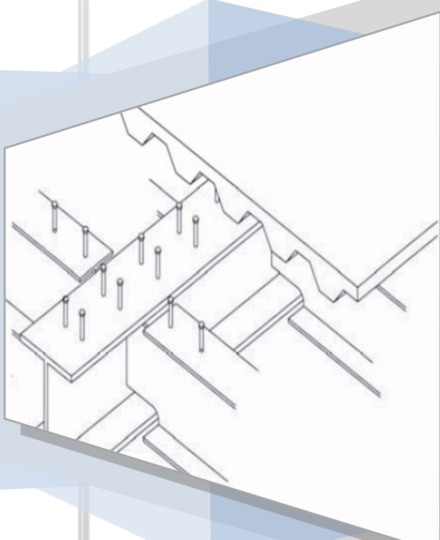
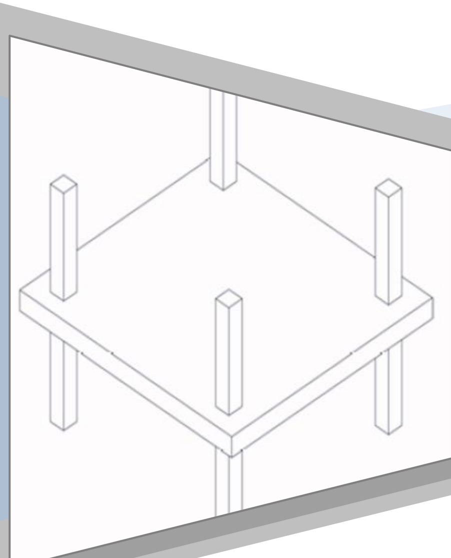
Ingleside at King Farm

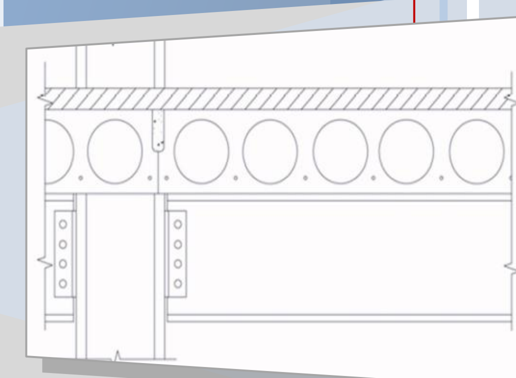
Technical Assignment # 2
Alternative Floor Systems
Investigation Report

Prepared by:
Stephen Dung Tat

Prepared for:
Professor Kevin M. Parfitt







The Pennsylvania State University
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EXECUTIVE SUMMARY:

Purpose

An investigation on alternative floor systems aside from the existing two-way post-tension flat plate concrete system of Ingleside at King Farm was made in this report. Three alternative systems were studied:

1) Two-Way Flat Plate with Reinforced Interior and Exterior Beams
2) Hollow Core Planks on Steel Girders
3) Composite Metal Deck on Steel Girders

Analysis

For the analysis of each floor system, design criteria and serviceability issues were addressed. These factors include cost, floor depth, system weight, deflection, fireproofing, impact on existing architectural and column layout, vibration, accoustics, and constructability.

A typical bay was chosen in one section of the building for simplification of analysis using hand calculations, structural theories, and design charts. Efforts were made to preserve the existing column layout, and any changes to the location of columns were kept to a minimum of 10 percent offset.

Results

The design criteria found for each floor system were compared with each other to determine its feasibility for further investigation. It seems that the existing system is the superior choice among the systems that were analyzed. The existing post-tension system preformed better than the three alternative systems in many of the categories. It was most predominate in deflection control, structural depth, cost, most flexible in terms of the building's floor geometry, time wise to construct, and in preserving the existing architectural plans and structural system.

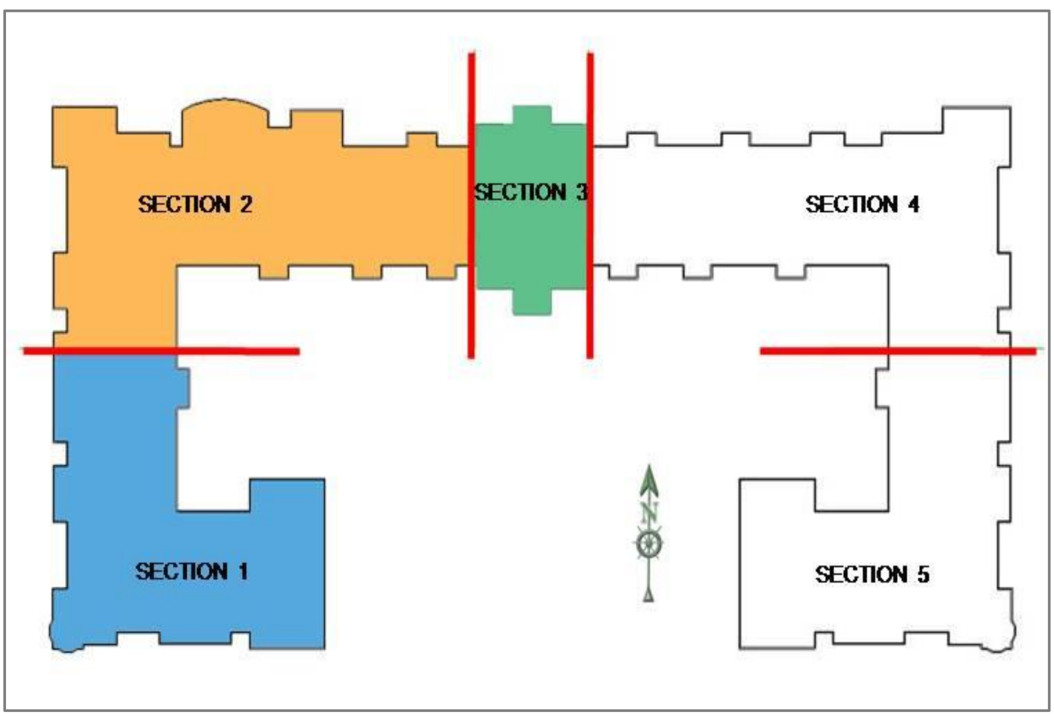
Further investigation topics on the two-way concrete systems include polymer fiber reinforcements, and ways to improve the shear capacity and decrease its construction time even more. As for hollow-core precast planks, the system is the most expensive, and least flexible of the four systems that were analyzed; and thus will be further studied. The composite system does offer some possibilities, and pulturded shapes may be studied for its higher strength and comparable cost to that of steel. Vibration is an associated issue with the use of light weight systems, and various damping techniques and construction can investigated with this composite system. In addition, column elements were not analyzed in this report, but will be in the next report for lateral resistance. A staggered truss system or an exterior load bearing system (possibly tubular steel frame) is likely to be used in conjunction with the feasible systems analyzed in this report.

INTRODUCTION

This pro-con structural study report examines the existing floor system of Ingleside at King Farm and three alternative floor systems. The existing floor system is primary a post-tension two-way flat plate system. Several alternative systems that were analyzed and compared with the existing system were reinforced concrete two-way flat plate with concrete beams, hollow core precast concrete panels on steel girders, and composite metal deck on steel girders. Gravity loads determined in technical report one were used to design the alternative floor systems, along with their respective self weight of the building materials used. Criteria to address and compared with for the floor systems include cost, system weight, floor depth, constructability, fire proofing, construction time, vibrations, and its impact on the existing architecture and structural layout.

There are four expansion joints built into the building. The primary reasons for these expansion joints are due to the shrinkage of the concrete, reduce the amount of strength lost caused by the relaxation in the tendons, and to maintain a continuous construction schedule by preventing idle time; while the concrete in one section of the building is curing, the formwork and layout of reinforcements or concrete placement may be possible in another building section. A majority of the structural analysis and floor system design was done in section one of the building. Section one of the building has a more regular column grid than the other sections. See **Figure 1** for the section divisions of the entire building, which has an approximate floor area of 790,000 square feet.

Figure 1: Building sections



EXISTING STRUCTURAL SYSTEM

Foundation

The sub level of the building is mainly used as a parking garage and contains most of the building's mechanical rooms. The loads from above are transferred down by 30" x 18" reinforced concrete columns with 10 #8 bars to spread footings. Beneath the spread footings is 3 feet of compact fill and then soil with a bearing capacity of 50 ksf. The 30" x 18" reinforced columns extends all the way to either the 6^{th} or 7^{th} floor. The structural slab in the foundation and sub level parking garage is a 5" concrete slab on grade reinforced with 6" x 6" W2.9 / W2.9 welded wire fabric over a vapor barrier and a 4" porous fill. It utilizes standard weight concrete with a 28 day minimum compressive strength of 4000 psi.

Typical Floor Frame

Ingleside at King Farm's primary structural system is a two-way flat plate post-tension concrete structure with 270 ksi unbonded ½ diameter 7 wire tendons. The post-tension concrete slabs are 8 inches thick for typical floors with a compressive strength of 4500 psi. All Concrete used in this building's construction is normal weight. There are no drop panels or beams supporting these typical slabs. The only drop panels in the building are found on the sub level columns holding up the 12 inch thick slab (f'c=6000 psi) that is supporting the weight of the court yard, and the 6^{th} floor columns supporting the 7^{th} floor loads due to the offset W 8 x 31 wide flange columns found on the 7^{th} floor. All the drop panels are 5' x 5' x 10".

Due to the irregular column gird of the building, bays range from 15 feet to 29.5 feet. For the analysis of alternative floor systems, a bay area of 30' x 30' is utilized for a more conservative design, which is the typical interior bay area for the building.

Lateral System

Ingleside at King Farm has eleven shear walls to resist lateral loads from the sub level up to the 7^{th} floor. Seven of the walls are ordinary reinforced concrete shear walls located at stairwells and elevator shafts with #4 horizontal reinforcing bars and #8 vertical reinforcing bars. Typical spacing of these bars is 12 inches. All these walls have a compressive strength of 5000 psi. The remaining four reinforced concrete shear walls have boundary elements and are 15 feet in length; two in east/west direction and two in north/south direction. Spacing of vertical and horizontal reinforcements is 30 inches and 12 inches respectively. Typical clear cover is 1 ½ inches for the reinforcements.

On the 7th floor, in addition to the shear walls, there are also moment connections to resist the lateral loads. Based on lateral load analysis in technical report one, it was discovered that the loads were largest at the 7th floor roof line. Thus, these moment connections (framed seated beam connection) justify the high wind loads that were calculated in technical report one.

Columns

The building contains over 140 reinforced columns, which are either 18" x 30" or 12" x 30". Due to the building's irregular column grid, some columns are miss-counted for in the column schedule. These reinforced concrete columns extend from the sub level to the 6th floor.

All 7th floor columns are W 8 x 31 steel rolled. There are approximately 152 of these steel columns and 33 of them are offset from the concrete reinforced concrete columns below. Thus, 5' x 5' x 10" drop panels are present on the 6th floor to aid with the load transfer and punching shear resistance for the offset columns.

The column schedule also does not account for the 6" x 6" x 3/8" steel tubular columns that are located in section two of the building where a majority of the public areas are found. These HSS columns support the gravity loads of areas whose roof line is at the first floor and second floor level.

Other Structural Elements

Several structural elements that have not been analyzed for this report, but they will be at a later time. They include structural components for the canopies, building envelope supports and load paths into the structural slabs, the steel joists and tubular steel members supporting the roof and roof up lift. An analysis of these structural members for structural strength and serviceability shall be done for the future, and as well as how the various systems work together.

Figure 2: Existing structure with Structural Elements Highlighted - West

Legend

- Reinforced Concrete Columns
- Shear Walls
- Expansion Joints

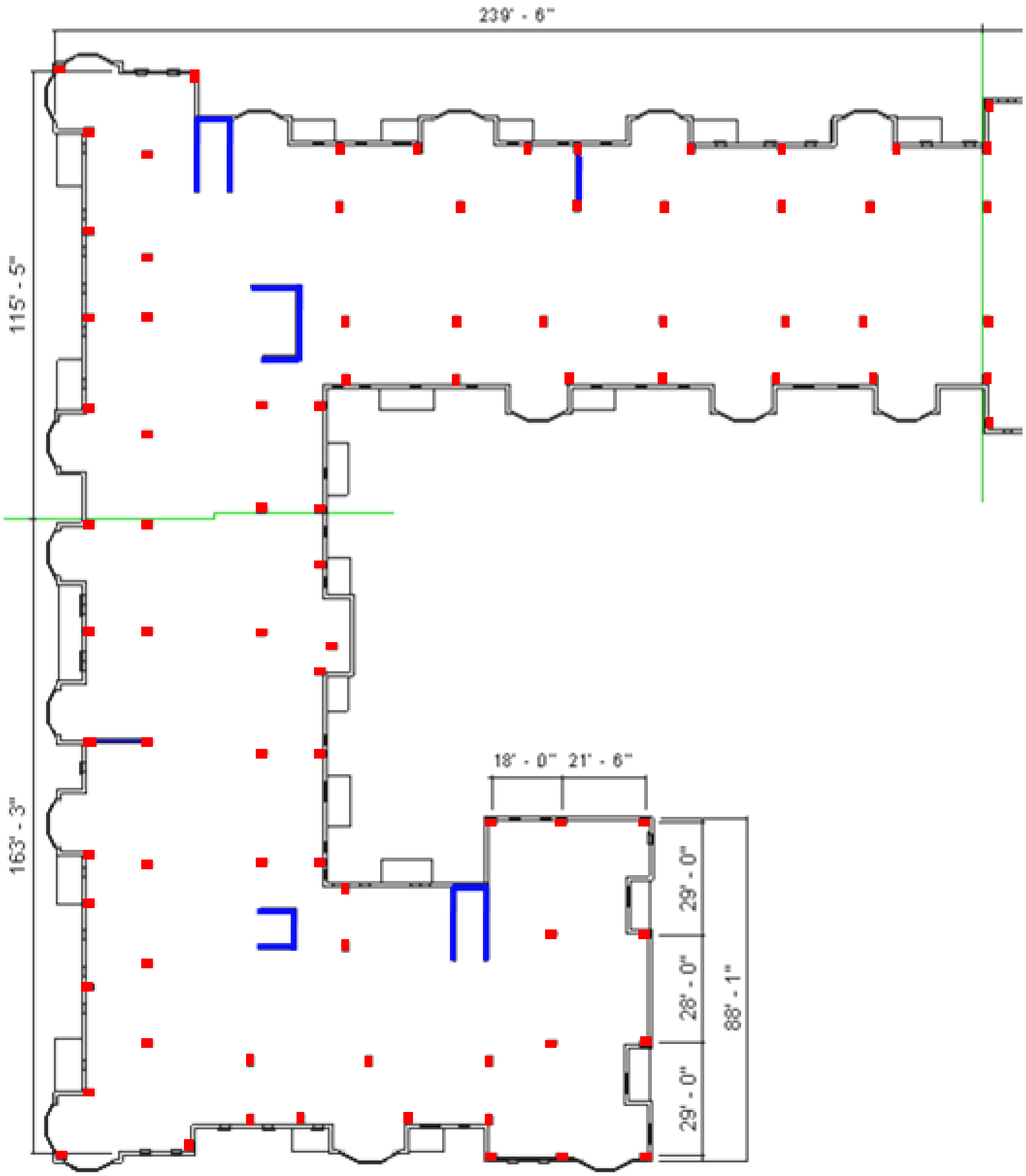
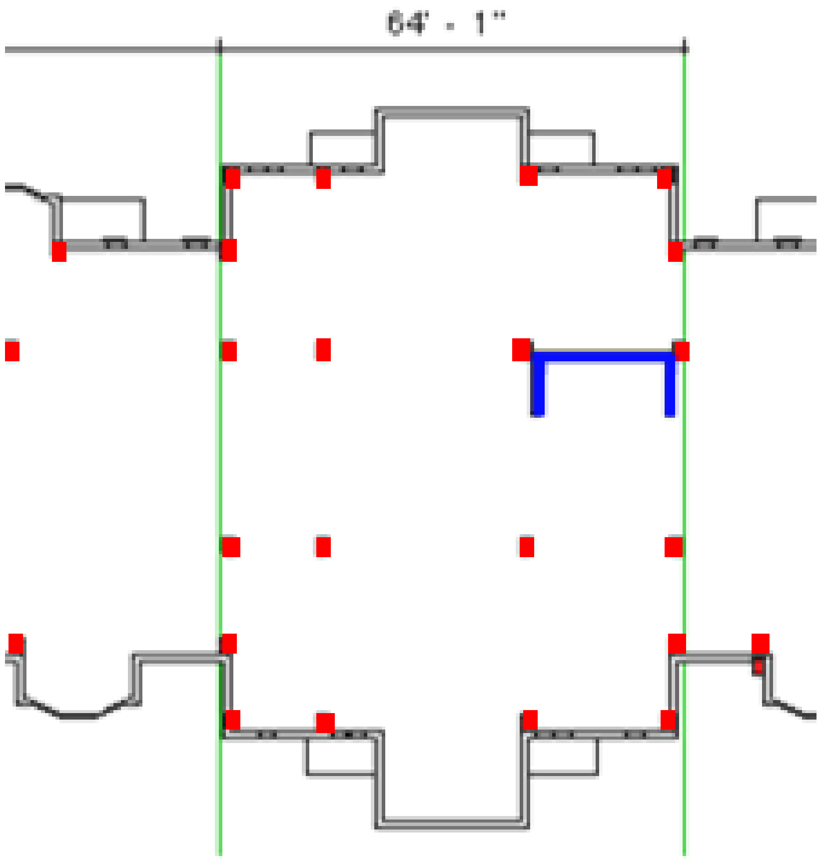


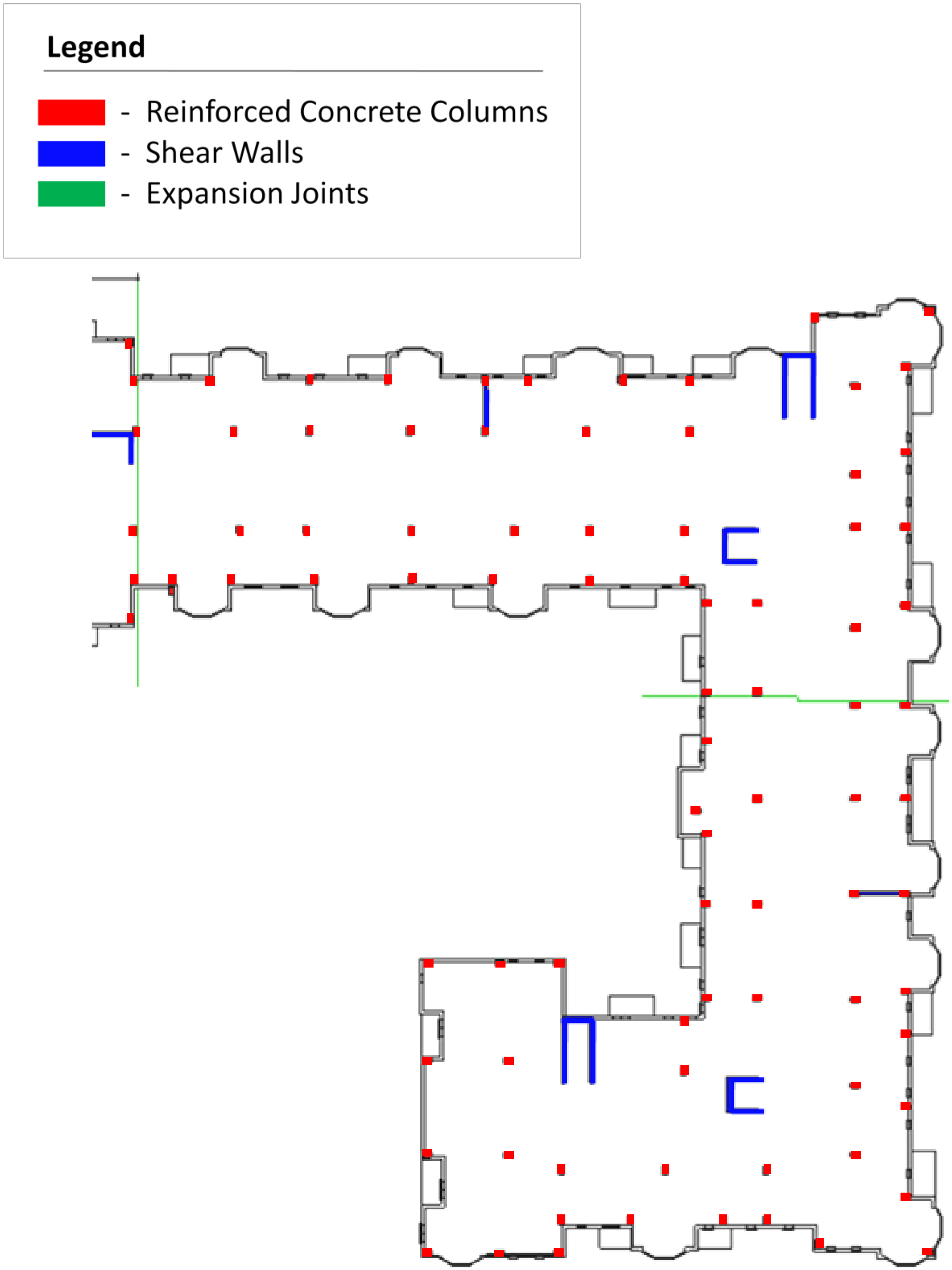
Figure 3: Existing structure with Structural Elements Highlighted - Center



Legend

- Reinforced Concrete Columns
- Shear Walls
- Expansion Joints

Figure 4: Existing structure with Structural Elements Highlighted - East



CODES AND STANDARDS

Codes and Standards in Original Design	Codes and Standards used for this Report
IBC 2003	International Building Code 2006
ASCE 7-98: Minimum Design Loads For Buildings and other Structures.	American Institute of Steel Construction 13th Edition
Rockville, MD City Codes: Local amendments	ASCE 7-05: Minimum Design Loads For Buildings and other Structures.
	American Concrete Institute: Building Code Requirements for Structural Concrete 318 - 05
	Post-Tensioning Institute (PTI) 1st edition

MATERIAL STRENGTH SUMMARY

Structural Steel	
Wide Flange Shapes	Fy= 50 ksi
Hollow Structural Steel (HSS)	Fy=46 ksi
Anchor Rods	Fy=55 ksi
Channels	Fy=36 ksi
Angles	Fy=36 ksi
Concrete	
Structural Slab Supporting Court Yard	F'_c = 6000 psi, Normal wt.
Slab on Grade/Foundation	F'_c = 4000 psi, Normal wt.
Floor Slab	F'_c = 4500 psi, Normal wt.
Cast-in-place Columns	F'_c = 5000 psi, Normal wt.
Cast-in-place Walls	F'_c = 5000 psi, Normal wt.
Shear Walls	F'_c = 5000 psi, Normal wt.
Reinforcements	
Deformed Bars	ASTM A615, Fy=60 ksi
Welded Wire Fabric	ASTM A18, Fy=70 ksi
Post-Tension Tendons	ASTM A-416-74, 270 ksi

BUILDING DESIGN LOAD DISCUSSION:

Gravity Loads

Static and dynamic loads acting on the building were determined in order to analyze the structural behavior of the building. Information regarding the building's weight, code compliant loadings and material specifications were provided and referenced from the construction documents, specifications, AISC 13th edition, ASCE 7 - 05, and IBC 2006. The table below summarizes the type of gravity loads and the system it applies to.

Floor System Loads			
Load Type	**Material / Usage**	**Load**	**Reference**
Dead Load	Normal Weight Concrete	150 pcf	ACS 318
	Cold-formed, light gauge steel stud walls with insulation and 5/8" gypsum board	5 psf	WDG
	Brick Masonry	40 psf	AISC 13th ed.
	Partition Walls	15 psf	Engineer's Judgment
	Miscellaneous	10 psf	Engineer's Judgment
Live Load	Lobbies and Common Spaces	100 psf	ASCE 7 - 05
	Theater Stage	100 psf	ASCE 7 - 05
	Corridors	100 psf	ASCE 7 - 05
	Living Units	40 psf	ASCE 7 - 05
	Balconies	60 psf	ASCE 7 - 05
	Parking Garage	40 psf	ASCE 7 - 05
	Retail Spaces	100 psf	ASCE 7 - 05

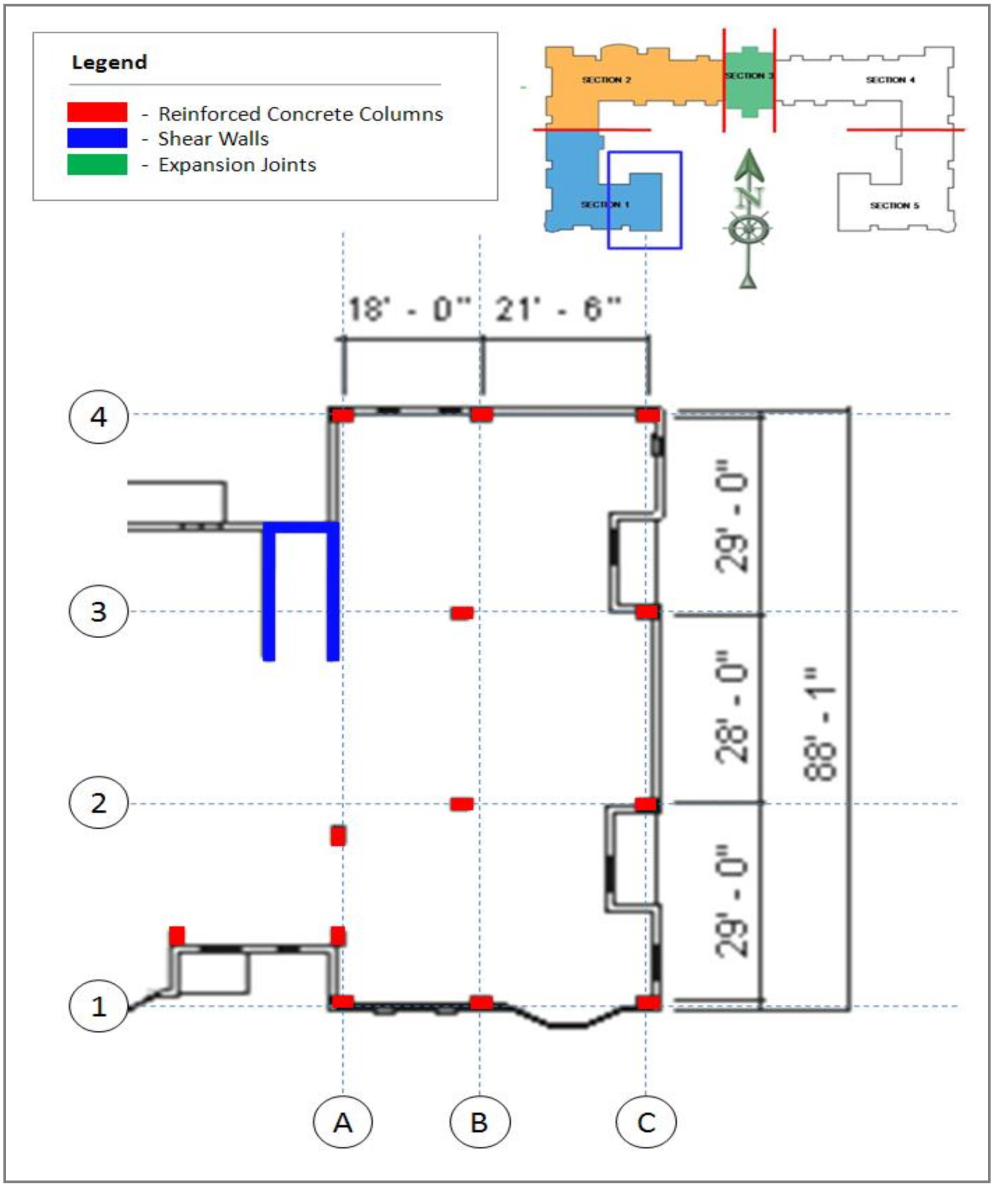
Roof and Terrace System Loads			
Load Type	**Material / Usage**	**Load**	**Reference**
Dead Load	Normal Weight Concrete	150 pcf	ACS 318
	Steel	by shape	AISC 13th ed.
	Steel Deck	2 psf	USD
	Green Roof	100 psf	ASCE 7 - 05
	Ballast, insulation, and waterproofing membrane	8 psf	AISC 13th ed.
	Miscellaneous (MEP, Ceilings, etc…)	15 psf	Engineer's Judgment
Live	Assembly Spaces	100 psf	ASCE 7 - 05
	Roof	30 psf	ASCE 7 - 05
Snow	Ground Snow Load	25 psf	ASCE 7 - 05 & IBC 2006
	Terrain Category	B	ASCE 7 - 05 & IBC 2006
	Ce Exposure	1	ASCE 7 - 05 & IBC 2006
	Ct Thermal Factor	1	ASCE 7 - 05 & IBC 2006
	Importance Factor	1	ASCE 7 - 05 & IBC 2006
	Flat Roof Snow	17.5 psf	ASCE 7 - 05 & IBC 2006

The miscellaneous gravity loads consist of lighting, plumbing, telecommunication, ACT, ductwork and anything that is not regarded as a live load. Because the building's roof is a mansard roof, snow drift will accumulate in the lower flat roof areas. The drift loads are not determined for this report, but will be for the analysis and design of the lateral system.

FLOOR SYSTEMS ANAYLSIS

The gross square footage of each floor level above grade is approximately 480,500 SF. Due to the massive size of the building and its irregular column grid, a small portion of the building was chosen for analysis and treated as a typical bay based on its column grid regularity, number of bays, and max span. The interior columns of **Frame B** is offset within less than 10 percent of the 18 feet span, and hence can be regarded as part of **Frame B** for frame analysis based on ACI code. The portion of the building that was chosen for the computational analysis of the existing floor system is shown in **Figure 5.**

Figure 5: Plan of floor section used for the analysis of the existing system



EXISTING FLOOR SYSTEM ANALYSIS (Two-way Post-tension Flat Plate)

The existing floor system, which is a two-way post-tension flat plate, was analyzed to serve as a reference in comparison with the alternative floor systems. The existing floor system design was hand calculated to verify the assumed basic loadings and design criteria with those used by the designer. The design calculations can be found in the Appendices of this report.

The numbers of banded tendons for **Frame B** were calculated to be the same as that specified by the designer, which is (18) tendons each with 7-wire strands. An exterior column of **Frame B, Column B1**, was chosen for punching shear analysis due to the nature of having the highest bending moment at the exterior span and support. It had failed in the punching shear analysis based on the calculations. Thus, reinforcement bars were needed.

Comparing the amount of reinforcements calculated with the designer's specifications, there seemed to be adequate top reinforcements for the critical section at **Column B1**. The designer's specified more reinforcements than the calculations had required. This was due to the dead load of the exterior wall system that was not factored into the calculations. If the exterior wall's dead load (brick masonry) was to be included, then the amount of rebar reinforcements calculated may be equivalent to that of the designer's specifications. The dead load from the brick masonry was not accounted for in the analysis of this report. It will be accounted for in future analysis as the transfer of the exterior walls' dead load to the slabs will be studied. The brick masonry does not envelope parts of the building where balconies and window dormers are present.

A computer model of the building's structural system will be made in the future for more accurate design. **Figure 6** compares the designer's structural specifications with the hand calculated design based on the assumed loading scenario.

Advantages and Disadvantages of a Two-way Post-tension Flat Plate System	
Pros	Cons
• Deflection and vibration control • Less floor depth • Crack control • Allows for the placement of columns in an irregular grid • Flexible floor design (geometry wise) • Reduced amount of steel reinforcements • Increase of construction speed • 2 hour fire rating	• Large amount of formwork • High labor cost for tendons layout

Figure 6: Existing system - comparison of calculated designed VS Designer's

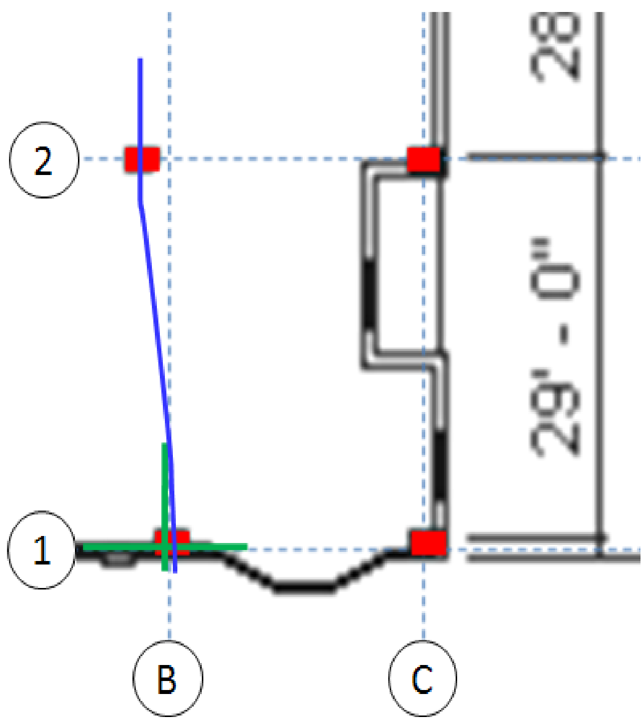
Note: Figures are not shown to scale

Legend

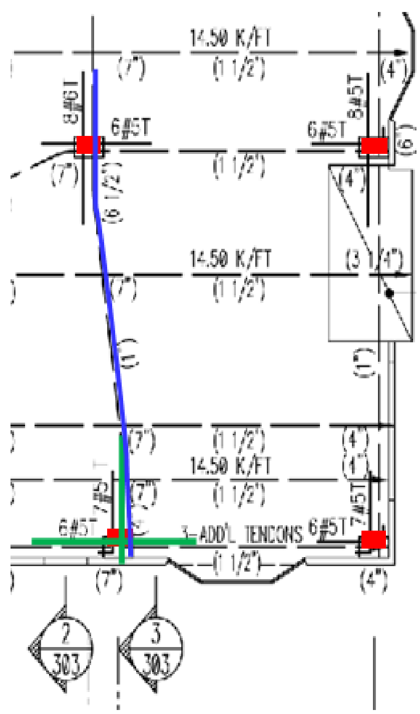
- Reinforced Concrete Columns
- Banded Tendons
- Top Reinforcement Bars

	Assumed Loading Scenario		Designer's Specifications	
	Frame B	Frame 1	Frame B	Frame 1
Steel Reinforcements	(7)#5 T	(4)#5 T	(7)#5 T	(6)#5 T
Tendons	18	NA	18	3

Design Based on Assumed Load Scenario



Designer's Specifications

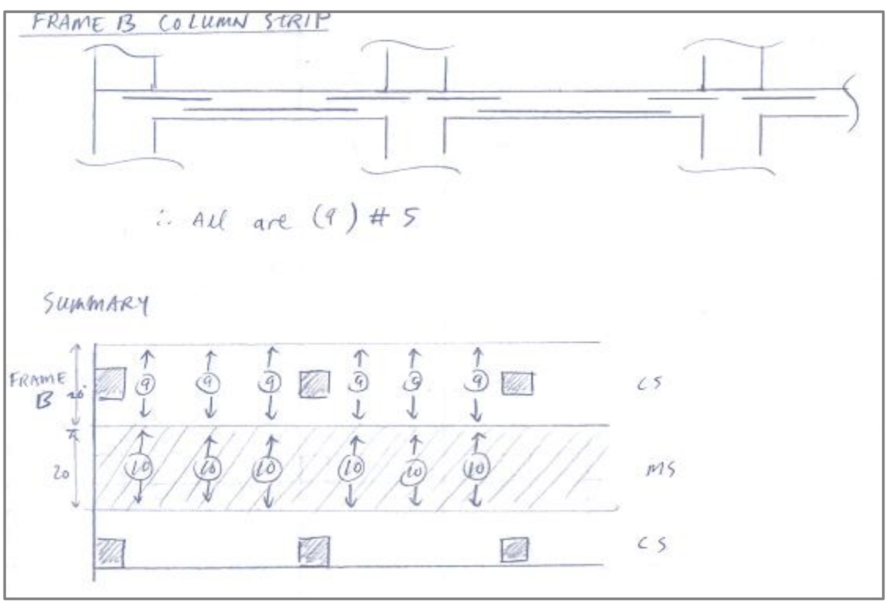


TWO-WAY REINFORCED CONCRETE FLATE PLATE WITH CONCRETE BEAMS

The same floor section used to analyze the existing floor system is used to analyze this alternative system. Instead of post-tension, it will utilize rebar reinforcements and concrete beams in order to give the floor slab more shear resistant. As shown in the calculations for the existing system, punching shear is a major issue around the columns, especially exterior columns. With the interior and edge beams, it will minimize the amount of reinforcements required for shear. However, based on the analysis and calculations, shear reinforcements is still required for punching shear. That can be solved by increasing the depth of the beams, or by increasing the thickness of the slab. Drop panels may also be used to remedy the shear resistance requirements. The disadvantage of this system is that the alignment of the columns had to adjust for the placement of beams and girders.

Advantages and Disadvantages of a Two-way Reinforced Concrete Flat Plate System with Interior and Exterior Beams	
Pros	Cons
• Deflection and vibration control • Provide more shear capacity for areas around columns • Flexible floor design (geometry wise) • 2 hour fire rating	• Large amount of formwork • More steel reinforcements are required • Relocation of columns for the placement of beams and girders

Figure 7: Two-way reinforced concrete flat plate with beams



HOLLOW-CORE PRECAST CONCRETE PLANK FLOOR SYSTEM

PCI design charts were used along with an altered column grid and girder layout to design this alternative system. The planks will rest on W 12 x 106 girders with 50 ksi strength based on calculations (see appendix). Loads are be transferred by W shape columns, which are not designed in this report. Per PCI 2.2.4, for deck members with 2 inch topping, 15 psf superimposed load, and 40 psf live load; the service load was 55 psf. Depth was not a factor since the largest plank depth listed in the charts is 12 inches, and the minimum story height of Ingleside at King Farm is 10 feet.

The primary design criteria that were used to determine the most efficient member size were the weight of the system, span length, and deflection. Light weight concrete is preferred due to the cost of transporting the materials to the site, and for other advantages such as higher thermal insulation and higher fire rating. As for the span factor, planks' span length of 15, 20, 23, 28, and 29 ft will be used (planks' width is 4 feet). See appendix for design charts. Columns were re-aligned (re-off setting in the north-south direction) for the bays to meet the span length of the panels used. Custom sized planks are needed for the floor areas such as balconies, around floor openings, and window dormers.

Design considerations for this alternative system include moment connections to help transfer lateral loads, and the redesigning of the column grid for the placement of steel girders and columns. This system will help reduce the construction time as curing and form work is not required. However, there is the issue with the geometry of floor sections where window dormers are located, which is the building's perimeter. Thus, custom sizes are required. The hollow planks will also reduce the overall weight of the building system.

Advantages and Disadvantages of Hollow-core Precast Concrete Plank Floor System	
Pros	Cons
• Building weight reduction • Faster construction compared to the existing system • No formwork • 2 hour fire rating	• Relocation of columns for the placement of beams and girders • Custom made shapes for the building's perimeter • Shipping cost (high oil prices) • Increased floor depth • Requires moment connections

The next few figures summarize the design of a typical floor using hollow-core precast concrete planks.

Figure 8 (a): Precast hollow core planks on steel girders – section one

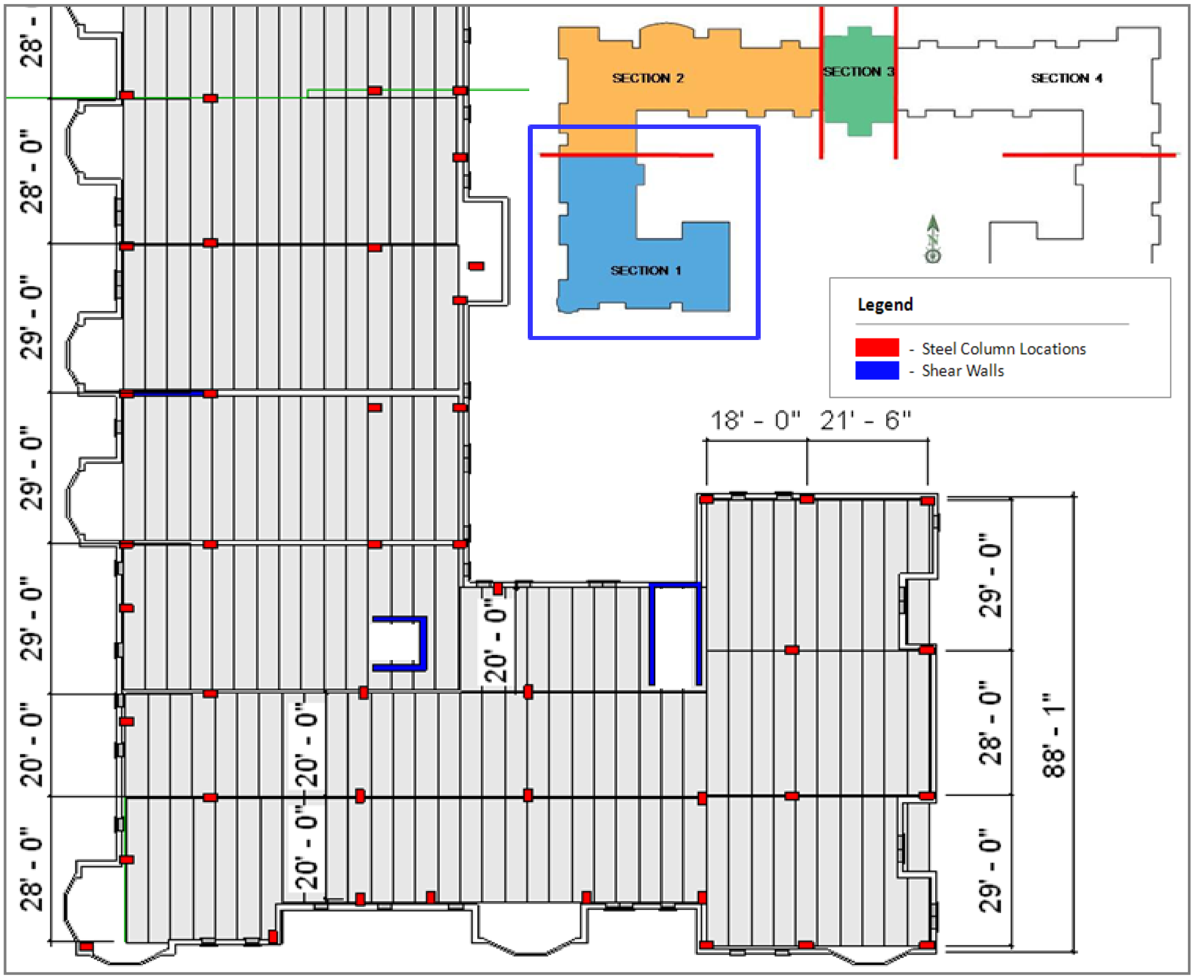


Figure 8 (b): Precast hollow core planks on steel girders – section two

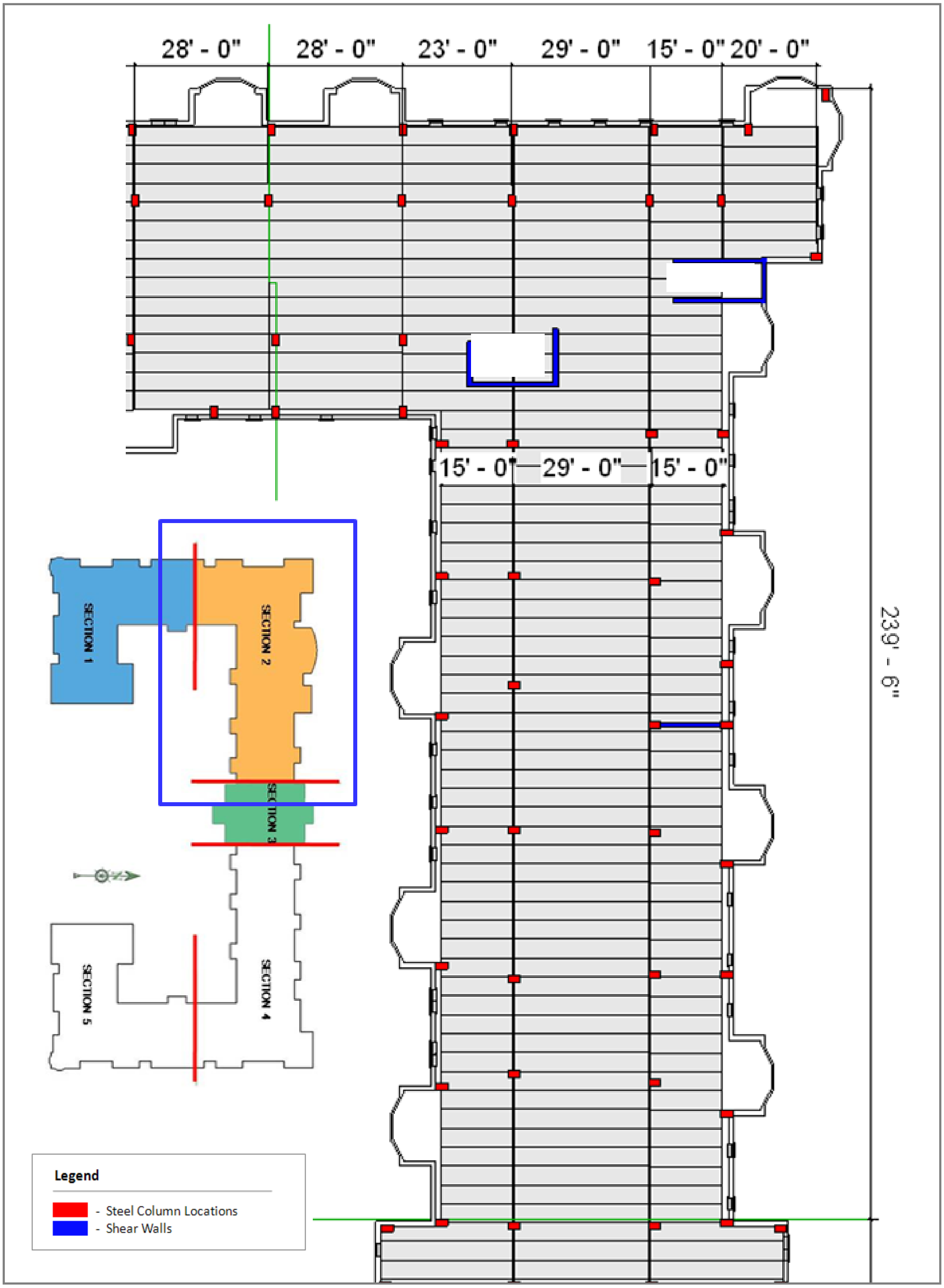
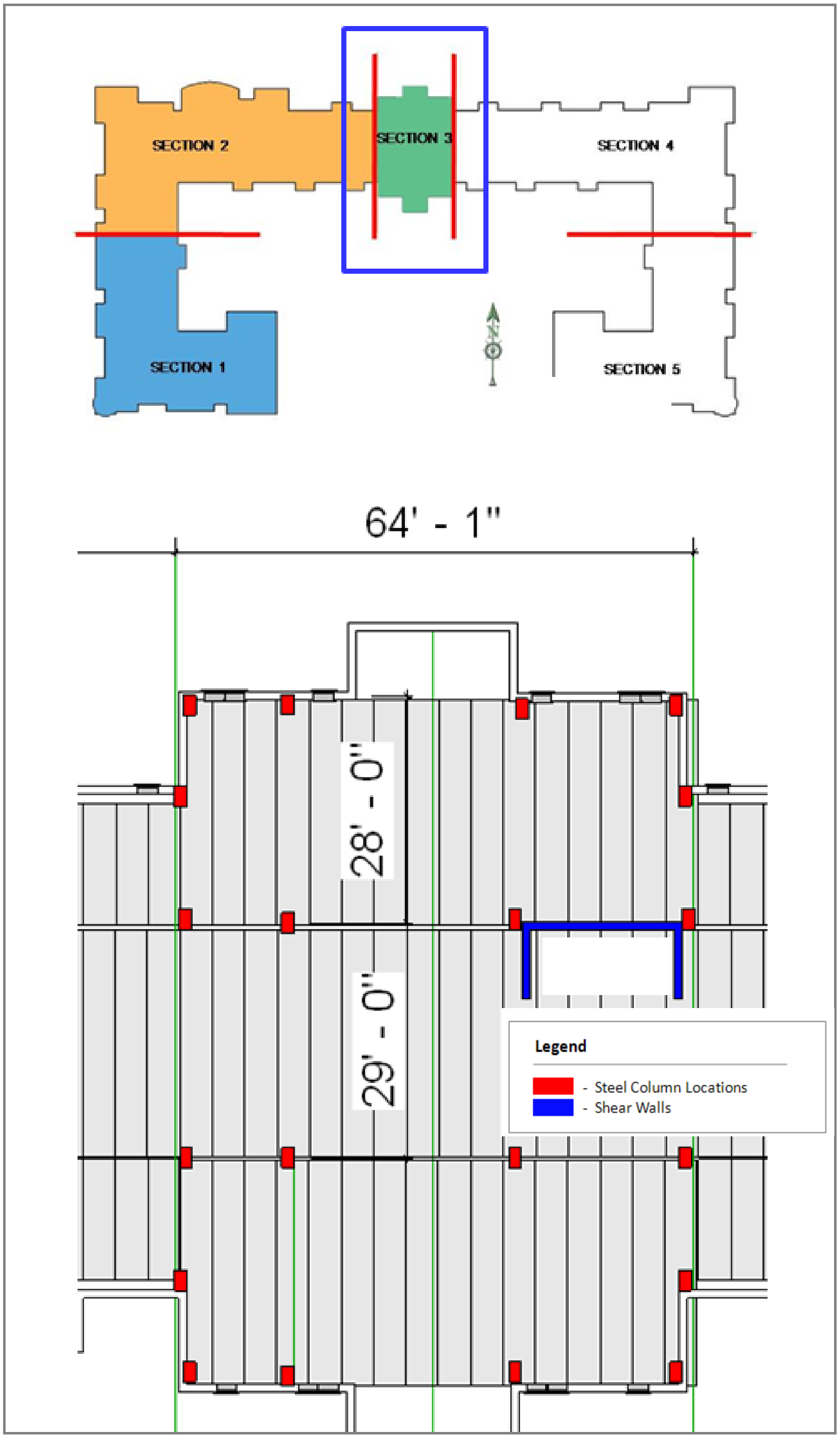


Figure 8 (c): Precast hollow core planks on steel girders – section three



COMPOSITE METAL DECK ON STEEL GIRDERS FLOOR SYSTEM

The United Steel Deck Catalog, along with hand calculations were used to determine the deck. The steel members were sized based on live loads and total loads deflection criteria, and were chosen from the AISC Steel Construction Manual 13th Edition. The composite action is contributed by ¾" diameter shear studs. The column gird used for the Hollow-core Precast Plank system was used for the design of this floor system as well.

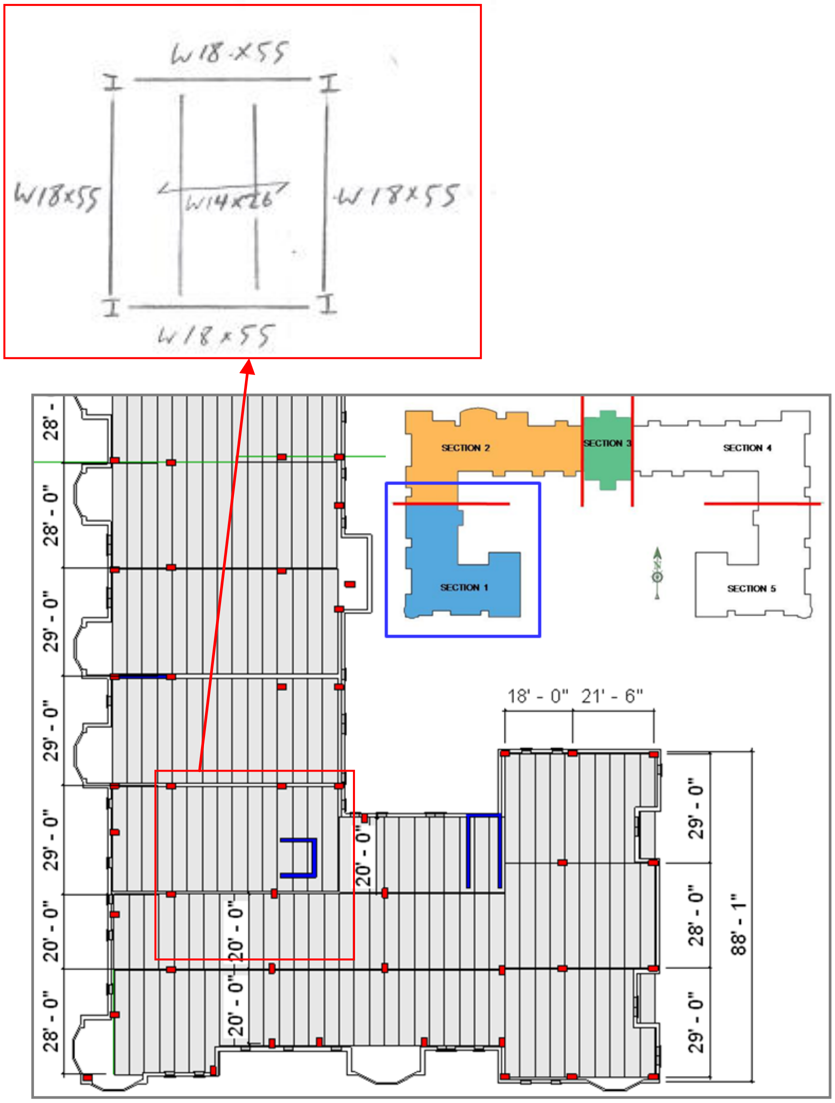
This composite system is simple to construct, light weight, and shallow. However, moment frames would be required to help transfer lateral loads and will likely to increase cost of materials. In addition, a large amount of shear studs are required resulting in an increase cost in labor hours.

A possible solution is to utilize a staggered truss system in which the amount of columns and moment connections could be reduced, and would result in longer bay spans. However, it would greatly impact the architectural plan of the building in which the trusses will have to cut through certain rooms, or partition walls would have to be relocated.

As for construction, formwork and cure time may not be needed, but additional labor cost, transportation cost, and the lead time due to mill procedures would be the disadvantages.

Advantages and Disadvantages of a Composite Steel and Metal Deck Floor System	
Pros	Cons
• Building weight reduction • Simple Construction • Faster construction compared to the existing system • No formwork • 2 hour fire rating with spray on fire proofing	• Relocation of columns for the placement of beams and girders • Shipping cost (high oil prices) • Long lead time due to shapes being rolled and shipped from the mill • Requires moment connections • Additional depth due to the girders

Figure 9: Composite metal deck on steel girders



FLOOR SYSTEMS COMPARISON

	System 1 (existing)	System 2	System 3	System 4
Issues to Address	**Two-way Post-tension Flat Plate**	**Reinforced Concrete Two-way Flat Plate With Beams**	**Precast Hollow Core Planks on Steel Girders**	**Composite Metal Deck on Steel Girders**
Cost	$17.18/sq ft	$19.95/sq ft	$23.88/sq ft	$19.35/sq ft
Floor Depth	8”	8” on 12” deep beams	6” slab with 2” topping on 12” girders	4.5” slab on deck, on 18” girders
System Weight	150 psf	150 psf	74 psf	34 psf
Architecture Plan Impact	None	None	None (Yes if used with a staggered truss system)	None (Yes if used with a staggered truss system)
Existing Column Grid Impact	None	Significant	Some	Significant
Fire Rating	2 hour	2 hour	2 hour (Spray on)	2 hour (Spray on)
Deflection	Little	Little	Medium	high
Vibration and Accoustics	Little to None	Little	Little	Medium to High
Construction Difficulty	Hard	Medium	Easy	Easy
Lead Time	Short	Short	Medium	Long
Further Investigation	Absolutely	Maybe	No	Yes

Comparison Criteria

When comparing the four floor systems, criteria of each system that were analyzed includes cost, floor depth, system’s weight, its impact on existing architectural plans and column grid, fire rating, vibration, construction difficulty, deflection, and lead time.

Cost

The main reference for the cost comparison was made using RS Means Assemblies 2009 data. The cost data indicated in the comparison table is based on a typical 30’ x 30’ bay. The cheapest system is the existing post tension system as less steel reinforcements are needed, and less building material due to a thinner floor depth. The most expensive is the precast hollow core planks system, which does not account for custom made shapes. Thus, using precast hollow core planks is out of the question.

Depth

The average floor depth for the alternative systems, which includes the depth of the supporting beams and girders are 20 inches. Ingleside at King Farm is a mixed used building with most of its commercial areas on the first floor, which is about 14 feet in height. The typical residential floor height is 10 feet. If the other alternative systems are used, the average 20" will greatly dwarf the height of the residential floors. A majority of the residential apartments are high priced suites and condos. Thus, the existing system is the superior choice.

Weight

The major factor in determining the weight each floor system is its thickness and material. Precast hollow planks and composite metal deck offers the lightest weight. However, the weight of a system will also affect the accoustical and vibration performances of a building.

Fireproofing

Ingleside is a mixed-use building. Thus, a 2-hour fire rating is the typical requirement for such construction type. The three alternative systems were initially chosen based on fireproofing requirements. While the Precast core planks and composite metal deck offers fireproofing, the steel girders they rest on does not. Spray on fireproofing is cost effective, but it is not environment friendly. Yet a steel system does compose more recycled components. A composite steel and concrete encased system is a possible further investigation if the composite metal deck is to be considered.

Layout Changes

Due to the utilization of beams and girders, the three alternative systems will require that the columns be relocated or additional columns are needed. This will result in the changes of the architectural plans. Thus, the existing system offers a more flexable structural floor design. In addition, the window dormers also contribute to the un-uniform perimeter of the building. Any precast systems will have to be custom made or manually adjusted.

Lead Time

Although the project is not fast track, time is still a considerable factor as it affects cost, such as the rental of cranes and other equipments. Unlike cast in place system, the composite steel system may acquire lead time for the shipment of materials from the mill. This also includes the hollow core planks as custom sizes are required. In addition, approximately 90% of the condos are sold out, and the date of completion is delayed. Systems that require more lead time will result in more unhappy clients/owners.

Deflection

The two-way concrete flat-plate systems offer the best deflection control. Ingleside being a mixed-use building, design loads cannot be 100 percent certain. A typical floor construction of a typical thickness and typical amount of reinforcements may offer great serviceability in one section of the building, but not another that is of public usage on the same floor. Having to deal with numerous member sizes and construction details on the same floor may affect the speed and cost of construction and labor.

Accoustics/Vibrations

Although accoustics and vibrations were not analyzed in depth in this report, the performance of the floor systems in these two areas can be predicted or categorized based on the stiffness of the structure and its weight. The denser and heavier a structural element is, the less sound energy it will be conducted or transferred by the material, and stiffer structural components will also help dampen the transfer of sounds. The concrete systems are likely to be the most affective systems in dealing with accoustical and vibration performances. Numerical statistics shall be obtained from models or calculations if the structural system is to be further investigated.

CONCLUSION

The evaluation of the feasibility of the floor systems was based on multiply factors. After careful analysis, it appears that the existing two-way post-tension flat plate is the best floor system of choice. Rockville is within proximity of Washington DC. Thus, a concrete system was the choice the designers made. Due to the un-uniform perimeter of the floor, a cast in place system was selected. Any precast systems will require additional changes or custom made components, and connections will complicate the cost of material and labor. The post-tension aspect of the system reduced the amount of long term creep and deflections. Disadvantages with the existing system are the shear capacity, and the affect of pre-stress lost due to time and shrinkage. If further investigation is decided for this system, a study on possible solutions for the system's disadvantages is possible.

Composite Steel is another viable option. The geometry is not as flexible as the two-way flat-plat concrete systems. It also requires a more regular aligned column grid, connections, and solutions to limit serviceability issues such as creep, deflection, accoustics, and vibrations. If further investigation is decided for this system, a study on staggered truss system and pultrusion polymer shapes or light gage is possible. A staggered truss system will reduce the amount of required columns and allows for longer spans.

The reinforced concrete two-way flat plate system with interior and exterior concrete beams is very much like the existing system. Its most apparent difference is the higher shear capacity, and the greater amount of steel reinforcements used. A possible topic for further investigation with this floor system includes the usage of polymer fiber reinforcements in place of the steel.

APPENDIX A: CALCULATIONS

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LOADS:

FRAMING DEAD LOAD = SELFWEIGHT
PARTITION WALLS = 15 PSF
SUPERIMPOSED D.L = 10 PSF (M/E, misc.)
LIVE LOAD = 40 PSF (RESIDENTIAL)

MATERIALS:

CONCRETE: NORMAL WEIGHT 150 PCF
f'_c = 4,500 psi
f'_{ci} = 3,000 psi

REBAR: f_y = 60,000 psi

PT: UNBONDED TENDONS
1/2 ϕ, 7 wire strands, $A_{ps} = 0.153\ in^2$
f_{pu} = 270 ksi

ESTIMATED PRESTRESS LOSSES = 15 ksi (ACI 18.6)
REDUCED EFFECTIVE STRESS $f_{se} = 0.7(270\ ksi) - 15\ ksi = 174\ ksi$ (ACI 18.5.1)
EFFECTIVE FORCE $P_{eff} = A \cdot f_{se} = (0.153)(174\ ksi) = 26.6$ Kips/tendon

DETERMINE PRELIMINARY SLAB THICKNESS

$L/h = 45$
LONGEST SPAN = 29'

$h = (29')(12")/45 = 7.73"$

∴ USE 8.0" SLAB THICKNESS

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LOADING

D.L. = SELFWEIGHT = 8"(150 PCF) = 100 PSF
SIDL & PARTITION WALLS = 25 PSF
LL_o = 40 PSF

IBC 2006, 1607.9.1 allows for LL reduction

EXTERIOR BAY: $A_T = (20')(29') = 580 \text{ ft}^2$

$K_{LL} = 1$

$LL = LL_o\left(0.25 + \frac{15}{\sqrt{1 \times 580}}\right) = LL_o(0.873)$

$LL = 35 \text{ psf}$

INTERIOR BAY: $A_T = (20')(29') = 580 \text{ ft}^2$

$K_{LL} = 1$

$LL = 35 \text{ PSF}$

DESIGN OF SOUTH-NORTH INTERIOR FRAME (FRAME B)

- USE EQUIVALENT FRAME METHOD, ACI 13.7 (EXCLUDING 13.7.7.4-5)

$LL/DL = 35/125 = 0.28 < 3/4$

∴ NO PATTERN LOADING REQUIRED (A.C.I. 13.7.6)

- CALCULATE SECTION PROPERTIES

TWO-WAY SLAB MUST BE DESIGNED AS CLASS U (ACI 18.3.3)
GROSS CROSS-SECTIONAL PROPERTIES ALLOWED (ACI 18.3.4)

$A = bh = (19.75')(12")(8") = 1896 \text{ in}^2$

$S = \frac{(19.75)(12")(8")^2}{6} = 2528 \text{ in}^3$

- IGNORE COLUMN STIFFNESS IN EQUATIONS FOR SIMPLICITY OF HAND CALCULATIONS.

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• SET DESIGN PARAMETERS

ALLOWABLE STRESSES : CLASS U (ACI 18.3.3)

AT TIME OF JACKING (ACI 18.4.1)

$f'_{ci} = 3{,}000$ PSI

COMPRESSION $= 0.60(f'_{ci}) = 0.6(3{,}000) = 1{,}800$ PSI

TENSION $= 3\sqrt{f'_{ci}} = 3\sqrt{3000} = 164$ PSI

AT SERVICE LOADS (ACI 18.4.2 (a) and 18.3.3)

$f'_c = 4{,}500$ PSI

COMPRESSION $= 0.45f'_c = 0.45(4500) = 2025$ PSI

TENSION $= 6\sqrt{f'_c} = 6\sqrt{4500} = 403$ PSI

AVERAGE PRECOMPRESSION LIMITS :

$P/A = 125$ PSI min. (ACI 18.12.4)

$= 300$ PSI max.

TARGET LOAD BALANCES :

CODES DO NOT PRESCRIBE LIMITATIONS FOR THESE PERCENTAGES, BUT WILL NEED TO DESIGN TO APPROPRIATE BALANCING LOADS TO LIMIT SLAB DEFLECTIONS AND CRACKING.

COMMON LOAD-BALANCING PERCENTAGES ARE IN THE 65-PERCENT TO 80-PERCENT RANGE AND IS KEPT CONSISTENT BETWEEN SPANS.

AVERAGE OF 65-PERCENT OF DEAD LOAD (SELFWEIGHT) FOR SLABS SHALL BE USED IN THIS CALCULATION.

$0.65_{W_{DL}} = 0.65(100 \text{ PSF}) = 65$ PSF

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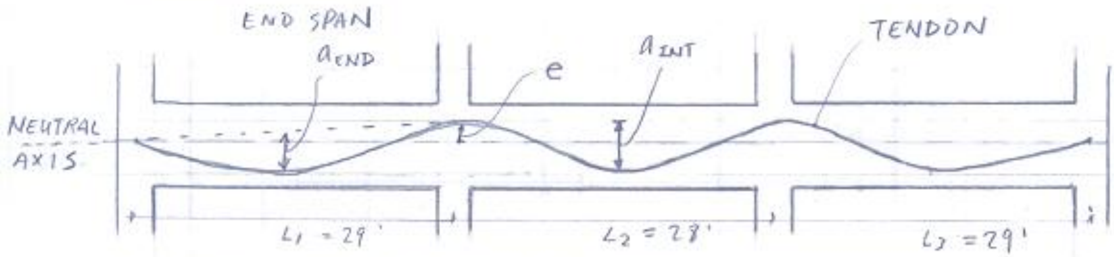
• TENDON PROFILE



* FIGURE NOT DRAWN TO SCALE

TENDON ORDINATE	TENDON CENTER OF GRAVITY LOCATION
EXTERIOR SUPPORT - ANCHOR	4.0"
INTERIOR SUPPORT - TOP	7.0"
INTERIOR SPAN - BOTTOM	1.0"
END SPAN - BOTTOM	1.75"

- LOCATION IS MEASURED FROM BOTTOM OF SLAB
- e : ECCENTRICITY ; IS THE DISTANCE FROM THE CENTER TO TENDON TO THE NEUTRAL AXIS, WHICH VARIES ALONG THE SPAN

$$a_{INT} = 7.0'' - 1.0'' = 6.0''$$
$$a_{END} = (4.0'' + 7.0'')/2 - 1.75'' = 3.75''$$

• REQUIRED PRESTRESS FORCE TO BALANCE 65% OF SELFWEIGHT DL

- DUE TO REDUCED TENDON DRAPE AT a_{END} AND THE EQUIVILENT LENGTH SPANS, THE END SPAN WILL GOVERN THE MAXIMUM REQUIRED POST-TENSION FORCE.

- DESIGNER SPECIFIED EQUIVALENT BALENCING LOAD : ω_b

$$\omega_b = 0.65\,\omega_{DL} = 0.65(100\ psf)(19.75\ ft) = 1284\ plf = 1.28\ ^k/_{ft}$$
(100 psf = self weight)

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

$$P = \omega_b L^2 / 8 a_{END}$$
$$= (1.28)(29)^2 / [8(3.75''/12'')]$$

$$P = 431.8\ \text{KIPS}$$

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- CHECK PRECOMPRESSION ALLOWANCE
  - DETERMINE NUMBER OF TENDONS TO ACHIEVE 431.8 K

$\# \text{TENDONS} = 431.8 / (26.6\ \text{K/TENDON})$
$= 16.23$

$\therefore$ USE 16 TENDONS

- ACTUAL FORCE FOR BANDED TENDONS

$P_{ACTUAL} = 16\,(26.6\text{K}) = 425.6\text{K}$

- BALANCED LOAD FOR THE END SPAN ADJUSTED

$W_{b_{\text{END SPAN ADJUSTED}}} = (425.6/431.8)(1.28\text{K/ft}) = \boxed{0.77\ \text{K/ft}}$ 1.42

- DETERMINE ACTUAL PRECOMPRESSION STRESS

$P_{ACTUAL}/A = (425.6)(1000)/(1896\ \text{in}^2) = \boxed{224.4\ \text{psi}}$

$224.4 > 125$ psi min ✓
< 300 psi max ✓

- CHECK INTERIOR SPAN FORCE

$P = (1.28\text{K/ft})(28\text{ft})^2 / [8\,(6''/12'')]$
$= 250.8 < 425.6\text{K}$ ok ✓

$\therefore$ LESS FORCE IS REQUIRED IN THE CENTER BAY

$W_{b_{\text{MID SPAN}}} = (425.6)(8)(6''/12'')/(28\text{ft})^2 = \boxed{2.17\ \text{K/ft}}$ 2.44

$W_b/W_{DL} = 2.17/1.995 = 109\%$ Acceptable ✓

$\therefore P_{eff} = 425$ KIPS

$W_{b_{AVG}} = (W_{b\ \text{END SPAN}} + W_{b\ \text{INT SPAN}} + W_{b\ \text{END SPAN}})/3$

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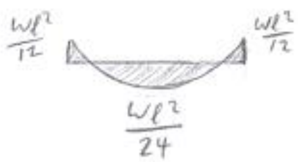
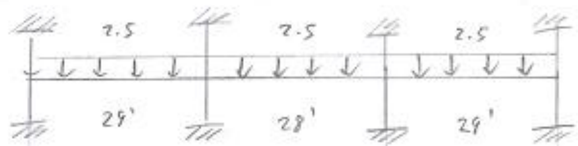
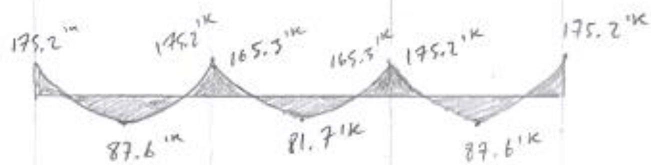
FRAME B:

- DEAD LOAD MOMENTS

$w_{DL} = 125\ PSF(14.75')/1000 = 2.5\ ^{K}/ft$

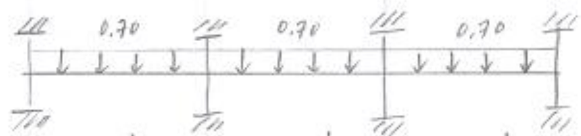


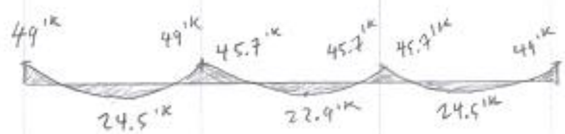




- LIVE LOAD MOMENTS

$W_{LL} = 35\ PSF(14.75')/1000 = 0.70\ ^{K}/ft$





END SPAN INT. SPAN END SPAN

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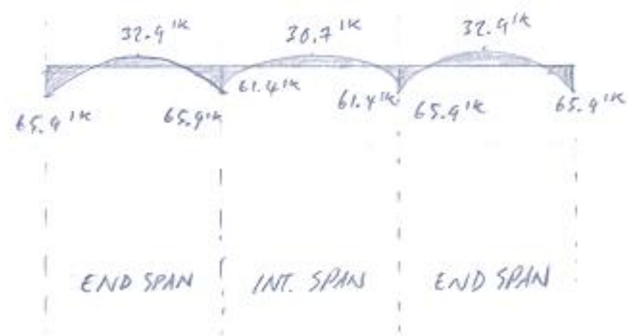
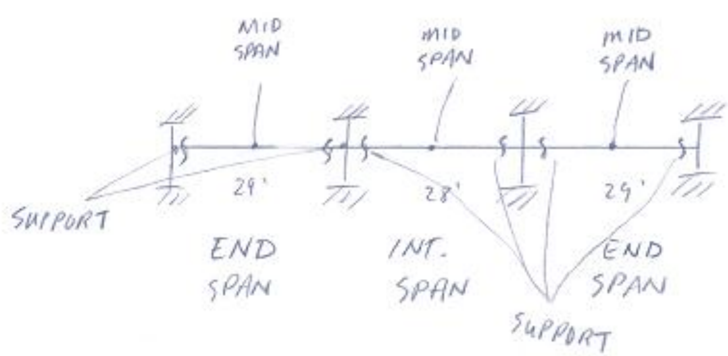
· TOTAL BALANCING MOMENTS, M_{bal}

$$w_{b_{AVG.}} = (w_{b_{END}} + w_{b_{INT}} + w_{b_{END}})/3$$

$$= \frac{0.77 + 1.27 + 0.77}{3} = -0.94 \text{ k/ft}$$



IMPORTANT NOTE: BE CAREFUL WITH IDENTIFYING THE TERMS "MID SPAN" AND "INTERIOR SPAN" IN THE CALCULATIONS ON THE NEXT PAGE.



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STAGE 1 : STRESSES IMMEDIATELY AFTER JACKING (DL + PT) (ACI 18.4.1)

• MID SPAN STRESSES

$f_{top} = (-M_{DL} + M_{bal})/S - P/A$

$f_{bot} = (+M_{DL} + M_{bal})/S - P/A$

• INTERIOR SPAN

$f_{top} = [(-81.7 + 30.7)(12)(1000)]/2560 - 224.5$

$= -463.5$ PSI (COMP.) $< 0.60 f'_{ci} = 1800$ Psi OK ✓

$f_{bot} = [(81.7 - 30.7)(12000)]/2560 - 224.5$

$= 14.6$ PSI (TENSION) $< 0.60 f'_{ci} = 1800$ Psi OK ✓

• END SPAN

$f_{top} = [(-87.6 + 32.9)(12000)]/2560 - 224.5$

$= -480.9$ PSI (COMP.) $< 0.60 f'_{ci} = 1800$ Psi OK ✓

$f_{bot} = [(87.6 - 32.9)(12000)]/2560 - 224.5$

$= 31.9$ PSI (TENSION) $< 0.60 f'_{ci} = 1800$ Psi OK ✓

• SUPPORT STRESSES

$f_{top} = (+M_{DL} - M_{bal})/S - P/A$

$f_{bot} = (-M_{DL} + M_{bal})/S - P/A$

• INTERIOR SPAN

$f_{top} = [(165.3 - 61.4)(12000)]/2560 - 224.5$

$= 262.6$ PSI (TENSION) $> 3\sqrt{f'_c} = 164$ Psi NG ✗

∴ TRY INCREASING # OF TENDONS TO 18

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TRIAL 2 : USE 18 TENDONS (BANDED)

- $P_{ACTUAL} = 18(26.6\text{ k}) = 478.8\text{ k}$

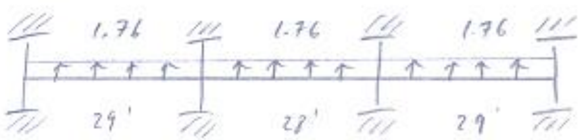
- BALANCE LOAD FOR END SPAN

$$W_{b\ \text{END SPAN ADJUSTED}} = (478.8/431.1)(1.28\ ^{k}/_{ft}) = \boxed{1.42\ ^{k}/_{ft}}$$

$$W_{b\ \text{MID SPAN ADJUSTED}} = (478.8)(8)(6''/12'')/(28\text{ ft})^2 = \boxed{2.44\ ^{k}/_{ft}}$$

TOTAL BALANCING MOMENTS M_{bal}

$$W_{b\ AVG} = [2(1.42) + 2.44]/3 = 1.76\ ^{k}/_{ft}$$



$$P/A = 478.8/1896 = 252.5\text{ psi}$$

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STAGE 1 : STRESSES IMMEDIATELY AFTER JACKING (DL+PT) (ACI 18.4.1)

- MID SPAN STRESSES

$f_{top} = (-M_{DL} + M_{bal})/S - P/A$
$f_{bot} = (+M_{DL} + M_{bal})/S - P/A$

· INTERIOR SPAN

$f_{top} = [(-81.7 + 57.5)(12000)]/2560 - 252.5$
$= -365.9$ PSI (COMP.) $< 0.60 f'_{ci} = 1800$ PSI OK ✓

$f_{bot} = [(81.7 - 57.5)(12000)]/2560 - 252.5$
$= -139.1$ PSI (COMP.) $< 0.60 f'_{ci} = 1800$ PSI OK ✓

· END SPAN

$f_{top} = [(-87.6 + 61.7)(12000)]/2560 - 252.5$
$= -373.9$ PSI (COMP) $< 0.60 f'_{ci} = 1800$ PSI OK ✓

$f_{bot} = [(87.6 - 61.7)(12000)]/2560 - 252.5$
$= -131.1$ PSI (COMP) $< 0.60 f'_{ci} = 1800$ PSI OK ✓

- SUPPORT STRESSES

$f_{top} = (+M_{DL} - M_{bal})/S - P/A$
$f_{bot} = (-M_{DL} + M_{bal})/S - P/A$

· INTERIOR SPAN

$f_{top} = [(165.3 - 115)(12000)]/2560 - 252.5$
$= -16.75$ (COMP) $< 3\sqrt{f'_{ci}} = 164$ PSI OK ✓

$f_{bot} = [(-165.3 + 115)(12000)]/2560 - 252.5$
$= -488.3$ (COMP) $< 0.60 f'_{ci} = 1800$ PSI OK ✓

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• END SPAN

$f_{top} = [(175.2 - 123.3)(12000)]/2560 - 252.5$

$= -9.25$ PSI (COMP) $< 3\sqrt{f'_{ci}} = 164$ PSI OK ✓

$f_{bot} = [(-175.2 + 123.3)(12000)]/2560 - 252.5$

$= -373.9$ PSI (COMP) $< 0.6 f'_{ci} = 1800$ PSI OK ✓

STAGE 2: STRESSES AT SERVICE LOAD (DL + LL + PT)(ACI 18.3.3 and 18.4.2)

• MID SPAN STRESSES

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

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

• INTERIOR SPAN

$f_{top} = [(-81.7 - 22.9 + 57.5)(12000)]/2560 - 252.5$

$= -473.3$ PSI (COMP) $< 0.45 f'_c = 2250$ PSI OK ✓

$f_{bot} = [(81.7 + 22.9 - 57.5)(12000)]/2560 - 252.5$

$= -31.75$ PSI (COMP) $< 6\sqrt{f'_c} = 424$ PSI OK ✓

• END SPAN

$f_{top} = [(-87.6 - 24.5 + 61.7)(12000)]/2560 - 252.5$

$= -488.8$ PSI (COMP) $< 0.45 f'_c = 2250$ PSI OK ✓

$f_{bot} = [(87.6 + 24.5 - 61.7)(12000)]/2560 - 252.5$

$= -16.3$ PSI (COMP) $< 6\sqrt{f'_c} = 424$ PSI OK ✓

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- SUPPORT STRESSES

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

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

- INTERIOR SPAN

$f_{top} = [(165.3 + 45.7 - 115)(12000)]/2560 - 252.5$

$= 197.5 \text{ PSI (TEN.)} < 6\sqrt{f'_c} = 424 \text{ PSI}$ OK ✓

$f_{bot} = [(-165.3 - 45.7 + 115)(12000)]/2560 - 252.5$

$= -702.5 \text{ PSI (COMP.)} < 0.45 f'_c = 2250 \text{ PSI}$ OK ✓

- END SPAN

$f_{top} = [(175.2 + 49 - 123.3)(12000)]/2560 - 252.5$

$= 220.4 \text{ PSI (TEN.)} < 6\sqrt{f'_c} = 424 \text{ PSI}$ OK ✓

$f_{bot} = [(-175.2 - 49 + 123.3)(12000)]/2560 - 252.5$

$= -488.8 \text{ PSI (COMP.)} < 0.45 f'_c = 2250 \text{ PSI}$ OK ✓

∴ ALL STRESSES ARE WITHIN THE PERMISSIBLE CODE LIMITS.

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ULTIMATE STRENGTH

PRIMARY POST-TENSIONING MOMENTS

$M_1 = P(e)$

$e = 0''$ at the exterior support

$e = 3.0''$ at the interior support (NA to the center of tendon)

$M_1 = (478.8\ k)(3\ in)/12 = 119.7^{'k}$

SECONDARY POST-TENSIONING MOMENTS

$M_{sec} = M_{bal} - M_1$

$= 123.3 - 0 = 123.3$ at interior supports

THE TYPICAL LOAD COMBINATION FOR ULTIMATE STRENGTH DESIGN IS:

$M_u = 1.2 M_{DL} + 1.6 M_{LL} + 1.0 M_{sec}$

AT MIDSPAN: $M_u = 1.2(87.6) + 1.6(24.5) + 1.0(-61.7) = 82.6^{'k}$

AT SUPPORT: $M_u = 1.2(-175.2) + 1.6(-49) + 1.0(123.3) = -165.3^{'k}$

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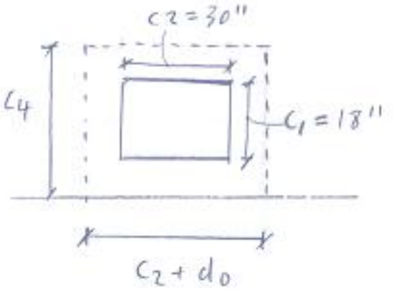
SHEAR CAPACITY IN EXTIOR EDGE COLUMN (B1)



$d_o = 0.8(8") = 6.4"$

$c_2 + d_o = 30 + 6.4 = 36.4"$

$c_4 = 18 + 6.4/2 = 21.2"$

$A_c = (b_1 + 2b_2) \times d_o = (2c_1 + c_2 + 2d_o)\,d_o$
$= (36 + 30 + 12.8)\,12.8$

$\boxed{A_c = 1068.64 \text{ in}^2}$

$$J_c = \frac{(c_1 + d_o/2)d_o^3}{6} + \frac{2d_o(c_3^3 + c_4^3)}{3} + d_o(c_2 + d_o)c_3^2$$

$$c_3 = \frac{d_o(c_1 + \frac{1}{2}d_o)^2}{A_c} = \frac{6.4(18 + 0.5(6.4))^2}{1008.64} = \boxed{2.85"}$$

$$J_c = \frac{(18 + 6.4/2)6.4^3}{6} + \frac{2(6.4)(2.85^3 + 21.2^3)}{3} + 6.4(30+6.4)2.85$$

$J_c = 926.1 + 40752.1 + 1892.2 = \boxed{43570 \text{ in}^3}$

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{c_1 + d_o/2}{c_2 + d_o}}} = 0.337$$

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FOR PUNCHING SHEAR :

$$\omega_u = 1.2(125\ \text{PSF}) + 1.6(35\ \text{PSF}) = 0.206\ ^{k}/_{ft}$$

$$V_u = \omega_u \times \text{Area}$$

$$V_u = 0.206\ ^{k}/_{ft}\left[(19.75 \times 29/2) - (18'' \times 30'')\left(\tfrac{1}{144}\right)\right]$$

$$V_u = 58.22\ ^{'k}$$

$$M_{u\,(support)} = -165.3\ ^{'k}$$

$$v_{u\,max} = \begin{cases} \dfrac{V_u}{A_c} + \dfrac{\gamma_v M_u C_4}{J_c} = \dfrac{58.22(1000)}{1008.64} + \dfrac{0.337(-165.3)(21.2)(12000)}{43570} \\[2ex] \dfrac{V_u}{A_c} + \dfrac{\gamma_v M_u C_3}{J_c} = \dfrac{58.22(1000)}{1008.64} + \dfrac{0.337(-165.3)(2.85)(12000)}{43570} \end{cases}$$

$$v_{u\,max} = \begin{cases} 57.72 + (-325.3) = -267.5 \leftarrow \text{governs} \\ 57.72 + (-43.7) = 13.99 \end{cases}$$

$$b_o = (2C_1 + C_2 + d_o) = (2 \times 18 + 30 + 6.4) = 72.4''$$

$$\alpha_s = 30 \text{ for edge column}$$

$$\beta_c = 30/18 = 1.667$$

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$$v_c = \text{Smallest} \begin{cases} \left(2+\frac{4}{\beta_c}\right)\sqrt{f'_c} = 4.399\sqrt{f'_c} \\ \left(2+\frac{\alpha_s d_o}{b_o}\right)\sqrt{f'_c} = 4.652\sqrt{f'_c} \\ 4\sqrt{f'_c} \Leftarrow \text{governs} \quad (f'_c = 4500\,psi) \end{cases}$$

$$v_c = 4\sqrt{4500} = 268.3$$

$$\phi v_c = 0.75(268.3) = 201.2 \text{ psi}$$

$$v_u = 267.5 > 201.2$$

∴ NG, FAIL IN PUNCHING SHEAR (DUE TO TRANSFER MOMENT)

SOLUTIONS:

- USE DROP PANEL
- USE REINFORCEMENTS
- INCREASE SLAB THICKNESS
- ADD EDGE BEAM
- INCREASE LARGER CRITICAL SECTION

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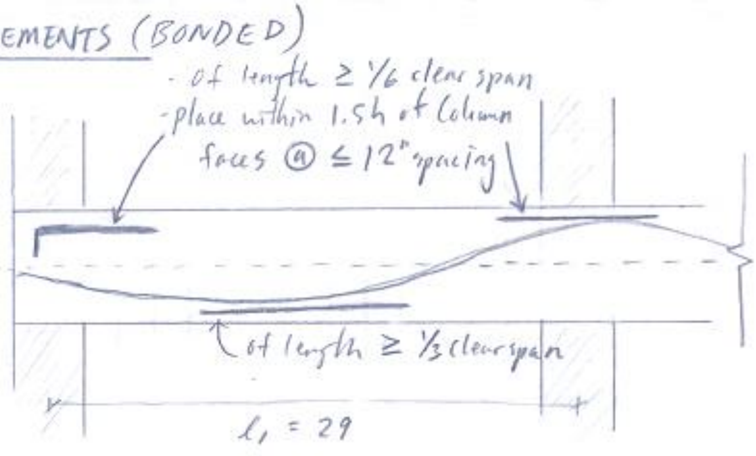
REINFORCEMENTS (BONDED)



- AT NEGATIVE MOMENT AREAS AT COLUMN SUPPORTS

$A_s = 0.00075 A_{cf}$

$A_{cf} = h \times \ell$ (ℓ = length of span in the direction parallel to that of the reinf.)

(ℓ_2 dir.) $A_{cf} = 8\text{ in} \times 19.75(12") = 1896\text{ in}^2$

$A_s = 0.00075(1896) = 1.422\text{ in}^2$

$\frac{1.422\text{ in}^2}{5} = 0.2844\text{ in}^2$

Note: Minimum # of bars is 4

$\therefore$ use 5 #5 $\Rightarrow 5(0.31) = 1.55 > 1.42\text{ in}^2$ ok ✓ along ℓ_2 direction

(ℓ_1 dir.) $A_{cf} = 8\text{in} \times 29(12) = 2784\text{ in}^2$

$A_s = 0.00075(2784) = 2.088$

$\frac{2.088\text{ in}^2}{7} = 0.298$

$\therefore$ use 7 #5 $\Rightarrow 7(0.31) = 2.17 > 2.088$ ok ✓

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DEFLETION

- NO DEFLECTION IS INDUCED UNDER DEAD LOAD BECAUSE OF LOAD BALANCING
- AS SLAB IS LEVEL, NO ADDITIONAL LONG-TERM DEFLECTION SHOULD THEORETICALLY BE RECORDED
- ESTIMATE ELASTIC DEFLECTION DUE TO LIVE LOAD

$$(\Delta_i) = K \frac{w l_a^4}{E_c h^3}$$

l_a = longer span of panel center to center

$$K = 0.11\left(1.5 - 0.5\frac{l_a}{l_b}\right)$$

in which $1 \le \frac{l_a}{l_b} \le 2$

$$K = 0.11\left[1.5 - 0.5\left(\frac{29}{21.5}\right)\right]$$

$$K = 0.0908$$

$$\frac{l_a}{l_b} = 1.35 \Rightarrow 1 \le 1.35 \le 2 \quad \text{ok} \checkmark$$

$$w = 35\ \frac{\text{lb}}{\text{ft}^2}\left(\frac{1\text{ft}}{12''}\right)^2 = 0.243\ \text{psi}$$

$$= \frac{0.0908(0.243)(12\times 29)^4}{57{,}000\sqrt{4500}\ (8)^3} = 0.165\ \text{in}$$

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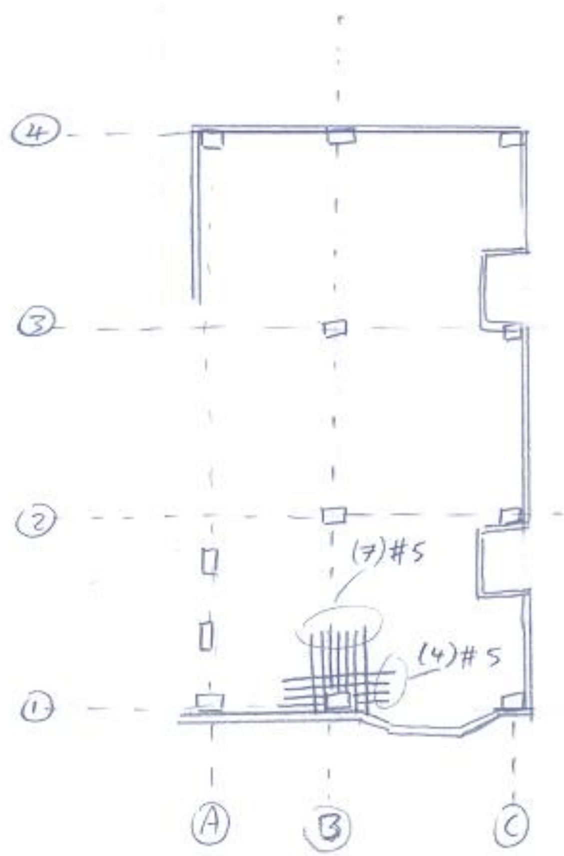
length ≥ 1/6 (29' − 18"/12) = 4.5'

1.5 (8") = 12" from face of columns

- AT POSITIVE MOMENT AREAS
  - NO NEED FOR REINFORCEMENTS

REINFORCEMENTS SUMMARY



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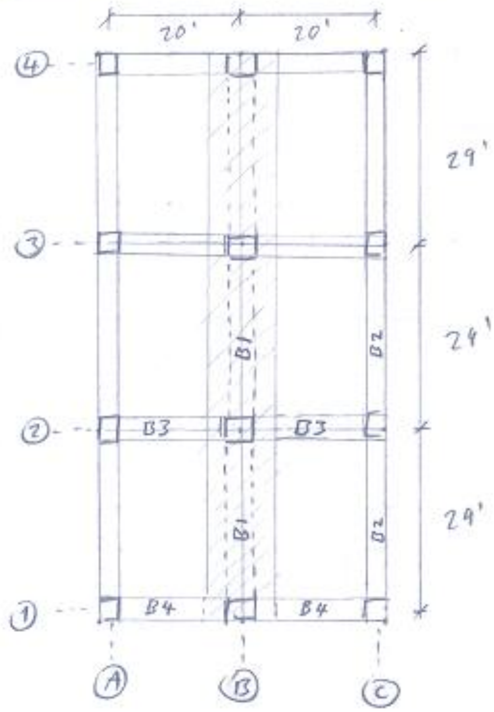
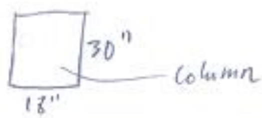
REINFORCED CONCRETE TWO-WAY SLAB



LL = 40 psf
DL = 100 psf → (150 pcf × 8"/12)

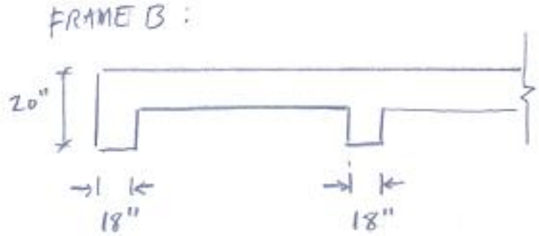
COLUMNS : 18"×30"
BEAMS : 18" web width

f'_c = 5000 psi
F_y = 60,000 psi



- CHECK IF DIRECT DESIGN METHOD CAN BE USED
  - HAS AT LEAST 3 SPANS ✓
  - $\frac{\ell_2}{\ell_1} = \frac{29}{20} = 1.45 \le 2$ ✓
  - NO OFFSETS ✓
  - $w_{LL} = 40 \le 2w_{DL} = 200$ psf

FRAME B :



$$\beta = \frac{\ell_n}{s_n} = (29' - 15"/12)/(20' - 30"/12) = 1.586$$

$$t_{min} = \frac{[(29)(12) - 15](0.8 + 60{,}000/200000)}{36 + 9(1.586)} = 7.286"$$

To be conservative, increase slab by 10%

∴ (1.10)(7.286) = 8.0"

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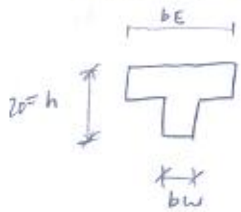
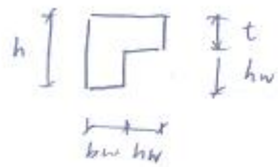
- check that $0.2 \le \frac{l_1^2/\alpha_1}{l_2^2/\alpha_2} \le 5$



$b_{E\,in} = b_w + 2h_w = 18" + 2(20" - 8") = 42"$ ← governs



$b_{E\,ext} = b_w + h_w = 18" + (20" - 8") = 30"$

$$K = \frac{1 + \left(\frac{b_E}{b_w} - 1\right)\left(\frac{t}{h}\right)\left[4 - 6\left(\frac{t}{h}\right) + 4\left(\frac{t}{h}\right)^2 + \left(\frac{b_E}{b_w} - 1\right)\left(\frac{t}{h}\right)^3\right]}{1 + \left(\frac{b_E}{b_w} - 1\right)\left(\frac{t}{h}\right)}$$

$$K_{int} = \frac{1 + \left(\frac{39}{18} - 1\right)\left(\frac{8}{20}\right)\left[4 - 6\left(\frac{8}{20}\right) + 4\left(\frac{8}{20}\right)^2 + \left(\frac{39}{18} - 1\right)\left(\frac{8}{20}\right)^3\right]}{1 + \left(\frac{39}{18} - 1\right)\left(\frac{8}{20}\right)}$$

↑ 2.17 ↑ 0.4

$$K_{int} = \frac{1 + (2.17 - 1)(0.4)\left[4 - 6(0.4) + 4(0.4)^2 + (2.17 - 1)(0.4)^3\right]}{1 + (2.17 - 1)(0.4)}$$

$$K_{int} = \frac{2.08}{1.47} = \boxed{1.42}$$

$$K_{ext} = \frac{1 + \left(\frac{27}{18} - 1\right)\left(\frac{8}{20}\right)\left[4 - 6\left(\frac{8}{20}\right) + 4\left(\frac{8}{20}\right)^2 + \left(\frac{27}{18} - 1\right)\left(\frac{8}{20}\right)^3\right]}{1 + \left(\frac{27}{18} - 1\right)\left(\frac{8}{20}\right)}$$

↳ 1.5

$$K_{ext} = \frac{1 + 0.5(0.4)\left[4 - 2.4 + 0.64 + 0.5(0.064)\right]}{1 + 0.5(0.4)}$$

$$K_{ext} = \frac{1.45}{1.20} = \boxed{1.208}$$

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$$\alpha = \frac{E\, I_{BEAM}}{E\, I_{SLAB}} \qquad I_b = \frac{k\, b_w h^3}{12} \qquad I_s = \frac{l_2 t^3}{12}$$

BEAMS:

EDGE BEAMS: $I_b = \frac{1.258\,(15")(20")^3}{12} = 12{,}080\ in^4$

INTERIOR BEAMS: $I_b = \frac{1.42\,(15")(20")^3}{12} = 14{,}200\ in^4$

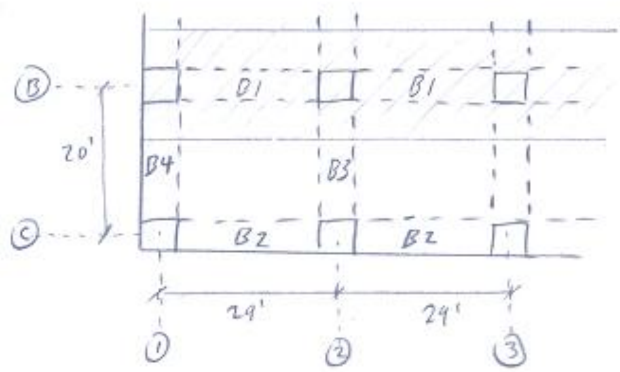
SLABS

29' INTERIOR: $I_s = \frac{(20\times 12)(8)^3}{12} = 10240\ in^4$

29' EXTERIOR: $I_s = \frac{(\frac{20}{2}\times 12)(8)^3}{12} = 5120\ in^4$

20' INTERIOR: $I_s = \frac{(29\times 12)(8)^3}{12} = 14848\ in^4$

20' EXTERIOR: $I_s = \frac{(\frac{29}{2}\times 12)(8)^3}{12} = 7424\ in^4$



$\alpha_{B1} = \frac{I_{b,int}}{I_{s,29,int}} = \frac{14{,}200}{10240} = 1.387$

$\alpha_{B2} = \frac{I_{b,ext}}{I_{s,29,ext}} = \frac{12080}{5120} = 2.359$

$\alpha_{B3} = \frac{I_{b,int}}{I_{s,20,int}} = \frac{14{,}200}{14848} = 0.956$

$\alpha_{B4} = \frac{I_{b,ext}}{I_{s,20,ext}} = \frac{12080}{7424} = 1.627$

$\alpha_m = \frac{\alpha_{B1} + \alpha_{B2} + \alpha_{B3} + \alpha_{B4}}{4}$

$\alpha_m = \frac{0.956 + 1.627 + 1.387 + 2.359}{4}$

$\alpha_m = 1.58 < 2.0$

∴ Medium stiff beam

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FOR MEADIUM STIFF BEAMS $\alpha_m < 2.0$ [ACI 9.5.3.3]

$$m_{int} = \frac{L_n(0.8 + f_y/200{,}000)}{36 + 5B(\alpha_{fm} - 0.2)}$$

$$= \frac{[(29)(12) - 15](0.8 + 60{,}000/200000)}{36 + 5(1.586)(1.58 - 0.2)} = 7.80 \Rightarrow 8.00 \text{ in is fine}$$

LOADING:

$$w_{TL} = 1.2(160 \text{ PSF}) + 1.6(40 \text{ PSF}) = 184 \text{ psf} = 0.184 \text{ KSF}$$

$$M_o = \frac{1}{8} l_2 l_n^2 w$$

$$= \frac{1}{8}(0.184)(20')(29 - 18''/12)^2 = 347.88'^k$$

FOR END SPAN MOMENTS [ACI 13.6.3.3]

Ext : $M^-_{int} = 0.70 M_o = 247.88'^k$

$M^+ = 0.57 M_o = 198.29'^k$

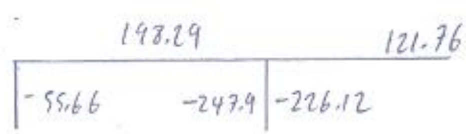
$M^-_{ext} = 0.16 M_o = 55.66'^k$

FOR INTERIOR MOMENTS [ACI 13.6.3.2]

Int : $M^- = 0.65 M_o = 226.12'^k$

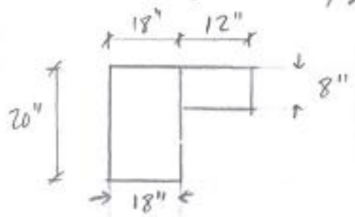
$M^+ = 0.35 M_o = 121.76'^k$

FRAME B Moments (ft·k)



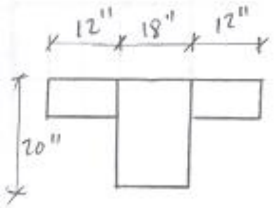
TORSION CONSTANT [ACI 18.7.5] (Sections that are chosen will give largest C)

$$C = \Sigma\left(1 - 0.63\frac{x}{y}\right)\left(\frac{x^3 y}{3}\right)$$



$$C = \left[\left(1-0.63\left(\frac{18}{20}\right)\right)\left(\frac{18^3 \cdot 20}{3}\right)\right] + \left[\left(1-0.63\left(\frac{8}{12}\right)\right)\left(\frac{8^3 \cdot 12}{3}\right)\right]$$

$$C = 16835.04 + 1187.84 = 18022.9$$



$$C = \left[\left(1-0.63\left(\frac{18}{20}\right)\right)\left(\frac{18^3 \cdot 20}{3}\right)\right] + 2\left[\left(1-0.63\left(\frac{8}{12}\right)\right)\left(\frac{8^3 \cdot 12}{3}\right)\right]$$

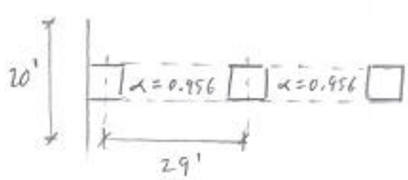
$$C = 16835.04 + 2(1187.84) = 19210.7$$

TRANSVERSE DISTRIBUTION OF LONG MOMENTS

Need: ℓ_2/ℓ_1, α_1, β

$$\ell_2/\ell_1 = \frac{20}{29} = 0.690$$

FRAME B



$$\alpha_1 = \frac{I_b}{I_s} = \frac{14200}{10240} = 1.387$$

$$\beta_1 = \frac{C}{2I_s} = \frac{19210.7}{2(10240)} = 0.938$$

$$\alpha_1\left(\frac{\ell_2}{\ell_1}\right) = 0.65964 = 0.660$$

mint to col strip [ACI 13.6.4.1]

ℓ_2/ℓ_1	0.5	0.69	1.0
$(\alpha_1 \ell_2/\ell_1) \geq 1.0$	90	84.3	75

$-247.9 \rightarrow$ 84.3% to CS $= -208.98^{'k} \rightarrow$ 85% to beam $= -177.6^{'k}$
$\searrow$ 15.7% to MS $= -38.92^{'k}$ $\searrow$ 15% to slab $= -31.3^{'k}$

$-226.12 \rightarrow$ 84.3% to CS $= -190.62^{'k} \rightarrow$ 85% to beam $= -162.03^{'k}$
$\searrow$ 15.7% to MS $= -35.5^{'k}$ $\searrow$ 15% to slab $= -28.59^{'k}$

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M^-_{ext} to Col. strip [ACI 13.6.4.2]

l_2/l_1	0.5	0.69	1.0
$\alpha l_2/l_1 \geq 0$			
$\beta_t = 0$	100	100	100
$\beta_t = 0.938$		94.3	
$\beta_t \geq 2.5$	90	84.3	75

-55.66 → 94.3% to CS = -52.49 ᶦᵏ → 85% to beam = -44.62 ᶦᵏ
↘ 5.7% to MS = -3.17 ᶦᵏ ↘ 15% to slab = -7.87 ᶦᵏ

M^+ to Col. strip [ACI 13.6.4.4]

l_2/l_1	0.5	0.69	1.0
$\alpha l_2/l_1 \geq 0$	90	84.3	75

+198.29 → 84.3% to CS = 163.12 ᶦᵏ → 85% to beam = 138.65 ᶦᵏ
↘ 15.7% to MS = 31.13 ᶦᵏ ↘ 15% to slab = 24.47 ᶦᵏ

+121.76 → 84.3% to CS = 102.64 ᶦᵏ → 85% to beam = 87.25 ᶦᵏ
↘ 15.7% to MS = 14.11 ᶦᵏ ↘ 15% to slab = 15.34 ᶦᵏ

FRAME B : Total width = 20', column strip = 10', middle strip = 10'

	M^-_{ext}	M^+	M^-_{int}	M^-	M^+
Total Mom.	-55.66	+198.29	-247.9	-226.12	+121.76
Beam	-44.62	+138.65	-177.6	-162.03	+87.25
CS Slab	-7.87	+15.39	-31.3	-28.59	+15.39
MS Slab	-3.17	+19.11	-37.92	-35.5	+19.11

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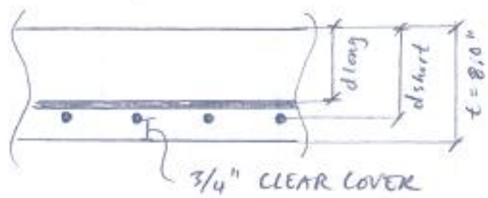
DESIGN OF SLAB REINFORCEMENTS

- Max Spacing = $2t = 2(8) = 16''$
- min steel = Temperature + shrinkage Reinforcement

$$A_{s,min} = 0.0018\,bt$$

CS slab width = $120'' - 15'' = 102''$

Assuming # 5 bars



$$d_{short} = 8 - 0.75 - 1/2(0.625) = 6.94''$$

$$d_{long} = 6.94 - 0.625 = 6.32'' \quad (d_{effective})$$

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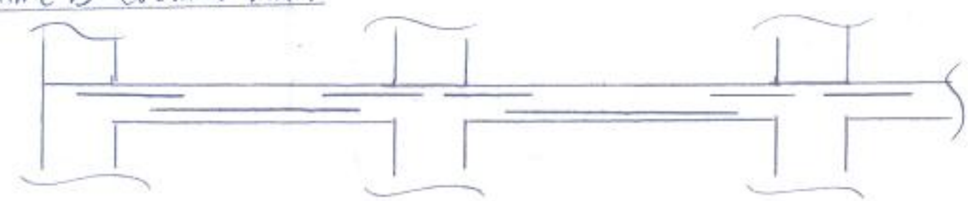
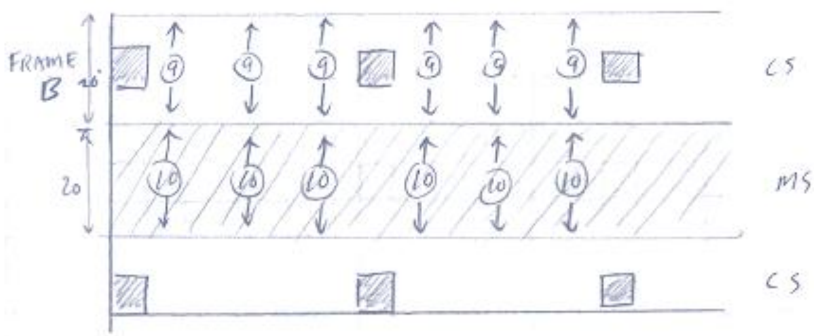
FRAME B COLUMN STRIP

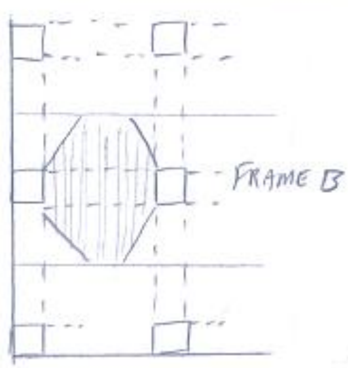


∴ All are (9) #5

SUMMARY



CHECK SHEAR CAPACITY IN SLAB



$w_u = 0.184$ ksf
Beam = 18" wide
Slab = 8" thick

$d_{slab} - d_{short} = 6.94"$

$$V_u = 0.184(1'\text{ strip})\left(10' - \frac{(18/12)}{2} - \frac{6.94}{12}\right)$$
$$= 1.595^k$$

$$\phi V_c = \phi 2\sqrt{f'_c}\,bd = 0.75(2)\sqrt{5000}\,(12)(6.94)\left(\frac{1}{1000}\right)$$
$$= 8.83^k > V_u = 1.595 \quad \text{ok} \checkmark$$

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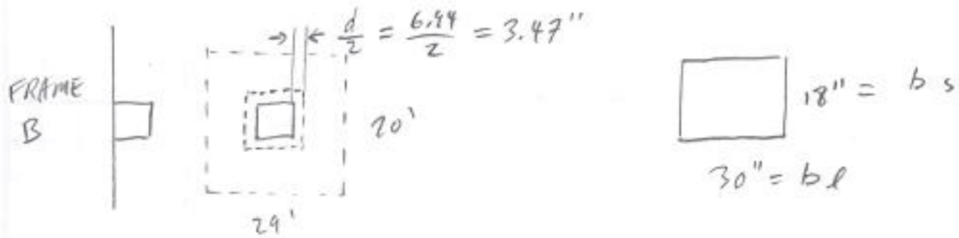
CHECK 2-WAY SHEAR (PUNCHING SHEAR) OF AN INTERIOR COLUMN



Perimeter $b_o = 2(30" + 6.94) + 2(18" + 6.94) = 123.76"$

$\frac{b_o}{d} = \frac{123.76}{6.94} = 17.83$ $\quad \alpha_s = 4.0$ for interior column

$\beta_c = \frac{b_l}{b_s} = \frac{30}{18} = 1.67$

$V_u = W_u \times Area = 0.184\ ksf \left| 24' \times 20' - \left[\left(\frac{30" + 6.94"}{12"}\right) \times \left(\frac{18" + 6.94'}{12}\right) \right] \right|$

$V_u = 105.5^k$

$V_c =$ smallest of:
① $4\sqrt{f'c}\, b_o d$
② $\left(2 + \frac{4}{\beta_c}\right)\sqrt{f'c}\, b_o d = 4.395\sqrt{f'c}\, b_o d$
③ $\left(\frac{\alpha_s}{\frac{b_o}{d}} + 2\right)\sqrt{f'c}\, b_o d = 2.224\sqrt{f'c}\, b_o d$ ⇐ governs

$V_c = 2.224\sqrt{5000}\,(123.76)(6.94)\left(\frac{1}{1000}\right) = 135^k$

$\phi V_c = 0.75\,(135) = 101.3^k < V_u = 105.5^k$

$\therefore \phi V_c < V_u$
$101.3 < 105.5^k$

SOME SHEAR REINFORCEMENTS ARE NEEDED

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TRIAL SIZE - HOLLOW CORE SLAB

USE : 4'-0" x 6" HOLLOW CORE SLAB WITH 2" TOPPING

$f'_c = 5{,}000$ psi   wt w/ topping = 74 psf
$f'_{ci} = 3{,}500$ psi

MAX SPAN LENGTH = 21.5'

∴ USE 96-S with 146 psf Safe superimposed service load

TRADE NAME: STANDARD SPANCRETE

LICENSING ORGANIZATION: SPANCRETE, MILWAUKEE, WISCONSIN

LOADING:

DEAD LOAD: 74 psf (self weight)
10 psf (partitions, duct work, etc...)
84 psf

LIVE LOAD: 40 psf (residential)

$1.2D + 1.6L = 1.2(84) + 1.6(40) = 164.8$ psf

TRIB WIDTH: $19.75'(164.8) = 3.25$ k/ft

$M_u = \frac{w\ell^2}{8} = \frac{3.25(24')^2}{8} = 341'^k$

GIRDERS:

$\Delta_{LL\,max} = \frac{\ell}{360} = \frac{24(12")}{360} = 0.97" = \frac{5(0.79/12)(29\times12)^4}{384(29000)I_{min}}$ ← $\frac{40\times19.75}{1000}$

$\Delta_{TL\,max} = \frac{\ell}{240} = \frac{29(12")}{240} = 1.45" = \frac{5(2.44/12)(29\times12)^4}{384(29000)I_{min}}$ ← $(40+84)19.75$

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$\Delta_{LL\,max}$: $\Rightarrow$ $I_{min} = 446.9 \text{ in}^4$

$\Delta_{TL\,max}$: $\Rightarrow$ $I_{min} = 923.4 \text{ in}^4 \Leftarrow$ governs

$\therefore$ Use W12×106 ($I_x = 933 \text{ in}^4$)

Check:

$M_u = 341'^k < 615'^k$ ✓ ok

$V_u = \frac{wl}{2} = \frac{3.25(29)}{2} = 47.1^k < 236^k$ ✓ ok

Strand Pattern Designation
76-S

- S = straight
- Diameter of strand in 16ths
- No. of Strand (7)

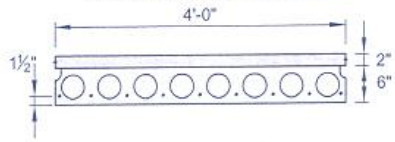
Safe loads shown include dead load of 10 psf for untopped members and 15 psf for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load.

Capacity of sections of other configurations are similar. For precise values, see local hollow-core manufacturer.

Key
444 – Safe superimposed service load, psf
0.1 – Estimated camber at erection, in.
0.2 – Estimated long-time camber, in.

HOLLOW-CORE

4'-0" x 6"
Normal Weight Concrete



$f'_c = 5{,}000$ psi
$f_{pu} = 270{,}000$ psi

Section Properties

	Untopped	Topped
A =	187 in.²	283 in.²
I =	763 in.⁴	1,640 in.⁴
y_b =	3.00 in.	4.14 in.
y_t =	3.00 in.	3.86 in.
S_b =	254 in.³	396 in.³
S_t =	254 in.³	425 in.³
wt =	195 plf	295 plf
DL =	49 psf	74 psf
V/S =	1.73 in.	

4HC6

Table of safe superimposed service load (psf) and cambers (in.) — **No Topping**

Strand Designation Code	Span, ft 10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
66-S	444	382	333	282	238	203	175	151	131	114	100	88	77	68	59	52	46	40	33	28	
	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.5	–0.7		
	0.2	0.2	0.2	0.2	0.3	0.3	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.3	–0.5	–0.7	–0.9	–1.2	–1.5	–1.9	
76-S		445	388	328	278	238	205	178	155	136	120	105	93	82	73	65	57	49	42	36	31
		0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0.1	0.0	–0.1	–0.3	–0.4	–0.6
		0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.7	–0.9	–1.2	–1.6	–2.0
96-S		466	421	386	338	292	263	229	201	177	157	139	124	110	99	88	78	68	60	53	46
		0.3	0.3	0.3	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.1	0.0	–0.1
		0.3	0.4	0.4	0.5	0.5	0.5	0.6	0.6	0.6	0.5	0.5	0.4	0.3	0.2	0.1	–0.1	–0.3	–0.6	–0.9	–1.3
87-S		478	433	398	362	322	290	264	240	212	188	167	149	134	119	107	95	85	76	68	60
		0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.7	0.8	0.8	0.7	0.7	0.7	0.6	0.5	0.4	0.3
		0.4	0.5	0.5	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.3	0.2	0.0	–0.3	–0.6
97-S		490	445	407	374	346	311	276	242	220	203	186	166	148	133	119	107	96	86	78	70
		0.4	0.4	0.5	0.5	0.6	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.9	1.0	0.9	0.9	0.9	0.8	0.7	0.6
		0.5	0.6	0.6	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.0	1.0	0.9	0.9	0.8	0.7	0.5	0.3	0.1	–0.2

4HC6 + 2

Table of safe superimposed service load (psf) and cambers (in.) — **2 in. Normal Weight Topping**

Strand Designation Code	Span, ft 12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
66-S	470	396	335	285	244	210	182	158	136	113	93	75	59	46	34				
	0.2	0.2	0.2	0.2	0.2	02	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2				
	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2	–0.3	–0.5	–0.7	–0.9	–1.2				
76-S		461	391	334	287	248	216	188	163	137	115	95	78	63	50	38	27		
		0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0.1	–0.0	–0.1	–0.3		
		0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.2	–0.3	–0.5	–0.7	–0.9	–1.2	–1.5		
96-S			473	424	367	319	279	245	216	186	160	137	116	98	82	68	55	43	33
			0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.1	0.0	–0.1
			0.4	0.4	0.4	0.4	0.4	0.4	0.3	0.3	0.2	0.1	–0.1	–0.3	–0.5	–0.7	–1.0	–1.4	–1.7
87-S			485	446	415	377	331	292	258	224	195	169	147	127	109	94	80	67	55
			0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.7	0.8	0.8	0.7	0.7	0.7	0.6	0.5	0.4	0.3
			0.5	0.5	0.5	0.6	0.6	0.6	0.5	0.5	0.4	0.4	0.2	0.1	–0.1	–0.3	–0.5	–0.8	–1.2
97-S			494	455	421	394	357	327	288	251	219	192	168	146	127	110	95	82	70
			0.5	0.6	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.9	1.0	0.9	0.9	0.9	0.8	0.7	0.6
			0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.5	0.4	0.2	0.0	–0.2	–0.5	–0.8

Strength is based on strain compatibility; bottom tension is limited to $7.5\sqrt{f'_c}$; see pages 2–7 through 2–10 for explanation.

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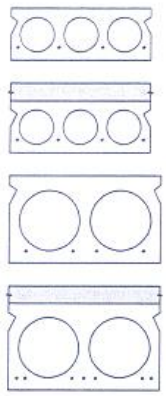
HOLLOW-CORE SLABS

Figure 2.5.3 Section Properties – Normal Weight Concrete **Flexicore**

Trade Name: Flexicore ®
Licensing Organization: The Flexicore Co. Inc., Dayton, Ohio



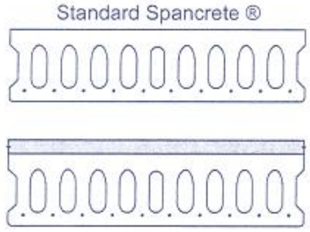
Section	Untopped				With 2 in. topping		
width x depth	A in.^{2}	y_b in.	I in.^{4}	wt psf	y_b in.	I in.^{4}	wt psf
1'-4" x 6"	55	3.00	243	43	4.23	523	68
2'-0" x 6"	86	3.00	366	45	4.20	793	70
1'-4" x 8"	73	4.00	560	57	5.26	1,028	82
2'-0" x 8"	110	4.00	843	57	5.26	1,547	82
1'-8" x 10"	98	5.00	1,254	61	6.43	2,109	86
2'-0" x 10"	138	5.00	1,587	72	6.27	2,651	97
2'-0" x 12"	141	6.00	2,595	73	7.46	4,049	98

Note: All sections are not available from all producers. Check availability with local manufacturers.

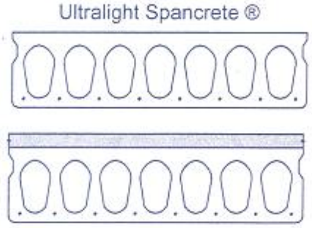
Figure 2.5.4 Section Properties – Normal Weight Concrete **Spancrete**

Trade Name: Spancrete ®
Licensing Organization: Spancrete Machinery Corp., Milwaukee, Wisconsin

Standard Spancrete ®



Ultralight Spancrete ®



Section	Untopped				With 2 in. topping		
width x depth	A in.^{2}	y_b in.	I in.^{4}	wt psf	y_b in.	I in.^{4}	wt psf
4'-0" x 4"	138	2.00	238	34	3.14	739	59
4'-0" x 6"	189	2.93	762	46	4.19	1,760	71
4'-0" x 8"	258	3.98	1,806	63	5.22	3,443	88
4'-0" x 10"	312	5.16	3,484	76	6.41	5,787	101
4'-0" x 12"	355	6.28	5,784	86	7.58	8,904	111
4'-0" x 15"	370	7.87	9,765	90	9.39	14,351	115

4'-0" x 8"	246	4.17	1,730	60	5.41	3,230	85
4'-0" x 10"	277	5.22	3,178	67	6.58	5,376	92
4'-0" x 12"	316	6.22	5,311	77	7.66	8,410	102

Note: Spancrete is also available in 40 in. and 96 in. widths. All sections are not available from all producers. Check availability with local manufacturers.

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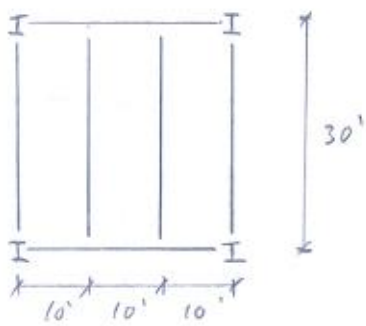
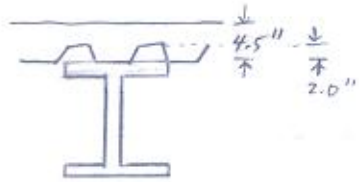
Stephen Dung Tat The Pennsylvania State University Architectural Engineering	Thesis: **Ingleside at King Farm**	

COMPOSITE BEAM WITH FORMED METAL DECK



SLAB:
$f'_c = 3000$ PSI
$F_y = 33$ KSI
$L = 10'$
USE LIGHT WEIGHT CONCR 115 PCF



SLAB DESIGN:

TRY 4.5" thick slab with 18 GAGE 2" LOK FLOOR DECK

UNIFORM LIVE SERVICE LOAD: 230 PSF > 1.6(40) = 64 PSF ✓ OK

MAX UNSHORE SPAN = 11.71' > 10' (3 SPAN) ✓ OK

∴ USE 4 ½" SLAB, 18 GAGE 2" LOK FLOOR DECK UNSHORED
6×6 W1.4×1.4 WWF

BEAM DESIGN

LOADS (UNFACTORED)
DEAD LOAD 0.48 Kip/ft
CONSTRUCTION LIVE 0.36 Kip/ft
SERVICE LIVE LOAD 0.40 Kip/ft > 0.36 Kip/ft

DL: 34 PSF COMPOSITE DECK
10 PSF SUPER IMPOSED (MEP)
40 lb/ft SUPER IMPOSED (SELFWT)
48 PSF Total

LOAD COMB: $1.2D + 1.6L = 1.2(48) + 1.6(40) = 121.6$ PSF

$$M_u = \frac{w\ell^2}{8} = \frac{121.6(10')(30)^2}{8(1000)} = 136.8^{\,\text{'k}}$$

Assume $a = 1$

$$Y_2 = 4.5 - \frac{1}{2} = 4.0 \text{ in} \qquad \text{PNA \# 4}$$

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DEFLECTIONS

$$\Delta_{DL+LL} = \frac{5\left[\frac{(88)10}{1000\times12}\right](30\times12)^4}{384(29000)(I_{LB})} < \frac{l}{240} = \frac{30(12")}{240} = 1.5"$$

$$I_{LB} < 368.7\ in^4 \Leftarrow \text{governs}$$

$$\Delta_{LL} = \frac{5\left[\frac{(40)10}{1000\times12}\right](30\times12)^4}{384(29000)(I_{LB})} < \frac{l}{360} = \frac{30(12")}{360} = 1"$$

$$I_{LB} < 251.4\ in^4$$

$$b_{eff}\ \min = \begin{cases} \frac{SPAN}{4} = \frac{30}{4} = 7.5' \leftarrow \text{governs} \\ \text{beam spacing} = 10' \end{cases}$$

[STL MANUAL TABLE 3-20]

TRY W14×26 ⇒ $I_{BL} = 587\ in^4$

[STL MANUAL TABLE 3-19]

W14×26 $\phi M_p = 151^{'k}$ $I_x = 118\ in^2$

$M_u = 267^{'k}$ @ PNA #4

$\Sigma Q_n = 226^k$ @ PNA #4

$$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{169}{0.85(3)(90)} = 0.736\ in < 1.0 \quad \checkmark ok$$

$$\Delta_{DL+LL} = \frac{5\left[\frac{88\times10}{1000\times12}\right](30\times12)^4}{384(29000)(587)} = 0.94" < 1.5" \quad \checkmark ok$$

$$\Delta_{LL} = \frac{5\left[\frac{(40)10}{1000\times12}\right](30\times12)^4}{384(29000)(587)} = 0.43" < 1" \quad \checkmark ok$$

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$\phi M_p = 151^{\prime k} > M_u = 136.8^{\prime k}$ ✓ OK

[STL. MANUAL TABLE 3-21]

USE
- Weak stud per rib = 1
- Light weight concr.
- stud diameter : 3/4"
- f'_c = 3 ksi
- Perpendicular to deck

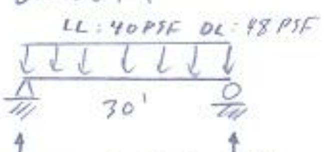
} $Q_n = 17.2$

$\# \text{ Studs} = \frac{226}{17.2} = 13.14 = 14$ studs per side (1 stud / 2.14')

∴ use 28 studs per 30' span beam

GIRDER DESIGN

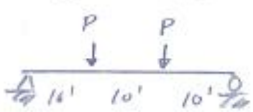
ON BEAM



$LL = 20(10)(30) = 6^k$

$DL = 24(10)(30) = 7.2^k$

ON GIRDER



Beam self weight assume 40 lb/lf = 1.2 k

$LL = 6^k \times 2 = 12^k$

$DL = (7.2^k)2 + 1.2^k = 15.6^k$

$$\Delta_{LL+DL} = \frac{27.6(30)^3(1728)}{28(29000)I_{LB}} \le \frac{l}{240} = 1.5''$$

$I_{LB} \ge 1057.2 \text{ in}^4$ ← governs

$$\Delta_{LL} = \frac{12(30)^3(1728)}{28(29000)I_{LB}} \le \frac{l}{360} = 1''$$

$I_{LB} \ge 689.5 \text{ in}^4$

Beam on both sides of Girder

$P_u = 1.2(15.6) + 1.6(12) = 21.07 \times 2 = 42.14\text{ k}$

Stephen Dung Tat
The Pennsylvania State University
Architectural Engineering

Thesis:
Ingleside at King Farm

$M_u = a(P_u)(L) = 0.333(42.14)(30) = 420^{1k}$

TRY W18×55 → $I_{BL} = 1880\ in^4 > 1057.2\ in^4$

$\phi M_p = 420^{1k} \geq M_u = 420^{1k}$

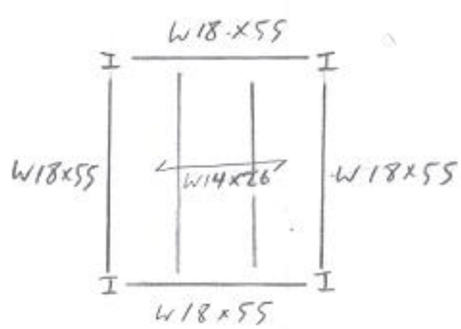
$\Sigma Q_n = 454^k$

$a = \frac{454}{0.85(3)(90)} = 1.98\ in < 4.5$ (avalible conc. depth above deck)

$\#\ studs = \frac{454}{17.2} = 26.3 \Rightarrow 27$ studs per side (1 stud/1.11')

∴ USE 54 studs on Girder

SUMMARY:



$\Delta_{DL+LL} = \frac{27.6(30)^3(1728)}{28(29000)1880} = 0.84'' < 1.5''$ ✓ OK

APPENDIX B: COST DATA

03 23 Stressing Tendons

03 23 05 – Prestressing Tendons

03 23 05.50 Prestressing Steel			Crew	Daily Output	Labor-Hours	Unit	2009 Bare Costs Material	Labor	Equipment	Total	Total Incl O&P
1050	143 kip	G	C-3	4200	.015	Lb.	1.12	.62	.02	1.76	2.26
1200	Ungrouted strand, 50' span, 100 kip	G	C-4	1275	.025		.62	1.13	.02	1.77	2.55
1250	300 kip	G		1475	.022		.62	.98	.02	1.62	2.30
1400	100' span, 100 kip	G		1500	.021		.62	.96	.02	1.60	2.27
1450	300 kip	G		1650	.019		.62	.87	.02	1.51	2.13
1600	200' span, 100 kip	G		1500	.021		.62	.96	.02	1.60	2.27
1650	300 kip	G		1700	.019		.62	.85	.02	1.49	2.09
1800	Ungrouted bars, 50' span, 42 kip	G		1400	.023		.78	1.03	.02	1.83	2.55
1850	143 kip	G		1700	.019		.78	.85	.02	1.65	2.26
2000	75' span, 42 kip	G		1800	.018		.78	.80	.02	1.60	2.18
2050	143 kip	G		2200	.015		.78	.66	.01	1.45	1.93
2220	Ungrouted single strand, 100' slab, 25 kip	G		1200	.027		.62	1.20	.02	1.84	2.67
2250	35 kip	G		1475	.022		.62	.98	.02	1.62	2.30

03 24 Fibrous Reinforcing

03 24 05 – Reinforcing Fibers

03 24 05.30 Synthetic Fibers			Crew	Daily Output	Labor-Hours	Unit	Material	Labor	Equipment	Total	Total Incl O&P
0010	**SYNTHETIC FIBERS**										
0100	Synthetic fibers, add to concrete					Lb.	4.43			4.43	4.87
0110	1-1/2 lb. per C.Y.					C.Y.	6.85			6.85	7.55

03 24 05.70 Steel Fibers			Crew	Daily Output	Labor-Hours	Unit	Material	Labor	Equipment	Total	Total Incl O&P
0010	**STEEL FIBERS**										
0150	Steel fibers, add to concrete	G				Lb.	.70			.70	.77
0155	25 lb. per C.Y.	G				C.Y.	17.50			17.50	19.25
0160	50 lb. per C.Y.	G					35			35	38.50
0170	75 lb. per C.Y.	G					54			54	59.50
0180	100 lb. per C.Y.	G					70			70	77

03 30 Cast-In-Place Concrete

03 30 53 – Miscellaneous Cast-In-Place Concrete

03 30 53.40 Concrete In Place			Crew	Daily Output	Labor-Hours	Unit	Material	Labor	Equipment	Total	Total Incl O&P
0010	**CONCRETE IN PLACE**										
0020	Including forms (4 uses), reinforcing steel, concrete, placement,										
0050	and finishing unless otherwise indicated	R033053-50									
0300	Beams, 5 kip per L.F., 10' span		C-14A	15.62	12.804	C.Y.	400	515	49	964	1,300
0350	25' span		"	18.55	10.782		430	430	41	901	1,200
0500	Chimney foundations, industrial, minimum		C-14C	32.22	3.476		166	133	.80	299.80	390
0510	Maximum		"	23.71	4.724		203	181	1.09	385.09	505
0700	Columns, square, 12" x 12", minimum reinforcing		C-14A	11.96	16.722		400	670	64	1,134	1,550
0720	Average reinforcing	R033105-85		10.13	19.743		720	790	75.50	1,585.50	2,100
0740	Maximum reinforcing			9.03	22.148		1,150	890	84.50	2,124.50	2,750
0800	16" x 16", minimum reinforcing			16.22	12.330		315	495	47	857	1,175
0820	Average reinforcing			12.57	15.911		625	640	61	1,326	1,750
0840	Maximum reinforcing			10.25	19.512		1,025	780	74.50	1,879.50	2,450
0900	24" x 24", minimum reinforcing			23.66	8.453		275	340	32.50	647.50	865
0920	Average reinforcing			17.71	11.293		570	455	43	1,068	1,375
0940	Maximum reinforcing			14.15	14.134		965	565	54	1,584	2,000
1000	36" x 36", minimum reinforcing			33.69	5.936		249	238	22.50	509.50	670
1020	Average reinforcing			23.32	8.576		505	345	33	883	1,125

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Crews

Crew No.	Bare Costs		Incl. Subs O & P		Cost Per Labor-Hour	
Crew C-15	**Hr.**	**Daily**	**Hr.**	**Daily**	**Bare Costs**	**Incl. O&P**
1 Carpenter Foreman (out)	$40.10	$320.80	$62.40	$499.20	$36.01	$55.70
2 Carpenters	38.10	609.60	59.30	948.80		
3 Laborers	30.25	726.00	47.05	1129.20		
2 Cement Finishers	37.00	592.00	54.30	868.80		
1 Rodman (reinf.)	43.00	344.00	70.55	564.40		
72 L.H., Daily Totals		$2592.40		$4010.40	$36.01	$55.70
Crew C-16	**Hr.**	**Daily**	**Hr.**	**Daily**	**Bare Costs**	**Incl. O&P**
1 Labor Foreman (outside)	$32.25	$258.00	$50.20	$401.60	$35.87	$55.68
3 Laborers	30.25	726.00	47.05	1129.20		
2 Cement Finishers	37.00	592.00	54.30	868.80		
1 Equip. Oper. (med.)	39.85	318.80	60.10	480.80		
2 Rodmen (reinf.)	43.00	688.00	70.55	1128.80		
1 Concrete Pump (small)		728.20		801.02	10.11	11.13
72 L.H., Daily Totals		$3311.00		$4810.22	$45.99	$66.81
Crew C-17	**Hr.**	**Daily**	**Hr.**	**Daily**	**Bare Costs**	**Incl. O&P**
2 Skilled Worker Foremen	$41.40	$662.40	$64.40	$1030.40	$39.80	$61.88
8 Skilled Workers	39.40	2521.60	61.25	3920.00		
80 L.H., Daily Totals		$3184.00		$4950.40	$39.80	$61.88
Crew C-17A	**Hr.**	**Daily**	**Hr.**	**Daily**	**Bare Costs**	**Incl. O&P**
2 Skilled Worker Foremen	$41.40	$662.40	$64.40	$1030.40	$39.81	$61.88
8 Skilled Workers	39.40	2521.60	61.25	3920.00		
.125 Equip. Oper. (crane)	40.95	40.95	61.75	61.75		
.125 Hyd. Crane, 80 Ton		161.25		177.38	1.99	2.19
81 L.H., Daily Totals		$3386.20		$5189.52	$41.80	$64.07
Crew C-17B	**Hr.**	**Daily**	**Hr.**	**Daily**	**Bare Costs**	**Incl. O&P**
2 Skilled Worker Foremen	$41.40	$662.40	$64.40	$1030.40	$39.83	$61.88
8 Skilled Workers	39.40	2521.60	61.25	3920.00		
.25 Equip. Oper. (crane)	40.95	81.90	61.75	123.50		
.25 Hyd. Crane, 80 Ton		322.50		354.75		
.25 Trowel, 48" Walk-Behind		9.95		10.95	4.05	4.46
82 L.H., Daily Totals		$3598.35		$5439.60	$43.88	$66.34
Crew C-17C	**Hr.**	**Daily**	**Hr.**	**Daily**	**Bare Costs**	**Incl. O&P**
2 Skilled Worker Foremen	$41.40	$662.40	$64.40	$1030.40	$39.84	$61.88
8 Skilled Workers	39.40	2521.60	61.25	3920.00		
.375 Equip. Oper. (crane)	40.95	122.85	61.75	185.25		
.375 Hyd. Crane, 80 Ton		483.75		532.13	5.83	6.41
83 L.H., Daily Totals		$3790.60		$5667.77	$45.67	$68.29
Crew C-17D	**Hr.**	**Daily**	**Hr.**	**Daily**	**Bare Costs**	**Incl. O&P**
2 Skilled Worker Foremen	$41.40	$662.40	$64.40	$1030.40	$39.85	$61.87
8 Skilled Workers	39.40	2521.60	61.25	3920.00		
.5 Equip. Oper. (crane)	40.95	163.80	61.75	247.00		
.5 Hyd. Crane, 80 Ton		645.00		709.50	7.68	8.45
84 L.H., Daily Totals		$3992.80		$5906.90	$47.53	$70.32
Crew C-17E	**Hr.**	**Daily**	**Hr.**	**Daily**	**Bare Costs**	**Incl. O&P**
2 Skilled Worker Foremen	$41.40	$662.40	$64.40	$1030.40	$39.80	$61.88
8 Skilled Workers	39.40	2521.60	61.25	3920.00		
1 Hyd. Jack with Rods		84.00		92.40	1.05	1.16
80 L.H., Daily Totals		$3268.00		$5042.80	$40.85	$63.03

Crew No.	Bare Costs		Incl. Subs O & P		Cost Per Labor-Hour	
Crew C-18	**Hr.**	**Daily**	**Hr.**	**Daily**	**Bare Costs**	**Incl. O&P**
.125 Labor Foreman (out)	$32.25	$32.25	$50.20	$50.20	$30.47	$47.40
1 Laborer	30.25	242.00	47.05	376.40		
1 Concrete Cart, 10 C.F.		56.40		62.04	6.27	6.89
9 L.H., Daily Totals		$330.65		$488.64	$36.74	$54.29
Crew C-19	**Hr.**	**Daily**	**Hr.**	**Daily**	**Bare Costs**	**Incl. O&P**
.125 Labor Foreman (out)	$32.25	$32.25	$50.20	$50.20	$30.47	$47.40
1 Laborer	30.25	242.00	47.05	376.40		
1 Concrete Cart, 18 C.F.		84.40		92.84	9.38	10.32
9 L.H., Daily Totals		$358.65		$519.44	$39.85	$57.72
Crew C-20	**Hr.**	**Daily**	**Hr.**	**Daily**	**Bare Costs**	**Incl. O&P**
1 Labor Foreman (outside)	$32.25	$258.00	$50.20	$401.60	$32.54	$49.98
5 Laborers	30.25	1210.00	47.05	1882.00		
1 Cement Finisher	37.00	296.00	54.30	434.40		
1 Equip. Oper. (med.)	39.85	318.80	60.10	480.80		
2 Gas Engine Vibrator		49.60		54.56		
1 Concrete Pump (small)		728.20		801.02	12.15	13.37
64 L.H., Daily Totals		$2860.60		$4054.38	$44.70	$63.35
Crew C-21	**Hr.**	**Daily**	**Hr.**	**Daily**	**Bare Costs**	**Incl. O&P**
1 Labor Foreman (outside)	$32.25	$258.00	$50.20	$401.60	$32.54	$49.98
5 Laborers	30.25	1210.00	47.05	1882.00		
1 Cement Finisher	37.00	296.00	54.30	434.40		
1 Equip. Oper. (med.)	39.85	318.80	60.10	480.80		
2 Gas Engine Vibrator		49.60		54.56		
1 Concrete Conveyer		172.80		190.08	3.48	3.82
64 L.H., Daily Totals		$2305.20		$3443.44	$36.02	$53.80
Crew C-22	**Hr.**	**Daily**	**Hr.**	**Daily**	**Bare Costs**	**Incl. O&P**
1 Rodman Foreman	$45.00	$360.00	$73.85	$590.80	$43.14	$70.55
4 Rodmen (reinf.)	43.00	1376.00	70.55	2257.60		
.125 Equip. Oper. (crane)	40.95	40.95	61.75	61.75		
.125 Equip. Oper. Oiler	35.10	35.10	52.95	52.95		
.125 Hyd. Crane, 25 Ton		98.25		108.08	2.34	2.57
42 L.H., Daily Totals		$1910.30		$3071.18	$45.48	$73.12
Crew C-23	**Hr.**	**Daily**	**Hr.**	**Daily**	**Bare Costs**	**Incl. O&P**
2 Skilled Worker Foremen	$41.40	$662.40	$64.40	$1030.40	$39.52	$61.10
6 Skilled Workers	39.40	1891.20	61.25	2940.00		
1 Equip. Oper. (crane)	40.95	327.60	61.75	494.00		
1 Equip. Oper. Oiler	35.10	280.80	52.95	423.60		
1 Lattice Boom Crane, 90 Ton		1567.00		1723.70	19.59	21.55
80 L.H., Daily Totals		$4729.00		$6611.70	$59.11	$82.65
Crew C-23A	**Hr.**	**Daily**	**Hr.**	**Daily**	**Bare Costs**	**Incl. O&P**
1 Labor Foreman (outside)	$32.25	$258.00	$50.20	$401.60	$33.76	$51.80
2 Laborers	30.25	484.00	47.05	752.80		
1 Equip. Oper. (crane)	40.95	327.60	61.75	494.00		
1 Equip. Oper. Oiler	35.10	280.80	52.95	423.60		
1 Crawler Crane, 100 Ton		1586.00		1744.60		
3 Conc. bucket, 8 C.Y.		576.00		633.60	54.05	59.45
40 L.H., Daily Totals		$3512.40		$4450.20	$87.81	$111.26
Crew C-24	**Hr.**	**Daily**	**Hr.**	**Daily**	**Bare Costs**	**Incl. O&P**
2 Skilled Worker Foremen	$41.40	$662.40	$64.40	$1030.40	$39.52	$61.10
6 Skilled Workers	39.40	1891.20	61.25	2940.00		
1 Equip. Oper. (crane)	40.95	327.60	61.75	494.00		
1 Equip. Oper. Oiler	35.10	280.80	52.95	423.60		
1 Lattice Boom Crane, 150 Ton		1885.00		2073.50	23.56	25.92
80 L.H., Daily Totals		$5047.00		$6961.50	$63.09	$87.02

B10 Superstructure — RB1010-100 Floor Systems Cost

Table B1010-101 Comparative Costs ($/S.F.) of Floor Systems/Type (Table Number), Bay Size, & Load

	Cast-in-Place Concrete						Precast Concrete			Structural Steel					Wood	
Bay Size	1 Way BM & Slab B1010-219	2 Way BM & Slab B1010-220	Flat Slab B1010-222	Flat Plate B1010-223	Joist Slab B1010-226	Waffle Slab B1010-227	Beams & Hollow Core Slabs B1010-236	Beams & Hollow Core Slabs Topped B1010-238	Beams & Double Tees Topped B1010-239	Bar Joists on Cols. & Brg. Walls B1010-248	Bar Joists & Beams on cols. B1010-250	Composite Beams & C.I.P. Slab B1010-252	Composite Deck & Slab, W Shapes B1010-254	Composite Beam & DK., Lt. Wt. Slab B1010-256	Wood Beams & Joists B1010-264	Laminated Wood Beams & Joists B1010-265
Superimposed Load = 40 PSF																
15 x 15	15.40	14.60	13.05	12.60	16.05	—	—	—	—	—	—	—	—	—	12.51	13.40
15 x 20	15.50	15.35	13.50	13.50	16.20	—	—	—	—	11.53	13.65	—	19.30	—	15.78	13.40
20 x 20	15.75	16.20	13.95	13.50	16.25	16.70	21.31	24.00	—	11.79	14.69	—	20.75	—	14.93	13.61
20 x 25	16.05	17.05	15.05	14.90	16.40	16.95	20.63	22.95	—	13.61	16.87	22.00	21.05	19.35	—	—
25 x 25	16.15	17.50	15.45	15.20	16.20	17.35	21.63	24.65	—	14.41	17.95	22.65	23.45	18.95	—	—
25 x 30	16.50	18.40	16.50	—	17.00	17.65	20.73	23.35	22.35	14.84	18.70	24.55	23.45	19.20	—	—
30 x 30	18.35	19.95	17.45	—	17.55	18.50	21.28	24.75	23.75	14.71	18.73	24.50	25.70	19.35	—	—
30 x 35	19.10	21.50	18.65	—	18.15	18.85	21.16	24.35		16.48	21.00	25.55	28.10	20.20	—	—
35 x 35	20.55	22.60	18.95	—	18.35	19.75	22.26	24.90	—	16.92	21.55	26.60	28.60	21.30	—	—
35 x 40	21.00	23.95	—	—	19.05	20.40	22.40	25.75	21.00	—	—	28.80	29.50	23.40	—	—
40 x 40	—	—	—	—	19.70	21.20	23.40	26.75	23.55	—	—	—	—	—	—	—
40 x 45	—	—	—	—	20.50	21.75	—	—	—	—	—	—	—	—	—	—
40 x 50	—	—	—	—	—	—	—	—	22.15	—	—	—	—	—	—	—
Superimposed Load = 75 PSF																
15 x 15	15.60	14.85	13.25	12.65	16.10	—	—	—	—	—	—	—	—	—	15.53	16.63
15 x 20	16.10	16.65	14.05	14.10	16.90	—	—	—	—	12.52	15.54	—	21.05	—	19.41	18.30
20 x 20	17.15	17.60	14.65	14.20	17.15	17.00	22.41	25.10	—	13.38	16.90	—	23.20	—	19.19	18.06
20 x 25	17.60	19.30	16.05	15.20	17.30	17.45	20.63	23.95	—	15.57	17.76	24.55	25.65	19.95	—	—
25 x 25	17.55	19.05	16.30	15.90	17.25	17.95	22.38	24.65	—	15.36	19.50	26.00	26.75	20.65	—	—
25 x 30	17.75	20.10	17.60	—	17.65	18.25	21.32	24.70	22.35	15.75	20.00	27.65	28.05	20.35	—	—
30 x 30	19.85	21.80	18.60	—	18.15	19.00	22.18	25.85	23.75	16.92	21.55	27.30	30.00	20.75	—	—
30 x 35	20.15	22.70	19.90	—	18.45	18.85	22.21	24.50	—	19.35	24.20	29.40	31.45	21.75	—	—
35 x 35	22.40	23.20	20.40	—	19.35	20.35	23.16	26.80	—	20.75	26.05	30.40	32.95	23.95	—	—
35 x 40	22.80	24.95	—	—	19.95	21.20	—	—	22.22	—	—	32.75	33.95	25.15	—	—
40 x 40	—	—	—	—	20.25	22.15	—	—	24.20	—	—	—	—	—	—	—
40 x 45	—	—	—	—	20.70	22.70	—	—	—	—	—	—	—	—	—	—
40 x 50	—	—	—	—	—	—	—	—	22.25	—	—	—	—	—	—	—
Superimposed Load = 125 PSF																
15 x 15	15.90	15.70	13.75	13.05	16.40	—	—	—	—	—	—	—	—	—	22.65	24.25
15 x 20	16.80	18.05	14.70	14.95	17.35	—	—	—	—	14.83	18.50	—	24.15	—	27.20	27.90
20 x 20	17.95	18.20	15.70	15.00	17.45	17.40	—	—	—	16.74	19.30	—	26.05	—	36.55	26.90
20 x 25	18.75	19.70	17.25	16.20	18.25	17.80	—	—	—	16.68	20.80	27.55	30.25	24.20	—	—
25 x 25	20.50	20.65	17.45	16.70	19.25	18.55	—	—	—	18.00	22.75	29.65	32.10	21.70	—	—
25 x 30	20.60	21.90	18.35	—	19.05	18.80	—	—	—	19.15	24.10	32.45	31.40	23.55	—	—
30 x 30	21.25	23.15	19.35	—	19.30	19.35	—	—	—	21.30	26.95	32.35	35.05	24.70	—	—
30 x 35	22.65	24.95	20.65	—	19.40	20.00	—	—	—	22.00	27.45	36.15	37.50	25.45	—	—
35 x 35	24.10	26.10	21.05	—	19.45	20.85	—	—	—	24.35	28.65	35.45	38.70	27.85	—	—
35 x 40	24.35	26.40	—	—	19.95	22.05	—	—	—	—	—	38.00	40.10	28.45	—	—
40 x 40	—	—	—	—	21.20	22.40	—	—	—	—	—	—	—	—	—	—
40 x 45	—	—	—	—	21.50	23.70	—	—	—	—	—	—	—	—	—	—
40 x 50	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
Superimposed Load = 200 PSF																
15 x 15	16.65	16.85	14.35	—	17.05	—	—	—	—	—	—	—	—	—	43.35	42.05
15 x 20	18.30	19.55	15.15	—	18.05	—	—	—	—	—	—	—	29.35	—	39.10	37.20
20 x 20	19.75	19.95	16.05	—	18.35	18.50	—	—	—	—	—	—	30.65	—	—	39.30
20 x 25	20.35	21.65	17.90	—	19.30	18.75	—	—	—	—	—	33.90	33.30	26.85	—	—
25 x 25	22.40	23.90	18.10	—	20.05	19.15	—	—	—	—	—	35.10	36.45	28.70	—	—
25 x 30	22.55	24.15	19.30	—	20.25	20.45	—	—	—	—	—	37.60	38.70	28.80	—	—
30 x 30	23.80	25.05	20.55	—	20.40	21.30	—	—	—	—	—	39.45	46.40	29.30	—	—
30 x 35	24.20	26.65	—	—	21.05	22.15	—	—	—	—	—	42.65	44.95	29.35	—	—
35 x 35	26.30	27.50	—	—	20.95	22.60	—	—	—	—	—	43.85	49.55	32.05	—	—
35 x 40	26.50	28.45	—	—	21.45	24.10	—	—	—	—	—	47.55	50.65	34.40	—	—
40 x 40	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
40 x 45	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
40 x 50	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—

B10 Superstructure

B1010 Floor Construction

B1010 229 Precast Plank with No Topping

	SPAN (FT.)	SUPERIMPOSED LOAD (P.S.F.)	TOTAL DEPTH (IN.)	DEAD LOAD (P.S.F.)	TOTAL LOAD (P.S.F.)	COST PER S.F.		
						MAT.	INST.	TOTAL
1700	45	40	12	70	110	9.15	1.88	11.03

B1010 230 Precast Plank with 2″ Concrete Topping

	SPAN (FT.)	SUPERIMPOSED LOAD (P.S.F.)	TOTAL DEPTH (IN.)	DEAD LOAD (P.S.F.)	TOTAL LOAD (P.S.F.)	COST PER S.F.		
						MAT.	INST.	TOTAL
2000	10	40	6	75	115	7.25	5.20	12.45
2100		75	8	75	150	8.35	4.74	13.09
2200		100	8	75	175	8.35	4.74	13.09
2500	15	40	8	75	115	8.35	4.74	13.09
2600		75	8	75	150	8.35	4.74	13.09
2700		100	8	75	175	8.35	4.74	13.09
2800	20	40	8	75	115	8.35	4.74	13.09
2900		75	8	75	150	8.35	4.74	13.09
3000		100	8	75	175	8.35	4.74	13.09
3100	25	40	8	75	115	8.35	4.74	13.09
3200		75	8	75	150	8.35	4.74	13.09
3300		100	10	80	180	9.05	4.41	13.46
3400	30	40	10	80	120	9.05	4.41	13.46
3500		75	10	80	155	9.05	4.41	13.46
3600		100	10	80	180	9.05	4.41	13.46
3700	35	40	12	95	135	9.50	4.15	13.65
3800		75	12	95	170	9.50	4.15	13.65
3900		100	14	95	195	10.15	3.94	14.09
4000	40	40	12	95	135	9.50	4.15	13.65
4500		75	14	95	170	10.15	3.94	14.09
5000	45	40	14	95	135	10.15	3.94	14.09

B10 Superstructure

B1010 Floor Construction

B1010 241 W Shape Beams & Girders

	BAY SIZE (FT.) BEAM X GIRD	SUPERIMPOSED LOAD (P.S.F.)	STEEL FRAMING DEPTH (IN.)	FIREPROOFING (S.F. PER S.F.)	TOTAL LOAD (P.S.F.)	COST PER S.F. MAT.	INST.	TOTAL
6550	25x30	40	16	.632	50	9.10	2.65	11.75
6600		40	21	.76	90	12.55	3.57	16.12
6650		75	24	.857	125	14.95	4.21	19.16
6700		125	30	.983	175	18.70	5.40	24.10
6750		200	33	1.11	250	23.50	5.50	29
6800	30x25	40	16	.532	50	8.35	2.40	10.75
6850		40	21	.672	96	12.80	3.56	16.36
6900		75	24	.702	131	15.20	4.15	19.35
6950		125	27	1.020	175	19.75	5.50	25.25
7000		200	30	1.160	250	25	6.85	31.85
7100	30x25	40	18	.569	50	8.75	2.51	11.26
7150		40	24	.740	90	12.20	3.47	15.67
7200		75	24	.787	125	15.25	4.23	19.48
7300		125	24	.874	175	19	5.40	24.40
7400		200	30	1.013	250	23.50	5.30	28.80
7450	30x25	40	16	.637	50	9.10	2.66	11.76
7500		40	24	.839	90	12.90	3.72	16.62
7550		75	24	.919	125	15.65	4.42	20.07
7600		125	27	1.02	175	19.75	5.70	25.45
7650		200	30	1.160	250	25	5.70	30.70
7700	30x30	40	21	.52	50	9.35	2.63	11.98
7750		40	24	.629	103	14.45	3.93	18.38
7800		75	30	.715	138	17.20	4.64	21.84
7850		125	36	.822	206	22.50	6.30	28.80
7900		200	36	.878	281	25.50	5.60	31.10
7950	30x30	40	24	.619	50	9.75	2.80	12.55
8000		40	24	.706	90	13.20	3.67	16.87
8020		75	27	.818	125	15.60	4.33	19.93
8040		125	30	.910	175	20	5.70	25.70
8060		200	33	.999	263	24.50	5.55	30.05
8080	30x30	40	18	.631	50	10.45	2.97	13.42
8100		40	24	.805	90	14.25	4	18.25
8120		75	27	.899	125	17	4.73	21.73
8150		125	30	1.010	175	21	6.05	27.05
8200		200	36	1.148	250	25	5.75	30.75
8250	30x35	40	21	.508	50	10.70	2.94	13.64
8300		40	24	.651	109	15.85	4.27	20.12
8350		75	33	.732	150	19.25	5.15	24.40
8400		125	36	.802	225	24	6.60	30.60
8450		200	36	.888	300	31.50	6.85	38.35
8500	30x35	40	24	.554	50	9.40	2.66	12.06
8520		40	24	.655	90	13.80	3.79	17.59
8540		75	30	.751	125	17.25	4.68	21.93
8600		125	33	.845	175	21	5.90	26.90
8650		200	36	.936	263	27.50	6.10	33.60
8700	30x35	40	21	.644	50	10.10	2.90	13
8720		40	24	.733	90	14.55	4.02	18.57
8740		75	30	.833	125	18.30	4.98	23.28
8760		125	36	.941	175	21	5.95	26.95
8780		200	36	1.03	250	28	6.25	34.25

B10 Superstructure

B1010 Floor Construction

B1010 229 Precast Plank with No Topping

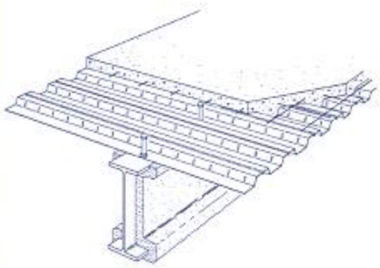
	SPAN (FT.)	SUPERIMPOSED LOAD (P.S.F.)	TOTAL DEPTH (IN.)	DEAD LOAD (P.S.F.)	TOTAL LOAD (P.S.F.)	COST PER S.F.		
						MAT.	INST.	TOTAL
1700	45	40	12	70	110	9.15	1.88	11.03

B1010 230 Precast Plank with 2″ Concrete Topping

	SPAN (FT.)	SUPERIMPOSED LOAD (P.S.F.)	TOTAL DEPTH (IN.)	DEAD LOAD (P.S.F.)	TOTAL LOAD (P.S.F.)	COST PER S.F.		
						MAT.	INST.	TOTAL
2000	10	40	6	75	115	7.25	5.20	12.45
2100		75	8	75	150	8.35	4.74	13.09
2200		100	8	75	175	8.35	4.74	13.09
2500	15	40	8	75	115	8.35	4.74	13.09
2600		75	8	75	150	8.35	4.74	13.09
2700		100	8	75	175	8.35	4.74	13.09
2800	20	40	8	75	115	8.35	4.74	13.09
2900		75	8	75	150	8.35	4.74	13.09
3000		100	8	75	175	8.35	4.74	13.09
3100	25	40	8	75	115	8.35	4.74	13.09
3200		75	8	75	150	8.35	4.74	13.09
3300		100	10	80	180	9.05	4.41	13.46
3400	30	40	10	80	120	9.05	4.41	13.46
3500		75	10	80	155	9.05	4.41	13.46
3600		100	10	80	180	9.05	4.41	13.46
3700	35	40	12	95	135	9.50	4.15	13.65
3800		75	12	95	170	9.50	4.15	13.65
3900		100	14	95	195	10.15	3.94	14.09
4000	40	40	12	95	135	9.50	4.15	13.65
4500		75	14	95	170	10.15	3.94	14.09
5000	45	40	14	95	135	10.15	3.94	14.09

B10 Superstructure

B1010 Floor Construction



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.

System Components	QUANTITY	UNIT	COST PER S.F. MAT.	INST.	TOTAL
SYSTEM B1010 256 2400					
20X25 BAY, 40 PSF S. LOAD, 5-1/2" SLAB, 17-1/2" TOTAL THICKNESS					
Structural steel	4.320	Lb.	7.26	1.73	8.99
Welded shear connectors 3/4" diameter 4-7/8" long	.163	Ea.	.12	.30	.42
Metal decking, non-cellular composite, galv. 3" deep, 22 gauge	1.050	S.F.	3.08	.90	3.98
Sheet metal edge closure form, 12", w/2 bends, 18 ga, galv	.045	L.F.	.26	.10	.36
Welded wire fabric rolls, 6 x 6 - W1.4 x W1.4 (10 x 10), 21 lb/csf	1.000	S.F.	.20	.34	.54
Concrete ready mix, light weight, 3,000 PSI	.333	C.F.	2.58		2.58
Place and vibrate concrete, elevated slab less than 6", pumped	.333	C.F.		.47	.47
Finishing floor, monolithic steel trowel finish for finish floor	1.000	S.F.		.78	.78
Curing with sprayed membrane curing compound	.010	C.S.F.	.06	.08	.14
Shores, erect and strip vertical to 10' high	.020	Ea.		.38	.38
Sprayed mineral fiber/cement for fireproof, 1" thick on beams	.483	S.F.	.28	.43	.71
TOTAL			13.84	5.51	19.35

B1010 256 Composite Beams, Deck & Slab

	BAY SIZE (FT.)	SUPERIMPOSED LOAD (P.S.F.)	SLAB THICKNESS (IN.)	TOTAL DEPTH (FT.-IN.)	TOTAL LOAD (P.S.F.)	COST PER S.F. MAT.	INST.	TOTAL
2400	20x25	40	5-1/2	1 - 5-1/2	80	13.85	5.50	19.35
2500	RB1010 -100	75	5-1/2	1 - 9-1/2	115	14.40	5.55	19.95
2750		125	5-1/2	1 - 9-1/2	167	17.70	6.50	24.20
2900		200	6-1/4	1 - 11-1/2	251	19.85	7	26.85
3000	25x25	40	5-1/2	1 - 9-1/2	82	13.70	5.25	18.95
3100		75	5-1/2	1 - 11-1/2	118	15.30	5.35	20.65
3200		125	5-1/2	2 - 2-1/2	169	15.95	5.75	21.70
3300		200	6-1/4	2 - 6-1/4	252	22	6.70	28.70
3400	25x30	40	5-1/2	1 - 11-1/2	83	14	5.20	19.20
3600		75	5-1/2	1 - 11-1/2	119	15.10	5.25	20.35
3900		125	5-1/2	1 - 11-1/2	170	17.60	5.95	23.55
4000		200	6-1/4	2 - 6-1/4	252	22	6.80	28.80
4200	30x30	40	5-1/2	1 - 11-1/2	81	13.95	5.40	19.35
4400		75	5-1/2	2 - 2-1/2	116	15.15	5.60	20.75
4500		125	5-1/2	2 - 5-1/2	168	18.40	6.30	24.70
4700		200	6-1/4	2 - 9-1/4	252	22	7.30	29.30
4900	30x35	40	5-1/2	2 - 2-1/2	82	14.65	5.55	20.20
5100		75	5-1/2	2 - 5-1/2	117	16.05	5.70	21.75
5300		125	5-1/2	2 - 5-1/2	169	19	6.45	25.45
5500		200	6-1/4	2 - 9-1/4	254	22	7.35	29.35