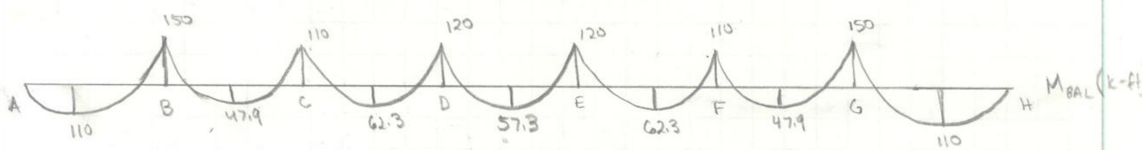
$w_D = (0.102 \text{ ksf} + 0.015 \text{ ksf})(36.33 \text{ ft}) = 4.25 \text{ klf}$



$w_L = (0.79)(0.08 \text{ ksf})(36.33 \text{ ft}) = 2.30 \text{ klf}$



$w_b = 3.21$ klf (AVERAGE ALL 7 BAYS)



STRESSES AFTER JACKING

MIDSPAN STRESSES:

$$f_{TOP} = \frac{-M_{DL} + M_{BAL}}{S} - \frac{P}{A}$$

$$f_{bot} = \frac{M_{DL} - M_{BAL}}{S} - \frac{P}{A}$$

INTERIOR SPAN (SPAN C-D)

$$f_{TOP} = \frac{(-82.5 + 62.3)\,12{,}000}{3560 \text{ in}^3} - 157 \text{ psi} = -225 \text{ psi} < 0.6 f'_{ci} \quad \boxed{\text{OK} \checkmark}$$

$$f_{bot} = \frac{(82.5 - 62.3)(12000)}{3560 \text{ in}^3} - 157 \text{ psi} = -88.9 \text{ psi} < 0.6 f'_{ci} \quad \boxed{\text{OK} \checkmark}$$

END SPAN (SPAN A-B)

$$f_{TOP} = \frac{(-145 + 110)\,12{,}000}{3560 \text{ in}^3} - 157 \text{ psi} = -275 \text{ psi} < 0.6 f'_{ci} \quad \boxed{\text{OK} \checkmark}$$

$$f_{bot} = \frac{(145 - 110)(12{,}000)}{3560 \text{ in}^3} - 157 \text{ psi} = -39.0 \text{ psi} < 0.6 f'_{ci} \quad \boxed{\text{OK} \checkmark}$$

POST-TENSIONING | TECH 2 | pg 5 OF 12

SUPPORT STRESSES:

$$f_{TOP} = \frac{M_{DL} - M_{BAL}}{S} - \frac{P}{A} \qquad f_{BOT} = \frac{-M_{DL} + M_{BAL}}{S} - \frac{P}{A}$$

INTERIOR SPAN (@ SUPPORT D)

$$f_{TOP} = \frac{(158 - 120)\,12{,}000}{3560\ in^3} - 157\ psi = -28.9\ psi < 0.6\, f'_{ci} \quad \boxed{OK ✓}$$

$$f_{BOT} = \frac{(-158 + 120)\,12000}{3560\ in^3} - 157\ psi = -285\ psi < 0.6\, f'_{ci} \quad \boxed{OK ✓}$$

END SPAN (@ SUPPORT B)

$$f_{TOP} = \frac{(198 - 150)\,12000}{3560\ in^3} - 157\ psi = 4.80\ psi < 3\sqrt{f'_{ci}} \quad \boxed{OK ✓}$$

$$f_{BOT} = \frac{(-198 + 150)\,12000}{3560\ in^3} - 157\ psi = -319\ psi < 0.6\, f'_{ci} \quad \boxed{OK ✓}$$

STRESS IN SERVICE

MIDSPAN STRESSES:

$$f_{TOP} = \frac{-M_{DL} - M_{LL} + M_{BAL}}{S} - \frac{P}{A} \qquad f_{BOT} = \frac{M_{DL} + M_{LL} - M_{BAL}}{S} - \frac{P}{A}$$

INTERIOR SPAN (SPAN C-D)

$$f_{TOP} = \frac{(-82.5 - 44.7 + 62.3)\,12000}{3560\ in^3} - 157\ psi = -376\ psi < 0.45\, f'_c \quad \boxed{OK ✓}$$

$$f_{BOT} = \frac{(82.5 + 44.7 - 62.3)\,12000}{3560\ in^3} - 157\ psi = 61.8\ psi < 6\sqrt{f'_c} \quad \boxed{OK ✓}$$

END SPAN (SPAN A-B)

$$f_{TOP} = \frac{(-145 - 78.9 + 110)\,12000}{3560\ in^3} - 157\ psi = -541\ psi < 0.45\, f'_c \quad \boxed{OK ✓}$$

$$f_{BOT} = \frac{(145 + 78.9 - 110)\,12000}{3560\ in^3} - 157\ psi = 227\ psi < 6\sqrt{f'_c} \quad \boxed{OK ✓}$$

SUPPORT STRESSES:

$$f_{TOP} = \frac{M_{DL} + M_{LL} - M_{BAL}}{S} - \frac{P}{A} \qquad f_{BOT} = \frac{-M_{DL} - M_{LL} + M_{BAL}}{S} - \frac{P}{A}$$

INTERIOR SPAN (SUPPORT @ D)

$$f_{TOP} = \frac{(158 + 85.7 - 120)\,12000}{3560\ in^3} - 157\ psi = 260\ psi < 6\sqrt{f'_c} \quad \boxed{OK ✓}$$

$$f_{BOT} = \frac{(-158 - 85.7 + 120)\,12000}{3560\ in^3} - 157\ psi = -574\ psi < 0.45 f'_c \quad \boxed{OK ✓}$$

END SPAN (SUPPORT @ B)

$$f_{TOP} = \frac{(198 + 107 - 150)\,12000}{3560\ in^3} - 157\ psi = 365\ psi < 6\sqrt{f'_c} \quad \boxed{OK ✓}$$

$$f_{BOT} = \frac{(-198 - 107 + 150)\,12000}{3560\ in^3} - 157\ psi = -679\ psi < 0.45\, f'_c \quad \boxed{OK ✓}$$

$\boxed{\text{ALL STRESSES ACCEPTABLE}}$

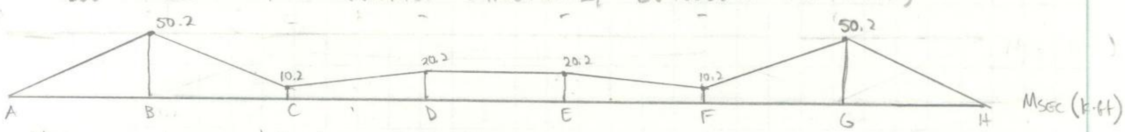
POST-TENSIONING TECH 2 pg 6 OF 12

ULTIMATE STRENGTH

$M_1 = Pe$
↳ $e = 0$ @ EXT. SUPPORT
↳ $e = 2.5$ in @ INT. SUPPORT

$M_1 = 479\text{ k}\left(\frac{2.5''}{12''}\right) = 99.8\text{ k-ft}$

$M_{SEC} = M_{BAL} - M_1$ (VARIES LINEARILY BETWEEN SUPPORTS)



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

BONDED REINFORCEMENT

POSITIVE MOMENT REGION:
INTERIOR SPAN (SPAN C-D)
$f_{TENS} = 61.8\text{ psi} < 2\sqrt{f'_c} = 141\text{ psi}$ ∴ NO REINFORCEMENT REQ'D (ACI 318-08 §18.9.3.1)

END SPAN (SPAN A-B)
$f_{TENS} = 227\text{ psi} > 2\sqrt{f'_c}$ ∴ REINF. REQ'D

$M_U = 1.2(145) + 1.6(78.9) + 1.0(0.5)(50.2 - 0) = 325\text{ k-ft}$

MIN REINFORCEMENT (ACI 318-08 § 18.9.3.2)

$y = \frac{f_{TENS}\,h}{f_{TENS} + f_{COMP}} = \frac{227\text{ psi}\,(7\text{ in})}{227\text{ psi} + 541\text{ psi}} = 2.07\text{ in}$

$N_c = \left(\frac{M_{DL} + M_{LL}}{S}\right) 0.5\, y\, t_2 = \left(\frac{145 + 78.9}{3560\text{ in}^3}\right) 0.5\,(2.07\text{ in})(36.33\text{ ft})(144\text{ in}^2/\text{ft}^2)$
$= 341\text{ K}$

$A_{s,min} = \frac{N_c}{0.5 f_y} = \frac{341\text{ k}}{0.5(60\text{ ksi})} = 11.4\text{ in}^2 \div 36.33\text{ ft} = 0.314\text{ in}^2/\text{ft}$

TRY #6 @ 12" O.C. $= 0.44\text{ in}^2/\text{ft}$ → VERIFY MIN. AREA IS OK

$A_{ps} = 0.153\text{ in}^2/\text{TENDON}\ (18\text{ TENDONS}) = 2.75\text{ in}^2$

$f_{ps} = f_{se} + 10{,}000 + \frac{f'_c\,bd}{300\,A_{ps}} = 174{,}000 + 10{,}000 + \frac{5000\,(36.33' \times 12)\,d}{300\,(2.75\text{ in}^2)}$
$= (184 + 2.642\,d)\text{ ksi}$

$d = 7'' - 1\tfrac{1}{2}'' - \tfrac{1}{4}'' = 6.25''$ ∴ $f_{ps} = 200.5\text{ ksi}$

$a = \frac{A_s f_y + A_{ps} f_{ps}}{0.85\,f'_c\,b} = \frac{15.84\text{ in}^2(60\text{ ksi}) + 2.75\text{ in}^2(200.5\text{ ksi})}{0.85\,(5\text{ ksi})(36.33\text{ ft} \times 12\text{ in})} = 0.81\text{ in}$

POST-TENSIONING | TECH 2 | pg 7 OF 12

$\phi M_n = \phi(A_s f_y + A_{ps} f_{ps})(d - \frac{a}{2}) = 0.9(15.84(60) + 2.75(200.5))(6.25 - \frac{0.81}{2})$
$= 658 \text{ k-ft} > M_u = 325 \text{ k-ft}$ $\boxed{\text{OK}}$

$\boxed{\text{USE \#6 @ 12'' O.C.}}$

NEGATIVE MOMENT REGION:
INTERIOR SPAN (SUPPORT @ D)
$2\sqrt{f'_c}$ LIMIT DOES NOT APPLY FOR NEGATIVE MOMENT

$M_u = 1.2(-158) + 1.6(-78.6) + 1.0(20.2) = -295 \text{ k-ft}$

MIN REINFORCEMENT (ACI 318-08 § 18.9.3.3)

$A_{cf} = \max \begin{cases} 21 \text{ ft} \\ 36.33 \text{ ft} \end{cases} > 36.33 \text{ ft}(7 \text{ in})(12 \text{ in/ft}) = 3050 \text{ in}^2$

$A_{s,min} = 0.00075 A_{cf} = 0.00075(3050) = 2.29 \text{ in}^2$

TRY (8) #5 w/ $A_s = 2.48 \text{ in}^2$

$d = 7'' - \frac{3}{4}'' - \frac{1}{4}'' = 6''$

$f_{ps} = 184 + 2.642(6) = 199.9 \text{ ksi}$

$a = \frac{2.48 \text{ in}^2(60 \text{ ksi}) + 2.75 \text{ in}^2(199.9 \text{ ksi})}{0.85(5 \text{ ksi})(36.33 \text{ ft} \times 12 \text{ in/ft})} = 0.38 \text{ in}$

$\phi M_n = \phi(A_s f_y + A_{ps} f_{ps})(d - \frac{a}{2}) = 0.9(2.48(60) + 2.75(199.9))(6 - \frac{0.38}{2})$
$= 304 \text{ k-ft} > M_u$ OK

$\boxed{\text{USE (8) \#5}}$

END SPAN, INTERIOR SUPPORT (SUPPORT @ B)

$M_u = 1.2(-198) + 1.6(-107) + 1.0(50.2) = -359 \text{ k}$

$A_{s,min} = 2.29 \text{ in}^2$ (WON'T BE ENOUGH)

TRY (8) #6 w/ $A_s = 3.52 \text{ in}^2$

$a = \frac{3.52 \text{ in}^2(60 \text{ ksi}) + 2.75(199.9 \text{ ksi})}{0.85(5 \text{ ksi})(36.33 \text{ ft} \times 12 \text{ in/ft})} = 0.41 \text{ in}$

$\phi M_n = 0.9(3.52(60) + 2.75(199.9))(6 - \frac{0.41}{2}) = 331 \text{ k-ft} < M_u$ NOT OK

TRY (12) #6 w/ $A_s = 5.28 \text{ in}^2$

$a = \frac{5.28 \text{ in}^2(60 \text{ ksi}) + 2.75 \text{ in}^2(199.9 \text{ ksi})}{0.85(5 \text{ ksi})(36.33 \text{ ft} \times 12 \text{ in/ft})} = 0.47 \text{ in}$

POST - TENSIONING TECH 2 Pg 8 OF 12

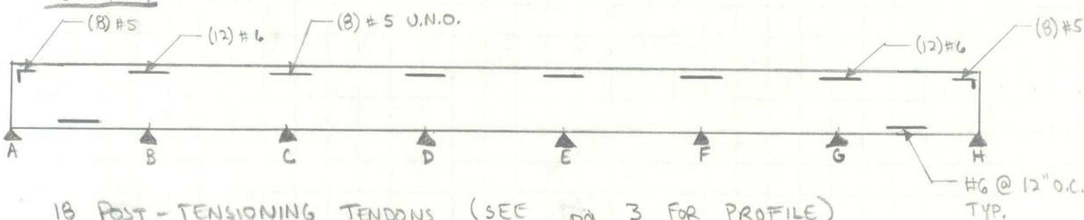
$\phi M_n = 0.9(5.28(60) + 2.75(199.9))(6 - \frac{0.43}{2}) = 375 \text{ k-ft} > M_u$ OK ✓

$\boxed{\text{USE (12) \#6}}$

END SPAN, EXTERIOR SUPPORT (SUPPORT @ A)
MIN REINFORCING WILL BE USED

$\boxed{\text{USE (8) \#5}}$

SUMMARY



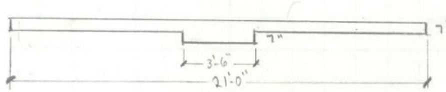
18 POST-TENSIONING TENDONS (SEE pg 3 FOR PROFILE)

DESIGN E-W DIRECTION (BANDED PT)

USE EQUIVALENT FRAME METHOD
IGNORE COLUMN STIFFNESS FOR SIMPLICITY

$\frac{LL}{DL} = \frac{0.79(80)}{102} = 0.62 < 0.75$ ∴ NO PATTERN LOADING REQUIRED

SECTION PROPERTIES



$A = 7\text{in}(21' \times 12\text{"/ft}) + 7\text{in}(3.5' \times 12\text{"/ft}) = 2060 \text{ in}^2$

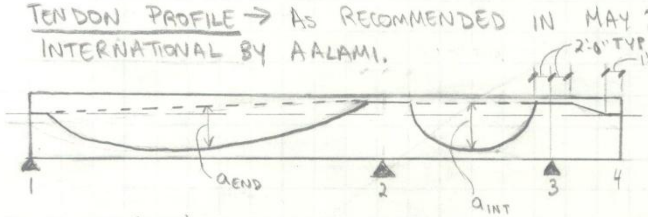
$\bar{y} = \frac{7"(21' \times 12)(10.5") + 7"(3.5' \times 12)(3.5")}{7"(21' \times 12) + 7"(3.5' \times 12)} = 9.5 \text{ in}$

$I = \frac{(21' \times 12)(7)^3}{12} + (21' \times 12)(7)(9.5 - 10.5)^2 + \frac{(3.5' \times 12)(7)^3}{12} + (3.5' \times 12)(7)(9.5 - 3.5)^2 = 20{,}752 \text{ in}^4$

$S_{top} = \frac{I}{y_t} = \frac{20752}{4.5} = 4612 \text{ in}^3$ $S_{BOT} = \frac{I}{y_b} = \frac{20{,}752}{9.5} = 2184 \text{ in}^3$

SET DESIGN PARAMETERS → SEE N-S DIRECTION

TENDON PROFILE → AS RECOMMENDED IN MAY 2003 ARTICLE IN CONCRETE INTERNATIONAL BY AALAMI.



DESCRIPTION	HEIGHT *
ANCHOR	9.5"
END-BOTTOM	1.75"
INT.-TOP	13"
INT.-BOTTOM	1.0"

* MEASURED FROM BOTTOM OF SLAB

$a_{END} = \frac{(13" - 9.5")}{36.33 \text{ ft}}(18 \text{ ft}) + (9.5" - 1.75") = 9.5 \text{ in}$

$a_{INT} = 13" - 1.0" = 12 \text{ in}$

POST – TENSIONING | TECH 2 | pg 9 of 12

REQUIRED PRESTRESS

$w_b = 0.7\,(102\text{ psf})\,(21\text{ ft}) = 1500\text{ plf} = 1.5\text{ klf}$

$P_{END} = \dfrac{w_b L^2}{8\,a_{END}} = \dfrac{1.5\text{ klf}\,(36.33\text{ ft})^2}{8\left(\frac{9.5''}{12''}\right)} = 313\text{ k}$

OF TENDONS → $\dfrac{313\text{ k}}{26.6^{k}/\text{TENDON}} = 11.8$ TENDONS → USE 15 TENDONS

$P_{ACT} = 15 \times 26.6 = 399\text{ k}$

$\dfrac{P}{A} = \dfrac{399}{2060} \times 1000 = 194\text{ psi}$ > 125 OK ✓ < 300 OK ✓

$P_{INT} = \dfrac{w_b L^2}{8\,a_{INT}} = \dfrac{1.5\,(17)^2}{8\left(\frac{12}{12}\right)} = 54.2\text{ k}$ (LESS PRESTRESS REQUIRED)

$w_{b,INT} = \dfrac{399\,(8)\left(\frac{12''}{12''}\right)}{17^2} = 11.0\text{ klf}$

$w_b / w_D = 516\%$ NOT GOOD → REDUCE DRAPE (DUE TO LARGE DISCREPANCY)

AIM FOR 175% w_D

$a_{INT,REQ'D} = \dfrac{1.75(0.102)(21')(17^2)}{399\,(8)} \times 12 = 4.07\text{ in}$ → USE 4.0 in

> 100% w_D RECOMMENDED FOR BEAMS PER MAY 2003 ARTICLE IN CONCRETE INTERNATIONAL BY AALAMI

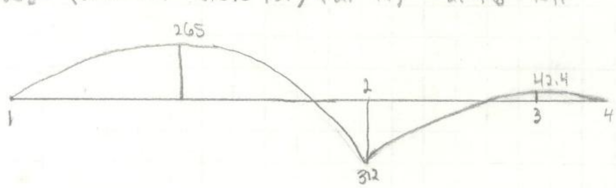
$w_{b,END} = \dfrac{399\,(8)\left(\frac{9.5}{12''}\right)}{36.33^2} = 1.91\text{ klf}$ (90% w_D) OK ✓

$w_{b,INT} = \dfrac{399\,(8)\left(\frac{4.0}{12}\right)}{17^2} = 3.68\text{ klf}$ (172% w_D) OK ✓

$P_{EFF} = 399\text{ k}$

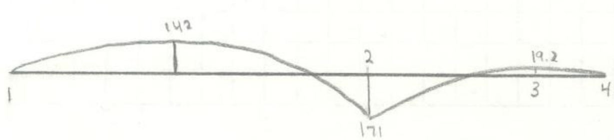
CHECK SLAB STRESSES

$w_D = (0.102\text{ ksf} + 0.015\text{ ksf})\,(21\text{ ft}) = 2.46\text{ klf}$



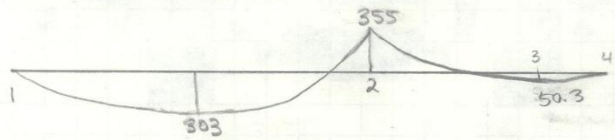
M_{DL} (k-ft)

$w_L = 0.79\,(0.08\text{ ksf})\,(21\text{ ft}) = 1.33\text{ klf}$



M_{LL} (k-ft)

$W_b = 2.80$ klf (AVERAGE ALL BAYS)



M_{BAL} (k-ft)

STRESSES AFTER JACKING

TENSION @ BOTTOM STRESSES:

MIDSPAN ON SPAN 1-2

$f_{TOP} = \dfrac{(-265+303)12000}{4612\text{ in}^3} - 194 = -84.4$ psi $< 0.6 f'_{ci}$ OK ✓

$f_{BOT} = \dfrac{(265-303)12000}{2184\text{ in}^3} - 194 = -403$ psi $< 0.6 f'_{ci}$ OK ✓

SUPPORT @ 3

$f_{TOP} = \dfrac{(-42.4+50.3)12000}{4612\text{ in}^3} - 194 = -173$ psi $< 0.6 f'_{ci}$ OK ✓

$f_{BOT} = \dfrac{(42.4-50.3)12000}{2184\text{ in}^3} - 194 = -237$ psi $< 0.6 f'_{ci}$ OK ✓

TENSION @ TOP STRESSES:

SUPPORT @ 2

$f_{TOP} = \dfrac{(312-355)12000}{4612\text{ in}^3} - 194 = -306$ psi $< 0.6 f'_{ci}$ OK ✓

$f_{BOT} = \dfrac{(-312+355)12000}{2184\text{ in}^3} - 194 = 42.3$ psi $< 3\sqrt{f'_c}$ OK ✓

STRESS IN SERVICE

TENSION @ BOTTOM STRESSES:

MIDSPAN ON SPAN 1-2

$f_{TOP} = \dfrac{(-265-142+303)12000}{4612\text{ in}^3} - 194 = -465$ psi $< 0.45 f'_c$ OK ✓

$f_{BOT} = \dfrac{(265+142-303)12000}{2184\text{ in}^3} - 194 = 377$ psi $< 6\sqrt{f'_c}$ OK ✓

SUPPORT @ 3

$f_{TOP} = \dfrac{(-42.4-19.2+50.3)12000}{4612\text{ in}^3} - 194 = -223$ psi $< 0.45 f'_c$ OK ✓

$f_{BOT} = \dfrac{(42.4+19.2-50.3)12000}{2184\text{ in}^3} - 194 = -132$ psi $< 0.45 f'_c$ OK ✓

TENSION @ TOP STRESSES:

SUPPORT @ 2

$f_{TOP} = \dfrac{(312+171-355)12000}{4612\text{ in}^3} - 194 = 139$ psi $< 6\sqrt{f'_c}$ OK ✓

$f_{BOT} = \dfrac{(-312-171+355)12000}{2184\text{ in}^3} - 194 = -897$ psi $< 0.45 f'_c$ OK ✓

$\boxed{\text{ALL STRESSES ACCEPTABLE}}$

POST-TENSIONING | TECH 2 | PG 11 OF 12

ULTIMATE STRENGTH

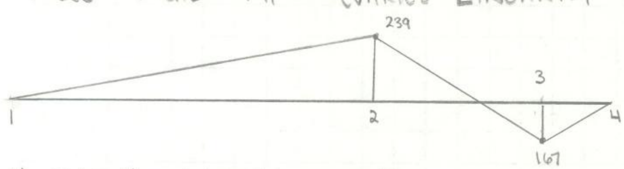
$M_1 = Pe$

↳ $e = 0$ in @ EXT. SUPPORT

↳ $e = 13'' - 9.5'' = 3.5$ in @ INT. SUPPORT

$M_1 = 399\ k\left(\frac{3.5''}{12''}\right) = 116.4$ k-ft

$M_{SEC} = M_{BAL} - M_1$ (VARIES LINEARLY BETWEEN SUPPORTS)



M_{SEC} (k-ft)

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

BONDED REINFORCEMENT

POSITIVE MOMENT REGION:

MIDSPAN ON SPAN 1-2

$f_{TENS} = 377$ psi $> 2\sqrt{f'_c}$ ∴ REINFORCEMENT REQ'D

$M_U = 1.2(265) + 1.6(142) + 1.0(0.5)(239-0) = 708$ k-ft

MIN. REINFORCEMENT (ACI 318-08 § 18.9.3.1)

$y = \frac{f_{TENS}\ h}{f_{TENS} + f_{COMP}} = \frac{377\ (14\ in)}{377 + 465} = 6.27$ in

$N_c = \left(\frac{265+142}{4612+2184}\right) 0.5(6.27\ in)(21\ ft)(144\ in^2/ft^2) = 568$ K

$A_{s,min} = \frac{568\ K}{0.5\ (60\ ksi)} = 18.9\ in^2$ → MUST GO IN BEAM

Ø = 1.27
As = 1.27
15 # 10
13 # 11
As = 1.56
Ø = 1.41

USE (13) #11 w/ $A_s = 20.28$ → SPACING $= \frac{42'' - 13(1.41\ in) - 2(0.75\ in)}{12} = 1.85$ in OK ✓

$A_{ps} = 15\ (0.153\ in^2/TENDON) = 2.30\ in^2$

$f_{ps} = (184 + 2.642\ a)$ ksi

$d = 14'' - 1\frac{1}{2}'' - 0.5(1.41\ in) = 11.8$ in ∴ $f_{ps} = 215.2$ ksi

$a = \frac{20.28\ in^2(60\ ksi) + 2.30\ in^2(215.2\ ksi)}{0.85\ (5\ ksi)(21\ ft \times 12\ in/ft)} = 1.60$ in

$\phi M_n = 0.9\left(20.28(60) + 2.30(215.2)\right)\left(11.8 - \frac{1.60}{2}\right) = 1412$ k-ft $> M_U$ OK ✓

$\boxed{\text{USE (13) \# 11}}$

POST - TENSIONING | TECH 2 | pg 12 OF 12

SUPPORT @ 3
NO TENSION IN SERVICE LOADS ∴ PROVIDE MIN REINFORCEMENT

$\boxed{\text{USE (13) \#11}}$

NEGATIVE MOMENT REGION:
SUPPORT @ 2
'$2\sqrt{f'_c}$' LIMIT DOES NOT APPLY @ NEGATIVE MOMENT

$M_u = 1.2(-312) + 1.6(-171) + 1.0(239) = -409$ k-ft

MINIMUM REINFORCEMENT

$$A_{cf} = \max\begin{cases} \frac{36.33+17}{2} = 26.7 \text{ ft} \\ 21 \text{ ft} \end{cases} > 26.7 \text{ ft } (14 \text{ in})(12 \text{ in/ft}) = 4486 \text{ in}^2$$

$A_{s,min} = 0.00075\ (4486) = 3.36$ in^2

TRY (8) #6 w/ $A_s = 3.52$ in^2

$d = 14" - 3/4" - 1/4" = 13$ in

$f_{ps} = 184 + 2.642\ (13) = 218.3$ ksi

$$a = \frac{3.52 \text{ in}^2 (60 \text{ ksi}) + 2.30 \text{ in}^2 (218.3 \text{ ksi})}{0.85\ (5 \text{ ksi})(21 \text{ ft} \times 12 \text{ in/ft})} = 0.67 \text{ in}^2$$

$\phi M_n = 0.9\left(3.52(60) + 2.30(218.3)\right)\left(13 - \frac{0.67}{2}\right) = 678$ k-ft $> M_u$ OK ✓

$\boxed{\text{USE (8) \#6}}$

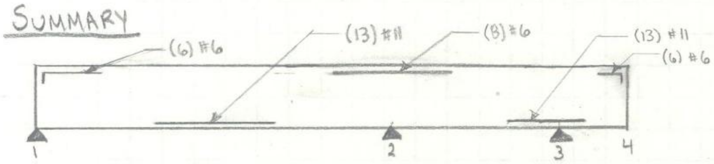
EXTERIOR SUPPORTS
NO MOMENT ∴ USE MINIMUM REINFORCING

$$A_{cf} = \max\begin{cases} \frac{36.33}{2} = 18.2 \text{ ft} \\ 21 \text{ ft} \end{cases} > 21 \text{ ft } (14 \text{ in})(12 \text{ in/ft}) = 3528 \text{ in}^2$$

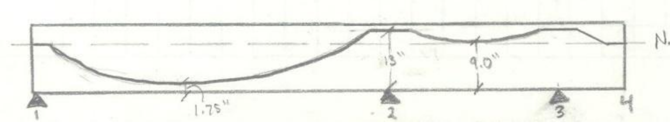
$A_{s,min} = 0.00075\ (3528 \text{ in}^2) = 2.64$ in^2

$\boxed{\text{USE (6) \#6}}$ w/ $A_s = 2.64$ in^2

SUMMARY



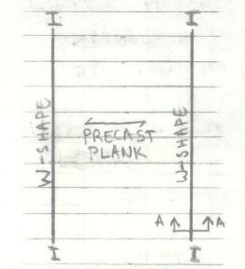
BONDED REINFORCING



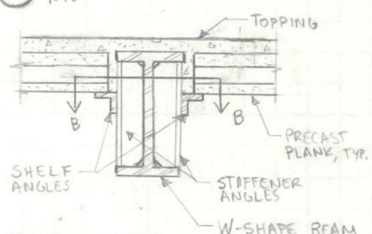
TENDON PROFILE
↳ 15 BANDED TENDONS

Appendix E: Precast Hollow core/Steel Girder Calculations

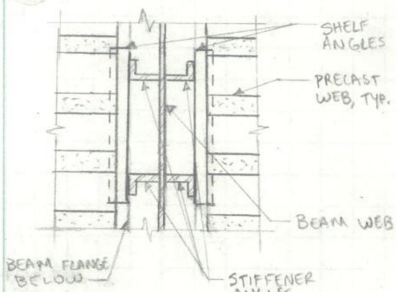
PRECAST PLANK/STEEL GIRDER | TECH 2 | PG 1 OF 2



PLAN VIEW
NTS



SECTION A-A
NTS



SECTION B-B
NTS

LOADS: DL = S.W. (ASSUME 10 psf FOR FRAMING)
SDL = 15 psf
LL = 80 psf (REDUCE AS ALLOWED)

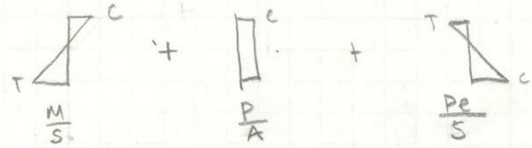
CHOOSE A PLANK → USE NITTERHOUSE
↳ MINIMIZE FLOOR DEPTH/WEIGHT
↳ MAINTAIN 2 HR FIRE RATING

TRY 6" HOLLOW CORE W/ 2" TOPPING
CHOOSE STRAND PATTERN:
SUPERIMPOSED LOAD = 15 psf + 80 psf = 95 psf

USE (4) ½"Ø PATTERN
CHECK STRESSES

$$M_u = \frac{(15+48.75+25+80)(4\text{ ft})(21\text{ ft})^2}{8} = 37.2\text{ k-ft} = 446.5\text{ k-in}$$

$$P_{EFF} = 0.6(270\text{ ksi})(0.153\text{ in}^2) = 24.8\text{ K/STRAND}$$



$$f_T = -\frac{446.5\text{ k-in}}{799} - \frac{24.8(4)}{253} + \frac{24.8(4)(2.65")}{799}$$
$$= -0.622\text{ ksi} = 622\text{ psi COMP.} < 0.45 f'_c = 2700\text{ psi}$$
OK ✓

$$f_b = \frac{446.5\text{ k-in}}{370} - \frac{24.8(4)}{253} - \frac{24.8(4)(2.65)}{370}$$
$$= 0.104\text{ ksi} = 104\text{ psi TENS.} < 10\sqrt{f'_c} = 775\text{ psi}$$
OK ✓

CHECK DEFLECTIONS

$$\Delta_{LL} = \frac{5(1.6)(0.08\text{ ksf})(4\text{ ft})(21\text{ ft})^4(1728)}{384(29000\text{ ksi})(1519\text{ in}^4)} = 0.051\text{ in} < \Delta_{LL,max} = 0.7\text{ in}$$
OK ✓

$$\Delta_{TL} = \frac{5[1.2(0.015+0.04875+0.025)+1.6(0.08)](4\text{ ft})(21\text{ ft})^4(1728)}{384(29000\text{ ksi})(1519\text{ in}^4)} = 0.093\text{ in} < \Delta_{TL,max} = 1.05\text{ in}$$
OK ✓

$$M_u = \frac{[1.2(0.015+0.04875+0.025)+1.6(0.08)]4\text{ ft}(21\text{ ft})^2}{8} = 51.7\text{ k-ft} < M_r = 67.4\text{ k-ft}$$
OK ✓

$\boxed{\text{6" HOLLOW CORE W/ 2" TOPPING \& (4) ½"Ø STRANDS}}$ IS OK

PRECAST PLANK/STEEL GIRDER | TECH 2 | pg 2 of 2

CHOOSE BEAM

$w_D = (15 + 48.75 + 25 + \overbrace{10}^{\text{FRAMING}})(21\text{ ft}) = 2074 \text{ plf} = 2.07 \text{ klf}$

$LL_r = 0.25 + \frac{15}{\sqrt{2(21)(36.33)}} = 0.634 > 0.5$ OK ✓

1526 > 400 OK ✓

$w_L = 0.634(80)(21\text{ ft}) = 1065 \text{ plf} = 1.07 \text{ klf}$

$w_u = 1.2(2.07) + 1.6(1.07) = 4.20 \text{ klf}$

$I_{req,LL} = \frac{5[1.6(1.07)](36.33\text{ ft})^4(1728)}{384(29000\text{ ksi})(36.33 \times 12/360)} = 1911 \text{ in}^4$

$I_{req,TL} = \frac{5(4.20)(36.33\text{ ft})^4(1728)}{384(29000\text{ ksi})(36.33 \times 12/240)} = 3119 \text{ in}^4$

FROM AISC TABLE 3-3, OPTIONS ARE:

1. W30x90 w/ $I = 3610 \text{ in}^4$
2. W27x94 w/ $I = 3270 \text{ in}^4$
3. W24x117 w/ $I = 3540 \text{ in}^4$
4. W21x132 w/ $I = 3220 \text{ in}^4$
5. W18x175 w/ $I = 3450 \text{ in}^4$
6. W14x257 w $I = 3400 \text{ in}^4$

TO REDUCE FLOOR DEPTHS, TRY W18x175 w/ d = 20.0"

$\phi M_n = 1490$ k-ft

$\phi V_n = 535$ k

$M_u = \frac{4.20 \text{ klf } (36.33\text{ ft})^2}{8} = 693 \text{ k-ft} < \phi M_n$ OK ✓

$V_u = 4.20 \text{ klf}(36.33\text{ ft})(0.5) = 76.3 \text{ k} < \phi V_n$ OK ✓

CHECK S.W. ASSUMPTION → $\frac{175 \text{ plf}}{21\text{ ft}} = 8.33 \text{ psf} < 10 \text{ psf}$ OK ✓

$\boxed{\text{USE W18x175}}$

NOTE: IT MIGHT BE POSSIBLE TO REDUCE THIS FURTHER IF COMPOSITE ACTION WAS ACHIEVED BETWEEN THE BEAM AND THE PRECAST/TOPPING

CHOOSE COLUMN

$P_u = 1.5(76.3\text{ k}) = 115\text{ k}$

USE K = 2.0 (VERY CONSERVATIVE) → KL = 28 ft

$\boxed{\text{USE W12x53}}$ w/ $\phi P_n = 192$ k

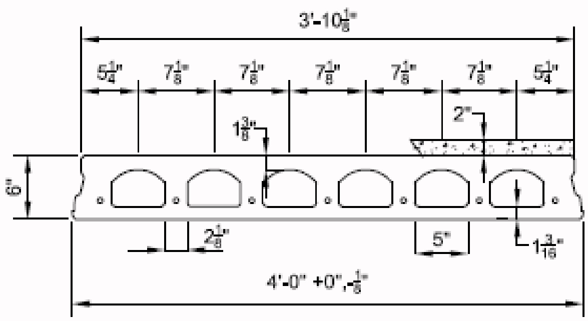
Prestressed Concrete
6"x4'-0" Hollow Core Plank

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_{tct} = 551 in.^{3}
Y_{tcp} = 1.90 in.	Precast S_{tcp} = 799 in.^{3}
Y_{tct} = 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