Technical Report 2

Rendering provided by DCS Design

Kingstowne Section 36A
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Kingstowne, VA 22315

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Structural Option

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October 12, 2012

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Executive Summary

Kingstowne Section 36A (KT36A) is a 200,000 SF mixed use building currently being constructed in Fairfax County Virginia. When completed, the lower half of the building will serve as a parking garage serving the office tenants of the upper half of the building. The parking garage levels utilize flat slab concrete construction while the office levels use a composite steel construction. A more thorough description of the existing structure can be found in the first half of this report.

The purpose of Technical Report 2 is to analyze the existing floor systems of KT36A and perform a pro-con study on three alternative floor systems that are believed to be possible candidates for consideration in the building. The alternative floor systems must be schematically designed so that an accurate comparison considering, strength, deflections, structure depth, system weight, lateral system impacts, foundation impacts, and costs can be made. A summary of the system comparison can be found in Figure 17 on page 25 of this report. The floor systems considered for comparison were:

- Two-way Flat Slab With Drop Panels (Garage Existing)
- Precast, Pretensioned Double Tees on Precast, Pretensioned Support Girders
- Composite Steel Deck on Composite Steel Beams and Girders (Office Existing)
- One-way Concrete Pan Joists With Wide Beams
- Steel Form Deck on Open-web Steel Joists and Wide Flange Girders

After conducting the analyses of the systems, it was clearer why a split structure was chosen for the building. Concrete was a good choice for the parking levels due to the minimal structural depth and the durability of concrete when exposed to a wet and salty environment. The composite steel construction in the office levels allowed large open bays for maximum flexibility of the office space. Of the alternative systems considered, the open web steel joist system seems to be the most probable alternative to the existing office level framing. Changing to this system would have virtually no impact on the existing systems since the weight is about the same and the braced and moment frames can still be used for the lateral system at the office levels. The precast double tee system is a possible alternative for the garage levels. Use of this system would allow for two lines of columns to be removed at grid lines B and E. However, lateral system impact from this alternative can be quite severe considering the designed lateral system consists of cast in place shear walls which typically frame into the cast in place columns. As for the one way concrete pan joist system, it appears that unless if the bay sizes can be made a little smaller, this system is simply too heavy and bulky to efficient construct the building with.

BUILDING INTRODUCTION

Kingstowne Section 36A (KT36A) is a 200,000 ft^2, 8 story office building to be located in Fairfax County Virginia. It will contain 4 levels of concrete structure parking garage and 4 levels of composite steel construction office space. Floor space has also been allocated for about 5,000 square feet of retail area on the ground floor (Parking Level 1). KT36A will be 86'-11" in height when measured from the average grade. The reason the building height is measured from average grade is because there is a significant grade elevation change from the south side of the building to the north side, on the order of 26'-8" (See Figure 1). This poses unique challenges in the structural design of the building since the geotechnical report states the soil placing a load of 60psf/ft in depth below grade surface on the structure. This means that there is more than 1600 psf of soil load on the foundation walls at the lowest slab levels. This load alone had enough impact on the building that six 12" thick shear walls had to be constructed at parking level 1 to transfer the loads safely.

When completed, KT36A will be part of a master planned development for retail and office space owned by the Halle Companies. Being a part of a master planned development, the building was designed to match the appearance of the surrounding buildings. This appearance can be characterized by a rectilinear footprint, pink velour brick, aluminum storefront with glass of blue/black appearance, and precast concrete bands around the circumference of the building.

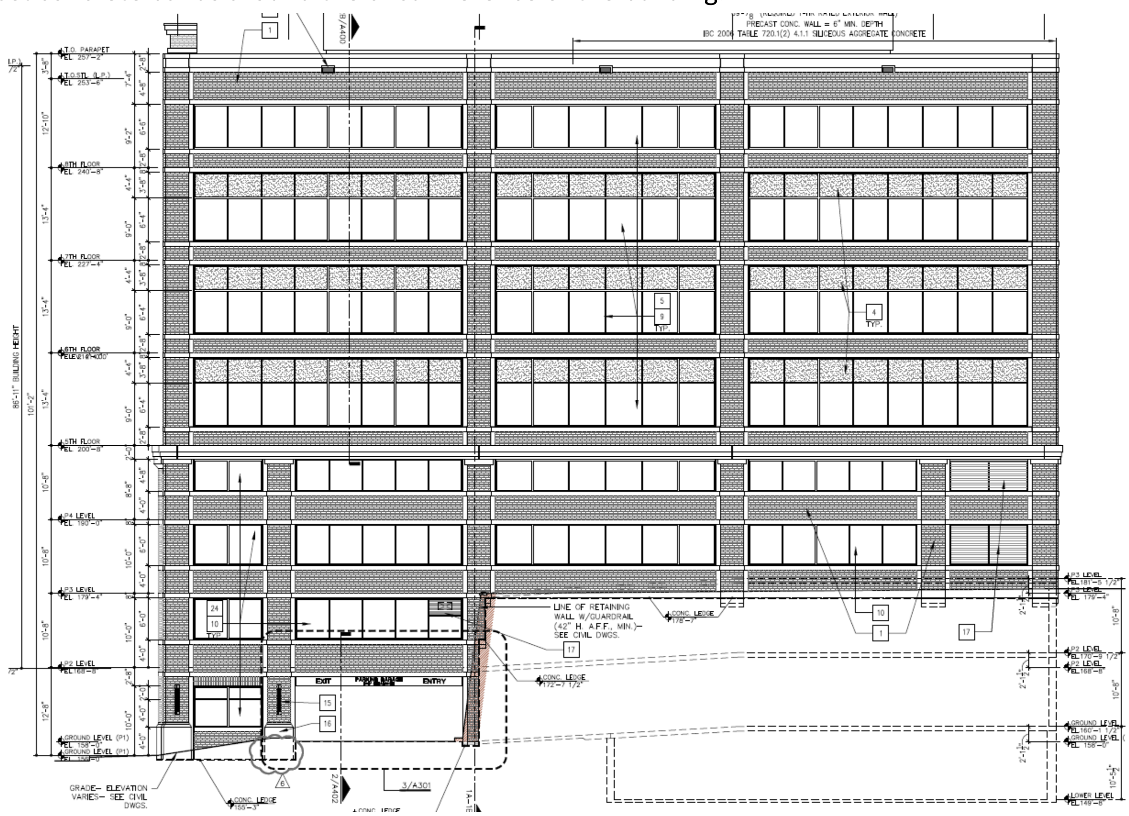


Figure 1: Elevation Looking East Showing Grade Differences (Source: DCS Design Drawing A-301)

STRUCTURAL OVERVIEW

Kingstowne Section 36A consists of two different primary structural systems; cast-in-place concrete for the lowest four floors of the building and a composite steel system for the remaining four floors. The concrete floors are used for the parking garage and retail space while the steel system is used at the office occupancy levels. Lateral forces in the concrete levels are resisted with 12” thick concrete shear walls of varying height. When the building transitions to steel construction, lateral forces are transferred to the concrete columns and shear walls through concentrically braced frames, eccentrically braced frames, and rigid moment frames. Per sheet S-001, components such as steel stairs and curtain wall/window systems were not included in the scope for the structural design of this building.

FOUNDATIONS

In their report submitted August of 2009, Burgess & Niple, Inc. (B&N) advised that shallow foundations not be used on this project due to settlement concerns based on subsurface conditions. They performed five new soil test borings, ranging from 45 to 100 feet in depth below the grade surface. In addition, they reviewed 14 borings from previous investigations, ranging in depth from 10 to 55 feet below grade surface.

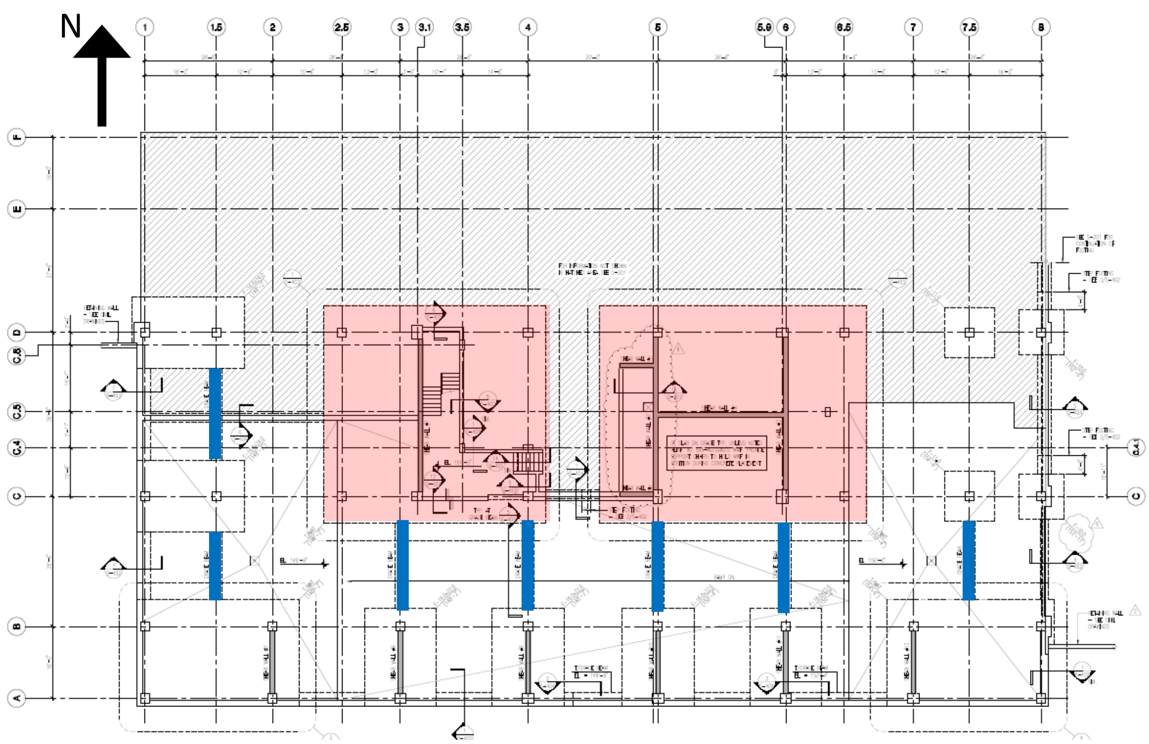


Figure 2: Foundation Plan (Level P0) Showing 48” Thick Mat Foundations Shaded in Red (Source: Cagley & Assoc. Drawing S-200)

Each of the borings found lean clay and fat clay fills with varying amounts of sand, residual soils consisting of lean to fat clay, and clayey to silty sands. Based on the fill materials being encountered between 4 and 48 feet below grade, B&N offered two foundation options. An intermediate foundation system consisting of spread and strip footings bearing on rammed aggregate piers (Geopiers) was chosen for KT36A over the alternate option of a deep system consisting of spread and strip footings bearing on caissons. Geopier diameters typically range from 24 to 36 inches and are compacted using a special high-energy impact hammer with a 45-degree beveled tamper. Per B&N report, footings supported by Geopier elements can be designed using a maximum bearing pressure of 7,000 psf.

Using the information provided by B&N, Cagley & Associates designed spread footings ranging from 27" to 44" in depth to support the columns of KT36A. 48" thick mat foundations bearing on Geopiers are located at the central core of the building to transfer forces in the main shear walls to the soil (See Figure 2). Grade beams (Blue lines in Figure 2) of 30" depth are used throughout level P0 to also transfer forces from the shear walls to the column footings. Foundation walls are supported by continuous wall footings designed for an allowable bearing pressure of 2,500 psf. All foundations are to bear a minimum of 30" below grade unless stated otherwise.

GARAGE LEVELS

FLOOR SYSTEM

As previously mentioned, KT36A utilizes cast-in-place concrete for the support structure in the garage. With the exception of the 5" thick slab on grade, this system consists of 8" thick two-way, flat slab construction with drop panels that project 8" below the bottom of structural slab. The drop panels are continuous between grid lines C and D to help the slab span the increased distance of 36'-6" in this bay, otherwise, they are typically 10'-0" x 10'-0" in size. Due to the need for vehicles to circulate vertically throughout the parking garage levels, the floor is sloped on 3 sides of the central core to achieve this.

Since a two-way, flat plate concrete floor system is subjected to both positive and negative moments, reinforcing steel is required in the top and bottom of the slab. The typical bottom mat of reinforcement in KT36A consists of #4 bars spaced at 12" on center in each direction of the slab. Additional bottom reinforcement in certain middle strips and continuous drop panels is also noted on the drawings. Top reinforcement is comprised of both #5 and #6 bars, both oriented in the same fashion as the bottom mat, with the #6 bars typically being used in the column strips to resist the larger negative moments present there (see Figure 3 for a typical bay layout). A typical bay size for the concrete levels is 28'-6" x 29'-0".

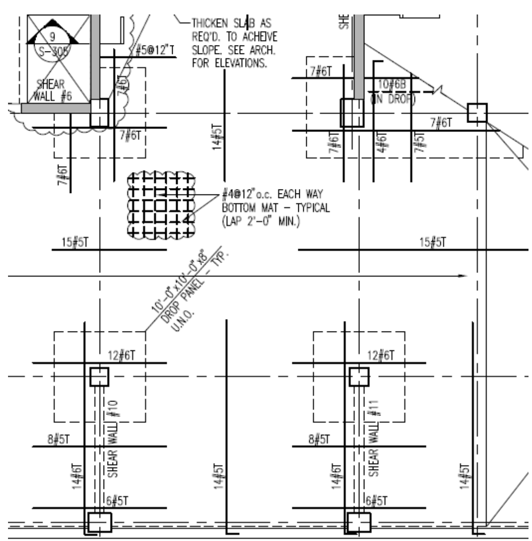


Figure 3: Partial Plan Level P1 (Source: Cagley & Assoc. Drawing S-201)

FRAMING SYSTEM

Supporting the floor slabs are cast-in-place concrete columns constructed of 5000 psi concrete. The most common column size is 24" x 24" reinforced with a varying number of #8 bars and either #3 or #4 ties. Columns of this size primarily account for the gravity resisting system of KT36A. The largest columns used are 36" x 30" reinforced with a varying number of #11 bars and #4 stirrups. The larger columns are located at the ends of the large shear walls in the central core of the building. A small number of concrete beams are also present in the project, typically at areas of the perimeter where additional façade support was needed and at large protrusions in the floor slab.

LATERAL SYSTEM

Cast-in-place concrete shear walls resist the lateral forces present in the parking garage levels of KT36A. All of the twelve walls present in the building are 12" thick and cast using 5000 psi concrete. Six of the shear walls (#1 - #6, see Red lines in Figure 4) extend 4-5 stories from the 48" thick mat foundations to office level 1 which is also the 5th elevated floor of the building. Three of the six walls are oriented to resist lateral forces in the N-S direction while the other three walls are oriented in the E-W direction. The remaining six walls (#7 - #12, Green lines in Figure 4) are only one story tall and are oriented to best resist the unique lateral soil load placed on KT36A. This load condition is further detailed in the lateral loads section of this report and will be further analyzed in Technical Report 3.

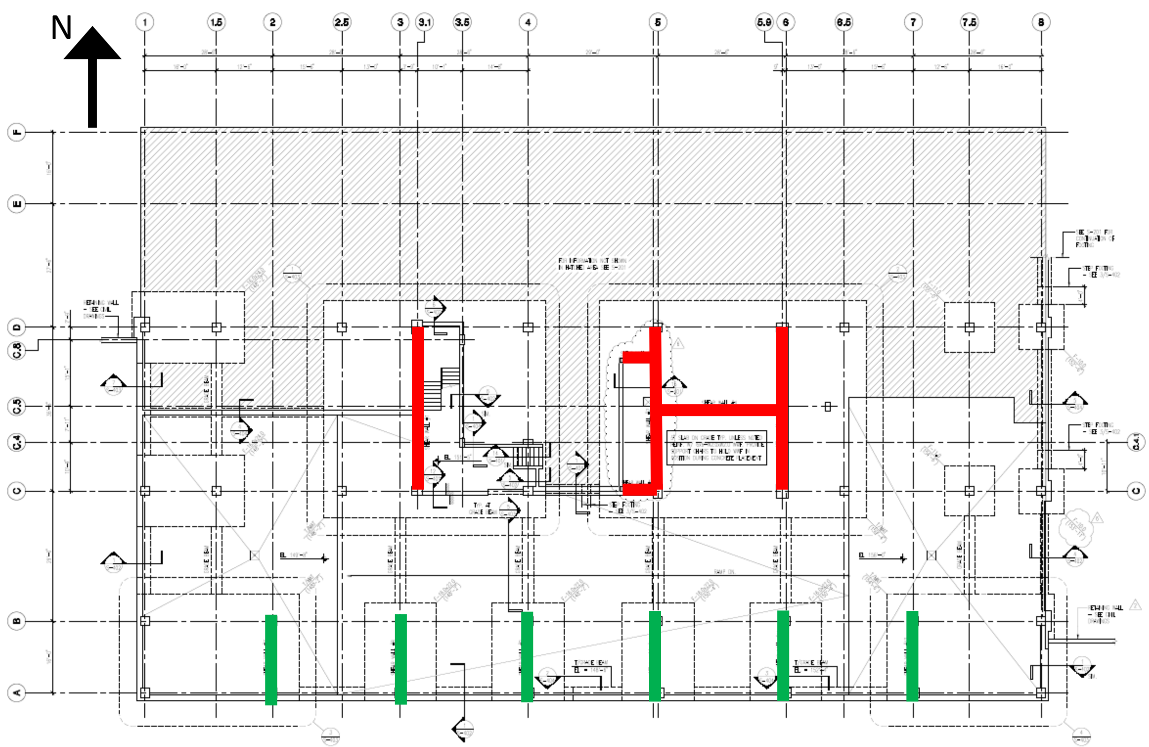


Figure 4: Foundation Plan (Level P0) Showing Shear Walls (Source: Cagley & Assoc. Drawing S-200)

Office Levels

FLOOR SYSTEM

Office level 1 is constructed of the same cast-in-place style of construction as the garage floors below it with the exception of the top of slab elevation being uniform throughout the floor. The remaining floors are constructed using a composite steel system. This system is comprised of 3 ¼" thick lightweight concrete on 2" x 18 gage galvanized composite steel decking. The 3000 psi lightweight concrete (115 pcf) coupled with the decking yields a total slab thickness of 5 ¼". Reinforcement for the slab is provided by 6x6-W2.1xW2.1 welded wire fabric.

According to sheet S-001, all decking should meet the three span continuous condition. The decking typically spans 9'-6" perpendicular to cambered beams of varying size. Shear studs of ¾" diameter placed along the length of the beams make this a composite system capable of more efficiently carrying the loads when compared to a non-composite system. The studs must be minimum length of 3 ½" but no longer than 4 ½" to meet designer and code requirements.

FRAMING SYSTEM

The composite floor system mentioned above is supported by structural steel framing comprised of primarily wide flange shapes. W21's and W18's account for most of the beams while the columns range in size from W12x40 to W14x109. A majority of the beams in KT36A are cambered between ¾" and 1 ¼", a function of the span and load demand on the beams. With the exception of four W30x99 sections cambered 1", most of the girders fall within the same size range as the beams. The four W30x99 girders each span 44'-0" which warrants the use of the camber to satisfy the total deflection criteria. The columns are all spliced just above the 7th floor (office level 3) where they are reduced in size to more economically carry the lighter axial loads. See Figure 5 below for a typical office floor level layout.

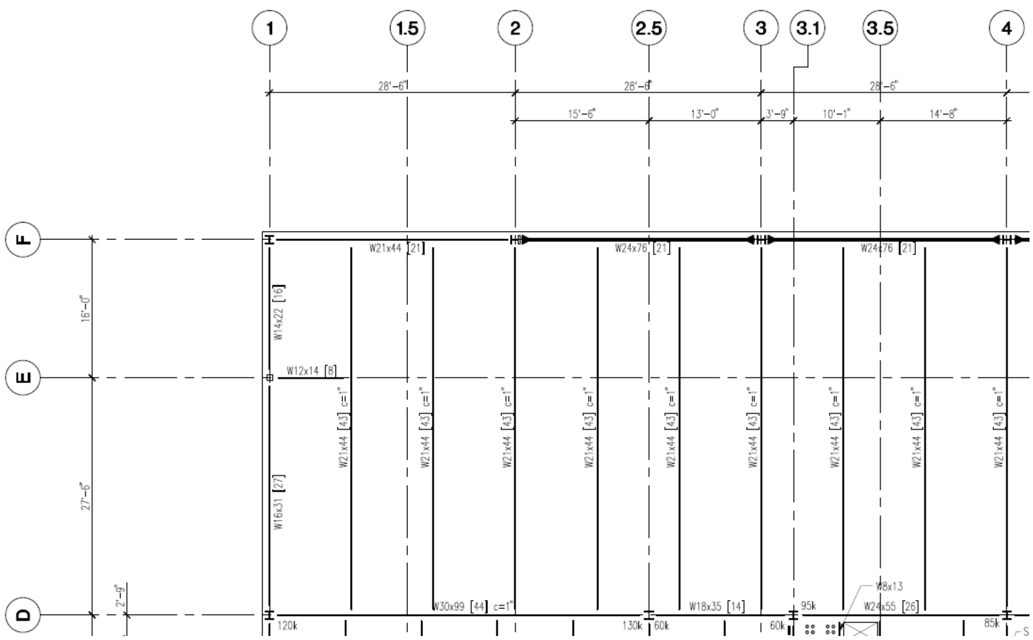


Figure 5: Typical Composite Slab Partial Plan (Level OL3) (Source: Cagley & Assoc. Drawing S-207)

LATERAL SYSTEM

Lateral forces at the office levels are transferred to the concrete shear walls through three different frame systems. Concentrically braced (Green Line) and eccentrically braced frames (Purple Lines) work in the north – south direction while ordinary steel moment frames (Orange Lines) resist the loads in the east – west direction. See Figure 6 for their location and orientation within the building. The eccentrically braced frames were necessary to maintain enough clearance for a corridor in that area of the building. Diagonal bracing for the frames consists of either HSS10x10 or HSS9x9 of varying thickness. Moment frames were most likely chosen for the east – west direction so as not to obstruct the occupants view to the exterior and lower lateral loads acting on the building in this direction.

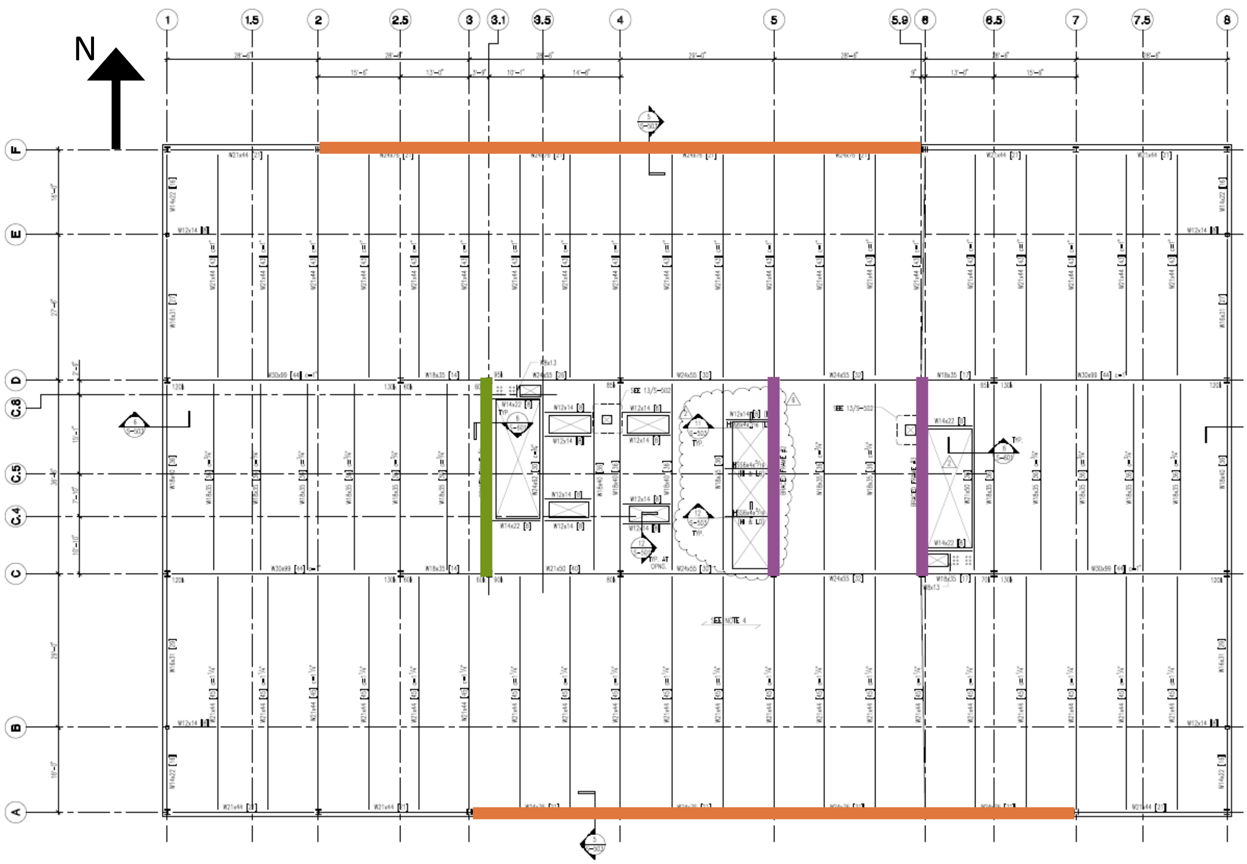
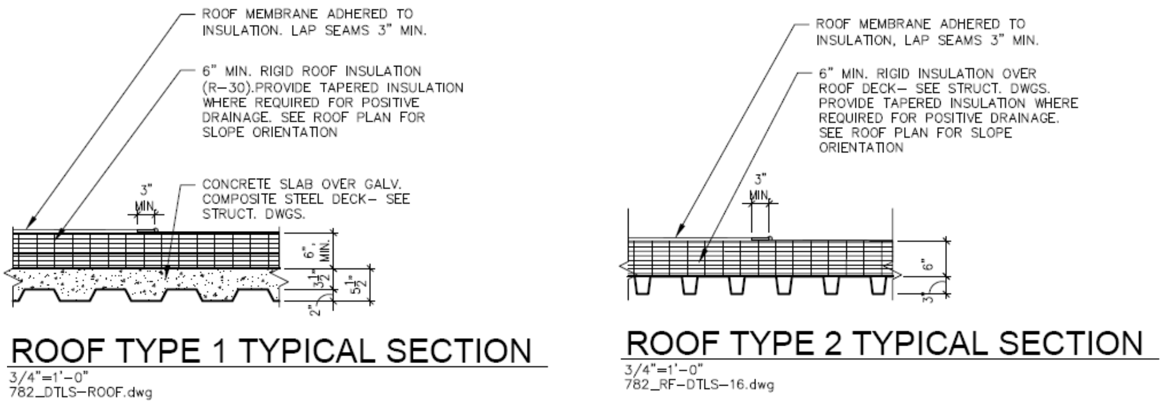


Figure 6: Typical Composite Slab Plan (Level OL3) (Source: Cagley & Assoc. Drawing S-207)

Roof System

The roofing system consists of a white EPDM membrane fully adhered over 6” minimum of R-30 continuous rigid roof insulation. The seams of the membrane must be lapped a minimum of 3” to ensure a watertight seal. Where mechanical equipment is located (see Figure 9), the roofing materials are supported by 2”x 18GA galvanized composite steel deck with a 3.25” thick light-weight concrete topping. The load carrying capacity that this type offers is required to support the four 17,000lb roof top mechanical units needed to condition the air for the building occupants. In all other areas of the roof, the system is supported by 3”x 20GA type N roof deck. Each of the roof types are supported by steel W-shapes that are sloped to achieve proper drainage.



Figures 7 and 8: Typical Roofing Details (Source: DCS Design Drawing A-410)

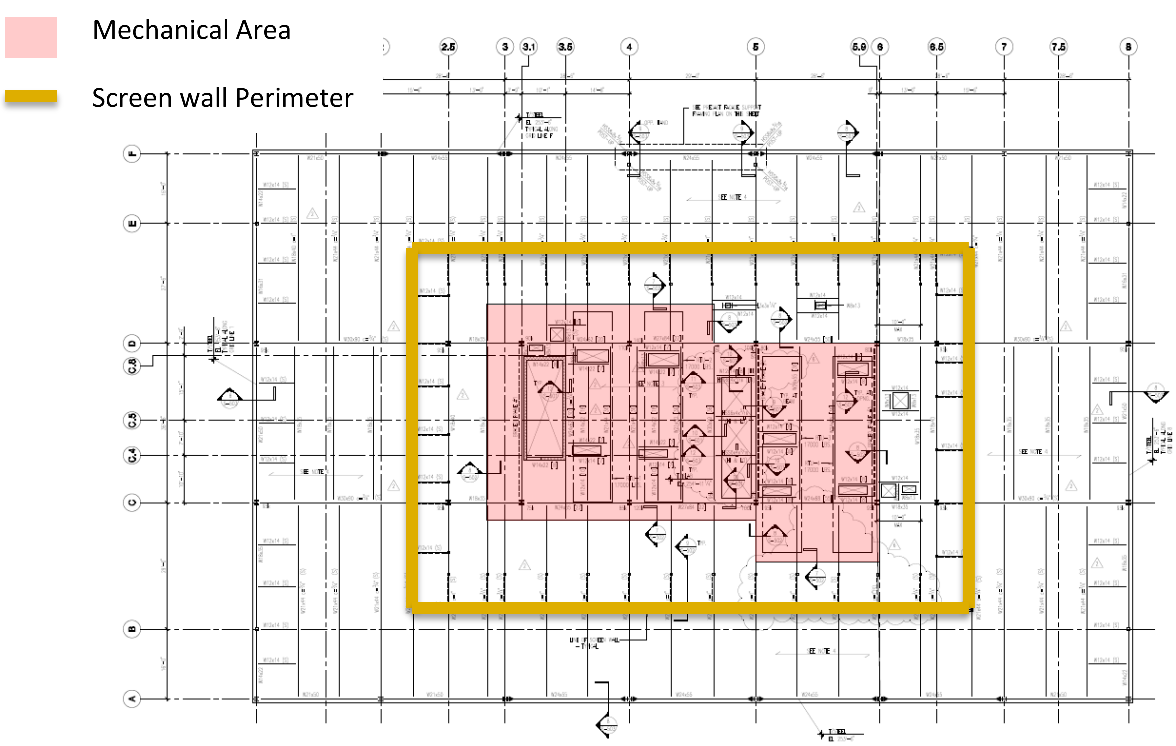


Figure 9: Structural Roof Plan (Source: Cagley & Assoc. Drawing S-209)

Design Codes

Per sheet S-001, Kingstowne Section 36A was designed in accordance with the following codes:

- 2006 International Building Code
- 2006 Virginia Uniform Statewide Building Code (Supplement to 2006 IBC)
- Minimum Design Loads for Buildings and Other Structures (ASCE 7-05)
- Building Code Requirements for Structural Concrete (ACI 318-08)
- ACI Manual of Concrete Practice, Parts 1 through 5
- Manual of Standard Practice (Concrete Reinforcing Steel Institute)
- Building Code Requirements for Masonry Structures (ACI 530, ASCE 5, TMS 402)
- Specifications for Masonry Structures (ACI 530.1, ASCE 6, TMS 602)
- AISC Manual of Steel Construction, 13th Edition
- Detailing for Steel Construction (AISC)
- Structural Welding Code ANSI/AWS D1.1 (American Welding Society)
- Design Manual for Floor Decks and Roof Decks (Steel Deck Institute)

Codes / Manuals referenced for the purposes of this report:

- 2009 International Building Code
- ASCE 7-10
- ACI 318-11
- AISC Manual of Steel Construction, 14th Edition
- 2008 Vulcraft Decking Manual

MATERIAL PROPERTIES

Minimum Concrete Compressive Strength	
Location	**28 Day f'c (psi)**
Footings	3000
Grade Beams	3000
Foundation Walls	5000
Shear Walls	5000
Columns	5000
Slabs-on-Grade	3500
Reinforced Slabs	5000
Reinforced Beams	5000
Elevated Parking Floors	5000
Light Weight on Steel Deck	3000

Max. Concrete W/C Ratios	
f'c @ 28 Days (psi)	**W/C (Max)**
$f'c \leq 3500$	0.55
$3500 < f'c < 5000$	0.50
$5000 \leq f'c$	0.45
Elevated Parking	0.40

Reinforcement:

- Deformed Reinforcing Bars — ASTM A615, Grade 60
- Welded Wire Reinforcement — ASTM A185
- Slab Shear Reinforcement — Decon Studrails or Equal

Masonry:

- Concrete Masonry Units — Light weight, Hollow ASTM C90, Min. f'c = 1900 psi
- Mortar — ASTM C270 – Type M (Below Grade)
  Type S (Above Grade)
- Grout — ASTM C476 – Min. f'c @ 28 days = 2000 psi
- Horizontal Joint Reinforcement — ASTM A951 – 9 Gage Truss-type Galvanized

Structural Steel:

- Wide Flange Shapes and Tees — ASTM A992, Grade 50
- Square/ Rectangular HSS — ASTM A500, Grade B, F_y = 46 ksi
- Base Plates and Rigid Frame Continuity Plates — ASTM A572, Grade 50
- All Other Structural Plates and Shapes — ASTM A36, F_y = 36 ksi
- Grout — ASTM C1107, Non-shrink, Non-metallic
  f'c = 5000 psi

GRAVITY LOADS

DEAD LOADS

Superimposed Dead Loads	
Plan Area	**Load (psf)**
Office Floors	15
Roof	30
Parking Garage Floors	5

Dead loads resulting from system self-weights were calculated and estimated based on the drawings provided. The loads and the assumptions used in their determination are detailed in Appendix A.

LIVE LOADS

Live Loads			
Plan Area	**Design Load (psf)**	**IBC Load (psf)**	**Notes**
Lobbies	100	100	
Mechanical	150	N/A	Non-reducible
Offices	80	80	Corridors used, otherwise 50 psf
Office Partitions	20	15	Minimum per section 1607.5
Parking Garage	50	40	
Retail	100	100	Located on first floor
Stairs and Exitways	100	100	Non-reducible
Storage (Light)	125	125	Non-reducible
Roof Load	30	20	

Floor System Analysis

The typical structural bay for Kingstowne 36A depends on what level of the building is being considered. At the garage levels, the bay size is commonly 27'-6" to 29'-0" by 28'-6" to 29'-0". Smaller bays and larger bays do exist, but this is the most common range. See Figure 10 for the garage bay used for the purposes of this report. As for the office levels, grid lines B and E terminate when the concrete structure is no longer used resulting in a typical bay size of 28'-6" to 29'-0" by 43'-6" to 45'-0". Again, smaller and larger bays do exist, but this is the most typical range. See Figure 11 for the office bay used for the purposes of this report.

Please note that only gravity loads were used for the purposes of this report. In order to properly estimate and understand the proposed floor systems, a lateral analysis for each system would also have to be completed. This, however, was not part of the scope of this report.

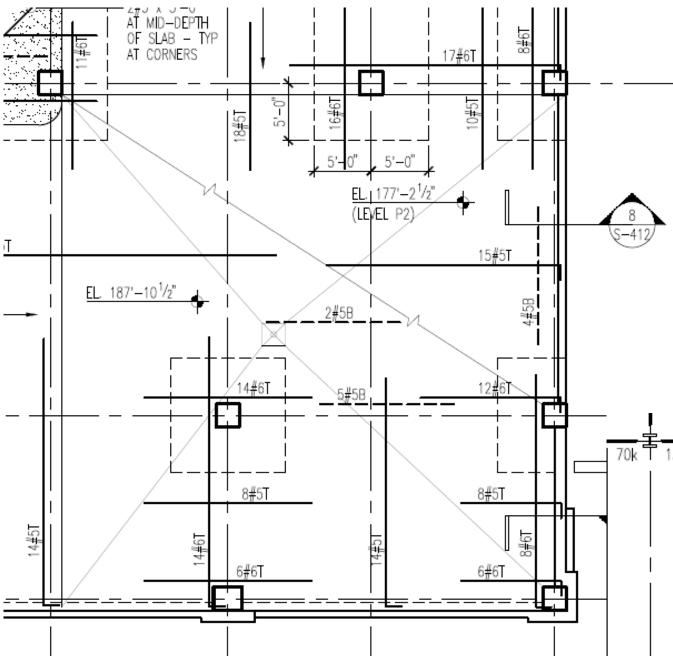


Figure 10: Typical Garage Bay (Level PL2)

(Source: Cagley & Assoc. Drawing S-203)

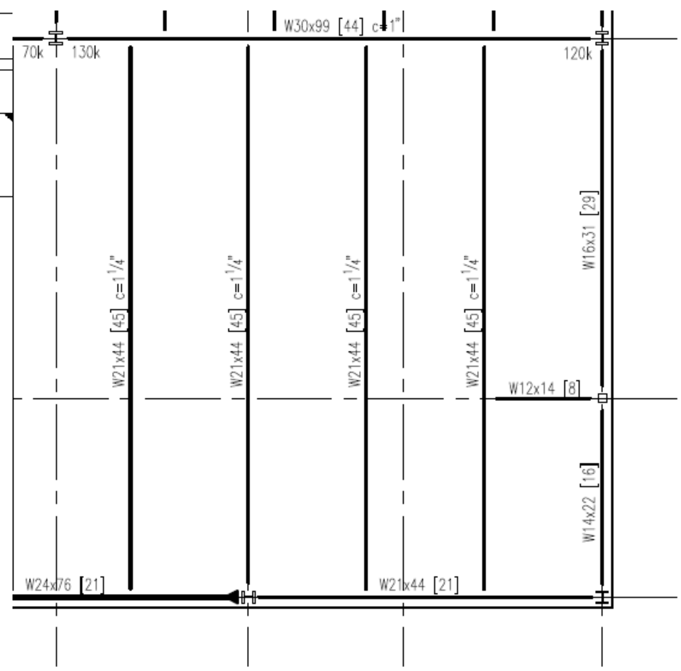


Figure 11: Typical Office Bay (Level OL3)

(Source: Cagley & Assoc. Drawing S-207)

Two-Way Flat Slab With Drop Panels

DESCRIPTION

The existing floor system used for the garage levels consists of an 8 inch thick 2-way flat slab with 8 inch thick drop panels at the columns. The drop panels are typically 10' by 10' and help to increase the punching shear capacity of the slab at the columns. Concrete with a compressive strength of 5000 psi and grade 60 reinforcement were used to construct the cast-in-place structure. Typical reinforcing for the bottom of the slab consists of #4 bars at 12" O.C each way. Top reinforcing for the slab typically consists of #5 and #6 bars.

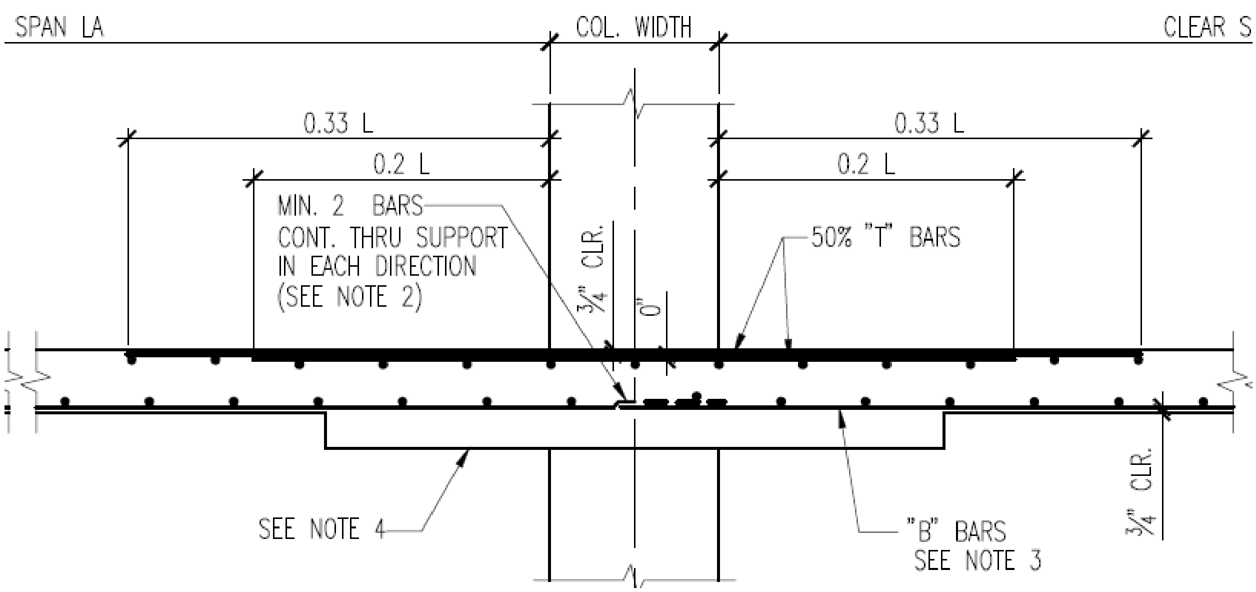


Figure 12: Typical Section at Column Strip (Source: Cagley & Assoc. Drawing S-410)

ADVANTAGES

Probably the most attractive feature of the flat slab is its ability to carry loads with absolutely minimal structural depth. This allows for a small floor to floor height, even after the mechanical systems are placed below the structure. The flat slab system typically has enough weight to it that vibrations are not a concern. Since there is so much flat surface area with this type of structure, formwork is generally simple and cheap when compared to concrete systems with beams and girders. This also eases constructability thus speeding up the construction time for this type of structure. Thanks to the inherent properties of the concrete, this system does not require additional fire-proofing to meet the 2 hour requirement.

DISADVANTAGES

The two-way flat slab is a significantly heavier system than any typical steel framing floor system. While not having an effect on KT36A, this extra weight could pose an issue at foundations if the flat slab system is being considered to replace a lighter existing system. Additionally, formwork required to cast the concrete is a cost that doesn't directly contribute to the strength of the floor system.

Precast, Pre-Tensioned Double-Tees On Precast, Prestressed Girders

DESCRIPTION

Intended to be an alternative for the garage levels of KT36A, this system was sized using a prestressed concrete textbook by Nawy and the PCI Design Handbook. For this alternative system, support along grid line B has been treated as though it is not there. It was decided to do this since the columns along this line terminate when the structure switches to steel construction. Based on hand calculations determining stress only, a 16” deep, 8’-0” wide double tee section with 14 - ½” diameter steel tendons was sized. The harped tendons with an ultimate strength of 270,000 psi required a 9.18 inch eccentricity at the center of the section and a 1 inch eccentricity at each end of the section to meet the stress limits in the tendons and concrete. Based on the double tee load table included in Appendix H of this report, an 8DT24 section appears to be a better solution for the 45’-0” span. The solution calculated by hand is likely governed by some other controlling factor such as deflection, resulting in the design tables conveying an 8DT24 as a more economical member. The double tees are supported by inverted T-beams at interior grid lines and L-beams at exterior grid lines. Using the PCI Design Handbook load tables, the support beams were designed as a 34IT28 with 20 - ½” diameter low relaxation strands for the interior and 20LB28 with 12 – ½” diameter low relaxation strands for the exterior of the bay. All of the precast members were evaluated using concrete with a 5000 psi compressive strength and tendons with an ultimate strength of 270,000 psi. See Figure 13 for a plan view of the system. Hand calculations for this system can be found in Appendix A.

ADVANTAGES

This style of precast, pretensioned sections is a common solution for parking garages. They offer great durability of the concrete since the members are cured under controlled conditions at the casting facility. A byproduct of this the nice, clean finish that the concrete obtains. While the erection procedure for this type of structure can be technical, the actual erecting of the precast members can go very quickly and efficiently. This quick method of construction has the ability to significantly reduce the duration of construction which could greatly reduce the project cost.

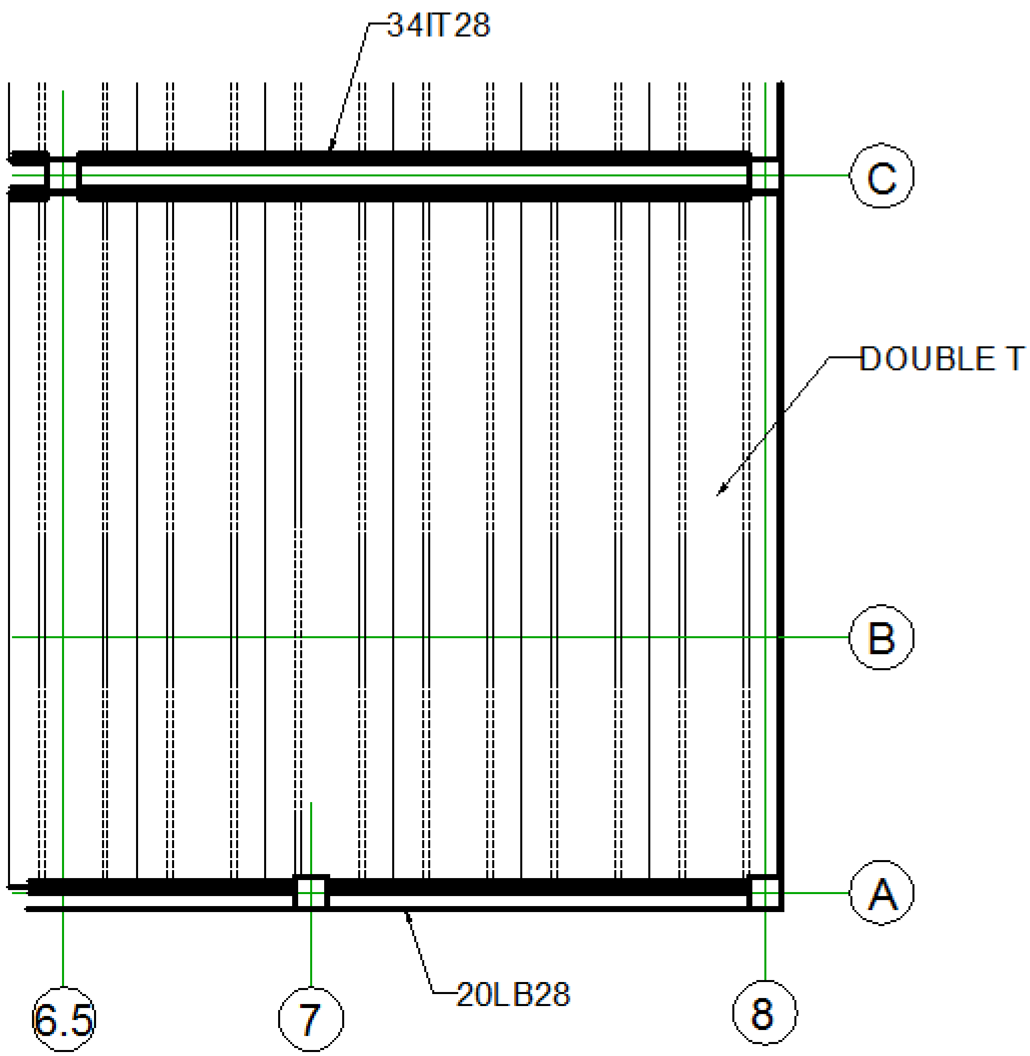


Figure 13: Schematic Double Tee Layout (Source: Chavanic)

DISADVANTAGES

As previously mentioned, erection of precast concrete structures can become quite technical. To start, two structural engineers are typically contracted in a precast concrete job, one to design the precast itself and one to design the foundations to support the precast. Precast structures are also poor performers against heavy lateral loads. A large lateral soil load is present in KT36A that would require careful consideration if this system was chosen to be designed. This load also works in conjunction with the controlling wind loads to make for an even more heavily loaded system.

Composite Steel Deck On Composite Beams And Girders

DESCRIPTION

The existing floor system used for the office levels of KT36A was evaluated using the Vulcraft deck catalog and RAM Structural System. Loads for this system are transferred in a one-way fashion along the load path, from slab deck to beam/joist, beam/joist to girder, and girder to column. As noted earlier, this system consists of a 2 inch 18 gage deck with a 3.25 inch lightweight concrete topping supported with composite steel wide flange members. According to the Vulcraft deck catalog, this slab will satisfy the 2 hour fire-rating. Looking at the bay used for the purposes of this report, the slab deck is supported by W21x44 beams spanning 45'-0" with 45 studs spaced evenly along the length of the beam. This is the most commonly used beam at the office levels. At the interior column line of this bay (column line C), the beams are supported by a W30x99 girder spanning 44'-0" with 44 studs. This is one of four girders spanning 44'-0", which is the largest girder span in the building. In order to satisfy the fire rating for the floor system, the beams and girders are coated with spray fire-proofing. A 3-D model of the bay used for analysis comparison can be seen in Figure 14. Hand calculations used as spot checks in Technical Report 1 and the RAM Structural System design for this system can be found in Appendix B.

ADVANTAGES

Composite deck and steel systems are very efficient in that they take advantage of the inherent compressive strength of the concrete. The composite action of the decking and the concrete allows greater spans between beams which equates to a lower number of beams needed to support the loads. By welding studs to the top of the beams, compressive forces are developed in the concrete which allows more of the steel member to be in tension. This allows a smaller steel section to be used which typically reduces structure depth and cost. When compared to a non-composite system on the same framing, a composite system can have longer span lengths and support more load with the same framing members.

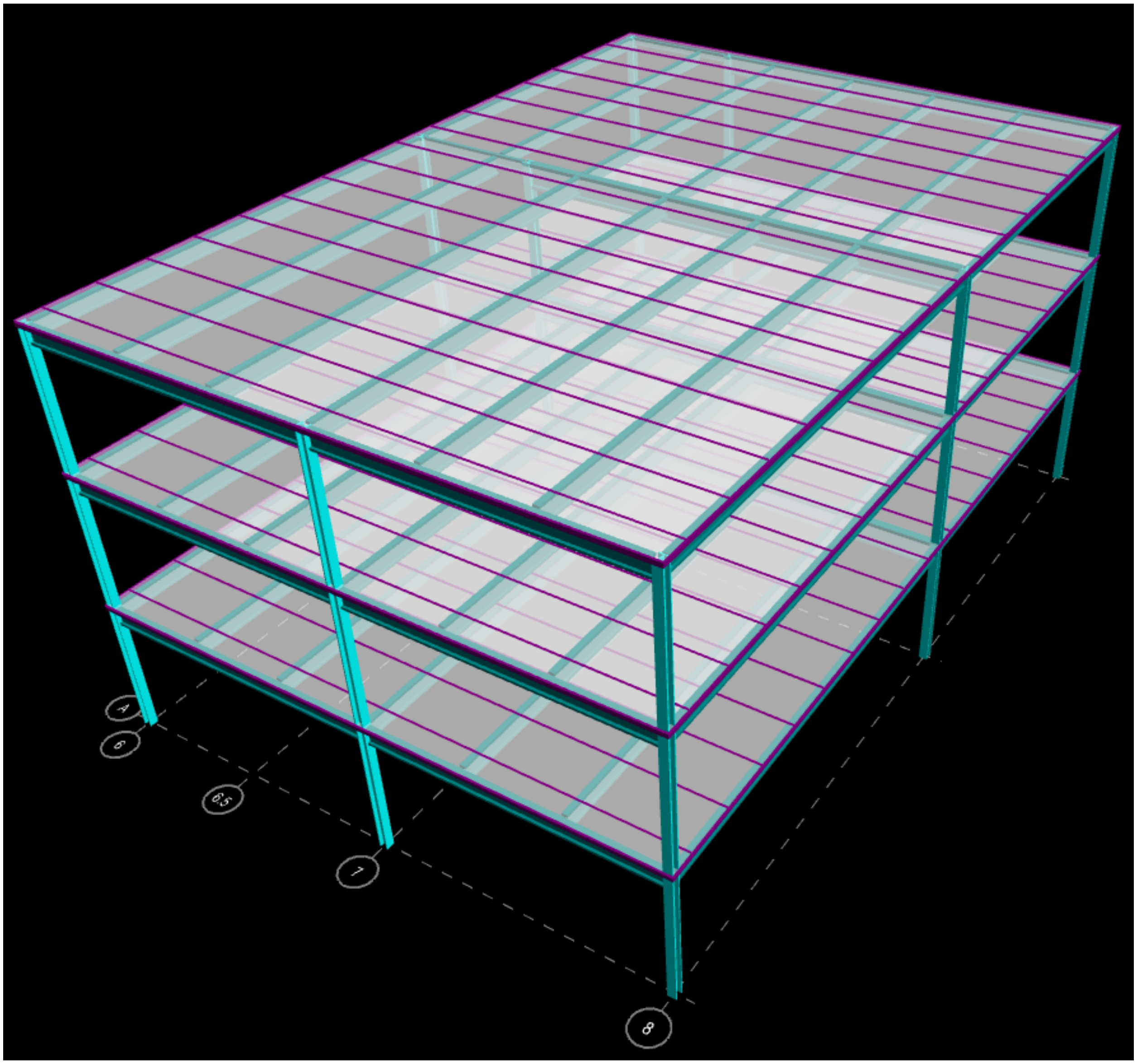


Figure 14: Model of Existing Office Level Bay Framing (Source: Chavanic)

DISADVANTAGES

When compared to a non-composite system, a composite system requires more time, labor, and inspections due to the shear studs being installed after the steel is erected and the deck has been laid down. The slab deck is typically controlled by the required fire rating; however, the supporting beams and girders must also meet this fire rating. In order to achieve this, spray fire-proofing is typically applied to the beams and girders. An alternative to this style of fire-proofing is using a gypsum board ceiling with batt insulation. A similar Underwriters Laboratories assembly can be seen in Appendix G (this assembly uses a steel joist instead of a rolled section).

One-Way Concrete Pan Joists With Wide Beams

DESCRIPTION

Intended as an alternative for the office floors of the building, this system was developed using ACI 318-11 provisions and standard pan joist dimensions. The joists span the 45'-0" long direction of the bay and are formed using 20 inch pans. Choosing a 6 inch joist width yields a 26 inch on center spacing of the joists. To meet the 2 hour fire rating requirement, a 4.5 inch thick one-way slab was chosen as the trial slab size. After checking the minimum thickness required to meet deflection criteria in ACI 318-11 Table 9.5(a), it was determined that a total system depth of 30 inches is needed to adequately span the 45'-0". Due to the standard joist pans being available in a maximum depth of 20 inches, a 10 inch solid slab is needed to make up the required depth of the system. Loads are transferred from the joists to the columns through wide beam girders that were constrained to the same depth of the joist and slab combination. To remain consistent with the design parameters used at the garage levels, concrete with a compressive strength of 5000 psi and grade 60 reinforcement were selected for use in the design.

Assuming one-way action in the slab, joist, and wide beam girder, reinforcement was designed based on the loads seen at the office floors. Analysis of the slab resulted in minimum steel for temperature and shrinkage controlling. For this, 1 #3 bar at 6 inches on center placed at the center of the slab was specified. Reinforcing for the joists was found to be 2 #6 bars and 1 #7 bar at the interior negative moment region, 1 #8 bar at the exterior negative moment region, and 1 #9 bar at the positive moment region of the joist. The interior 44'-0" span girder was designed for this system as it is the controlling girder span present. After a couple of design iterations, a 30 inch deep by 54 inch wide rectangular section was found to carry the loads. Required reinforcement for the girder was found to be 20 #11 bars in 2 layers at the interior negative moment region of the girder, and 20 #9 bars in 2 layers at the positive moment and exterior negative moment regions of the girder.

Considering the excessive weight of this structural floor system and the significant impacts it would have on the building foundations, it was deemed not feasible for this application. However, with a consideration to reducing bay sizes, this floor system could become an economical option for KT36A. This system could also be a potential alternative at the parking garage levels which also warrants reconsidering its feasibility. A plan view of the proposed layout can be seen in Figure 15. Hand calculations for this system can be found in Appendix C.

ADVANTAGES

A one-way concrete pan joist system can be very effective at spanning large distances, just not ones quite as long as the 45'-0" spans in KT36A. The shear massiveness of the system allows vibration considerations to essentially be ignored. In addition to the voids creating the joists reducing the self-weight of the system, they also allow space for routing mechanical ducts. Since the concrete has inherent fire-proofing, additional spray fire-proofing is not necessary for this type of system, reducing the overall cost of the floor.

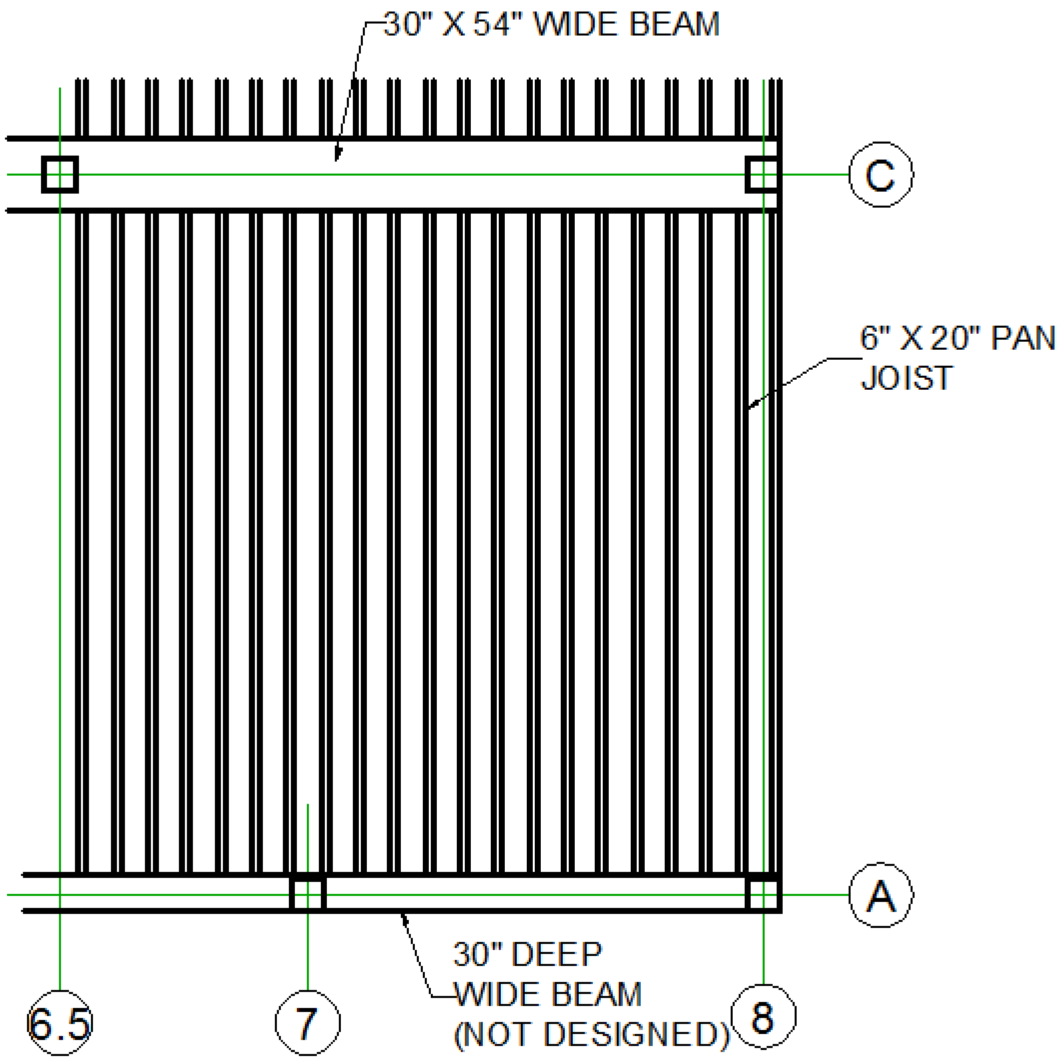


Figure 15: Schematic Layout of One-way Concrete Pan Joist System (Source: Chavanic)

DISADVANTAGES

While the massiveness of the system is great for dampening floor vibrations, it causes some major drawbacks with the rest of the building. In order to support all of the weight, concrete columns will have to increase in size and contain heavier amounts of reinforcement. This directly leads to foundation redesign in order to support the increased loads. In the case of KT36A, the intermediate foundation system consisting of spread footings bearing on rammed aggregate piers will most likely have to be changed to a more expensive, deep foundation system. The significant increase in weight will also affect the forces induced on the building from seismic ground motion. Said forces may increase enough to make the seismic loading control over the wind loading which would then affect the design of the lateral system.

STEEL FORM DECK ON OPEN-WEB STEEL JOISTS AND WIDE FLANGE GIRDERS

DESCRIPTION

Intended as an alternative for the office floors of the building, this system was sized using design guides from Vulcraft and Table 3-10 from the Steel Construction Manual. Similar to the existing system, loads are transferred in a one-way fashion along the load path, from slab deck to beam/joist, beam/joist to girder, and girder to column. The major difference with this alternative is that no composite action exists in either the slab or supporting members. In order to maintain the two-hour fire rating required for the office levels, a 1.5 inch 24 gage deck with a 2.5 inch normal-weight concrete topping reinforced with 6x6-W2.9xW2.9 welded wire fabric was chosen.

For the initial design of the floor systems, a 4'-0" spacing of the joists was chosen. Due to the joists spanning in the 45'-0" long direction, a 4'-0" tributary width on them provided too much load per joist to use a K-series joist. Because of this, two sub-options were examined for this alternative. Option one is using 28LH09 series joists at 4'-0" on center. Referencing RSMeans CostWorks, this style of joist costs approximately $24.89/linear foot (based on linear interpolation of costs between a 28LH06 and 28LH11). Option two is using 26K10 joists at 2'-0" on center. Again using RSMeans CostWorks, this style of joist costs $15.35/linear foot. Since twice as many K-series joists will be needed, the LH series joist looks to be the more economical option. If this system were to be utilized, a more detailed cost estimate considering price reductions for ordering bulk quantities would be warranted to see if the K-series joists have the chance of being the better option.

Each of the sub-options resulted in the same girder design. While not being the most economical section, a W30x173 with a ¾" camber was chosen for the girder spanning 44'-0" along column line C. This section would be stressed at 95% when subjected to full loading (not considering live load reduction). In order to not impact the architecture of the office levels, this girder was restricted to a 30" depth which resulted in the heavier weight being required. Based on the factors taken into account in Figure 17, this system, when constructed with either of the joist options, proved to be a feasible alternative. Hand calculations for this system can be found in Appendix D.

ADVANTAGES

Steel web joists covered by non-composite deck can economically and lightly span large distances such as those found in the bays of KT36A. When considered throughout a building, the weight savings can lead to decreased loads on footings and reduced seismic loading. For the case of KT36A, this floor system weighs about the same as the existing system, approximately 50 psf when including the distributed weight of the framing. Another advantage of a steel web joist system is that it allows the mechanical equipment in the ceiling to pass through the structure, which can't be done when wide-flange beams are being used. Even though the joists are sized deeper than the existing composite beams of KT36A, overall ceiling to floor depth may be decreased due to this feature.

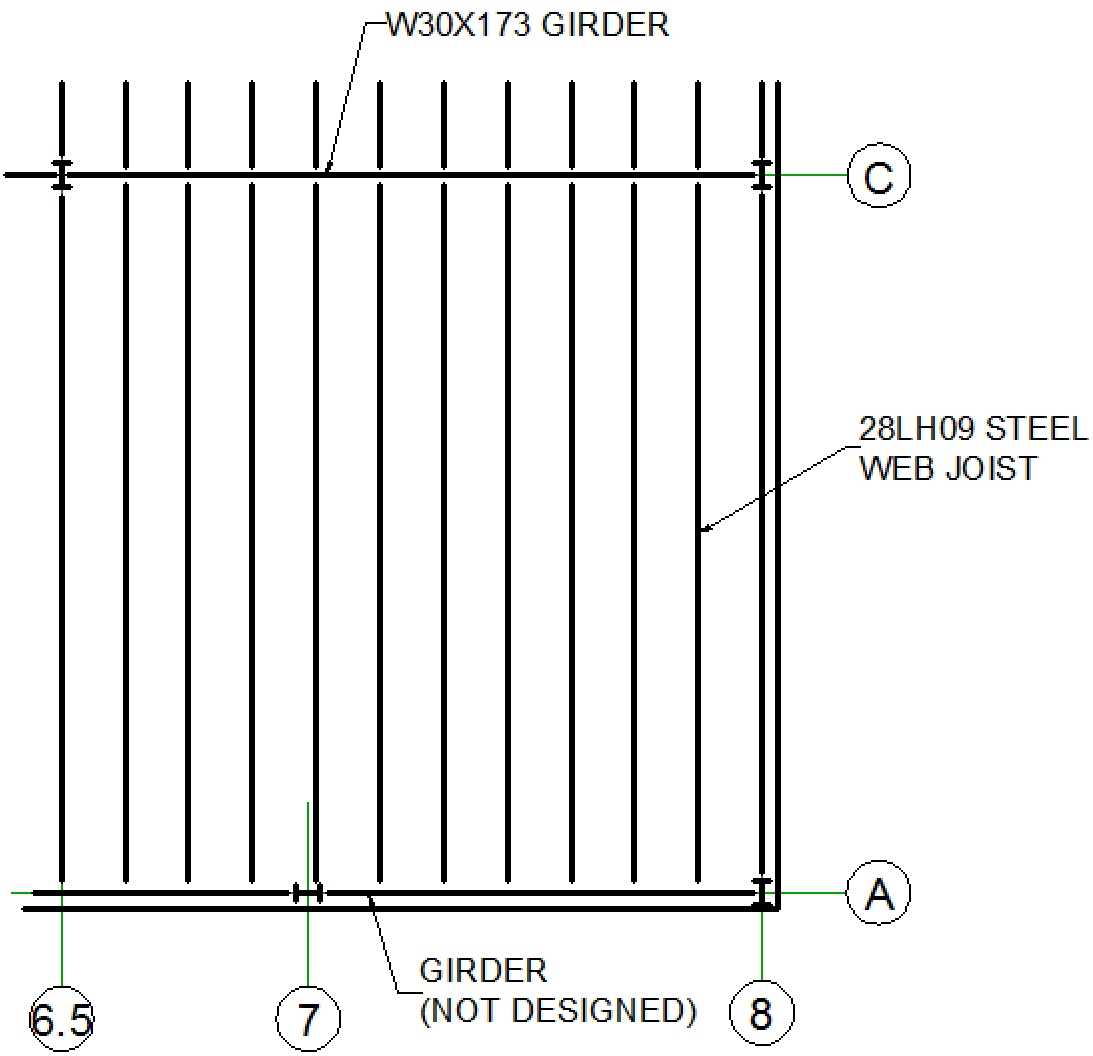


Figure 16: Schematic Layout of Open-web Steel Joist System (Source: Chavanic)

DISADVANTAGES

The lightweight nature of this system can also lend itself to concerns when considering floor vibrations. To help inhibit this, normal weight concrete was chosen for the deck topping as opposed to a lightweight concrete topping. While the deck itself meets the fire rating, the joists supporting the deck need some sort of fire-proofing in order to meet the requirement. A common option for wide flange supports, spray fire-proofing is quite messy when applied to joists. It is also difficult to ensure an even coating over the entire surface area of the joists. Due to the difficulty and expense of the spray fire-proofing, a U.L. certified assembly using gypsum board and batt insulation (such as the one in Appendix G) is commonly used with steel web joist floor systems.

Summary Comparison

The chart below provides a side-by-side comparison of the existing and proposed alternative systems. In this chart, the final row provides a personal opinion on whether or not the system is feasible for KT36A based on engineering judgment of the information uncovered in this report.

		Systems					
		Office Levels (80 psf LL)				Garage Levels (40 psf LL)	
		Existing	Alternatives			Existing	Alternative
Consideration		Composite Steel Deck on Composite Beams and Girders	One-way Concrete Pan Joists on Wide Girders	Non-Composite Steel Deck on Open Web Joists		2-Way Flat Slab With Drop Panels	Precast, Prestressed Double Tees on Precast, Prestressed Beams
Sub-Options				LH Series Joists (4' O.C.)	K-Series Joists (2' O.C.)		
System Stats							
	Slab Weight	44 psf	125 psf	45 psf	45 psf	100 psf	67 psf
	System Weight	49 psf	207 psf	50 psf	52 psf	121 psf	72 psf
	Slab Depth	5.25"	10"	4"	4"	8"	4"
	System Depth	35.25"	30"	39"	36.5"	16"	30"
	Assembly Cost	\$25.38/SF	\$20.97/SF	\$15.32/SF	\$18.89/SF	\$16.60/SF	\$23.25/SF
Architectural							
	Bay Size	28'-6" x 45'-0"	28'-6" x 45'-0"	28'-6" x 45'-0"	28'-6" x 45'-0"	28'-6" x 29'-0"	28'-6" x 45'-0"
	Fire Rating	2 HR - UL Assembly	2 HR	2 HR - UL Assembly	2 HR - UL Assembly	2 HR	< 2 HR
	Other	Additional fire-proofing needed to protect framing members	Decrease in floor to floor height Structure changes to concrete	Increase in floor to floor height, however, may be offset by running mechanical entities through joists	Increase in floor to floor height, however, may be offset by running mechanical entities through joists	Smaller bays needed to make system economical	Provides ability to eliminate 2 column lines in the garage levels Increased floor to floor height over existing slab
Structural							
	Gravity System Alterations	No Change	Concrete joists w/ wide beam girders Concrete columns	Beams become open web joists	Beams become open web joists	No Change	Precast beams, girders, and columns
	Lateral System Alterations	No Change	Extend shear walls from garage levels up to office levels	Little to no change	Little to no change	No Change	Little to no change
	Foundation Alterations	No Change	Significant Impact Increased footing sizes Deep foundation system may be necessary	Little to no change	Little to no change	No Change	May be able to reduce footing sizes due to reduced self-weight
Construction							
	Formwork Required	Minimal	Yes	Minimal	Minimal	Yes	None
	Constructability	Technical	Technical	Easy	Easy	Technical	Technical
	Lead Time	Standard	Standard	Standard	Standard	Standard	Long
Serviceability							
	Vibration Control	Mediocre	Great	Mediocre	Mediocre	Great	Good
Feasible		Yes	No	Yes	Yes	Yes	Yes

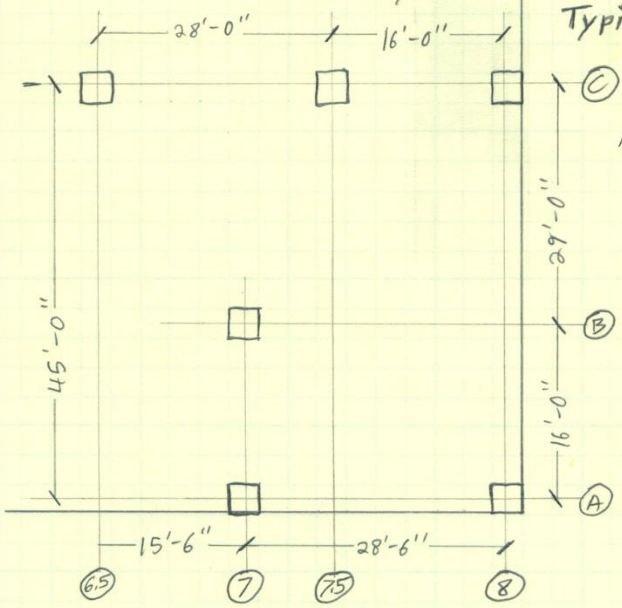
Figure 17: System Comparison Matrix (Source: Chavanic)

CONCLUSION

After examining Figure 17, the one-way concrete pan joist system was the only alternative considered not feasible for Kingstowne Section 36 A. The spans lengths of 45'-0" found in the end bays are simply too much for this floor system. The long spans yield a system that needs to be 30 inches deep in order to satisfy ACI 318-11 Table 9.5(a). Since pan joists come in a maximum depth of 20 inches, this meant that a 10 inch thick slab had to be chosen to get the total depth to 30 inches. This resulted in a massive system that weighs 4 times the existing system at the office levels! If the owner permitted, bay sizes could be adjusted making this a very plausible system. The remaining alternatives were all deemed feasible and are worthy of being considered for further study. Of special note is the precast double tee system which has the potential to be used in all levels of the building. This would pose an interesting problem to solve regarding the handling of lateral loads. If the columns along grids B and E were permitted to come up through the office levels, continuing the flat slab construction into the office levels could become a very probable alternative.

APPENDIX: A

Alternative #3 | Oneway PT Double Tee | James Chavanic | 1



Typical Parking Garage Bay

Design Precast - Double Tee section

Assumptions
NWC: $f'_c = 5000psi$
$f'_{ci} = 75\%$ of 28 day strength
losses $= 35{,}000psi$ → Table 3.2 in Nawy Text
$f_{pu} = 270{,}000psi$
$f_t = 12\sqrt{f'_c}$

← propose to eliminate column line (B)
∴ 45'-0" span from center of support to center of support
✱ This will impact column design and foundation design

Loads:

Dead
SIDL: 5psf
Self wt.: ??
2" topping: 25psf

Live: 40psf → IBC 2009
50psf was used at time of design
Estimate 900 lb/ft selfweight, check later

– Try design with the 8' wide module

$$w_u = 1.2[(5+25)8+900] + 1.6\cdot 40\cdot 8$$
$$w_u = 1880 \text{ lb/ft}$$

self wt
$$w_D = 1.2[25\cdot 8 + 900]$$
$$w_D = 1320 \text{ lb/ft}$$
$$M_D = \frac{w_D l^2}{8} = \frac{1320\cdot 45ft^2}{8}\cdot 12$$
$$M_D = 4{,}009{,}500 \text{ in}\cdot\text{lb}$$

superimposed load
$$w_{S+L} = 1.2\cdot 5\cdot 8 + 1.6\cdot 40\cdot 8$$
$$w_{S+L} = 560 \text{ lb/ft}$$
$$M_{S+L} = \frac{w_{S+L}\, l^2}{8} = \frac{560 \text{ lb/ft}\cdot 45ft^2}{8}\cdot 12$$
$$M_{S+L} = 1{,}701{,}000 \text{ in}\cdot\text{lb}$$

$$f'_{ci} = -0.75\cdot 5000psi = -3{,}750psi$$

Stress limit in tendon at initial prestress
$$f_{pi} = 0.7\cdot f_{pu} = 0.7\cdot 270{,}000psi$$
$$f_{pi} = 189{,}000psi$$

Effectives stress after prestress
$$f_{pe} = 189{,}000psi - 35000psi = 154{,}000psi$$

$$\gamma = 1 - \frac{\text{losses}}{f_{pi}}$$
$$\gamma = 1 - \frac{35000}{189000} = 0.815$$
$$= 81.5\%$$

Alternative #3 One way PT Double-Tee James Chavanic 2

Find required section Modulus

$$S^{top} \geq \frac{(1-\gamma)M_D + (M_{SD}+M_L)}{\gamma f_{ti} - f_c}$$

$f_{ti} = 3\sqrt{f'_{ci}} = 184\,psi$

$f_c = 0.45 f'_c = -2250\,psi$

$f_t = 6 \cdot 7\sqrt{f'_c} = 424.3\,psi$

$f_{ci} = 0.6 f'_{ci} = -2250\,psi$

$$S^{top} \geq \frac{(1-0.815)(4,009,500 + 1,701,000}{(0.815 \cdot 184 - -2250)}$$

$$S^{top} \geq 440.2\ in^3$$

$$S^{bottom} \geq \frac{(1-\gamma)M_D + M_{SD} + M_L}{f_t - \gamma f_{ci}}$$

$$S^{bottom} \geq \frac{(1-0.815)(4,009,500 + 1,701,000)}{424.3 - 0.815 \cdot -2250}$$

$$S^{bottom} \geq 468\ in^3$$

use 8DT16

$S^{top} = 1630\,in^3$

$S_{bott} = 556\,in^3$

$b_f = 96''$ $h = 16''$

$t_f = 2''$

$b_{w\,bott} = 3.75''$ $A_c = 325\,in^2$

$b_{w\,top} = 5.75''$ $I_c = 6634\,in^2$

Values from PCI Design Handbook

$r^2 = 20.41\,in^2$

$e_e = 3.93$ $Y_b = 11.93''$

$e_c = 9.18$ $Y_t = 4.07''$

$wt_{w/top} = 539\,plf < 900$ OK

Design of Strands and check of stresses

$$M_D = \frac{539\,plf}{900 + 25.8} \cdot (4,009,500) = 1,964,655\ in \cdot lb$$

stresses at transfer

$$f_t = -\frac{P_i}{A_c}\left(1 - \frac{ec_t}{r^2}\right) - \frac{M_D}{S_t} \leq f_{ti} = 184\,psi$$

$$\frac{-\left(184\,psi + \frac{M_D}{S_t}\right) \cdot A_c}{\left(1 - \frac{ec_t}{r^2}\right)} = P_i \quad P_i = \frac{-\left(184 + \left(\frac{1964655}{1630}\right)\right) \cdot 325}{1 - \frac{9.18 \cdot 4.07}{20.41}}$$

Max $P_i = 543,612\ lb$

$$\text{Max Required \# of tendons} = \frac{543,612}{189,000 \cdot 0.153} = 18.8\ \tfrac{1}{2}''\text{ dia tendons}$$

Try 14 ½" dia strands for the standard section

$A_{ps} = 14 \cdot 0.153\,in^2 = 2.142\,in^2$

$P_i = 2.142\,in^2 \cdot 189,000 = 404,838\ lb$ $P_e = 2.142\,in^2 \cdot 154000 = 329868\ lb$

Alternative #3 | Oneway PT Double Tees | James Chavanic | 3

Analysis of Stresses at service load at Midspan

$P_e = 329{,}868 \text{ lb}$

$M_{s+L} = 1{,}701{,}000 \text{ in}\cdot\text{lb}$

$M_{total} = M_D + M_{s+L} = 3{,}665{,}655 \text{ in-lb}$

$$f^t = -\frac{P_e}{A_c}\left(1 - \frac{e c_t}{r^2}\right) - \frac{M_T}{S^t}$$

$$= -\frac{329868 \text{ lb}}{325 \text{ in}^2}\left(1 - \frac{9.18 \cdot 4.07}{20.41}\right) - \frac{3{,}665{,}655}{1630}$$

$f^t = -1406 \text{ psi} < f_c = -2250 \text{ psi}$ O.K.

$$f^b = -\frac{P_e}{A_c}\left(1 + \frac{e c_b}{r^2}\right) + \frac{M_T}{S_b}$$

$$= -\frac{329868 \text{ lb}}{325 \text{ in}^2}\left(1 + \frac{9.18 \cdot 11.93}{20.41}\right) + \frac{3665655}{556}$$

$f^b = 131.7 \text{ psi (Tension)} < 424.3 \text{ psi} = f_t$ O.K.

Analysis of Stresses at Support Section

$e_e = 3.93''$ $\quad f_{ti} = 6\sqrt{f'_c} = 368 \text{ psi}$

simply supported

$f_t = 12\sqrt{f'_c} = 849 \text{ psi}$

At transfer

$$f^t = -\frac{P_i}{A_c}\left(1 - \frac{e c_t}{r^2}\right) - 0 = -\frac{404838 \text{ lb}}{325 \text{ in}^2}\left(1 - \frac{3.93 \cdot 4.07}{20.41}\right) - 0$$

$f^t = -269 \text{ psi} < f_{ci} = -2250 \text{ psi}$ O.K.

compression

$$f^b = -\frac{P_i}{A_c}\left(1 + \frac{e c_b}{r^2}\right) + 0 = \frac{-404838 \text{ lb}}{325 \text{ in}^2}\left(1 + \frac{3.93 \cdot 11.93}{20.41}\right) + 0$$

$f^b = -4107 \text{ psi} > -2250 \text{ psi} = f_{ci}$ N.G.

Alternative #3 | One way PT Double Tees | James Chavanic | 4

At Service Load

$$f^t = \frac{-P_e}{A_c}\left(1 - \frac{e c_t}{r^2}\right) - \frac{M_T}{S^t} = -\frac{329868\text{lb}}{325\text{in}^2}\left(1 - \frac{3.93 \cdot 4.07}{20.41}\right) - 0$$

$$f^t = -219.6\text{ psi} \underset{\text{compression}}{<} f_{ci} = -2250\text{psi} \quad \underline{\underline{\text{O.K.}}}$$

$$f^b = \frac{-P_e}{A_c}\left(1 + \frac{e c_b}{r^2}\right) + \frac{M_T}{S^T} = -\frac{329868}{325\text{in}^2}\left(1 + \frac{3.93 \cdot 11.93}{20.41}\right) + 0$$

$$f^b = -3346\text{ psi} > f_c \quad \therefore \text{ eccentricity at end needs changed and checked}$$

using a 1" eccentricity

$$f^t = -812.6\text{ psi} < f_c = -2250\text{psi} \quad \underline{\text{O.K.}}$$

$$f^b = -1608\text{ psi} < f_c = -2250\text{psi} \quad \underline{\text{O.K.}}$$

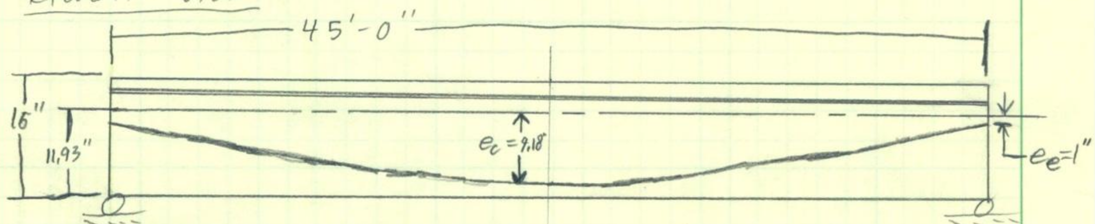
Recheck at transfer

$$f^t = -997.3\text{psi} < f_{ci} = -2250\text{psi} \quad \underline{\underline{\text{O.K.}}}$$

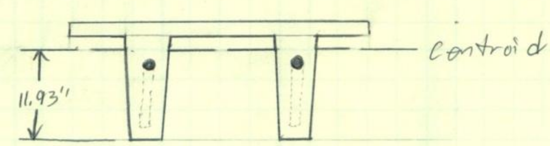
$$f^b = -1974\text{ psi} < f_{ci} = -2250\text{psi} \quad \underline{\text{O.K.}}$$

Use 8DT16 with 14 ½" diameter tendons
mid span eccentricity $= e_c = 9.18''$
end eccentricity $= e_e = 1.00''$

Elevation View

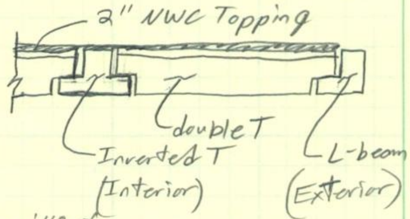


Section View



Alternative #3 | One way PT double Tees | James Chavanic | 5

Design Support Girders



Interior Girders (Inverted T)

worst span = 28'-0"

Loads:

Dead: 2" topping : 25 psf; Double T : 42 psf; SI DL : 5 psf

Live: Garage : 40 psf

$$\text{Trib width} = \left(\frac{45' + 36.5'}{2}\right) = 40.75'$$

$$w_u = 1.2 \cdot (25+42+5) \cdot 40.75 + 1.6 \cdot 40 \cdot 40.75$$

$$w_u = 6130 \text{ lb/ft}$$

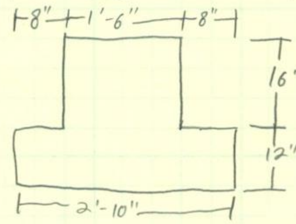
Precast members are simply supported

$$M_u = \frac{w_u \ell^2}{8} = \frac{6.13 \text{ k/ft} \cdot (28')^2}{8}$$

$$M_u = 600 \text{ k}\cdot\text{ft}$$

From PCI Design Handbook Load Tables

use: 34IT28 w/ 20 1/2 Ø low relax strands, $f'_c = 5000$ psi, $f_{pu} = 270{,}000$ psi



Exterior Girders (L-beam)

worst span = 28'-6"

Loads:

Dead: 2" topping : 25 psf; Double T : 42 psf; SI DL : 5 psf

Live: Garage : 40 psf

$$\text{Trib width} = \frac{45'}{2} = 22.5'$$

$$w_u = 1.2(25+42+5) \cdot 22.5 + 1.6 \cdot 40 \cdot 22.5$$

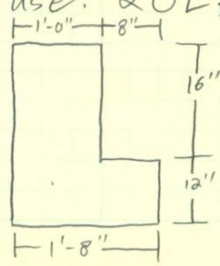
$$w_u = 3384 \text{ lb/ft}$$

$$M_u = \frac{w_u \ell^2}{8} = \frac{3.384 \text{ k/ft} \cdot (28.5')^2}{8}$$

$$M_u = 344 \text{ k}\cdot\text{ft}$$

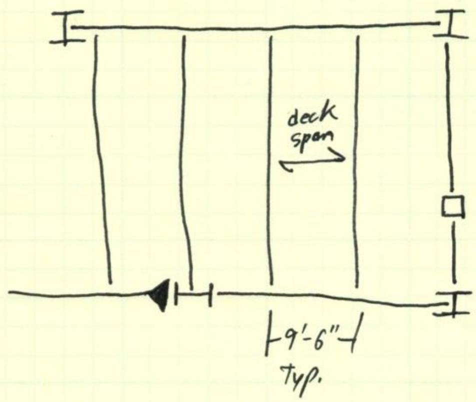
From PCI Design Handbook Load Tables

use: 20LB28 with 12 1/2" Ø low relax strands, $f'_c = 5000$ psi, $f_{pu} = 270{,}000$ psi



APPENDIX: B

James Chavanic | Tech 1 | Decking Spot Check | C.1



Typical bay at office levels 2-4

Span = 9'-6" typical

3¼" LWC on 2"x18 GA deck

using 2VLI18 from 2008 Vulcraft Catalog

SDI Max unshored Clr span

1 span = 10'-6" > 9'-6" ✓

2 span = 12'-7" > 9'-6" ✓

3 span = 12'-7" > 9'-6" ✓

Max Superimposed LL = 222 psf

$SILL_{req} = \underbrace{7\,psf}_{framing} + \underbrace{15\,psf}_{SIDL} + \underbrace{5\,psf}_{MEP} + \underbrace{80\,psf}_{office} + \underbrace{15\,psf}_{partitions}$

$SILL_{req} = 122\,psf$

222 psf > 122 psf ∴ plenty strong enough

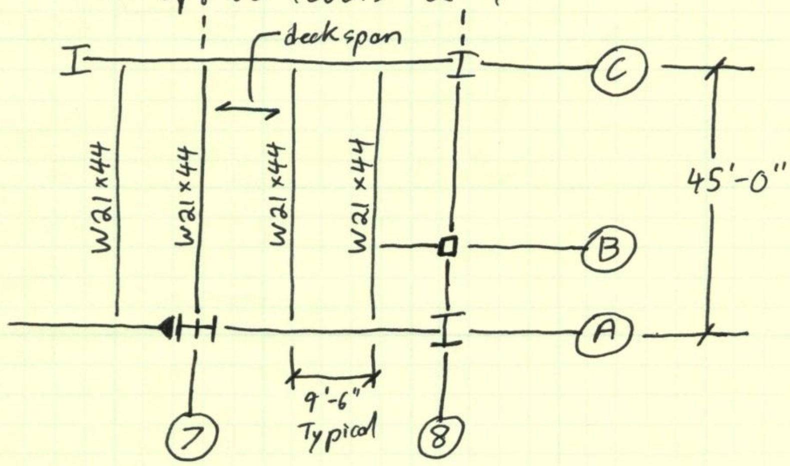
Possibly oversized to carry storage loads

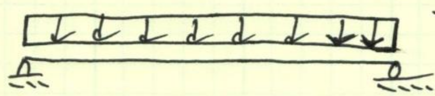
Concrete thickness needed for fire protection

Even though S-001 states that decks shall meet the 3 span condition, they still pass if a single span section is needed

James Chavanic | Tech 1 | Beam Spot Check | G2

Typical W21x44 beam between Grids A and C
at office levels 2-4





Span: 45'-0"
Trib width: 9'-6"
$f'_c = 3000$ psi LWC
$t = 5.25''$
$A_s = 13.0 \text{ in}^2$ $f_y = 50$ ksi
$d = 20.7''$ $b_f = 6.5''$

Dead Loads
SW: 44 psf
SID: 15 psf
MEP: 5 psf
Beam SW: 44 plf

Live Loads
Office: 80 psf
Partitions: 15 psf minimum

Live Load Reduction

$$L = L_0\left(0.25 + \frac{15}{\sqrt{K_{LL} \cdot A_T}}\right)$$

$$L = 95\text{psf}\left(0.25 + \frac{15}{\sqrt{2 \cdot 45' \cdot 9.5'}}\right)$$

↳ Int. beam

$$L = 73 \text{ psf}$$

$$w_u = 1.2\, w_D + 1.6\, w_L \quad \boxed{\text{LRFD}}$$

$$w_u = 1.2\left[(44+15+5)9.5+44\right] + 1.6 \cdot 73 \cdot 9.5$$

$$w_u = 1892 \text{ lb/ft} \approx 1.9 \text{ klf}$$

James Chavanic | Tech 1 | Beam Spot Check (cont.)

C.3

$M_u = \frac{w_u l^2}{8} = \frac{1.9klf \cdot (45ft)^2}{8}$ $\underline{M_u = 481\ k \cdot ft}$

$b_{eff} = \min \begin{cases} \text{Trib width} = 9.5' \rightarrow \text{controls} \\ span/4 = 45/4 = 11.25' \end{cases}$ $b_{eff} = 9.5' = 114''$

$V_{c\,max} = 0.85 \cdot f'_c \cdot b_{eff} \cdot t$

$V_{c\,max} = 0.85 \cdot 3000 \cdot 114'' \cdot 5.25''$

$V_{c\,max} = 1526\ k$

$V_{s\,max} = A_s \cdot f_y$

$V_{s\,max} = 13.0 in^2 \cdot 50ksi$

$V_{s\,max} = 650\ K$

$\Sigma Q_n = \frac{45 studs}{2} \cdot 17.2 K/stud$ (Table 3-21)

$\Sigma Q_n = \underline{387K}$

→ ⊥ deck
weak stud position

→ $< V_{s\,max} < V_{c\,max}$ ∴ ~~PNA in concrete~~ Partially Composite

N/A in flange?

~~$x = \frac{V_{s\,max} - \Sigma Q_n}{2 \cdot 50 ksi}$~~

$x = \frac{V_{s\,max} - \Sigma Q_n}{2 \cdot f_y \cdot b_f}$

$x = \frac{650k - 387k}{2 \cdot 50ksi \cdot 6.5''} = 0.4046'' < t_f = 0.45''$

∴ PNA is in top flange

$M_n = \Sigma Q_n \cdot (t - a/2) + A_s \cdot f_y \cdot d/2 - 2 \cdot f_y \cdot b_f \cdot x \cdot \frac{x}{2}$

where $a = \frac{\Sigma Q_n}{0.85 \cdot b_{eff} \cdot f'_c}$ and $x = 0.4046''$

$a = \frac{387K}{0.85 \cdot 114'' \cdot 3ksi} = 1.331''$

$M_n = 387K \cdot (5.25'' - \frac{1.331''}{2}) + 650K \cdot \frac{20.7''}{2} - 2 \cdot 50ksi \cdot 6.5'' \cdot \frac{0.4046^2}{2}$

$M_n = 704\ k \cdot ft$

$\phi M_n = 0.9 \cdot 704 k \cdot ft$

$\phi M_n = 634\ K \cdot ft$ ⟹ strength capacity OK

$\phi M_n > M_u$

James Chavanic | Tech 1 | Beam Spot Check (cont) G4

Check deflections

wet Concrete

$$\Delta_{wc\,max} = \frac{\ell}{240} = \frac{45'\cdot 12''}{240} = 2.25'' \quad I_{w21\times44} = 843\,in^4$$

$$\omega_{wc} = 44psf \cdot 9.5' + 44plf = 0.462\,klf$$

$$\Delta_{wc} = \frac{5w\ell^4}{384EI} = \frac{5\cdot 0.462klf\cdot(45\cdot 12)^4\cdot\frac{1}{12}}{384\cdot 29000ksi\cdot 843in^4} = 1.744'' < 2.25'' \checkmark$$

Live Load

Find I_{LB}, need $\bar{y}$ $\quad y_2 = t - a/2 = 5.25'' - \frac{1.331}{2} = 4.585''$

$$\bar{y} = \frac{A_s\cdot d/2 + \frac{\Sigma Q_n}{F_y}(d+y_2)}{A_s + \frac{\Sigma Q_n}{F_y}} = \frac{13.0in^2\cdot\frac{20.7}{2} + \frac{387k}{50ksi}(20.7'' + 4.585'')}{13in^2 + \frac{387k}{50ksi}}$$

$$\bar{y} = 15.924''$$

$$I_{LB} = I_x + A_s\left(\bar{y} - d/2\right)^2 + \frac{\Sigma Q_n}{F_y}\left(d + y_2 - \bar{y}\right)^2$$

$$I_{LB} = 843in^4 + 13in^2\left(15.924 - \frac{20.7}{2}\right)^2 + \frac{387k}{50ksi}\left(20.7 + 4.585 - 15.924\right)^2$$

$$I_{LB} = 1925\,in^4$$

$$\Delta_{LL\,max} = \frac{\ell}{360} = \frac{45\cdot 12}{360} = 1.5'' \quad \omega_{LL} = 95psf\cdot 9.5' = 0.903\,klf$$

$$\Delta_{LL} = \frac{5w\ell^4}{384EI_{LB}} = \frac{5\cdot 0.903klf/12\cdot(45\cdot 12)^4}{384\cdot 29{,}000ksi\cdot 1925in^4} = 1.49'' < 1.50'' \checkmark$$

Check unshored strength

$$\phi M_p = 358\,k\cdot ft$$

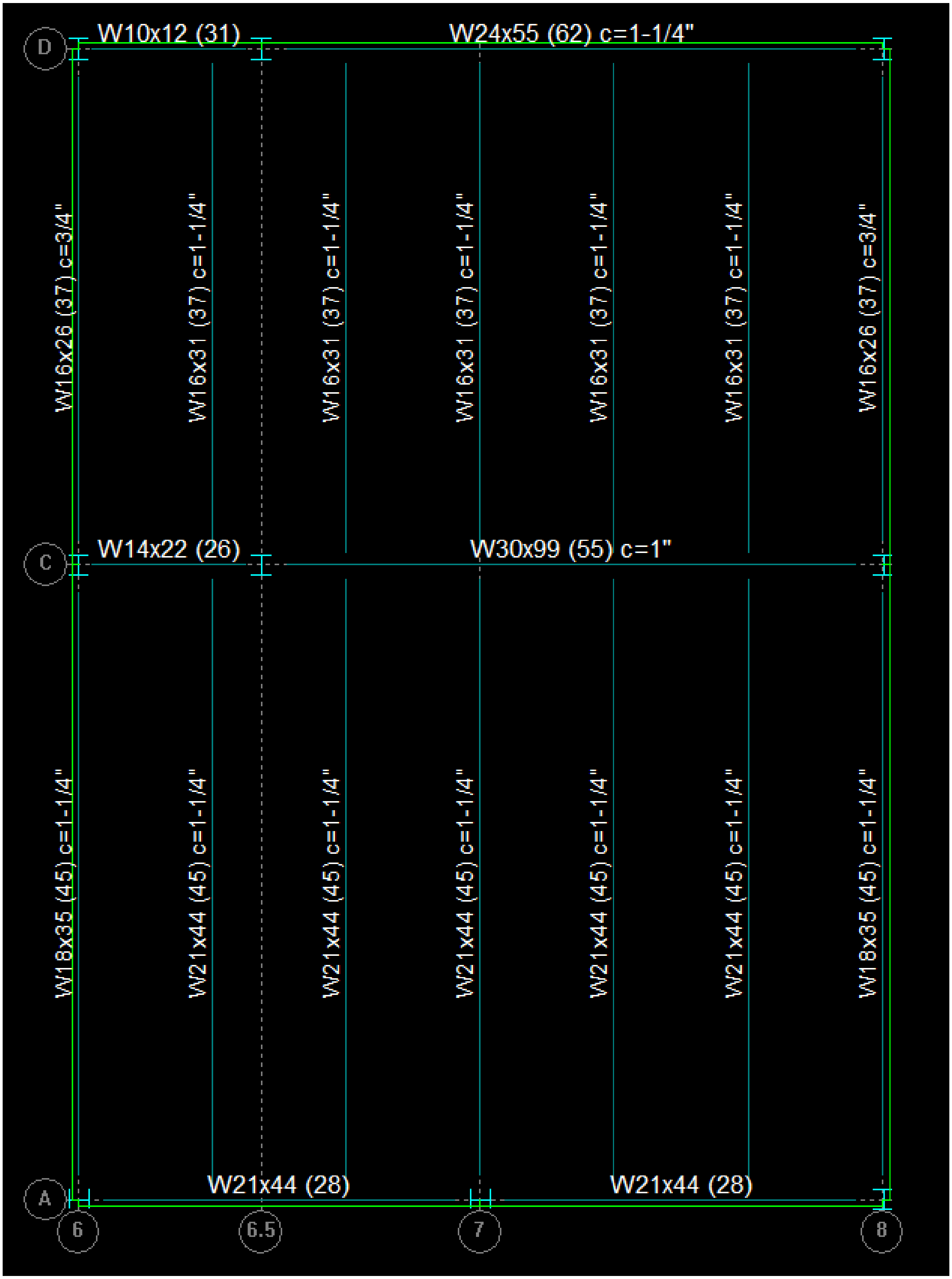
$$\omega_{u\,DL} = 1.4\cdot(44\cdot 9.5 + 44) = 0.647\,klf$$

$$\omega_{u\,CL} = 1.2\cdot(44\cdot 9.5 + 44) + 1.6\underbrace{(20\cdot 9.5)}_{\text{construction LL}} = 0.858\,klf$$

$$M_u = \frac{\omega_{uCL}\,\ell^2}{8} = \frac{0.858klf\cdot 45^2}{8}$$

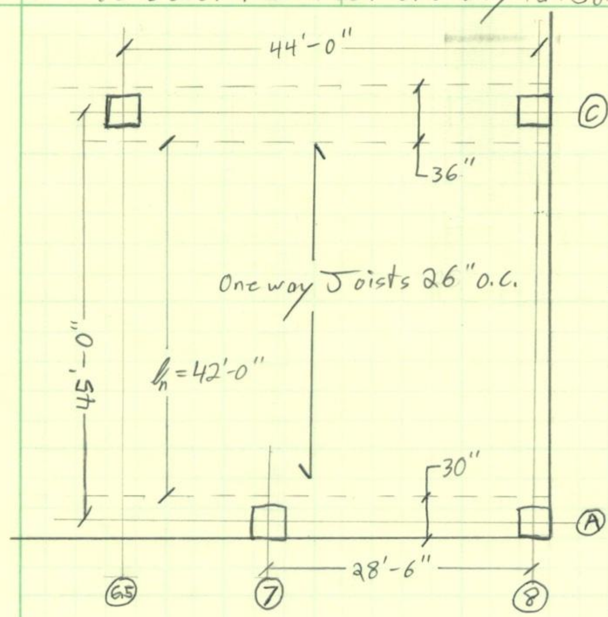
$$M_u = 217\,k\cdot ft \quad \phi M_p > M_u \Rightarrow \text{Good}$$

$$\boxed{\text{W21}\times\text{44 with 45 studs is adequate}}$$


D
C
A
6
6.5
7
8
W10x12 (31)
W24x55 (62) c=1-1/4"
W14x22 (26)
W30x99 (55) c=1"
W21x44 (28)
W21x44 (28)
W16x26 (37) c=3/4"
W16x31 (37) c=1-1/4"
W16x31 (37) c=1-1/4"
W16x31 (37) c=1-1/4"
W16x31 (37) c=1-1/4"
W16x31 (37) c=1-1/4"
W16x26 (37) c=3/4"
W18x35 (45) c=1-1/4"
W21x44 (45) c=1-1/4"
W21x44 (45) c=1-1/4"
W21x44 (45) c=1-1/4"
W21x44 (45) c=1-1/4"
W21x44 (45) c=1-1/4"
W18x35 (45) c=1-1/4"

APPENDIX: C

Office Level Alternative 1 One-Way Pan Joist James Chavanic 1



- Assume Column sizes come up at same size as floors below 24"x24" in this bay
- Keep grid spacing to satisfy arch. requirements
- Building is 204'-6" from (1) to (8) Try 20" pans with 6" rib width
- 2 Hr fire rating required → 4.5" slab min
  1" min. cover
- max rib depth = 3.5 · ribwidth = 21" ∴ use 20" deep pans (ACI 318-11 8.13.2)

$f'_c = 5000$ psi
$f_y = 60{,}000$ psi

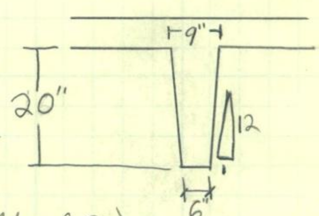
- 10% increase on V_c permitted (ACI 318-11 8.13.8)

Loads

Self-weight

$-\text{slab} = 150\,\text{lb}/\text{ft}^3 \cdot \frac{10''}{12''} = 125\,\text{psf}$

$-\text{joist} = \frac{150\,\text{lb}/\text{ft}^3 \cdot \left(\frac{7.5''}{12''}\right)\left(\frac{20''}{12''}\right)}{\left(26''/12''\right)} = 72\,\text{psf}$ over trib width



Deflection Criteria (ACI 318-11 Table 9.5(a))

$l/18.5 \Rightarrow 45' \cdot 12''/18.5 = h_{min} = 29.2''$ Use 30" Total depth ∴ need 10" thick slab

SIDL = 15 psf
MEP = 5 psf
SW = 125 psf + 72 psf = 197 psf

LL = 80 psf
$\sqrt{K_{LL} \cdot A_T} < 20$ ∴ No Live Load Reduction

OL Alternative #1 | one-way Pan Joist | James Chavanic | 2

Design 1' wide strip of slab

$$w_u = \frac{26''}{12''}\left[1.2(125+15+5)+1.6\cdot 80\right] = 0.655\ klf$$

Using ACI 318-11 8.3.3 moment coefficients

$$M_u^- = \frac{w_u l_n^2}{11} = \frac{0.655\cdot(20/12)^2}{11} = 0.166\ k\cdot ft/ft\ \text{of slab}$$

$$M_u^+ = \frac{w_u l_n^2}{14} = \frac{0.655\cdot(20/12)^2}{14} = 0.130\ k\cdot ft/ft\ \text{of slab}$$

↳ end condition

Check Minimum Reinforcement

(ACI 318-11 7.12.2.1) Shrinkage and Temperature $A_{s\,min} = 0.0018\cdot A_{gross}$

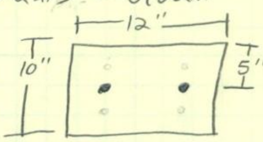
$A_{s\,min} = 0.0018\cdot 10''\cdot 12''$

$A_{s\,min} = 0.216\ in^2 \Rightarrow 2\#3 = 0.22\ in^2$

Max spacing:

(ACI 318-11 10.5.4) $S_{max} = 3t \leq 18''$ → controls

$3t = 3\cdot 10 = 30''$



Try 1#3 @ 6" which will still give 2#3 bars per 12" wide section

$$a = \frac{A_s\cdot f_y}{0.85 f'_c\cdot b} = \frac{0.22in^2\cdot 60000}{0.85\cdot 5000\cdot 12''}$$

$a = 0.259''$

$c = a/\beta_1$ $\quad \beta_1 = 0.85 - \frac{0.05}{1000}(5000-4000)$

$c = 0.259''/0.8$ $\quad \beta_1 = 0.80$

$c = 0.324''$

check $\varepsilon_s > \varepsilon_y$ assumption

$$\varepsilon_s = \frac{\varepsilon_u}{c}(d-c) = \frac{0.003}{0.324}(5-0.324) \qquad \varepsilon_s = 0.0433 > 0.005 \therefore \phi = 0.9$$

$\phi M_n = \phi A_s\cdot f_y\,(d - a/2)$

$= 0.22in^2\cdot 60ksi\left(5-\frac{0.259}{2}\right)\cdot 0.9$

$\phi M_n = 4.82\ k\cdot ft > 0.166\ k\cdot ft$ ✓
$> 0.130\ k\cdot ft$ ✓

→ use 10" deep slab with #3 @ 6" o.c.

Design Joist

$$w_u = \frac{26''}{12''}\left[1.2(197+5+15)+1.6\cdot 80\right] = 0.842\ k/f$$

using ACI 318-11 8.3.3 moment coefficients

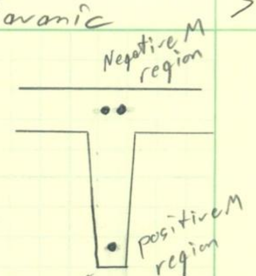
$$M_{u,int}^- = \frac{w_u l_n^2}{11} = \frac{0.842klf\cdot(42')^2}{11} = 135\ k\cdot ft$$

$$M_{u,ext}^- = \frac{w_u l_n^2}{24} = \frac{0.842klf\cdot(42')^2}{24} = 62\ k\cdot ft$$

$$M_u^+ = \frac{w_u l_n^2}{14} = \frac{0.842klf\cdot(42')^2}{14} = 106\ k\cdot ft$$

OL Alternative #1 | One way Pan Joist | James Chavanic | 3

Int. Negative Moment Reinforcement



$d = 30'' - 5'' = 25''$

$A_s = \frac{M_u}{4d} = \frac{135\,k\cdot ft}{4\cdot 25''} = 1.35\ in^2 \rightarrow$ Try 2#6+#7 $A_s = 1.48\,in^2$

$\rho = \frac{A_s}{bd} = \frac{1.48}{6\cdot 25''} = 0.0099 = 1\%$ $f'_c = 5000\,psi$ $> \therefore \frac{M_u}{4d}$ is conservative ✓

$a = \frac{A_s \cdot f_y}{0.85 f'_c \cdot b} = \frac{1.48\,in^2 \cdot 60{,}000\,psi}{0.85 \cdot 5000 \cdot 6''}$

$a = 3.48''$ $c = a/\beta_1 = 3.48/0.8$

$c = 4.35''$

$\varepsilon_{ts} = \frac{\varepsilon_u}{c}(d-c) = \frac{0.003}{4.35}(25-4.35)$

$\varepsilon_{ts} = 0.0142 > 0.004$ ✓ > 0.005 ✓ $\phi = 0.9$

$\phi M_n = \phi \cdot A_s \cdot f_y (d - a/2)$

$\phi M_n = 0.9 \cdot 1.48 \cdot 60{,}000 (25 - \frac{3.48}{2})$

$\phi M_n = 155\,k\cdot ft > 135\,k\cdot ft$ ✓ use 2#6 and 1#7

Ext. Negative Moment Reinforcement

$d = 30'' - 5'' = 25''$

$A_s = \frac{M_u}{4d} = \frac{62\,k\cdot ft}{4\cdot 25''} = 0.62\,in^2 \rightarrow$ Try 1#8 bar $A_s = 0.79\,in^2$ ✓

$\rho = \frac{A_s}{b\cdot d} = \frac{0.79}{6\cdot 25''} = 0.0053 = 0.4\%$ $f'_c = 5000\,psi$ $> \therefore \frac{M_u}{4d}$ is conservative ✓

$a = \frac{A_s \cdot f_y}{0.85 f'_c \cdot b} = \frac{0.79\cdot 60}{0.85\cdot 5\cdot 6''} = 1.86''$

$c = a/\beta_1 = 1.86/0.8 = 2.33''$

$\varepsilon_{ts} = \frac{\varepsilon_u}{c}(d-c) = \frac{0.003}{2.33}(25'' - 2.33'')$

$\varepsilon_{ts} = 0.029 > 0.004$ ✓ > 0.005 ✓ $\phi = 0.9$

$\phi M_n = \phi A_s f_y (d - a/2)$

$\phi M_n = 0.9 \cdot 0.79 \cdot 60 (25 - \frac{1.86}{2})$

$\phi M_n = 85.6\,k\cdot ft > 62\,k\cdot ft$ ✓

use 1#8 bar

Positive Moment Reinforcement

$d = 30'' - \underbrace{1.5''}_{cc} - \underbrace{0.5''}_{\#4\ stirrup} - \frac{1.00}{2} = 27.5''$

$A_s = \frac{M_u}{4d} = \frac{106}{4\cdot 27.5} = 0.964\,in^2 \rightarrow$ Try 1#9 bar $A_s = 1.00\,in^2$

Check T-beam behavior $M_u > M_u$ t-beam

$b_{eff} = \min \begin{cases} 6 + 16\cdot 10 = 166 \\ 6 + 2\cdot \frac{40}{2} = 26 \rightarrow \text{controls} \\ 1/4 \cdot 45 \cdot 12 = 135 \end{cases}$

M_u t-beam $= \phi\, 0.85 \cdot f'_c \cdot b \cdot h_f (d - h_f/2)$ (k·in)

$= 0.9 \cdot 0.85 \cdot 5000 \cdot 26 \cdot 10 (27.5 - 10/2)$

$= 1864\,k\cdot ft$ $\therefore$ no t-beam behavior

OL Alternative #1 | One-way Pan Joist | James Chavanic | 4

$\rho = \frac{A_s}{b_{eff} \cdot d_{slab} + b_w(d - d_{slab})} = \frac{1.00 in^2}{26 \cdot 10 + 6(27.5 - 10)} = 0.0027 = 0.27\% > \frac{M_u}{4d}$ is conservative, $f'_c = 5000 psi$

$a = \frac{1.00 \cdot 60}{0.85 \cdot 5 \cdot 26''} = 0.543''$

$\phi M_n = \phi A_s \cdot f_y (d - a/2)$

$\phi M_n = 0.9 \cdot 1.00 \cdot 60 (27.5 - \frac{0.543}{2})$

$c = a/\beta_1 = \frac{0.543}{0.8} = 0.679''$

$\phi M_n = 122.5 k \cdot ft > 106 k ft$ ✓

$\varepsilon_{ts} = \frac{\varepsilon_u}{c}(d - c) = \frac{0.003}{0.679}(27.5 - 0.679)$

use 1 #9 bar

$\varepsilon_{ts} = 0.1185 > 0.004$ ✓
> 0.005 ✓ $\phi = 0.9$

Shear Check

$V_u = \frac{w_u l_n}{2} = \frac{0.842 klf \cdot 42'}{2} = 17.7k$

$\phi V_c = 1.1 \cdot \phi \cdot 2 \cdot \sqrt{f'_c} \cdot b_w \cdot d$

↳ 10% increase per ACI 318-11 8.13.8

$\phi V_c = 1.1 \cdot 0.75 \cdot 2 \cdot \sqrt{5000} \cdot 6'' \cdot 27.5''$

$\phi V_c = 19.25k$ check $V_u < 0.5 \phi V_c = 9.63k$ ✗

need reinforcing steel

$V_s = V_u/\phi - V_c = 17.7k/0.75 - 25.7$

$V_s = -2.1$ → steel not required for strength, just as a safety measure.

$V_s = 0 \leq 8\sqrt{f'_c} b_w \cdot d$ ✓

$\leq 4\sqrt{f'_c} b_w \cdot d$ ✓ $s_{max} = \min \left| \begin{array}{l} d/2 = 27.5/2 = 13.75'' \\ 24'' \end{array} \right.$

$s_{max} = 13''$

$A_{v_{min}} = \max \left| \begin{array}{l} 0.75\sqrt{f'_c} b_w \cdot s/f_{yt} = 0.75 \cdot \sqrt{5000} \cdot 6'' \cdot 13/60000 = 0.069 in^2 \\ 50 b_w \cdot s/f_{yt} = 50 \cdot 6 \cdot 13/60{,}000 = 0.065 in^2 \end{array} \right.$

$A_{v_{min}} = 0.069 in^2 \Rightarrow$ use #3 (one leg) $A_v = 0.11 in^2$

use #3 @ 13" o.c.

refine to find when no longer needed for further economy

OL Alternative #1 One-way Pan Joist James Chavanic 5

Design Girder (Interior)

Interior girder @ 44'-0" span is worst case girder

Say Girder is same depth as joist/slab system and is 36" wide

self weight $= 150\,lb/ft^3 \cdot \frac{30}{12} \cdot \frac{36}{12} = 1125\,lb/ft$

Deflection criteria (ACI 318-11 Table 9.5(a))

$$h_{min} = \ell/18.5 = \frac{44' \cdot 12''}{18.5} = 28.5'' \therefore 30'' \text{ good} \checkmark$$

$$w_u = 1.2(197+5+15)\cdot\left(\frac{45+36.5}{2} - 3\right) + 1.2\cdot 1125 + 1.6\cdot 80\cdot 45$$

$$w_u = 16.94\,klf$$

Use frame analysis to determine design moments

M_u^- int $= -2682\,k\cdot ft$

M_u^- ext $= -1848\,k\cdot ft$

$M_u^+ = 1856\,k\cdot ft$

used uniform loading and pattern loading to determine worst case scenario

Interior negative Moment Reinforcement

$$d = 30'' - 1.5'' - 0.5'' - \frac{1.00''}{2} = 27.5''$$

$A_s = \frac{M_u}{4d} = \frac{2682}{4\cdot 27.5} = 24.4\,in^2 \Rightarrow$ Try 16 #11 bars $A_s = 24.96\,in^2$

Assuming 1" Aggregate size 16 bars > # allowed in single layer

$\therefore$ need 2 layers of bars $\Rightarrow$ reevaluate d

$$d = 30'' - 1.5'' - 0.5'' - (1.56'' + \underbrace{S_c}_{1.56''} + 1.56'')/2 = 25.66''$$

$A_s = \frac{M_u}{4d} = \frac{2682}{4\cdot 25.66} = 26.13\,in^2 \Rightarrow$ Try 18 #11 bars $A_s = 28.08\,in^2$

check max # of bars for a single layer in 36" wide beam

$$b - 2c_c - 2d_{tr} > n\,d_b + (n-1)S_c$$

$$36'' - 2\cdot 1.5'' - 2\cdot 0.5'' > n\cdot 1.56 + (n-1)1.56$$

$n < 10.76$ bars $\Rightarrow$ max allowed = 10 bars

4 bars < 9 bars/layer < 10 bars

↳ minimum to satisfy crack control

$\rho = \frac{A_s}{b\cdot d} = \frac{28.08\,in^2}{36''\cdot 25.66''} = 0.304 = 3.04\% \Rightarrow A_s = \frac{M_u}{4d}$ may not work ☆

Check $\rho_{max} = 0.85\beta_1 \cdot \frac{f'_c}{f_y} \cdot \frac{\varepsilon_u}{\varepsilon_u + 0.004} = 0.85\cdot 0.8\,\frac{5}{60}\cdot\frac{0.003}{0.003+0.004} = 0.0243 = 2.43\%$

OL Alternative #1 | One-way Pan Joist | James Chavanic | 6

Since $\rho_{max} = 2.43\%$ $\Rightarrow$ need to make beam wider to satisfy code.

Maintaining same d, find b to satisfy ρ_{max}

$$\rho_{max} = \frac{A_{s\,used}}{b_{req} \cdot d} \quad b_{req} = \frac{A_{s\,used}}{\rho_{max} \cdot d} = \frac{28.08}{0.0243 \cdot 25.66}$$

$b_{req} = 45.03''$ ∴ make beam 46" wide

46" wide beam will impact previous design but will only make it more conservative due to the slightly smaller span of the joists

Check beam

$$a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{28.08\,in^2 \cdot 60\,ksi}{0.85 \cdot 5\,ksi \cdot 46''}$$

$a = 8.618''$ $\quad c = a/\beta_1 = \frac{8.618}{0.8}$

$c = 10.77''$

$$\varepsilon_{ts} = \frac{\varepsilon_u}{c}(d_t - c) = \frac{0.003}{10.77}(25.66 - 10.77)$$

$\varepsilon_{ts} = 0.00415 > 0.004$ ✓

> 0.005 ✗ find ϕ

$$\phi = 0.65 + 0.25\,\frac{0.00415 - 0.00207}{0.005 - 0.00207}$$

$\phi = 0.827$

$\phi M_n = \phi A_s \cdot f_y (d - a/2)$

$\phi M_n = 0.827 \cdot 28.08 \cdot 60 (25.66 - \frac{8.618}{2})$

$\phi M_n = 2480\ k \cdot ft > 2682$ ✗

Beam No Good = Redesign

Try 54" wide beam $\Rightarrow$ increase in A_s allowed

$A_{s\,max} = \rho_{max} \cdot b \cdot d = 2.43\% \cdot 54'' \cdot 25.66$

$A_{s\,max} = 33.67\,in^2$ $\Rightarrow$ try 20 #11 bars

$A_s = 31.2\,in^2$

$$a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{31.2 \cdot 60}{0.85 \cdot 5 \cdot 54}$$

$a = 8.157''$ $\quad c = a/\beta_1 = \frac{8.157}{0.8}$

$c = 10.196$

$$\varepsilon_{ts} = \frac{0.003}{10.196}(25.66 - 10.196)$$

$\varepsilon_{ts} = 0.00455 > 0.004$ ✓

> 0.005 ✗ find ϕ

$$\phi = 0.65 + 0.25\,\frac{0.00455 - 0.00207}{(0.005 - 0.00207)}$$

$\phi = 0.8616$

$\phi M_n = 0.8616 \cdot 31.2 \cdot 60 (25.66 - \frac{8.157}{2})$

$\phi M_n = 2900\ K \cdot ft$

$2900\,k \cdot ft > 2726\,k \cdot ft$ ✓

use 20 #11 bars in 2 layers

Find adjusted M_u values for larger beam

self weight $= 150 \cdot \frac{54}{12} \cdot \frac{30}{12} = 1688$ lb/ft

$M_u^-\ int = -2726\ k \cdot ft$

$M_u^-\ ext = -1880\ k \cdot ft$

$M_u^+ = 1890\ k \cdot ft$

OL Alternative #1 | One-way Pan Joist | James Chavanic | 7

Since M_u^+ and M_u^- ext are nearly the same, I will just do one design that will cover both

Assume 2 layers of bars will be needed => $d \approx 25.66''$

$A_s = \frac{M_u}{4d} = \frac{1890}{4 \cdot 25.66} = 18.41\,in^2 \rightarrow$ Try 20 #9 bars $A_s = 20\,in^2$

max # of #11 bars allowed in single layer

$54'' - 2 \cdot 1.5'' - 2 \cdot 0.5'' > n \cdot 1.56 + (n-1)1.56$

$n < 16.5$ bars => max allowed = 16 #11

$\rho = A_s/bd = \frac{20\,in^2}{54'' \cdot 25.66''} = 0.0144 = 1.44\%$ > $\frac{M_u}{4d}$ should be close

$f'_c = 5000\,psi$

$a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{20 \cdot 60}{0.85 \cdot 5 \cdot 54}$

$a = 5.229''$ $c = a/\beta_1 = \frac{5.229}{0.8}$

$c = 6.536''$

$\varepsilon_{ts} = \frac{\varepsilon_u}{c}(d_t - c) = \frac{0.003}{6.536}(25.66 - 6.536)$

$\varepsilon_{ts} = 0.00878 > 0.004$ ✓

> 0.005 $\phi = 0.9$

$\phi M_n = \phi A_s \cdot f_y (d - a/2) = 0.9 \cdot 20 \cdot 60 (25.66 - \frac{5.229}{2})$

$\phi M_n = 2074\,k \cdot ft > 1890\,k \cdot ft$ ✓ use 20 #9 bars in 2 layers

Shear Check (worst case @ interior face)

$V_u = 400\,Kips$ => from same frame analysis used to find moments.

$\phi V_c = \phi \cdot 2 \cdot \sqrt{f'_c} \cdot b_w \cdot d$

$= 0.75 \cdot 2 \cdot \sqrt{5000} \cdot 54'' \cdot 25.66''$

$\phi V_c = 147\,K$

$V_u < 0.5 \phi V_c$ X ∴ need steel

$V_s = V_u/\phi - V_c = 400^k/0.75 - 196k$

$V_s = 338\,Kips$

check $V_s \leq 8\sqrt{f'_c}\, b_w \cdot d$

$\leq 784k$ ✓

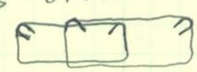
$\leq 4\sqrt{f'_c}\, b_w \cdot d = 392k$ ✓

∴ $s_{max} = \min \begin{cases} d/2 = 25.66/2 = 12.83'' \\ 24'' \end{cases}$

$s_{max} = 12''$

$Av_{min} = \max \begin{cases} 0.75\sqrt{f'_c} \cdot \frac{b_w \cdot s}{f_{yt}} = 0.5728\,in^2 \text{ (controls)} \\ 50 \frac{b_w \cdot s}{f_{yt}} = 0.54\,in^2 \end{cases}$

→ 3 #4 => $A_s = 0.6\,in^2$

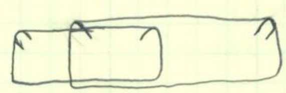
Try 4# in  $A_s = 0.8\,in^2$

$s = A_v \cdot f_{yt} \cdot d / V_s$

$s = 0.8 \cdot 60 \cdot 25.66'' / 338k$

$s = 3.64''$

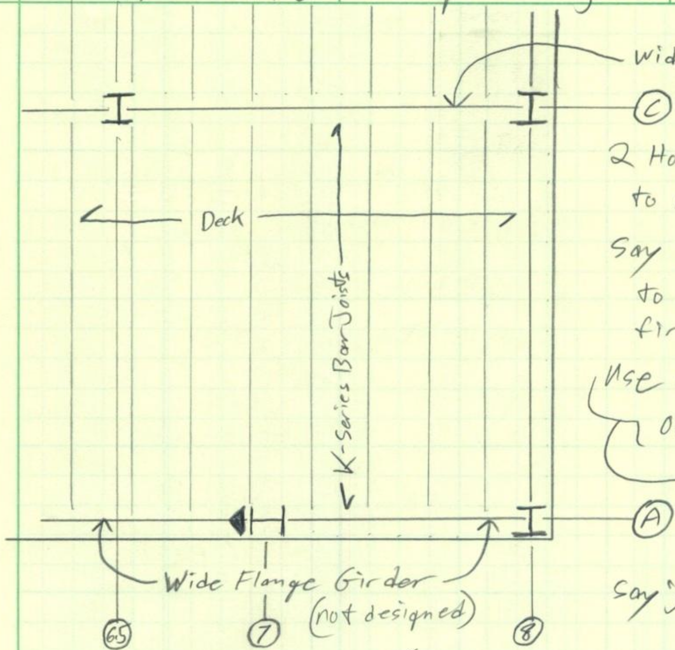
∴ use 3''

use  #4 @ 3'' O.C.

refine to find when no longer needed for further economy

APPENDIX: D

OL Alternative #2 | Non-Comp. on bar joists | James Chavanic | 1



2 Hour fire rating needed to match existing system

Say Gypsum Board Protection to avoid spray fiber fire proofing on joists (messy)

Use 2½ NWC

On 1.5C Conform deck

Used Vulcraft deck catalog to determine this

Say joists will be 4'-0" O.C.

Say 24 GA. Deck (1.5C24)

Max. Construction clear spans

1 span = 5'-1" > 4' ✓

2 span = 6'-9" > 4' ✓

3 span = 6'-10" > 4' ✓

w = 43 psf for concrete weight

Loads

Deck = 1.44 psf say 2 psf

Concrete = 43 psf

SIDL = 15 psf

MEP = 5 psf

LL = 80 psf

Total = 145 psf $F_b = 36{,}000$

LL = 80 psf Defl = $\ell/240$

Allowable Uniform Load

1.5C24 1 span @ 4'-0" 198 psf (total load) > 145 psf ✓

check deflection criteria

1.5C24 1 span @ 4'-0" 140 psf (Defl = $\ell/240$) > 80 psf LL

2 and 3 span conditions have even higher capacities

∴ 1.5C24 deck is good

Use 1.5C24 deck with 2½" NWC topping ($d_{total} = 4.0"$)

with WWF 6×6 – W2.9×W2.9 to meet

ACI criteria for temp. and shrinkage

OL Alternative #2 Non-Comp. on bar joist James Chavanic 2

Size Joist

Trib width = 4'-0"

Loads Deck = 2psf
concrete = 43psf
SIDL = 15psf
MEP = 5psf
Joist selfwt + GWB+Insulation = 5psf

LL = 80psf

DL = 70psf

$w_{LL} = 80 \cdot 4ft$
$w_{LL} = 320\ lb/ft$

$w_u = 1.2DL + 1.6LL$
$w_u = 212psf \cdot 4ft$
$w_u = 848\ lb/ft$ → no k-series joist

Options: 1. Use LH series joist
2. Reduce joist spacing to 2'-0" o.c.

Option 1
$w_{TL} = 848\ lb/ft$
$w_{LL} = 320\ lb/ft$
using SJI catalog
28LH09 wt = 21lbs/ft < 879 TL, 351 LL
Still good once corrected for joist self wt.
☆ choose based on cost comparison ☆

Option 2
$w_{uTL} = 212psf \cdot 2ft = 424\ lb/ft$
$w_{LL} = 80 \cdot 2ft = 160\ lb/ft$
using SJI Catalog Economy Tables
~~30K8~~ wt = 13.2 lbs/ft < 436 TL, 192 LL
✓26K10 wt = 13.8 lbs/ft < 486 TL, 182 LL
✓24K12 wt = 16 lbs/ft < 580 TL, 199 LL
Adjust for actual selfweight

Size Girder (check for both options)

Option 1
Idealize as distributed load
l_b = joist spacing = 4'-0"
DL = 75psf → adjusted for original low allowance of Joist selfwt.
LL = 80psf
$w_u = [1.2 \cdot 75 + 1.6 \cdot 80]\left(\frac{45+36.5}{2}\right)$
$w_u = 8.89\ k/ft$
$M_u = \frac{w_u l^2}{8} = \frac{8.89 \cdot 44^2}{8} = 2150\ k \cdot ft$
W30×173 $\phi M_n = 2280\ k \cdot ft$
Deflection check $w_{LL} = 3.26\ k/ft$
$\Delta_L = \frac{5 w_L l^4}{384EI} = \frac{5 \cdot 3.26 \cdot (44ft)^4 \cdot (12in/ft)^3}{384 \cdot 29{,}000 \cdot 8230 in^4}$
$\Delta_{TL} = \frac{5 w_{TL} l^4}{384EI} = \frac{5 \cdot 6.316 \cdot 44^4 \cdot 12^3}{384 \cdot 29{,}000 \cdot 8230 in^4} = 2.23"$ ∴ camber girder 3/4"
use W30×173 with 3/4" Camber

Option 2
Idealize as distributed load
l_b = joist spacing = 2'-0"
DL = 75psf LL = 80psf
$w_u = 8.89\ k/ft$
$M_u = 2150\ k \cdot ft$
W30×173 $\phi M_n = 2280\ k \cdot ft$
Deflection check
$\Delta_{max} = l/240 = 44 \cdot 12/240 = 2.2"$
More Stringent $\Delta_{max} = l/360 = \frac{44 \cdot 12}{360} = 1.47"$
$\Delta = 1.152" < 1.47"$ ✓

APPENDIX: E

Note: All tables and figures in this appendix were obtained from RSMeans CostWorks website.

Assembly B10102506150 **Based on National Average Costs**

Floor, concrete, slab form, open web bar joist @ 2' OC, on W beam and column, 25'x30' bay, 29" deep, 100 PSF superimposed load, 145 PSF total load

Description	Quantity	Unit	Material	Installation	Total
Welded wire fabric, sheets, 6 x 6 - W1.4 x W1.4 (10 x 10) 121 lb. per C.S.F., A185, incl...	0.01000	C.S.F.	0.15	0.36	0.51
Structural concrete, ready mix, normal weight, 3000 psi, includes local aggregate, san...	0.21000	C.F.	0.87	0.00	0.87
Structural concrete, placing, elevated slab, pumped, less than 6" thick, includes strike...	0.21000	C.F.	0.00	0.32	0.32
Concrete finishing, floors, for specified Random Access Floors in ACI Classes 1, 2, 3 an...	1.00000	S.F.	0.00	0.86	0.86
Concrete surface treatment, curing, sprayed membrane compound	0.01000	C.S.F.	0.08	0.09	0.17
Structural steel project, apartment, nursing home, etc, 100-ton project, 1 to 2 stories,...	4.36800	Lb.	6.03	1.83	7.86
Open web bar joist, K Series, 40-ton job lots, 30' to 50' spans, shop fabricated, incl sh...	5.70000	Lb.	4.73	1.48	6.21
Metal decking, steel, slab form, galvanized, 9/16" D, 28 gauge, type UFS	1.02000	S.F.	1.32	0.75	2.07
Total			**$13.20**	**$5.69**	**$18.89**

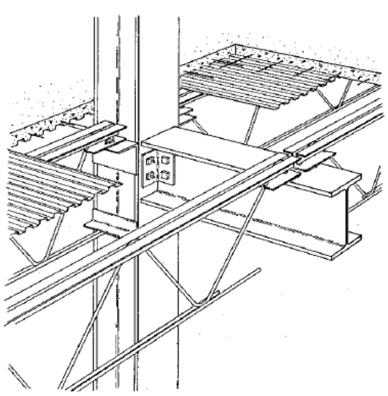


Table below lists costs for a floor system on steel columns and beams using open web steel joists, galvanized steel slab form, and 2-1/29 concrete slab reinforced with welded wire fabric.

Design and Pricing Assumptions:

- Structural Steel is A36.
- Concrete f'c = 3 KSI placed by pump.
- WWF 6 x 6 – W1.4 x W1.4 (10 x 10)
- Columns are 128 high.
- Building is 4 bays long by 4 bays wide.
- Joists are 28 O.C. 6 and span the long direction of the bay.

Joists at columns have bottom chords extended and are connected to columns.

Slab form is 28 gauge galvanized. Column costs in table are for columns to support 1 floor plus roof loading in a 2-story building; however, column costs are from ground floor to 2nd floor only. Joist costs include appropriate bridging. Deflection is limited to 1/360 of the span. Screeds and steel trowel finish.

Design Loads	Min.	Max.
S.S. & Joists	6.3 PSF	15.3 PSF
Slab Form	1.0	1.0
2-1/29 Concrete	27.0	27.0
Ceiling	3.0	3.0
Misc.	5.7	1.7
	43.0 PSF	48.0 PSF

Cost Estimate Report
CostWorks
RSMeans

Unit Detail Report

Year 2012

Date: 11-Oct-12

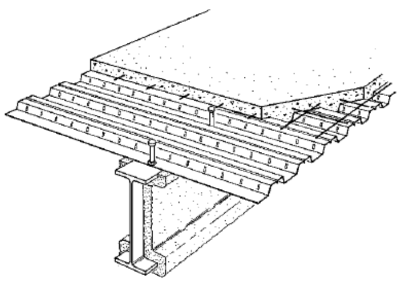
Prepared By:
James Chavanic
Penn State University

Line Number			Description	Quantity	Unit	Total Incl. O&P	Ext. Total Incl. O&P
Division 05 Metals							
052116502320			Longspan joist, LH Series, 40-ton job lots, 28LH06, 16 plf, spans to 96', shop fabricated, incl shop primer, bolted cross bridging	1	L.F.	$19.73	$19.73
052116502340			Longspan joist, LH Series, 40-ton job lots, 28LH11, 25 plf, spans to 96', shop fabricated, incl shop primer, bolted cross bridging	1	L.F.	$28.33	$28.33
052119100640			Open web bar joist, K Series, 40-ton job lots, 26K10, 13.8 plf, 30' to 50' spans, shop fabricated, incl shop primer, horizontal bridging	1	L.F.	$15.35	$15.35
Division 05 Metals Subtotal							$63.41

Assembly B10102568000 **Based on National Average Costs**

Floor, composite metal deck, shear connectors, 5.5" slab, 35'x40' bay, 29.5" total depth, 125 PSF superimposed load, 171 PSF total load

Description	Quantity	Unit	Material	Installation	Total
Welded wire fabric, sheets, 6 x 6 - W1.4 x W1.4 (10 x 10) 121 lb. per C.S.F., A185, incl...	0.01100	C.S.F.	0.17	0.40	0.56
Structural concrete, placing, elevated slab, pumped, less than 6" thick, includes strike...	0.33300	C.F.	0.00	0.51	0.51
Structural concrete, ready mix, lightweight, 110 #/C.F., 3000 psi, includes local aggre...	0.33300	C.F.	2.41	0.00	2.41
Concrete finishing, floors, for specified Random Access Floors in ACI Classes 1, 2, 3 an...	1.00000	S.F.	0.00	0.86	0.86
Concrete surface treatment, curing, sprayed membrane compound	0.01000	C.S.F.	0.08	0.09	0.17
Weld shear connector, 3/4" dia x 4-7/8" L	0.15300	Ea.	0.11	0.31	0.42
Structural steel project, apartment, nursing home, etc, 100-ton project, 3 to 6 stories,...	8.34000	Lb.	11.68	3.59	15.26
Metal floor decking, steel, non-cellular, composite, galvanized, 3" D, 18 gauge	1.05000	S.F.	2.88	1.10	3.98
Metal decking, steel edge closure form, galvanized, with 2 bends, 12" wide, 18 gauge	0.02700	L.F.	0.11	0.07	0.17
Sprayed fireproofing, cementitious, normal density, beams, 1 hour rated, 1-3/8" thick...	0.65400	S.F.	0.38	0.65	1.03
Total			**$17.80**	**$7.58**	**$25.38**



Description: Table below lists costs ($/S.F.) for a floor system using composite steel beams with welded shear studs, composite steel deck, and light weight concrete slab reinforced with W.W.F. Price includes sprayed fiber fireproofing on steel beams.

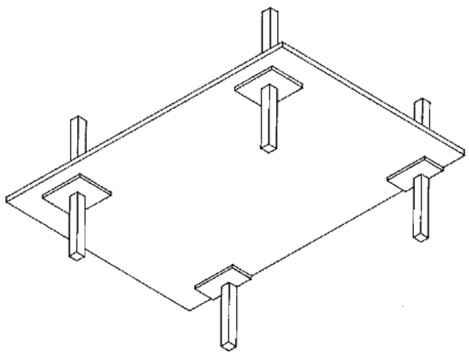
Design and Pricing Assumptions:
- Structural steel is A36, high strength bolted.
- Composite steel deck varies from 22 gauge to 16 gauge, galvanized.
- Shear Studs are 3/4".
- W.W.F., 6 x 6 – W1.4 x W1.4 (10 x 10)
- Concrete f'c = 3 KSI, lightweight.
- Steel trowel finish and cure.
- Fireproofing is sprayed fiber (non-asbestos).

Spandrels are assumed the same as interior beams and girders to allow for exterior wall loads and bracing or moment connections.

Assembly B10102224000 **Based on National Average Costs**

Flat slab, concrete, with drop panels, 8.5" slab/8.5" panel, 20" column, 25'x25' bay, 125 PSF superimposed load, 243 PSF total load

Description	Quantity	Unit	Material	Installation	Total
C.I.P. concrete forms, beams and girders, exterior spandrel, plywood, 12" wide, 4 use...	0.03400	SFCA	0.03	0.35	0.38
C.I.P. concrete forms, elevated slab, flat slab with drop panels, to 15' high, 4 use, incl...	0.99400	S.F.	1.27	5.81	7.09
Reinforcing Steel, in place, elevated slabs, #4 to #7, A615, grade 60, incl labor for acc...	3.87600	Lb.	2.17	1.67	3.84
Structural concrete, ready mix, normal weight, 3000 psi, includes local aggregate, san...	0.78700	C.F.	3.27	0.00	3.27
Structural concrete, placing, elevated slab, pumped, 6" to 10" thick, includes strike of...	0.78700	C.F.	0.00	1.02	1.02
Concrete finishing, floors, for specified Random Access Floors in ACI Classes 1, 2, 3 an...	1.00000	S.F.	0.00	0.86	0.86
Concrete surface treatment, curing, sprayed membrane compound	0.01000	C.S.F.	0.08	0.09	0.17
Total			**$6.80**	**$9.80**	**$16.60**



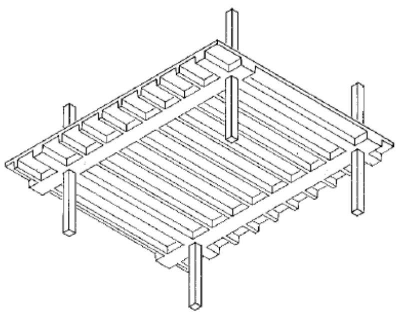
General: Flat Slab: Solid uniform depth concrete two-way slabs with drop panels at columns and no column capitals.

Design and Pricing Assumptions:
- Concrete f'c = 3 KSI, placed by concrete pump.
- Reinforcement, fy = 60 KSI.
- Forms, four use.
- Finish, steel trowel.
- Curing, spray on membrane.
- Based on 4 bay x 4 bay structure.

Assembly B10102269800 **Based on National Average Costs**

Joist slab, cast-in-place concrete, multi-span, 20" deep rib, 28" column, 40'x45' bay, 125 PSF superimposed load, 234 PSF total load

Description	Quantity	Unit	Material	Installation	Total
C.I.P. concrete forms, beams and girders, exterior spandrel, plywood, 12" wide, 4 use...	0.16700	SFCA	0.15	1.71	1.86
C.I.P. concrete forms, beams and girders, interior, plywood, 12" wide, 4 use, includes...	0.13900	SFCA	0.15	1.17	1.32
C.I.P. concrete forms, elevated slab, floor, with 1-way joist pans, 4 use, includes shori...	0.92300	S.F.	2.96	5.86	8.82
C.I.P. concrete forms, elevated slab, edge forms, alternate pricing, to 6" high, 1 use, i...	0.01000	SFCA	0.01	0.06	0.07
Reinforcing Steel, in place, elevated slabs, #4 to #7, A615, grade 60, incl labor for acc...	2.88000	Lb.	1.61	1.24	2.85
Structural concrete, ready mix, normal weight, 4000 PSI, includes local aggregate, sa...	0.91000	C.F.	3.80	0.00	3.80
Structural concrete, placing, elevated slab, pumped, over 10" thick, includes strike off...	0.91000	C.F.	0.00	1.23	1.23
Concrete finishing, floors, for specified Random Access Floors in ACI Classes 1, 2, 3 an...	1.00000	S.F.	0.00	0.86	0.86
Concrete surface treatment, curing, sprayed membrane compound	0.01000	C.S.F.	0.08	0.09	0.17
Total			$8.75	$12.22	$20.97



General: Combination of thin concrete slab and monolithic ribs at uniform spacing to reduce dead weight and increase rigidity. The ribs (or joists) are arranged parallel in one direction between supports.

Square end joists simplify forming. Tapered ends can increase span or provide for heavy load.

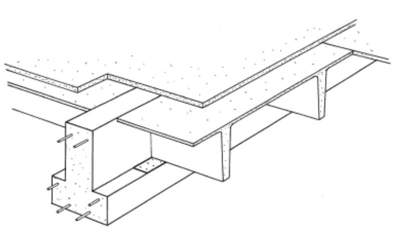
Costs for multiple span joists are provided in this section. Single span joist costs are not provided here.

Design and Pricing Assumptions:
- Concrete f'c = 4 KSI, normal weight placed by concrete pump.
- Reinforcement, fy = 60 KSI.
- Forms, four use.
  - 4-1/29 slab.
  - 309 pans, sq. ends (except for shear req.).
  - 69 rib thickness.
  - Distribution ribs as required.
- Finish, steel trowel.
- Curing, spray on membrane.
- Based on 4 bay x 4 bay structure.

Assembly B10102393100 **Based on National Average Costs**

Precast double T, 2" topping, on precast beams, 38" deep, 25'x30' bay, 75 PSF superimposed load, 168 PSF total load

Description	Quantity	Unit	Material	Installation	Total
C.I.P. concrete forms, elevated slab, bulkhead with keyway, 2 piece, 1 use, includes s...	0.01000	L.F.	0.02	0.06	0.08
C.I.P. concrete forms, elevated slab, edge forms, to 6" high, 4 use, includes shoring, e...	0.03700	L.F.	0.01	0.15	0.16
Welded wire fabric, sheets, 6 x 6 - W1.4 x W1.4 (10 x 10) 121 lb. per C.S.F., A185, incl...	0.01000	C.S.F.	0.15	0.36	0.51
Structural concrete, ready mix, normal weight, 3000 psi, includes local aggregate, san...	0.17000	C.F.	0.71	0.00	0.71
Structural concrete, placing, elevated slab, pumped, less than 6" thick, includes strike...	0.17000	C.F.	0.00	0.26	0.26
Concrete finishing, floors, for specified Random Access Floors in ACI Classes 1, 2, 3 an...	1.00000	S.F.	0.00	0.86	0.86
Concrete surface treatment, curing, sprayed membrane compound	0.01000	C.S.F.	0.08	0.09	0.17
Precast concrete beam, 5000 psi, T-shaped, 25' span, 12" x 36"	0.03100	L.F.	6.32	0.45	6.78
Precast concrete beam, 5000 psi, L-shaped, 25' span, 12" x 28"	0.02100	L.F.	2.86	0.27	3.13
Precast tees, double, roof, 30' span, 16" x 8' wide, prestressed	0.00412	Ea.	8.96	1.65	10.62
Total			$19.10	$4.15	$23.25



General: Beams and double tees priced here are for plant produced prestressed members transported to the site and erected.

The 2" structural topping is applied after the beams and double tees are in place and is reinforced with W.W.F.

Design and Pricing Assumptions:
Prices are based on 10,000 S.F. to 20,000 S.F. projects and 50 mile to 100 mile transport.

Concrete for prestressed members is f'c 5 KSI.

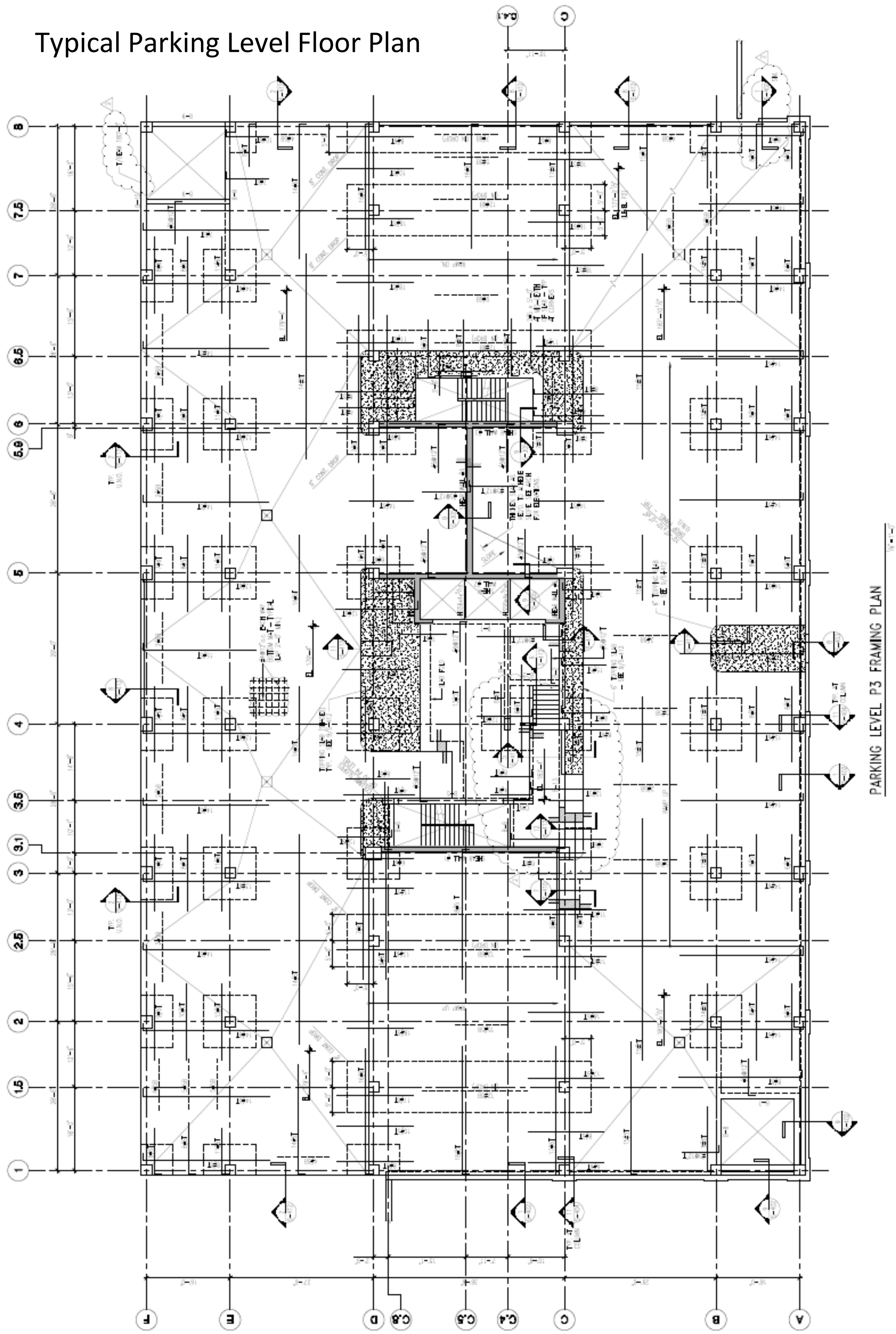
Concrete for topping is f'c 3000 PSI and placed by pump.

Prestressing steel is fy = 250 or 300 KSI.

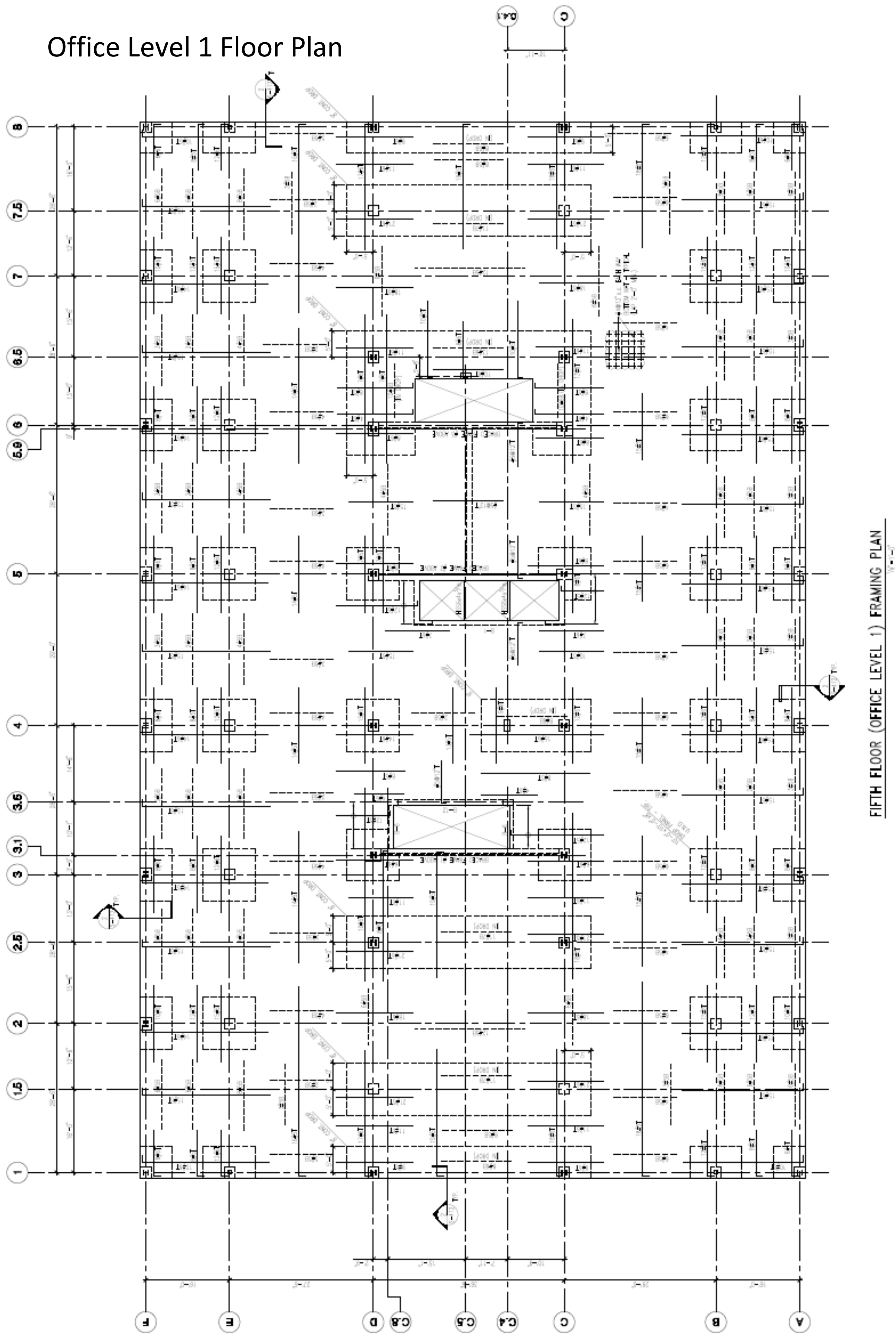
W.W.F. is 6 x 6 – W1.4 x W1.4 (10x10).

APPENDIX: F

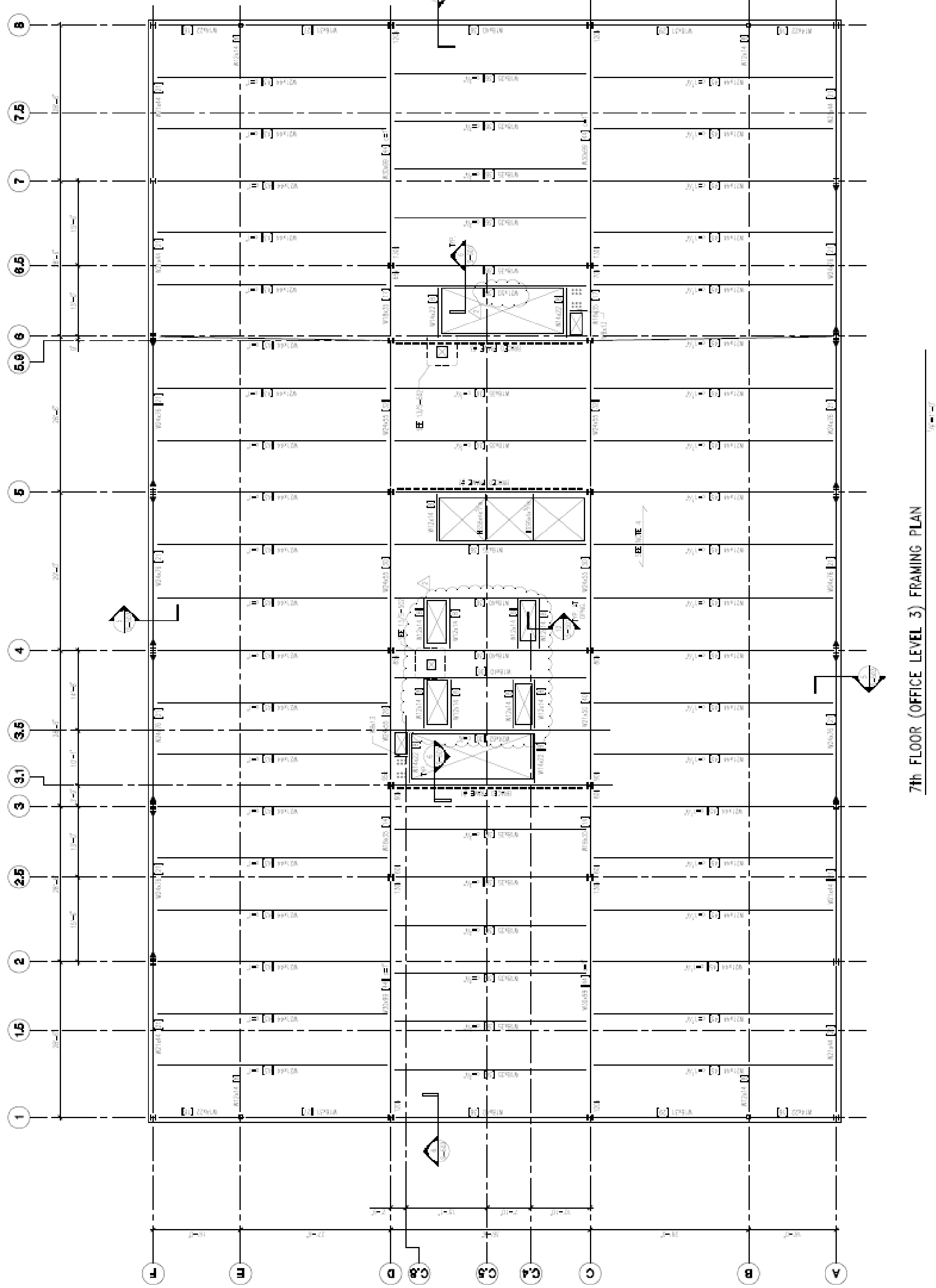
Typical Parking Level Floor Plan



Office Level 1 Floor Plan



Typical Office Level Floor Plan



APPENDIX: G

ONLINE CERTIFICATIONS DIRECTORY

Design No. G559
BXUV.G559
Fire Resistance Ratings - ANSI/UL 263

Page Bottom

Design/System/Construction/Assembly Usage Disclaimer

- Authorities Having Jurisdiction should be consulted in all cases as to the particular requirements covering the installation and use of UL Listed or Classified products, equipment, system, devices, and materials.
- Authorities Having Jurisdiction should be consulted before construction.
- Fire resistance assemblies and products are developed by the design submitter and have been investigated by UL for compliance with applicable requirements. The published information cannot always address every construction nuance encountered in the field.
- When field issues arise, it is recommended the first contact for assistance be the technical service staff provided by the product manufacturer noted for the design. Users of fire resistance assemblies are advised to consult the general Guide Information for each product category and each group of assemblies. The Guide Information includes specifics concerning alternate materials and alternate methods of construction.
- Only products which bear UL's Mark are considered as Classified, Listed, or Recognized.

Fire Resistance Ratings - ANSI/UL 263

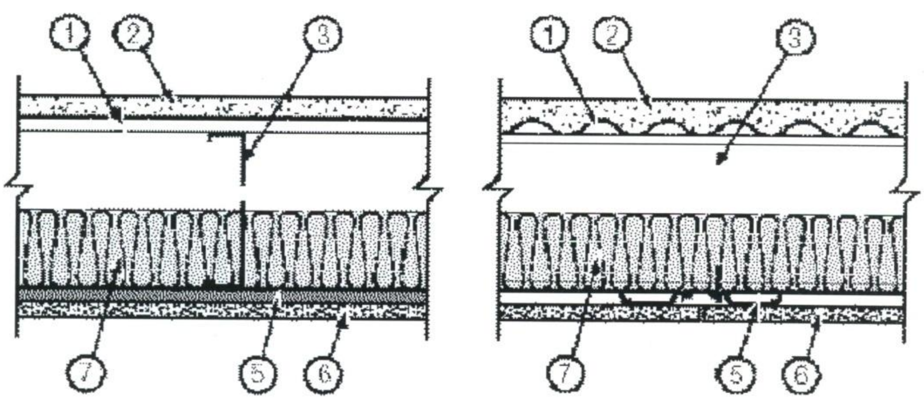
See General Information for Fire Resistance Ratings - ANSI/UL 263

Design No. G559

March 19, 2012

Unrestrained Assembly Rating - 2 Hr.

Load Restricted for Canadian Applications — See Guide BXUV7



1. **Steel Deck** — Min 9/16 in. deep, 22 MSG galv corrugated fluted steel deck. Attached to each joist with #10 3/4 in. long screws at each side joint and no more than 12 in. OC between sides.

2. **Floor Topping Mixture*** — Compressive strength to be 3500 psi min. Minimum thickness to be 1 in. as measured from the top plane of the deck. Refer to manufacturer's instructions accompanying the material for specific mix design. An ethylene vinyl acetate adhesive may be applied to the steel deck prior to the installation of the floor topping mixture at a maximum application rate of 0.025 lbs./ft^2.

UNITED STATES GYPSUM CO — LEVELROCK® Brand ™ CSD or LEVELROCK™ CSD RH

2A. **Floor Topping Mixture*** — (As an alternate to Item 2, not shown) — Various types of insulating concrete prepared and applied in the thickness indicated below:

A. **Vermiculite Concrete** — 6 cu ft of Vermiculite Aggregate* to 94 lbs. of Portland cement and 0.5 lb. of air entraining agent. Min 2-1/2 in. thickness above top plane of steel deck when no foamed plastic insulation boards (Item 9) are used. When foamed plastic insulation boards are used, min thickness above foamed plastic is 2 in. and min thickness between the top plane of the steel deck and the foamed plastic is 1/8 in. The max vermiculite concrete thickness shall be determined by job site conditions.

SIPLAST INC

THE STRONG CO INC

VERMICULITE PRODUCTS INC

B. **Cellular Concrete — Roof Topping Mixture*** — Foam concentrate mixed with water and Portland cement per manufacturer's specifications. Cast dry density and 28-day compressive strength of min 190 psi as determined in accordance with ASTM C495-66. Thickness of cellular concrete topping to be 2-3/4 in. min above top plane of steel deck when no foamed plastic insulation boards (Item 9A) are used. When foamed plastic is used, a 1/8 in. min slurry coat of cellular concrete, as measured to the top of the steel form unit corrugations, shall be employed. The cellular concrete topping thickness above foamed plastic, shall be 2 in. min.

CELCORE INC — Type Celcore with cast dry density of 31 (+ or - 3.0) pcf or Type Celcore MF with cast dry density of 29 (+ or - 3.0) pcf.

CELLULAR CONCRETE L L C — Cast dry density 37 (+ or -) 3.0 pcf.

ELASTIZELL CORP OF AMERICA — Type II. Mix #1 of cast dry density 39 (+ or -) 3.0 pcf, Mix #2 of cast dry density 40 (+ or -) 3.0 pcf, Mix #3 of cast dry density 47 (+ or -) 3.0 pcf.

LITE-CRETE INC — Cast dry density of 29 (+ or -) 3.0 pcf.

C. **Perlite Concrete** — Mix consists of 6 cu ft of Perlite Aggregate* to 94 lb of Portland cement and 1-1/2 pints of air entraining agent. Min 2-1/2 in. thickness above top plane of steel deck when no foamed plastic insulation boards (Item 9B) are used. When foamed plastic boards are used, min thickness above foamed plastic is 2 in. and min thickness between the top plane of the steel deck and the foamed plastic is 1/8 in.

See **Perlite Aggregate** (CFFX) category in Fire Resistance Directory for names of Classified companies.

D. **Cellular Concrete — Roof Topping Mixture*** — Foam Concentrate mixed with water, Portland Cement and UL Classified Vermiculite Aggregate per manufacturer's application instructions. Cast dry density of 33 (+ or -) 3.0 pcf and 28-day compressive strength of min 250 psi as determined in accordance with ASTM C495-86. A 1/8 in. min slurry coat shall be employed below the foamed plastic (Item 9A or 9B). The cellular concrete topping thickness, above the foamed plastic, shall be 2 in. min.

CELLULAR CONCRETE L L C — Mix #3.

SIPLAST INC — Mix #3.

2B. **Lightweight Concrete** — (As an alternate to Items 2 and 2A, not shown) -- Lightweight concrete, expanded shale or slate aggregate by rotary-kiln method or expanded clay aggregate by rotary-kiln or sintered-grate method, 107 - 113 pcf unit weight, 3000 psi compressive strength, vibrated, 4 to 7 per cent entrained air. Min. thickness as measured from the top plane of the steel deck, 2-1/2 in.

3. **Structural Steel Members*** — The proprietary joists are channel-shaped, 9-1/4 in. min depth. Joists are fabricated from min No. 16 MSG galv steel. Joists spaced max 24 in. OC. Joists attached to rim joist with three #10 3/4 in. long self-drilling screws at the rim track clip to the outside of the web joist, and a #10 1/2 in. long screw through the top and bottom flange of the joists to the top and bottom flange of the rim track. At rim joist splices bearing on supports, rim joists are connected using an overlapping section of a 12 in. long splice plate (a joist piece), with six 3/4 in. long self-drilling #10 screws to each rim piece.

CALIFORNIA EXPANDED METAL PRODUCTS CO — Type SSCJ floor joists, SSRT rim joists

4. **Joist Bridging —** Not shown — Installed immediately after joists are erected and before construction loads are applied. The structural bridging, Type CEMCO Sure Bridging, consisting of No. 18 MSG galv steel, 2-1/2 in. wide by 25-1/2 in. long with 1-5/16 in. long legs structural bridging staggered between the steel joists and attached to the bottom joist flange with two #10 1/2 in. long self-drilling screws at each end tab of bridging. Solid bridging consisting of cut to length joist sections placed between outer joists and at center joist with 8 ft OC max spacing. Solid bridging is seated in the structural bridging and is screw-attached at joist web using Type CEMCO Sure-Support Clips (1-1/2 in. by 1-1/2 in. by 7 in. long, 16 MSG, min 50 ksi support clip) with three #10 3/4 in. long self-drilling screws per leg on one side and the other side with Type CEMCO Sure-Support Clips (4 in. by 1-1/2 in. by 7 in. long, 16 MSG, min 50 ksi support clip) with three #10 3/4 in. long self-drilling screws per leg.

5. **Resilient Channels —** 1/2 in. deep, formed of 25 MSG galv steel, spaced 12 in. OC perpendicular to joists. Channel splices overlapped 4 in. beneath steel joists. Channels secured to each joist with 1/2 in. Type S-12 low profile screws. Channels oriented opposite at wallboard butt joints (spaced 5-1/2 in. OC) as shown in the above illustration.

5A. **Alternate Steel Framing Members —** (Not Shown) - As an alternate to Item 5, main runners, cross tees, cross channels and wall angle as listed below:

a. **Main Runners —** Nom 10 or 12 ft long, 15/16 in. or 1-1/2 in. wide face, spaced 4 ft. OC. Main runners suspended by min 12 SWG galv steel hanger wires spaced 48 in. OC. Hanger wires to be located adjacent to main runner/cross tee intersections. Hanger wires inserted through holes drilled through web of joists and twist-tied.

b. **Cross Tees —** Nom 4 ft long, 1-1/2 in. wide face, installed perpendicular to the main runners, spaced 16 in. OC. Additional cross tees or cross channels used at 8 in. from each side of butted gypsum panel end joints. The cross tees or cross channels may be riveted or screw attached to the wall angle or channel to facilitate the ceiling installation.

c. **Cross Channels —** Nom 4 or 12 ft long, installed perpendicular to main runners, spaced 16 in. OC.

d. **Wall Angle or Channel —** Painted or galv steel angle with 1 in. legs or channel with 1 in. legs, 1-9/16 in. deep attached to walls at perimeter of ceiling with fasteners 16 in. OC. To support steel framing member ends and for screw-attachment of the gypsum panel.

CGC INC — Type DGL or RX.

USG INTERIORS LLC — Type DGL or RX.

6. **Gypsum Board* —** Nom 5/8 in. thick, 48 in. wide gypsum panels. When resilient channels (Item 5) are used, gypsum panels installed with long dimension perpendicular to resilient channels. Gypsum panels secured with 1 in. long Type S bugle-head screws spaced 8 in. OC in both the field and the perimeter, and 1-1/2 in. from side edges of the board. When Steel Framing Members (Item 5A) are used, gypsum panels installed with long dimension perpendicular to cross tees with side joints centered along main runners and end joints centered along cross tees. Panels fastened to cross tees with 1 in. long Type S bugle-head screws spaced 8 in. OC in the field and along end joints. Panels fastened to main runners with 1 in. long Type S bugle-head screws spaced midway between cross tees. Screws along sides and ends of panels spaced 3/8 to 1/2 in. from panel edge. End joints of panels shall be staggered with spacing between joints on adjacent panels not less than 2 ft OC.

CGC INC — Types C, IP-X2, IPC-AR

UNITED STATES GYPSUM CO — Types C, IP-X2, IPC-AR

USG MEXICO S A DE C V — Types C, IP-X2, IPC-AR

7. **Batts and Blankets* —** Glass fiber insulation, min 3-1/2 in. thick, bearing the UL Classification Marking for Surface Burning Characteristics and/or Fire Resistance. Insulation fitted in the concealed space, draped over the resilient channel/gypsum panel ceiling membrane. See **Batts and Blankets** (BKNV or BZJZ) Categories for names of Classified companies.

8. **Joint System —** Not Shown — Vinyl, dry or premixed joint compound, applied in two coats to joints and screw heads; paper tape, 2 in. wide, embedded in first layer of compound over all joints.

9. **Foamed Plastic* —** Optional — For vermiculite concrete applications — Foamed plastic insulation boards with holes and/or slots. Nom 24 by 48 in. size. Thickness 1 in. to max 8 in.

VERMICULITE PRODUCTS INC

9A. **Foamed Plastic*** — Nom 24 by 48 in., 48 by 48 in. or 30 by 60 in. by max 8 in. thick polystyrene foamed plastic insulation boards with holes symmetrically placed having a max density of 2.0 pcf. For use only with cellular concrete roof topping mixture.

STARRFOAM MFG INC

9B. **Foamed Plastic*** — Nominal 24 by 48 by max 8 in. thick polystyrene foamed plastic insulation boards having a density of 1.0 + 0.1 pcf encapsulated within cellular or perlite concrete topping (Item 3B or 3C). Each insulation board shall contain six nom 3 in. diameter holes oriented in two rows of three holes each with the holes oriented in two rows of three holes each with the holes spaced 12 in. OC, transversely and 16 in. OC longitudinally.

See **Foamed Plastic*** (BRYX) category in Building Materials Directory or **Foamed Plastic*** (CCVW) category in Fire Resistance Directory for list of manufacturers.

*Bearing the UL Classification Mark

Last Updated on 2012-03-19

APPENDIX: H

3

3.4 Double-Tee Load Tables

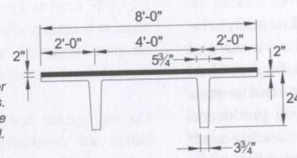
Strand Pattern Designation

1 2 8 - D 1
- 12 = Number of strand (12)
- S = straight D = depressed
- 8 = Diameter of strand in 16ths
- 1 = Number of depression points

Safe loads shown include dead load of 10 lb/ft² for untopped members and 15 lb/ft² for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load.

Key
- 186 – Safe superimposed service load, lb/ft²
- 0.7 – Estimated camber at erection, in.
- 0.9 – Estimated long-time camber, in.

8'-0" × 24"
Normalweight Concrete



f'_c = 5000 psi

f_{pu} = 270,000 psi

½-in.-diameter regular strand

Section Properties

	No Topping	2 in. Topping
A =	401 in.²	–
I =	20,985 in.⁴	27,720 in.⁴
y_b =	17.15 in.	19.27 in.
y_t =	6.85 in.	6.73 in.
S_b =	1224 in.³	1439 in.³
S_t =	3064 in.³	4119 in.³
wt =	418 lb/ft	618 lb/ft
DL =	52 lb/ft²	77 lb/ft²
V/S =	1.41 in.	

Check with regional producers for availability.

8DT24

No Topping

Table of safe superimposed service load, lb/ft², and cambers, in.

Strand pattern	y_s(end) y_s(center) in.	32	34	36	38	40	42	44	46	48	50	52	54	56	58	60	62	64	66	68	70	72	74	76
68-S	4.00	186	161	140	122	106	93	81	71	62	55	48	42	36	31	27								
	4.00	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.0	1.0	1.0	0.9	0.8	0.7	0.6	0.5								
		0.9	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.0	0.9	0.8	0.6	0.4	0.1	–0.2								
88-S	5.00			185	162	143	126	112	99	88	78	70	62	55	49	43	38	33	29					
	5.00			1.1	1.2	1.3	1.3	1.4	1.4	1.5	1.5	1.5	1.5	1.4	1.4	1.3	1.1	1.0	0.8					
				1.4	1.5	1.6	1.6	1.7	1.7	1.7	1.7	1.6	1.5	1.4	1.2	0.9	0.6	0.3	–0.2					
108-S	6.00				197	174	155	138	123	110	98	88	79	71	64	57	51	45	39	34	29			
	6.00				1.4	1.5	1.6	1.7	1.7	1.8	1.9	1.9	1.9	1.9	1.9	1.9	1.8	1.7	1.5	1.3	1.1			
					1.8	1.9	2.0	2.1	2.1	2.1	2.1	2.1	2.1	2.0	1.9	1.8	1.6	1.3	0.9	0.5	0.0			
128-S	7.00							159	142	128	114	101	90	81	73	66	59	53	47	42	37	32	27	
	7.00							1.8	1.9	2.0	2.1	2.1	2.2	2.2	2.2	2.2	2.2	2.1	2.0	1.9	1.7	1.5	1.2	
								2.3	2.4	2.4	2.5	2.5	2.4	2.4	2.3	2.2	2.1	1.8	1.6	1.2	0.8	0.3	–0.3	
128-D1	11.67												114	103	92	83	74	66	59	53	48	43	39	35
	3.25												2.5	2.6	2.6	2.7	2.7	2.6	2.6	2.5	2.3	2.1	1.9	1.6
													2.9	2.9	2.9	2.8	2.6	2.4	2.2	1.8	1.5	1.1	0.6	0.0
148-D1	12.86																		71	64	58	52	46	42
	3.50																		3.1	3.1	3.0	2.9	2.7	2.5
																			3.0	2.7	2.4	2.0	1.6	1.1

8DT24 + 2

2 in. Normalweight Topping

Table of safe superimposed service load, lb/ft², and cambers, in.

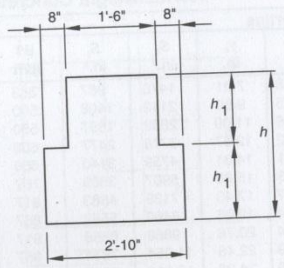
Strand pattern	y_s(end) y_s(center) in.	28	30	32	34	36	38	40	42	44	46	48	50	52	54	56	58	60	62	64	66	68	70
48-S	3.00	169	141	117	97	81	67	55	45	36	29												
	3.00	0.4	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.4												
		0.4	0.4	0.4	0.4	0.3	0.3	0.2	0.0	–0.1	–0.3												
68-S	4.00			189	161	138	118	101	87	74	64	54	45	38	30								
	4.00			0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.0	1.0	1.0	0.9	0.8								
				0.7	0.8	0.8	0.7	0.7	0.6	0.6	0.4	0.2	0.0	–0.3	–0.6								
88-S	5.00					188	163	142	124	108	94	82	71	62	52	42	33						
	5.00					1.1	1.1	1.3	1.3	1.4	1.4	1.5	1.5	1.5	1.5	1.4	1.4						
						1.1	1.2	1.1	1.1	1.1	1.0	0.9	0.7	0.5	0.3	–0.1	–0.5						
108-S	6.00							176	155	136	120	104	90	77	66	56	47	39	31				
	6.00							1.5	1.6	1.7	1.7	1.8	1.9	1.9	1.9	1.9	1.9	1.9	1.8				
								1.4	1.4	1.4	1.4	1.3	1.2	1.0	0.8	0.6	0.3	–0.1	–0.6				
128-S	7.00									160	140	121	104	90	77	67	57	48	41	33	27		
	7.00									1.8	1.9	2.0	2.1	2.1	2.2	2.2	2.2	2.2	2.2	2.1	2.0		
										1.6	1.6	1.6	1.5	1.3	1.1	0.9	0.6	0.3	–0.1	–0.6	–1.1		
128-D1	11.67														105	91	79	68	58	49	40	34	28
	3.25														2.5	2.6	2.6	2.7	2.7	2.6	2.6	2.5	2.3
															1.6	1.4	1.1	0.8	0.4	0.0	–0.6	–1.2	–1.8

Strength is based on strain compatibility; bottom tension is limited to $12\sqrt{f'_c}$; see pages 3-8 through 3-11 for explanation.
Shaded values require release strengths higher than 3500 psi.

3

3.11 Inverted-Tee Beam Load Tables (cont.)

Normalweight Concrete



Designation	h in.	h_1/h_2 in.	A in.^{2}	I in.^{4}	y_b in.	S_b in.^{3}	S_t in.^{3}	Wt lb/ft
34IT20	20	12/8	488	16,082	8.43	1908	1390	508
34IT24	24	12/12	624	27,825	10.15	2741	2009	650
34IT28	28	16/12	696	44,130	11.79	3743	2722	725
34IT32	32	20/12	768	65,856	13.50	4878	3560	800
34IT36	36	24/12	840	93,616	15.26	6135	4514	875
34IT40	40	24/16	976	128,656	16.85	7635	5558	1017
34IT44	44	28/16	1048	171,157	18.58	9212	6733	1092
34IT48	48	23/16	1120	221,906	20.34	10,910	8023	1167
34IT52	52	36/16	1192	281,504	22.13	12,721	9424	1242
34IT60	60	44/16	1336	439,623	25.78	17,053	12,847	1392

Section Properties

1. Check local area for availability of other sizes.
2. Safe loads shown include 50% superimposed dead load and 50% live load. 800 psi top tension has been allowed, therefore, additional top reinforcement is required.
3. Safe loads can be significantly increased by use of structural composite topping.

f'_c = 5000 psi

f_{pu} = 270,000 psi

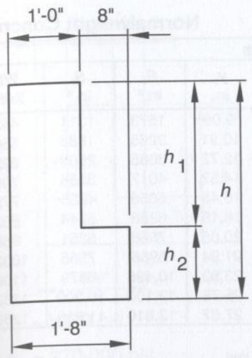
1/2-in.-diameter, low-relaxation strand

Key
7820 – Safe superimposed service load, lb/ft
0.4 – Estimated camber at erection, in.
0.1 – Estimated long-time camber, in.

Table of safe superimposed service load, lb/ft, and cambers, in.

Designation	Number strand	y_s in.	Span, ft 16	18	20	22	24	26	28	30	32	34	36	38	40	42	44	46	48	50
34IT20	14	2.29		7820	6250	5090	4200	3520	2970	2530	2170	1870	1620	1410	1230	1080				
				0.4	0.5	0.6	0.7	0.7	0.8	0.9	1.0	1.1	1.1	1.2	1.2	1.2				
				0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1				
34IT24	17	2.59			9220	7520	6230	5220	4430	3780	3260	2820	2460	2150	1880	1660	1460	1290	1140	1000
					0.4	0.5	0.6	0.7	0.7	0.8	0.9	1.0	1.1	1.1	1.2	1.2	1.2	1.3	1.3	1.2
					0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.1	0.0	–0.1
34IT28	20	3.00					8640	7270	6180	5300	4580	3990	3490	3070	2710	2400	2130	1900	1690	1510
							0.5	0.6	0.7	0.7	0.8	0.9	1.0	1.0	1.1	1.2	1.2	1.3	1.3	1.3
							0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.1
34IT32	23	3.48						9580	8170	7030	6090	5320	4670	4120	3650	3250	2900	2590	2320	2090
								0.5	0.6	0.7	0.8	0.8	0.9	1.0	1.0	1.1	1.2	1.2	1.3	1.3
								0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2
34IT36	24	3.50								9220	8010	7010	6170	5460	4860	4330	3880	3490	3140	2840
										0.6	0.7	0.7	0.8	0.9	0.9	1.0	1.1	1.1	1.2	1.2
										0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
34IT40	30	4.40									9720	8510	7490	6630	5900	5270	4730	4250	3830	3460
											0.6	0.7	0.8	0.9	0.9	1.0	1.1	1.1	1.2	1.3
											0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.4	0.4	0.4
34IT44	30	4.40											9360	8300	7400	6630	5950	5370	4850	4400
													0.7	0.7	0.8	0.9	0.9	1.0	1.0	1.1
													0.2	0.2	0.2	0.2	0.2	0.3	0.2	0.2
34IT48	33	4.73													8960	8030	7230	6530	5910	5370
															0.8	0.8	0.9	1.0	1.0	1.1
															0.3	0.3	0.3	0.3	0.3	0.3
34IT52	36	5.22														9500	8560	7740	7020	6390
																0.8	0.9	0.9	1.0	1.0
																0.3	0.3	0.3	0.3	0.3
34IT56	39	5.59																	8260	7530
																			1.0	1.0
																			0.3	0.3
34IT60	40	6.00																	9560	8720
																			0.8	0.9
																			0.3	0.3

3.10 L-Beam Load Tables



Normalweight Concrete

Section Properties									
Designation	h in.	h_1/h_2 in.	A in.^{2}	I in.^{4}	y_b in.	S_b in.^{3}	S_t in.^{3}	Wt lb/ft
20LB20	20	12/8	304	10,160	8.74	1163	902	317
20LB24	24	12/12	384	17,568	10.50	1673	1301	400
20LB28	28	16/12	432	27,883	12.22	2282	1767	450
20LB32	32	20/12	480	41,600	14.00	2971	2311	500
20LB36	36	24/12	528	59,119	15.82	3737	2930	550
20LB40	40	24/16	608	81,282	17.47	4653	3608	633
20LB44	44	28/16	656	108,107	19.27	5610	4372	683
20LB48	48	32/16	704	140,133	21.09	6645	5208	733
20LB52	52	36/16	752	177,752	22.94	7749	6117	783
20LB56	56	40/16	800	221,355	24.80	8926	7095	833
20LB60	60	44/16	848	271,332	26.68	10,170	8143	883

1. Check local area for availability of other sizes.
2. Safe loads shown include 50% superimposed dead load and 50% live load. 800 psi top tension has been allowed, therefore, additional top reinforcement is required.
3. Safe loads can be significantly increased by use of structural composite topping.

f'_c = 5000 psi

f_{pu} = 270,000 psi

$^1/_2$-in.-diameter, low-relaxation strand

Key
6560 – Safe superimposed service load, lb/ft
0.3 – Estimated camber at erection, in.
0.1 – Estimated long-time camber, in.

Table of safe superimposed service load, lb/ft, and cambers, in.

Designation	Number strand	y_s in.	Span, ft 16	18	20	22	24	26	28	30	32	34	36	38	40	42	44	46	48	50
			6560	5130	4100	3340	2760	2310	1960	1670	1430	1240	1070							
20LB20	9	2.44	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.0	1.1	1.2							
			0.1	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.2							
			9570	7490	6000	4900	4060	3410	2890	2470	2130	1850	1610	1410	1240	1090	969			
20LB24	10	2.80	0.3	0.3	0.4	0.5	0.5	0.6	0.7	0.8	0.9	0.9	1.0	1.0	1.1	1.1	1.2			
			0.1	0.1	0.1	0.1	0.1	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.1	0.0	0.0			
					8220	6730	5590	4710	4000	3440	2970	2590	2270	2000	1760	1560	1390	1240	1110	992
20LB28	12	3.33			0.4	0.4	0.5	0.6	0.6	0.7	0.8	0.9	0.9	1.0	1.1	1.1	1.2	1.2	1.2	1.3
					0.1	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	0.0
						8940	7440	6280	5350	4610	4000	3490	3070	2710	2400	2140	1910	1710	1540	1380
20LB32	14	3.71				0.4	0.5	0.5	0.6	0.7	0.7	0.8	0.9	1.0	1.0	1.1	1.2	1.2	1.3	1.3
						0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.2	0.2	0.2	0.2	0.1
							9450	7980	6820	5880	5110	4470	3940	3480	3100	2770	2480	2230	2010	1810
20LB36	16	4.25					0.4	0.5	0.5	0.6	0.7	0.8	0.8	0.9	1.0	1.1	1.1	1.2	1.2	1.3
							0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2
								9810	8380	7230	6290	5510	4850	4300	3830	3420	3070	2760	2490	2250
20LB40	18	4.89						0.4	0.5	0.6	0.6	0.7	0.8	0.8	0.9	1.0	1.0	1.1	1.1	1.2
								0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3
										8950	7800	6840	6040	5360	4780	4280	3850	3470	3140	2850
20LB44	19	5.05								0.5	0.6	0.6	0.7	0.8	0.8	0.9	0.9	1.0	1.1	1.1
										0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
											9220	8100	7150	6360	5670	5090	4580	4140	3750	3400
20LB48	21	5.81									0.5	0.6	0.6	0.7	0.8	0.8	0.9	0.9	1.0	1.1
											0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3
												9630	8520	7570	6770	6080	5480	4950	4490	4090
20LB52	23	6.17										0.6	0.6	0.7	0.7	0.8	0.9	0.9	1.0	1.0
												0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3
													9950	8860	7920	7120	6420	5820	5280	4810
20LB56	25	6.64											0.6	0.7	0.7	0.8	0.8	0.9	1.0	1.0
													0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3
															9080	8170	7380	6680	6080	5540
20LB60	27	7.33													0.7	0.7	0.8	0.9	0.9	1.0
															0.3	0.3	0.3	0.3	0.3	0.3