The New York Times Building

New York, NY



IPD/BIM Thesis Technical Report #2

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EXECUTIVE SUMMARY

In this second technical report, alternative floor systems were investigated for the New York Times Building. A typical bay of 30'-0" x 40'-0" was analyzed for four floor systems, which includes the existing system. The four systems were then compared based on framing impacts, total structural depth, column sizes, constructability, fire rating and fireproofing, lead time, foundation and lateral system impact, structural weight, and relative cost. The existing floor system is a composite steel system with normal weight concrete and two infill beams. The additional three systems investigated include:

- o Composite Steel Deck with Lightweight Concrete and Three Infill Beams
- o One-Way Slab Concrete with two different beam layouts
- o Two-Way Post-Tensioned Slab

The design of composite steel deck with lightweight concrete and three infill beams floor system resulted in a 5" thick lightweight composite deck with a structural depth of approximately $22 \frac{1}{2}$ ". This system is lighter than the existing system, but may prove problematic in lateral and vibration analyses. In addition, the shallower members did not even add an inch more to a typical bay which would not give the mechanical, electrical, and plumbing systems much more room. In terms of relative cost, it was the second cheapest of the system with \$30.07 per square foot. Further investigation into vibration and lateral system impacts is required to determine if this system is a viable option.

The one-way slab floor system resulted in two designs being investigated to determine which layout would yield a shallower depth. The beams spanning the 40'-0" direction resulted in a structural depth of 28" from top of slab to bottom of girder with 24x20 beam and 28x24 girders. Though the structural depth is shallower than that of the other option, its structural self weight is 25% greater than the other option where the beams span the 30'-0" direction. The second option where the beams span the 30'-0" direction resulted in a deeper structural depth of 33" from the top of the slab to the bottom of the girder with 20x28 beams and 33x28 girders. The cost of the long span beams versus the short span beams was \$31.40 per square foot and \$29.44 per square foot respectively. This aspect of this layout makes it the cheapest of all the systems analyzed. Even though this system is easily constructed, it is approximately two times heavier than the existing system and will result in foundation and lateral system changes.

The post-tensioned two-way slab was designed to limit the structural depth and weight of a concrete floor system, compared to the bulky one-way system designed before it. It was determined that a 11 ¹/₂" thick slab was needed to span the typical 40'-0" direction. In addition to the post-tensioned slab, ¹/₂" drop panels were necessary to prevent punching shear. The cost of the post-tensioned slab is \$30.52 per square foot which is approximately the average cost of the four floor systems. At the interior supports, a substantial amount of reinforcement was required for ultimate strength. A post-tensioned system will be further investigated, due to its economical shallow depths and long span capabilities. However, the post-tensioned system is implemented it will affect the column grid due to concrete columns, and the foundations and lateral system due to a weight of approximately two and a half times greater than the existing floor system.

INTRODUCTION



Figure 1: Typical Tower Framing Plan

The New York Times Headquarters Building is home to the New York Times newsroom and twenty six floors of Times offices, as well as several law firms whose offices are leased through Forest City Ratner. Designed by architect Renzo Piano in association with FFFOWLE Architects, it was intended to be a flagship building promoting sustainability, lightness, and transparency. The architectural façade reflects the ever changing environment surrounding the building, an appropriate acknowledgement of the heart of New York City.

The building rises fifty two stories with a height of 744 feet to the main roof. A 300 feet mast then extends up into the sky topping out at 1048 feet above Eighth Avenue between 40th and 41st Streets. The New York Times building totals 1.5 million square feet with the New York Times Company owning 800,000 square feet and Forest City Ratner Companies owning the other 700,000 square feet. It has one 16'-0" level below grade. The ground level contains a lobby, retail space and a glass-enclosed garden. The New York Times' newsroom occupies the entire five-story podium which is east of the tower structure. The tower ascends above the podium an additional forty eight stories. Story heights average approximately 13'-9" in the tower, lending a great view to the open office plans. At the mechanical floors on levels twenty eight and fifty one though, the floor height is approximately 27'-0" to accommodate equipment and two-story outriggers.

The steel structural system is comprised of composite floor beams and columns configured as shown in Figure 1, with lateral chevron braces in both the East-West and North-South directions in the core. Foundations are a combination of concrete spread footings and caissons to develop the required capacity. Many structural elements are also architectural details, including the exposed X bracing on the exterior of the structure and the built-up columns at the corner notches. Overall, the building exhibits ingenuity in design and construction.

The remainder of this report evaluates the existing floor framing system, as well as three alternative solutions. All designs are schematic, as the objective of this report is to study various floor systems that can be applied to the New York Times Building. Several variables are taken into account when comparing floor systems, such as framing layout, structural depths, fire protection, lead time, cost, and other structural impacts. All alternative floor systems will be designed and compared using a typical 30'-0" x 40'-0" interior bay, as seen boxed in red on Figure 1.

Design Codes and References

Design Codes

National Model Code:
1968 Building Code of the City of New York with latest supplements
Structural Standards:
ASCE 7-98, Minimum Design Loads for Buildings and other Structures
Structural Design Codes:
AISC – LRFD, Steel Construction Manual 2nd edition, American Institute of Steel Construction
ACI 135-74 Manual of standard Practice for detailing Reinforced Concrete Structures
ACI 318-99 American Concrete Institute Building Code Requirements for Reinforced Concrete
ACI 530-95 Building Code Requirements for Masonry Structures
National Building Code of Canada, 1995 Uniform Building Code, 1997

Thesis Codes

National Model Code:

2006 International Building Code

Structural Standards:

ASCE 7-05, Minimum Design Loads for Buildings and other Structures

Design Codes:

AISC – LRFD, Steel Construction Manual 13th edition, American Institute of Steel Construction

Design Deflection Criteria

Lateral Deflections: Total building sway deflection for ten year wind loading is limited to H/450 Thermal Deflections:

The shortening and elongating effects due to thermal fluctuations is designed to L/300.

Thesis Deflection Criteria

Gravity Deflections: Live load deflections for floor members are limited to L/360 Total load deflections for floor members are limited to L/240 Lateral Deflections: Total building sway deflection for ten year wind loading is limited to H/450

Allowable inter-story drift due to wind is H/400 to H/600 (ASCE 7-05 § CC.1.2)

Building story sway deflection for seismic loading is limited to 0.015h_{sx} (ASCE 7-05 TABLE 12.12-1)

Thermal Deflections:

The shortening and elongating effects due to thermal fluctuations is designed to L/300.

Material Strengths

Concrete:
Foundation Walls, Buttresses, S.O.GCompressive strength of 4,000 psi, Normal Weigh
Footings and PiersCompressive strength of 5,950 psi, Normal Weigh
Concrete on Metal DeckCompressive strength of 4,000 psi, Normal Weigh
Concrete Pads, Fill SlabsCompressive strength of 3,000 psi, Light Weight (115 PCF
All Other ConcreteCompressive strength of 4,000 psi, Normal Weight
ReinforcingASTM A-615, Grade 60
Welded Wire FabricASTM A185
Rock Anchor:
Dywidag Threadbars AnchorsASTM A722, Grade 150 ks
High Strength PVC Corrugated SheathingCompressive strength of 7,000 ps
PlatesATSM A30
Structural Steel:
Rolled Shapes and ChannelsASTM A572 or A992, Minimum yield strength of 50 ks
Miscellaneous AnglesASTM A36, Minimum yield strength of 36 ks
"UAP" ChannelsEuropean Code EC3, Grade S-235JRG2, Minimum yield strength of 46 ks
TubesASTM A500, Grade B, Minimum yield strength of 42 ks
PipesASTM A500, Grade B, Minimum yield strength of 46 ks
Plate Material used for Built-Up MembersASTM A572, Minimum yield strength of 50 ks
Connections & Base PlateASTM A36 (36 ksi), A529 (42 ksi), A572 & A588 (50 ksi
Diagonal & X-Braced RodsASTM A572, Grade 65
Metal Decking:
3" Composite DeckASTM A653 SQ, Grade 40, Minimum yield strength of 40 ks
Headed Shear Studs 3/4"ASTM A108, Type I
Connections:
BoltsASTM A325 or A490
NutsASTM A563
WashersASTM A-F436
Anchor Bolts/ RodsASTM F-1554, Grade 55
Welding Electrodes E70XX
Masonry:
MortarType M or S
GroutCompressive strength of 3,000 ps
Concrete Masonry UnitsCompressive strength of 3,000 ps
ReinforcingASTM A-615, Grade 60

Fire Protection and Fire Ratings

The fire rating of the existing structure is 2 hours. Therefore, to adequately compare fire ratings and fire protection of the existing floor system, the alternative systems must obtain a minimum rating of 2 hours if possible. All structural steel members must be protected against fire and must meet Underwriters Laboratories minimum requirement. Approximate weight and application of fire protection will be considered for the various methods such as cementitious fireproofing, sprayed fiber, intumescent paint, and gypsum board encasement. All reinforced concrete must be protected against fire and must meet ACI 318-08 minimum clear cover. For additional fire protection information for each system, see Appendix A: Existing Framing System, Appendix B: Lightweight Composite Framing System, Appendix C: One-Way Reinforced Concrete System, and Appendix D: Two-Way Post-Tensioned Concrete System.

Gravity Loads

The construction dead load for a typical floor system in this report includes the self weight of the floor system and a superimposed dead load of 25 psf for the ceiling, as well as mechanical, lighting and plumbing in the raised floor system and in the ceiling.

Live Load:									
Load Description	ASCE 7-05 &	Design Load							
	NYC Bldg Code								
Office:	50 psf	50+20 (for partitions) = 70 psf							
Technology Floors:	100 psf	100 psf							
Elevator Lobbies:	75 psf	75 psf							
Corridors above First Floor:	80/75 psf	75 psf							
All Other Lobbies & Corridors:	100 psf	100 psf							
Exit Facilities:	100 psf	100 psf							
Retail Areas:	100 psf	100 psf							
Kitchen:	100 psf	150 psf							
Cafeteria:	100 psf	100 psf							
Auditorium (with fixed seats):	60 psf	100 psf							
Light Storage Area:	125/100 psf	100 psf							
Loading Dock:	250 psf	250 psf or actual weight whichever is greater							
Mechanical Floors:	125 psf	150 psf or actual weight whichever is greater							
Mechanical/Fan Rooms:	75 psf	75 psf or actual weight whichever is greater							
Sidewalks	250 psf	600 psf							
Roofs:	20 psf	30 psf + Drift							
Roof Garden	100 psf	Not Specified							

The typical floor live load used in this report is 70 psf for office areas, 100 psf for the core and cafeteria floors, and 150 psf for mechanical floors. The typical floor system analyzed is in the office area with 70 psf. Live load reduction will not be considered in this report. For specific gravity loads of each system, see Appendix A: Existing Framing System, Appendix B: Alternative Composite Framing System, Appendix C: Alternative One-Way Concrete System, and Appendix D: Alternative Two-Way Concrete System.

Structural System Overview

Foundation

The foundation of the New York Times Headquarters combines typical spread footings with caissons to achieve its maximum axial capacity. Below the building's 16-foot cellar, the tower and podium mostly bear on rock; Class 1-65 and 2-65 per the New York City Building Code, with a capacity of 20 - 40 ton per square foot. However, the rock at the southeast corner of the tower only had an 8 ton per square foot capacity; Class 4-65. Of the seven columns that fall within this area (indicated in Figure 2) 24-inch diameter concrete-filled steel caissons were used. Each caisson was designed to support a load of 2,400 kips with 6,000 psi concrete.

Under the other 21 columns (indicated on Figure 2) spread footings of unknown dimensions with a compressive strength of 6,000 psi are used to support the loads. The columns which fall in the cantilevered areas do not directly transfer load to the ground which removes the need for footings at these locations.



The New York City Subway does pass the north and eastern sides of the New York Times Building. However, this is not a major site restriction since the transit system passes below Eighth Avenue and 41st Street and not directly beneath the structure. Although, vibration effects on the foundation and building structure may have had an impact on the design.

Floor System

The floor system is a composite system with a typical bay size of 30'-0"x 40'-0"surrounding the 90'-0"x 65'-0" core. There are 60'-0"x 20'-0" cantilever bays on the north and south sides of the tower. The floor system is made up of $2 \frac{1}{2}"$ normal weight concrete on 3" metal deck, typically spanning 10'-0" from W12x19 to W18x35 infill beams. The W12x19 and W18x35 beams span into W18x40 girders. The girders frame into the various built-up columns, box columns along the exterior and built-up non-box columns in the



Figure 3: 'Dog-leg' beam connection, courtesy of Thornton Tomasetti

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core. Framing of the core consists of W12 and HSS shapes framing into W14 and W16 shapes which frame into W33 girders that frame into the core columns.

In the New York Times spaces, the structural slab is 16" below the finish floor and the spandrel panel, due to the raised floor system for the under floor mechanical systems. For all the exterior steel of the building to maintain a centerline at the center of the spandrel panel, a crooked connection or 'dog-leg' was used. See Figure 3 for an interior view of the 'dog-leg' connection during construction. The 'dogleg' connection allows for the end of the beam to rise 10" before it leaves the interior of the building and penetrates the building envelope. Figure 4 shows the 'dog-leg' connection after the beam has penetrated the building envelope.



Figure 4: 'Dog-leg' penetrating building envelope

Columns

The 30"x30" box columns at the exterior notches (Figure 5) of the tower consist of two 30" long flange plates and two web plates inset 3" from the exterior of the column on either side. The two web plates of the welded box column vary from 7" thick at the ground floor to 1" thick at the fifty second floor. This is to account for the different steel areas needed for the higher forces at the bottom of the building. To maintain consistent proportions at all floors, a hierarchy of flange plate thicknesses was developed. At the ground floor, each flange plate is 4" thick and decreases to 2" thick at the fifty second floor. See Figure 6 for box column hierarchy. Although the yield strength of the plates also varies with tower height, the strength was assumed to be a uniform 50 ksi for calculations. Interior columns are a combination of built-up sections and rolled shapes. Column locations stay consistent throughout the height of the building, and every perimeter column is engaged in the lateral system which will be described later.





Figure 6: Box Column hierarchy, courtesy of Thornton Tomasetti

Vierendeel System

A Vierendeel system was used at the 20 foot cantilever sections of the tower. Renzo Piano did not want columns obstructing the glass storefronts at the ground level, so these sections were cantilevered from the main structure. The middle line of the cantilevered bays have beams moment connected to the columns thus creating the Vierendeel system and engaging every floor except at the outrigger levels. At the outrigger level; floor twenty eight and fifty two, large diagonal braces tie the middle line back to the core through the outrigger trusses. In extreme loading conditions, this provides a redundant load path. See Figure 7 for Vierendeel frame location. At the exterior beam lines of the cantilever, 2" diameter steel rods were connected from the columns to the ends of the beams to control deflection at every floor. This allowed the beams to be designed only for strength, thus avoiding bulky exterior members.



Figure 7: Cantilevered bays from exterior

Lateral System

The main lateral load resisting system for the tower of the New York Times Building consists of a centralized, steel braced frame core, with outriggers on the two mechanical floors (Levels twenty eight and fifty one). The structural core consists of concentric braces behind elevator shafts and eccentric braces at the elevator lobby entrances. At this time, the member sizes of these braces have yet to be disclosed, but the members were sized for strength. The core configuration remains consistent from the ground level to the twenty seventh floor as shown in Figure 8 and Figure 9. Above the twenty eighth floor, the low rise elevators were no longer required, and the number of bracing lines in the North-South direction was reduced from two to one.



Figure 8: Mechanical Floor Framing Plan, Floor 28





Figure 9: Core bracing during construction

The outriggers on the mechanical floors engage all columns of the tower in the lateral system. The outriggers consist of single diagonal braces shown in Figure 8 and Figure 10. The outrigger system was designed to increase the stiffness of the tower by engaging the perimeter columns in the lateral system.



Figure 10: Outrigger bracing on mechanical floor

During the design of the tower, the engineers at Thornton Tomasetti sized the members of the main lateral force resisting system merely for strength. In order to reduce lateral drift and acceleration, the structural engineers utilized the double story steel rod X-braces instead of increasing the member sizes of the main lateral force resisting system. These X-braces can be seen in Figure 8 on previous page and in Figure 11. The paired rods eliminate a center node and load sharing, in addition to eliminating eccentricities at the columns. The high strength steel rods transition from 2.5" to 4" in diameter and were prestressed to 210 kips. This induced tensile load prevents the need for large compression members, which prevents the members from buckling and conforms to the architectural vision of the exterior.

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Although the X-braces did reduce the need for an overall member size increase, the lateral system still did not completely conform to the deflection criterion. Therefore, some of the 30" by 30" base columns were designed as built-up solid sections which reduced the building drift caused by the building's overturning moment. After combining these solid base columns and the X-braces with the main lateral force resisting system, the calculated deflection of the tower due to wind was L/450 with a 10 year return period and a building acceleration of less than 0.025g for non-hurricane winds.

Thermal differentials had to be considered due to interior steel members being maintained at room temperature and exposed steel members undergoing extreme temperature changes. Thornton Tomasetti designed the structure using a range of -10°F to 130 °F. Due to the temperature deformation of the exterior columns and not the interior ones, differential deflection at upper floors exceeded L/100. To combat these thermal differentials, the outrigger trusses were utilized to even out the differential deflections. Thermal trusses were added along the east and west face at the twenty eighth and fifty first floors (Figure 12). These trusses provide bonus redundancy and limited deflection to L/300.

According to information obtained from the structural engineer, the podium of the New York Times Building was designed with a separate lateral system. Though information about the podium was not disclosed by the owner, an educated guess can be made about its lateral system. The podium contains the New York Times Newsroom and it can therefore be assumed that steel bracing, which would



Figure 11: Exposed exterior X-braced rods



Figure 12: Thermal Truss, in green, located at the 28th and 51st floor, courtesy of Thornton Tomasetti

cut down on the usable floor space, would not be used. Also, the use of concrete shear walls would go against the architect's "transparent" and open plan layout of the building design. Therefore, it can be assumed that the lateral system of the podium, from the ground to sixth floor, is designed as a steel moment resisting frame.

EXISTING FLOOR FRAMING SYSTEM

The existing floor system is a composite system with a typical bay size of 30'-0"x 40'-0". 2 ¹/₂" normal weight concrete on 3" metal deck, typically span 10'-0" from infill beams. W12x19 and W18x35 beams span into a W18x40 girder. Figure 13 shows the typical bay analyzed for the existing floor framing system calculation. The calculations included in appendix A refer to the 13th edition of AISC for beam and girder design and Vulcraft for the steel deck design. The deck was checked to meet a 2 hour fire rating with spray on fireproofing and strength requirements. The beams were checked to meet strength requirements in addition to deflection requirements of L/240 for total load and L/360 for live load. It was also found that the calculated shear and flexural forces in the beams were fifteen percent less than Thornton Tomasetti's designed values. This is due to the fifteen percent increase Thornton Tomasetti added in to account for potential changes in office space and expansion of light MEP systems.



Advantages and Disadvantages Analysis:

Composite steel is fairly simple to construct and combines the strength of concrete and steel, resulting in a system that is light and shallow. The current beams and girders are approximately 18" with a 5 $\frac{1}{2}$ " normal weight composite deck to make a combined structural depth of approximately $23 \frac{1}{2}$ ". The current system has a fire rating of 2 hours and structural weight of 57.5 psf including beams and girders. Currently, moment frames, bracings, outriggers, and built-up box columns are contributing to resisting lateral loads. This may have added additional cost to the system, where shear walls could have decreased cost, but at the time New York City labor union laws prevented concrete work occurring at the same time as steel work. Architecturally, this system allows for long open space without a really deep ceiling-to-floor sandwich. With the current column grid there are built up columns approximately every 30-40 feet. Though this allows for open space, the columns are big, bulky and conspicuous in the spaces where there are exposed; for example, in the lobby. The color of the fireproofing matches the window mullions, which exposes the viewer to look into the open air atrium if a bulky column is not obstructing their view. With the beams being 10 feet on center, it allows the mechanical systems to have room between the beams. However, due to the under floor air distribution system, the actual floor is 16" above the structural slab. In addition, to keep exterior beam line consistent a 'dog-leg' connection was designed for the end, which most likely added cost to the project.

Alternative Floor Framing System



Figure 14: Typical 30'-0" x 40'-0" interior bay used

Three additional alternative floor systems were design and compared for the typical interior bay in Figure 14. The following floor systems were selected based on architectural, structural, and construction impacts on the existing building:

Lightweight Concrete Composite Deck on three infill beams One-Way Reinforced Concrete Slab with two different beam layouts Two-Way Post-Tensioned Concrete Slab

Composite Floor Framing System

For this alternative floor system the normal weight concrete was replaced with lightweight concrete and a 5 $\frac{1}{2}$ " metal deck was replaced with a 5" metal deck. The metal deck spans 6'-7" from infill beams. W12x19 and W16x26 beams span into a W18x35 girder. However, the W12x19s were used only to help size the girder for the bay. Figure 15 shows the typical bay analyzed for the alternative floor framing system calculation overlaid on the existing system. The calculation included in appendix B refers to the 13th edition of AISC for beam and girder design and Vulcraft for the steel deck design. The deck was checked to meet a 2 hour fire rating with spray on fireproofing and strength requirements. The beams were checked to meet strength requirements in addition to deflection requirements of L/240 for total load and L/360 for live load.



Advantages and Disadvantages Analysis:

Utilizing composite action of the beams and concrete, the resulting system is lighter and shallower. The alternative beams and girder are approximately 16"-18" deep. Including the 5" deck the combine structural depth is approximately 21"-23". The current system has a fire rating of 2 hours and a structural weight of 40.37 psf including beams and girders. Vibration criteria was not evaluated for this report, however further investigation may indicate that the beams may need to be deeper or the slab may need to be thicker to prevent noticeable vibrations. The lightness of this system could affect the lateral system, causing additional bracing to resist lateral forces. For the foundation, the lightness could reduce the size. The addition of the extra beam would not affect the architecture of the space below or above, unless it is exposed or there are vibration issues. The thinner structural depth of the floor system is not so drastic as to change any heights of the mechanical systems in the ceiling or the under floor air distribution system. In addition, the column grid would not have to change, therefore keeping with Renzo Piano's open, light, and transparent feel for the building. Though the slab depth does not drastically change, it could possibly be cheaper to install, but on the other hand it could be slightly more expensive due to admixtures. The beams would need to be fireproofed, but since the current system already has composite beams with fireproofing, there might not be too much of a change, due to an addition beam with smaller depth. However, if cementitious fireproofing or spraved fiber is replaced with intumescent paint, the cost will increase. Another construction issue that may arise is the coordination between the trades, because the additional beam may interfere with mechanical ductwork and electrical conduit.

One-Way Reinforced Concrete Floor System

This floor system uses a one-way reinforced concrete slab to transfer loads to concrete infill beams which frame into girders, which in turn transfer gravity loads to the columns. The typical interior bay size of $30'-0" \ge 40'-0"$ was used to design the floor system. For ease of calculation, it was assumed that this interior bay is continuous on both sides with same dimensions, therefore making slabs, beams, and girder typical. An additional assumption was that the slab and beams are cast integrally with 4000 psi concrete. The structural elements were designed to ACI 318-08 and meet flexural, shear, and deflection requirements stated by the code. PCA Column was use to approximate an appropriate column size to carry the gravity load with live load reduction. The columns assumed in the calculations are 33"x33" at the ground floor. A 2 hour fire rating was obtained by providing a minimum clear cover of 3/4" for slabs and 1 1/2" for beams and girders. See appendix C for design calculations.



Advantages and Disadvantages Analysis:

Two options were analyzed to compare sizes of beam within the one-way concrete slab alternative. Option one shown in Figure 16 has beams spanning the long direction, and option two shown in Figure 17 has beams spanning the short direction. Option one resulted in a $4\frac{1}{2}$ " thick slab with 24x20 beams and 28x24 girders. Option two resulted in a 4 1/2" thick slab with 20x16 beams and 33x28 girders. (Refer to appendix C for reinforcing.) The self weight of the floor system is approximately 112 psf and 110 psf for option one and two respectively. This alternative results in the second heaviest floor system analyzed, which will alter the foundation design and the lateral system design. Attempting to keep the existing spans resulted in the sizes of the beam and girder stated above. This obviously changes the ceiling-to floor sandwich which can cause mechanical, electrical, and plumbing restriction and issues resulting in an even deeper ceiling-to floor sandwich. To decrease the depth of the beams, the spans should be shortened, which would affect the column sizes and placement, changing the space around them. One-way concrete systems are relatively easy to construct, due to less complex structural connections. Formwork will be required and will add an additional cost to the project, but concrete has a lesser lead time than steel. Relative costs of these options are \$31.40 per square foot and \$29.44 per square foot for one and two respectively. Option two is the cheaper alternative system analyzed, but will most likely result in the most drastic structural and architectural changes if implemented. The 2 hour fire rating is built into the concrete and therefore no additional cost for fireproofing will be needed.

Two-Way Post-Tensioned Floor System

This floor system uses a two-way post-tensioned slab. The typical interior bay size of $30'-0" \ge 40'-0"$ was analyzed using ACI 318-08 and resulted in a $11 \frac{1}{2}"$ thick slab with (36) $\frac{1}{2}"$ diameter 270 ksi 7-wire strands in both directions. Minimum mild reinforcement was provided at midspan, while negative moment reinforcement at the supports was determined by strength requirements. An additional $\frac{1}{2}"$ was added for drop panels at the columns, which was required to meet punching shear requirements. A 2 hour fire rating was obtained by providing a $1 \frac{1}{2}"$ clear cover at the bottom of the slab. (See appendix D for design and calculations.) For ease of calculation, it was assumed that this interior bay is continuous on both sides with the same dimensions.



Figure 18:Two-way post-tensioned floor system (www.wikipedia.com)

Advantages and Disadvantages Analysis:

A two-way post-tensioned floor system is efficient when spanning long distances and carrying heavy loads. This system has the smallest structural depth of all the floor systems analyzed in this report. This system is in keeping with Renzo Piano's open layout of the building. The thin ceiling-to-floor sandwich allows for more space for mechanical, electrical, and plumbing systems. The self weight of this alternative system is 144.8 psf. Though this system has the thinnest depth it is the heaviest of the alternative systems. This alternative system would also impact the foundations and lateral system due to the weight, causing the foundations and the lateral systems to be bigger in size. Construction for this system is difficult and requires an experienced construction team. Prior to construction, slab penetrations must be planned to avoid cutting post-tensioning strands. This system's relative cost is \$30.52 per square foot. When compared to the lightweight composite steel system and option two of the one-way concrete system, the post-tension system is slightly more expensive. In addition, the post-tension system takes more time to construct, but with increased spans it has the ability to be more economical and efficient.

Conclusions

Ste	eel		Concrete				
Existing Composite Steel	LW Composite Steel	One-Way Long Beams	One-Way Short Beams	Two-Way Post Tensioned Slab			
N/A	Minor	Major	Major	Yes			
5.50	5.00	4.50	4.50	11.50			
23.40	22.50	28.00	33.00	11.50			
30x30	30x30	33x33	33x33	33x33			
N/A	Minor	Major	Major	Minor-Medium			
Medium	Medium	Easy	Easy	Hard			
2	2	2	2	2			
Yes	Yes	Built-in	Built-in	Built-in			
Minimal	Minimal	Yes	Yes	Yes			
Medium	Medium	Short	Short	Short			
\$31.50/SF	\$30.07/SF	\$31.40/SF	\$29.44/SF	\$30.52/SF			
N/A	Possibly	Yes	Yes	Yes			
N/A	Possibly	Yes	Yes	Yes			
57.5	40.37	112	110	144.8			
N/A		Additional Inv	vestigation Req	uired			
N/A	Maybe	No	No	Yes			
	Existing Composite Steel N/A 5.50 23.40 30x30 N/A 30x30 N/A 30x30 N/A 30x30 N/A 30x30 N/A Steel N/A N/A	Steel Existing Composite Steel LW Composite Steel N/A Minor N/A Minor 5.50 5.00 23.40 22.50 30x30 30x30 30x30 30x30 Minor 4 Medium Minor Yes Yes Minimal Minimal Medium Medium Yes Yes Minimal Minimal Medium Hedium Medium Hedium Moritianal Minimal Moritianal Minimal Minimal Minimal Medium Hedium Solor/SF \$30.07/SF N/A Possibly N/A Possibly N/A Houst N/A Possibly N/A Maybe	Steel LW Composite Steel One-Way Long Beams N/A Minor Major N/A Minor Major 23.40 22.50 28.00 30x30 30x30 33x33 N/A Minor Major Medium Major 4.50 23.40 22.50 28.00 30x30 30x30 33x33 N/A Minor Major Medium Major 4.50 Yes Yes Built-in Minimal Minimal Yes Medium Medium Short Minimal Yes 31.40/SF %30.07/SF \$31.40/SF 112 N/A Possibly Yes N/A Possibly Yes N/A Possibly Yes N/A Maybe No	SteelConcretExisting Composite SteelLW Composite BeamsOne-Way Short BeamsN/AMinorMajorN/AMinorMajor5.50 5.00 4.50 23.40 22.50 28.00 $30x30$ $30x30$ $33x33$ $30x30$ $30x30$ $33x33$ N/AMinorMajorMediumEasy 2 YesYesBuilt-inMinimalMediumShort $31.50/SF$ $$30.07/SF$ $$31.40/SF$ $$7.5$ 40.37 112 N/APossiblyYesYesYesN/APossiblyYesN/AMaybeNo			

 Table 2: Comparison of floor systems analyzed

* The system cost is a rough estimate using RS Means Assemblies Cost Data and RS Means Facilities Construction Cost Data.

After reviewing each floor system, it seems that the two way post-tensioned system is the most possible alternative to the current floor system, due to the system's relative cost, structural weight, structural depth, and impacts on mechanical, electrical, and plumbing systems. In regards to the construction process of post-tensioned system, experienced contractors with knowledge of post-tension construction methods and understanding are required to ensure proper construction. The column sizes for this system will be larger than the current system and it will possibly impact the architectural space and flow. Therefore, if this system is implemented, architectural attention must be maintained.

The 11 $\frac{1}{2}$ " slab of the post-tension system allows for a thinner ceiling-to-floor sandwich which in turn increases the height of the space below and allows for open space with the long spans. Vibrations and deflections can be reduced due to the balanced load that is produced by the $\frac{1}{2}$ " diameter 270 ksi 7-wire strands. However, at the interior supports, substantial amount of mild reinforcement was required for ultimate strength. No additional fireproofing is required.

The other two systems; composite steel framing with lightweight concrete and one-way slab with beams, had benefits which were outweighed by the disadvantages. The disadvantage for the one-way slab with beams system is the total structural depth increased 20%-40% of the existing system. The composite steel framing with lightweight concrete did not result in significant changes of structural depth and cost when compared to the existing system.

It has been determined that due to the information investigated through these sets of analyses that further investigation of the two-way post-tensioned slab system is required to determine feasibility of a possible proposal topic for AE Senior Thesis.

Appendix A: Existing Framing System

Checking the 5 1/2" Composite Deck

It was determined from Thornton Tomasetti's guidance and the architectural plans that the typical office bay metal decking chosen was a 20 gage, 3 inch deep deck with yield strength of 40 ksi, with 2.5 inches of concrete topping. The loading is as follows:

Superimposed Dead Loads:		
Ceiling	5	psf
MEP in raised floor system	12	psf
MEP in ceiling	8	psf
Fireproofing	2	psf
Total SIDL for Floor System Design:	27	psf
Typical Floor Live Loads:		
Office:	50	psf
Partitions:	20	psf
Total LL for Floor System Design:	70	psf
Total Superimposed Live Load for table	97	psf

Clear span = 10'-0'' - 6'' (thick of beam flange) = 9'-6''

The following table was taken from the 2001 Vulcraft catalog on page 48 for a 3 inch deep deck:

()						- (• /											
Total		5	SDI Max. U	nshored								Superir	nposed l	ive Load	, PSF				
Slab	Deck		Clear	Span								С	lear Spa	n (ftin.)					
Depth	Type	1 Span	2 Span	3 Span	7'-0	7'-6	8'-0	8'-6	9'-0	9'-6	10'-0	10'-6	11'-0	11'-6	12'-0	12'-6	13'-0	13'-6	14'-0
	3VLI22	7'-8	9'-7	9'-7	216	195	149	133	120	109	99	90	83	76	70	64	59	54	50
5"	3VLI21	8'-11	11'-3	11'-4	230	206	187	170	128	116	106	96	88	81	74	68	63	58	54
	3VLI20	9"-6	11'-11	12'-4	241	216	196	178	163	150	111	101	93	85	78	72	66	61	57
(t=2")	3VLI19	10'-8	13'-2	13'-7	265	237	214	194	178	163	151	140	102	94	86	79	73	67	62
	3VLI18	11'-8	14'-1	14'-6	289	261	238	218	201	186	173	161	151	142	106	98	92	86	80
44 PSF	3VLI17	12'-7	14'-11	15'-5	309	278	253	231	212	196	182	170	159	150	141	133	97	91	85
	3VLI16	13'-4	15'-8	15'-11	327	294	267	243	223	206	191	178	167	156	147	139	132	96	89
	3VLI22	7'-0	8'-9	8'-9	247	190	170	152	137	124	113	103	94	87	80	73	67	62	57
5 1/2"	3VLI21	8'-4	10'-4	10'-4	262	235	213	162	146	133	120	110	101	92	85	78	72	66	61
	3VLI20	9"-0	11'-5	11'-9	275	247	223	203	186	140	127	116	106	97	89	82	76	70	65
(t=2 1/2")	3VLI19	101.1	101.7	1010	302	270	244	222	203	186	172	128	117	107	98	90	83	77	71
	3VLI18	11'-1	13'-5	13'-11	330	298	271	248	229	212	197	184	173	130	121	112	105	98	92
50 PSF	3VLI17	11'-11	14'-3	14'-9	352	317	288	263	242	224	208	194	182	171	128	119	111	104	97
	3VLI16	12'-8	15'-0	15'-5	373	335	304	277	255	235	218	203	190	178	168	159	117	109	102

(N=9) NORMAL WEIGHT CONCRETE (145 PCF)

Since 140 psf > 97 psf \therefore the deck capacity is **ok**

The following table was taken from the 2001 Vulcraft catalog on page 61 to check for fire protection:

Restrained Assembly	Type of	Concrete Thickness &	U.L. Design	Classified [Deck Type	Unrestrained Beam
Rating	Protection	Type (1)	No. (2,3,4)	Fluted Deck	Cellular Deck (5)	Rating
		2" NW&LW	859 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2,3 Hr.
			022	2121,0121	2.461,0461	
			825 *	1.5VLI,2VLI,3VLI	2VLP, 3VLP	1,1.5,2 Hr.
			831 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2 Hr.
			832 *	1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2,3 Hr.
		2 1/2" NW&LW	833 *	1.5VLI,2VLI,3VLI	2VLP, 3VLP	1.5 Hr.
	Sprayed Fiber		847 *	2VLI,3VLI	3VLP	1,1.5,3 Hr.
			858 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2,4 Hr.
			861 *	12VLI,3VLI		1,1.5 Hr.
			870 *	1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,2 Hr.
			871 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2,3 Hr.
		0.1/all LW/	000 *	0\/112\/11		4 1 14
		2 1/2" NW	864 *	3VLI	3VLP	1.5 Hr.
2 Hr.		3 ¹ /4" LW	860 *	2VLI,3VLI		1,1.5,2 Hr.
(continued)			733 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5Hr.
			826 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2 Hr.

A 3VL20 with 2.5" of normal weight concrete meets the 2 hour fire rating there ok

Typical Beam (W18x35 [40] c=1.5")			
Material Properties:				
Concrete	$f_c =$		4	ksi
Beam	$F_y =$		50	ksi
	$F_u =$		65	ksi
Spacing:	10	ft		
Span:	40	ft		
Loads:				
Dead Loads:				
Slab:	0.053	ksf		
Beam Weight:	0.004	ksf		
MEP/Ceiling:	0.027	ksf		
Live Loads:				
Non-Reduced:	0.070	ksf		
Total dead load:	0.835	klf		
Total live load:	0.700	klf		
Const. dead load (unshored):	0.565	klf		
Const. live load (unshored):	0.200	klf		
w _u =1.2D+1.6L=	2.122	klf		
$V_u = W_u l/2 =$	42.440	k		
$M_u = w_u l^2 / 8 =$	424.400	ftk		

b _{eff} = min of (span/8, 1/2 distance to adj. bm, dist. To edge of slab)=	120.000	in					
Assume a=	1	in					
$Y_2 = t_{slab} - a/2 =$	5.000	in	Table	3-19			
Check I _{reg} :							
Δ=l/240+camber=	3.500	in					
$I_{req} = 5w_{CDL}l^4/(384E\Delta) =$	320.631	in ⁴	<	510	in ⁴	OK	Table 3- 20
Check member strength as un-sho	ored:						
$w_{u(unshored)} = 1.2D + 1.6L =$	0.998	klf					
$M_{u(unshored)} = W_u l^2/8 =$	199.600	ftk	<	249	ftk	OK	Table 3- 19
PNA location=	5						
ΣΟ =	260	ŀ					Table 3-
2Qn	200	ĸ					19
Check member strength:							
φM-=	435	ftk	>	424 400	ftk	OK	Table 3- 19
φV ₀ =	159	k	>	42.440	k	OK	17
T - 11	,					011	
Check a:							
$a=\Sigma Qn/0.85 f_c b_{eff}=$	0.637	in	<	1	in	OK	
Chook A							
	1 2 2 2	:	т_	1170	:4	T-11	2.20
$\Delta_{LL} = 1/300 =$	1.333	in 	I _{LB} –	11/0	in :		2 3-20
$\Delta_{LL} - 5W_{LL}I / (384EI_{LB}) =$	1.188	in	<	1.333	In	ÛK	
CACON STRUGT				Table 3-			
Q _n =	17.2	kips/stud		21			

Since the number of shear studs actually used - 40 studs - is greater than 32 studs, therefore shear studs OK.

Typical Beam (W12x19 [3] c=0"	')							
Material Properties:								
Concrete	$f_c =$		4	ksi				
Beam	$F_v =$		50	ksi				
	$F_u =$		65	ksi				
Spacing:	10	ft						
Span:	5.33	ft						
Loads:								
Slab:	0.053	ksf						
Beam Weight	0.000	ksf						
MEP/Ceiling	0.002	ksf						
Live Loads:	0.027	KSI						
Non-Reduced:	0.070	ksf						
Total dead load:	0.819	klf						
Total live load:	0.700	klf						
Const. dead load (unshored):	0.549	klf						
Const. live load (unshored):	0.200	klf						
w.,=1 2D+1 6L=	2 103	klf						
$V_{u} = \frac{1}{2} \frac{1}{2}$	5 607	k						
$M = w l^2/8$	דרא ד	ftk						
Iviu wui /o	/.+//	ΠK						
b _{eff} =	16.000	in						
Assume a=	1.5	in						
$Y_2 = t_{slab} - a/2 =$	4.750	in						Table 3-19
Check I _{req} :								
$\Delta = 1/240 + \text{camber} =$	0.267	in						
$I_{req} = 5 w_{CDL} l^4 / (384 E \Delta) =$	1.292	in ⁴		<	130	in ⁴	OK	Table 3-20
Charle manh an staran sth as an a	h a wa da							
-1 2D+1 4I -	0.070	ելե						
$W_{u(unshored)} = 1.2D \pm 1.0L =$	0.979	KII C1			0.0	01	OT	m 11 0 10
$M_{u(unshored)} = W_u l^2 / 8 =$	3.480	ttk		<	92.6	ttk	OK	Table 3-19
PNA location=	(0.7	1.						Table 2, 10
2Q _n =	69./	K						1 able 3-19

Check me	mber strength:						
	$\phi \mathbf{M}_{n}$ =	141.5	ftk	>	7.477	ftk	OK Table 3-19
	φV _n =	85.7	k	>	5.607	k	OK
Check a:							
	$a = \Sigma Qn/0.85 f_c b_{eff} =$	1.281	in	<	1.5	in	ОК
Check Δ_{LI}	;						
	Δ _{LL} =I/360=	0.178	in	$I_{LB} =$	261	in ⁴	Table 3-20
	$\Delta_{LL}=5w_{LL}I^4/(384EI_{LB})=$	0.0017	in	<	0.178	in	OK
Check stu	ds:						
	$Q_n =$	17.2	kips/stud		Table 3-21		
	# of studs= $\Sigma Q_n/Q_n$ = Total studs=	4.052 10	use	5	studs/side		

Since the number of shear studs actually used - 3 studs - is less than 10 studs, it could be possible that there is a live reduction occurring in the area where the W12x19 are present, or the level of partial composite action is different, therefore shear studs OK.

Typical Girder (W18x40 [30] c=3/4"	')							
Material Properties:								
Concrete	$f_c =$		4	ksi				
Beam	$F_v =$		50	ksi				
	$F_u =$		65	ksi				
Span:	30.000	ft						
Loads:								
Dead Loads:								
P _{W18x35} :	16.700	k						
P _{W12x19} :	2.184	k						
Beam Weight:	0.040	klf						
Live Loads:								
P _{W18x35} :	14.000	k						
P _{W12x19} :	1.867	k						
Total dead load (P _u):	18.884	k						
Total dead load (w _u):	0.040	klf						
Total live load(P _u):	15.867	k						
Const. dead load (unshored):	12.764	k						
Const. dead load (unshored):	0.040	klf						
Const. live load (unshored):	4.533	k						
$P_u = 1.2D + 1.6L =$	48.047	k						
$w_u = 1.2D + 1.6L =$	0.048	klf						
$V_u = W_u l/2 + P_u =$	48.767	k						
$M_u = w_u l^2 / 8 + P_u l / 3$	485.875	ftk						
$b_{eff} =$	90	in						
Assume a=	1.5	in						
V -4 /2-	175	•						Table 3-
$Y_2 = t_{slab} - a/2 =$	4.75	in						19
Check L								
CHECK I req: $A = 1/240 \pm comb cm^{-1}$	2 250	in						
$\Delta - 1/240 \pm camber =$	2.230	111						Table 3-
$I_{req} = 5w_{DL}l^4/(384E\Delta) + P_{DL}l^3/(28E\Delta) =$	337.126	in ⁴		<	612	in ⁴	OK	20

Check member strength as un-shor						
$P_{u(unshored)} = 1.2D + 1.6L =$	22.570	k				
$w_{u(unshored)} = 1.2D + 1.6L =$	0.048	klf				
$M_{u(unshored)} = W_u l^2 / 8 + P_u l / 3 =$ PNA location=	231.101 4	ftk	<	294	ftk	Table OK 19
ΣQ _n =	351	k				Table 19
Check member strength:						
ϕM_n =	516.5	ftk	>	485.875	ftk	OK
$\phi V_n =$	169	k	>	48.767	k	OK
Check a:						
$a=\Sigma Qn/0.85 f_c b_{eff}=$	1.147	in	<	1.5	in	OK
Check Δ_{LL} :						
∆ _{LL} =I/360=	1.000	in	$I_{LB} =$	1440	in ⁴	Table 3-20
$\Delta_{LL} = P_{LL}I^3/(28EI_{LB}) =$	0.633	in	<	1	in	OK
Check studs:						
0 =	17 200	kins/stud		Table 3-		
Q_n^- # of studs= $\Sigma \Omega / \Omega =$	20 407	nips/stuu	21	∠ı studs/side		
Total studs=	42	450	<u>~</u> 1	57445/5140		

Since the number of shear studs actually used - 30 studs - is less than 42 studs, it could be possible that there is a live reduction occurring in the area where the W12x19 are present, therefore affecting the W18x40 girder, or the level of partial composite action is different, therefore shear studs OK.

System Weight:			
Item	Number/bay	Weight (lbs)	Total
W18x35	3	1400	4200
W18x40	1	1200	1200
Composite System + Deck	1	63600	63600
Self weight (PSF)=			57.50

Appendix B: Lightweight Composite Framing System

Checking the 5" Composite Deck

An alternative typical office bay metal decking chosen was a 20 gage, 3 inch deep deck with yield strength of 40 ksi, with 2 inch of concrete topping. The loading is as follows:

Superimposed Dead Loads:		
Ceiling	5	psf
MEP in raised floor system	12	psf
MEP in ceiling	8	psf
Fireproofing	2	psf
Total SIDL for Floor System Design:	27	psf
Typical Floor Live Loads:		
Office:	50	psf
Partitions:	20	psf
Total LL for Floor System Design:	70	psf
Total Superimposed Live Load for table	97	psf

Clear span = 7'-6''

The following table was taken from the 2001 Vulcraft catalog on page 49 for a 3 inch deep deck:

(14-14		WLIG		NONL	(1101	U)												
Total			SDI Max. U	nshored								Superir	nposed L	ive Load	I, PSF				
Slab	Deck		Clear	Span								C	lear Spa	n (ft-in.)					
Depth	Туре	1 Span	2 Span	3 Span	8'-0	8'-6	9'-0	9'-6	10'-0	10'-6	11'-0	11'-6	12'-0	12'-6	13'-0	13'-6	14'-0	14'-6	15'-0
	3VLI22	9'-1	11-5	11'-5	141	127	115	83	75	67	60	54	49	45	40				
5'	3VLI21	Q' 10	12:1	12'0	153	138	125	113	82	74	67	60	54	49	45	41			
	3VLI20	10'-6	13'-0	13'-5	163	147	133	121	110	102	72	65	59	54	49	44	40		
(t=2")	3VLI19	11-10	14'-4	14'-10	185	166	150	136	124	114	105	97	68	62	57	52	47	43	
	3VLI18	13'-0	15-4	15'-10	244	222	204	188	174	162	151	142	133	126	119	90	85	79	75
34 PSF	3VLI17	14'-0	16-3	16'-6	262	238	218	201	185	172	161	150	141	133	126	119	113	85	80
	3VLI16	14'-5	16'-11	16'-11	277	254	234	217	202	189	177	166	157	149	141	134	127	99	94
	3VLI22	8'-5	10'-6	10'-6	161	121	107	95	85	77	69	62	56	51	46	42			
5 1/2"	3VLI21	9'-5	11'-10	12'-2	175	157	142	105	94	84	76	69	62	56	51	47	42		
	3VLI20	10'-0	12-6	12'-11	186	167	151	138	126	91	82	/4	67	61	56	51	46	42	
(t=2 1/2")	3VLI19	11'-3	13-9	14'-3	211	189	171	155	142	130	120	86	78	71	65	59	54	49	45
	3VLI18	12'-4	14-8	15'-2	278	253	232	214	198	184	172	161	152	118	110	103	97	91	85
39 FSF	3VLI17	13'-4	15-7	16' 0	299	272	248	229	211	196	183	171	161	152	143	110	103	97	91
	3VLI16	14'-0	16'-5	16'-5	316	289	267	247	230	215	202	190	179	170	161	153	146	114	107

(N=14) LIGHTWEIGHT CONCRETE (110 PCF)

Since 163 psf > 97 psf \therefore the deck capacity is **ok**

The following table was taken from the 2001 Vulcraft catalog on page 61 to check for fire protection:

Restrained Assembly	Type of	Concrete Thickness &	U.L. Design	Classified Deck Type		Unrestrained Beam
Rating	Protection	Type (1)	No. (2.3.4)	Eluted Deck	Cellular Deck (5)	Rating
		2" NW&LW	859 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2,3 Hr.
	•		825 *	1.5VLI,2VLI,3VLI	2VLP, 3VLP	1,1.5,2 Hr.
			831 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2 Hr.
			832 *	1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2,3 Hr.
		2 1/2" NW&LW	833 *	1.5VLI,2VLI,3VLI	2VLP, 3VLP	1.5 Hr.
	Sprayed Fiber		847 *	2VLI,3VLI	3VLP	1,1.5,3 Hr.
			858 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2,4 Hr.
			861 *	12VLI,3VLI		1,1.5 Hr.
			870 *	1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,2 Hr.
			871 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2,3 Hr.
		2 1/2" LW	862 *	2VLI,3VLI		1 Hr.
		2 1/2" NW	864 *	3VLI	3VLP	1.5 Hr.
2 Hr.		3 1/4" LW	860 *	2VLI,3VLI		1,1.5,2 Hr.
(continued)			733 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5Hr.
			826 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2 Hr.
			840 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.

A 3VL20 with 2" of lightweight concrete meets the 2 hour fire rating there ok

Typical Beam (W16x26 [40] c=1.5'	')			
Material Properties:				
Concrete	$f_c =$		4	ksi
Beam	$F_y =$		50	ksi
	$F_u =$		65	ksi
Spacing:	7.5	ft		
Span:	40	ft		
Loads:				
Dead Loads:				
Slab:	0.036	ksf		
Beam Weight:	0.0026	ksf		
MEP/Ceiling:	0.027	ksf		
Live Loads:				
Non-Reduced:	0.070	ksf		
Total dead load:	0.493	klf		
Total live load:	0.525	klf		
Const. dead load (unshored):	0.291	klf		
Const. live load (unshored):	0.150	klf		
w _u =1.2D+1.6L=	1.432	klf		
$V_u = w_u l/2 =$	28.633	k		
$M_u = w_u l^2 / 8 =$	286.332	ftk		

b _{eff} = min of (span/8, 1/2 distance to adj. bm, dist. To edge of slab)=	90.000	in					
Assume a=	1	in					T 11 2
$Y_2 = t_{slab} - a/2 =$	5.000	in					1 able 3- 19
Check I _{rea} :							
Δ=l/240+camber=	3.500	in					
$I_{req} = 5 w_{CDL} l^4 / (384 E \Delta) =$	164.884	in ⁴	<	301	in ⁴	OK	Table 3- 20
Check member strength as un-sho	red:						
$w_{u(unshored)} = 1.2D + 1.6L =$	0.589	klf					
$M_{\rm w}$ (unchanged) = $w_{\rm w} l^2/8$ =	117 732	ftk	<	166	ftk	OK	Table 3- 19
PNA location=	4			100		011	
$\Sigma Q_n =$	242	k					
Check member strength:							Table 3-
ϕM_n =	315	ftk	>	286.332	ftk	OK	19
$\phi V_n =$	106	k	>	28.633	k	OK	
Check a: $a=\Sigma On/0.85f$ b $a=$	0 701	in	-	1	in	OK	
a - 2QII/0.001 cOeff	0.791	111		1	111	ŰK	
Check Δ_{LL} :							
Δ _{LL} =I/360=	1.333	in	I _{LB} =	791	in ⁴	Table	3-20
$\Delta_{LL}=5w_{LL}I^4/(384EI_{LB})=$	1.318	in	<	1.333	in	OK	
Check studs:							
Q _n =	17.2	kips/stud		Table 3-21			
# of studs= $\Sigma Q_n/Q_n$ =	14.070	use	15	studs/side			
Total studs=	30						

Since the number of shear studs used - 40 studs - is greater than 30 studs, therefore shear studs OK

Typical Beam (W12x19 [3] c=0"))							
Material Properties:								
Concrete	$f_c =$		4	ksi				
Beam	$F_v =$		50	ksi				
	$F_u =$		65	ksi				
Spacing:	10	ft						
Span:	5.33	ft						
Loads:								
Slab:	0.026	kaf						
Stab. Beam Weight:	0.030	KSI kef						
MEP/Ceiling:	0.002	ksi						
Live Loads:	0.027	K21						
Non-Reduced	0 070	ksf						
Total dead load:	0.650	klf						
Total live load:	0.700	klf						
Const. dead load (unshored):	0.380	klf						
Const. live load (unshored):	0.200	klf						
$w_u = 1.2D + 1.6L =$	1.900	klf						
$V_u = w_u l/2$	5.068	k						
$M_u = w_u l^2/8$	6.757	ftk						
b _{eff} =	16.000	in						
Assume a=	1.5	in						
$Y_2 = t_{slab} - a/2 =$	4.750	in						Table 3-19
Check I_{req} :	0.267	:						
$\Delta = 1/240 + \text{camber} =$	0.267	1n				4		
$I_{req} = 5 w_{CDL} l^4 / (384 E\Delta) =$	0.895	in ⁴		<	130	in ⁴	OK	Table 3-20
Check member strength as up sk	arad.							
$\frac{-1}{2D+1} \frac{1}{6I} = -1$	0.776	1/1						
$W_{u(unshored)} = 1.2D + 1.0L -$	0.770	KII						T 11 A 40
$M_{u(unshored)} = W_u l^2/8 =$	2.761	ttk		<	92.600	ttk	OK	Table 3-19
PNA location=	/	1.						T-11 0 10
$2Q_n =$	69.7	K						Table 3-19

Check member strength:						
ϕM_n =	141.5	ftk	>	6.757	ftk	OK Table 3-19
φV _n =	85.7	k	>	5.068	k	OK
Check a:						
$a=\Sigma Qn/0.85 f_c b_{eff}$	= 1.281	in	<	1.5	in	OK
Check A						
					. 4	
Δ _{LL} =I/360=	0.178	in	$I_{LB} =$	261	in ⁴	Table 3-20
$\Delta_{LL}=5w_{LL}I^4/(384EI_{LB})=$	0.0017	in	<	0.178	in	OK
Check studs:						
Q _n =	= 17.2	kips/stud		Table 3-21		
# of studs= $\Sigma Q_n/Q_n$ =	4.052	use	5	studs/side		
Total studs=	= 10					

Since the number of shear studs used - 3 studs - is less than 10 studs, it could be possible that there is a live reduction occurring in the area where the W12x19 are present therefore shear studs OK

Please note: this beam was kept the same in order to relatively size the girder.

Typical Girder (W18x35 [30] c=3/4")							
Material Properties:								
Concrete	$f_c =$		4	ksi				
Beam	$F_v =$		50	ksi				
	$F_{u} =$		65	ksi				
	ű							
Span:	30.000	ft						
Loads:								
Dead Loads:								
\mathbf{P}_{W16x26} :	9.861	k						
P _{W12x19} :	1.734	k						
Beam Weight:	0.035	klf						
Live Loads:								
P _{W16x26} :	10.500	k						
P _{W12x19} :	1.867	k						
Total dead load (P _u):	11.595	k						
Total dead load (w _u):	0.035	klf						
Total live load(P_{μ}):	12.367	k						
Const. dead load (unshored):	6.825	k						
Const. dead load (unshored):	0.035	klf						
Const. live load (unshored):	0.533	k						
$P_u = 1.2D + 1.6L =$	33.701	k						
$w_u = 1.2D + 1.6L =$	0.042	klf						
$V_u = w_u l/2 + P_u =$	34.331	k						
$M_{u} = w_{u}l^{2}/8 + P_{u}l/3$	341 736	ftk						
	0 11,00							
h _{eff} =	90 000	in						
0011	,							
Assume a=	1	in						
								Table 3-
$Y_2 = t_{slab} - a/2 =$	5	in						19
Check I _{req} :								
$\Delta = 1/240 + \text{camber} =$	2.250	in						T 11 2
$I_{reg} = 5 W_{DI} l^4 / (384 E \Lambda) + P_{DI} l^3 / (28 E \Lambda) =$	184 076	in ⁴		<	510	in ⁴	OK	1 able 3-
$I_{req} = 5w_{DL}l^4/(384E\Delta) + P_{DL}l^3/(28E\Delta) =$	184.076	in ⁴		<	510	in ⁴	OK	Table 3- 20

Check member strength as un-shored	d:					
$P_{u(unshored)} = 1.2D + 1.6L =$	9.044	k				
$w_{u(unshored)} = 1.2D + 1.6L =$	0.042	klf				- 11 -
$M_{u(unshored)} = w_u l^2 / 8 + P_u l / 3 =$ PNA location=	95.163 7	ftk	<	249	ftk	Table 3 OK 19
ΣQ _n =	129	k				Table 3 19
Check member strength:						
$\phi M_n =$	363	ftk	>	341.736	ftk	OK
$\phi V_n =$	159	k	>	34.331	k	OK
Check a:						
$a=\Sigma Qn/0.85 f_c b_{eff}=$	0.422	in	<	1	in	OK
Check Δ_{LL} :						
Δ _{LL} =I/360=	1.000	in	$I_{LB} =$	906	in ⁴	Table 3-20
$\Delta_{LL} = P_{LL}I^3/(28EI_{LB}) =$	0.784	in	<	1	in	OK
Check studs:						
0.=	17.200	kips/stud		Table 3- 21		
# of studs= $\Sigma Q_n/Q_n$ = Total studs=	7.500 16	use	8	studs/side		

Since the number of shear studs used, 30 studs, is greater than 16 studs, therefore shear studs OK

System Weight:			
Item	Number/bay	Weight (lbs)	Total
W16x26	4	1040	4160
W18x36	1	1080	1080
Composite System + Deck	1	43200	43200
Self weight (PSF)=			40.37

Appendix C: One-Way Reinforced Concrete System

Option 1: One Way Slab Design						
Material Properties:						
Concrete in slab	$f_{c} =$	4000	psi			
Concrete in beams	$f_{c} =$	4000	psi			
Reinforcement	$f_v =$	60000	psi			
	,		I			
Loads:						
Superimposed Dead Loads:						
Ceiling:	0.005	ksf				
MEP in raised floor system:	0.012	ksf				
MEP in ceiling:	0.008	ksf				
Total:	0.025	ksf				
Concrete self weight:	0.150	kcf				
Live Loads:						
Non-Reduced:	0.070	ksf				
Option 1:						
Slab span (l _n):	10	ft				
Beam span:	40	ft				
Girder span:	30	ft				
Preliminary h:	1/20 -	4.20			4 5	
h _{slab} :	1/28 =	4.29	111	use	4.5	111
h _{beam} :	1/21 =	0.00	111	use	24	111
h _{girder} :	1/21 =	22.86	in	use	18	111
Beam	24	Х	20			
Girder	28	Х	24			
Assumed Col	33	Х	33			
Slab Design:						
	0.025	lrof				
WD, superimposed —	0.025	KS1				
$W_{D, slab contribution} = hx150/12 =$	0.056	kst				
$w_L =$	0.070	ksf				
Applycic 1 ft width b -	12	in				
marysis i it width, b –	12					

Moments (assume continuous interior span):

Benjamin R. Barben IPD/BIM Structural Option Dr. Andres Lepage 10/28/2009					The Ne	w York Tim New Technical	es I 7 Yo Re
$M = w_u l_n^2 / 11$	-22.85	kin					
$M^{+} = w_u l_n^2 / 16$	15.71	kin					
$V_u = w_u l_n / 2$	1.05	k					
Assume #	4	bars	for s	stirrups			
Assume #	4	bars	for f	lexure			
clear cover=	0.75	in					
d = h-cover-stirrup-0.5 $d_{flexure}$ =	3.00	in					
$A_s =$	0.4	in ²					
Check A _{s,min} :							
$3\sqrt{f_{c}bd}/f_{y} =$	0.114	in^2					
$200 \text{bd}/f_y =$	0.120	in^2					
$A_{s,min} =$	0.120	in ²	<	0.4	OK		
Check A _{s,max} :							
$\rho_{max} =$	0.0206						
$A_{s,max} = \rho bd =$	0.7431	in ²	>	0.4	OK		
Check A _{s,temp} :							
$A_{s,temp} = 0.0018bh =$	0.0972	in^2	<	0.2	OK		
Use $A_{s,temp} =$	0.2	in ²	@	18	in		
Determine M _n :							
$a=A_{s}f_{y}/.85f_{c}b=$	0.588	in					
$\beta =$	0.850						
$c = a/\beta =$	0.692	in					
$\epsilon_s = 0.003 (d-c)/c =$	0.0100	in/in					
$\varepsilon_y = 60/29000 = \phi =$	0.0021	in/in	<	0.0100	OK		
$\phi M_n = \phi A_s f_y(d-a/2) =$	58.45	kin	>	22.85	OK		
Maximum Number of Bars (Table A.7)							
Max numbers of bars =	4		>	2	OK		
Minimum Number of Bars (Table A.8)	, for Crack	Control			07		
Min numbers of bars $=$	2		<	2	OK		

Determine shear strength of beam without	t stirrups:				
$\lambda =$	1				
$V_c = 2 \lambda \sqrt{f_c} b_w d =$	4.55 k	τ	>	1.05	NO SHEAR REINF.
$\phi =$	0.75	-		1.00	
$\phi V_{p} = 0.5 \phi V_{c} =$	1.71 k	ζ.			
Beam Design:					
w _{D, superimposed} =	0.150	ksf			
$w_{D, slab contribution} = hx150/12 =$	0.056	ksf			
$w_{D, beam \text{ contribution}} = (h-t_{slab})xbx150/144 =$	0.406	klf			
$w_L =$	0.070	ksf			
Analysis Trib width10 ft =	120	in			
$w_u = 1.2D + 1.6L =$	4.083	klf			
Moments (assume continuous interior spa	n):				
$M^{-} = w_u l_n^2 / 11$	-6431.05	kin			
$M^{+} = w_u l_n^2 / 16$	4421.35	kin			
$V_u = w_u l_n / 2$	77.57	k			
Assume #	4	bars	for s	tirrups	
Assume #	9	bars	&	0	bars for flexu
clear cover=	1.5	111			
b _w =	20	1n			
$d = h$ -cover-stirrup-0.5 $d_{flexure} =$	21.44	in	0	0	0
Number of bars =			æ	0	0
$A_s \equiv$	7.00	1n ²			
Check If T-beam behavior occurs:					
h=	4 5	in			
h _r h +16h-	92	in			
$b_{\rm W} = 10 {\rm mfm}$	100				
$D_{W} \pm 2(.5c) car (distance) = 25 span length = 25 spa$	100	in			
.25span tengui-	02				
$D_{eff,int} = A 0.95 \ell b (d b / 2) =$	24 207	lin	~	6 121	ΝΟ Τ ΒΕΛΜ
$M_{u,T-Beam} - \psi \ 0.851 \ cDn_f(d-n_f/2) =$	24,307	KIĤ	/	0,431	INU I-DEAM
Determine M_{-1} for $0 = 0$					
$p = 0.85(f / f) \beta(0.003 / (0.003 + 0.005)) =$	0.0181				
$p_{max\phi} = 0.05(1 c/1y)p(0.005/(0.005+0.005)) =$	7 744	in ²			
$A_{s1} = \rho_{max\phi} bd =$	1./44	1N ²			

$a=A_{s1}f_y/.85f_cb=$	6.833	in^2			
$M_{n1}=0.85f_{c}ab(d-a/2)=$	8,374	kin	>	6,431	REINFORCED
Check A _{s,min} :					
$3\sqrt{f_{c}bd}/f_{y} =$	1.356	in ²			
$200 \text{bd/f}_{y} =$	1.429	in ²			
$A_{s,min} =$	1.429	in ²	<	7.00	OK
Check A _{s,max} :					
$\rho_{max} =$	0.0206				
$A_{s,max} = \rho bd =$	8.8506	in ²	>	7.00	OK
Determine M _n :					
$a = A_s f_y / .85 f_c b =$	6.176	in			
β =	0.850				
$c = a/\beta =$	7.266	in 			
$\varepsilon_s = 0.003 (d-c)/c =$	0.0059	in/in			
$\varepsilon_y = 60/29000 = \phi =$	0.0021 0.9	in/in	<	0.0059	OK
$\phi M_n = \phi A_s f_y (d-a/2) =$	6,936	kin	>	6,431	OK
Maximum Number of Bars:					
$b_{min} = 2c_c + 2d_{tr} + nd_b + (n-1)4/3 =$	19.88		<	20	ОК
Minimum Number of Bars (Table A.8), for	Crack C	ontrol			
Min numbers of bars =	3		<	7	ОК
Determine shear strength of beam without	stirrups:				
$\lambda =$	1				
$V_c = 2 \lambda \mathbf{V} \mathbf{f}_c \mathbf{b}_w \mathbf{d} =$	54.23	k	<	77.57	SHEAR REINF.
$\phi =$	0.75				
$\phi V_n = 0.5 \phi V_c =$	20.34	k			
Determine shear strength required by shea	r reinforc	ing:			
$V_u @ d =$	70.27	k			
$\mathrm{V}_{\mathrm{s}} = \mathrm{V}_{\mathrm{u}}/\varphi\text{-}\mathrm{V}_{\mathrm{c}} =$	39.47	k			
$V_s \le 8 V f'_c b_w d =$	216.93	k		OK	
No reinforcing required at:	68.59	in			

Determine maximum spacing of shear reinforcing: $V_v \le 4 \sqrt{f_c} b_w d = 108.47$ k OK $s = 24$ 24 in $s_{max} = 10.72$ in use 10 Determine minimum shear reinforcement: $A_v = 0.75 \sqrt{f_c} b_w s/f_v = 0.158$ in ² $A_v = 500 b_w s/f_v = 0.167$ in ² use 4 $A_{v,min} = 0.167$ in ² use 4 $A_{v,min} = 0.4$ in ³ 0 0 Design Shear Reinforcement: S = $A_v f_v d/V_v = 13.04$ in Use (2) # 4 stirrups: 1 @ 2", 7 @ 10 in each end Girder Design: WD, specimposed = 0.150 ksf w ₀ , sbc constrution = (h-tab.)b150/144x10ft = 0.041 ksf Multiple constrution = (h-tab.)b150/144x10ft = 0.041 ks							
$V_{v} \leq 4 \text{ vf}_{c} b_{w} d = 108.47 \text{ k} OK$ $s = d/2 10.72 \text{ in}$ $s = 24 24 \text{ in}$ $s = 10.72 \text{ in} \text{use} 10 \text{ in}$ Determine minimum shear reinforcement: $A_{v} = 0.75 \text{ vf}_{c} b_{w} s/f_{v} = 0.158 \text{in}^{2}$ $A_{v} = 50 b_{w} s/f_{v} = 0.167 \text{in}^{2}$ $A_{v, sind} = 0.167 \text{in}^{2} \text{ use} 4$ $A_{v, used} = 0.4 \text{in}^{3}$ Design Shear Reinforcement: $s = \Lambda_{v} f_{y} d/V_{v} = 13.04 \text{ in}$ $Use (2) \# 4 \text{ stirrups: } 1 @ 2^{n}, 7 @ 10 \text{ in} \text{each end}$ $\frac{Girder Design:}{V_{v} s = 13.04} \text{ in}$ $Use (2) \# 4 \text{ stirrups: } 1 @ 2^{n}, 7 @ 10 \text{ in} \text{each end}$ $\frac{Girder Design:}{V_{v} s = 0.056} \text{ ksf}$ $w_{D, supermposel} = 0.150 \text{ ksf}$ $w_{D, supermposel} = 0.150 \text{ ksf}$ $w_{D, supermposel} = 0.150 \text{ ksf}$ $w_{D, supermposel} = 0.070 \text{ ksf}$ $M_{v} = 0.070 \text{ ksf}$ $Aralysis Trib width 30 \text{ ft} = 360 \text{ in}$ $w_{u} = 1.2D + 1.6L = 12.953 \text{ klf}$ $\frac{M \text{cments}}{V_{u} = w_{u}l_{u}^{2}/11 \qquad 10492.41 \text{ kin}$ $M^{4} = w_{u}l_{u}^{2}/16 \qquad 7213.53 \text{ kin}$ $V_{u} = w_{u}l_{u}/2 \qquad 176.48 \text{ k}$ $Assume \# 4 \text{ bars for stirrups}$ $Aralysis fir box (24 \text{ stirrups} - 25.38 \text{ in}$ $d_{v} = \text{h-cover-stirrup} - 0.5d_{hexture} = 25.38 \text{ in}$ $d_{v} = \text{h-cover-stirrup} - 0.5d_{hexture} = 24 \text{ in}$	Determine maximum spacing of shear re	einforcing:					
$s = d/2 10.72 \text{ in}$ $s = 24 24 \text{ in}$ $s = 10.72 \text{ in} \text{use} 10 \text{ in}$ Determine minimum shear reinforcement: $A_v = 0.75 \text{ yf}_c \text{ b}_w \text{ s/f}_v = 0.158 \text{ in}^2$ $A_v = 50 \text{ b}_w \text{ s/f}_v = 0.167 \text{ in}^2 \text{ use} 4$ $A_vassel = 0.4 \text{ in}^3$ Design Shear Reinforcement: $s = \Lambda_v f_y d/V_v = 13.04 \text{ in}$ Use (2) # 4 stirrups: 1 @ 2", 7 @ 10 in each end Girder Design: $w_{D, superimposel} = 0.150 \text{ ksf}$ $w_{L} = 0.070 \text{ ksf}$ $M_{L} = 0.070 \text{ ksf}$ $M_{u} = 1.2D + 1.6L = 12.953 \text{ klf}$ Moments (assume continuous interior span): $M' = w_u l_v^2/11 \qquad -10492.41 \text{ kin}$ $M' = w_u l_v^2/16 \qquad 7213.53 \text{ kin}$ $V_u = w_u l_u/2 \qquad 176.48 \text{ k}$ $Assume \# \qquad 4 \text{ bars for stirrups}$ $Assume \# \qquad 9 \text{ bars } \& 10 \text{ bars for}$ $d_{u} = h-cover-stirrup-0.5d_{heaver} = 25.38 \text{ in}$ $d_{u} = h-cover-stirrup-1.5d_{heaver} = 24.13 \text{ in}$	$V_s \leq 4 \sqrt{f_c} b_w d =$	= 108.47	k		OK		
$s = 24 \qquad 24 \qquad \text{in}$ $s_{max} = 10.72 \qquad \text{in} \qquad \text{use} \qquad 10 \qquad \text{in}$ Determine minimum shear reinforcement: $A_v = 0.75 \forall f_c b_w s/f_v = 0.158 \qquad \text{in}^2$ $A_v = 50 b_w s/f_v = 0.167 \qquad \text{in}^2$ $A_{v,min} = 0.167 \qquad \text{in}^2 \qquad \text{use} \qquad 4$ $A_{v,used} = 0.4 \qquad \text{in}^3$ Design Shear Reinforcement: $s = A_v f_y d/V_s = 13.04 \qquad \text{in}$ Use (2) # 4 stirrups: 1 @ 2", 7 @ 10 \quarkstarrow 10 \quarkstarr	s = d/2	2 10.72	in				
$s_{max} = 10.72 \text{ m} \text{ use} 10 \text{ in}$ $Determine minimum shear reinforcement:$ $A_v = 0.75 \sqrt{f_c} b_w s/f_v = 0.158 \text{ in}^2$ $A_v = 50 b_w s/f_v = 0.167 \text{ in}^2$ $A_{v,min} = 0.167 \text{ in}^2 \text{ use} 4$ $A_{v,used} = 0.4 \text{ in}^3$ $Design Shear Reinforcement:$ $s = A_v f_y d/V_s = 13.04 \text{ in}$ $Use (2) \# 4 \text{ stirrups} 1 @ 2^n, 7 @ 10 \text{ in} \text{ each end}$ $Girder Design:$ $w_{D, superimposd} = 0.150 \text{ ksf}$ $w_{D, superimposd} = (h-t_{stat})b150/144 \times 10ft = 0.041 \text{ ksf}$ $w_{D, girder courblution} = (h-t_{stat})b150/144 = 0.588 \text{ klf}$ $w_{L} = 0.070 \text{ ksf}$ $Analysis Trib width 30 \text{ ft} = 360 \text{ in}$ $w_u = 1.2D+1.6L = 12.953 \text{ klf}$ $M \text{ ensula}^2/11 \qquad -10492.41 \text{ kin}$ $M^+ = w_u l_u^2/16 \qquad 7213.53 \text{ kin}$ $V_u = w_u l_u/2 \qquad 176.48 \text{ k}$ $Assume \# 4 \text{ bars for stirrups}$ $Assume \# 9 \text{ bars & 10 \text{ bars for stirrups}}$ $Assume \# 9 \text{ bars & 10 \text{ bars for stirrups}}$ $Assume \# 9 \text{ bars & 210 \text{ bars for stirrups}}$ $Assume \# 24 \text{ in}$ $d_e \text{ h-cover-stirrup-0.5d}_{desare} = 25.38 \text{ in}$ $d_e \text{ h-cover-stirrup-1.5d}_{desare} = 24.13 \text{ in}$	s= 24	4 24	111		• ~		
$\begin{array}{l} \label{eq:constraints} \textbf{Determine minimum shear reinforcement:} \\ A_v = 0.75 \textbf{Vf}_c b_w s/f_y = \ 0.158 \ in^2 \\ A_v = 50 b_w s/f_y = \ 0.167 \ in^2 \\ A_{v,min} = \ 0.167 \ in^2 \ usc \ 4 \\ A_{v,weel} = \ 0.4 \ in^3 \end{array}$	s _{max} =	= 10.72	ın	use	10	ın	
$\begin{array}{cccccccccccccccccccccccccccccccccccc$	Determine minimum shear reinforcemen	nt:					
$\begin{array}{cccccccccccccccccccccccccccccccccccc$	$A_v = 0.75 $ Vf c $b_w $ s/f $_y =$	= 0.158	in ²				
$\begin{array}{cccccccccccccccccccccccccccccccccccc$	$A_v = 50 b_w s/f_y =$	= 0.167	in ²				
$A_{v,used} = 0.4 \text{ in}^3$ $Design Shear Reinforcement:$ $s = A_v f_v d/V_s = 13.04 \text{ in}$ $Use (2) # 4 \text{ stirrups: } 1 @ 2", 7 @ 10 \text{ in} \text{ each end}$ $Girder Design:$ $w_{D, superimposed} = 0.150 \text{ ksf}$ $w_{D, superimposed} = 0.150 \text{ ksf}$ $w_{D, superimposed} = 0.056 \text{ ksf}$ $w_{D, girder contribution} = hx150/12 = 0.056 \text{ ksf}$ $w_{D, girder contribution} = (h-t_{slab})b150/144 x 10ft = 0.041 \text{ ksf}$ $w_{L} = 0.070 \text{ ksf}$ $Analysis Trib width 30 ft = 360 \text{ in}$ $w_u = 1.2D+1.6L = 12.953 \text{ klf}$ $Mr = w_u l_a^2/11 - 10492.41 \text{ kin}$ $M^+ = w_u l_a^2/16 - 7213.53 \text{ kin}$ $V_u = w_u l_a/2 - 176.48 \text{ k}$ $Assume # 4 \text{ bars for stirrups}$ $Assume # 9 \text{ bars } \& 10 \text{ bars for}$ $clear cover = 1.5 \text{ in}$ $b_w = 24 \text{ in}$ $d_r = h-cover-stirrup-0.5d_{flexare} = 25.38 \text{ in}$ $d_r = h-cover-stirrup-1.5d_{flexare} = 24.13 \text{ in}$	A _{v,min} =	= 0.167	in ²	use	4		
Design Shear Reinforcement: $s = \Lambda_v f_y d/V_s = 13.04$ in Use (2) # 4 stirrups: 1 @ 2", 7 @ 10 in each end Girder Design: w_{D_s} superimposed = 0.150 ksf w_{D_s} superimposed = 0.150 ksf w_{D_s} superimposed = 0.056 ksf w_{D_s} superimposed = 0.056 ksf w_{D_s} superimposed = 0.056 ksf w_{D_s} girder contribution = (h-t_stab)b150/144x10ft = 0.041 ksf w_{D_s} girder contribution = (h-t_stab)b150/144 = 0.588 klf $w_{L} = 0.070$ ksf Analysis Trib width 30 ft = 360 in $w_u = 1.2D+1.6L=$ 12.953 klf Moments (assume continuous interior span): M ⁺ = wula ² /11 -10492.41 kin M ⁺ = wula ² /16 7213.53 kin $v_u = wula/2$ 176.48 k Assume # 4 bars for stirrups Assume # 9 bars & 10 bars for clear cover= 1.5 in $b_w = 24$ in $b_w = 24$ in $d_v = h$ -cover-stirrup-0.5d _{Resure} = 25.38 in in	$A_{v,used} =$	= 0.4	in ³				
Design Shear Reinforcement: $s = A_v f_y d/V_s =$ 13.04 in Use (2) # 4 stirrups: 1 @ 2", 7 @ 10 in Use (2) # 4 stirrups: 1 @ 2", 7 @ 10 in Girder Design: $w_{D, superimposed} =$ 0.150 ksf $w_{D, beam contribution} = hx150/12 =$ 0.056 ksf $w_{D, beam contribution} = (h-t_{stab})b150/144x10ft =$ 0.041 ksf $w_{L} =$ 0.070 ksf $w_{L} =$ 0.070 ksf Analysis Trib width 30 ft = 360 in $w_u = 1.2D + 1.6L =$ 12.953 klf Mr = wula ² /11 -10492.41 kin $M^+ = w_u l_n^2/16$ 7213.53 kin $V_u = w_u l_n/2$ 176.48 k Assume # 4 bars for stirrups Assume # 4 bars for stirrups Assume # 9 bars & 10 bars for Mr = w_u l_n/2 176.48 k 10 bars for <td col<="" td=""><td>,</td><td></td><td></td><td></td><td></td><td></td></td>	<td>,</td> <td></td> <td></td> <td></td> <td></td> <td></td>	,					
$s = \Lambda_v f_y d/V_s = 13.04 \text{ in}$ Use (2) # 4 stirrups: 1 @ 2", 7 @ 10 in each end Girder Design: $w_{D, superimposed} = 0.150 \text{ ksf}$ $w_{D, slab contribution} = hx150/12 = 0.056 \text{ ksf}$ $w_{D, bean contribution} = (h-t_{slab})b150/144x10ft = 0.041 \text{ ksf}$ $w_{D, girder contribution} = (h-t_{slab})b150/144 = 0.588 \text{ klf}$ $w_{L} = 0.070 \text{ ksf}$ Analysis Trib width 30 ft = 360 in $w_u = 1.2D + 1.6L = 12.953 \text{ klf}$ Moments (assume continuous interior span): $M^{c} = w_u l_n^2/11 - 10492.41 \text{ kin}$ $M^{+} = w_u l_n^2/16 - 7213.53 \text{ kin}$ $V_u = w_u l_n/2 - 176.48 \text{ k}$ Assume # 4 bars for stirrups Assume # 9 bars & 10 bars for clear cover = 1.5 in $b_w = 24 \text{ in}$ $d_c = h$ -cover-stirrup-0.5d _{Rexue} = 25.38 in d = h-cover-stirrup-1.5d _{Rexue} = 24.13 in	Design Shear Reinforcement:						
Use (2) # 4 stirrups: 1 @ 2", 7 @ 10 in each end Girder Design: $W_{D, superimposed} = 0.150$ ksf $w_{D, slab contribution} = hx150/12 = 0.056$ ksf $w_{D, bean contribution} = (h-t_{slab})b150/144 x10ft = 0.041$ ksf $w_{D, girder contribution} = (h-t_{slab})b150/144 x10ft = 0.588$ klf $w_{L} = 0.070$ ksf Analysis Trib width 30 ft = 360 in $w_{u} = 1.2D+1.6L= 12.953$ klf Moments (assume continuous interior span): $M' = w_{u}l_{n}^{2}/11$ -10492.41 kin $M' = w_{u}l_{n}^{2}/16$ 7213.53 kin $V_{u} = w_{u}l_{n}/2$ 176.48 k Assume # 4 bars for stirrups Assume # 9 bars & 10 bars for clear cover= 1.5 in $b_{w} = 24$ in $d_{v} = h-cover-stirrup-0.5d_{flexure} = 25.38$ in $d_{v} = h-cover-stirrup-1.5d_{flexure} = 24.13$ in	$s = A_v f_y d/V_s =$	= 13.04	in				
Gree (2) # + surrups. (@ 2, ' / ' @ ' to' in' ' each end Girder Design: $w_{D, superimposed} =$ 0.150 ksf $w_{D, slab contribution} = hx150/12 =$ 0.056 ksf $w_{D, girder contribution} = (h-t_{slab})b150/144x10ft =$ 0.041 ksf $w_{L} =$ 0.070 ksf $w_{L} =$ 0.070 ksf $w_{L} =$ 0.070 ksf $M_{u} = 1.2D + 1.6L =$ 12.953 klf Mements (assume continuous interior span): $M^+ = w_u l_n^2/11$ -10492.41 kin $M^+ = w_u l_n^2/16$ 7213.53 kin $V_u = w_u l_n/2$ 176.48 k Assume # 4 bars for stirrups Assume # 9 bars 40 bars for bars & 10 bars for def cover-stirrup-0.5d _{hesure} = 25.38 in def h-cover-stirrup-1.5d _{flesure} = 24.13 in	Use (2) # A stimule 1 @ 2"	7	\widehat{a}	10	in	each and	
Girder Design: $w_{D, superimposed} =$ 0.150 ksf $w_{D, slab contribution} = hx150/12 =$ 0.056 ksf $w_{D, beam contribution} = (h-t_{slab})b150/144x10ft =$ 0.041 ksf $w_{D, girder contribution} = (h-t_{slab})b150/144 =$ 0.588 klf $w_{L} =$ 0.070 ksf $M_L =$ 0.070 ksf $Analysis Trib width 30 ft =$ 360 in $w_u = 1.2D + 1.6L =$ 12.953 klf Mr = w_u l_n^2/11 -10492.41 $M^+ = w_u l_n^2/16$ 7213.53 kin $V_u = w_u l_n/2$ 176.48 k Assume # 4 bars for stirrups Assume # 9 bars & 10 bars for def clear cover= 1.5 in d_t = h-cover-stirrup-0.5d_flexure = 25.38 in	Use(2) # 4 suitups. $I (@ 2)$, /	W	10	111	each end	
$ \begin{split} & w_{D, superimposed} = & 0.150 \ \text{ksf} \\ & w_{D, slab contribution} = hx150/12 = & 0.056 \ \text{ksf} \\ & w_{D, girder contribution} = (h-t_{slab})b150/144x10ft = & 0.041 \ \text{ksf} \\ & w_{D, girder contribution} = (h-t_{slab})b150/144 = & 0.588 \ \text{klf} \\ & w_{L} = & 0.070 \ \text{ksf} \\ & \text{Analysis Trib width 30 ft} = & 360 \ \text{in} \\ & w_{u} = 1.2D+1.6L= & 12.953 \ \text{klf} \\ \end{split} $	Girder Design:						
$ \begin{split} & w_{D, slab contribution} = hx150/12 = 0.056 & ksf \\ & w_{D, beam contribution} = (h-t_{slab})b150/144x10ft = 0.041 & ksf \\ & w_{D, girder contribution} = (h-t_{slab})b150/144 = 0.588 & klf \\ & w_{L} = 0.070 & ksf \\ & Analysis Trib width 30 ft = 360 & in \\ & w_{u} = 1.2D+1.6L = 12.953 & klf \\ \hline \\ & \\ \hline Moments (assume continuous interior span): \\ & M^{-} = w_{u}l_{n}^{2}/11 & -10492.41 & kin \\ & M^{+} = w_{u}l_{n}^{2}/16 & 7213.53 & kin \\ & V_{u} = w_{u}l_{n}/2 & 176.48 & k \\ \hline & \\ & \\ & \\ & \\ & \\ & \\ & \\ & \\ & \\$	WD, superimposed =	0.150	ksf				
$ \begin{split} & w_{D, beam contribution} = (h-t_{slab}) b150/144 x10ft = 0.041 ksf \\ & w_{D, girder contribution} = (h-t_{slab}) b150/144 = 0.588 klf \\ & w_{L} = 0.070 ksf \\ & Analysis Trib width 30 ft = 360 in \\ & w_{u} = 1.2D+1.6L= 12.953 klf \\ \hline \\ & \\ \hline & \\ Moments (assume continuous interior span): \\ & M^{-} = w_{u} l_{n}^{2}/11 -10492.41 kin \\ & M^{+} = w_{u} l_{n}^{2}/16 7213.53 kin \\ & V_{u} = w_{u} l_{n}/2 176.48 k \\ \hline & \\ & \\ & \\ & \\ & \\ & \\ & \\ & \\ & \\$	$w_{D, slab \text{ contribution}} = hx150/12 =$	0.056	ksf				
	$w_{D,\ beam\ contribution} = (h\text{-}t_{slab})b150/144x10ft =$	0.041	ksf				
	$w_{D, \text{ girder contribution}} = (h-t_{slab})b150/144 =$	0.588	klf				
Analysis Trib width 30 ft = 360 in $w_u = 1.2D+1.6L=$ 12.953 klf Moments (assume continuous interior span):	$w_L =$	0.070	ksf				
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$	Analysis Trib width $30 \text{ ft} =$	360	in				
Moments (assume continuous interior span): $M^- = w_u l_n^2/11$ -10492.41 kin $M^+ = w_u l_n^2/16$ 7213.53 kin $V_u = w_u l_n/2$ 176.48 k Assume # 4 bars for stirrups Assume # 9 bars & 10 bars for stirrups $Assume #$ 9 bars & 10 bars for stirrups $Assume #$ 9 bars & 10 bars for stirrups $Assume #$ 9 bars & 10 bars for stirrups $Assume #$ 9 bars & 10 bars for stirrups $Assume #$ 9 bars & 10 bars for stirrups $b_w =$ 24 in $d_t =$ h-cover-stirrup-0.5d_flexure 25.38 in $d =$ h-cover-stirrup-1.5d_flexure 24.13 in	$w_u = 1.2D + 1.6L =$	12.953	klf				
$\begin{array}{rrrrrrrrrrrrrrrrrrrrrrrrrrrrrrrrrrrr$	Moments (assume continuous interior sr	oan):					
$M^{+} = w_{u}l_{n}^{2}/16$ $V_{u} = w_{u}l_{n}/2$ $Assume \#$ $4 bars for stirrups$ $Assume \#$ $9 bars & 10 bars for$ $clear cover = 1.5 in$ $b_{w} = 24 in$ $d_{t} = h-cover-stirrup-0.5d_{flexure} = 25.38 in$ $d = h-cover-stirrup-1.5d_{flexure} = 24.13 in$	$M^{-} = W_{u} l_{0}^{2} / 11$	-10492.41	kin				
$V_{u} = w_{u}l_{n}/2$ $V_{u} = w_{u}l_{n}/2$ $Assume \# \qquad 4 bars for \ stirrups$ $Assume \# \qquad 9 bars \& \qquad 10 bars for$ $clear \ cover = \qquad 1.5 in$ $b_{w} = \qquad 24 in$ $d_{t} = \ h-cover-stirrup-0.5d_{flexure} = \qquad 25.38 in$ $d = \ h-cover-stirrup-1.5d_{flexure} = \qquad 24.13 in$	$M^+ = W_{\rm p} l_{\rm p}^2 / 16$	7213.53	kin				
Assume # 4 bars for stirrups Assume # 9 bars & 10 bars for clear cover= 1.5 in $b_w = 24$ in $d_t = h$ -cover-stirrup-0.5d _{flexure} = 25.38 in d = h-cover-stirrup-1.5d _{flexure} = 24.13 in	$V_{\mu} = W_{\mu} l_{\rho}/2$	176.48	k				
Assume #4barsfor stirrupsAssume #9bars&10barsforclear cover=1.5in b_w =24in d_t = h-cover-stirrup-0.5d _{flexure} =25.38in d_t =24.13in	· u · · u·ii) —	1,0,10					
Assume #9bars&10barsfeclear cover=1.5in b_w =24in d_t = h-cover-stirrup-0.5d _{flexure} =25.38ind= h-cover-stirrup-1.5d _{flexure} =24.13in	Assume #	4	bars	for st	irrups		
$clear cover = 1.5 in$ $b_w = 24 in$ $d_t = h-cover-stirrup-0.5d_{flexure} = 25.38 in$ $d = h-cover-stirrup-1.5d_{flexure} = 24.13 in$	Assume #	9	bars	&	10	bars f	
$b_w = 24 \text{ in}$ $d_t = \text{h-cover-stirrup-0.5d}_{flexure} = 25.38 \text{ in}$ $d = \text{h-cover-stirrup-1.5d}_{flexure} = 24.13 \text{ in}$	clear cover=	1.5	in				
$d_t = h\text{-cover-stirrup-0.5} d_{\text{flexure}} = 25.38 \text{ in}$ $d = h\text{-cover-stirrup-1.5} d_{\text{flexure}} = 24.13 \text{ in}$	$b_w =$	24	in				
$d = h$ -cover-stirrup-1.5 $d_{flexure} = 24.13$ in	d_t = h-cover-stirrup-0.5 $d_{flexure}$ =	25.38	in				
	$d=h$ -cover-stirrup-1.5 $d_{flexure} =$	24.13	in				

	d '= cover+stirrup+0.5d _{flexure} =	2.56	in			
	Number of bars =	5	9	&	5	10
	$A_s =$	11.35	in^2			
	-					
Check If	T-beam behavior occurs:					
	h _f =	4.5	111			
	$b_w + 16h_f =$	96	1n			
	$b_w + 2(.5clear distance) =$	456	in			
	.25span lengtn-	90	in			
	$b_{eff,int} =$	90	1n			
	$M_{u,T-Beam} = \phi \ 0.85f'_{c}bh_{f}(d-h_{f}/2) =$	27,989	kın	>	10,492	NO T-BEAM
	$A_{sf}=0.85f_{c}(b-b_{w})h_{f}/f_{y}=$	16.83	in ²			
	$M_{nf}=0.85f_{c}(b-b_{w})h_{f}(d-h_{f}/2)=$	23,352	kin			
	$M_{nw}=M_u/\phi-M_{nf}=$	(11,693)	kin			
	$\phi M_{nw} =$	(10,278)	kin			
Determin	ne M_{n1} for $\rho = \rho_{max\phi}$ and compare	to \mathbf{M}_{nw} :				
$\rho_{max\phi} = 0$	$0.85(f_c/f_y)\beta(0.003/(0.003+0.005)) =$	0.0181				
	$A_{s1} = \rho_{max\phi} bd =$	10.458	in ²			
	$a=A_{s1}f_y/.85f_cb=$	7.690	in ²			
	$\beta = \beta =$	0.850	•			
	c - a/p - b	9.047	111			SINGLY
	$M_{n1}=0.85f_{c}ab(d-a/2)=$	13,510	kin	>	(11,693)	REINFORCED
Determin	ne M _{n2} :	(25.2.2.2)				
	Mn2 = Mnw - Mn1 =	(25,203)	kın			
Determi	ne As2 assuming $fs = fy$:					
	As2 = Mn2/fv(d-d') =	-19.481	in ²			
Find req	uired A'sw:					
Find req	uired A'sw: f's=0.003(c-d')Es/c =	62358	psi			

	A'sw = As2(fy/f's) =	-19.48	in ²
Required Steel:			
	As= Asf+Asw=	8	in ²
	A's=A'w=	(19)	in ²

Check A _{s,min} :						
	$3\sqrt{f_cbd}/f_y =$	1.831	in ²			
	$200 \text{bd}/f_{\text{v}} =$	1.930	in ²			
	$A_{s,min} =$	1.930	in ²	<	11.35	ОК
Check A _{s,max} :						
	$\rho_{max} =$	0.0206				
	$A_{s,max} = \rho bd =$	11.9522	in ²	>	11.35	ОК
Determine M _n :						
	$a=A_sf_y/.85f_cb=$	8.346	in			
	$\beta =$	0.850				
	$c = a/\beta =$	9.818	1n . ,.			
	$\varepsilon_s = 0.003 (d-c)/c =$	0.0048	1n/1n			
	$\epsilon_y = 60/29000 =$	0.0021	in/in	<	0.0048	ОК
	$-\psi$	0.88	1		10 402	OV
	$\psi M_n - \psi A_s I_y (d-a/2) -$	11,945	K111	/	10,492	ÛK
Maximum Number o	f Bars (Table A.7)					
Max	x numbers of bars =	9		>	5	ОК
Max	x numbers of bars =	8		>	5	ОК
Minimum Number of	f Bare (Table A 8) f	or Crack (ontrol			
Mir	n numbers of bars =	3	,0111101	<	9	ОК
Mir	n numbers of bars =	3		<	8	OK
Determine shear stree	ngth of beam withou	ıt stirrups:				
	$\lambda =$	1				
	$V_c = 2 \lambda V f_c b_w d =$	73.24	k	<	176.48	SHEAR
	$\psi = \psi$	0.75	1			
	$\phi V_n = 0.5 \phi V_c =$	27.46	K			
			ina			
Determine shear stree	ngth required by she	ear reinfore	ung.			
Determine shear stre	ngth required by sh $V_u @ d =$	ear reinforo 150.44	k			
Determine shear stre	ngth required by she $V_u @ d =$ $V_s = V_u/\phi - V_c =$	150.44 127.35	k k			
Determine shear stre	ngth required by she $V_u @ d =$ $V_s = V_u/\phi - V_c =$ $V_s \le 8 \sqrt{f_c} b_w d =$	150.44 127.35 292.95	k k k		ОК	

Determine maximum spacing of shear rea	inforcing:				
$V_s \leq 4 v f_c b_w d =$	146.48	k		OK	
s = d/2	12.06	in			
s= 24	24	in			
s _{max} =	12.06	in	use	12	in
Determine minimum shear reinforcemen	f•				
A = 0.75 yf b s/f =	0.228	in2			
$A_v = 50 \text{ h}_c \text{ b}_v \text{ s/r}_y = 4$	0.220	in-			
$A_v = 50 \text{ by } \text{s/} \text{ly} =$	0.240	1112		4	
$A_{v,min} =$	0.240	111 ²	use	4	
$A_{v,used} =$	0.40	1N ³			
Design Shear Reinforcement:					
$s = A_v f_y d/V_s =$	4.55	in			
					1
Use (2) $\#$ 4 stirrups: 1 @ 2",	22	(a)	4	in	each end
		0			
System Weight:					
Item	#/bay	Spa	n (ft)	Total	
Beams	3	38	3.00 7.05	48750	LBS
Slab	1	Ζ.	1.25	17625	LBS
Self weight (PSF)=	1			112	PSF
				Top	over
Тор			#4 Stirrups (1,) @ 2", (22) @ 4" fron	n each end ຊີ
$\frac{\pi 4}{2} \frac{1}{1000} \frac{\pi 4}{1000} \frac{\pi 4}{1000} \frac{\pi 4}{1000} \frac{\pi 4}{1000} \frac{\pi 4}{1000} \frac{\pi 4}{10000} \frac{\pi 4}{1000000000000000000000000000000000000$	(5)	# 9 bar	s		
(7) #9 bars	(5)	# 10 bars at midspar	s -		1.50 m
1.50					
	i i				
	h 24.00				
Doer					1 COVEr
(7) #9 bars	. (5)	# 10 bars # 9 bars	, _ · _ · · 0		
at supports	<u>↓</u>	at support	s Li		
1.59 in					1.50 in
1.50 in 1.50 in		1.	50 in	<u> </u>	
300 COVEr 0		side c	la la	24.00 in	 !
! Bottom				Bottom	
Figure 19: 24x20 Beam				Fig	ure 20: 28x

Figure 19: 24x20 Beam

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Option 2: One Way Slab Design with Shor	rt Span Bea	ams				
Material Properties:	-					
Concrete in slab	$f'_c =$	4000	psı			
Concrete in beams	$f_c =$	4000	psi			
Reinforcement	$f_y =$	60000	psi			
Loads:						
Superimposed Dead Loads:						_
Ceiling:	0.005	ksf				
MEP in raised floor system:	0.012	ksf				
MEP in ceiling:	0.008	ksf				
Total:	0.025	ksf				
Concrete self weight:	0.150	kcf				
Live Loads:						
Non-Reduced:	0.070	ksf				
Option 2:						
Slab span (l _n):	10	ft				
Beam span:	30	ft				
Girder span:	40	ft				
Preliminary h:						
hslab:	1/28 =	4.29	in	use	4.5	in
hbeam:	l/21 =	0.00	in	use	18	in
hgirder:	l/21 =	17.14	in	use	24	in
beam	20	х	16			
Girder	33	х	28			
Assumed Col	33	X	33			
Slab Design:						
W _{D, superimposed} =	0.025	ksf				
$W_{D, slab \text{ contribution}} = hx150/12 =$	0.056	ksf				
w _I =	0.070	ksf				
Analysis 1 ft width, $b =$	12	in				
$w_u = 1.2D + 1.6L =$	0.210	klf				
Moments (assume continuous interior spa	an):					
$M^{-} = w_u l_n^2 / 11$	-22.85	kin				

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$M^+ = w_u l_n^2 / 16$	15.71	kin					
$V_u = w_u l_n / 2$	1.05	k					
Assume # Assume # clear cover= d= h-cover-stirrup-0.5d _{flexure} = A _s =	4 0.75 3.00 0.4	bars bars in in	for s for f	tirrups lexure			
Check A _{s min} :							
$3\sqrt{f_c}bd/f_v =$	0.114	in ²					
$200 \text{bd/f}_{\text{y}} =$	0.120	in ²					
$A_{s,min} =$	0.120	in ²	<	0.4	OK		
Check A _{s,max} :							
$\rho_{max} =$	0.0206						
$A_{s,max} = \rho bd =$	0.7431	in ²	>	0.4	OK		
Check A _{s.temp} :							
$A_{s,temp} = 0.0018bh =$	0.0972	in ²	<	0.2	OK		
Use $A_{s,temp} =$	0.2	in ²	@	18	in		
Determine M _n :							
$a=A_{s}f_{y}/.85f_{c}b=$	0.588	in					
$\beta =$	0.850						
$c = a/\beta =$	0.692	in					
$\epsilon_s = 0.003 (d-c)/c =$	0.0100	in/in					
$\varepsilon_y = 60/29000 =$	0.0021	in/in	<	0.0100	OK		
$\varphi = \phi M_n = \phi A_s f_y (d-a/2) =$	58.45	kin	>	22.85	OK		
Maximum Number of Bars (Table A 7)							
Max numbers of bars =	4		>	2	OK		
Minimum Number of Bars (Table A.8),	for Crack Co	ontrol					
Min numbers of bars =	2		<	2	OK		
Determine shear strength of beam witho	ut stirrups:						

$\lambda = V_{\rm c} = 2 \lambda \mathbf{V} \mathbf{f}_{\rm c} \mathbf{b}_{\rm w} \mathbf{d} =$	1 4.55 k		>	1.05	NO SHEAR REINF.	
$\phi = \\ \phi V_n = 0.5 \phi V_c =$	0.75 1.71 k					
Beam Design:						
WD, superimposed =	0.150	ksf				
$w_{D, slab \text{ contribution}} = hx150/12 =$	0.056	ksf				
$w_{D, \text{ beam contribution}} = (h-t_{slab})xbx150/144 =$	0.258	klf				
$w_{\rm L}$ =	0.070	ksf				
Analysis 10 ft width, $b =$	120	in				
$w_u = 1.2D + 1.6L =$	3.905	klf				
Moments (assume continuous interior span):						
$M^{-} = w_u l_n^2 / 11$	-3260.79	kin				
$M^{+} = w_u l_n^2 / 16$	2241.80	kin				
$V_u = w_u l_n/2$	54.02	k				
		,	C			
Assume $\#$	4	bars	for 8-	stirrups) have for	foruro
clear cover=	1.5	in	æ	(J Dais 101	Ilexuie
b.=	16	in				
$d = h_{cover_{stirrup}} 0.5 d_{s} =$	17 44	in				
Number of bars =	5	9	&	(0 0	
$A_s =$	5.00	in ²				
Check If T beem behavior occures						
h.=	4 5	in				
h_r +16h=	88	in				
$b_{\rm w}$ + 70 for distance)	104	in				
$D_w + 2(.5Clear distance) = 25span length=$	90	in				
h=	88	in				
$M_{u,T-Beam} = \phi \ 0.85 f_c b h_f (d-h_f/2) =$	18,404	kin	>	3,261	NO T-BEA	М
				,		
Determine M_{n1} for $\rho = \rho_{max\phi}$:						
$\rho_{max\phi} = 0.85 (f_c/f_y)\beta(0.003/(0.003+0.005)) =$	0.0181					
$A_{s1} = \rho_{max\phi} bd =$	5.039	in ²				

$a=A_{s1}f_y/.85f_cb=$	5.558	in^2			
$M_{n1}=0.85f_{c}ab(d-a/2)=$	4,432	kin	>	3,261	REINFORCED
Check A _{s,min} :					
$3\sqrt{f_{c}bd}/f_{y} =$	0.882	in ²			_
$200 \text{bd}/\text{f}_{y} =$	0.930	in ²			
$A_{s,min} =$	0.930	in ²	<	5.00	OK
Check A _{s,max} :					
$ ho_{ m max} =$	0.0206				
$A_{s,max} = \rho bd =$	5.7594	in ²	>	5.00	ОК
Determine M _n :					
$a = A_s f_y / .85 f_c b =$	5.515	in			
β=	0.850				
$c = a/\beta =$	6.488	1n . ,.			
$\varepsilon_s = 0.003 (d-c)/c =$	0.0051	1n/1n			
$\varepsilon_y = 60/29000 =$	0.0021	ın/ın	<	0.0051	ОК
	3,964	kin	>	3,261	OK
Maximum Number of Bars:					
$b_{min} = 2c_c + 2d_{tr} + nd_b + (n-1)4/3 =$	14.96		<	16	ОК
Minimum Number of Bars (Table A.8), for C	rack Contro	1			
Min numbers of bars =	3		<	5	ОК
Determine shear strength of beam without st	irrups:				
$\lambda =$	1				
$V_{c} = 2 \lambda \mathbf{V} \mathbf{f}_{c} \mathbf{b}_{w} \mathbf{d} =$	35.29	k	<	54.02	SHEAR REINF.
$\phi =$	0.75				
$\phi V_n = 0.5 \phi V_c =$	13.23	k			
Determine shear strength required by shear r	einforcing:				
$V_u @ d =$	48.34	k			
$V_s = V_u/\varphi - V_c =$	29.17	k			
$V_s \le 8 \ \textrm{Vf}_c \ b_w \ d =$ No reinforcing required at:	141.16 57.55	k in		OK	

$V_{s} \leq 4 \ \sqrt{r} C_{b} W_{a} d = 70.58 \ k \qquad OK$ $s = d/2 \qquad 8.72 \ in$ $s = 24 \qquad 24 \ in$ $s_{max} = 8.72 \ in \qquad use \qquad 8 \ in$ $Determine minimum shear reinforcement:$ $A_{v} = 0.75 \ \sqrt{r} C_{b} W_{s} S/f_{y} = 0.101 \ in^{2}$ $A_{v} = 50 \ b_{v} S/f_{y} = 0.107 \ in^{2}$ $A_{v} = 50 \ b_{v} S/f_{y} = 0.107 \ in^{2}$ $A_{v,min} = 0.107 \ in^{2} \qquad use \qquad 4$ $A_{v,min} = 0.107 \ in^{2} \qquad use \qquad 4$ $A_{v,min} = 0.4 \ in^{3}$ $Design Shear Reinforcement:$ $s = A_{v} f_{v} d/V_{s} = 14.35 \ in$ $Use (2) \# 4 \ stirrups: 1 @ 2", \qquad 8 \ @ \qquad 8 \ in \ each \ end$ $Girder Design:$ $W_{D, specimprosed} = 0.150 \ ksf$ $W_{D, labe contribution} = hx150/12 = 0.056 \ ksf$ $W_{D, labe contribution} = (h-t_{abb})b150/144x10ft = 0.026 \ ksf$ $W_{D, girder contribution} = (h-t_{abb})b150/144 = 0.831 \ klf$ $W_{L} = 0.070 \ ksf$ $Analysis Trib width 30 \ ft = 300 \ in$ $W_{u} = 1.2D \pm 1.6L = 12.713 \ klf$ $Moments (assume continuous interior span):$ $M^{-} = W_{u} l_{u}^{2}/11 \qquad -19242.97 \ kin$ $M^{+} = W_{u} l_{u}^{2}/16 \qquad 13229.54 \ kin$ $V_{u} = W_{u} l_{u}^{2}/16 \qquad 13229.54 \ kin$ $V_{u} = W_{u} l_{u}^{2}/16 \qquad 13229.54 \ kin$	Determine maximum spacing of shear rein	forcing					
$s = d/2 in s = d/2 in s = d/2 in s = 24 24 in s_{max} = 8.72 in usc 8 in$ Determine minimum shear reinforcement: $A_v = 0.75 v f_c b_w s/f_v = 0.101 in^2 A_v = 50 b_w s/f_v = 0.107 in^2 usc 4 A_{v,nin} = 0.107 in^2 usc 4 A_{v,nin} = 0.107 in^2 usc 4 A_{v,ased} = 0.4 in^3$ Design Shear Reinforcement: $s = A_v f_y d/V_s = 14.35 in Usc (2) \# 4 stirrups: 1 @ 2", 8 @ 8 in each \ end$ Girder Design: $w_{D, superimposel} = 0.150 ksf w_{D, superimposel} = 0.026 ksf w_{D, superimposel} = 0.026 ksf w_{D, superimposel} = 0.070 ksf Mu_v = 0.070 ksf Mu_v = 1.2D+1.6L = 12.713 klf$ Moments (assume continuous interior span): $M = w_u l_u^2/11 \qquad 19242.97 kin M^* = w_u l_u^2/16 \qquad 13229.54 kin V_u = w_u l_u^2/16 \qquad V_u = w_u l_u^2/16 \qquad V_u = v_u l_u^2/16 \qquad V_u$	V < 4 Jf b $d =$	70	58 ŀ		OK		
$s=24 \qquad 24 in$ $s=24 \qquad 24 in$ $s_{max} = 8.72 in \qquad use \qquad 8 in$ Determine minimum shear reinforcement: $A_v = 0.75 yf_c b_w \ s/f_v = 0.101 in^2$ $A_v = 50 b_w \ s/f_v = 0.107 in^2$ $A_v = 0.4 in^3$ Design Shear Reinforcement: $s = A_v f_y d/V_s = 14.35 in$ Use (2) # 4 stirrups: 1 @ 2", 8 @ 8 in each end $Girder Design$ $w_{D, superimposel} = 0.150 ksf$ $w_{D, superimposel} = 0.150 ksf$ $w_{D, superimposel} = 0.150 ksf$ $w_{D, superimposel} = 0.056 ksf$ $w_{D, bala contribution} = (h-t_{stab})b150/144 \times 10ft = 0.026 ksf$ $w_{D, bala contribution} = (h-t_{stab})b150/144 = 0.831 klf$ $w_{L} = 0.070 ksf$ $m_{w_{L}} = 12.0713 klf$ Moments (assume continuous interior span): $M = w_u l_u^2/11 \qquad -19242.97 kin$ $M^{-} = w_u l_u^2/16 \qquad 13229.54 kin$ $V_u = w_u l_u^2/16 \qquad 13229.54 kin$ $V_u = w_u l_u^2/16 \qquad 13229.54 kin$	$v_s \leq 4 v_{1c} b_w d =$ s = d/2	70	72 in		OK		
$s_{max} = 8.72 \text{ in use } 8 \text{ in}$ Determine minimum shear reinforcement: $A_v = 0.75 \text{ Vf}_v b_w \text{ s/} f_v = 0.101 \text{ in}^2$ $A_v = 50 \text{ b}_w \text{ s/} f_v = 0.107 \text{ in}^2$ $A_v = 50 \text{ b}_w \text{ s/} f_v = 0.107 \text{ in}^2$ $A_v = 0.4 \text{ in}^3$ Use 4 $A_{v,used} = 0.4 \text{ in}^3$ Design Shear Reinforcement: $s = A_v f_v d/V_v = 14.35 \text{ in}$ Use $(2) \# 4 \text{ stirrups: } 1 @ 2^v$, 8 @ 8 in each end Girder Design: $w_{D, superimposed} = 0.150 \text{ ksf}$ $w_{D, superimposed} = 0.150 \text{ ksf}$ $w_{D, superimposed} = 0.150/12 = 0.056 \text{ ksf}$ $w_{D, superimposed} = 0.150/144 \text{ stirrups: } 1 @ 2^v$, 8 @ 8 in each end $M_v = 1.2D + 1.6L = 12.713 \text{ klf}$ Moments (assume continuous interior span): $M = w_v \ln^2/11 \qquad -19242.97 \text{ kin}$ $M = w_v \ln^2/16 \qquad 13229.54 \text{ kin}$ $v_u = w_u \ln^2/2 \qquad 236.77 \text{ k}$	s = 24		24 in				
Determine minimum shear reinforcement: $A_v = 0.75 \ Vf_c \ b_w \ s/f_v = 0.101 \ in^2$ $A_v = 50 \ b_w \ s/f_v = 0.107 \ in^2$ $A_v = 50 \ b_w \ s/f_v = 0.107 \ in^2$ $A_{v,min} = 0.107 \ in^2$ $A_{v,ased} = 0.4 \ in^3$ Design Shear Reinforcement: s = $A_v f_y d/V_v = 14.35 \ in$ Use (2) # 4 stirrups: 1 @ 2", 8 @ 8 in each end Girder Design: $w_{D, superimposed} = 0.150 \ ksf$ $w_{D, superimposed} = 0.160 \ ksf$ $w_{D, superimposed} = 0.150 \ ksf$ $w_{D, superimposed} = 0.070 \ ksf$ $w_{D, superimposed} = 0.070 \ ksf$ $w_{u} = 1.2D + 1.6L = 12.713 \ klf$ M = w_u l_n^2/11 \ -19242.97 \ kin $M^{-} = w_u l_n^2/16 \ 13229.54 \ kin$ $v_u = w_u l_n/2 \ 236.77 \ k$	s _{max} =	8.	72 in	use		8	in
Determine minimum shear reinforcement: $A_{v} = 0.75 \text{ Vf}_{c} \text{ bw} \text{ s/fy} = 0.101 \text{ in}^{2}$ $A_{v} = 50 \text{ bw} \text{ s/fy} = 0.107 \text{ in}^{2} \text{ use } 4$ $A_{v, arian} = 0.107 \text{ in}^{2} \text{ use } 4$ $A_{v, aread} = 0.4 \text{ in}^{3}$ Design Shear Reinforcement: $s = \Lambda_{v} f_{v} d/V_{s} = 14.35 \text{ in}$ $Use (2) \# 4 \text{ stirrups: } 1 @ 2", 8 @ 8 \text{ in each end}$ Girder Design: w _D , superimposed = 0.150 ksf w _D , superimposed = 0.150 ksf w _D , superimposed = 0.056 ksf w _D , bab contribution = hx150/12 = 0.056 ksf w _D , girder contribution = (h-t_{slab})b150/144 = 0.831 klf w _L = 0.070 ksf Analysis Trib width 30 ft = 360 in w _u = 1.2D+1.6L= 12.713 klf Moments (assume continuous interior span): M' = w_{u}l_{u}^{2}/11 - 19242.97 kin M' = w_{u}l_{u}^{2}/16 13229.54 kin V _u = w_{u}l_{u}/2 236.77 k							
$\begin{array}{llllllllllllllllllllllllllllllllllll$	Determine minimum shear reinforcement:						
$\begin{array}{rcl} A_v = 50 \ b_w \ s/f_y = & 0.107 & in^2 \\ A_{v,min} = & 0.107 & in^2 & use & 4 \\ A_{v,used} = & 0.4 & in^3 \end{array}$	$A_v = 0.75 \mathbf{v} f_c \mathbf{b}_w \mathbf{s} / \mathbf{f}_y =$	0.1	01 in ²				
$\begin{array}{rrrrrrrrrrrrrrrrrrrrrrrrrrrrrrrrrrrr$	$A_v = 50 b_w s/f_y =$	0.1	07 in ²				
$A_{v,used} = 0.4 \text{ in}^{3}$ Design Shear Reinforcement: $s = A_v f_y d/V_s = 14.35 \text{ in}$ Use (2) # 4 stirrups: 1 @ 2", 8 @ 8 in each end Girder Design: WD, superimposed = 0.150 ksf wD, superimposed = 0.150 ksf wD, superimposed = 0.026 ksf wD, beam contribution = (h-tstab)b150/144 x10ft = 0.026 ksf wD, girder contribution = (h-tstab)b150/144 = 0.831 klf wL = 0.070 ksf Analysis Trib width 30 ft = 360 in wu = 1.2D+1.6L= 12.713 klf Moments (assume continuous interior span): M = wu l_a^2/11 - 19242.97 kin M^ = wu l_a^2/16 13229.54 kin Vu = wu l_a/2 236.77 k	$A_{v,min} =$	0.1	07 in ²	use		4	
Design Shear Reinforcement: $s = A_v f_y d/V_s =$ 14.35 in Use (2) # 4 stirrups: 1 @ 2", 8 @ 8 in each end Girder Design: 0.150 ksf $w_{D, superimposed} =$ 0.150 ksf $w_{D, superimposed} =$ 0.056 ksf $w_{D, babe contribution} = hx150/12 =$ 0.056 ksf $w_{D, beam contribution} = (h-t_{slab})b150/144 x10ft =$ 0.831 klf $w_{L} =$ 0.070 ksf $Analysis Trib width 30 ft =$ 360 in $w_u = 1.2D + 1.6L =$ 12.713 klf Moments (assume continuous interior span): M $M^{+} = w_u l_n^2/16$ 13229.54 kin $V_u = w_u l_n/2$ 236.77 k	$A_{v,used} =$	C).4 in ³				
Design Shear Keinforcement: $s = A_v f_y d/V_s =$ 14.35 in Use (2) # 4 stirrups: 1 @ 2", 8 @ 8 in each end Girder Design: $w_{D, superimposed} =$ 0.150 ksf $w_{D, slab contribution} = hx 150/12 =$ 0.056 ksf $w_{D, beam contribution} = (h-t_{slab}) b150/144x 10ft =$ 0.026 ksf $w_{D, girder contribution} = (h-t_{slab}) b150/144x =$ 0.831 klf $w_{L} =$ 0.070 ksf $Analysis Trib width 30 ft =$ 360 in $w_u = 1.2D + 1.6L =$ 12.713 klf Mr = w_u l_n^2/11 -19242.97 $M^+ = w_u l_n^2/16$ 13229.54 kin $V_u = w_u l_n/2$ 236.77 k							
$s = A_v t_y d/V_s = 14.35 \text{ in}$ Use (2) # 4 stirrups: 1 @ 2", 8 @ 8 in each end Girder Design: WD, superimposed = 0.150 ksf WD, slab contribution = hx150/12 = 0.056 ksf WD, beam contribution = (h-t_{slab})b150/144x10ft = 0.026 ksf WD, girder contribution = (h-t_{slab})b150/144 = 0.831 klf WL = 0.070 ksf Analysis Trib width 30 ft = 360 in Wu = 1.2D+1.6L= 12.713 klf Moments (assume continuous interior span): M' = wuln ² /11 -19242.97 kin M' = wuln ² /16 13229.54 kin Vu = wuln/2 236.77 k Arrow the contribution of the time for the span of the time of the time.	Design Shear Reinforcement:		о г .				
Use (2) # 4 stirrups: 1 @ 2", 8 @ 8 in each end Girder Design: $w_{D, superimposed} = 0.150 \text{ ksf}$ $w_{D, slab contribution} = hx150/12 = 0.056 \text{ ksf}$ $w_{D, beam contribution} = (h-t_{slab})b150/144x10ft = 0.026 \text{ ksf}$ $w_{D, girder contribution} = (h-t_{slab})b150/144 = 0.831 \text{ klf}$ $w_{L} = 0.070 \text{ ksf}$ Analysis Trib width 30 ft = 360 in $w_{u} = 1.2D+1.6L= 12.713 \text{ klf}$ Moments (assume continuous interior span): $M^{+} = w_{u}l_{a}^{2}/11 - 19242.97 \text{ kin}$ $M^{+} = w_{u}l_{a}^{2}/16 - 13229.54 \text{ kin}$ $v_{u} = w_{u}l_{n}/2 - 236.77 \text{ k}$	$s = A_v f_y d/V_s =$	14.	35 in				
Girder Design: $w_{D, superimposed} =$ 0.150 ksf $w_{D, slab contribution} = hx150/12 =$ 0.056 ksf $w_{D, beam contribution} = (h-t_{slab})b150/144x10ft =$ 0.026 ksf $w_{D, girder contribution} = (h-t_{slab})b150/144x10ft =$ 0.831 klf $w_{L} =$ 0.070 ksf $Analysis Trib width 30 ft =$ 360 in $w_u = 1.2D + 1.6L =$ 12.713 klf Moments (assume continuous interior span): $M^{\circ} = w_u l_n^2/11$ -19242.97 kin $M^{\circ} = w_u l_n^2/16$ 13229.54 kin $V_u = w_u l_n/2$ 236.77 k	Use (2) # 4 stirrups: 1 @ 2".	8	a	8	in		each end
Girder Design: $w_{D, superimposed} =$ 0.150 ksf $w_{D, slab contribution} = hx150/12 =$ 0.056 ksf $w_{D, beam contribution} = (h-t_{slab})b150/144x10ft =$ 0.026 ksf $w_{D, girder contribution} = (h-t_{slab})b150/144x10ft =$ 0.831 klf $w_{L} =$ 0.070 ksf $M_{L} =$ 0.070 ksf $w_{u} = 1.2D + 1.6L =$ 12.713 klf Moments (assume continuous interior span): Mr = w_u l_n^2/11 -19242.97 kin $M^+ = w_u l_n^2/16$ 13229.54 kin Vu = w_u l_n/2 236.77 k			Ŭ				
$ \begin{array}{llllllllllllllllllllllllllllllllllll$	Girder Design:						
$ \begin{array}{llllllllllllllllllllllllllllllllllll$	WD, superimposed =	0.150	ksf				
$ \begin{split} & w_{D, beam \ contribution} = (h-t_{slab})b150/144x10ft = 0.026 \ ksf \\ & w_{D, girder \ contribution} = (h-t_{slab})b150/144 = 0.831 \ klf \\ & w_{L} = 0.070 \ ksf \\ & Analysis \ Trib \ width \ 30 \ ft = 360 \ in \\ & w_{u} = 1.2D+1.6L= 12.713 \ klf \\ \hline \begin{tabular}{lllllllllllllllllllllllllllllllllll$	$w_{D, slab contribution} = hx150/12 =$	0.056	ksf				
$ \begin{split} w_{D, \text{ girder contribution}} &= (h-t_{\text{slab}})b150/144 = 0.831 \text{ klf} \\ w_{L} &= 0.070 \text{ ksf} \\ & \text{Analysis Trib width 30 ft} = 360 \text{ in} \\ w_{u} &= 1.2D+1.6L = 12.713 \text{ klf} \end{split} $	$w_{D, \text{ beam contribution}} = (h-t_{slab})b150/144x10ft =$	0.026	ksf				
$w_{L} = 0.070 \text{ ksf}$ Analysis Trib width 30 ft = 360 in $w_{u} = 1.2D+1.6L= 12.713 \text{ klf}$ Moments (assume continuous interior span): $M^{-} = w_{u}l_{n}^{2}/11 -19242.97 \text{ kin}$ $M^{+} = w_{u}l_{n}^{2}/16 13229.54 \text{ kin}$ $V_{u} = w_{u}l_{n}/2 236.77 \text{ k}$	$w_{D, \text{ girder contribution}} = (h-t_{slab})b150/144 =$	0.831	klf				
Analysis Trib width 30 ft = 360 in $w_u = 1.2D+1.6L=$ 12.713 klf Moments (assume continuous interior span): -19242.97 kin $M^- = w_u l_n^2/11$ -19242.97 kin $M^+ = w_u l_n^2/16$ 13229.54 kin $V_u = w_u l_n/2$ 236.77 k	$w_L =$	0.070	ksf				
$w_u = 1.2D+1.6L=$ 12.713 klf Moments (assume continuous interior span):	Analysis Trib width 30 ft =	360	in				
Moments (assume continuous interior span): $M^- = w_u l_n^2 / 11$ -19242.97 kin $M^+ = w_u l_n^2 / 16$ 13229.54 kin $V_u = w_u l_n / 2$ 236.77 k	$w_u = 1.2D + 1.6L =$	12.713	klf				
Moments (assume continuous interior span): $M^- = w_u l_n^2 / 11$ -19242.97 kin $M^+ = w_u l_n^2 / 16$ 13229.54 kin $V_u = w_u l_n / 2$ 236.77 k							
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$	Moments (assume continuous interior span	n):					
$M^{+} = w_{u} l_{n}^{2} / 16 $ $V_{u} = w_{u} l_{n} / 2 $ $M^{+} = w_{u} l_{n} / 2 $	$M^{-} = w_{u}l_{n}^{2}/11$	-19242.97	kin				
$V_u = w_u l_n/2$ 236.77 k	$M^+ = w_u l_n^2 / 16$	13229.54	kin				
	$V_u = W_u l_n/2$	236.77	k				
A contact T // base to strendor	A course #	Л	bare	for stimes	26		
Assume # 9 bars & 10 bars for flexure	Assume #	4	bars	&	,s 10	h	ars for flexure
clear cover= 1.5 in	clear cover=	1.5	in		••	5	
$b_w = 28$ in	b _w =	28	in				
$d_t = h$ -cover-stirrup-0.5 $d_{flexure} = 30.38$ in	d_t = h-cover-stirrup-0.5 $d_{flexure}$ =	30.38	in				
	d= h-cover-stirrup-1.5d _{flexure} =	29.13	in				
	d= h-cover-stirrup-1.5d _{flexure} =	29.13	in				

d '= cover+stirrup+0.5d _{flexure} =	2.56	in			
Number of bars =	7	9	&	7	10
$A_s =$	15.89	in ²			
Check If T-beam behavior occurs:					
h _f =	4.5	in			
$b_w + 16h_f =$	100	in			
$b_w + 2(.5 \text{clear distance}) =$	332	in			
.25span length=	120	in			
$b_{eff,int} =$	100	in			
$M_{u,T-Beam} = \phi \ 0.85 f_c b h_f (d-h_f/2) =$	37,753	kin	>	19,243	NO T-BEAM
$A_{sf} = 0.85 f_{c} (b - b_{w}) h_{f} / f_{v} =$	18.36	in ²			
$M_{nf} = 0.85 f_{c}(b-b_{w})h_{f}(d-h_{f}/2) =$	30,983	kin			
$M_{ow} = M_{u} / \phi - M_{of} =$	(9.601)	kin			
φM _{am} =	(8 424)	kin			
Ψ ⁱⁿ InW	(0,121)	KIII			
Determine M_{n1} for $\rho = \rho_{max\phi}$:					
$\rho_{max\phi} = 0.85(f_c/f_y)\beta(0.003/(0.003+0.005)) =$	0.0181				
$A_{s1} = \rho_{max\phi} bd =$	14.730	in ²			
$a = A_{s1} f_v / .85 f_c b =$	9.284	in ²			
$\beta =$	0.850				
$c = a/\beta =$	10.922	in			
$M_{p1}=0.85f_{c}ab(d-a/2)=$	22.743	kin	>	(9.601)	SINGLY REINFORCED
	,+ 10			(,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,	
Determine M _{n2} :					
Mn2 = Mnw - Mn1 =	(32,344)	kin			
Determine A_{2} assuming $f_{2} = f_{2}$					
$As^2 = Mn^2 / fr(d, d') =$	20.295	in2			
1152 - 10112/19(d-d) -	-20.293	111-			
Find required A'sw:					
f's=0.003(c-d')Es/c =	66588	psi			
fs=	60000	psi			
A'sw=As2(fy/f's)=	-20.29	in ²			
Required Steel:					
	12	in?			
$A_{1} = A_{1} = A_{1$	(20)	in ²			
A S - A W =	(20)	1114			

Check A _{s,min} :					
$3\sqrt{f_cbd}/f_y =$	2.579	in ²			
$200 \text{bd/f}_{y} =$	2.718	in ²			
$A_{s,min} =$	2.718	in ²	<	15.89	ОК
Check A _{s,max} :					
$ ho_{max} =$	0.0206				
$A_{s,max} = \rho bd =$	16.8343	in ²	>	15.89	ОК
Determine M _n :					
$a=A_sf_y/.85f_cb=$	10.015	in			
$\beta =$	0.850				
$c = a/\beta =$	11.782	in			
$\epsilon_s = 0.003 (d-c)/c =$	0.0047	in/in			
$e_y = 60/29000 =$	0.0021	in/in	<	0.0047	ОК
$\phi =$	0.88				
$\phi M_n = \phi A_s f_y (d-a/2) =$	20,173	kin	>	19,243	ОК
Maximum Number of Bare (Table A 7)					
Max numbers of bars =	10		>	7	OK
Max numbers of bars =	10		>	7	OK
Minimum Number of Bars (Table A.8), for	Crack Cor	ntrol			
Min numbers of bars =	3		<	9	ОК
Min numbers of bars =	3		<	8	ОК
Determine shear strength of beam without	stirruns				
$\lambda =$	1				
$V_c = 2 \lambda \sqrt{f_c} b_w d =$	103.15	k	<	236.77	SHEAR REINF.
$\phi =$	0.75				
$\phi V_n = 0.5 \phi V_c =$	38.68	k			
Determine shear strength required by shear	r reinforcin	g:			
$V_u @ d =$	205.92	k			
$V_s = V_u/\varphi - V_c =$	171.40	k			
$V_a \leq 8 \sqrt{f_a} b_m d =$	410 (1	1-		OK	
	412.01	K		UK	

Determine maximum spacing of shear reinf	orcing:				
$V_s \le 4 v f_c b_w d =$	206.31	k		OK	
s = d/2	14.56	in			
s= 24	24	in			
s _{max} =	14.56	in	use	14	in
Determine minimum shear reinforcement:					
$A_{v} = 0.75 v f_{c} b_{w} s/f_{y} =$	0.310	in ²			
$A_v = 50 b_w s/f_v =$	0.327	in ²			
$A_{v,min} =$	0.327	in ²	use	4	
$A_{yused} =$	0.40	in ³			
.,					
Design Shear Reinforcement:					
$s = A_v f_y d/V_s =$	4.08	in			
Use (2) # 4 stirrups: 1 @ 2",	31	@	4	in	each end
System Weight:					
Item	#/bay	Sna	n (ft)	Total	
Beams	π/Day 1	3 ра . 27	n (n) 7.67	31000	IBS
Girder	+ 1	37	.07	33250	LBS
Slab	1	57	.23	67500	LBS
Salf weight (DSE) =	1			110	DSE
				110	
۲ ор 8 <u># 4 Stirrups (1)</u> @ 2°, (8) @ 8° from each end				Тор	
	··-·+	# 4	Stirrups (1) () 2", (31) @ 4" from eac	hend g
(5) #9 bars	(7)	#9 bars		000000	
1.56	(7)	# 10 bars−· at midspan		0-0-0-0-0 -0-(●
	11 20.00 ir				h no on in
					ŝ
1 20 API					Dover
(5) #9 bars $- \cdot - $	(7)	# 10 bars			•
at supports		at supports			
8					20 in
1.50 m side cover b _π		1.50 il side cove		b	1.50 in
16.00 in			-	28.00 in	
: Bottom				i Bottom	
Figure 21: 20x16 Beam				Figure 22: 3	3x28 Girder

APPENDIX D: TWO-WAY POST-TENSIONED CONCRETE SYSTEM

Two-Way Post-Te	ensioned Design					
Loads:				0.16		
Framing Dead Load	ł		=	Self		
Superimposed Dea	d Load		=	25	psf	
Live Load			=	70	psf	
2 hour fire-rating					L	
0						
Materials:						
Concrete:	Normal weight			150	pcf	
		\mathbf{f}_{c}	=	5,000	psi	
		$\mathbf{f'_{ci}}$	=	3,000	psi	
					-	
Rebar:		f_v	=	60,000	psi	
Post Tension:	Unbonded tendons				I	
	$1/2"\phi$, 7-wire strands	А	=	0.153	in^2	
		$\mathbf{f}_{\mathbf{p}}$				
		u	=	270	ksi	_
	Estimated prestress losses		=	15	ksi ACI 18.6	
	$f_{se} = 0.7 f_{pu}$ - losses		=	174	ksi ACI 18.5	.1
	$Peff = A*f_{r}$		=	26.62	kips/tendo	
	i chi ili ise			20.02		
Determine Prelim	inary Slab Thickness:					
	Long Span:					
		L_1	=	30	ft	
		L_2	=	40	ft	
	h=L _{dormet} /45	h	=	10.67	in	
	preliminary slab thickness	h	=	11.50	in	
	I man je me i se					
Loading:						
Framing Dead Load	$d = self weight = t_{slab}(150pcf)$		=	143.75	psf	
Superimposed Dea	d Load		=	25	psf	
Live Load			=	70	psf	
Design of East W	est Frame (40 ft span 30ft wi	dth)				
Section Properties	3:					
	$A = bh = (360 \text{ in})t_{slab}$		=	4140	in^2	
	$S = bh^2/6 = (360)$		_	5005	• 2	
	$(t_{slab})^2/6$		=	7935	1 n ⁵	

Design Parameters:					
Allowable stresses: Class U (ACI 18.3.3):					
At time of jacking (ACI 18.4.1)	:				
f	ci	=	3,000	psi	
Compression = $0.60 f_{ci}$		=	1,800	psi	
Tension = $3\sqrt{f_{ci}}$		=	164	psi	
At service loads (ACI 18	.4.2((a) :	and 18.3.3):		
i	f'c	=	5,000	psi	
Compression = 0.45 f_{c}		=	2,250	psi	
Tension = $6\sqrt{f_c}$		=	424	psi	
Average precompression limits:					
P/A		=	125	psi	min (ACI 18.1
		=	300	psi	max
Target load balances:			0.4		
	//0	=	0.6	~	
% WDL		=	86	pst	
2 Hour Fire Rating:					
Restrained Slabs:			0.75	in	bottom
Unrestrained Slabs:			1.5	in	bottom
			0.75	in	top
Tendon Profile:					
$a_{int} = t_{slab} - 2 \times cover - d_{tendon}$		=	9.50	in	
$0.5 \text{ x} \text{ d}_{\text{tendon}}$		=	6.3750	in	
Pre-stress Force Required to Balance % of S.W.					
$w_b = 0 / w_{DL}$		=	2.588	klf	
Force needed to counteract load in end bay:					
$P = w_b L^2 / 8a_{end}$		=	974.12	k	
Check Precompression Allowance:					
# tendons		=	36.59	use	tendor 36 s
Actual force for banded tendons					
$P_{act} = \#$ tendons x P_{act}	eff	=	958.4	k	

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Adjust En	d Span Balanced Load:						
	Wb	=	2.546	klf			
Determine	e actual Precompression stress						osi min
	$P_{actual} / A = P_{actual} \ge 1000 / A$	=	231.50	psi	>	125	ря нин. ОК
					<	300	psi max. OK
Check In	terior Span Force:						Tana
							Force
	$P = w_b L^2 / 8a_{int}$	=	653.68	k	<	958.4	Required
	$w_b = P_{act} \ge 8 \ge a_{int}/l^2$	=	3.794	klf			
	w_b / w_{DL}	=	0.880		OK		
	Γ						
	Effective prestress force, P _{eff}	=	958.4	k			
Check SI	ab Strassas						
CHECK OI	10 51163563.						
	Dead Load Moments:						
	Dead Load Moments: WDL	=	5.063	klf			
	Dead Load Moments: W _{DL} M ⁻	=	5.063 810.00	klf kft			
	Dead Load Moments: W _{DL} M ⁻ M ⁺ _{ext}	= =	5.063 810.00 648.00	klf kft kft			
	Dead Load Moments: WDL M ⁻ M ⁺ ext M ⁺ int	= = =	5.063 810.00 648.00 202.50	klf kft kft kft			
	Dead Load Moments: WDL M ⁻ M ⁺ _{ext} M ⁺ _{int}	= = =	5.063 810.00 648.00 202.50	klf kft kft kft			
	Dead Load Moments: WDL M ⁻ M ⁺ _{ext} M ⁺ _{int} Live Load Moments:	=	5.063 810.00 648.00 202.50	klf kft kft kft			
	Dead Load Moments: WDL M ⁻ M ⁺ ext M ⁺ int Live Load Moments: WLL ML	=	5.063 810.00 648.00 202.50 2.100 226.00	klf kft kft kft klf			
	Dead Load Moments: WDL M ⁻ M ⁺ ext M ⁺ int Live Load Moments: WLL M ⁻		5.063 810.00 648.00 202.50 2.100 336.00	klf kft kft kft klf			
	Dead Load Moments: WDL M ⁻ M ⁺ ext M ⁺ int Live Load Moments: WLL M ⁻ M ⁺ ext M ⁺ ext	= = = = =	5.063 810.00 648.00 202.50 2.100 336.00 268.80	klf kft kft kft klf kft			
	Dead Load Moments: WDL M ⁺ M ⁺ _{ext} M ⁺ _{int} Live Load Moments: WLL M ⁻ M ⁺ M ⁺ M ⁺ M ⁺		5.063 810.00 648.00 202.50 2.100 336.00 268.80 84.00	klf kft kft kft kft kft kft			
	Dead Load Moments: WDL M ⁺ M ⁺ ext M ⁺ int Live Load Moments: WLL M ⁻ M ⁺ ext M ⁺ ext M ⁺ ext M ⁺ ext M ⁺ ext		5.063 810.00 648.00 202.50 2.100 336.00 268.80 84.00	klf kft kft kft kft kft kft			
	Dead Load Moments: WDL M ⁺ M ⁺ ext M ⁺ int Live Load Moments: WLL M ⁺ M ⁺		5.063 810.00 648.00 202.50 2.100 336.00 268.80 84.00 3.170	klf kft kft kft kft kft kft			
	Dead Load Moments: WDL M' M*ext M*int Live Load Moments: WLL M' M' M*ext M*int Total Balancing Moments: Wb M'		5.063 810.00 648.00 202.50 2.100 336.00 268.80 84.00 3.170 507.15	klf kft kft kft kft kft kft kft			
	Dead Load Moments: WDL M ⁺ M ⁺ ext M ⁺ int Live Load Moments: WLL M ⁻ M ⁺ ext Total Balancing Moments: Wb M ⁺ M ⁺		5.063 810.00 648.00 202.50 2.100 336.00 268.80 84.00 3.170 507.15 405.72	klf kft kft kft kft kft kft kft kft			
	Dead Load Moments: WDL M ⁺ M ⁺ ext M ⁺ int Uive Load Moments: WLL M ⁺ M ⁺ ext M ⁺ ext M ⁺ ext M ⁺ int		5.063 810.00 648.00 202.50 2.100 336.00 268.80 84.00 3.170 507.15 405.72 126.79	klf kft kft kft kft kft kft kft kft kft			

Stage 1: Stresses Immediately after Jacking (DL + PT):

Midpsan Stresses:					
$f_{top}=(-M_{DL}+N_{DL})$	(I _{BAL})/S - P/A				
f_{bot} =(+ M_{DL} - N	(I _{BAL})/S - P/A				
Interior Span:					
f_{top}	=	-346.00	<	1800	ОК
f_{bot}	=	-117.00	<	1800	ОК
End Span:					
${ m f}_{ m top}$	=	-597.89	<	1800	ОК
$\mathbf{f}_{\mathrm{bot}}$	=	134.90	<	164	ОК
Support Stresses:					
$f_{top}=(+M_{DL}N)$	$(I_{BAL})/S$ - P/A				
$f_{bot} = (-M_{DL+}N)$	(I _{BAL})/S - P/A				
$\mathrm{f_{top}}$	=	226.50	>	164	NEED REINF.
$\mathbf{f}_{\mathrm{bot}}$	=	-689.49	<	1800	ОК
Stage2: Stresses at Service Load (DL	+ LL + PT):				
Midpsan Stresses:					
f_{top} =(- M_{DL} - M_{DL} + M_{BAL})/S	- P/A				
f_{bot} =(+ M_{DL} + M_{DL} - M_{BAL})/S Interior Span:	- P/A				
f_{top}	=	-473.03	<	1350	ОК
f_{bot}	=	10.04	<	329	ОК
End Span:					
f_{top}	=	-1004.40	<	1350	ОК
$\mathbf{f}_{\mathrm{bot}}$	=	541.41	>	329	NEED REINF.
Support Stresses:					
$f_{top} = (+M_{DL} + M_{DL} - M_{BAL})/S$	- P/A				
$f_{bot}\text{=}(\text{-}M_{DL}\text{-}M_{DL}\text{+}M_{BAL})/S$	- P/A				
${ m f}_{ m top}$	=	734.63	>	329	NEED REINF.
$\mathbf{f}_{\mathrm{bot}}$	=	-1197.62	<	1350	ОК
Ultimate Strength:					
	$M_1 = Pe =$	379.36	kft		
Ν	$M_{sec} = M_{BAL} - M_1 =$	127.79	kft	@ inte	rior supports

	M_u =1.2 M_{DL} +1.6 M_L	$_{L}$ +1.0M _{SEC}					
		M_{u}	=	1271.57	kft	@ midspan	
		M_{u}	=	1573.49	kft	@ support	
Minimum Bor	ded Reinforcement:						
Positive Momen	nt Region:						
Inte	erior Span:	\mathbf{f}_{t}	=	10.04	psi	< 141	NO REINF. REQ. NEED
Exterior Span:		f_t	=	541.41	psi	> 141	REINF.
	y=	$= f_t/(f_t+f_c)h$	=	4.03	in		
	$N_c = M_{DL+LL}$	/S*.5*y*L1	=	1005.19	кıр s		
	A _{s,m}	in=Nc/.5fy	=	33.51	in ²		
Dis	tribute reinforcement evenl	y across the	widtl	n of the slal	b		
	A _{s,mi}	$A_{s,min}/L_1$	=	1.117	in²/ft		
		Bottom	=	1.2	in²/ft		
Negative Mome	ent Region: erior Supports :						
	$A_{cf} =$	t _{slab} x L x12	=	5520	in ²		
	A _{s,min} =	0.00075A _{cf}	=	4.14	in ²		
	Use 1	l4 #5 Top		4.34	in ²		
Ext	erior Supports :						
	$A_{cf} =$	t _{slab} x L x12	=	4140	in^2		
	A _{s,min} =	0.00075A _{cf}	=	3.105	in^2		
	Use	11 #5 Top		3.41	in^2		
Bar At l	s span minimum of 1/6 cle east 4 bars in each direction	ar span each 1	side	of support			
	Max I	Bar Spacing	=	12	in		
Check if minir	num reinforcement is su	fficient for u	ıltim	ate streng	th:		
At Supports							
Inte orts	erior Supp :						

	$d_{supports} = t_{slab}$ - cover- 0.5 x d_{tendon}	=	10.50	in		
	$A_{ps} = 0.153 \text{ in}^2 x$ (number of tendons)	=	5.51	in^2		
	$f_{ps, supports} = f_{se} + 10,000 + (f_cbd)/(300A_{ps})$	=	195,438	psi		
	$a = (A_s f_y + A_{ps} f_{ps}) / (0.85 f_c b)$	=	0.87	in		
	ϕ Mn = ϕ (Asfy + Apsfps) (d-a/2)	=	1,009	kft	<	1,573 kft
			\mathbf{U}	TL. S	STGTH	REINF. GOVERNS
	$A_{s, reqd}$	=	17.74	in ²		
	Use #7 @ 12" O.C. Bottom at end span		A_s	=	18.00	in ²
At						
Midspan:						
	$d_{supports} = t_{slab}$ - cover- 0.5 x d_{tendon}	=	9.75	in		
	$A_{ps} = 0.153 \text{ in}^2 x \text{ (number of tendons)}$	=	5.51	in^2		
	$f_{ps,supports} = f_{se} + 10,\!000 + (f_cbd)/(300A_{ps})$	=	194,621	psi		
	$a = (A_s f_y + A_{ps} f_{ps}) / (0.85 f_c b)$	=	2.01	in		

$$\label{eq:main_star} \begin{split} \phi \mathrm{Mn} &= \phi (\mathrm{Asfy} + \mathrm{Apsfps}) \; (\mathrm{d}\text{-a}/2) \; = \; 2,021 \quad \mathrm{kft} \; > \; 1,272 \; \mathrm{kft} \\ \mathbf{MIN. \; REINF.\; OK} \end{split}$$

Use #7 @ 6" O.C. Bottom at end spans

Design of North South				
Section Properties:				
	$A = bh = (360 \text{ in})t_{slab}$	=	5520	in ²
	$S = bh^2/6 = (360 \text{ in})(t_{slab})^2/6$	=	10580	in ³
Design Barameters:				
Allerent le stranger Clear				
Allowable stresses: Class	S U (ACI 18.3.3):			
	At time of jacking (ACI 18.4.1):			
	\mathbf{f}_{ci}	=	3,000	psi
	$Compression = 0.60 \text{ f}_{ci}$	=	1,800	psi
	Tension = $3\sqrt{\mathbf{f}_{ci}}$	=	164	psi
	At service loads (ACI 18.4.2(a) and 18.3.3):			
	\mathbf{f}_{c}	=	5,000	psi
	Compression = 0.45 f_{c}	=	2,250	psi

Tension = $6\sqrt{f_c}$	=	424	psi	
Average precompression limits:				
D/A	_	105	psi min (A	CI
P/A	_	125 300	18.12.4)	
Taroet load balances:	_	300	psimax	
1 alget 10 ad 5 alaitees. %	=	0.6		
⁰ / ₀ Wpt	=	86	nsf	
70 WDL		00	P31	
2 Hour Fire Rating:				
Restrained Slabs:		0.75	in bottom	
Unrestrained Slabs:		1.5	in bottom	
		0.75	in top	
Tendon Profile:				
$a_{int} = t_{slab} - 2 \ge cover - d_{tendon}$	=	9.50	in	
$a_{end} = 0.5 \text{ x} (1.5 \text{ x} t_{slab}\text{-cover-}0.5 \text{ x} d_{tendon})\text{-cover-}0.5 \text{ x} d_{tendon}$	= 6.	3750	in	
Pre-stress Force Required to Balance % of S.W.				
$w_b = 0/W_{DL}$	= :	3.450	klf	
Force needed to counteract load in end bay:				
$P = w_b L^2 / 8a_{end}$	= 7.	30.59	k	
tendors = 27.4	4 1150	2	28 tondy	
# tendons $-2/.4$	4 use		.o tenuc	5115
Actual force for banded tendons				
$P_{crr} = \#$ tendons x $P_{crr} = 74^{\circ}$	54 k			
	., K			
Adjust End Span Balanced Load:				
$w_{\rm b} = 3.5$	20 klf			
Determine actual Precompression stress				
				psi min.
$P_{actual} / A = P_{actual} \ge 1000 / A = 135.$	04 psi	>	> 125	OK
				max.
		<	< 300	OK
Check Interior Span Force:				
$P = w_b L^2 / 8a_{int} = 490.$	26 k	<	745.4	ł
Less	Force Re	quired	1	

	$w_b = P_{act} x$	$8 \ge a_{int}/l^2$	=	5.246	klf	
		w _b / w _{DL}	=	0.912		ОК
						_
	Effective exections f		_	745 4	1-	
Chaols Slab S	Effective prestress fo	orce, P _{eff}	_	/45.4	K	
LINECK STAD S	Dead Load Moments:					
	Dead Load Moments.					
		WDL	=	6.750	klf Þf	
		M-	=	607.50	t	
		M^+_{ex}			kf	
		t	=	486.00	t	
					kf	
		M^+_{int}	=	151.88	t	
	Live Load Moments:					
		WLL	=	2.800	klf	
					kf	
		M^{-}	=	252.00	t	
		$\mathrm{M}^{+}\mathrm{ex}$			kf	
		t	=	201.60	t 1-6	
		M+:	=	63.00	KI t	
		ivi nu		05.00	ι	
	Total Balancing Moments:					
		Wb	=	4.383	klf	
				a a 4 : -	kf	
		M ⁻	=	394.45	t 1-6	
		IVI ' ex	=	315 56	KI t	
		t	_	515.50	kf	
		$M^{+}_{int} \\$	=	98.61	t	
tage 1: Stres	ses Immediately after Jacking (DL + PT):			
Aidpsan Stres	ses:					

End Span:

$ \begin{aligned} & \int_{\text{floor}} e = 328.3 & < 1800 \text{ OK} \\ & \int_{\text{floor}} e = 328.3 & < 164 \text{ OK} \end{aligned} \end{aligned}$ Support Stresses: $ \begin{aligned} & \int_{\text{floor}} e = (-M_{\text{DL}}, M_{\text{BAU}})/S \cdot P/A \\ & \int_{\text{floor}} e = 106.61 & < 164 \text{ OK} \end{aligned} \end{aligned}$ Support Stresses: $ \begin{aligned} & \int_{\text{floor}} e = 376.68 & < 1800 \text{ OK} \end{aligned}$ Stage2: Stresses at Service Load (DL + LL + T): Midpsan Stresses: $ \begin{aligned} & \int_{\text{floor}} e = 376.68 & < 1800 \text{ OK} \end{aligned}$ Stage2: Stresses at Service Load (DL + LL + T): Midpsan Stresses: $ \begin{aligned} & \int_{\text{floor}} e = -266.91 & < 1350 \text{ OK} \end{aligned}$ Support Stresses: $ \begin{aligned} & \int_{\text{floor}} e = -3.17 & < 329 \text{ OK} \end{aligned}$ Support Stresses: $ \begin{aligned} & \int_{\text{floor}} e = -357.01 & < 1350 \text{ OK} \end{aligned}$ Support Stresses: $ \begin{aligned} & \int_{\text{floor}} e = -357.01 & < 1350 \text{ OK} \end{aligned}$ Support Stresses: $ \begin{aligned} & \int_{\text{floor}} e = -392.43 & > 329 \text{ OK} \end{aligned}$ Support Stresses: $ \begin{aligned} & \int_{\text{floor}} e = -392.43 & > 329 \text{ OK} \end{aligned}$ Support Stresses: $ \begin{aligned} & \int_{\text{floor}} e = -392.43 & > 329 \text{ OK} \end{aligned}$ Support Stresses: $ \begin{aligned} & \int_{\text{floor}} e = -392.43 & > 329 \text{ OK} \end{aligned}$ Support Stresses: $ \begin{aligned} & \int_{\text{floor}} e = -392.43 & > 329 \text{ OK} \end{aligned}$ Support Stresses: $ \begin{aligned} & \int_{\text{floor}} e = -392.43 & > 329 \text{ OK} \end{aligned}$ Support Stresses: $ \begin{aligned} & \int_{\text{floor}} e = -392.43 & > 329 \text{ OK} \end{aligned}$ Support Stresses: $ \begin{aligned} & \int_{\text{floor}} e = -392.43 & > 329 \text{ OK} \end{aligned}$ Support Stresses: $ \begin{aligned} & \int_{\text{floor}} e = -392.43 & > 329 \text{ OK} \end{aligned}$ Support Stresses: $ \begin{aligned} & \int_{\text{floor}} e = -392.43 & > 329 \text{ OK} \end{aligned}$ Support Stresses: $ \begin{aligned} & \int_{\text{floor}} e = -392.43 & > 329 \text{ OK} \end{aligned}$ Support Stresses: $ \begin{aligned} & \int_{\text{floor}} e = -392.43 & > 329 \text{ OK} \end{aligned}$ Support Stresses: $ \begin{aligned} & \int_{\text{floor}} e = -392.545 \text{ Str} (@ \text{ interior supports} \end{aligned}$ $ \end{aligned}$ $ \end{aligned}$ $ \begin{aligned} & \int_{\text{floor}} e = 955.45 \text{ Str} (@ \text{ midspan} \\ & \int_{\text{Maintended Reinforcement:} \end{aligned}$ Positive Moment Region: $ \end{aligned}$ $ \end{aligned}$ $ \end{aligned}$ $ \end{aligned}$											
$\int_{bot} e^{-1} = 58.28 < 164 \text{ OK}$ Support Stresses: $\int_{bot} (+M_{DL}, M_{BAL})/S \cdot P/A$ $\int_{bot} e^{-1} (-M_{DL}, M_{BAL})/S \cdot P/A$ $\int_{bot} e^{-1} = 376.68 < 1800 \text{ OK}$ Stage2: Stresses at Service Load (DL + LL + PT): Midpsan Stresses: $\int_{bot} e^{-1} (-M_{DL}, -M_{DL}, +M_{BAL})/S \cdot P/A$ $\int_{bot} e^{-1} e^{-266.91} < 1350 \text{ OK}$ $\int_{bot} e^{-1} e^{-3.17} < 329 \text{ OK}$ End Span: $\int_{bot} e^{-1} e^{-3.17} < 329 \text{ OK}$ Support Stresses: $\int_{cop} e^{-1} (-M_{DL}, +M_{DL}, -M_{BAL})/S \cdot P/A$ $\int_{bot} e^{-1} e^{-3.17} < 329 \text{ OK}$ End Span: $\int_{bot} e^{-1} e^{-3.17} < 329 \text{ OK}$ Support Stresses: $\int_{cop} e^{-1} (-M_{DL}, +M_{DL}, -M_{BAL})/S - P/A$ $\int_{bot} e^{-1} e^{-3.17} < 329 \text{ OK}$ Support Stresses: $\int_{cop} e^{-1} (-M_{DL}, +M_{DL}, -M_{BAL})/S - P/A$ $\int_{bot} e^{-1} e^{-3.17} < 329 \text{ OK}$ Support Stresses: $\int_{cop} e^{-1} (-M_{DL}, +M_{DL}, -M_{BAL})/S - P/A$ $\int_{bot} e^{-1} (-1)^{-3.15} $				ftop	=	328	- 8.36	<	1800	ОК	
Support Stresses: $f_{op}=(+M_{DL},M_{BAL})/S - P/A$ $f_{op} = 106.61 < 164 \text{ OK}$ $f_{op} = 376.68 < 1800 \text{ OK}$ Stage2: Stresses at Service Load (DL + LL + PT): Midpsan Stresses: $f_{op}=(+M_{DL} - M_{DL} + M_{BAL})/S - P/A$ $f_{oot} = -3.17 < 329 \text{ OK}$ Support Stresses: $f_{op}=(+M_{DL} + M_{DL}, M_{BAL})/S - P/A$ $f_{oot} = -3.17 < 329 \text{ OK}$ Support Stresses: $f_{op}=(+M_{DL} + M_{DL}, M_{BAL})/S - P/A$ $f_{oot} = -3.17 < 329 \text{ OK}$ Support Stresses: $f_{op}=(+M_{DL} + M_{DL}, M_{BAL})/S - P/A$ $f_{oot} = -266.91 < 1350 \text{ OK}$ $f_{oot} = -3.17 < 329 \text{ OK}$ Support Stresses: $f_{op}=(+M_{DL} + M_{DL}, M_{BAL})/S - P/A$ $f_{oot} = -266.94 < 329 \text{ OK}$ Support Stresses: $f_{op}=(+M_{DL} + M_{DL}, M_{BAL})/S - P/A$ $f_{oot} = -662.51 < 1350 \text{ OK}$ $The the the the the the the the the the t$				fun	=	58	8 28	<	164	ОК	
Support Stresses: $\begin{aligned} \int_{c_{00}} = (+M_{DL}, M_{BAL})/S - P/A \\ \int_{b_{00}} = (-M_{DL}, M_{BAL})/S - P/A \\ \int_{b_{00}} = 376.68 < 160 OK \\ \hline \\ \hline \\ Stresses: & 1800 OK \\ \hline \\ Stresses: & 1800 OK \\ \hline \\ Stresses: & 1800 OK \\ \hline \\ \\ \hline \\ \\ \\ \\ \\ \\ \\ \\ \\ \\ \\ \\ \\ \\ \\$				Tbot		50	0.20		104	OK	
$\begin{array}{cccccccccccccccccccccccccccccccccccc$	Support Stresses:										
$f_{bot} = (M_{DL} + M_{BAL})/S - P/A$ $f_{bot} = 106.61 < 164 \text{ OK}$ $f_{bot} = 376.68 < 1800 \text{ OK}$ Stage2: Stresses at Service Load (DL + LL + PT): Midpsan Stresses: $f_{wp}=(M_{DL} + M_{BAL})/S - P/A$ $f_{wp}=(M_{DL} + M_{BAL})/S - P/A$ Interior Span: $f_{wp} = -266.91 < 1350 \text{ OK}$ $f_{bot} = -3.17 < 329 \text{ OK}$ End Span: $f_{wp} = -557.01 < 1350 \text{ OK}$ $f_{bot} = 286.94 < 329 \text{ OK}$ Support Stresses: $f_{wp}=(+M_{DL} + M_{DL} - M_{BAL})/S - P/A$ $f_{bot} = -662.51 < 1350 \text{ OK}$ $f_{bot} = 1280.94 + (M_{DL} - M_{DL} - M_{DL$	11	$f_{\rm tra} = (+M_{\rm DI} M_{\rm PAI})/S$	- P/A								
$f_{top} = 106.61 < 164 \text{ OK}$ $f_{top} = 376.68 < 1800 \text{ OK}$ Stage2: Stresses at Service Load (DL + LL + PT): Midpsan Stresses: $f_{cop}=(-M_{DL} - M_{DL} + M_{BAI})/S \cdot P/A$ $f_{bot}=(+M_{DL} + M_{DL} - M_{BAI})/S \cdot P/A$ Interior Span: $f_{top} = -266.91 < 1350 \text{ OK}$ $f_{bot} = -3.17 < 329 \text{ OK}$ End Span: $f_{top} = -557.01 < 1350 \text{ OK}$ $f_{bot} = 286.94 < 329 \text{ OK}$ Support Stresses: $f_{top}=(+M_{DL} + M_{DL} - M_{BAI})/S \cdot P/A$ $f_{bot} = -30.17 < 329 \text{ OK}$ End Span: $f_{top} = -557.01 < 1350 \text{ OK}$ $f_{bot} = -286.94 < 329 \text{ OK}$ Support Stresses: $f_{top}=(+M_{DL} + M_{DL} - M_{BAI})/S \cdot P/A$ $f_{bot} = -662.51 < 1350 \text{ OK}$ EITINI $f_{top} = -662.51 < 1350 \text{ OK}$ $M_u = -955.45 \text{ kft} @ \text{ midspan}$ $M_u = 1181.89 \text{ kft} @ \text{ support}$ Minimum Bonded Reinforcement: Positive Moment Region: Interior Span: $f_t = -3.17 \text{ psi} < 141 \text{ NEQ.}$		$f_{\rm top} = (M_{\rm DL} M_{\rm DAL})/S$	D/Λ								
$\begin{split} F_{top} &= 100.01 < 104 \text{ UK} \\ F_{tor} &= 376.68 < 1800 \text{ OK} \\ \end{split}{0.5} \\ \begin{array}{r} \text{Stage2: Stresses at Service Load (DL + LL + PT):} \\ \hline \text{Midpsan Stresses:} \\ f_{top} = (-M_{DL} - M_{DL} + M_{BAI})/S - P/A \\ f_{bos} = (-M_{DL} + M_{DL} - M_{BAI})/S - P/A \\ \hline \text{Interior Span:} \\ f_{top} &= -266.91 < 1350 \text{ OK} \\ \hline f_{bos} &= -3.17 < 329 \text{ OK} \\ \hline \text{End Span:} \\ f_{top} &= -3.17 < 329 \text{ OK} \\ \hline \text{End Span:} \\ f_{top} &= -286.94 < 329 \text{ OK} \\ \hline \text{Support Stresses:} \\ f_{top} = (-M_{DL} + M_{DL} - M_{BAI})/S - P/A \\ f_{box} &= 286.94 < 329 \text{ OK} \\ \hline \text{Support Stresses:} \\ f_{top} = (-M_{DL} - M_{DL} + M_{BAI})/S - P/A \\ f_{box} &= -662.51 < 329 \text{ OK} \\ \hline \text{Utimate Strength:} \\ \hline M_{1} = \text{Pe} &= 295.06 \text{ kft} \\ M_{sec} = M_{BAL} - M_{1} &= 99.39 \text{ kft} (@ interior supports \\ M_{u} = 1181.89 \text{ kft} (@ midspan \\ M_{u} &= 1181.89 \text{ kft} (@ support \\ \hline \text{Minimum Bonded Reinforcement:} \\ \hline \text{Positive Moment Region:} \\ \hline \text{Interior Span:} & f_{i} &= -3.17 \text{ psi} < 141 \text{ REQ.} \\ \hline \end{array}$		$1_{\text{pot}} - (-1_{\text{MDL}} + 1_{\text{MBAL}}) / 3 -$	1/11	c		10	<i>((</i>)		1.64	OV	
$\begin{array}{cccccccccccccccccccccccccccccccccccc$				I _{top}	_	100	0.01 -	<	104	OK	
Stage2: Stresses at Service Load (DL + LL + PT): Midpsan Stresses: $f_{top}=(-M_{DL} - M_{DL} + M_{BAL})/S - P/A$ $f_{bor}=(+M_{DL} + M_{DL} - M_{BAL})/S - P/A$ Interior Span: $f_{top} = -266.91 < 1350 \text{ OK}$ $f_{bor} = -3.17 < 329 \text{ OK}$ End Span: $f_{top} = -557.01 < 1350 \text{ OK}$ $f_{bor} = 286.94 < 329 \text{ OK}$ Support Stresses: $f_{top}=(+M_{DL} + M_{DL} - M_{BAL})/S - P/A$ $f_{bor}=(-M_{DL} - M_{DL} + M_{BAL})/S - P/A$ $f_{top}= -392.43 > 329 \text{ OK}$ PLUTIMATE Strength: $M_{1}=Pe = 392.43 > 329 \text{ OK}$ $M_{1}=12M_{DL} + M_{DL} - M_{BAL}/S - P/A$ $f_{top}= -662.51 < 329 \text{ OK}$ $M_{1}=12M_{DL} + 1.6M_{LL} + 1.0M_{SEC}$ $M_{u}=1.2M_{DL} + 1.6M_{LL} + 1.0M_{SEC}$ $M_{u} = 955.45 \text{ kft } @ \text{ midspan}$ $M_{u} = 1181.89 \text{ kft } @ \text{ support}$ $M_{1}=12M_{DL} + 1.6M_{LL} + 1.0M_{SEC}$ $M_{u} = 1181.89 \text{ kft } @ \text{ support}$ $M_{1}=12M_{DL} + 1.6M_{LL} + 1.0M_{SEC}$ $M_{u} = 1181.89 \text{ kft } @ \text{ support}$ $M_{1}=12M_{DL} + 1.6M_{LL} + 1.0M_{SEC}$ $M_{1} = 955.45 \text{ kft } @ \text{ midspan}$ $M_{u} = 1181.89 \text{ kft } @ \text{ support}$ $M_{1}=12M_{DL} + 1.6M_{LL} + 1.0M_{SEC}$ $M_{1} = 955.45 \text{ kft } @ \text{ midspan}$ $M_{1} = 1181.89 \text{ kft } @ \text{ support}$ $M_{1}=12M_{DL} + 1.6M_{LL} + 1.0M_{SEC}$ $M_{1} = 955.45 \text{ kft } @ \text{ midspan}$ $M_{1} = 1181.89 \text{ kft } @ \text{ support}$ $M_{1} = -3.17 \text{ psi } < 141 \text{ REV}$				$f_{\rm bot}$	=	370	6.68	<	1800	ОК	
Midpsan Stresses: $f_{top}=(-M_{DL} - M_{DL} + M_{BAL})/S - P/A$ $f_{bot}=(+M_{DL} + M_{DL} - M_{BAL})/S - P/A$ Interior Span: $f_{top} = -266.91 < 1350$ OK $f_{bot} = -3.17 < 329$ OK End Span: $f_{top} = -557.01 < 1350$ OK $f_{bot} = 286.94 < 329$ OK Support Stresses: $f_{top}=(+M_{DL} + M_{DL} - M_{BAL})/S - P/A$ $f_{bot}= -662.51 < 1350$ OK EINI $f_{top} = -392.43 > 329$ OK EINI $f_{top} = -662.51 < 1350$ OK $IUtimate Strength:$ $M_{1}=Pe = 295.06$ kft $M_{u}=1.2M_{DL} + 1.6M_{LL} + 1.0M_{SEC}$ $M_{u}=1.2M_{DL} + 1.6M_{LL} + 1.0M_{SEC}$ $M_{u} = 1181.89$ kft @ interior support $M_{u} = 1181.89$ kft @ midspan $M_{u} = 1181.89$ kft @ support $Iuterior Span:$ $f_{t} = -3.17$ psi < 141 NEELINI	Stage2: Stresses at	Service Load (DL +	LL +	PT):							
$\begin{aligned} & f_{top} = (-M_{DL} - M_{DL} + M_{BAL})/S - P/A \\ & f_{bot} = (+M_{DL} + M_{DL} - M_{BAL})/S - P/A \\ & Interior Span: \\ & f_{top} = -266.91 < 1350 \text{ OK} \\ & f_{bot} = -3.17 < 329 \text{ OK} \end{aligned}$ $\begin{aligned} & \text{End Span:} \\ & f_{top} = -357.01 < 1350 \text{ OK} \\ & f_{bot} = 286.94 < 329 \text{ OK} \end{aligned}$ Support Stresses: $f_{top} = (+M_{DL} + M_{DL} - M_{BAL})/S - P/A \\ & f_{bot} = -286.94 < 329 \text{ OK} \end{aligned}$ Support Stresses: $f_{top} = (+M_{DL} + M_{DL} - M_{BAL})/S - P/A \\ & f_{bot} = -662.51 < 1350 \text{ OK} \end{aligned}$ $\begin{aligned} \text{Millimate Strength:} \\ & \text{Illimate Strength:} \\ & M_{1} = Pe = 392.43 \text{ Points of the strength:} \\ & \text{Illimate Strength:} \\ & M_{1} = Pe = 295.06 \text{ kft} \\ & M_{sec} = M_{BAL} - M_{1} = 99.39 \text{ kft @ interior supports } \\ & M_{u} = 1.2M_{DL} + 1.6M_{LL} + 1.0M_{SEC} \\ & M_{u} = 1181.89 \text{ kft @ midspan} \\ & M_{u} = 1181.89 \text{ kft @ support} \end{aligned}$ $\begin{aligned} \text{Minimum Bonded Reinforcement:} \\ & Positive Moment Region: \\ & Interior Span: & f_{t} = -3.17 \text{ psi } < 141 \text{ NEED} \end{aligned}$	Midpsan Stresses:										
$ \begin{split} & f_{bot} = (+M_{DL} + M_{DL} - M_{BAL})/S - P/A \\ & Interior Span: \\ & f_{top} = -266.91 < 1350 \\ & f_{bot} = -3.17 < 329 \\ \hline OK \\ \hline C \\ End Span: \\ & f_{top} = -557.01 < 1350 \\ & f_{bot} = 286.94 < 329 \\ \hline OK \\ \hline Support Stresses: \\ & f_{top} = (+M_{DL} + M_{DL} - M_{BAL})/S - P/A \\ & f_{bot} = -286.94 < 329 \\ \hline OK \\ \hline Support Stresses: \\ & f_{top} = (+M_{DL} + M_{DL} - M_{BAL})/S - P/A \\ & f_{bot} = -662.51 < 329 \\ \hline OK \\ \hline Ultimate Strength: \\ & M_{1} = Pc = 392.43 \\ & f_{bot} = -662.51 < 350 \\ \hline OK \\ \hline Ultimate Strength: \\ & M_{1} = Pc = -295.06 \\ & Kft \\ & M_{ucc} = M_{BAL} - M_{1} = 99.39 \\ & Kft @ interior supports \\ & M_{u} = 1.2M_{DL} + 1.6M_{LL} + 1.0M_{SEC} \\ & M_{u} = 1181.89 \\ & Kft @ midspan \\ & M_{u} = 1181.89 \\ & Kft @ support \\ \hline Minimum Bonded Reinforcement: \\ \hline Positive Moment Region: \\ & f_{t} = -3.17 \\ \hline Strength = -3.17 \\ & psi < 141 \\ \hline OK \\ \hline \end{tabular}$	$f_{top} = (-1)^{-1}$	M_{DL} - M_{DL} + M_{BAL})/S -	P/A								
Interior Span: $f_{top} = -266.91 < 1350$ OK $f_{bot} = -3.17 < 329$ OK End Span: $f_{top} = -3.17 < 329$ OK End Span: $f_{top} = -557.01 < 1350$ OK $f_{top} = -286.94 < 329$ OK Support Stresses: $f_{top} = -286.94 < 329$ OK Support Stresses: $f_{top} = -392.43 > 329$ NEED $f_{top} = -662.51 < 329$ OK Utimate Strength: $M_1 = Pe = -662.51 < 329$ OK $M_1 = Pe = -662.51 < 329$ OK $M_u = -393.93$ kft @ interior supports MEED $M_u = -395.45$ kft @ midspan Mu = 1.2M_{DL}+1.6M_{LL}+1.0M_{SEC} $M_u = -395.45$ kft @ support Mu = 1.2M_{DL}+1.6M_{LL}+1.0M_{SEC} $M_u = -3955.45$ kft @ support KEUN Distive Moment Region: $f_t = -3.317$ psi < 141	$f_{bot} = (+)$	$M_{DL} + M_{DL} - M_{BAL})/S$ -	P/A								
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$		Interior Span:									
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$			f _{top}	=	-266.9	91	<		1	350	ОК
End Span: $f_{top} = -557.01 < 1350 \text{ OK}$ $f_{bot} = 286.94 < 329 \text{ OK}$ Support Stresses: $f_{top}=(+M_{DL} + M_{DL} - M_{BAL})/S - P/A$ $f_{bot}=(-M_{DL} - M_{DL} + M_{BAL})/S - P/A$ $f_{top} = 392.43 > 329 \text{ REINI}$ $f_{bot} = -662.51 < 1350 \text{ OK}$ Ultimate Strength: $M_1 = Pe = 4295.06 \text{ kft}$ $M_{sec} = M_{BAL} - M_1 = 99.39 \text{ kft} @ \text{ interior supports}$ $M_u = 1181.89 \text{ kft} @ \text{ support}$ Minimum Bonded Reinforcement: Positive Moment Region: $f_t = -3.17 \text{ psi} < 141 \text{ REQ}.$			fbot	=	-3.3	17	<			329	ОК
End Span: f_{top} = -557.01 <			501		-						_
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$		End Span:									
$f_{bot} = 286.94 < 329 \text{ OK}$ Support Stresses: $f_{top} = (+M_{DL} + M_{DL} - M_{BAL})/S - P/A$ $f_{bot} = (-M_{DL} - M_{DL} + M_{BAL})/S - P/A$ $f_{top} = 392.43 > 329 \text{ NEED}$ $f_{top} = -662.51 < 1350 \text{ OK}$ Ultimate Strength: $M_1 = Pe = -662.51 < 1350 \text{ OK}$ Ultimate Strength: $M_1 = Pe = 295.06 \text{ kft}$ $M_{sec} = M_{BAL} - M_1 = 99.39 \text{ kft} @ \text{ interior supports}$ $M_u = 12M_{DL} + 1.6M_{LL} + 1.0M_{SEC}$ $M_u = 1181.89 \text{ kft} @ \text{ support}$ Minimum Bonded Reinforcement: Positive Moment Region: $f_t = -3.17 \text{ psi} < 141 \text{ NEEL}$			f _{top}	=	-557.0	01	<		1	350	ОК
Support Stresses: $f_{top}=(+M_{DL} + M_{DL} - M_{BAL})/S - P/A$ $f_{bot}=(-M_{DL} - M_{DL} + M_{BAL})/S - P/A$ $f_{top} = 392.43 > 329$ REINI $f_{bot} = -662.51 < 1350$ OK Ultimate Strength: $M_1 = Pe = 295.06 \text{ kft}$ $M_{sec} = M_{BAL} - M_1 = 99.39 \text{ kft } @ \text{ interior supports}$ $M_u = 955.45 \text{ kft } @ \text{ midspan}$ $M_u = 1181.89 \text{ kft } @ \text{ support}$ Minimum Bonded Reinforcement: Positive Moment Region: Interior Span: $f_t = -3.17 \text{ psi} < 141$ NEED			fbot	=	286.9	94	<			329	ОК
Support Stresses: $f_{top}=(+M_{DL} + M_{DL} - M_{BAL})/S - P/A$ $f_{bot}=(-M_{DL} - M_{DL} + M_{BAL})/S - P/A$ $f_{top} = 392.43 > 329$ $RED f_{bot} = -662.51 < 1350 OK Ultimate Strength:M_{1}=Pe = 295.06 \text{ kft} M_{sec}=M_{BAL}-M_{1} = 99.39 \text{ kft} @ \text{ interior supports} M_{u}=1.2M_{DL}+1.6M_{LL}+1.0M_{SEC} M_{u} = 955.45 \text{ kft} @ \text{ midspan} M_{u} = 1181.89 \text{ kft} @ \text{ support} Minimum Bonded Reinforcement: Positive Moment Region:f_{t} = -3.17 \text{ psi} < 141 \text{ REQ}.$			-001		_000.					02/	011
$f_{top}=(+M_{DL}+M_{DL}-M_{BAL})/S - P/A$ $f_{bot}=(-M_{DL}-M_{DL}+M_{BAL})/S - P/A$ $f_{top} = 392.43 > 329$ REINI $f_{bot} = -662.51 < 1350$ OK Ultimate Strength: $M_{1}=Pe = 295.06 \text{ kft}$ $M_{sec}=M_{BAL}-M_{1} = 99.39 \text{ kft} @ \text{ interior supports}$ $M_{u}=1.2M_{DL}+1.6M_{LL}+1.0M_{SEC}$ $M_{u} = 955.45 \text{ kft} @ \text{ midspan}$ $M_{u} = 1181.89 \text{ kft} @ \text{ support}$ Minimum Bonded Reinforcement: Positive Moment Region: $f_{t} = -3.17 \text{ psi} < 141 \text{ REQ}.$	Support Stresses:										
$f_{bot} = (-M_{DL} - M_{DL} + M_{BAL})/S - P/A$ $f_{top} = 392.43 > 329$ $REINI$ $f_{bot} = -662.51 < 1350$ OK Ultimate Strength: $M_1 = Pe = 295.06 \text{ kft}$ $M_{sec} = M_{BAL} - M_1 = 99.39 \text{ kft @ interior supports}$ $M_u = 1181.89 \text{ kft @ midspan}$ $M_u = 1181.89 \text{ kft @ support}$ Minimum Bonded Reinforcement: Positive Moment Region: $f_t = -3.17 \text{ psi} < 141 \text{ REQ}.$	$f_{top} = (+)$	$M_{DL} + M_{DL} - M_{BAL})/S$ -	P/A								
$f_{top} = 392.43 > 329$ $REINI$ $f_{top} = -662.51 < 1350$ $M_{t} = -295.06 \text{ kft}$ $M_{t} = -295.06 \text{ kft}$ $M_{u} = -99.39 \text{ kft} @ \text{ interior supports}$ $M_{u} = 1.2M_{DL} + 1.6M_{LL} + 1.0M_{SEC}$ $M_{u} = -955.45 \text{ kft} @ \text{ midspan}$ $M_{u} = -1181.89 \text{ kft} @ \text{ support}$ $M_{t} = -3.17 \text{ psi} < 141 \text{ REQ}.$	fbot=(-	$M_{DI} - M_{DI} + M_{RAI})/S -$	P/A								
$\begin{array}{rcl} f_{top} &=& 392.43 \\ f_{bot} &=& -662.51 \\ \end{array} > & 1350 \\ \hline \mbox{OK} \end{array}$	-Dot (1 / 11								NEED
f_{bot} = -662.51 <			f_{top}	=	392.4	43	>			329	REINF.
Ultimate Strength: $M_1=Pe = 295.06 \text{ kft}$ $M_1=Pe = 295.06 \text{ kft}$ $M_{sec}=M_{BAL}-M_1 = 99.39 \text{ kft}$ @ interior supports $M_u=1.2M_{DL}+1.6M_{LL}+1.0M_{SEC}$ $M_u = 955.45 \text{ kft}$ @ midspan $M_u = 1181.89 \text{ kft}$ @ supportNO REINUNO REINUInterior Span: $f_t = -3.17 \text{ psi} < 141 \text{ REQ.}$			$f_{bot} \\$	=	-662.	51	<		1	350	ОК
$\begin{split} M_1 = Pe &= 295.06 kft \\ M_{sec} = M_{BAL} - M_1 &= 99.39 kft @ \text{ interior supports} \\ M_u = 1.2M_{DL} + 1.6M_{LL} + 1.0M_{SEC} \\ M_u &= 955.45 kft @ \text{ midspan} \\ M_u &= 1181.89 kft @ \text{ support} \\ \end{split}$	Ultimate Strength:										
$\begin{split} M_{sec}=M_{BAL}-M_{1} &= 99.39 & \text{kft } @ \text{ interior supports} \\ M_{u}=1.2M_{DL}+1.6M_{LL}+1.0M_{SEC} \\ M_{u} &= 955.45 & \text{kft } @ \text{ midspan} \\ M_{u} &= 1181.89 & \text{kft } @ \text{ support} \end{split}$		Ν	M₁=Pe	=	295	.06	kft				
$\begin{array}{rrrrrrrrrrrrrrrrrrrrrrrrrrrrrrrrrrrr$		$M_{sec}=M_{H}$	_{BAL} -M ₁	=	99	.39	kft	@ interior su	apports	5	
$\begin{array}{rrrrrrrrrrrrrrrrrrrrrrrrrrrrrrrrrrrr$	M_{u}	$_{u}$ =1.2M _{DL} +1.6M _{LL} +1.6	OM _{SEC}								
$M_{u} = 1181.89 \text{ kft } @ \text{ support}$ $Minimum Bonded Reinforcement:$ Positive Moment Region: Interior Span: $f_{t} = -3.17 \text{ psi} < 141 \text{ REQ.}$			M_{u}	=	955	.45	kft	(a) midspan			
Minimum Bonded Reinforcement:Positive Moment Region:Interior Span: $f_t = -3.17$ psi $<$ 141REQ.			M_{μ}	=	1181	.89	kft	a support			
Positive Moment Region:NO REINIInterior Span: $f_t = -3.17$ psi< 141	Minimum Bonded	Reinforcement:	u					0 11			
Interior Span: $f_t = -3.17 \text{ psi} < 141 \text{ REQ.}$	Positive Moment Re	egion:									
$1_t3.1/p_{s1} > 141$ REQ.	Intonica Span		f -	_	2 -	17	.		1 / 1	NO DE	REINF.
NEED	merior span:		I _t -	_	-3.	1/	psi		141	NE	ر. ED
Exterior Span: $f_t = 286.94 \text{ psi} > 141 \text{ REINF.}$	Exterior Span:		f _t =	=	286.9	94	psi	>	141	RE	INF.
$y=f_t/(f_t+f_c)h = 3.91$ in		$y=f_t/(f_t+$	f _c)h =	=	3.9	91	in				

$N_{c} = M_{DL+LL} / S^{*}.5^{*}y^{*}L_{1}$	=	731.83	kips
$A_{s,min}$ =Nc/.5fy	=	24.39	in^2

Distribute reinforcement evenly across the width of the slab

$$A_{s,min} = A_{s,min}/L_1 = 0.610 \text{ in}^2/\text{ft}$$

Use #7 @ 10" O.C. Bottom = 0.72 in^2/ft

Negative Moment Region:

Interior Supports :

	$A_{cf} = t_{slab} \ge L \ge 12$	=	5520	in ²
	$A_{s,min}$ =0.00075 A_{cf}	=	4.14	in^2
Use 14 #5 Top			4.34	in ²

Exterior Supports :

	$A_{cf} = t_{slab} \ge L \ge 12$	=	5520	in^2
	$A_{s,min}$ =0.00075 A_{cf}	=	4.140	in^2
Use 14 #5 Top			4.34	in ²

Bars span minimum of 1/6 clear span each side of support At least 4 bars in each direction

Max Bar Spacing	=	12	in			
Check if minimum reinforcement is sufficient for ultimative	ate s	trength	:			
At Supports:						
$d_{supports} = t_{slab}$ - cover- 0.5 x d_{tendon}	=	10.50	in			
$A_{ps} = 0.153 \text{ in}^2 \text{ x} \text{ (number of tendons)}$	=	4.28	in ²			
		203,6				
$f_{ps, supports} = f_{se} + 10,000 + (f_cbd)/(300A_{ps})$	=	08	psi			
$a = (A_s f_y + A_{ps} f_{ps}) / (0.85 f_c b)$	=	0.56	in			
$\phi M p = \phi (Asfy + Apsfps) (d a/2)$	_	868	lzft	<	1 1 8 2	ŀ-f+
ψ	_	000	KIU		1,102	КIL
	UΊ	TL. STG	TH F	REINF.	GOVEF	NS
A _{s, reqd}	=	11.43	in ²			
Use #7 @ 12" O.C. Bottom at end span		As	=	12.00) in ²	
		5				

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At	Mids	pan:
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At Midspan:						
	$d_{supports} = t_{slab}$ - cover- 0.5 x d_{tendon} = $A_{ps} = 0.153 \text{ in}^2 \text{ x}$ (number of		9.75 in			
	tendons) =		4.28 in ²			
	$f_{ps, supports} = f_{se} + 10,000 + (f_cbd)/(300A_{ps}) =$		202,2 07 psi			
	$a = (A_s f_y + A_{ps} f_{ps}) / (0.85 f_c b) =$		1.14 in			
	ϕ Mn = ϕ (Asfy + Apsfps) (d-a/2) =		1,604 kft	>	955	5 kft
			MIN.	REIN	F. OK	
	Use #7 @ 10" O.C. Bottom at end span	ns				
Punching She	ear:					
Framing Dead	$Load = self weight = t_{slab}(150pcf)$	=	143.75	psf		
Superimposed	Dead Load	=	25	psf		
Live Load		=	/0	pst		
	$w_u = 1.2DL + 1.6LL$	=	314.5	psf		
	Area=A	=	1193.75	ft²		
	$V_u = w_u A$	=	375.43	k		
	d _{average}	=	10.125	in		
	b _o	=	160.5	in		
Vc=mi	n of: 4 v f _c b _o d	=	459.64	k		
	$(2+4/\beta)\sqrt{f_c b_o d}$	=	689.46	k		
	$(\alpha d/b_o+2)\sqrt{f_cb_od}$	=	519.78	k		
	Vc	=	459.64	k		
	177	_	244 72	1	_	
	ψv_c	_	344.73	К		3/3.43 К
Size Drop Par	nel:					
	V_u	=	$\phi 4(\mathbf{f}_{c})^{1/2}\mathbf{b}_{o}\mathbf{d}$			
	375.43	=	0.75x4x √ f [•] _c (4(30+d))d	
	d	=	10.84	in		
	ϕV_c	=	375.65	k	> 3	875.43 k OK
		Need	11.84	in	thick d	lrop panel
		use	12	in		

Beam Shear:

At Panel:

Drop Panel

		Area=A	=	600	ft²		
		$V_u = w_u A$	=	188.7	k		
		b _w d ∳V _c =0.75x2 √ f _c b _w d	=	168 11 196.01	in in k	assu drop >	med 14 ft 9 panel 188.70 k OK
	At Slab:	A rec= A	_	300	£+2		
		V = W A	_	122.66	11- 1-		
		$v_u - w_u T$	=	360	к in		
		d d	=	10.84	in		
		$\phi V_c = 0.75 x 2 V f_c b_w d$	=	413.91	k	>	122.66k OK
System Weight:							
Item		#/bay		Total			
Slab		1		172500	lbs		

Provide (36) 1/2" dia. 270 ksi 7-wire strand in each direction & each bay

Self weight (PSF)=



1

1225

144.77 PSF

lbs