TECHNICAL REPORT 2

BRIDGESIDE POINT II

PITTSBURGH, PA

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Structural Option

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EXECUTIVE SUMMARY

In this second technical report alternate floor systems for Bridgeside Point II are investigated through preliminary design analysis. The designs of five systems (including the existing system) are analyzed and compared. Several factors provide the comparative basis and they include: constructability, fire protection, cost, serviceability, and architecture. The existing floor system is composite steel, which adequately carries the large live loads and achieves the required lengthy spans. The other four systems being analyzed are as follows:

- Two-way post-tensioned slab
- Two-way flat slab
- Girder-Slab
- Non-composite steel with lightweight concrete

Upon review of these systems, the post-tensioned and two-way systems seem to provide the best alternatives to the composite steel system. Both can handle the large spans and loads while they also reduce the amount of total floor thickness. However, changing the building from steel to concrete could have significant implications in both the architecture and substructure of the building. The lateral system would no longer be a braced-frame system and the necessary shear walls would have to be seamlessly incorporated into the floor plan. Technical report three will address these issues and provide a better understanding of the lateral systems, and confirm or reject the use of a concrete floor system.

INTRODUCTION: BRIDGESIDE POINT II

The Bridgeside Point II project consists of five above grade stories with a combination of office and laboratory space. It is located in the Pittsburgh Technology Center, which is just east of downtown Pittsburgh, Pennsylvania. The building conveys a feeling of progression from a historic steel mill town to a fast-paced, innovation driven city through its use of clean lines, visible lateral system, and open plan. A glass curtain wall lends itself for a feeling a transparency on the upper floors, while dense, pre-cast panels wrap the ground floor.

The building is approximately 150,000 square feet and reaches a height of 75 feet above grade. The building floor template is an open plan with a design core capable of housing office and laboratory spaces as each floor is roughly 15 feet floor to floor. A typical bay is 30 feet by 32 feet, and is comprised of composite steel with a concrete slab on deck. The lateral system is a series of braced frames, two in the east – west building direction and three in the north – south building direction. The foundation system is a driven pile system. A typical pile cap hosts between three and seven piles and has a thickness of 3'-6" to 4'-6". The ground floor is a reinforced slab on grade with grade beams around the perimeter.

Flexibility is the main concept this building expresses. At the time of design, no definite tenant has been selected; therefore two design criterions are at the forefront to create a flexible space. The desired large bays require a heavy uniform live load, thus larger structural members; and placement of the lateral system is limited.

This report examines four alternate floor systems for Bridgeside Point II. Discussion of each system includes a discussion of the system's viability for implementation in the building. Factors for discussion include: constructability, fire protection, cost, serviceability, and architecture. This paper's main goal is to stimulate thought and provide a better understanding of the framing options available for further consideration as part of my thesis proposal. It should be noted that all calculations and designs are schematic in nature, as this is not an exhaustive analysis of each floor system.

Codes and References

The 2006 International Building Code as amended by the City of Pittsburgh.

The Building Code Requirements for Structural Concrete (ACI 318-05), American Concrete Institute.

Steel Construction Manual, Thirteenth Edition, American Institute of Steel Construction.

Minimum Design Loads for Buildings and Other Structures (ASCE 7-05), American Society of Civil Engineers.

Deflection Criteria

L/240 Total Load and L/360 Live Load

L/600 Curtain Wall Load

EXISTING COMPOSITE STEEL SYSTEM

Floor System

The floor system of Bridgeside Point II is a composite system with a typical bay size of 30'-0" by 32'-0" (Figure 1). A 3" concrete slab rests on 3" composite steel decking. Shear studs ¾" diameter (5 ½" long) are used to create composite action. This assembly provides a 1.5 to 2 hour fire rating which meets IBC requirements. Infill beams are W21x44 spaced at 10'-0" center to center which frame into W24x62 girders. This report will use this typical bay as a benchmark for comparison of the floor systems under investigation.

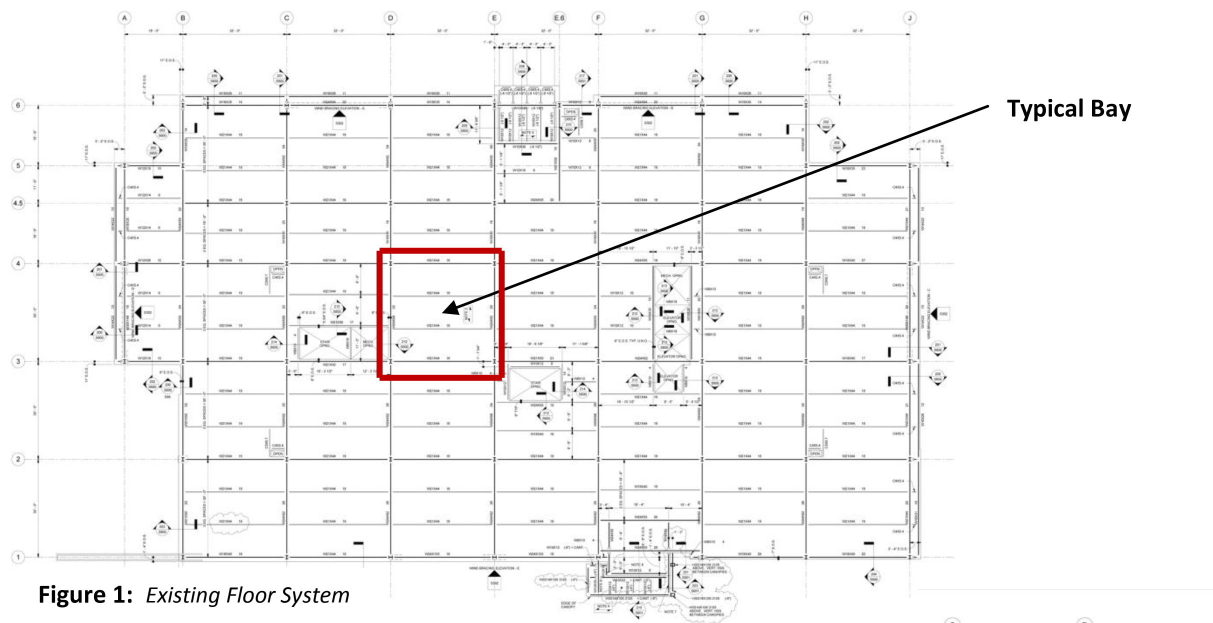


Figure 1: *Existing Floor System*

Lateral System

Large braced frames make up the building's lateral load resisting system. In order to increase the flexibility of the building plan, the perimeter was chosen for the bracing. Four of the five bracing frames are exposed via windows. In these bays, large HSS8x8x3/8 and HSS10x10x1/2 provide the bracing at the second through fifth floors and are K-Braces, which create a two story "X" in the window (Figure 2). On the first floor these four frames have an eccentric brace, whereas the large fifth frame is two bays wide and is comprised of all W-shape eccentric braces.

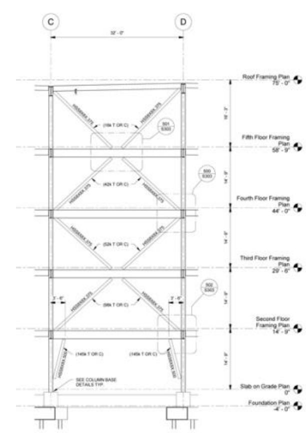


Figure 2: *Typical Lateral Frame*

Foundations

A driven pile system with pile caps containing between two and nine piles provides the foundation system for the building with an end bearing capacity of 105 to 130 tons per pile. The pile caps vary in thickness from 3'-6" to 4'-6" and have between 9 and 12 No. 9 reinforcing bars. Depending on their location within the site, they are driven to a depth of 45 to 55 feet. These piles support the framing system as well as a 4" thick concrete slab on grade.

Pro-Con Analysis: Existing Composite Steel System

This system is adequate to handle the structural requirements of Bridgeside Point II. It is very effective in its ability to handle lengthy spans and heavy distributed loads, both of which are essential to the building's success. The end result is an open floor plate with 14'-6" floor to floor heights. The thick deck and slab combination meet the requirement for a 2 hour fire rating (Figure 3). The large steel sections minimize the deflection per floor, and the pile driven foundation is more than acceptable in handling a heavy superstructure. The construction of the composite system is a very efficient method. Formwork and shoring is not required, and the minimal number of openings in the slab results in fast slab pouring. Steel erection is also much more efficient and faster than forming and pouring concrete beams and columns. The overall system cost is relatively cheap (approximately \$48.00 per ft^2) and relatively easy to construct.

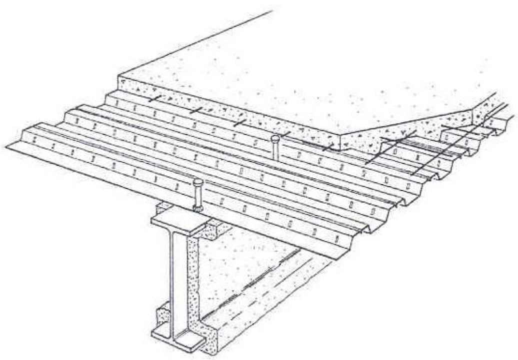


Figure 3: *Composite System*

However, the deep sections and thick deck create a total floor depth of nearly 30 inches, as well as an enormous weight for the foundation to shoulder.

Overall, this system is a very good choice for this project as it meets both the architectural and structural requirements of the project.

ALTERNATE FLOOR SYSTEMS

For this report four alternate floor systems were investigated for Bridgeside Point II. The decision to analyze these systems stemmed from a desire to achieve smaller total floor thickness with the hope of adding an additional floor for more leasable space, and to provide the tenant with the maximum amount of usable space. The systems are listed in the order in which they will be discussed.

- Two-way post-tensioned slab
- Two-way flat slab
- Girder-Slab
- Non-composite steel with lightweight concrete

Various reference manuals were used for investigation of design and cost analysis.

- AISC Specification for Structural Steel Buildings 13th Edition
- ACI 318-05 Building Code and Commentary
- VSL Post-Tensioned Slabs
- Girder-Slab Design Guide v1.3
- Manual for the Design of Hollow Core Slabs 2nd Edition
- RS Means Assemblies Cost Data, 2006 Edition
- RS Means Square Foot Cost Data, 2007 Edition

The concrete floor systems utilized the existing column grid; however, the Girder-Slab and Non-composite steel systems required reallocation of columns (Figure 4). Justification for this comes from the much smaller total floor thicknesses and member sizes that can be achieved. It should be noted that with the additional columns comes additional footings. These footings will be carrying far less load, but more piles and pile caps will need to be constructed. The viability of this will be overviewed in detail later in this report (see Figure 11 on page 12).

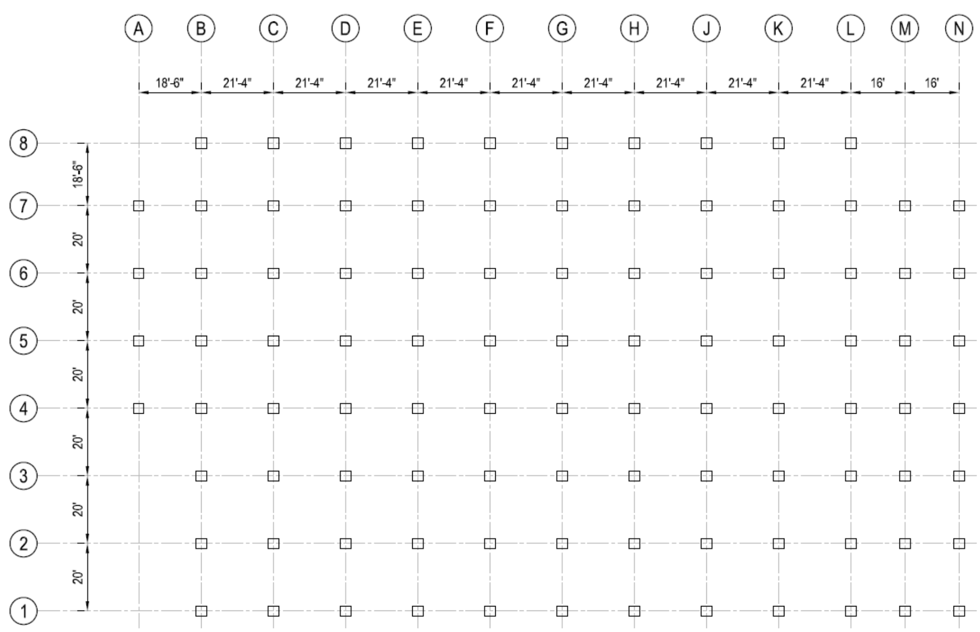


Figure 4: *Redefined column grid*

TWO-WAY POST-TENSIONED SYSTEM

This system uses a two-way post-tensioned slab and concrete columns. The square-like bay affords itself for a very efficient design. For this report, only a typical interior bay was designed and analyzed (calculations are provided in Appendix B). The design intent was minimizing floor thickness while also avoiding drop panels. This system achieved a floor thickness of 9 inches; however, to resolve punching shear and keep the floor slab to a minimum, 2 inch thick by 38 inch square drop panels needed to be added around the columns. Even with the thin thickness, a 2 hour fire rating can be attained. The post-tensioned system is very effective in carrying heavy loads while spanning long distances. It would be worth investigating the length at which this system could achieve under current loading while still providing a slim floor thickness, and will be done in a future report. This would allow for even more open spaces and a possible reduction in the number of driven piles systems needed.

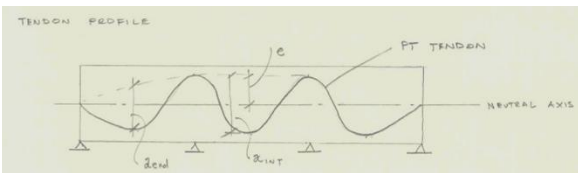


Figure 5: *Tendon Profile*

Figure 6: *Typical tendon layout. Taken from www.concreteconstruction.net*

Pro-Con Analysis: Two-Way Post-Tensioned Slab

This system and its ability to achieve large spans through a thin slab thickness allows for an open plan, which could possibly be increased for even more space. The system also justifies adding an additional as floor to floor heights could be reduced, thus the amount of material needed for an additional floor is minimum. Larger drop panels could possibly reduce the overall thickness more and reduce the column size; however, for this design drop panels are minimized where possible to reduce their impact on the floor plan. Cost-wise this system is expensive, but with greater spans the cost would go down, making it much more efficient and economical.

As mentioned above, the concrete system itself needs a completely different lateral system verses the current steel system. The additional loads from the concrete and resulting lateral system could add a fair amount of load to the foundations, but I do not foresee this as a significant issue. However, this is a very difficult system to construct and requires a very experienced design and construction team (Figures 5 & 6). Supervision of the post-tensioning process is mandatory and specifications may require a testing agency to monitor construction. Also, adding openings after installation is prohibited as it may severe a tendon, so this means a fair amount of pre-construction planning is required by the client.

Overall, this system is a viable option for this project as it meets several structural and architectural requirements of the project, but further investigation will need to be conducted to determine whether the lateral system and larger bays fit the current floor plan.

TWO-WAY FLAT SLAB SYSTEM

This system uses a two-way reinforced concrete slab. The existing, nearly square bay afforded a perfect canvas to use such a system. For this report, only a typical interior bay was designed and analyzed (calculations are provided in Appendix C). The initial design intent was to avoid the use of drop panels; however, this schematic design did include these panels to eliminate punching shear and minimize slab thickness (Figure 7). Column capitals were not designed, but were considered and disregarded for cost and constructability reasons. The design yielded a total floor thickness of 14.5 inches; however, ballasts or other means will need to be provided to conceal the mechanical ductwork, which will inevitably increase the total floor thickness. The governing force in the design was punching shear, which given the large span and heavy distributed load, it was expected to control.

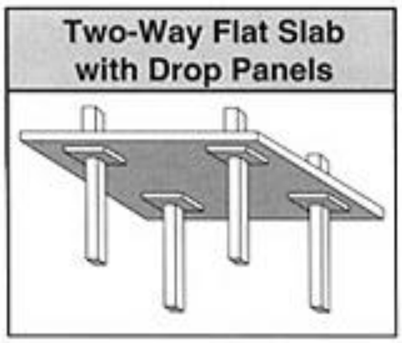


Figure 7: *Two-Way Slab with Drop Panels. Taken from www.crsi.org*

Pro-Con Analysis: Two-Way Slab System

This system performs very well with the current square grid at Bridgeside Point II. No new columns are needed for this system which lends itself well to a true alternative. The minimal total floor thickness is nearly half that of the composite steel system, even with the addition of drop panels. The drop panels allow for smaller columns, which were not analyzed for this report, and also reduce the amount of reinforcement and concrete needed to achieve the same slab strength.

However, the drop panels do cause some problems with the ceiling and mechanical components as they would need to adjust to the slab thickness increase. The concrete system itself needs a completely different lateral system verses the current steel system. One of the features of this building is its dramatic "X" in the window (Figure 8); a concrete system would not have this feature because a shearwall would most likely be tucked into the elevator core and/or stairwells. The additional weight of the concrete system could burden the current foundation system, but I would expect its impact to minimal in nature. A deeper look at the lateral systems will be discussed in Technical Report 3.

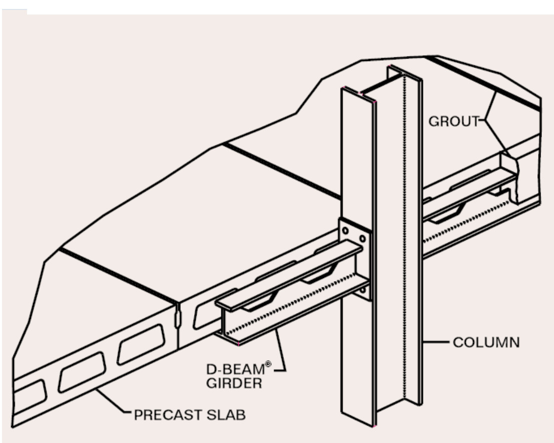
Overall, this system is a viable option for this project as it meets several structural requirements of the project, but further investigation will need to be conducted to determine whether the lateral system fits the current floor plan.

Figure 8: *Visible lateral system in its existing design*

GIRDER-SLAB SYSTEM

This system is a very unique system because it uses a modified steel shape to support precast planks (Figures 9a and 9b) that are grouted together to make the system integral. In order to fully utilize this system, the grid was modified considerably. The justification for this was that with thinner floor thickness and less weight, it could be possible to add an additional floor, which would allocate another 33,000 ft^2 to the tenant and owner for relatively low construction costs. For this report, only a typical interior bay was designed and analyzed (calculations are provided in Appendix D). The design did result in a 10 inch floor thickness and did reduce the column size by almost half the weight, but an additional 40 columns and 40 footings are required to achieve this design.

Figures 9a & 9b: *Girder-Slab details. Taken from www.girder-slab.com*



Pro-Con Analysis: Girder-Slab System

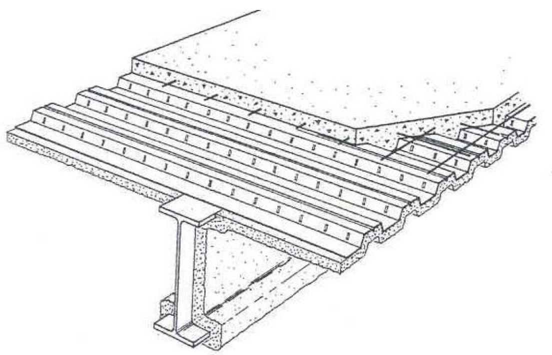
This is a very fast system to erect as everything is brought to the site and hoisted into position. It is the most lightweight system analyzed in this report and the most innovative. The current lateral system could still work, but would need to be analyzed in more detail. Only the steel girders and columns would need fireproofing which does help on cost.

However, the system is very costly and is not very efficient spanning large distances. The additional columns and footings seem to offset the reduced floor thickness. If shallow footings were present, this could prove to be a nice alternative; however, the cost to add additional driven pile footings is a very steep cost as seen in the cost comparison on page 12.

This system is not a viable alternative, but could easily be used in the residential market where loads and spans are much smaller, or in an area that permitted shallow footings.

NON-COMPOSITE STEEL SYSTEM

This system is a fairly basic steel system as it is uses lightweight steel shapes and a thin deck/slab configuration (Figure 10). In order to fully utilize this system, the grid was modified considerably. The justification for this was that with thinner floor thickness and less weight, it could be possible to add an additional floor, which would allocate another 33,000 ft^2 to the tenant and owner for relatively low construction costs. For this report, only a typical interior bay was designed and analyzed (calculations are provided in Appendix E). The design did result in a 18.5 inch floor thickness and did reduce the column size by almost half the weight, but an additional 40 columns and 40 footings are required to achieve this design. Infill beams are W12x19 and girders are W14x34 which do provide enough space to route most mechanical ductwork, and fire protection can easily reach a 2 to 3 hour rating.



Figures 10: *Non-composite system*

Pro-Con Analysis: Non-composite Steel System

This is a relatively fast system to erect because shear studs are not required. The lightweight characteristics and the thin floor thickness make it a very desirable choice. The shorter spans reduce the possibility of significant vibrations. This system would utilize the existing lateral system, which meets both the structural and architectural concepts put forth in this project.

However, the main drawback is the additional substructures and columns needed to reduce the span lengths. The open plan would be somewhat confined even if an additional floor was added.

This system is not a viable alternative, but could easily be used in a design where shorter spans are present. For the purposes of this project, reducing the bay area for an additional floor is not justifiable.

SYSTEM COMPARISON

The following (Figure 11) compares each system on several criterion.

Floor System Comparison - Typical Bay

Criterion	Floor Systems				
	Existing Composite Steel	Post-Tensioned Slab	Two-Way Slab	Girder-Slab	Non-composite Steel
System Weight (psf)	72	112	137	75	54
Slab Depth (in)	6.0	9.0	11.0	10.0	4.5
Total Depth (in)	29.7	11.0	14.5	10.0	18.5
Column Size	W12x87	18"x18"	18"x18"	W10x49	W10x49
Constructability	Medium	Hard	Medium	Medium	Easy
Foundation Impact	-	Little	Little	* Yes	* Yes
Fire Rating (hr)	1.5 to 2	2	2	2 to 3	1.5 to 2
Materials Cost per ft^2	$12.45	$17.22	$9.00	$12.12	$11.47
Labor Cost per ft^2	$5.65	$8.95	$9.15	$6.20	$6.06
Column Cost per ft^2	$5.92	$6.31	$6.31	$20.03	$20.03
Foundation Cost per ft^2	$23.96	$23.96	$23.96	$86.13	$86.13
Total Cost per ft^2	**$47.98**	**$56.44**	**$48.42**	**$124.48**	**$123.69**
Possible Alternative	-	Yes	Yes	No	No
Additional Study	-	Yes	Yes	No	No

* System requires additional columns and footings

Figures 11: *Side by side system comparison*

From a cost standpoint, the existing composite system is the cheapest and is fairly easy to construct. Given the context of the city (Pittsburgh), steel is a preferred choice; and, in the design of Bridgeside Point II, the architect takes advantage of it in the lateral system. The contractors, designers, and workers are very familiar with steel construction, so working with concrete could present a challenge. However, it is a very easy problem to overcome as the Grant Street Transportation Center in Pittsburgh is currently being constructed using a post-tensioned system. Based on the research and design of these systems, I believe that either concrete system would be a viable option.

CONCLUSION

The schematic designs presented in this report are intended to stimulate the considerations engineers encounter on each project. A schematic design was presented for each system and examined for is feasibility as an alternative to the current composite steel system. The results of the analysis suggested that reducing bay size is not a viable option given the architectural intent of an open plan and due to the additional columns and foundations needed. Even with the prospect of an additional floor to offset these conditions and costs, smaller bay sizes and expensive driven pile footings reduce the marketability of the design to the building owner.

However, the post-tensioned and two-way slab systems appear to be viable alternatives to the composite steel without considering the lateral and constructability issues. Both take advantage of the current grid and have minimal impact of the foundation system. The cost of each system is similar to the existing system, and proposing an additional floor to the building is a good possibility because no additional columns or footings would be required. One drawback at present is the uncertainty of the lateral system, which requires a more detailed analysis. Another would be the difficulty with post-tensioned construction. These concrete systems, along with the existing composite steel system will be analyzed further in Technical Report 3. At that point, a more conclusive decision will be made on the viability of each alternate concrete system.

All design values were done in accordance with the applicable codes. Detailed notes, tables, and figures are provided in the appendices for further review. Any questions and/or comments should be directed to Antonio Verne through email: adv118@psu.edu.

APPENDIX A: BUILDING LAYOUT

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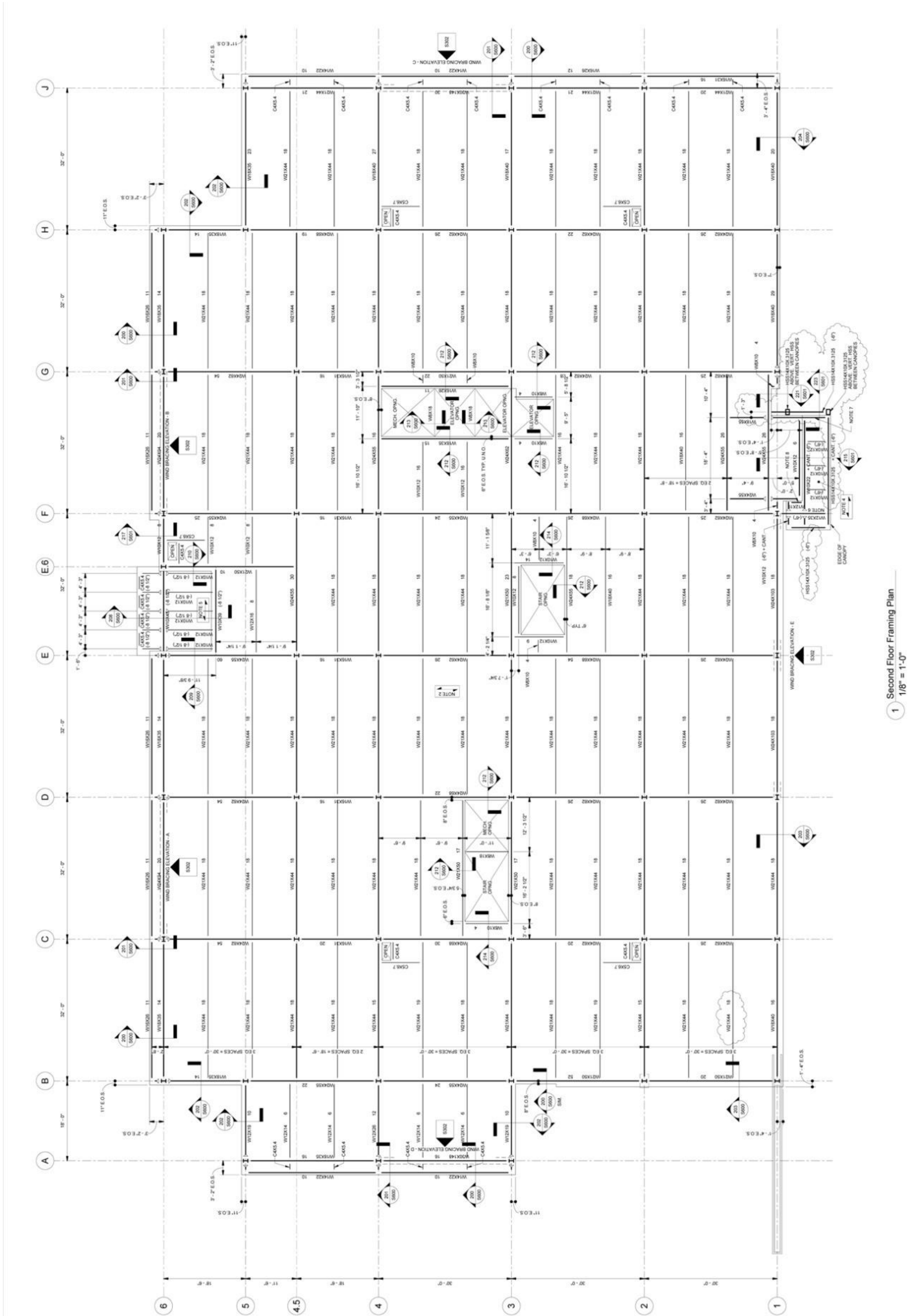
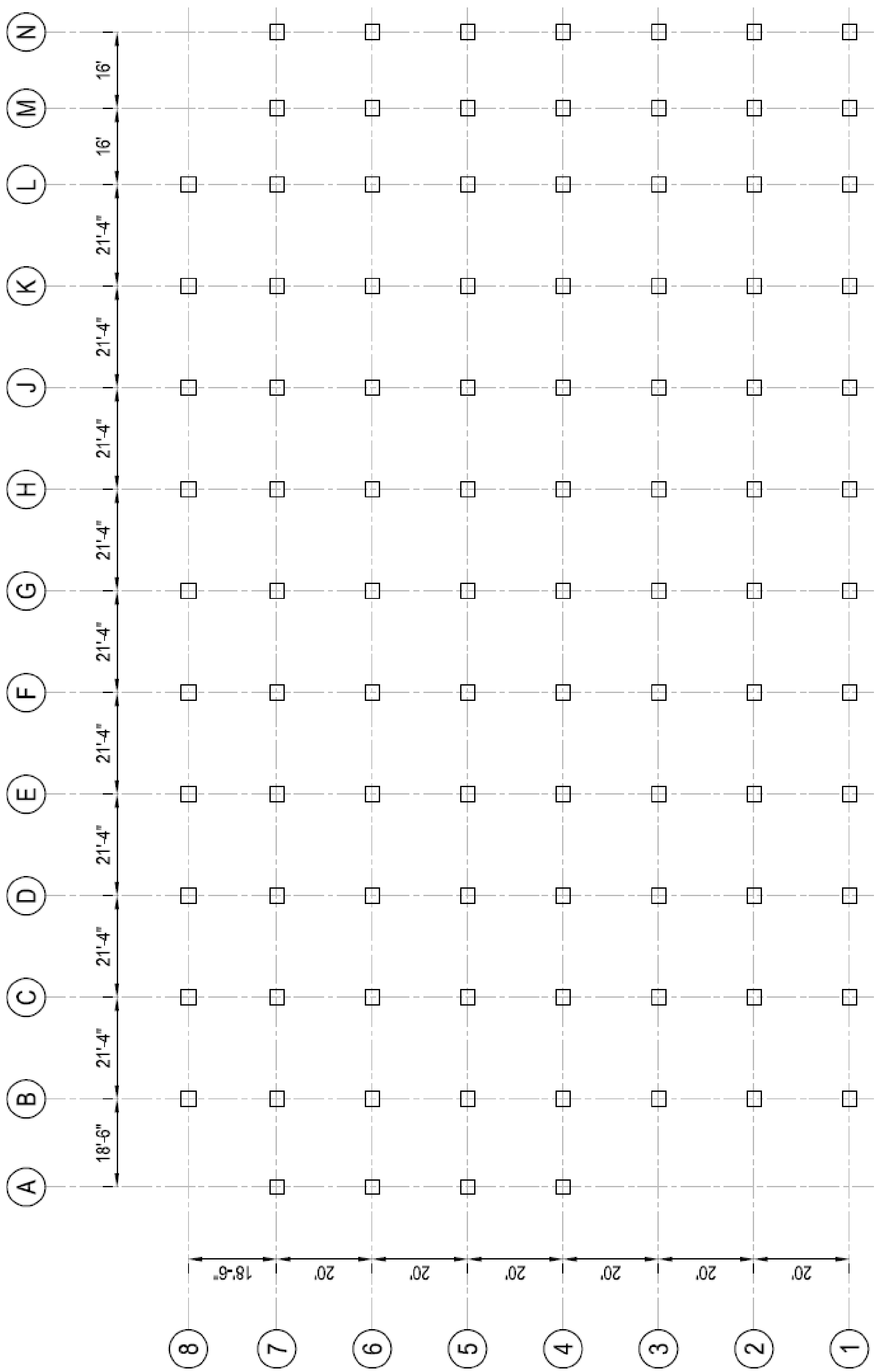
Typical Floor Layout



Alternative Floor Layout



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APPENDIX B: POST-TENSIONED SLAB ANALYSIS

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TWO-WAY POST-TENSIONED SLAB

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

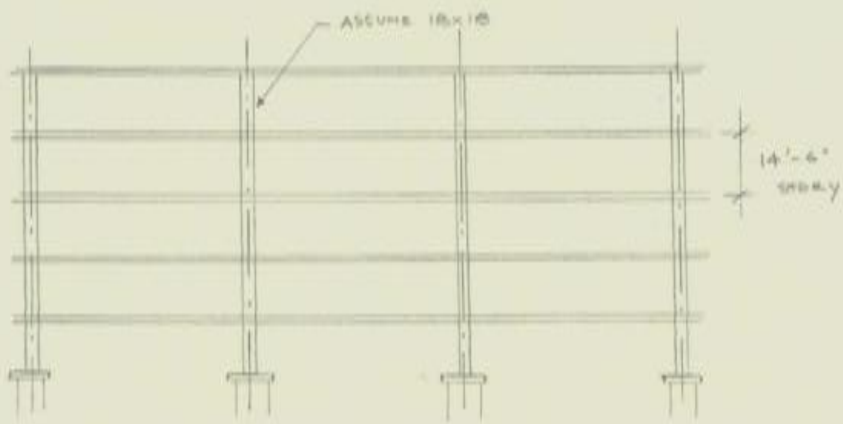
$w_c = 150$ pcf

LIVE LOAD = 60 PSF (OFFICE / LAB)

PARTITION LOAD = 20 PSF

SUPERIMPOSED DL = 10 PSF

TYPICAL BAY :



ADDITIONAL MATERIALS :

$f'_{ci} = 3000$ psi

UNBONDED TENDONS : $\frac{1}{2}"$ ϕ, 7-WIRE STRANDS, $A = 0.153$ in^2
$f_{pu} = 270,000$ psi

LOSSES ≈ 15 ksi (FROM PRESTRESS)

$f_{se} = (0.7)(270 \text{ ksi}) - 15 \text{ ksi} = 174$ ksi

$P_{eff} = (0.153 \text{ in}^2)(174 \text{ ksi}) = 26.6^{k} /$ TENDON

SLAB THICKNESS : $L/h = 45$ SPAN = 32 ft

$h = (32 \text{ ft})(12 \text{ in}/\text{ft})/45 = 8.533$ in $\longrightarrow$ TRY 9 in

DL = 112.5 psf (SW)

SUPERIMPOSED = PARTITION = 30 psf / LL = 60 psf

LL REDUCTION = $L_o \sqrt{0.25 + 15/\sqrt{A_I}}$ A_I : 3840 ft^2 INT. BAY

$L = 0.49 L_o = (0.49)(60 \text{ psf}) = 30$ psf

FOR E-W FRAME
AREA = $(30 \text{ ft})(12 \text{ in}/\text{ft})(9 \text{ in}) = 3240$ in^2
$S = bh^2/6 = (360 \text{ in})(9 \text{ in})^2/6 = 4860$ in^3

DESIGN PARAMETERS :

AT TIME OF JACKING : $f'_{ci} = 3000$ psi

$C = (0.6)(3000 \text{ psi}) = 1800$ psi
$T = (3)\sqrt{3000 \text{ psi}} = 164$ psi

AT SERVICE LOADS : $f'_c = 5000$ psi

$C = (0.45)(5000 \text{ psi}) = 2250$ psi
$T = (6)\sqrt{5000 \text{ psi}} = 424$ psi

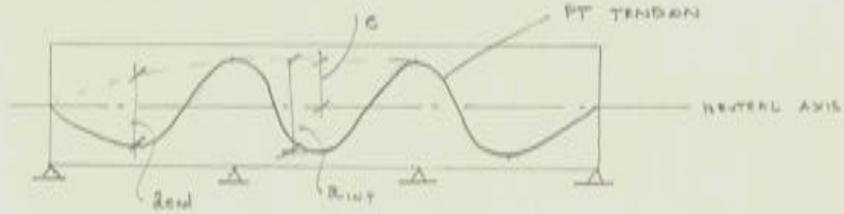
PRECOMPRESSION LIMITS : P/A = 125 psi MIN / 300 psi MAX

TARGET LOAD BALANCES : 75% $\Longrightarrow$ $W_{DL} = (0.75)(112.5) = 84.4$ psf

COVER REQUIREMENTS : 2-HOUR RATING

— RESTRAINED = 3/4" —

TENDON PROFILE



EXTERIOR SUPPORT ANCHOR : 4.5"
INTERIOR SUPPORT - TOP : 8.0"
INTERIOR SPAN - BOTTOM : 1.0"
END SPAN BOTTOM : 1.75"

e_{INT} = 7"

e_{end} = 4.5"

PRESTRESS FORCE

$W_b = (84.4\ psf)(30\ ft) = 2.532\ ^k/ft$

FORCE = $P = W_b l^2 / 8 e_{INT} = (2.532\ ^k/ft)(30\ ft)^2 / (8)(7/12\ ft)$

$P = 556^k$ FOR INTERIOR BAY

TENDONS = $556^k / 26.6^k$ /TENDON

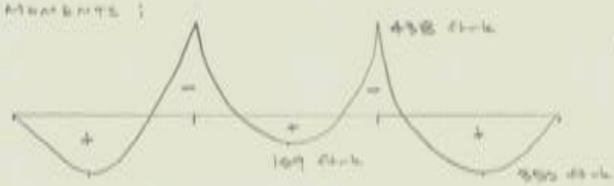
TENDONS = 21 TENDONS

$P_{ACTUAL} = 559^k$ & $W_b = 2.59\ ^k/ft$

PRECOMPRESSION STRESS : $P/A = (559^k / 3240\ in^2)(1000) = 172\ psi$ ✓ OK

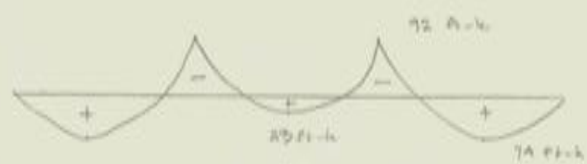
$\boxed{\text{EFFECTIVE STRESS FOR E-W FRAME} = 559^k}$

MOMENTS :



DEAD LOAD :

$W = (30\ ft)(142.5\ psf)$
$W = 4.275$ KLF



LIVE LOAD :

$W = (80\ psf)(30\ ft) = 2.4\ klf$

BALANCING : 1 = 2.55 k/ft

STAGE 1 : IMMEDIATELY AFTER JACKING (DL + PT)

INTERIOR MIDSPAN STRESSES

$f_{TOP} = (-M_{DL} + M_{BAL})/S - P/A$

$= (-109 + 65)(12)(1000)/4860 - 172 = -281$ psi $< 0.6 f'_{ci}$ ✓ OK

$f_{BOT} = (M_{DL} - M_{BAL})/S - P/A$

$= (109 - 65)(12)(1000)/4860 - 172 = -49$ psi ✓ OK

INTERIOR SUPPORT STRESSES

$f_{TOP} = (M_{DL} - M_{BAL})/S - P/A$

$= (438 - 261)(12)(1000)/4280 - 172 = 324$ psi $> 3\sqrt{f'_{ci}}$

SO : $0.6 f_y$ REINF.

$f_{BOT} = (-M_{DL} + M_{BAL})/S - P/A$

$= (-438 + 261)(12)(1000)/4280 - 172 = -787$ psi $< 0.6 f'_{ci}$ ✓ OK

STAGE 2 : STRESSES AT SERVICE LOAD (DL + LL + PT)

MIDSPAN

$f_{TOP} = -360$ psi $< 0.45 f'_c$ ✓ OK

$f_{BOT} = 16$ psi $< 6\sqrt{f'_c}$ ✓ OK

SUPPORT

$f_{TOP} = 582$ psi $> 6\sqrt{f'_c}$ ⇒ $0.6 f_y$ REINF.

$f_{BOT} = -926$ psi $< 0.45 f'_c$ ✓ OK

ULTIMATE STRENGTH :

$M_1 = Pe = (554^k)(3.5\text{ in})/12 = 163$ ft-k

$M_{SEC} = M_{BAL} - M_1 = 261 - 163 = 98$ ft-k

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

@ MIDSPAN : $M_U = (1.2)(109) + (1.6)(23) + 49 = 217$ ft-k

@ SUPPORT : $M_U = (1.2)(-438) + (1.6)(-92) + (98) = -575$ ft-k

BONDED REINFORCEMENT:

$f_t = 16$ psi $< 2\sqrt{f'_c} = 141$ psi ✓ OK NO REINF. NEEDED

NEGATIVE MOMENT:

$A_{s,min} = 0.00075 A_{cf}$

INTERIOR SUPPORTS: $A_{cf} = (9\text{ in})\left[(64\text{ ft})/2\right](12) = 3456\text{ in}^2$

$A_{s\,min} = (0.00075)(3456\text{ in}^2) = 2.6\text{ in}^2$

13 – #4 TOP (2.6 in^2)

REQ'D: 6h IN EACH SIDE OF SUPPORT (SPAN)
4 BARS MIN IN EACH DIRECTION
PLACE WITHIN 13 IN OF SUPPORT FACE, E.S.
MAX SPACING = 12 IN

CHECK MINIMUM REQUIREMENTS:

$\phi M_n = \phi(A_s f'_y + A_{ps} f_{ps})(d - a/2)$; $f'_y = 1.6 f_y = 96$ ksi

$A_{ps} = (0.153\text{ in}^2)(21\text{ TENDONS}) = 3.213\text{ in}^2$
$f_{ps} = f_e + 10000 + \left[(f'_c b d)/(A_{ps})(300)\right]$
$= 174{,}000 + 10000 + \left[(5000)(360)d / (300)(3.213)\right]$

$f_{ps} = 184{,}000 + 1867\,d$

$a = (A_s f_y + A_{ps} f_{ps})/0.85 f'_c b \Rightarrow a = 0.48$ in

AT SUPPORTS: $d = 9'' - 3/4'' - 1/4'' = 8''$

$f_{ps} = 198{,}789$ psi

$\phi M_n = (0.9)(732.8)(7.76)/12$

$\phi M_n = 426.5$ ft-k < 575 ft-k

A_s REQ'D $= 5.91\text{ in}^2$

$\boxed{18 - \#4}$ TOP AT INTERIOR SUPPORTS

E-W FRAME

FOR N-S FRAME

AREA = (32 ft)(12 in/ft)(9 in) = 3456 in^2

S = (384 in)(9 in)^2/6 = 5184 in^3

w_b = (84.4 psf)(32 ft) = 2.70 k/ft

FORCE = P = $w_b l^2 / 8 a_{INT}$ = (2.7)(36)^2/(8)(7/12)

P = 521 k

TENDONS = 20 TENDONS

P_{ACTUAL} = 532 k w_b = 2.76 k/ft

P/A = 154 psi > 125 psi ✓ ok

$\boxed{\text{EFFECTIVE STRESS FOR N-S FRAME = 532 k}}$

DEAD LOAD MOMENTS:

w = (32 ft)(142.5 psf) = 4.56 klf

M^- = −410 ft-k

M^+ = 103 ft-k

LIVE LOAD MOMENTS:

w = (32 ft)(30 psf) = 0.96 klf

M^- = −87 ft-k

M^+ = 22 ft-k

BALANCING MOMENTS:

w = 2.76 klf

M^- = −249 ft-k

M^+ = 62 ft-k

STAGE 1:

MIDSPAN

f_{TOP} = −249 psi ✓ ok

f_{BOT} = −66 psi ✓ ok

SUPPORT

f_{TOP} = 219 psi > $3\sqrt{f'_{ci}}$; 0.6 f'_{ci}

f_{BOT} = 927 psi ✓ ok

STAGE 2 :

MIDSPAN

$f_{TOP} = -300$ psi ✓ OK

$f_{BOT} = -8.2$ psi ✓ OK

SUPPORT

$f_{TOP} = 420$ psi $< 6\sqrt{f'_c}$ ✓ OK

$f_{BOT} = -732$ psi ✓ OK

ULTIMATE STRENGTH :

$M_1 = Pe = (532^k)(35\ in)/12 = 155$ ft-k

$M_{SEC} = M_{BAL} - M_1 = 94$ ft-k

M_u @ MIDSPAN = 216 ft-k

M_u @ SUPPORT = −537 ft-k

REINFORCEMENT :

NO REINF. NEEDED @ POSITIVE MOMENT

NEGATIVE MOMENT :

$A_{s\,min} = 2.43\ in^2 \Rightarrow 13 - \#4 \quad (A_s = 2.6\ in^2)$

CHECK MIN. REQUIREMENTS :

$A_{PS} = 3.06\ in^2 \quad ; \quad d = 8\ in \qquad f_y = 0.8 f_{py}$

$f_{ps} = 246{,}732$ psi

$a = 0.434$ in

$\phi M_n = (0.9)(708)(7.783)/12$

$\phi M_n = 413.2$ ft-k < 537 ft-k

A_s REQ'D = 3.38 in^2

$\boxed{17 - \#4}$ TOP AT INTERIOR SUPPORTS

N-S FRAME

CHECK PUNCHING SHEAR :

ASSUME INTERIOR COLUMN :

$d = 9" - 1" = 8"$

$$V_c = \min \begin{cases} 4\sqrt{f'_c}\, b_o d \\ \left(\dfrac{\alpha_s d}{b_o} + 2\right)\sqrt{f'_c}\, b_o d \\ \left(2 + 4/\beta_c\right)\sqrt{f'_c}\, b_o d \end{cases}$$

ASSUME 18×18 COLUMNS

$b_o = 104\ \text{IN} \qquad \alpha_s = 40$

$$V_c = \min \begin{cases} (4)\sqrt{5000}\,(121\ \text{in})(8\ \text{in}) = 274^k \\ \left[\dfrac{(40)(8\ \text{in})}{(104\ \text{in})} + 2\right]\sqrt{5000}\,(121\ \text{in})(8\ \text{in}) = 348^k \\ (6)\sqrt{5000}\,(121\ \text{in})(8\ \text{in}) = 411^k \end{cases}$$

$\phi V_c = (0.75)(274^k) = 206^k$

$DL = (1.2)(142.5\ \text{psf})$
$LL = (1.6)(65\ \text{psf})$ $\Big\}$ 267 PSF

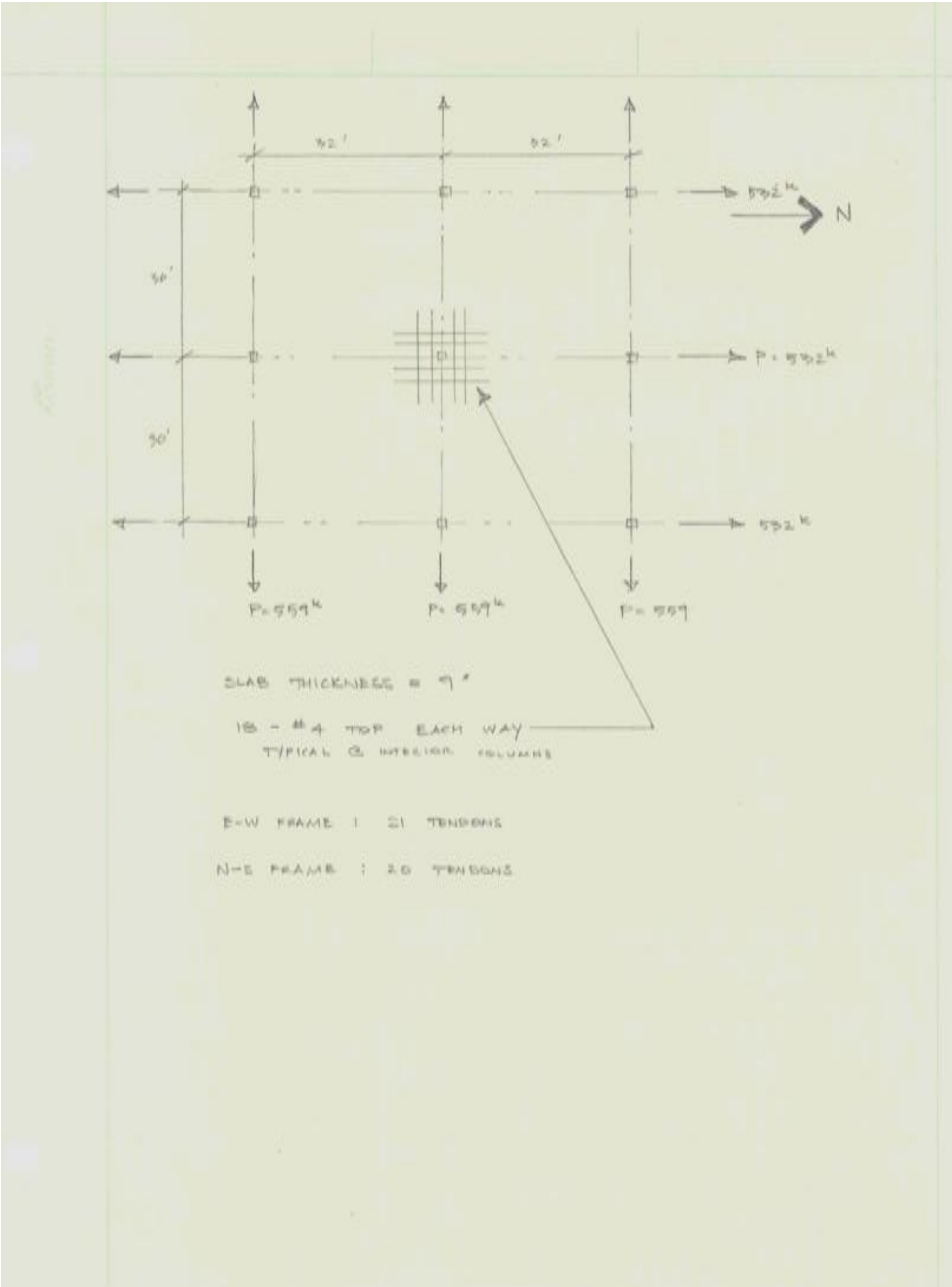
$A_T = 960\ ft^2$

$V_u = (960\ ft^2 - 2.25\ ft^2)(267\ \text{psf}) = 256^k$ ∴ NEED DROP PANELS

$(256^k / 0.75) = 4\sqrt{5000}\, b_o\,(8") \Rightarrow b_o = 151\ \text{IN}$

$151"/4 = 18 + d \qquad d = 19.75\ \text{IN} \Rightarrow 20"$ / SLAB = 9"

NEED 11" DROP HEAD



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APPENDIX C: TWO-WAY SLAB ANALYSIS

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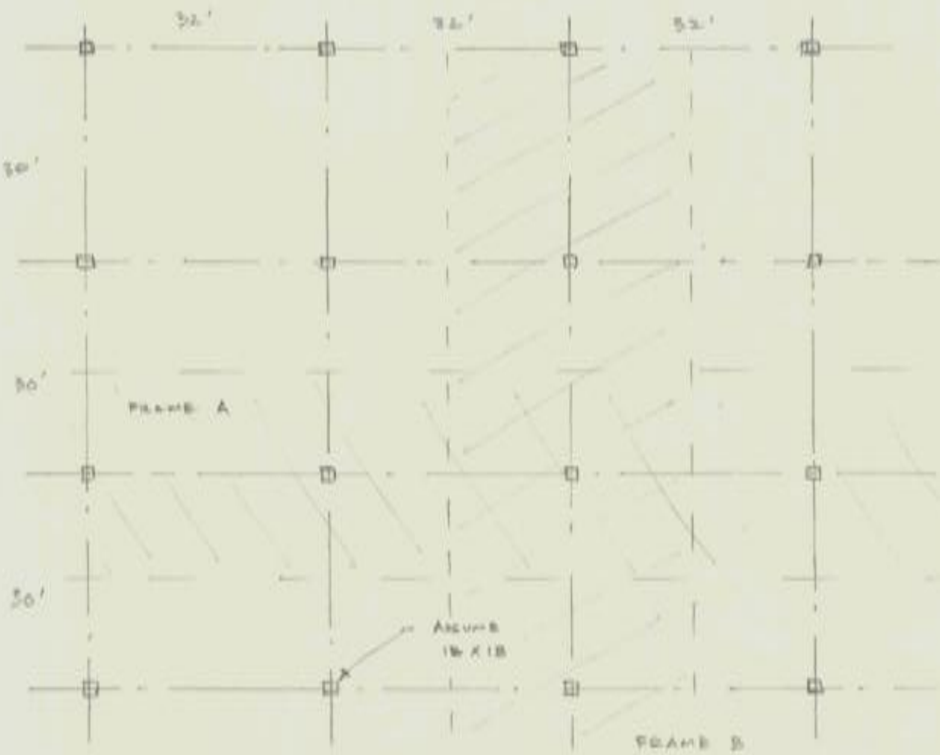
TWO-WAY FLAT SLAB

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

LIVE LOAD = 60 PSF
PARTITION LOAD = 20 PSF
SUPERIMPOSED DEAD LOAD = 10 PSF

TYPICAL INTERIOR BAY:

STORY HEIGHT = 14'-6" (TYP.)



SLAB THICKNESS: BASED ON SHEAR ⟹ NO DROP PANEL

TRIAL $t = \ell_n / 33$; $\ell_n = 32' - 18/12 = 30.5$ ft

$t_{min} = (30.5\text{ ft})(12\text{ in/ft}) / 33 = 11$ in

$W_u = (1.2)\left[(11/12)(150\text{ pcf}) + 30\text{ psf}\right] + (1.6)(60\text{ psf}) = 297$ psf

SHEAR CHECK :

$V_u = w_u \, AREA = (297\ psf)(30\ ft)(32\ ft) - \left(\frac{18 \times 18}{144}\right)$

$V_u = 285^k$

PUNCHING SHEAR

$b_o = 72\ in$ $\qquad b_o/d \ ; \quad d = 10\ in$ ASSUME

$b_o/d = 7.2 \qquad \alpha_s = 40$

$$V_c = \min \begin{cases} 4\sqrt{f'_c}\, b_o d = (4)(\sqrt{5000})(72)(10) = 204^k \\ 6\sqrt{f'_c}\, b_o d \\ \left(\frac{\alpha_s}{b_o/d} + 2\right)\sqrt{f'_c}\, b_o d \end{cases}$$

$\phi V_c = (0.75)(204^k) = 153^k$ NO GOOD $\Rightarrow$ NEW t

$$V_u = \phi V_c = \min \begin{cases} 4\sqrt{f'_c}\, b_o d \\ \left(\frac{\alpha_s}{b_o/d} + 2\right)\sqrt{f'_c}\, b_o d \end{cases}$$

FIND d_{MAX}

NOT ECONOMICAL

$285^k = (0.75)(4)\sqrt{5000}\,(72)\,d \implies d = 18.7\ in$

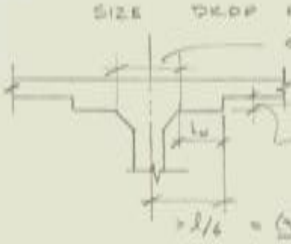
$\therefore$ INVESTIGATE DROP PANELS USING $\ell_n / 36$

$t_{min} = (30.5\ ft)(12^{in}/ft)\ /36$

$t_{min} = 10.5\ in$

— CHECK LATER —

SIZE DROP PANEL



0.2 to $0.33\ \ell_{avg} = (0.21)(31\ ft) = 78\ in$ vs. 39 IN PER SIDE

t_d

$\geq t/4$ BUT $< 1/4\, h_w$ $\qquad h_w = 25\ in$

$t_d = 7.5\ in$

$> \ell/6 = \frac{(32\ ft)(12^{in}/ft)}{6} = 64\ in$

FRAME A :

INCREASED FROM PANEL

$$M_o = (1/8)(300\ psf)(30\ ft)(30.5\ ft)^2 = 1047\ ft\text{-}k$$

FRAME B :

$$M_o = (1/8)(0.300\ ksf)(32\ ft)(28.5\ ft)^2 = 975\ ft\text{-}k$$

MOMENT	FRAME A	FRAME B	
M^-	-681	-634	$0.65 M_o$
M^+	366	341	$0.35 M_o$

MOMENT DISTRIBUTION :

75 % M^- TO C.S. / 25 % M^- TO M.S.

60 % M^+ TO C.S. / 40 % M^+ TO M.S.

COLUMN STRIP = 15 ft , $M.S._A$ = 15 ft , $M.S._B$ = 17 ft

FRAME A

M_{TOTAL}	-681	366	-681
$M_{C.S.}$	-511	220	-511
$M_{M.S.}$	-170	146	-170

FRAME B

M_{TOTAL}	-634	341	-634
$M_{C.S.}$	-476	205	-476
$M_{M.S.}$	-158	136	-158

ASSUME $d_A = d_B$
#7 BARS

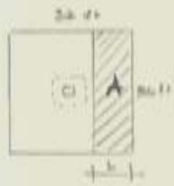
$$d_A = 14 - \frac{0.875}{2} - 0.75 = 12.81\ in$$

$$d_B = 11.94\ in$$

ITEM	DESCRIPTION	FRAME A				FRAME B			
		C.S.		M.S.		C.S.		M.S.	
		M^-	M^+	M^-	M^+	M^-	M^+	M^-	M^+
1	MOMENT	-511	220	-170	146	-476	205	-158	136
2	b DEEP	80	180	180	180	80	180	204	204
3	d effective	12.81	9.31	9.31	9.31	11.94	8.44	8.44	8.44
4	M_A	-568	245	-189	162	-529	228	-176	151
5	R	519	188	147	125	557	213	145	125
6	ρ	0.0098	0.0032	0.0025	0.0021	0.016	0.0037	0.0026	0.0021
7	A_s	9.98	5.36	4.19	3.52	9.55	5.62	4.30	3.77
8	ρ_{min}	0.0019	0.0021	0.0021	0.0021	0.0016	0.0022	0.0022	0.0022
9	N	16	9	7	6	16	10	8	7
10	N_{min}	10	10	10	10	10	10	10	10

SHEAR CHECK :

$$d_{AVG} = \frac{12.81 + 11.94}{2} = 12.375 \text{ IN} \quad @ \text{ PANEL}$$



$$L = 9.64 \text{ ft}$$

$$A = (30 \text{ ft})(9.64 \text{ ft}) = 289 \text{ ft}^2$$

$$V_u = (300 \text{ psf})(289 \text{ ft}^2) = 87^k$$

$$\phi V_c = (0.75)(2)\sqrt{5000}\,(30)(12)(8.375)$$

$$\phi V_c = 339^k \quad \checkmark \text{ OK}$$

PUNCHING SHEAR :

AREA OF DROP

$$V_u = (300 \text{ psf})\left[(30 \text{ ft})(32 \text{ ft}) - 114 \text{ ft}^2\right]$$

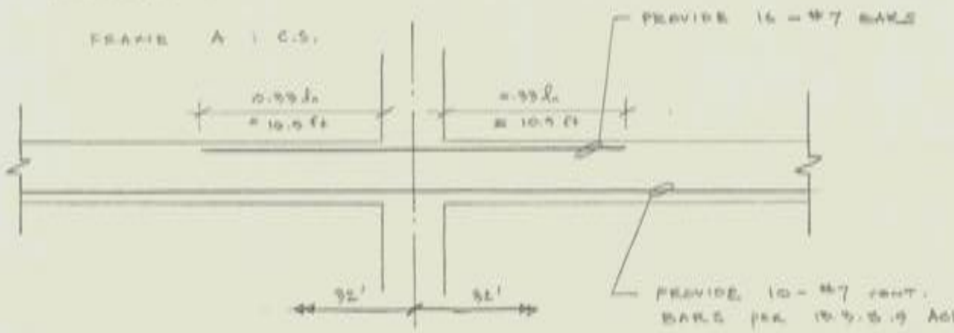
$$V_u = 254^k$$

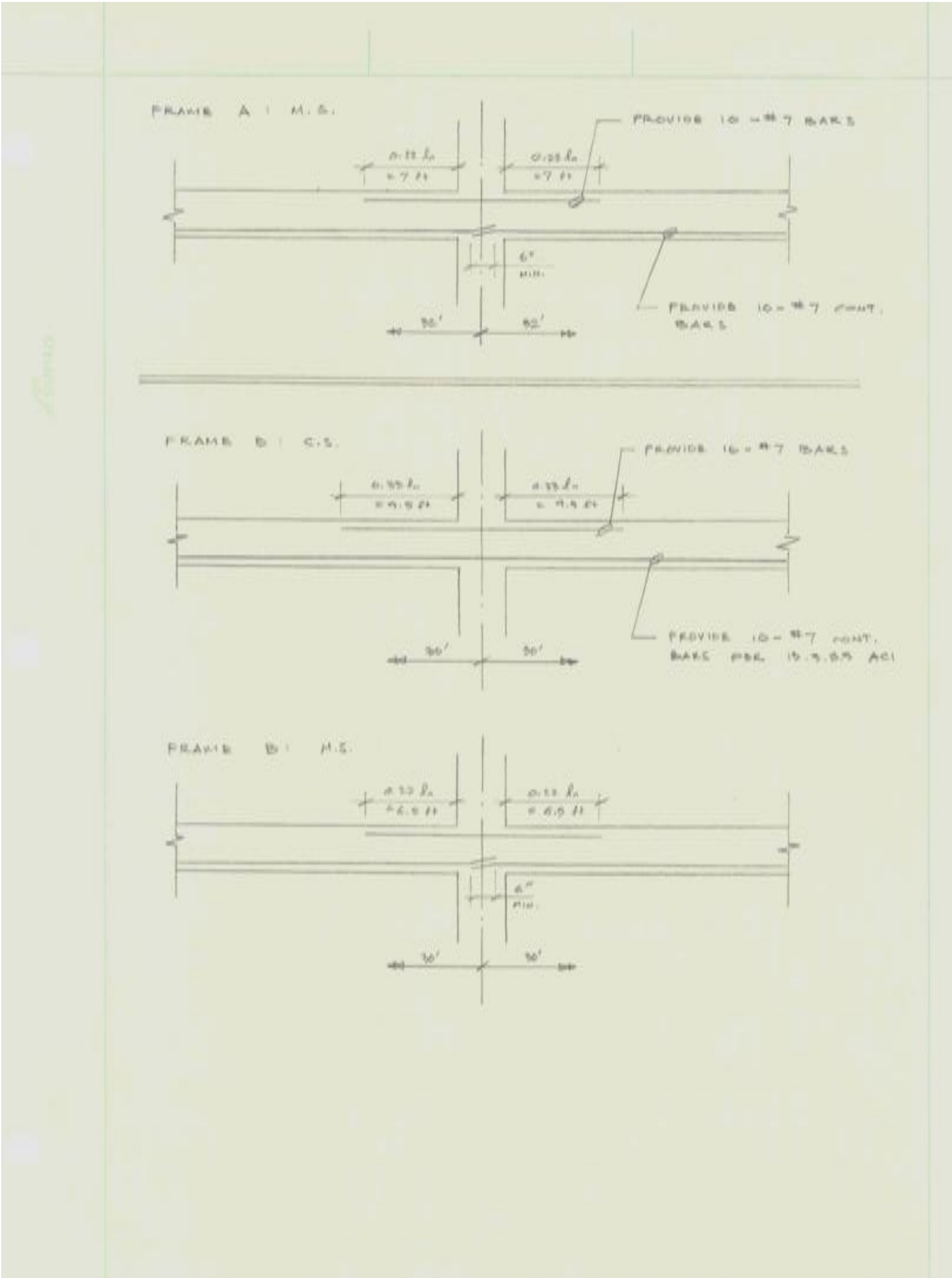
$$\phi V_c = \min \left| \begin{array}{l} 4\sqrt{5000}\,(256 \text{ in})(12.375 \text{ in})(0.75) = 672^k \\ \left(\frac{40}{21} + 2\right)\sqrt{5000}\,(256 \text{ in})(12.375 \text{ in})(0.75) = 661^k \end{array} \right.$$

$$\phi V_c = 661^k > 254^k \quad \checkmark \text{ OK}$$

CUT-OFFS :

FRAME A : C.S.





-End of Section-

APPENDIX D: GIRDER-SLAB ANALYSIS

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GIRDER - SLAB

MODIFIED BAYS : 20'-0" x 21'-4" (SEE CAD DWG.)

PLANK : DY-CORE 4'-0" x 8" WITH 2" TOPPING

DEAD LOAD = 75 PSF

PARTITION LOAD = 20 PSF

LIVE LOAD = 60 PSF
REDUCED TO 46 PSF

TRY DB 9 x 46

$I_s = 195 \text{ in}^4$
$S_t = 33.7 \text{ in}^3$
$S_b = 50.8 \text{ in}^3$
$M_{s\,cap} = 84 \text{ ft-k}$
$t_w = 0.375 \text{ in}$
$b = 5.75 \text{ in}$

$I_{tr} = 596 \text{ in}^4$
$S_t = 68.6 \text{ in}^3$
$S_b = 80.6 \text{ in}^3$

$\Delta_{ALLOW} = L/360 = (20 \text{ ft})(12 \text{ in}/\text{ft}) / 360 = 0.67 \text{ in}$

INTIAL LOAD PRECOMPOSITE :

$M_{DL} = (21.333 \text{ ft})(0.075 \text{ ksf})(20 \text{ ft})^2 / 8$

$M_{DL} = 80 \text{ ft-k} < M_{s\,cap} = 84 \text{ ft-k}$ ✓ ok

$$\Delta_{DL} = \frac{(5)(21.3 \text{ ft})(0.075 \text{ ksf})(20 \text{ ft})^4 (1728 \text{ in}^3/\text{ft}^3)}{(384)(29000 \text{ ksi})(195 \text{ in}^4)} = 1.02$$

$\Delta = L/236$ ok ∴ within 5 %

TOTAL LOAD - COMPOSITE :

$M_{sup} = (21.33 \text{ ft})(0.02 \text{ ksf} + 0.046 \text{ ksf})(20 \text{ ft})^2 / 8$

$M_{sup} = 70.4 \text{ ft-k}$

$M_{TL} = M_{DL} + M_{sup} = 150.4 \text{ ft-k}$

$S_{REQ} = (150.4 \text{ ft-k})(12 \text{ in}/\text{ft}) / (0.6)(50 \text{ ksi}) = 60.2 \text{ in}^3$

$S_{REQ'D} = 60.2 \text{ in}^3 < S_t = 68.6 \text{ in}^3$ ✓ ok

$$\Delta_{sup} = \frac{(5)(21.3 \text{ ft})(0.066 \text{ ksf})(20 \text{ ft})^4 (1728 \text{ in}^3/\text{ft}^3)}{(384)(29000)(596 \text{ in}^4)} = 0.49 \text{ in}$$

✓ ok

CHECK COMPRESSIVE STRESS ON CONCRETE :

$N_{VALUE} = E_s / E_c = 29000 / (57000)\sqrt{4000} = 8.04$

$S_{tc} = (8.04)(68.6 \text{ in}^3) = 551.5 \text{ in}^3$

$f_c = (70.4 \text{ ft-k})(12^{in}/\text{ft}) / 551.5 \text{ in}^3$

$f_c = 1.53 \text{ ksi}$

$F_c = (0.45)(4 \text{ ksi}) = 1.80 \text{ ksi} > f_c$ ✓ OK

CHECK BOTTOM FLANGE TENSION STRESS (TOTAL LOAD) :

$$f_b = \frac{(80 \text{ ft-k})(12^{in}/\text{ft})}{50.8 \text{ in}^3} + \frac{(70.4 \text{ ft-k})(12^{in}/\text{ft})}{80.6 \text{ in}^3}$$

$f_b = 29.4 \text{ ksi}$

$F_b = (0.9)(50 \text{ ksi}) = 45 \text{ ksi} > f_b$ ✓ OK

CHECK SHEAR :

TOTAL LOAD = 141 PSF

$W = (0.141 \text{ ksf})(21.33 \text{ ft}) = 3.0 \text{ klf}$

$R = (3.0 \text{ klf})(20 \text{ ft}) / 2 = 30^k$

$f_v = 30^k / (0.375 \text{ in})(5.75 \text{ in})$

$f_v = 14 \text{ ksi}$

$F_v = (0.4)(50 \text{ ksi}) = 20 \text{ ksi} > f_v$ ✓ OK

∴ DB 9x46 WITH 8" HOLLOW CORE PLANK + 2" TOPP.

-End of Section-

APPENDIX E: NON-COMPOSITE STEEL ANALYSIS

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NONCOMPOSITE WITH LIGHTWEIGHT CONCRETE

MODIFIED BAYS : 20'-0" × 21'-4" (SEE CAD DWG)

LOADS :

DECKING : 1.5C18 (4½" TOTAL THICKNESS — 3" CONCRETE)

DEAD : STRUCTURAL = 15 PSF
M.E.P. = 10 PSF
PARTITIONS = 20 PSF
SUPERIMPOSED = 5 PSF
SLAB / DECK = 39 PSF
89 PSF

LIVE : OFFICE / LAB = 50 PSF ⟹ L_o = 46 PSF

$W_u = (1.2)(89 \text{ psf}) + (1.6)(46 \text{ psf})$

$W_u = 0.18$ ksf

BEAMS : TRIB WIDTH OF 6'-8"

$M_u = \frac{w_u l^2}{8} = \frac{(0.18 \text{ ksf})(6.67 \text{ ft})(21 \text{ ft})^2}{8}$

$M_u = 67$ ft-k

ASSUME FULLY BRACED

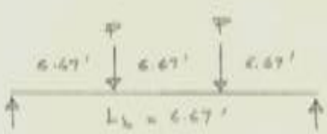
W12 × 16 $\phi M_p = 75.4$ ft > 67 ft-k ✓ OK

DEFLECTION CHECK :

$\Delta_L = L/360 = 0.70$ in

$\Delta = \frac{(5)(0.31 \text{ klf})(21 \text{ ft})^4 (1728 \text{ in}^3/\text{ft}^3)}{(384)(29000 \text{ ksi})(103 \text{ in}^4)} = 0.45$ in ✓ OK

GIRDERS :



$P = 12.6^k + 12.6^k$

$M_u = (25.2^k)(6.67 \text{ ft}) = 168$ ft-k

W14 × 30 $\phi M_p = 177$ ft-k > 168 ft-k ✓ OK

PICKED A W14 TO MINIMIZE COPING TOP & BOTTOM FLANGES

DEFLECTION CHECK :

$$\Delta_L = L/360 = 0.67 \text{ in}$$

$$\Delta = \frac{Pl^3}{28EI} = \frac{(12.89^k)(20 \text{ ft})^3(1728 \text{ in}^3/\text{ft}^3)}{(28)(29000)(291 \text{ in}^4)} = 0.75 \text{ in}$$

$$I_{REQ'D} = 328 \text{ in}^4 \quad \therefore \text{W14x34}$$

BEAM $\Delta_T = L/240 = 1.05$ in

$$\Delta = \frac{(5)(0.9 \text{ klf})(21 \text{ ft})^4(1728 \text{ in}^3/\text{ft}^3)}{(384)(29000 \text{ ksi})(103 \text{ in}^4)}$$

$$\Delta = 1.32 \text{ in}$$

FIND REQUIRE I_x

$$I_{REQ'D} = 127.8 \text{ in}^4$$

$\boxed{\text{USE W12X19}}$ FOR ALL INTERIOR BEAMS

GIRDER $\Delta_T = L/240 = 1.0$ in

$$I_{REQ'D} = \frac{(18.9^k)(20 \text{ ft})^3(1728 \text{ in}^3/\text{ft}^3)}{(28)(29000 \text{ ksi})(1.0 \text{ in})} = 322 \text{ in}^4$$

$\boxed{\text{W14x34}}$ OK FOR ALL INTERIOR GIRDERS

DESIGN PERIMETER GIRDERS

$$\Delta_T = L/400 = 0.6 \text{ in}$$

$$I_{REQ'D} = \frac{(9.5^k)(20 \text{ ft})^3(1728 \text{ in}^3/\text{ft}^3)}{(28)(29000 \text{ ksi})(0.6)} = 269.6 \text{ in}^4$$

$$I = 328 \text{ in}^4 \checkmark \text{ OK}$$

$\boxed{\text{W14x34}}$

-End of Section-

APPENDIX F: SYSTEM COMPARISON

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Floor System Comparison - Typical Bay

Criterion	Floor Systems				
	Existing Composite Steel	Post-Tensioned Slab	Two-Way Slab	Girder-Slab	Non-composite Steel
System Weight (psf)	72	112	137	75	54
Slab Depth (in)	6.0	9.0	11.0	10.0	4.5
Total Depth (in)	29.7	11.0	14.5	10.0	18.5
Column Size	W12x87	18"x18"	18"x18"	W10x49	W10x49
Constructability	Medium	Hard	Medium	Medium	Easy
Foundation Impact	-	Little	Little	* Yes	* Yes
Fire Rating (hr)	1.5 to 2	2	2	2 to 3	1.5 to 2
Materials Cost per ft^2	$12.45	$17.22	$9.00	$12.12	$11.47
Labor Cost per ft^2	$5.65	$8.95	$9.15	$6.20	$6.06
Column Cost per ft^2	$5.92	$6.31	$6.31	$20.03	$20.03
Foundation Cost per ft^2	$23.96	$23.96	$23.96	$86.13	$86.13
Total Cost per ft^2	**$47.98**	**$56.44**	**$48.42**	**$124.48**	**$123.69**
Possible Alternative	-	Yes	Yes	No	No
Additional Study	-	Yes	Yes	No	No

* System requires additional columns and footings

Available upon request are paper copies of RS Means data.

-End of Section-