Technical Report 2

Alternate Floor Systems
Faculty Advisor: Dr. Thomas E. Boothby
October 12, 2012

Office Building

Sayre, PA

Seth M. Moyer | Structural

Table of Contents

Executive Summary

The purpose of Technical Report 2 is to propose and design three alternate floor systems for potential use in the Office Building and compare them to the existing floor system and to one another. All four systems will be compared and assessed in terms of weight, cost and depth and the general, architectural, structural, serviceability and constructability impacts of the systems will be discussed. All cost data was taken from RSMeans Building Construction Cost Data 2012 and includes overhead and profit with a location factor applied to account for the construction site.

Hand calculations were completed in order to design the alternate systems for strength and serviceability requirements. A single bay was used to design and compare all the systems. The bay selected for this assessment spans 18'-10" between grids B and C and 36'-0" between grids 1 and 3. The 36'-0" is the longest span in the building and was chosen as it would likely control the design. The existing system is composite deck slab on open web steel joists (non-composite). The three alternate systems considered were composite deck slab on composite steel beams/girders, one-way slabs and precast hollow-core planks on steel girders.

The composite steel system was designed as 2 1/2" of lightweight concrete on 1 1/2" thick composite deck. The beams supporting the deck are W10x12 with (12) 3/4" shear studs spaced evenly along their lengths. The girders are W21x44 with (32) 3/4" shear studs spaced evenly along their lengths. As both the lightest and least expensive of all systems considered, the many benefits of this floor outweighed the poor vibration control properties (which still need to be investigated further and addressed) and led to it being classified as the only viable option for an alternate floor in the Office Building of the ones that were assessed.

The one-way slab system was designed as an 8 1/2" thick slab spanning between 18"x28" concrete beams. Tension reinforcing was designed for the negative and positive moments in both the slab and the beams. Shrinkage and temperature reinforcing was also calculated for the slab and transverse reinforcing for shear was designed for the beam. Due primarily to the significantly higher costs, more difficult construction and much higher dead weight of this design, which will have a major impact on the rest of the building structure (including the foundations and lateral system), the one-way slab system was deemed an unfeasible alternative for the Office Building.

The precast hollow-core planks system was designed as 4HC6 planks (4'-0" by 6" thick) with a 66-S strand designation code (untopped). This plank was chosen from the PCI Design Handbook, 6th Edition based on the allowable superimposed load capacity. The girders supporting the planks were designed as W24x68 wide flange members. It was assumed that the girder is fully braced by the precast planks. Based on the increased cost, more difficult construction and greater weight of this system and benefits similar to those offered by the composite steel design (a less expensive, lighter option), the pros of the precast plank system

do not justify its use as an alternate system when compared to other floor systems with greater potential.

Supplemental figures in the form of images and tables are provided and referenced throughout the report as well as appendices, following the conclusion, that contain further detailed hand calculations.

Building Introduction

The Office Building is being constructed as part of a multi-phase office complex development project in Sayre, PA. Upon completion, currently slated for April 2013, the building will provide office and meeting space. It will also feature a fitness wing and locker rooms for employees on the second floor. With five stories (all above grade) extending up to 67'-0" at the mean roof height (top of parapet elevation = 74'-5"), the 85,075 sq ft Office Building has been designed for a total occupancy load of 1134.

The footprint of the Office Building is laid out in an off-centered "H" configuration (See Figure 1). The façade enclosing the east and west wings is primarily made up of insulated metal panels on 6" cold formed metal studs. 6' high horizontal glazing strips break up the exterior at each story. The portion of the building that connects the two wings is enclosed with a curtain wall glazing system. Figure 2 shows an elevation of the south-facing (main entrance) side of the building in which you can see both the wings and connecting portion. The parapet extends up past the roof to a maximum height of 74'-5" along both the east and west facades. It tapers down to a height of 68'-2 1/2" at the interior edge of the wings and continues at that elevation across the connecting segment.
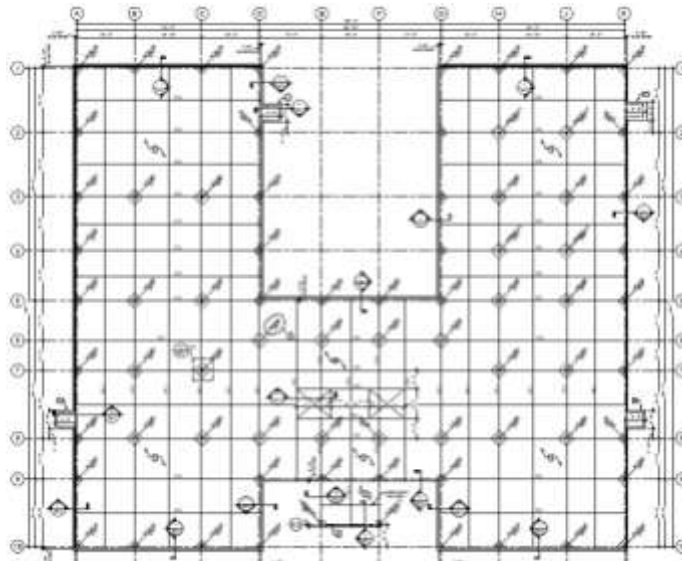

Figure 1: First Floor Slab Plan
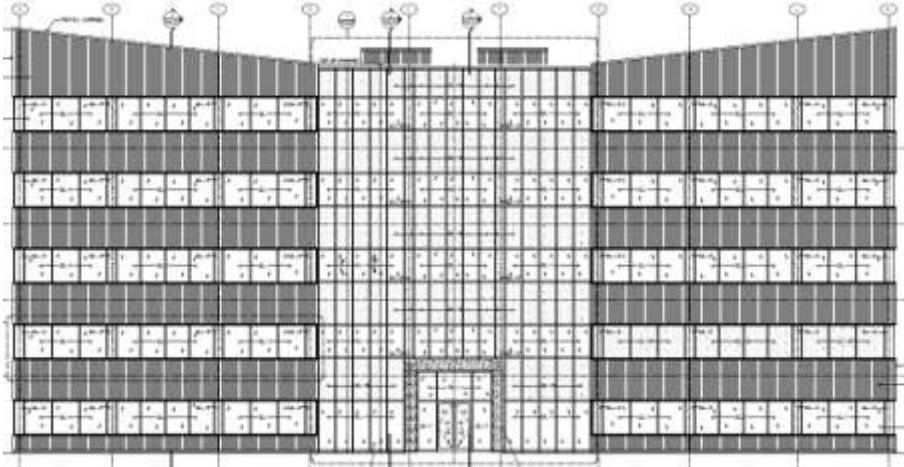(Image Credit: Larson Design Group)

Figure 2: South Elevation
(Image Credit: Silling Associates, Inc.)

Structural Overview

The Office Building structure is founded on spread, combined and strip footings which support the concrete piers, pier walls, foundation walls and columns directly to transfer the loads from the superstructure to the soil they bear upon. The floor system is made up of 4" thick (total) composite deck floor slabs on open web steel joists (non-composite for joists/beams). The joists frame into wide flange steel beams which transfer the loads to wide flange steel columns. The lateral system consists of braced frames in both the N-S and E-W directions, which all extend up to the roof.

Foundations

The geotechnical report conducted by CME Associates, Inc. for the Office Building site subsurface conditions indicates that spread and continuous footing foundations may be designed for an allowable soil bearing pressure of 4,000 psf. The report also specifies that spread footings should not be less than 3'-3" square and continuous strip footings should not be less than 2'-3" wide to prevent excessive settlements.

Typical interior columns are supported directly by spread footings just under the slab-on-grade. Typical perimeter columns sit on concrete piers that extend down to the spread footings. To protect against frost heave, perimeter footings have a minimum of 4'-0" of soil above their bearing elevation, measured from the bottom of the footing to finish grade. Both 8" and 12" thick concrete foundation walls run continuously along the outside perimeter of the building footprint, centered on 2'-3" strip footings, between the perimeter piers and footings.

At the braced frame locations outlined in Figure 3, 28" thick pier walls extend between the individual column piers. Combined footings also extend from pier to pier. The combined footings help to resist the overturning moments that result from lateral loading along their longitudinal axis. They also help to prevent differential settlement of the individual columns that form the braced frame.
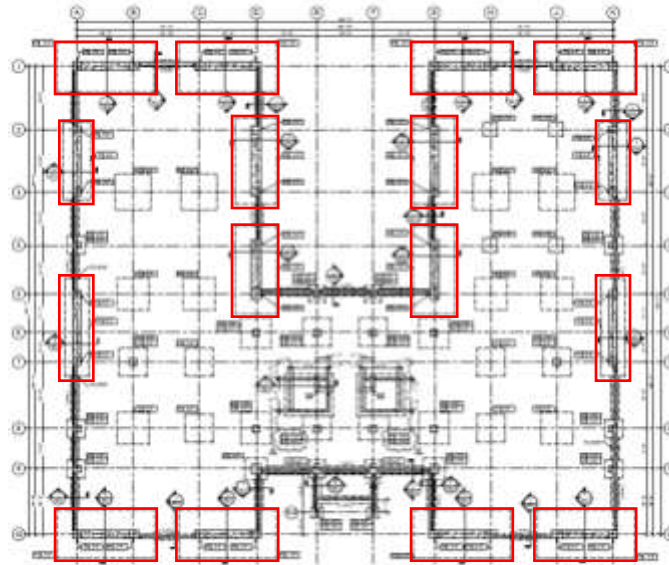
Figure 3: Braced Frames/Combined Footing Locations
(Image Credit: Larson Design Group)

Floor and Framing System

The first floor is a 4" thick slab-on-grade with WWR 6x6 – W2.9xW2.9 at mid-depth. Floors 2-5 consist of 2 1/2" thick normal weight concrete on 20 gauge 1 1/2" composite deck with WWR 6x6 – W4.0xW4.0 at mid-depth (4" total slab thickness). The composite deck slab is supported by open web steel joists (typically 16K2 up to 16K4) spaced at 3'-0" on center max. The floor joists distribute the gravity loads to the wide flange beams (interior beams are typically W24s and the exterior beams range from W12 to W16). Neither the joists nor beams are designed as composite as there is no way specified to transfer the necessary shear forces between the composite deck slab and the framing members. The maximum beam span is 36', between grid lines 1 and 3, for the W24x76 interior beams along grid lines B,C,H and J.

The beams carry the loads to wide flange columns to then be dispersed to the foundation. Typical column sizes include W12x53, W12x65, W12x79 and W12x106. All typical columns are spliced at 30'-8" above first floor (4' above the third floor). Where the fitness room is located in the east wing on level 2, HSS6x6x1/4 columns run up to the bottom of the W24x55 and W24x76 beams at grid points H2, H4, J2 and J4. The primary purpose of these one story columns is to reduce vibrations in the bays supporting the fitness center activities, which might otherwise create a serviceability issue with the light system of framing being utilized.

An enlarged portion of the typical floor framing plan can be seen in Figure 4 below.
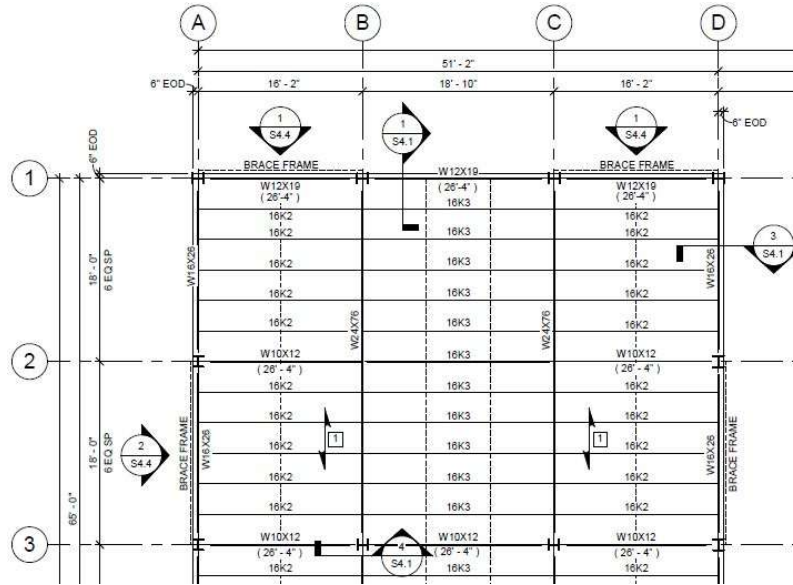
Figure 4: Typical Floor Framing Plan (Enlarged)
(Image Credit: Larson Design Group)

Roof and Framing System

The roof structure is made up of 1 1/2" Type B 20 gauge wide rib roof deck. A maximum thickness of 4" of rigid insulation is laid on top of the deck and is covered with fully adhered EPDM roof membrane. The deck is typically supported by 16KCS2 and 24K4 open web steel joists spaced at 6'-0" on center max. The joists then rest on W21x44 interior beams (towards which they slope down from the perimeter beams) and either W12x19 or W14x22 exterior beams. All gravity loads are then transferred to the wide flange columns.

An enlarged portion of the typical roof framing plan can be seen in Figure 5 below.
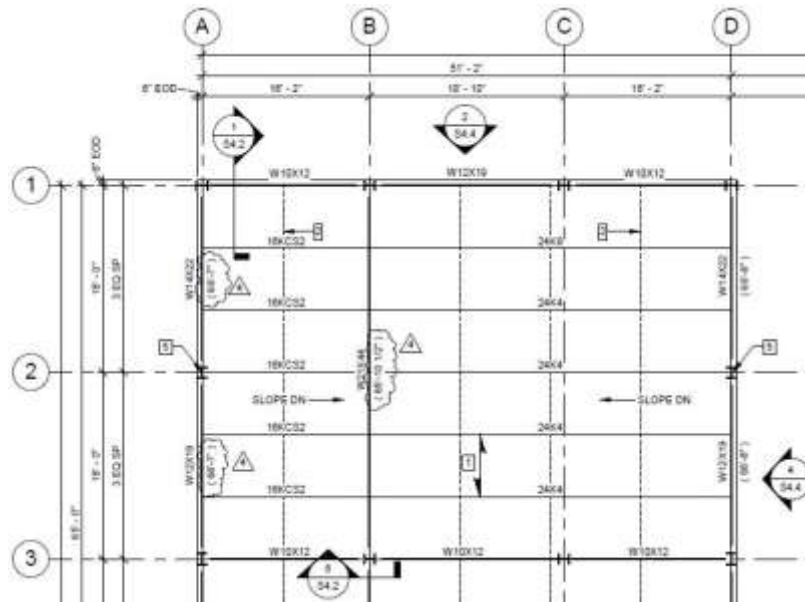
Figure 5: Typical Roof Framing Plan (Enlarged)
(Image Credit: Larson Design Group)

Lateral System

The lateral force resisting system of the Office Building is made up of 16 "K" braced frames (8 in each the N-S and E-W directions) (See Figure 3 for plan locations). The double angles brace the center work point of the perimeter beam at each floor down to the horizontal double angle-to-column intersection points above the windows of the floor below and up to the horizontal double angle-to-column intersection points below the windows of the floor above (double angles brace the base of the columns to the center work point of the horizontal wide flange beam below the windows at level 1) (See Figure 6 for typical bracing details).

Wind pressures on the exterior of the building are collected by the façade and the resultant forces are transferred into the floor/roof diaphragms. The diaphragms at each story act rigidly and transfer the story shear forces to the braced frames that run parallel to the direction of the loading. The braced frames resist the lateral loads based on the proportion of their relative stiffness. These story forces accumulate at each floor, moving down through the building until the total base shear is transferred into the ground via the foundation.

Similarly, for seismic loads induced by the buildings response to ground motion/acceleration, the total base shear is distributed to the diaphragms at each story as a function of the respective heights and weights attributed to each level. Once distributed, the seismic forces travel through the diaphragms and into the braced frames based on relative stiffness. Similarly, the story forces accumulate and are eventually transferred down to the bearing soils through the foundation.
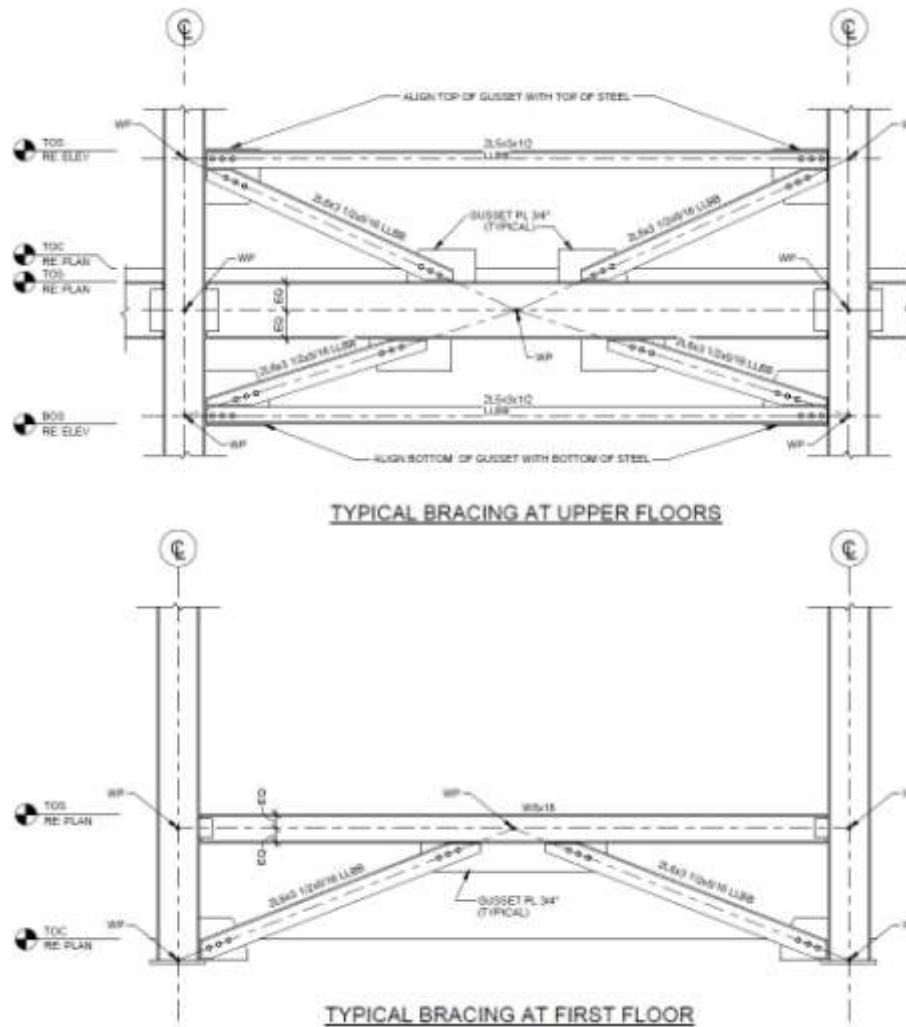
Figure 6: Typical Bracing Details
(Image Credit: Larson Design Group)

Design Codes

The major model and design codes and standards used in the design of the Office Building:

- Pennsylvania Uniform Construction Code (PAUCC)
- International Building Code 2009 (IBC 2009) (as adopted and modified by the PAUCC)
- Minimum Design Loads for Buildings and Other Structures (ASCE 7-05)
- Specification for Structural Concrete (ACI 301-05)
- Building Code Requirements for Structural Concrete (ACI 318-08)
- Specification for Structural Steel Buildings (AISC 360-05)
- Standard Specifications for Open Web Steel Joists, K-Series (SJI-K-1.1 05)
- Design Manual for Composite Decks, Form Decks, Roof Decks and Cellular Metal Floor Deck with Electrical Distribution, SDI Pub. No. 29

The same codes and standards are being referenced for use in this technical report with the following exceptions:

- ASCE 7-10
- AISC Steel Construction Manual, 14th Edition, LRFD
- Specification for Structural Steel Buildings (AISC 360-10)
- Building Code Requirements for Structural Concrete (ACI 318-11)

Materials Used

Materials were referenced from Sheets S0.1 and S0.2 and are summarized below in Figure 7.

Steel		
Type	ASTM Standard	Grade
W and WT Shapes	A992	50
Standard Shapes	A36	N/A
Angles, Channels and Plates	A36	N/A
HSS	A500	B
Pipe	A53, E or S	B
Anchor Rods	F1554	N/A
Shear/Anchor Studs	A108	N/A
Deformed Anchors	A496	N/A
Bolts (Plain)	A307	N/A
Bolts (High Strength)	A325	N/A
Nuts	A563	C
Hardened Washers	F436	N/A
Plate Washers	A36	N/A
Deformed and Plain Bars	A615	60
Welded Wire Reinforcement	A185	N/A
Steel Deck	A611	C,D,E
or Steel Deck	A653-94	33
Zinc Coated Steel Sheet	A1003	N/A
Hot Dipped, Galvanized Finish	A123	N/A
Load-Bearing Cold-Formed	C955-07	N/A
SS Pipes and Tubes	A312	N/A
SS Bars and Fittings	A582	N/A
Alum. Pipes and Tubes	B429	N/A
Alum. Bars and Fittings	B221	N/A
SS Fasteners	A240/A666	N/A

Concrete		
Usage	Weight	f'c (psi)
Foundation Walls	Normal	4500
Column Piers	Normal	4500
Combined Footings	Normal	4500
Exterior Slabs-on-Grade	Normal	4500
Specified Column Piers	Normal	5500
Elements Not Specified	Normal	3000

Miscellaneous	
Type	Standard
Grout (6000 psi)	ASTM C1107
Weld Electrodes	AWS Class E7018

Figure 7: Materials Summary

Gravity Loads

Dead, live and snow loads will be calculated and compared to the design loads used by the structural engineer. Spot checks of various typical framing members will then be made using the loads that were calculated.

Dead and Live Loads

Dead loads for the roof and floors were calculated using the actual weights of construction materials and additional allowances to account for superimposed loads due to MEP and ceiling materials as well as various structural framing. The calculated values of both the roof and floor dead loads matched the design values (See Figure 8 below). Refer to Appendix A for a detailed breakdown of the dead load calculations.

Dead Loads (psf)		
	Design	Calculated
Roof	20	20
Floor	60	60

Figure 8: Dead Load Summary

Live loads for the roof and floors were determined from ASCE 7-10, Table 4-1 for office buildings and roofs. For optimal flexibility of the Office Building in years to come, 80 psf for corridors above the first floor was selected as well as an additional allowance of 20 psf for partitions. This total load of 100 psf for the floors will allow for a variety of configurations of the office space instead of just designing for the corridors where they fall in the current layout. The calculated values for both the roof (minimum live load from Table 4-1) and floors matched the design values (See Figure 9 below).

Live Loads (psf)		
	Design	Calculated
Roof	20	20
Floor	100	100

Figure 9: Live Load Summary

Snow and Drift Loads

The flat roof snow load was determined to be 21 psf from a ground snow load value of 30 psf (Refer to Appendix A for flat roof snow load calculation details). 21 psf is less than the design snow load of 24 psf. This is due to the fact that the design value was calculated using a thermal

factor of 1.1 as opposed to the 1.0 used for the calculation in this report. It was assumed that the roof could be considered warm, since the structure is heated and the roof is not openly ventilated, and therefore $C_t=1.0$. However, using the thermal factor of 1.1 is conservative.

The maximum value of the snow drift load was calculated for the longest stretch of roof ($l_u=155.33'$) upwind of the full-height parapet. In this case, the drift snow load was found to be a maximum of 57.8 psf directly against the parapet at the east or west exterior walls. This value is superimposed onto the flat roof snow load and results in a maximum snow load value of 78.8 psf at the inside face of the parapet. Refer to Appendix A for the hand calculations of the drift load as well as a loading diagram at the parapet.

Floor Systems

The primary objective of this technical report is to analyze the existing floor system of the Office Building and to propose and investigate three other alternate floor design options. All four systems will be compared and assessed in terms of weight, cost and depth and the general, architectural, structural, serviceability and constructability impacts of the systems will be discussed.

A single bay will be used to design and compare all the systems. The bay selected for this assessment spans 18'-10" between grids B and C and 36'-0" between grids 1 and 3. The 36'-0" is the longest span in the building and was chosen as it would likely control the design. The bay is outlined in red below in Figure 10 and an enlarged plan of the bay can be seen in Figure 11.
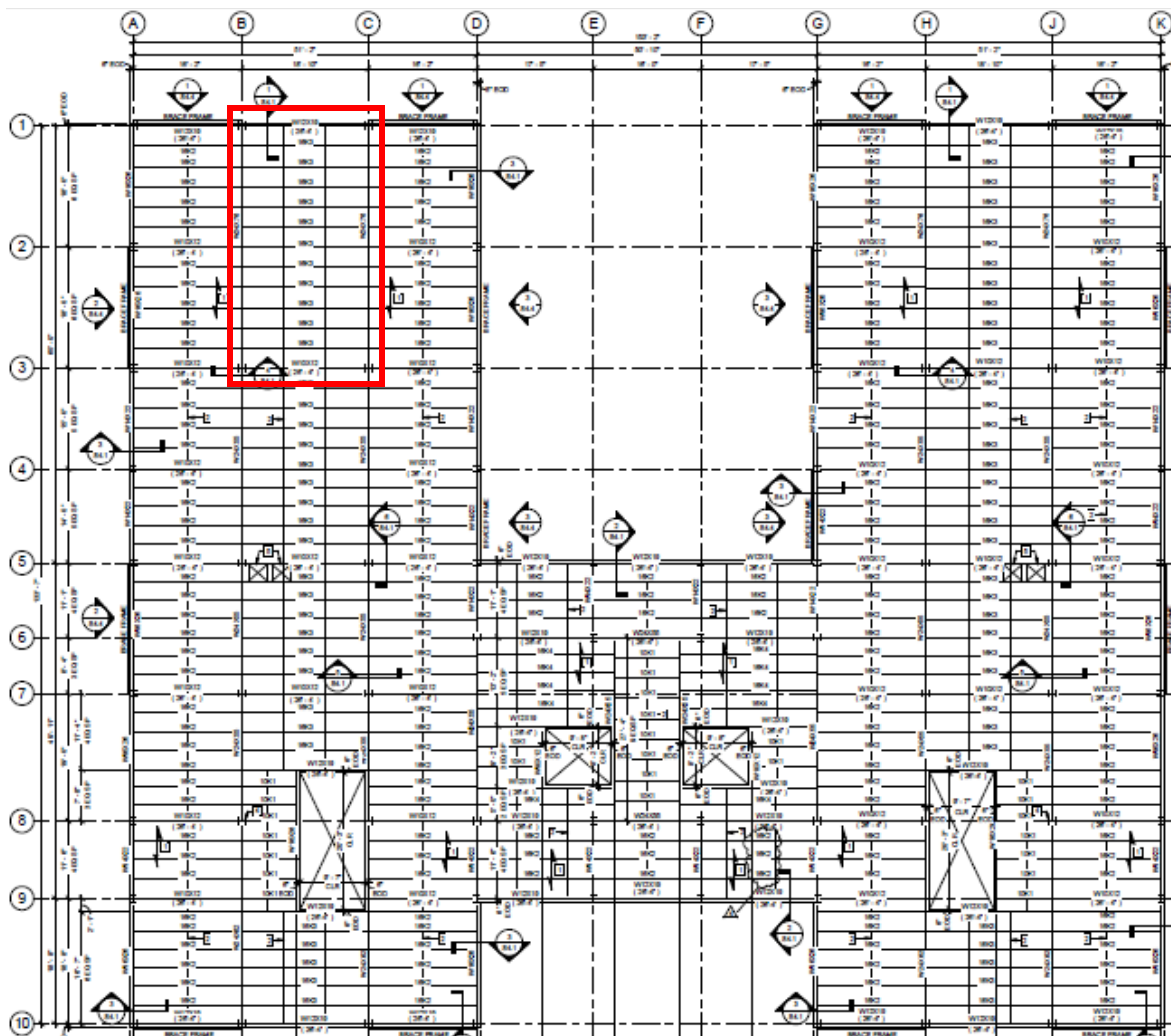

Figure 10: Typical Framing Plan
(Image Credit: Larson Design Group)

Composite Deck Slab on Open Web Steel Joists
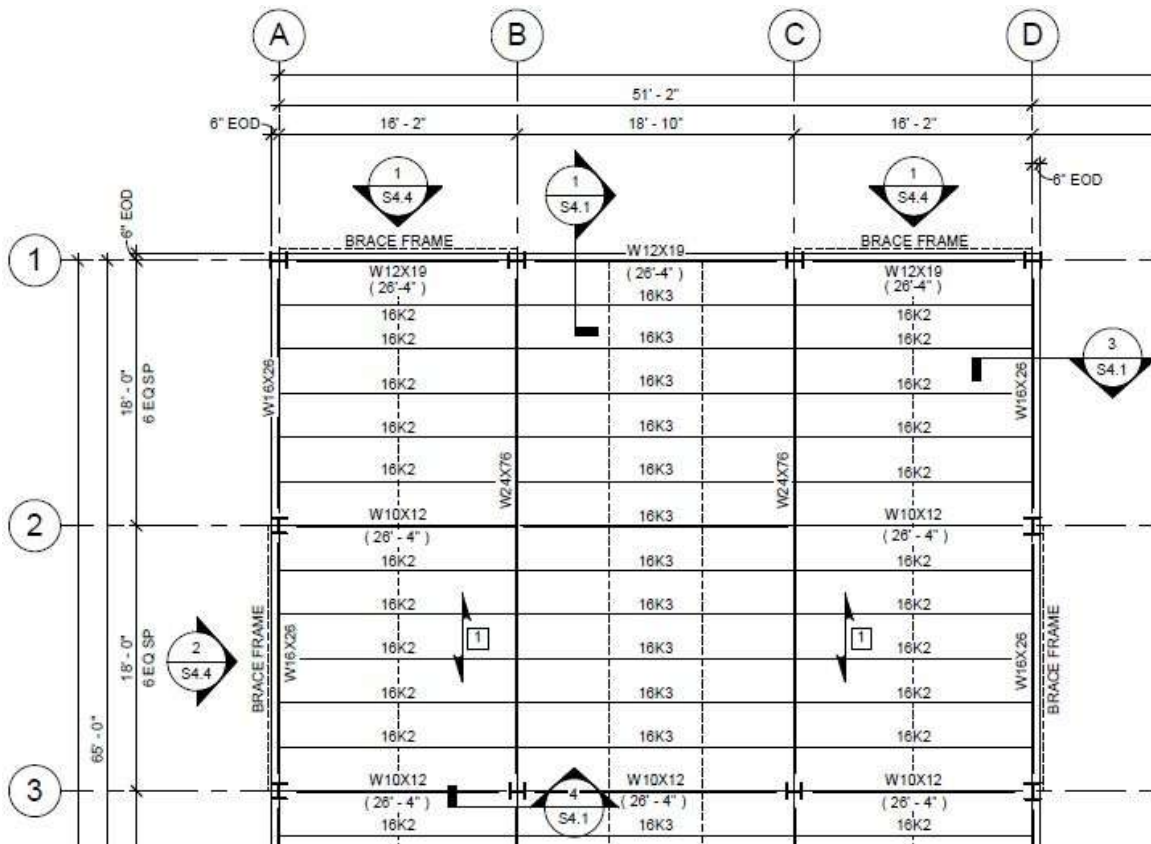
Figure 11: Enlarged Plan of Floor Framing Bay
(Image Credit: Larson Design Group)

The existing elevated floors of the Office Building are made up of 2 1/2" thick normal weight concrete topping on 20 gauge 1 1/2" composite deck (4" total slab thickness). The deck is supported by 16K3 open web steel joists at 3'-0" on center. At each end of the 18'-10" span, the joists rest on W24x76 wide flange girders that span 36'-0" between wide flange column supports.

Although composite floor deck is used, the overall design is non-composite with respect to the joists and girders. There is no connection specified between the slab and steel members to provide the shear transfer strength necessary for composite behavior. The 1 1/2" composite deck is a standard product that is specified by the structural engineer. The 3'-0" deck spans are based on the capacity of the 16" deep steel joists, which were selected based on the depth allowed by HVAC and ceiling heights.

General

The current floor system was found to weigh 45.4 pounds per square foot (psf), with only the composite steel beams/girders system weighing less out of the four options considered. This value will be used as the basis for assessing the alternate floor system weights and their potential impacts on the structure with respect to foundations and lateral systems.

The depth, from the top of the slab, to the bottom of the steel joists is 20". This is the effective depth occupied by the structure in the interior portion of the bay (excluding the perimeter framing members). This 20" is the deepest of the floor systems considered. The maximum total depth including the W24x76 girder is 30 1/2" (includes 2 1/2" for the joist seat).

The cost of the composite deck slab on steel joists floor was estimated to be $14.52/sf. This came out as the second most economical system behind composite steel beams/girders. Using RSMeans Building Construction Cost Data 2012, separate costs were found for the deck, concrete, placement, joists and girders and broken down into equivalent values per square foot. Those values were then combined to come up with a unit assembly cost for comparison and assessment.

Architectural

The 20" depth of the slab and joists will be the base value used for comparison with the alternate floor systems, since the 16" deep joists were chosen based on allowances for mechanical equipment and ceiling heights actually used in the Office Building. Although 20" is the greatest depth for any of the floors considered, the use of open web steel joists also means that some MEP equipment could potentially run through the open web space instead of beneath the joists altogether.

All typical structural elements in the Office Building require a 1-hour fire rating (except the supporting structure for the stairs and elevator shafts, which require a 2-hour rating). In order to achieve this rating for the floor system and bay considered, sprayed-on cementitious fireproofing is required for all elements. This includes application to the bottom of the deck, since only 2 1/2" of normal weight concrete topping is specified and 3 1/2" would be required to leave the deck unprotected.

Structural

The existing foundations are primarily shallow spread and strip footings and the lateral system is made up of double angle braced frames. The foundations and lateral system have been designed for the gravity and/or lateral loads associated with this specific floor system. Thus, they will serve as the basis for comparison of the alternate floor systems and the potential impacts they will have on the overall building structure.

Serviceability

While deflections were not actually calculated for the open web steel joists, the members are loaded to around 78% of the allowable capacity for live load deflection and 83% of the allowable capacity for total load deflection. The maximum deflections due to live and total loads on the W24x76 girders are 0.73" and 1.38", respectively.

This type of floor system is commonly susceptible to vibration, due to its light weight and low overall stiffness properties. The 4" total slab thickness was specified, in part, to add some extra weight to resist vibrations. A few edge girders throughout the building were also upsized to help limit vibrations in accordance with AISC Design Guide 11.

Construction

The type of steel construction for this floor system is very typical and can be constructed quickly and efficiently. The deck acts as leave-in-place formwork for the concrete. Also, the deck does not need to be shored based on the distance it needs to span, which will speed up installation. However, fireproofing does need to be applied to all of the structural elements in this system, which adds cost and time to the overall schedule.

Summary

The existing floor system is rather typical for office buildings such as this. Its use is well justified based on the quick easy construction, light overall weight and relatively low cost.

Composite Deck Slab on Composite Steel Beams/Girders
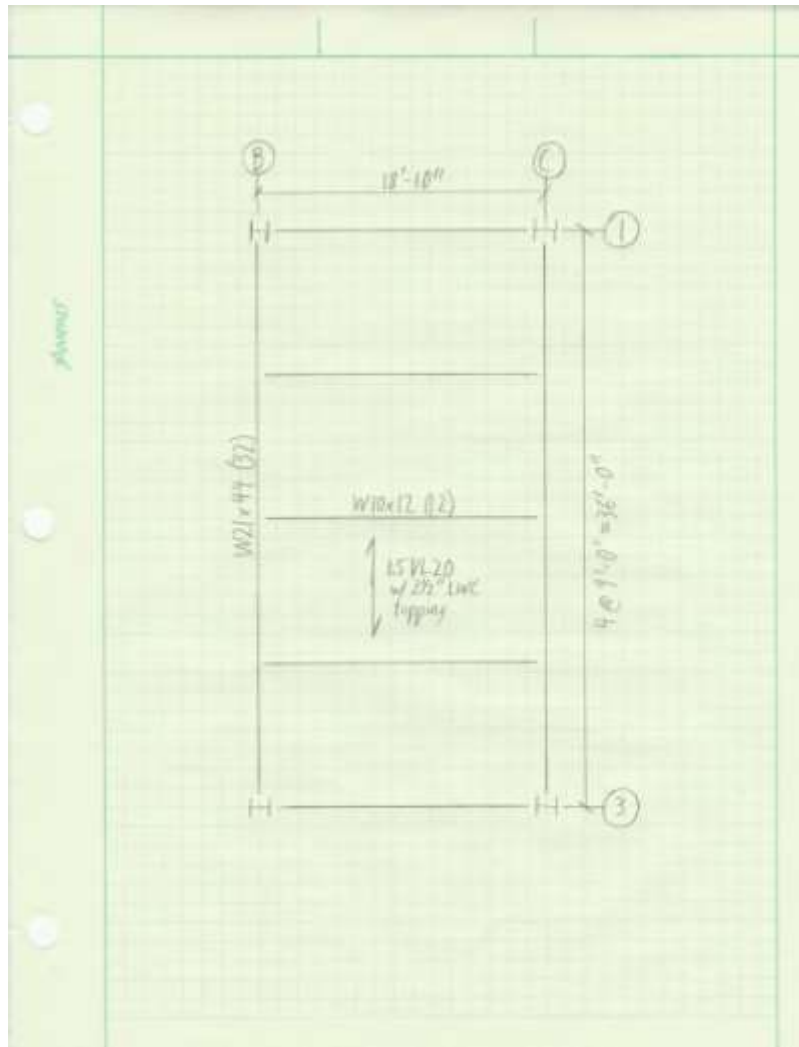
Figure 12: Sketch of Composite Steel Layout

The first alternate floor system considered for the Office Building is actually quite similar to the existing system. However, in this case the composite deck and steel support members have been designed to act together compositely. This is accomplished by providing adequate shear transfer strength between the composite slab and beams/girders through the use of shear studs.

Based on a 9'-0" beam spacing, which worked well for both this 36'-0" long bay and the 18'-0" long adjacent bays, and the required 1-hour fire rating for the floor, a 1 1/2" composite deck was selected for use with 2 1/2" of lightweight concrete topping. The beams are W10x12 with twelve 3/4" shear studs spaced evenly along the length of each beam. The girders are W21x44 with thirty-two 3/4" shear studs spaced evenly along their lengths.

General

The composite steel system was found to weigh 33.8 pounds per square foot (psf), which is significantly lighter than any of the other three floor options. This makes the composite system nearly 26% lighter than the existing floor design.

The depth, from the top of the slab, to the bottom of the steel beams is 14". This is the effective depth occupied by the structure in the interior portion of the bay (excluding the perimeter framing members). At 14", this system is the second deepest of the floor systems considered at 6" less than the existing floor. The maximum total depth including the W21x44 girder is 25".

The cost of the composite steel floor was estimated to be $13.69/sf. This is the least expensive system at $0.83/sf less than the existing floor design. Using RSMeans Building Construction Cost Data 2012, separate costs were found for the deck, concrete, placement, beams, girders and shear studs and broken down into equivalent values per square foot. Those values were combined to get a unit assembly cost to be used for comparison and assessment.

Architectural

The 14" depth of the slab and beams could potentially permit a reduction of the floor-to-floor heights by 6", while still providing the allowances for mechanical equipment and ceiling heights specified for the original design. A 6" reduction of the 13'-4" story heights would add up to a total building height reduction of 30" (or 2'-6"). There is also a 5 1/2" decrease in the maximum total depth to the bottom of the girder for the composite system. If the story heights were reduced by 6" as mentioned, the bottom of the girder would project 1/2" lower than the girder in the original layout. There would be 1/2" less clearance between the finish floor and the bottom of the girder, which would likely be a tolerable change.

As previously described for the existing floor system, all typical structural elements in the Office Building require a 1-hour fire rating (typical elements do not include the supporting structure for the stairs and elevator shafts). For the composite system and bay considered, sprayed-on cementitious fireproofing is required for the beams and girders. However, providing 2 1/2" of lightweight concrete topping over the 1 1/2" composite deck allows the bottom of the deck to be left unprotected while still providing a 1-hour fire rating.

Structural

With the significant reduction in overall floor system weight, it is likely that the shallow foundations may be able to be reduced in size and/or have the amount of reinforcing reduced. Column sizes could also be reduced. Since the lateral loading, calculated for Technical Report 1, due to wind was found to be significantly greater than that due to seismic (by a factor of about 1.7), the reduction of the floor weight may not have much of an impact on the lateral system.

However, it may influence the lateral design through second-order (P-Δ) effects. Under less load, these effects could be minimized to where a reduction of the bracing member sizes or a reduction in the number of braces and/or braced frames could be possible.

Serviceability

Maximum deflections of the W10x12 beam under live and total loads were found to be 0.623" and 0.907", respectively. These values were calculated using the lower-bound moment of inertia of the composite section (I_{lb}=141 in^4). Both deflections are within 4% of the maximum allowable values for the span distance.

Maximum deflections of the W21x44 girder under live and total loads were found to be 0.935" and 1.56", respectively. These were also calculated using the lower-bound moment of inertia of the composite section (I_{lb}=1560 in^4).

Similar to the existing floor design, this type of floor system is commonly susceptible to vibration problems. In this case, the system is even lighter. It makes use of lightweight concrete and raises serious concern over vibrations which can be excited by smaller amounts of force impacting the floor because of its decreased mass. While superimposed loads from the ceiling and mechanical equipment should help in damping the floor, other methods of damping may need to be investigated if this design is to be considered further.

Construction

The type of steel construction for this floor system is very commonly used in modern office buildings. It is easy and quick to construct. While shear studs need to be individually welded along the lengths of the beams and girders, there are fewer beams to be installed than there are joists for the existing system. The deck acts as leave-in-place formwork for the concrete. Also, the deck does not need to be shored before the slab is poured based on the allowable SDI maximum unshored clear span, which will speed up installation. Fireproofing does need to be applied to each of the beams and girders, but not to the underside of the steel deck.

Summary

The composite floor system is both the lightest and least expensive option being considered. The smaller depth of the beams may allow for smaller floor-to-floor heights to decrease the overall height of the building and cut down on both construction costs and schedule. Construction of such a typical modern framing layout would be relatively quick and easy. Perhaps the greatest concern with this system is its potential for vibration problems, which are likely the worst out of the three alternate systems considered.

It would be worthwhile to look into the vibration concerns further and figure out the best way to deal with this issue. Otherwise, the composite floor system is a very viable alternate.

One-way Slab
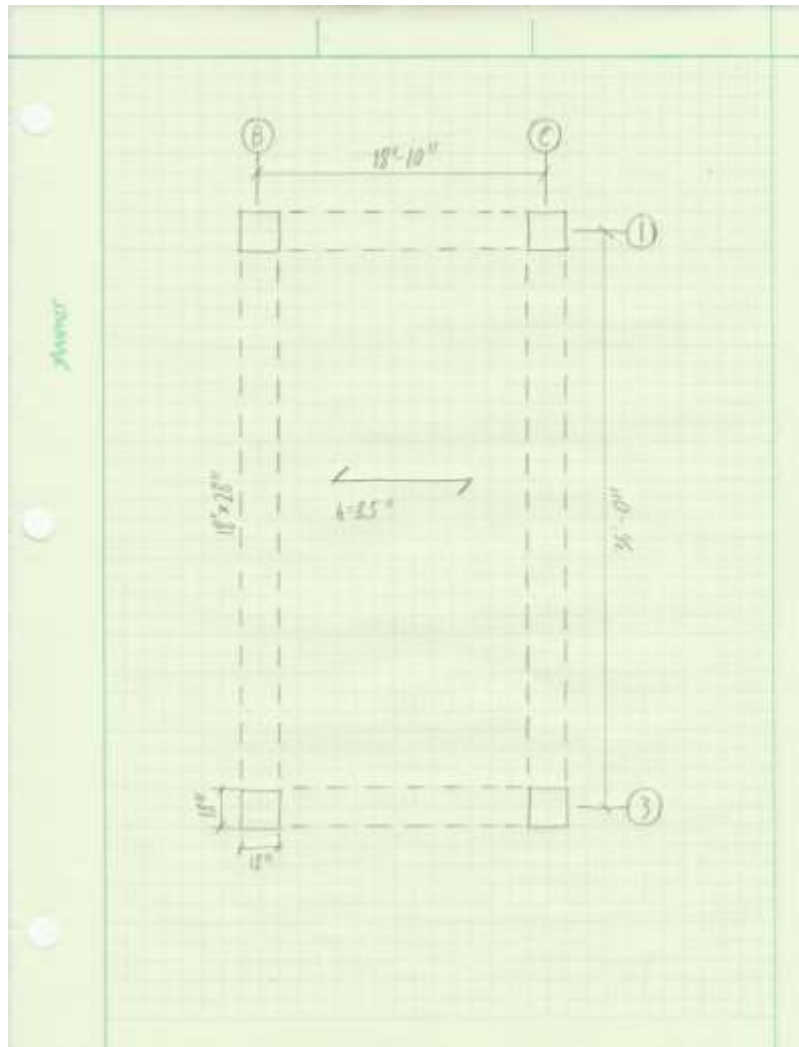
Figure 13: Sketch of One-way Slab Layout

The second alternate floor system being considered for the Office Building is a one-way slab. One-way slabs were considered because the ratio of the long to short spans of the bay is nearly 2. The slab was designed to span between grids B and C (18'-10") to concrete beams along grids B and C which span between grids 1 and 3 (36'-0"). Normal weight concrete with a compressive strength (f'_c) of 4 ksi and grade 60 steel reinforcement were used in the design. Also, to determine the clear spans for the beams, concrete columns were assumed to be 18"x18".

The slab was designed as 8 1/2" inches thick. The negative moment reinforcement at each end of the slab (at the slab-beam interface) is #4 at 8" and the positive moment reinforcement at midspan is #4 at 12". In the direction perpendicular to the slab span, #4 at 12" are used for shrinkage and temperature reinforcement.

The beam was designed based on an assumed width of 18" to match the assumed column width. The beam was designed to be 18"x28" (h=28"). Because the beam is located at an end span, the negative moment at the face of the first interior support is larger than the negative moment at the face of the exterior support. The negative moment (tension) reinforcement at the exterior support is (2) #9 and (2) #8. The positive moment (tension) reinforcement at midspan is (4) #9. The negative moment (tension) reinforcement at the interior support is (4) #9 and (2) #8.

The shear in the beam was increased by 15% at the face of the first interior support because it is an end member. The shear reinforcement in the beam from the face of the interior support is (1) at 2", (13) at 6" and (9) at 12". From the face of the exterior support, the shear reinforcement is (1) at 2", (6) at 8" and (9) at 12". Shear reinforcement is #3 bars in the form of a "U" stirrup at each location (equal to two legs of #3 bar resisting shear at each location).

General

The one-way slab system was found to weigh 127 pounds per square foot (psf), which is more than twice as heavy as any of the other three floor options. This makes the concrete system about 180% heavier than the existing floor design.

The depth, from the top to the bottom of the slab is 8 1/2". This is the effective depth occupied by the structure in the interior portion of the bay (excluding the perimeter beams). At 8 1/2", this system is the second most shallow of the floor systems considered and is 11 1/2" less than the existing floor. The maximum total depth including the beams is 28".

The cost of the one-way slab floor was estimated to be $18.19/sf. This is the most expensive system and is $3.67/sf more than the existing floor design. Using RSMeans Building Construction Cost Data 2012, separate costs were found for the formwork, concrete, placement and reinforcement and broken down into equivalent values per square foot for both the slab and the beam. Those values were combined to get a unit assembly cost to be used for comparison and assessment.

Architectural

The 8 1/2" depth of the slab could potentially permit a reduction of the floor-to-floor heights by 11 1/2", while still providing the allowances for mechanical equipment and ceiling heights specified for the original design. An 11 1/2" reduction of the 13'-4" story heights would add up to a total building height reduction of 57 1/2" (or 4'-9 1/2"). There is also a 2 1/2" decrease in the maximum total depth to the bottom of the beam for the concrete system. If the story heights were reduced by 11 1/2" as mentioned, the bottom of the concrete beam would project 9" lower than the steel girder in the original layout. There would be 9" less clearance

between the finish floor and the bottom of the beam, which is significant and would likely not be tolerable.

With the concrete construction proposed for this system of slabs and beams, no additional fireproofing is required to be added to the structure for it to reach a 1-hour fire rating. For this reason, the structure does not necessarily need to be concealed. While a drop ceiling would still likely be added to hide different elements of the building systems and for acoustic reasons, all or parts of the structure (such as protruding deep beams) could potentially be left exposed.

Structural

This is by far the heaviest system considered and it would have a large impact on the rest of the building structure. The increased dead weight of the structure would certainly impact the size of the foundations and maybe even the type. Currently the foundations include shallow spread and strip footings. The sizes and/or amount of reinforcement would need to be increased for the greater loads and other options such as deep foundations might need to be considered.

Perhaps the biggest structural impact this floor system could have is on the lateral system. With the increased weight, seismic loads would likely be significantly greater than wind loads and the lateral system would need to be designed to resist the seismic loads induced by the buildings response to ground motion/acceleration. Cast-in-place shear walls and/or dual systems would likely be considered as options for this floor system. The increased lateral forces would also have a direct impact on the foundations.

Serviceability

Table 9.5 (a) of ACI 318-11 specifies minimum depths for both slabs and beams, above which deflections need not be calculated. From this table, the minimum depths of the slab and beam were found to be 8.08" and 23.4", respectively. The final depth of the slab was 8 1/2" and the depth of the beam was 28". Therefore, deflections were not required to be checked.

The one-way slab system with a slab depth of 8 1/2" is easily the best system for controlling vibrations. The high level of stiffness and considerable mass of the slab are ideal properties when dealing with vibrations in a floor system.

Construction

One-way slabs are not all that typical for this type of office building and its location in northern Pennsylvania. Because of this and the increased labor that goes into forming, reinforcing and placing concrete, this system would likely have the longest construction time. Also, required curing times for the concrete to gain strength would lengthen the overall schedule. However, fireproofing does not need to be added to any of the structural elements due to concrete's

inherent fire resistant properties. This could have the potential to shorten the construction time some.

Summary

The one-way slab system turned out to be the most expensive and heaviest option considered. It would have the biggest impact on the rest of the building structure, including foundations and the lateral system. The construction process is also a lot more labor intensive and the schedule will reflect that by being lengthened.

Because of the minimal slab depth, nearly a foot could potentially be taken off of the typical story height. Although, there would need to be a further check into the beam depth and its projection before reducing the floor-to-floor heights. Vibration control is very good for a system with this kind of mass and with the greater overall stiffness gained through monolithic construction. There is also no need for further fire protection with this system.

One-way slabs do not appear to be a viable option for an alternate system at this time. This is based on the increased cost and much greater weight of the system, which will have huge impacts structurally on both the foundations and lateral force resisting system.

Precast Hollow-core Planks on Steel Girders
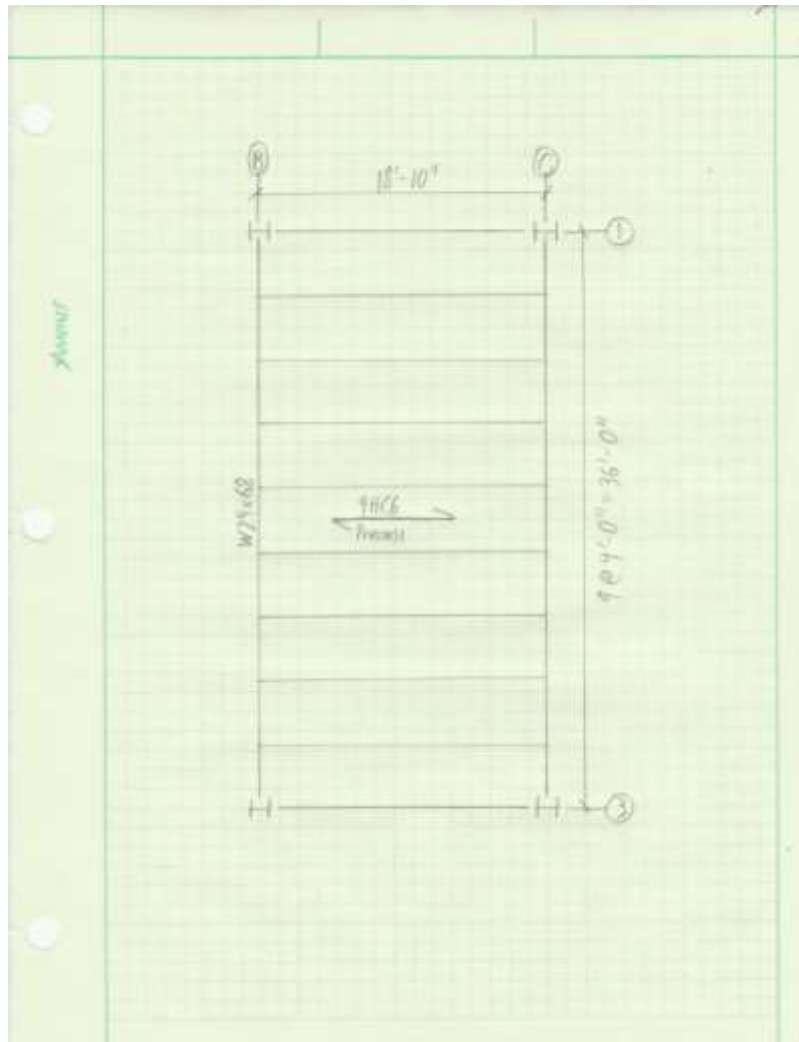
Figure 14: Sketch of Precast Hollow-core Plank Layout

The third and final alternate floor system considered was precast hollow-core planks supported by steel wide flange girders. All plank design data was taken from the PCI Design Handbook, 6th Edition. To span the 18'-10" between grids B and C, 4HC6 hollow-core planks (4'-0" wide by 6" thick) with a 66-S strand designation code were chosen (untopped). The girders that span 36'-0" between grids 1 and 3 are W24x68 wide flanges.

General

The precast plank system was found to weigh 52.9 pounds per square foot (psf), which is the second heaviest of all the floor options. This makes the precast plank system nearly 17%

heavier than the existing floor design. However, the planks are still well less than half the weight of the one-way slab system.

The depth, from the top to the bottom of the planks is 6". This is the effective depth occupied by the structure in the interior portion of the bay (excluding the perimeter girders). At just 6", this system is the shallowest of the floor systems considered and is 14" less than the existing floor. The maximum total depth including the girders is 30" if the planks rest on the top flange of the steel girders. The planks can also be installed so that the top of the plank matches the top of the steel girder. This could be done by welding steel plate supports to the web of the girders, with the bearing elevation at 6" below the top of the flange, and setting the planks onto those supports. In this case, the maximum total depth with the girder included would be just 24". For the rest of the precast plank system assessment, the second installation option will be used and the tops of the girders and the tops of the planks will be assumed flush with one another.

The cost of the hollow-core plank floor was estimated to be $15.52/sf. This is the second most expensive system and it costs $1.00/sf more than the existing floor design. Using RSMeans Building Construction Cost Data 2012, separate costs were found for the precast planks and steel girders and broken down into equivalent values per square foot. Those values were combined to get a unit assembly cost to be used for comparison and assessment.

Architectural

The 6" depth of the planks could potentially permit a reduction of the floor-to-floor heights by 14", while still providing the allowances for mechanical equipment and ceiling heights specified for the original design. A 14" reduction of the 13'-4" story heights would add up to a total building height reduction of 70" (or 5'-10"). There is also a 6 1/2" decrease in the maximum total depth to the bottom of the girder for the precast plank system. If the story heights were reduced by 14" as mentioned, the bottom of the girder would project 7 1/2" lower than the steel girder in the original layout. There would be 7 1/2" less clearance between the finish floor and the bottom of the girder, which is significant and would likely not be tolerable.

With the precast concrete planks proposed for this floor system, no additional fireproofing should be required for the planks to reach a 1-hour fire rating. For this reason, the structure does not necessarily need to be concealed. Oftentimes with precast hollow-core plank construction, the underside of the plank is exposed and serves as the ceiling in many cases where it is used. While a drop ceiling would still likely be added to hide different elements of the building systems and for acoustic reasons, parts of the structure where the planks are used could potentially be left exposed. The steel girders do require a sprayed-on cementitious fireproofing to be applied to reach the required 1-hour fire rating.

Structural

The increased dead weight of the structure with the precast plank floor system would likely increase the size of the foundations (spread and strip footings) and/or their reinforcement a little, but not too significantly. Because this is not a drastic increase in the weight of the building, wind loading will probably still be greater than seismic loads. With the additional weight though, second-order (P-Δ) effects could cause the need for an increase in size/capacity of the steel bracing members and/or of the shallow foundations.

Serviceability

Because the precast hollow-core planks were selected based on allowable superimposed loads, direct calculations for deflections were not required. The service live and total loads calculated were within 4% of the maximum allowable loads from the PCI table so the deflections should be very near the limits based on span length.

The maximum deflections of the W24x68 girder under live and total loads are 0.84"and 1.62", respectively.

While the exact vibration susceptibility of the precast plank system is not known, it seems that this system should fall somewhere between the one-way slab and composite steel system behaviors. There is a greater amount of mass for the precast planks for damping than with the composite steel floor. On the other hand, the planks do not have the high overall system stiffness that is achieved through the continuous spans of monolithic concrete construction. The precast plank system will be considered average for vibration control.

Construction

The biggest concern for the construction of the precast plank system is the labor intensive nature of the potential bearing plate-to-girder web welded connection with stiffener plates that would be needed for the top of the girder flange to align with the top of the precast panels. This could lengthen the construction time for this system significantly. The large precast panels would also likely be expensive to transport to the building site and could affect the schedule depending on how long it takes for them to be delivered to the site and in what quantities.

However, for the planks there is no need for formwork of any kind and no time is taken on the job for the concrete to cure. The planks do not require any additional fireproofing, while the girders still do. If the planks are simply supported on top of the girders instead of being dropped down to be flush with the flanges, no special connection detailing will be required and both cost and schedule will be decreased.

Summary

The precast hollow-core plank system turned out to be the second most expensive and second heaviest option considered. It would likely have some impact on the rest of the building structure, including foundations and the lateral system, but nothing extremely significant. The construction process could be a lot more labor intensive than with the existing and composite steel floor options, especially with the welded connections proposed for along the web of the girders to act as the plank seat. The construction schedule would reflect this by being lengthened.

Because of the minimal plank depth, over a foot could potentially be taken off of the typical story height. Although, there would need to be a further check into the girder depth and its projection before reducing the floor-to-floor heights. It is important to note that the girder would need to be concealed somehow due to the sprayed-on fireproofing it requires. Vibration control is average to above average for a system with this kind of mass and overall stiffness. There is also no need for further fire protection of the planks with this system, but the girders still need to be sprayed with fireproofing.

The cost estimate for the plank system did not include any allowances for the welded connections proposed for the girder web. As a result, the estimate should probably be closer to the cost calculated for the all-concrete system, currently the most expensive option. Also of concern is the potential lower overall rigidity of the floor diaphragm since each of the planks is independent of the others. Perhaps special connection detailing could help the diaphragm to act in a more rigid manner. This should be investigated further to see what kind of impacts the rigidity of the floor diaphragm could have on the distribution of lateral loads to the lateral system and foundations. Another potential issue to consider is that, if the planks are supported by connections to the girder web, uneven adjacent spans could cause differential torsional loads to be transferred directly into the web of the girders.

At this time, it does not appear that the precast planks are a viable option for an alternate system. This is based on the increased cost and greater weight of the system, which will have some impact structurally on both the foundations and lateral force resisting system. While the system could provide a reduction of the typical story height, to do so would require that the special welding connections be designed and detailed to support the planks at the girder web. This will add costs to the installation. Due to the projection of the girder below the planks, the most the story height could likely be reduced would be 6 1/2", which is very close to the potential 5 1/2" or 6" that could be reduced with the significantly less expensive and less heavy composite steel floor. In the end, the benefits of hollow-core planks do not justify its use as an alternate system.

Floor Systems Summary Comparison

Consideration		System			
		Composite Deck on Steel Joists	Composite Steel	One-way Slab	Precast Hollow-core Planks
General	Weight (psf)	45.4	33.8	127	52.9
	Cost ($/sf)	14.52	13.69	18.19	15.52
	Primary Slab/Framing Depth (in)	20	14	8.5	6
	Total System Depth (in)	30.5	25	28	24 or 30
Architectural	Fire Rating	1-hr	1-hr	1-hr	1-hr
	Other	Open web joists provide more accessible plenum space	Could reduce story height by upwards of 6"	Could reduce story height by upwards of 11.5"	Could reduce story height by upwards of 14"
Structural	Foundation	Shallow spread/ strip footings	Reduced foundation sizes and/or reinforcement	Increased sizes or possible deep foundations	Increased foudation sizes and/or reinforcement
	Lateral System	Double angle "K" braced frames	Possible reduction in no. and/or size of braces	Possibly cast-in-place concrete shear walls	Possible increase in size of braces
Serviceability	Maximum Deflection (in)	1.38	1.56	N/A	1.62
	Vibration Control	Poor to average	Poor	Good	Average to good
Construction	Additional Fire Protection	Sprayed-on for all elements	Sprayed-on for beams/girders	None	Sprayed-on for girders
	Schedule	N/A	May reduce duration	May increase duration significantly	May increase duration
	Constructability	Easy	Easy	Medium to Difficult	Medium to Difficult
Viable Alternate		N/A	Yes	No	No

Conclusion

Technical Report 2 has analyzed the existing floor system of the Office Building as well as three other alternate framing systems. The existing system of composite deck slab on steel joists was used as the basis for comparison of the three alternate systems, which include composite deck slab on composite steel beams/girders, one-way slabs and precast hollow-core planks on steel girders. The assessment was made for use in a single bay of the building, which would likely control the design of the overall floor system.

All four systems were compared and assessed in terms of weight, cost and depth and the general, architectural, structural, serviceability and constructability impacts of the systems were discussed. After looking into all of these aspects of the various floor systems, a conclusion was reached for each system as to whether or not it is viable and should be considered further for use in the Office Building.

The composite steel system turned out to be both the lightest and least expensive of all systems considered. Another benefit of this system was the potential for reducing floor-to-floor heights in the building. While this system is particularly susceptible to vibration and this issue still needs to be investigated further and addressed, this floor came out as the only viable option for an alternate floor that competed with the low cost and weight of the original design for the Office Building (of the alternates that were considered).
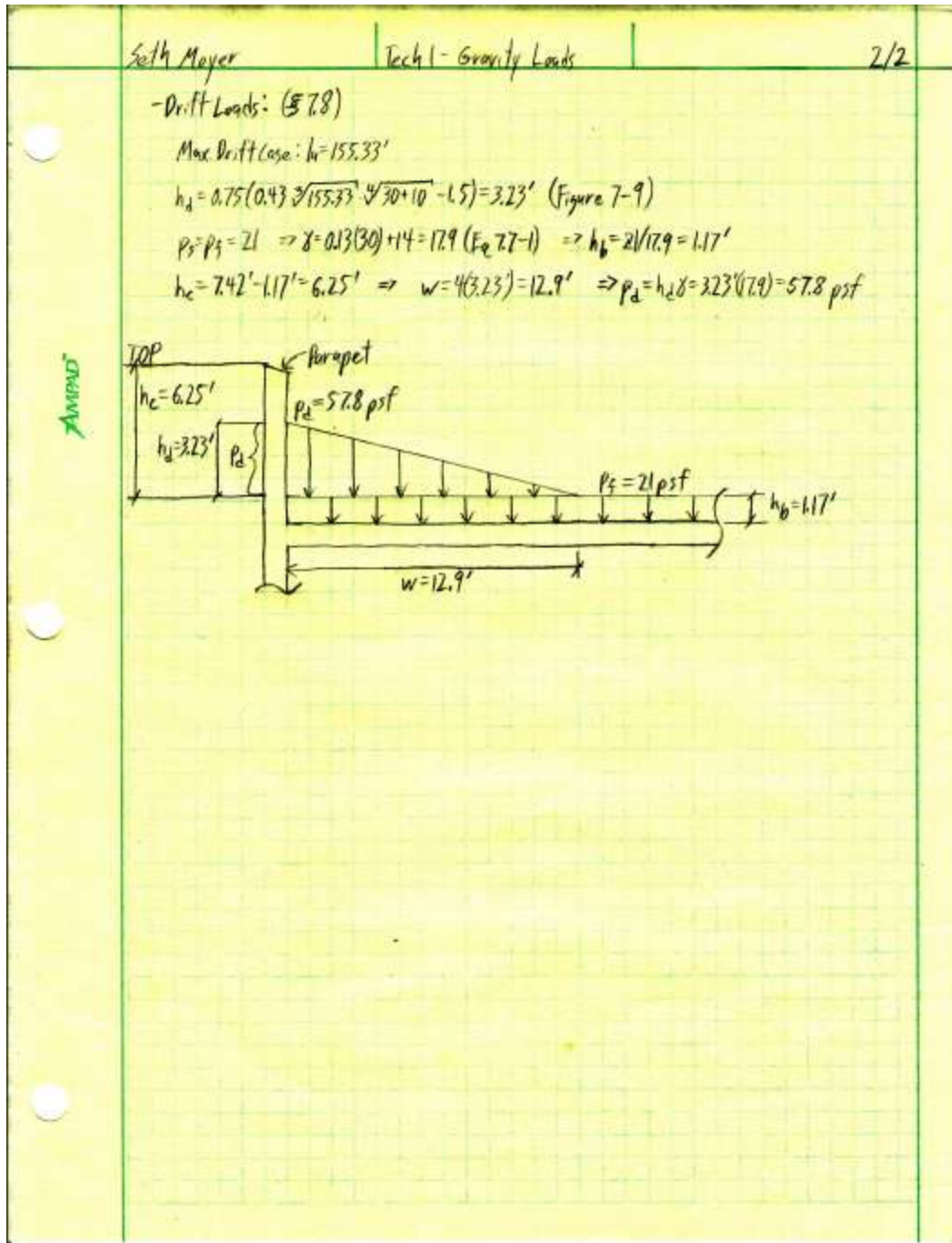
The one-way slab system was the heaviest and most expensive option. The greatly increased dead loads from the concrete structure would necessitate a redesign of the foundations and lateral system, which seismic would have a much greater effect on. Although the system did provide inherent fireproofing and vibration control due to its stiffness and mass, the one-way slab system was deemed an unfeasible alternative for the Office Building.

The precast hollow-core planks system had an increased cost (could potentially be even higher after taking welded connections to the girder web into account), more difficult construction and greater weight than the existing system. Unfortunately, the benefits it presented were similar to those offered by the composite steel design (a less expensive, lighter option). In the end, the pros of the precast plank floor did not justify its use as an alternate system when compared to other floor systems with greater potential.

Appendix A

Seth Moyer | Tech 1 - Gravity Loads | 1/2

-Dead Loads:

   Roof: 1½" Type B 20 ga. wide rib roof deck = 2.14 psf   (Vulcraft Deck Cat.)

      24K4 @ 6' OC = 8.4 plf/6' = 1.4 psf   (SJI)

      4" Rigid Insulation = 6 psf

      EPDM = 0.7 psf

      MEP/Ceiling = 10 psf

      Total = 2.14 + 1.4 + 6 + 0.7 + 10 = 20.24 => 20 psf

   Floor: 2½" thk conc. slab on 20 ga. 1½" composite deck = 39 psf (Vulcraft Deck Cat., 1.5 VL20)

      16K4 @ 3' OC = 7.0 plf/3' = 2.33 psf   (SJI)

      Bms/Girders = 7 psf

      MEP/Ceiling = 10 psf

      Total = 39 + 2.33 + 7 + 10 = 58.33 => 60 psf

-Live Loads: (ASCE 7-10, Table 4-1)

   Roof: 20 psf

   Floor: Corridors above first floor = 80 psf

      Partitions = 20 psf

      Total = 80 + 20 = 100 psf

-Snow Loads:

   -Ground Snow Load: $p_g = 30$ psf   (ASCE 7-10, Figure 7-1)

   -Exposure Factor: $C_e = 1.0$ (Partially Exposed)   (Table 7-2)

   -Thermal Factor: $C_t = 1.0$ (Table 7-3)

   -Importance Factor: $I_s = 1.0$ (Table 1.5-2)

   -Flat Roof Snow Load: $p_f = 0.7(1.0)(1.0)(1.0)(30) = 21$ psf (Eq. 7.3-1)

Seth Moyer — Tech 1 - Gravity Loads — 2/2

- Drift Loads: (§ 7.8)

Max. Drift Case: $h = 155.33'$

$h_d = 0.75(0.43 \sqrt[3]{155.33} \cdot \sqrt[4]{30+10} - 1.5) = 3.23'$ (Figure 7-9)

$p_s = p_f = 21 \Rightarrow \gamma = 0.13(30) + 14 = 17.9$ (Eq 7.7-1) $\Rightarrow h_b = 21/17.9 = 1.17'$

$h_c = 7.42' - 1.17' = 6.25' \Rightarrow w = 4(3.23') = 12.9' \Rightarrow p_d = h_d \gamma = 3.23'(17.9) = 57.8 \text{ psf}$

TOP   Parapet

$h_c = 6.25'$

$h_d = 3.23'$ p_d

$p_d = 57.8 \text{ psf}$

$p_f = 21 \text{ psf}$

$h_b = 1.17'$

$w = 12.9'$

Appendix B

Seth Moyer   Tech 2 - Composite Deck/Beam   1/5

DL = 10 psf (superimposed - MEP)

LL = 100 psf

TL = 110 psf (superimposed)

Based on required 1-hr fire rating and a 9' beam spacing (to work w/ 18' adjacent bays), use Vulcraft 1.5 VL 20 w/ 2½" LW concrete topping

SDI max. unshored clear span (3-span cond.) = 9'-9" > 9'-0" ∴ OK

Allowable superimposed load @ 9'-0" sp = 151 psf > 110 psf ∴ OK

Total slab depth = 4"

Total slab weight = 30 psf

Beam:

DL : Slab/deck wt. = 30 psf
    Superimposed DL = 10 psf   } Total DL = 45 psf
    Beam self wt. allow = 5 psf

LL: L_0 = 100 psf $K_{LL}A_T$ = 18(18 + 10/12) = 339 sf < 400 sf ∴ not reducible

W_u = 1.2(45) + 1.6(100) = 214 psf

w_u = 214(9')/1000 = 1.93 klf

M_u = 1.93 (18 + 10/12)^2/8 = 85.6 ft-k

$I_{req'd}$ for Wet Conc. Deflection:

w_{wc} = (30 + 5) 9' = 315 plf

$\Delta_{wc} = \frac{(18 + 10/12)12}{240} = 0.94" \geq \frac{5(0.315)(18 + 10/12)^4(1728)}{384(29,000) I_{req'd}}$ ⟹ $I_{req'd} \geq 32.7$ in^4 ⟹ Try W10x12 (I = 53.8 in^4)

Seth Moyer   Tech 2 - Composite Deck/Beam   2/5

Check ϕM_n

$b_{eff} = 2 \times \min \begin{cases} (18'\cdot 12/12)\cdot 12/8 = 28.4" \\ 9(12)/2 = 54" \end{cases} = 56.5"$

If $a \approx 1 \Rightarrow Y2 = 4 - 0.5 = 3.5"$

W10x12 w/ $\Sigma Q_n = 93.8$ $Y2 = 3.5" \Rightarrow \phi M_n = 89.5$ ft-k > 85.6 ft-k $\therefore$ OK

$a = 93.8/[0.85\ (3)(56.5)] = 0.65" < 1" \therefore OK$

Shear Studs:

Deck $\perp$; LWC; Weak Stud Position; 1 stud/rib; $f'_c = 3$ ksi

$17.2^k/$stud   ($\frac{3}{4}"$ stud)

$93.8/17.2 = 5.45 \Rightarrow 6$ studs $\times 2 = 12$ studs/beam

Actual $\Sigma Q_n = 6(17.2) = 103.2^k \Rightarrow 103.2/[0.85(3)(56.5)] \Rightarrow a = 0.72" < 1" \therefore OK$

Check Unshored Strength

W10x12 $\Rightarrow \phi M_p = 46.9$ ft-k   Const. LL

$w_u = 1.2[30(9') + 12] + 1.6[20(9')] = 0.626$ klf

$M_u = 0.626 (18 + 10/12)^2/8 = 27.8$ ft-k < 46.9 ft-k $\therefore$ OK

Check LL Deflection:

$w_{LL} = 100(9') = 0.9$ klf

I_{LB} w/ $\Sigma Q_n = 93.8$, $Y2 = 3.5" \Rightarrow 141$ in^4

$\Delta_{LL} = \dfrac{5(0.9)(18 + 10/12)^4 (1728)}{384(29,000)(141)} = 0.623" < \dfrac{18 + 10/12(12)}{360} = 0.628" \therefore OK$

$\Rightarrow$ Use $\boxed{\text{W10x12 w/ (12) } \frac{3}{4}" \text{ shear studs}}$ spaced evenly along length

Check beam self wt allow $12/9' = 1.33$ psf < 5 psf assumed $\therefore$ OK

Check TL Deflection

$w_{TL} = 145(9') = 1.31$ klf $\Rightarrow \Delta_{TL} = \dfrac{5(1.31)(18 + 10/12)^4 (1728)}{384(29,000)(141)} = 0.907" < \dfrac{18 + 10/12(12)}{240} = 0.942" \therefore OK$

Seth Moyer | Tech 2: Composite Deck/Beam | 3/5

-Girder:

DL = 45 psf

$LL = 100(0.25 + 15/\sqrt{(18+10/12 + 16 + 3/12) \times 36}) = 67.3$ psf

$W_u = 1.2(45) + 1.6(67.3) = 162$ psf

$P_u = 162(9')(18 + 10/12 + 16 + 2/12)/2/1000 = 25.5^k$

$M_u = 459$ ft-k

J req'd for Wet Conc. Deflection

$W_{wc} = (30 + 5)9' = 315$ plf

$P = 315(18 + 10/12 + 16 + 2/12)/2/1000 = 5.51^k$

Superposition of Δ_{max} for load at center & than equal symmetrically placed loads

$\Delta_{wc} = \frac{36(12)}{240} = 1.8" = \frac{5.51(9')(3(36)^2 - 4(9)^2)(1728)}{24(29,000) I_{req'd}} + \frac{5.51(36)^3(1728)}{48(29,000) I_{req'd}} \Rightarrow I_{req'd} \geq 421 in^4$

$\Rightarrow$ Try W18x35 $(I = 510 in^4)$

Check ϕM_n

$b_{eff} = \min \begin{vmatrix} 36(12)/8 = 54" \\ (18 + 10/12)(12)/2 = 113" \end{vmatrix} + \min \begin{vmatrix} 54" \\ (16+2/12)(12)/2 = 97" \end{vmatrix} = 108"$

If $a = 1" \Rightarrow Y2 = 4 - 0.5 = 3.5"$

W18x35 w/ $\Sigma Q_n = 451$, Y2 = 3.5" $\Rightarrow \phi M_n = 460$ ft-k > 459 ft-k $\therefore$ OK

$a = 451/[0.85(3)(108)] = 1.64" > 1" \therefore$ calc ϕM_n

$A_s F_y = 10.3(50) = 515^k \Rightarrow a = 515/[0.85(3)(108)] = 1.87" < 2.5" \therefore$ OK (topping thickness)

$\phi M_n = 0.9(515)(17.7/2 + 4 - 1.87/2)/12 = 460$ ft-k > 459 ft-k $\therefore$ OK

Seth Moyer　　　　　　　Tech 2 - Composite Deck/Beam　　　　　　4/5

Shear Studs:

Deck ∥; LWC; $f'_c = 3$ ksi; $w_r/h_r = 1.75/1.5 = 1.17 < 1.5$

17.1 ^{k}/stud (3/4" stud)

$\Sigma Q_n = 515/17.1 = 30.1 \Rightarrow 31$ studs $\times 2 = 62$ studs/girder $\Rightarrow$ a lot of studs, check diff sizes

W16×40 ✓ $\Sigma Q_n = 502 < Y2 = 3$ (a=2) $\Rightarrow \phi M_n = 466$ ft-k > 459 ft-k ∴ OK

$a = 502/[0.85(10.0)(3)] = 1.82" < 2"$ ∴ OK

$502/17.1 = 29.4 \Rightarrow 30$ studs $\times 2 = 60$ studs/girder

W18×40 ✓ $\Sigma Q_n = 274$ × Y2 3.5 (a=1) $\Rightarrow \phi M_n = 462$ ft-k > 459 ft-k ∴ OK

$a = 274/[0.85(10.0)(3)] = 0.99 < 1"$ ∴ OK

$274/17.1 = 16.02 \Rightarrow 17$ studs × 2 = 34 studs/girder　　$= 17(17.1) = 290.7 = \Sigma Q_n$　$\Rightarrow 290.7/0.85(10.0)(3)$
$a = 1.06 = 1"$

"Assuming cost of 1 stud (installed) ≈ cost of 10 lbs of steel weight

W18×35 = 35(36') + 62(10) = 1880 lb

W16×40 = 40(36') + 60(10) = 2040 lb

W18×40 = 40(36') + 34(10) = 1780 lb $\Rightarrow$ most economical w/o increasing depth

Check Unshored Strength:

W18×40 $\Rightarrow \phi M_p = 294$ ft-k

$w_c = 1.2(35)(9') + 1.6(20)(9') = 0.666$ klf　← Const LL

$P_u = 0.666(35/2) = 11.7^k$

$M_u = 211$ ft-k < 294 ft-k ∴ OK

Check Δ_{LL}:　I_{LB} ✓ $\Sigma Q_n = 274$, Y2 35 $\Rightarrow 1190$ in^4

$w_{LL} = 67.3(9') = 606$ plf

$P_a = 0.606(35/2) = 10.6^k$

$\Delta_{LL} = \dfrac{10.6(12")[3(36)^2 - 4(9)^2](1728)}{24(29,000)(1190)} + \dfrac{10.6(36)^3(1728)}{48(29,000)(1190)} = 1.23" > \dfrac{36'(12)}{360} = 1.2"$ ∴ NG $\Rightarrow$ (ok I_{LB})

11.7^k

4@9'-0"

12.55

5.85

158'' 211''^{k}

Seth Moyer | Tech 2- Composite Deck/Beam | 5/5

$$\bar{y} = \frac{11.8(17.9/2) + 2.91((17.9+4-10.6/2)/50)}{11.8 + 2.91/50} = 13.05''$$

$$I_{LB} = 612 + 11.8(13.05-17.9/2)^2 + 2.91((17.9+4-10.6/2-13.05)^2/50 = 1213.2 \text{ in}^4$$

$$\Delta_{LL} = \frac{10.6(9')[3(36)^2-4(9)^2](1728)}{24(29,000)(1213.2)} + \frac{10.6(36)^3(1728)}{48(29,000)(1213.2)} = 1.20'' = \frac{36'(12)}{360} = 1.2'' \therefore OK$$

Check beam+girder self wt. allow: 12/9' + 40/(35/2) = 3.62 psf < 5 psf assumed ∴ OK

Check Δ_{TL}

$$w_{TL} = (45+67.3) 9' = 1.01 \text{ klf}$$

$$P_4 = 1.01(35/2) = 17.7^K$$

$$\Delta_{TL} = \frac{17.7(9')[3(36)^2-4(9)^2](1728)}{24(29,000)(1213.2)} + \frac{17.7(36)^3(1728)}{48(29,000)(1213.2)} = 2.01 > \frac{36'(12)}{240} = 1.8'' \therefore NG$$

$$I_{LB,req'd} = 2.01/1.8 \times 1213.2 = 1355 \text{ in}^4$$

Try W21×44 w/ $\Sigma Q_n = 260$ & Y2=3.5

$$a = 260/[0.85(108)(0)] = 0.949 < 1'' \therefore OK$$

260/17.1 = 15.2 => 16 studs × 2 = 32 studs/girder   16(21) = 273.6 = ΣQ_n ∴ a = 273.6/[0.85(108)(3)]
                                                                                = 0.99 < 1'' ∴ OK

Check Δ_{TL}

$$I_{LB} = 1560 \text{ in}^4$$

$$\Delta_{TL} = \frac{17.7(9')[3(36)^2-4(9)^2](1728)}{24(29,000)(1560)} + \frac{17.7(36)^3(1728)}{48(29,000)(1560)} = 1.56'' < 1.8'' \therefore OK$$

Check beam+girder sw allow: 12/9' + 44/(35/2) = 3.85 psf < 5 psf assumed ∴ OK

=> Use W21×44 w/ (32) 3/4" shear studs spaced evenly along length

Appendix C

Seth Moyer   Tech 2 - One-Way Slab   1/5

$f'_c = 4\,ksi$

$f_y = 60\,ksi$

Assume 18"×18" columns

h = 8.5"

$h_{slab} = (18 + 10/2)(12)/28 = 8.07"$

Check adjacent bay (one end continuous) $h_{slab} = (16 + 2/12)(12)/24 = 8.08"$ {ACI 318-11, Table 9.5(a)}

⇒ Try $h_{slab} = 8.5"$ d = 7.5"

DL = 8.5/12 (150) = 107 psf

Superimposed DL = 10 psf

LL = 100 psf

$W_u = 1.2(107 + 10) + 1.6(100) = 300\,psf$

Design 1' slab strip

$w_u = 300\,plf$ Assume $b_w = 18"$ (match column width)

$M_u^- = 0.3(18 + 10/12 - 18/12)^2/11 = 8.19\,ft\text{-}k$

$M_u^+ = 0.3(18 + 10/12 - 18/12)^2/16 = 5.63\,ft\text{-}k$

$A_s^- = M_u/4d = 8.19/(4)(7.5) = 0.273\,in^2/ft$ ⇒ Use #4 @ 8" OC $(0.2 \cdot 12/8 = 0.3\,in^2/ft)$

d = 8.5 - 0.75 - 0.5/2 = 7.5"

Check ϕM_n: Assume $\varepsilon_t \geq \varepsilon_y$

$a = 0.3(60,000)/[0.85(4,000)(12)] = 0.441$ $c = a/\beta = 0.441/0.85 = 0.519$

$\varepsilon_s = \dfrac{0.003(7.5 - 0.519)}{0.519} = 0.040 > 0.005$ ⇒ $\phi = 0.9$

$\phi M_n = 0.9(0.3)(60)(7.5 - 0.441/2)/12 = 9.83\,ft\text{-}k > M_u^- = 8.19\,ft\text{-}k$ ∴ OK

Seth Moyer   Tech 2 - One-way Slab   2/5

Check ϕM_n^+ Assume $\varepsilon_t \geq \varepsilon_y$

$A_s^+ = M_u/4d = 5.63/[4(0.5)] = 0.188 \ in^2/ft \Rightarrow$ Use #4 @ 12" OC (0.2 in^2/ft)

$a = 0.2(60)/[0.85(4)(12)] = 0.294 \quad c = 0.294/0.85 = 0.346$

$\varepsilon_t = \dfrac{0.003(7.5-0.346)}{0.346} = 0.062 > 0.005 \Rightarrow \phi = 0.9$

$\phi M_n = 0.9(0.2)(60)(7.5-0.294/2)/12 = 6.62 \ ft\text{-}k > M_u^+ = 5.63 \ ft\text{-}k \therefore OK$

Check Shrinkage + Temp Reinf:

$A_{req'd} = 0.0018 (12)(8.5) = 0.184 \ in^2/ft$

#4 @ 12" OC = $0.2 \ in^2/ft > 0.184 \ in^2/ft \therefore OK$

Check Crack Control

$s \leq 15 - 2.5(0.75) = 13\frac{1}{8}" \quad 13\frac{1}{8}" > 12" > 8" \therefore OK$

-Beam Design

$h_{min} = 36(12)/18.5 = 23.4"$

$DL = 8.5/12 (150) = 107 \ psf$

$SDL = 10 \ psf$

$LL = 100(0.25 + 15/\sqrt{36(35')}) = 67.3 \ psf$

$W_u = 1.2(107+10) + 1.6(67.3) = 248 \ psf$

$w_u = 248(35/2) = 4.34 \ klf \qquad$ member self-wt estimate

$M_{ext}^- = 4.34(36'-1.5')^2/16 = 322.9 \ ft\text{-}k \times 1.1 = 355.2 \ ft\text{-}k$

$M^+ = 4.34(36'-1.5')^2/14 = 369 \ ft\text{-}k \times 1.1 = 405.9 \ ft\text{-}k$

$M_{int}^- = 4.34(36'-1.5')^2/10 = 516.6 \ ft\text{-}k \times 1.1 = 568.3 \ ft\text{-}k$

$bd^2 \geq 20 M_u = 20(18.3)^2 = 20(568.3) \Rightarrow d = 25.1" \Rightarrow Try \ h = 28" \Rightarrow d = 28 - 2.5" = 25.5"$ $> 23.4" \therefore OK$

$18(28-8.5)/144 \times 150 = 366 \ plf$

$w_u = 4.34 + 1.2(0.366) = 4.78 \ klf$

Seth Moyer   Tech2 - One-way Slab   3/5

$M_{net}{}^- = 4.78 (36.15)^2/10 = 568.9$ ft-k

$A_s = M_u/4d = 568.9/4(25.5) = 5.58$ in^2 $=> (4)$ #9 $+ (2)$ #8 $= 5.58$ in^2

$d = 28 - 1.5 - 0.5 - \left(\frac{(4(1.0)(1.28) + 2(0.79)(1.0))}{4(1.0) + 2(0.79)} \right)/2 = 25.45"$

$a = 5.58(60)/[0.85(4)(18)] = 5.47$ $c = 5.47/0.85 = 6.44$

$\varepsilon_s = \frac{0.003(25.45 - 6.44)}{6.44} = 0.0089 > 0.005 \Rightarrow \phi = 0.9$

$\phi M_n = 0.9(5.58)(60)(25.45 - 5.47/2)/12 = 570.4$ ft-k $> M_u = 568.9$ ft-k $\therefore$ OK

From Table A.7: Max #bars as single layer $= 6 = 6 \therefore$ OK

From Table A.8: Min #bars $= 3 < 6 \therefore$ OK

$b_{min} = 2(1.5) + 2(0.5) + 4(1.128) + 2(1) + 5(1.128) = 16.2" < 18" \therefore$ OK

$A_{s,min} = 3\sqrt{4000}(18)(25.45)/60,000 = 1.45$ in$^2 < 200(18)(25.45)/60,000 = 1.53$ in$^2 < 5.58$ in$^2 \therefore$ OK

$-M^+ = 4.78(36 - 1.5)^2/14 = 406.4$ ft-k

$A_s = 406.4/4(25.5) = 3.98$ in^2 $=> (4)$ #9 $= 4.0$ in^2

$d = 28 - 1.5 - 0.5 - 1.128/2 = 25.44"$

$a = 4(60)/[0.85(4)(18)] = 3.92$ $c = 3.92/0.85 = 4.61$

$\varepsilon_s = \frac{0.003(25.44 - 4.61)}{4.61} = 0.014 > 0.005 \Rightarrow \phi = 0.9$

$\phi M_n = 0.9(4)(60)(25.44 - 3.92/2)/12 = 422.6$ ft-k $> M_u = 406.4$ ft-k $\therefore$ OK

From Table A.7 Max #bars $= 6 > 4 \therefore$ OK

From Table A.8: Min #bars $= 3 < 4 \therefore$ OK

$b_{min} = 2(1.5) + 2(0.5) + 4(1.28) + 3(1.128) = 11.9" < 18" \therefore$ OK

$A_{s,min} = 3\sqrt{4000}(18)(25.44)/60,000 = 1.45$ in$^2 < 200(18)(25.44)/60,000 = 1.53$ in$^2 < 4$ in$^2 \therefore$ OK

Seth Moyer　　　　　| Tech 2 – One-way Slab　　　　　　　　　　　4/5

$-M_{ext}^- = 4.78 (36-1.5)^2/16 = 355.6 \text{ ft-k}$

$A_s = 355.6/4(25.5) = 3.49 \text{ in}^2 \Rightarrow 7(2)\text{''}9 + (2)\text{''}8 = 3.58 \text{ in}^2$

$d = 28 - 1.5 - 0.5 - \left(\dfrac{(2.0)(1.128) + 2(0.79)(1)}{2(1) + 7(0.79)}\right)/2 = 25.46\text{''}$

$a = 3.58(60)/[(0.85(4)18] = 3.51 \qquad c = 3.51/0.85 = 4.13$

$\varepsilon_s = \dfrac{0.003(25.46 - 4.13)}{4.13} = 0.015 > 0.005 \Rightarrow \phi = 0.9$

$\phi M_n = 0.9(3.58)(60)(25.46 - 3.51/2)/12 = 381.9 \text{ ft} > M_u = 355.6 \text{ ft-k} \therefore OK$

From Table A.7 Max # bars = 6 > 4 ∴ OK

From Table A.8 Min # bars = 3 < 4 ∴ OK

$b_{min} = 2(0.5) + 2(0.5) + 2(1.128) + 2(1) + 3(1.128) = 11.6\text{''} < 18\text{''} \therefore OK$

$A_{smin} = 3\sqrt{4000}(18)(25.46)/60,000 = 1.45 \text{ in}^2 < 200(18)(25.46)/60,000 = 1.53 \text{ in}^2 < 3.58 \text{ in}^2 \therefore OK$

– Shear Reinf:

$w_u = 4.78 \text{ klf}$

$V_{u,int} = 1.15 (4.78)(36-1.5)/2 = 94.8^K$

$V_{u,ext} = 4.78(36-1.5)/2 = 82.5^K$

$V_u @ d = 94.8 - (25.45/12)(4.78) = 84.7^K$

$V_c = 2\sqrt{4000}(18)(25.45)/1000 = 57.9^K$

$\phi V_c = 0.5 (0.75)(57.9) = 21.7^K \Rightarrow 21.7/4.78 = 4.54' \quad 4.5 - 2'7'' = 1'-11'' \text{ left of midspan}$

No shear reinf req'd from 1'-11'' left of midspan to 4'-6'' right of midspan

$V_s = 84.7/0.75 - 57.9 = 55.0^K \qquad 55.0^K < 4\sqrt{4000}(18)(25.45)/1000 = 115.9^K \checkmark$

$S_{max} \overset{min}{=} \left| \begin{array}{l} 25.45/2 \\ 24 \end{array} \right. \quad S = 12.7'' \Rightarrow 12''$

Seth Moyer   Tech2 - One way Slab   5/5

$$A_{v,min} = \begin{vmatrix} 0.75\sqrt{4000}(18)(12)/60,000 = 0.171 \text{ in}^2 \\ 50(18)(12)/60,000 = 0.18 \text{ in}^2 \end{vmatrix} <= \text{(2) legs of } ^{\#}3 = 0.22 \text{ in}^2 > 0.18 \text{ in}^2 \therefore OK$$

$$S = 0.22(60)(25.95)/55.0 = 6.11 = 76''$$

Use (1) $^{\#}3 \times \square$ @ 6" OC at member end, starting 2" from face of support.

Check 12" sp: $\phi V_n = 0.75(5.29 + 0.22(60)(25.95)/12) = 64.4^k$

$$\ell_V = \frac{(99.5 - 64.4)(12)}{4.78} = 76.3''$$

From face of int. support: $1@2''; 13@6''; 9@12''$

- Shear Reinf from ext support

$$V_{u@d} = 82.5 - (25.95/12)(4.78) = 72.4^k$$

$$V_c = 58.0^k$$

$$\phi V_c = 0.5(0.75)(58.0) = 21.8^k \quad \therefore 21.8/4.78 = 4.56'$$

$$V_s = 72.4/0.75 - 58 = 38.5^k \quad 38.5^k < 115.9^k \checkmark$$

$$S_{max} = 12''$$

$$S = 0.22(60)(25.46)/38.5 = 8.73'' => 8''$$

Use (1) $^{\#}3 \times \square$ @ 8" OC at member end, starting 2" from face of support.

Check 12" sp: $\phi V_n = 0.75(58 + 0.22(60)(25.46)/12) = 64.5^k$

$$\ell_V = \frac{(82.5 - 64.5)(12)}{4.78} = 45.2''$$

From face of ext support: $1@2''; 6@8''; 9@12''$

Appendix D

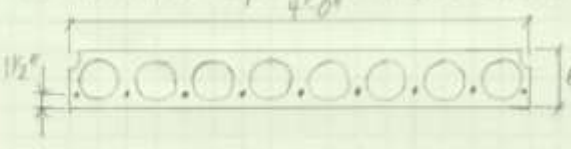Seth Moyer   Tech 2 - Hollow-core Planks   1/2

→ All plank design data taken from PCI Design Handbook, 6th Ed

Superimposed DL = 10 psf ⎫
                        ⎬ Total Superimposed Service Load = 110 psf
LL = 100 psf   ⎭

For a 4'-0" ×6" NWC hollow-core plank w/ 19' span, safe superimposed service load
= 114 psf > 110 psf ∴ OK

For allowable deflections, LL_{allow} = 114 - 10 = 104 psf > 100 psf ∴ OK

→ Use 4HC6 hollow-core plank (4'-0" ×6") w/ 66-S strand designation code, untopped

4'-0"

A = 187 in^2 f'$_c$ = 5000 psi
I = 763 in^4 f$_{pu}$ = 270,000 psi
wt = 195 plf
DL = 49 psf

Design Girder (36' span)

DL: 49 psf (planks) ⎫
    10 (superimposed) ⎬ Total DL = 63 psf
    4 (girder s.w. allow) ⎭

LL = 100 (.25 + 15/√(36×35)) = 67.3 psf

W_u = 1.2(63) + 1.6(67.3) = 183.3 psf

w_u = (35/2)(183.3) = 3.21 klf

M_u = 3.21(36^2)/8 = 520 ft-k

Find $I_{req'd}$ for Δ_{TL}

w_{TL} = (63 + 67.3)(35/2) = 2.28 klf

$\Delta_{TL} = \frac{5(0.28)(36)^4(1728)}{384(29,000)I_{req'd}} \leq \frac{36(12)}{240} = 1.8"$ → $I_{req'd} \geq 1651 in^4$

→ Try W24×68 (I_x = 1830 in^4)

ϕM_p = 664 ft-k > 520 ft-k ∴ OK   (assume beam is fully braced by precast planks)

Check Δ_{LL}

$\Delta_{LL} = \frac{5(67.3)(35/2)/1000(36)^4(1728)}{384(29,000)(1830)}$ 0.84" < $\frac{36(12)}{360}$ = 1.2" ∴ OK

Seth Moyer   Tech 2 - Hollow-core Planks   2/2

Check girder self weight allow.:

68 plf /(35%) = 3.89 psf < 4 psf estimated ∴ OK

⇒ Use W24x68 girder

Appendix E

Seth Moyer   Tech 2 - Floor System Weights   1/1

Composite Deck Slab on Open Web Steel Joists (Existing System: Non-composite Design)

- 2½" thk conc. slab on 20 ga. 1½" composite deck: 39 psf

- Joists: 16K3 => 6.3 plf/3' = 2.1 psf

- Girders: W24x76 => 76 plf/(35½) = 4.34 psf

=> Total Weight: 39 + 2.1 + 4.34 = $\boxed{45.4 \text{ psf}}$

Composite Deck Slab on Composite Beams/Girders ((Composite Design)

- 2½" thk LW conc. slab on 20 ga. 1½" composite deck: 30 psf

- Beams: W10x12 => 12 plf/9' = 1.33 psf

- Girders: W21x44 => 44 plf/(35½) = 2.51 psf

=> Total Weight: 30 + 1.33 + 2.51 = $\boxed{33.8 \text{ psf}}$

One-way Slab

- Slab: (8.5/12)(150) = 106.25 psf

- Beams: (28-8.5)(8)(144)(150) = 365.625 plf/(35½) = 20.89 psf

=> Total Weight: 106.25 + 20.89 = $\boxed{127 \text{ psf}}$

Hollow-core Planks on Steel Girders

- Planks (6" thk.): 49 psf

- Girders: W24x68: 68 plf/(35½) = 3.89 psf

=> Total Weight: 49 + 3.89 = $\boxed{52.9 \text{ psf}}$

Appendix F

Seth Moyer　　　　　　　Tech 2 - Floor System Costs　　　　　　　1/3

=> All cost data taken from RSMeans Building Construction Cost Data 2012

=> Bradford County, PA Location Weighted Factor of 0.947 will be applied to all cost values

Composite Deck Slab on Open Web Steel Joists (Existing System: Non-composite Design)

- Deck: $2.88/sf

- Concrete (NW): Volume = 678 sf (2.5/12) + 678 (1.5/12)(0.35) = 171 cf / 27 = 6.33 cy

　　Structural Concrete: 6.33 (113/cy) = $715.29 / 678 sf = $1.055/sf

　　Placement: 6.33 (25/cy) = $158.25 / 678 sf = $0.233 + 0.75 = $0.983/sf　(finishing)

- Joists: $10.15/ft / 3' = $3.38/sf

- Girders: $123/ft / (35½) = $7.03/sf

=> Total Cost: 2.88 + 1.055 + 0.983 + 3.38 + 7.03 = $15.33/sf × 0.947 = $14.52/sf

Composite Deck Slab on Composite Beams / Girders (Composite Design)

- Deck $2.88/sf　　　　　(LW conc. premium or NW)

- Concrete (LW): 6.33 (113/cy)(1.35) = $965.64 / 678 sf = $1.42/sf

　　Placement: 6.33 (25/cy)(1.2) = $189.90 / 678 sf = $0.28/sf + 0.75 = $1.03/sf
　　　(LW placement premium)　　(finishing)

- Beams: $28.50/ft / 9' = $3.17/sf

　　Shear Studs: ($14/stud)(12) = $168/(18 + 10/12)/9' = $0.99/sf

- Girders: $74.50/ft / (35½) = $4.26/sf

　　Shear Studs: ($14/stud)(32) = $448/36'/(35½) = $0.71/sf

=> Total Cost: 2.88 + 1.42 + 1.03 + 3.17 + 0.99 + 4.26 + 0.71 = $14.46/sf × 0.947 = $13.69/sf

☆ Assume cost of (1) stud, installed, = 10 lbs of steel weight.

Seth Moyer　　　　　　　Tech 2 - Floor System Costs　　　　　　　　　　　2/3

One-way Slab

- Concrete

　Slab: $\$113/cy /27 = \$4.185/cf (8.5/12) = \underline{\$2.96/sf}$

　Beam: $(28-8.5)(18)/144 = 2.4375 \, sf (36'-1.5') = 84.66 \, cf/27 = 3.02 \, cy \Rightarrow$

　　　$\Rightarrow 3.02 (\$113/cy) = \$341.26/(35\,1/2)/36' = \underline{\$0.54/sf}$

- Formwork

　Slab: $\underline{\$6.62/sf}$

　Beam: $[(28-8.5)/2 + 18]/12 \, (36'-1.5') = 163.875 \, sfca \, (\$8.56/sfca) = \$1402.77/(35\,1/2)/36' = \underline{\$2.23/sf}$

- Placement (pumped):　　　　　　　　　　　　　　　　　　(finishing)

　Slab: $\$28.50/cy /27 = \$1.056/cf (8.5/12) = \$0.75/sf + 0.75 = \underline{\$1.50/sf}$

　　　　　　　　　　　　　　　　　　　　　　　(finishing)

　Beam: $3.02 \, cy (\$75.50/cy) = \$228.01/(35\,1/2)/36' = \$0.36/sf + 0.75 = \underline{\$1.11/sf}$

- Reinforcement

　Slab: $A_s^- \Rightarrow \#4 @ 8": 0.668 \, plf (12/8) = 1.002 \, psf$

　　　$A_s^+ \Rightarrow \#4 @ 12" = 0.668 \, psf$

　　　Shrinkage/Temp Reinf $\Rightarrow \#4 @ 12" = 0.668 \, psf$

　　　$\Rightarrow 1.002 + 0.668(2) = 2.338 \, psf \, (\$2025/ton)/(2000 \, lbs/ton) = \underline{\$2.37/sf}$

　Beam: $M_{int}^- \Rightarrow (4) \#9 + (2)\#8 = 4(3.4 \, lb/ft) + 2(2.67 \, lb/ft) = 18.94 \, plf (36/2) = 340.9 \, lb/(35\,1/2)/36' = 0.54 \, psf$

　　　$M^+ \Rightarrow (4) \#9 = 4(3.4 \, lb/ft) = 13.6 \, plf (36') = 489.6 \, lb/(35\,1/2)/36' = 0.777 \, psf$

　　　$M_{ext}^- \Rightarrow (2)\#9 + (2)\#8 = 2(3.4 \, lb/ft) + 2(2.67 \, lb/ft) = 12.14 \, plf (36/2) = 218.5 \, lb/(35\,1/2)/36' = 0.347 \, psf$

　　　Transverse: $\#3 \Rightarrow 6.67'/stirrup \times 39 = 260' (0.376 \, lb/ft) = 97.76 \, lb/(35\,1/2)/36' = 0.155 \, psf$

　　　$\Rightarrow 0.54 + 0.777 + 0.347 = 1.665 \, psf \, (\$2000/ton)/(2000 \, lbs/ton) = \underline{\$1.67/sf}$

　　　$=$ (transverse): $0.155 \, psf \, (\$2650/ton)/(2000 \, lbs/ton) = \underline{\$0.21/sf}$

$\Rightarrow$ Total Cost: $2.96 + 0.54 + 6.62 + 2.23 + 1.50 + 1.11 + 2.37 + 1.67 + 0.21 = \$19.21/sf \times 0.947 = \boxed{\$18.19/sf}$

Seth Moyer   Tech 2- Floor System Costs   3/3

Hollow-core Planks on Steel Girders

  - 6" Precast Planks: $10.05/sf

  - Girders: W24x68: $(111/14/(35/2)) = $6.34/sf

⇒ Total Cost: 10.05 + 6.34 = $16.39/sf × 0.947 = $15.52/sf