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Structural Option
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Oct. 31, 2005

Structural Technical Report 2
Pro-Con Structural Study of Alternative Floor Systems

Executive Summary

This report is a comparison of various floor systems that could be used in the design of the College of Business Administration building for Northern Arizona University. For the calculations included with this report, a typical framing bay was chosen to model the different floor systems. The loads used are based on the 2003 version of the International Building Code. Some factors that went into the comparison of they systems were ease of construction, costs, depth of system, fire protection, and weight of system.

Included in this report is an analysis of the floor system of the College of Business Administration. Also included are investigations into other floor systems that could possibly used when doing a redesign of the College of Business Administration. The first system checked was a system made up of precast concrete double tee beams and inverted tee beams as girders. Also looked at was a composite steel and concrete system. Another system explored was open web steel joists which frame into steel W-shaped girders. Two cast in place concrete systems were also examined. First, a one-way pan joist system, which used the CRSI Design Handbook was considered. The last system which was studied was a post-tensioned flat slab which was analyzed using the direct design method. All of these systems included in this report were designed for a typical bay which is seen throughout the College of Business Administration building. These are preliminary ideas used to see which systems deserve a closer look.

It was concluded in this report that the only investigated systems that are worth further investigation are the composite steel and concrete system, the post-tensioned flat slab, and the precast double tee system. With the long spans and high loads that are seen in the College of Business administration, the other systems were concluded to be uneconomical even from a preliminary look.

Introduction

Floor Loads

The typical loads used for this analysis are shown below. Live loads reductions are used where applicable. This analysis is being done using the typical bay specified above and does not represent the conditions throughout the entire building.

- Live Load: 100 PSF (Owner Specified over entire floor)
- Dead Load: 138 PSF
    - Structural Members: 128 PSF
    - MEP: 5 PSF
    - Miscellaneous: 5 PSF

To determine loads for new designs, the assumed self weight of the original design was replaced by the self weight of the new system. Added to the self weight was the 10 PSF dead load, as seen above.

Fireproofing

The College of Business Administration falls mostly into the Assembly Group A occupancy classification according to the IBC. From Table 302.3.3, it was found that a 2 hour fire rating should be used throughout this building. Table 719.1(3) and the *Underwriters Laboratories Directory, Fire Resistance-Volume 1* will be used to determine if the different floor systems have the minimum 2 hour fire rating that is needed. The table found in the comparison portion of the report will show if the structural materials alone will have the fire rating needed. If a system does not meet the rating with the materials alone it can still be used, but will increase the price and will be a factor to whether or not the system will be investigate further.

Existing Floor System Design

The College of Business Administration is comprised of four stories plus a mechanical mezzanine and is used mostly for classrooms and faculty offices. The four floors are constructed with structural precast elements and are similar from floor to floor. The main difference in the floors is the locations of the openings, but these openings follow a pattern and do not interrupt the column lines. Each floor has the same size beams and floor planks. A floor plan can be found in the appendix of this report.

The current design of the floors in the CBA is made up of precast concrete planks, with three inch structural concrete topping, which span between precast inverted t-beams. The planks are 10 inch hollow core planks and span 36 feet over the entire floor plan. These planks each have seven 0.5 inch diameter pre-stressing strands in them. The beams are 34"x 27" inverted t-beams which vary in length from 11 feet to 36 feet. For this study, a typical floor framing bay spanning from column lines 4 to 5 and C to D will be used. This bay contains 1296 square feet and has span lengths of 36 feet. Concise Beam 4.39 was used to analyze the original precast concrete system.

Advantages:

Erection Time: Since most of the work is done off-site, the construction time is cut in half because there is no form work or curing to be done on site.

Cost: Precast concrete tends to be less expensive than most other forms of construction. See the cost analysis shown at the end of this report.

Disadvantages:

Weight of system: This concrete system weighs 128 pounds per square foot which is higher than some other systems.

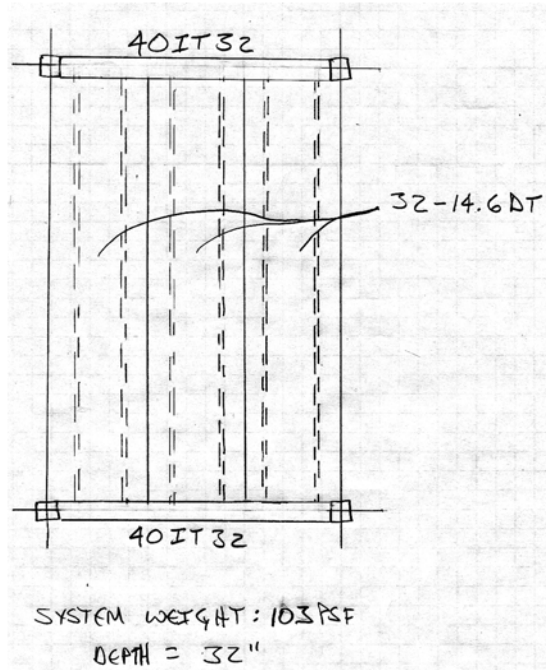
Availability: Precast concrete can be hard to find in some locations. If a manufacturer is not nearby, precast concrete is ruled out due to the very high costs in transporting the large members.

Floor Depth: The depth from floor to floor must be at least 30 inches because the inverted t-beam is 27 inches and it has a three inch topping.

Alternate Floor System Designs

Precast Double T-Beams and Inverted T-Beam

The first floor system investigated is an alternate design using precast, pre-stressed concrete members. The hollow core planks have been replaced with double t-beams. For this design two bays were used since double t-beams are usually more economical when spanning long distances. The span length used was 56 feet which is the distance between column lines C and E. This was chosen because spanning two, 36 foot bays would not be possible without using beams with depths much greater than wanted. This change would result in the ability to remove the columns along column line D. This would give the building more options on floor plan layout and the freedom for future changes if it was desired. However, a change like this could have very large impacts on the column sizes and the foundation. The loads on the interior columns would almost double and the loads on the exterior columns would increase be approximately 150%. It was shown that the same size 24" square columns could be used, but the rebar used would be 24 - #11 compared to the 12 - #10 used in the original design.



To find a double t-beam that would work, Nitterhouse Concrete Product's design tables were used. It was found that a 32-14.6DT could be used. This means a 32" depth beam, 30" precast double tee with 2" cast in place topping, with 14 0.6" diameter pre-stressing strands were used. To find the size of the girder needed, the PCI Design Handbook was referenced. A 40IT32 was chosen for the 36 foot span. This is a very large beam that weighs 1000 pounds per foot of length. This system is one that is used much more often in buildings such as parking garages due to its large depth. There is not a lot of room in this system for the complex mechanical systems used in the College of Business Administration.

Advantages:

Erection Time: Since most of the work is done off-site, the construction time is cut in half because there is no form work or curing to be done on site.

Cost: Precast concrete tends to be less expensive than most other forms of construction. See the cost analysis shown at the end of this report.

Open Floor Plan: This system would be designed so that the columns along column line D would be able to be eliminated. This would allow the space to be used in a more open fashion if it was preferred.

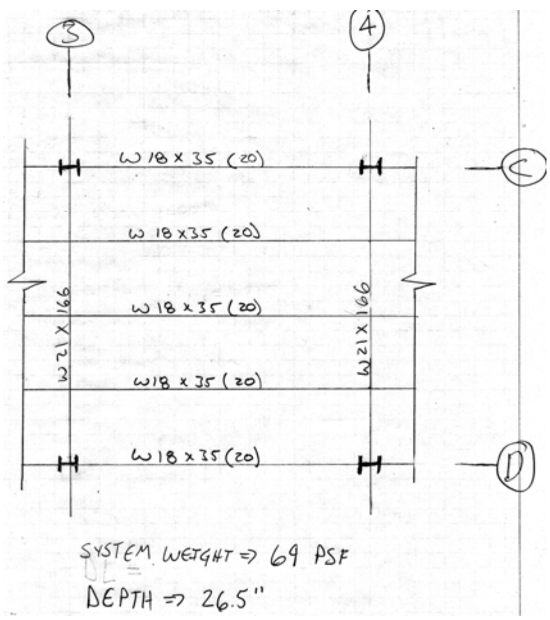
Disadvantages:

Weight of system: This concrete system weighs 103 pounds per square foot which is higher than some other systems.

Availability: Precast concrete can be hard to find in some locations. If a manufacturer is not nearby, precast concrete is ruled out due to the very high costs in transporting the large members.

Floor Depth: The depth of the floor system is 32 inches which is an increase from the original system. This could be an issue with zoning and with the architecture.

Structural Steel with Composite Deck



A possible alternate design for the floor system is cast-in-place concrete on composite metal decking supported by composite steel beams. The steel beams will span 36 feet and will be spaced nine feet apart. The composite metal deck was chosen to be the B-LOK 22 gage 1.5 x 6" deck from the United Steel Deck, Inc Design Manual and Catalog. The use of ¾" shear studs will allow for full composite action between the steel beams and concrete slab. There will be four inches of normal weight concrete plus the inch and a half deck. The concrete will be 4 ksi concrete.

The beam chosen after analysis was a W18 x 35 with 20 shear studs and the girder chosen was a W21 x 166. The girder was to be designed with the same depth as the beam. However, there were no W18 members that would hold the moment necessary so the next smallest W shape that could hold the loads was chosen. This system is somewhat less in weight than the original design and seems to be a good fit for bays of this size.

Advantages:

Floor Depth: The depth of this system will be only 23.5 inches. This is 6.5 inches less than the original design.

Weight of system: Steel construction will keep the building weight to a minimum. This decrease in weight will impact the foundation.

Construction time: Steel construction is usually able to be done faster than some other types of construction. Construction time is very often an important factor in determining what type of building is best for a project.

Disadvantages:

Cost: Composite steel construction is much more expensive than the original design. See the cost analysis shown at the end of this report.

Procurement time: The time it takes from when the design is done to when the steel can be on-site is a long time. The decision of system needs to be made early on in the project so that the steel can be manufactured and shipped by the time the contractors are ready for it.

Open Web Steel Joist

Another possible floor system would be to use open web steel joists which frame into steel girders. The joists will span 36 feet and be spaced four feet apart. There will be 3 inches on normal weight concrete on top of steel deck. The New Columbia Joist Company Steel Joists and Joist Girder catalog was used to size the joists.

A 24LH09 open web steel joist was chosen for this design. This member has a depth of 24" and a weighs 21 pounds per foot. Using the reactions from each joist, an equivalent uniformly distributed load was determined and moment was found to determine the size of the steel girder. A W21 x 93 was chosen for the 36 foot span. The open web steel joist system is very low in weight compared to the other systems examined in this report. The biggest drawback to this design is the need for fireproofing in some fashion. This takes time and can get expensive in some cases. Deflection and vibration could also be an issue with this system and should be checked if further investigation is warranted.

Advantages:

Weight of system: The open web steel joist system is by far the lightest of the systems checked. This can have an impact on the lateral system since seismic forces are based on the total weight of the structure.

Depth of system: The overall depth of this system turns out to be27 inches. This in itself is not much different from the original design, but since the webs are open, some mechanical and electrical equipment may be able to be placed between the webs.

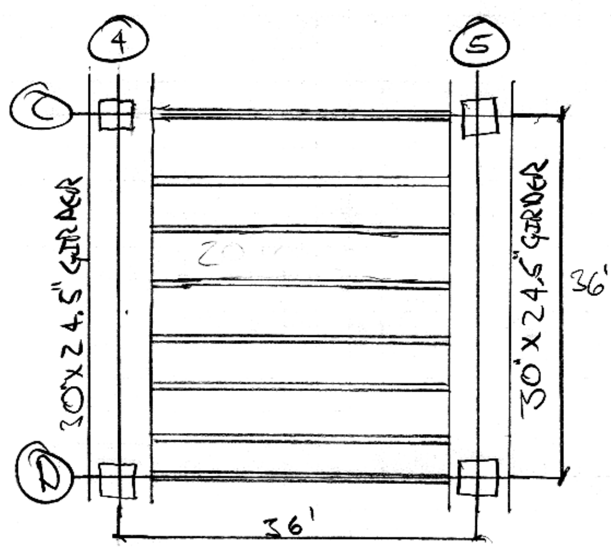
Disadvantages:

Connections: The number of connections in this system is higher than that of a traditional steel system. Also, the joists need to be welded on site to the girder. On-site welding is something that designers want to avoid when necessary because they are costly and time consuming.

Erection time: The time to construct this system is longer than most other. A lot of this is due to the large amounts of on-site welding. Also having W shape steel, open web joists, and cast in place concrete makes for a detailed construction process.

One way Concrete Pan Joist System

A one-way pan-joist concrete system was explored as a possible alternative floor system. The span length will be 36 feet and will be designed as an interior bay since there are multiple similar bays around the one chosen for design. The CRSI Concrete Design Handbook was used to size this floor system. Since the CRSI load tables are based on dead load and live load factors of 1.4 and 1.7 respectively, the loads for this design used those factors in order to be able to accurately compare loads. The complete calculations for this design can be found in the appendix of this report.



The design which was chosen uses 30" forms and 6" ribs. The ribs will be 20" deep with a 4.5 inch topping. The top reinforcing will need to be #6 bars at 11" on center and the bottom reinforcing will be 2-#7 bars in each rib. A one-way slab is usually better fitted in bays that are less square. The construction process is hit the hardest due to this system.

Advantages:

Depth of system: This system will only be 24.5 inches deep. This is 5.5 inches less than the original design.

Constructability: This design was chosen to be this size because there are standard forms that are used for this type of pan-joist system. This makes this system much easier to construct.

Disadvantages:

Weight of system: As with the other concrete systems examined, the weight of this system is high. It is similar to that of the original design, but higher than the steel systems.

Construction time: Since this system is cast-in-place, the curing time of the concrete will affect the schedule.

Post-Tensioned Concrete Flat Slab System

Another alternative design that was chosen was to use a post-tensioned concrete flat slab system. For this design, the bay used was located between column lines 4 and 5 and C and D. The direct design method was used to find the design moments. The slab was broken into a column strip and a middle strip in each direction for the bay, as shown in the calculations. The column strip was found to be 18 feet, the same size as the middle strip. The calculated moments were broken into mid-span moments and support moments as per the direct design method, and were further broken into column strip moments and middle strip moments.

The moments were used to find the minimum force needed to keep the section from cracking. This force was found to be 846 kips. One strand which is 0.6" diameter equals 35 kips of force, based on using 60% of the yield strength of 270 ksi. It was shown that the tendons will need to be placed at approximately nine inches on center in the column strip, which would be the worst case. This seems to be a little close, but could be off a little since this was just a preliminary design. This system has a higher weight than that of the original design which would force a look at other structural features, especially the foundation. The foundation may need to be resized since the loads it will be carrying will be higher.

Advantages:

Depth of system: The post-tensioned slab has the smallest depth of all the systems evaluated in this report.

Column size: With a post-tensioned slab, the columns could possibly be smaller since the tendons will take some of the shear.

Disadvantages:

Weight of system: This system has a higher weight than the original design. This is a very high weight which would mean the foundation would need to be checked and redesigned if it cannot handle the extra weight.

Construction: A post-tensioned system is one that requires a lot of detail during construction. The tendons have to be placed correctly and the pull forces must be accurate. Many times, an engineer is present during the placing of the concrete to ensure everything is done as it needs to be.

Comparison

System	Depth	Advantages	Disadvantages	Fireproofing 2HR?	Approximate Costs	Investigate More?
Existing System	30 in	Erection Time Cost	Weight Availability Depth	YES	Mat. $6.76 Inst. $3.04 Tot. $9.80	**-**
Precast Double Tee	34 in	Erection Time Cost Less Columns	Weight Availability Depth	YES	Mat. $6.58 Inst. $3.15 Tot. $9.73	**YES**
Composite Steel and Concrete	26.5 in	Depth Weight Const. Time	Cost Procurement Time	NO	Mat. $11.20 Inst. $5.15 Tot. $16.35	**YES**
Open-Web Steel Joist	27 in	Weight Depth	Connections Erection Time	NO	Mat. $9.85 Inst. $4.47 Tot. $14.32	**NO**
One-Way Pan-Joist	24 in	Depth Constructability	Weight Construction Time	YES	Mat. $6.75 Inst. $9.40 Tot. $16.15	**NO**
Post-Tensioned Slab	9.5 in	Depth Column Size	Weight Construction Time	YES	Mat. $6.26 Inst. $7.48 Tot. $13.64	**YES**

Conclusion

Based on the information above, it was decided that the systems that deserve a more detailed examination are the post-tensioned flat slab, the composite steel and concrete systems and the precast double tee system. The other systems do not seem to work well in this building. The College of Business Administration has long spans and high loads which cause some of these designs to call for very large and costly members. All of the analyses done were somewhat elementary and would need to be expanded to be a full design of a floor system of a building.

I feel the precast double tee system is not one I would want to explore more in the future for this building. A double tee floor system is one that is used more in parking garages not as much in educational buildings. The space between the floor and the ceiling below is not nearly as open as in other systems. I am very interested in doing a detailed design of a post-tensioned slab and the structure that would go with it. The analysis of this system was by far the most elementary since I do not know the details of post-tensioned construction. The system of composite concrete and steel is another that seems to work well in the CBA building. This system would also use steel columns which would need designed.

I feel there are really only two systems that would realistically work and need further examination. These are: post-tensioned slab, and composite steel and concrete.

Appendix

Structural Technical Report 2
Michael A. Troxell
College of Business Administration

ORIGINAL DESIGN

PRECAST HOLLOW CORE PLANKS ON
PRECAST INVERTED T-BEAMS

LOADS

DL:

PLANK = 90
3" TOPPING = 38
MEP = 5
MISC = 5

DL = 138 PSF
LL = 100 PSF

BEAM → CONCISE BEAM

10" HOLLOW CORE PLANK
w/ 7 - ½"φ PRESTRESSING STRANDS

GIRDER → CONCISE BEAM

34"x27" INVERTED T-BEAM
w/ 32-0.6"φ PRESTRESSING STRANDS
+ 4 #9 BARS TOP REINFORCING

AMPAD 22-141 50 SHEETS 22-142 100 SHEETS 22-144 200 SHEETS

AMPAD 22-141 50 SHEETS 22-142 100 SHEETS 22-144 200 SHEETS

PRECAST DOUBLE TEE

SPAN: 56' w/ 2" TOPPING

LOADS: DL = 40 PSF

LL = 100 PSF

TOTAL LOAD = 140 PSF

NITTERHOUSE CONCRETE PRODUCTS DESIGN TABLE

32-14.6 DT CAPACITY = 151 PSF

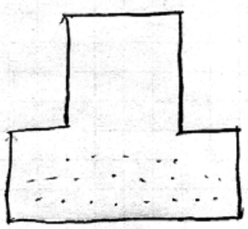
GIRDER

SPAN = 36 FT

R_x @ EACH TEE

R_x = 110 PSF(12 FT)($\frac{56}{2}$ FT)

R_x = 40^K @ 6'



EQUIVELENT DISTRIBUTED LOAD $\omega = 5.13^{KLF}$

PCI DESIGN HANDBOOK

pg 2-46

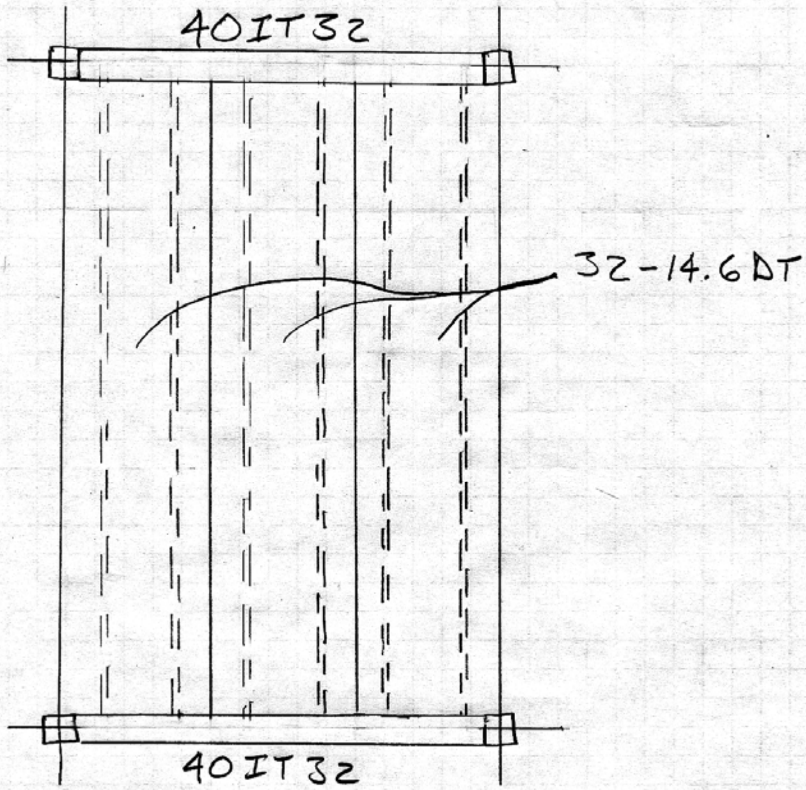
USE 40IT32 CAP = $6.375^{KLF} > 5.13^{KLF}$

∴ OK

GIRDER – 40IT32

WT – 1,000 plf

PRECAST DOUBLE TEE



SYSTEM WEIGHT: 103PSF
DEPTH = 32"

CAMPAD 22-141 50 SHEETS 22-142 100 SHEETS 22-144 200 SHEETS

COLUMN COMPARISON FOR PRECAST DOUBLE TEE FLOOR SYSTEM

ORIGINAL DESIGN:

24" SQUARE COLUMN

12 - #10 BARS

LOADS:
$DL = 93$ PSF
$LL = 100$ PSF

$P_U = 1.2D + 1.6L = 271.6$ PSF

$P_{UD} = 113^K$ PER FLOOR

* $P_{UL} = 81^K$ PER FLOOR

Reduced $P_{UL} = 49^K$ PER FLOOR

* ONLY LL ON ONE SIDE; THIS WILL CAUSE LARGEST MOMENT.

LL REDUCTION

$L = L_o\left(.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right)$

$= 81\left(.25 + \frac{15}{\sqrt{4(2016)}}\right)$

$LL = 81(.42) \Rightarrow$ LARGE REDUCTION

USE .6 INSTEAD

$LL = 81(.6) = 48.6$

$P_U = 1706^K$

$M_U = 68^{'K}$

INTERACTION DIAGRAM

$\rho = 0.06$

$A_{ST} = 34.6\ in^2$

INSTEAD OF USING 12 - #10 BARS

24 - #11 BARS COULD BE USED

SEE IF 7 - #11 BARS PER SIDE FITS

DIAMETER OF #11 = 1.41 in

$1.41(7) + 1.41(6) + 2.5(2) = 23.3" < 24\ in$ **OK**

AMPAD 22-141 50 SHEETS 22-142 100 SHEETS 22-144 200 SHEETS

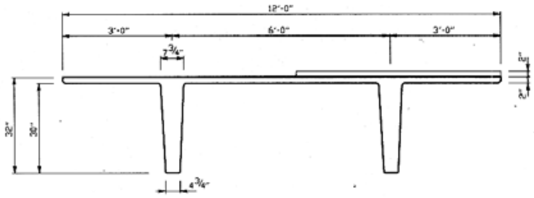
Prestressed Concrete

32" x 12' Double Tee

(2" C.I.P. TOPPING)

PHYSICAL PROPERTIES
Composite

A' = 951 in.^{2}	S'_b = 3198 in.^{3}
I' = 80,182 in.^{4}	S'_t = 11,570 in.^{3} (at top of D.T.)
Y_b = 25.07 in.	S'_{tt} = 8,979 in.^{3} (at top of topping)
Y'_t = 6.93 in. (to top of D.T.)	Wt'. = 991 PLF
Y'_{tt} = 8.93 in. (to top of topping)	Wt'. = 83 PSF



Depth	Strands 6 - 0.6" Ø	Parallel	Draped	Topping
32	6.6	P	D	T

DESIGN DATA

1. Precast strength @ RELEASE = 3000 PSI (min.)
2. Precast strength @ 28 days = 6000 PSI
3. Precast Density = 150 PCF
4. Strand = 0.6" Ø 270k LO-relaxation
5. Topping Strength = 3,000 PSI
6. Topping Density = 150 PCF
7. Maximum bottom tensile stress is $12\sqrt{f'c}$ = 930 PSI.
8. All superimposed load is treated as live load in the flexural strength analysis.
9. Flexural capacity is based on stress/strain strand relationships.
10. Maximum moment capacity is critical at midspan for parallel stands and is critical near 0.4 span for draped strands.
11. All loads shown refer to allowable loads after the topping has hardened.

Table of Safe Superimposed Loads (lbs. per sq. ft.)

Section	Ø M_n (in. Kips)	Span in Feet																				
		46	48	50	52	54	56	58	60	62	64	66	68	70	72	74	76	78	80	82	84	86
32 - 6.6 PT	9,334	76	64	54	45	36																
32 - 8.6 PT	11,900	116	101	88	76	66	56	48	40	33												
32 - 10.6 PT	14,322	153	135	119	105	92	81	71	62	53	46	39	33									
32 - 12.6 PT	16,423	185	165	146	130	116	103	91	81	71	63	55	48	41	35	30						
32 - 14.6 DT	20,943	256	230	206	186	167	151	136	123	110	99	89	80	72	64	57	51	45	39	33		
32 - 16.6 DT	22,860							161	146	132	120	109	99	88	78	69	60	53	48	42	38	33
32 - 18.6 DT	24,486													105	94	84	75	66	58	51	44	39

This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths.

NITTERHOUSE
CONCRETE PRODUCTS

2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17201-0813
717-267-4505 • FAX: 717-267-4518

WD 5M 5/94

COMPOSITE FLOOR SYSTEM
STEEL BEAMS w/ METAL DECK

AMPAD 22-141 50 SHEETS 22-142 100 SHEETS 22-144 200 SHEETS

METAL DECKING

LOADS:

LL = 100 PSF

DL:

5½" SLAB = 69

MEP = 5

MISC. = 5

DL = 79 PSF

FACTORED LOADS : 1.2 DL + 1.6 LL

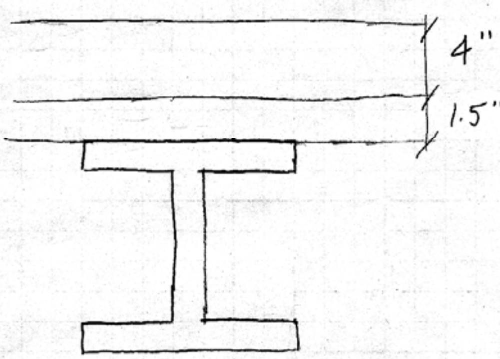
= 1.2(79) + 1.6(100) = 255 PSF

SPAN = 9 FT

UNITED STEEL DECK, INC.
DESIGN MANUAL + CATALOG   Pg 23

USE B-LOK 22 GAGE   CAPACITY = 255 PSF

STEEL BEAM



STEEL BEAM DESIGN

LOADS:

DL = 79 PSF
LL = 100 PSF

SPAN = 36 FEET
TRIB = 9 FEET
$f'_c = 4$ ksi

FACTORED LOADS: $w_u = 255$ PSF

$$M_u = \frac{(255)(9')(36')^2}{8}$$

$$M_u = 372'^K$$

$$b_{eff} = \frac{36(12)}{4} = 108 \qquad b_{eff} = 108''$$
$$9' = 108$$

ASSUME: $a = 1''$

$$Y_2 = 5\tfrac{1}{2} - \tfrac{1}{2} = 5''$$

TABLE 5-14 LRFD MANUAL
ASSUME P.N.A. IS @ PT. 6

TRY W18X35 $\phi_b M_n = 382'^K$, $\Sigma Q_n = 194^K$

CHECK ASSUMPTION $a = 1''$

$$a = \frac{\Sigma Q_n}{.85 f'_c\, b} = \frac{194}{.85(4)(108)} = .53'' < 1'' \quad \underline{OK}$$

$$\phi M_n = 382'^K > M_u = 372'^K \quad \underline{OK}$$

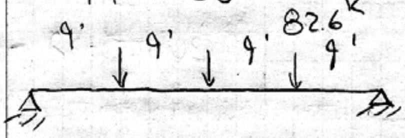
ASSUME 1 SHEAR STUD TRANSFERS 21^K.

$$\frac{\Sigma Q_n}{Q_n} = \frac{194}{21} = 9.23 \Rightarrow 10 \text{ PER SIDE} \Rightarrow 20 \text{ SHEAR STUDS}$$

USE W18X35 w/20 SHEAR STUDS

GIRDER DESIGN

– SPAN = 36'



REACTION FROM EACH BEAM:

$$R_x = \frac{2(w_u)(TRIB)(LENGTH)}{2}$$

$$R_x = \frac{2(.255)(9)(36)}{2}$$

$$R_x = 82.6^K$$

$M = aPL$

$a = 0.5$

$P = 82.6^K$

$L = 36'$

$M_u = .5(82.6)(36) = 1487^{'K}$

NO W18 IS SATISFACTORY
SO TRY W21

TRY W21 x 166 $\phi M_p = 1620^{'K}$

SELF WT MOMENT: $\frac{(.166)(1.6)(36)^2}{8} = 43^{'K}$

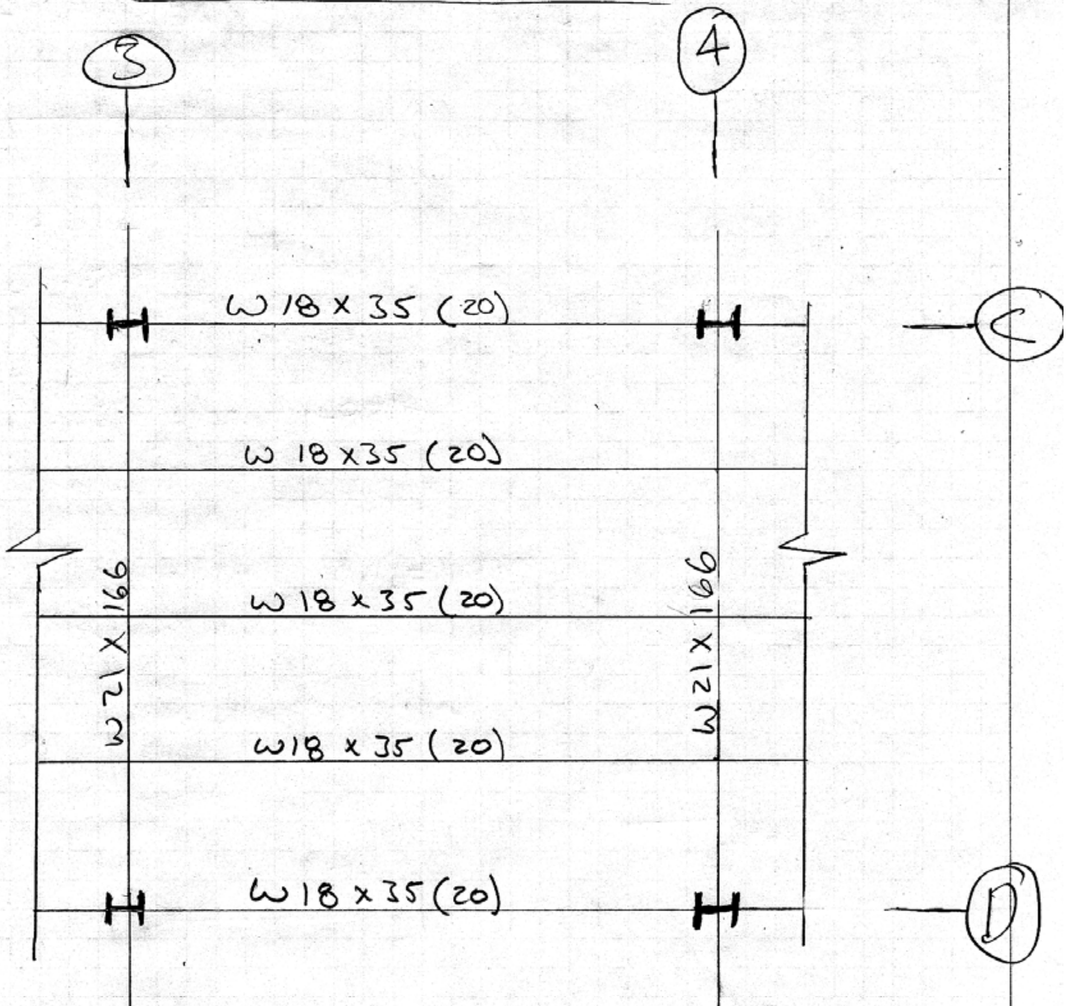
$M_u = 1487 + 43 = 1530^{'K}$

$\phi M_n = 1620^{'K} > 1530^{'K}$ OK

USE W21 x 166 GIRDERS

COMPOSITE (4)

STEEL + COMPOSITE DECK



SYSTEM WEIGHT => 69 PSF

DEPTH => 26.5"

OPEN WEB STEEL JOIST

SPAN = 36 FT

SPACING = 4 FT

CONCRETE – 2½"

LOADS: DL = 10 PSF + 38 PSL (SELF WT) = 48 PSF
LL = 100 PSF

UNIFORMLY DISTRIBUTED LOAD: 148 PSF (4 FT)

$\omega_u = 592$ plf

$\omega_{LL} = 400$ plf

NEW COLUMBIA JOIST COMPANY
STEEL JOISTS + JOIST GIRDERS

USE: 24LH09 → DEPTH – 24"
WEIGHT – 21 PLF

GIRDER DESIGN

– SPAN: 36'

– POINT LOADS: 22.1^{K} @ 4' o.c.

EQUIVALENT DISTRIBUTED LOAD:

$\omega_u = 4.904$ klf

REACTION FROM JOIST

$$R_x = \frac{(592+21)(36)^2(2)}{2}$$

$R_x = 22.1^{K}$

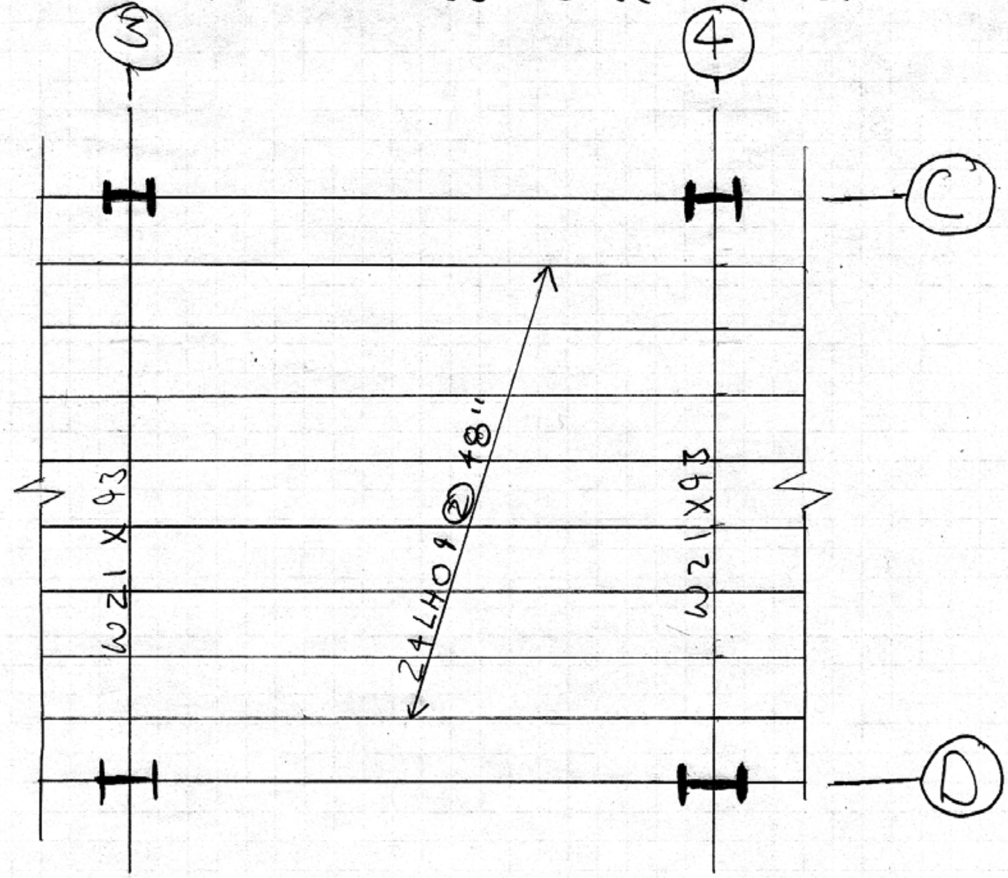
$$M = \frac{(4.9)(36)^2}{8} = 795^{'K}$$

W21X93 $\phi M_n = 829^{'K}$

$$M_{selfwt} = \frac{(.093)(36)^2}{8} = 15^{'K}$$

$\phi M_n = 829^{'K} > M = 810^{'K}$ OK.

OPEN WEB STEEL JOIST



SYSTEM WEIGHT: 43 PSF

DEPTH: 26.5"

AMPAD 22-141 50 SHEETS 22-142 100 SHEETS 22-144 200 SHEETS

ONE-WAY PAN-JOIST SYSTEM.

LOADS:
LL = 100 PSF
DL = 10 PSF

4.5" SLAB

TOTAL LOAD: 1.4(10) + 1.7(100) = 184 PSF

CRSI DESIGN HANDBOOK

USE: 6"x20" deep RIB w/ 30" FORM

TOP REINF: #6 @ 11
BOTTOM REINF: (2) #7

GIRDER: 30 x 24.5"

TOP REINF: (5) #7
BOTTOM REINF: (8) #8

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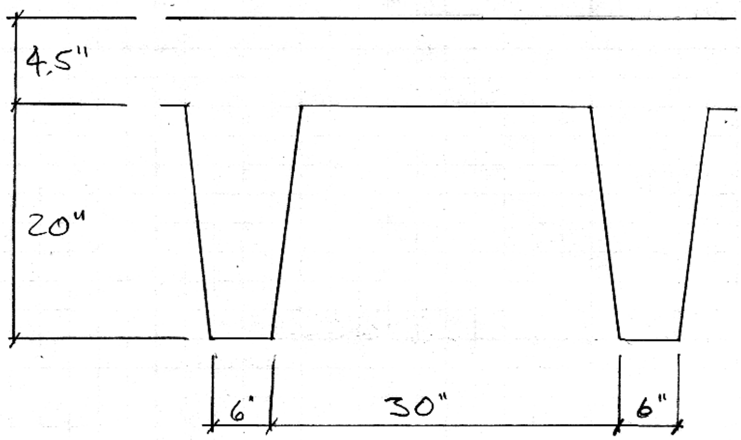
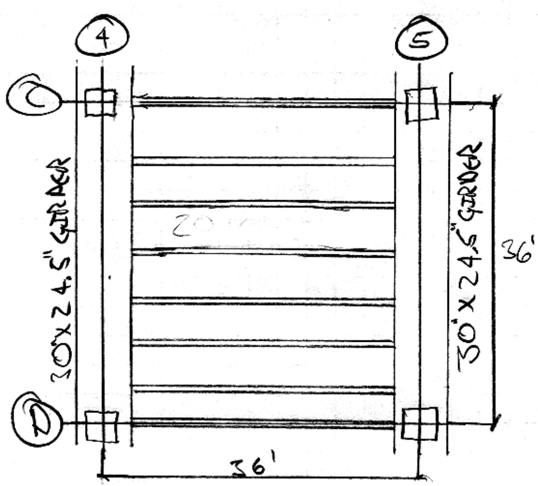
ONE-WAY PAN-JOIST SYSTEM





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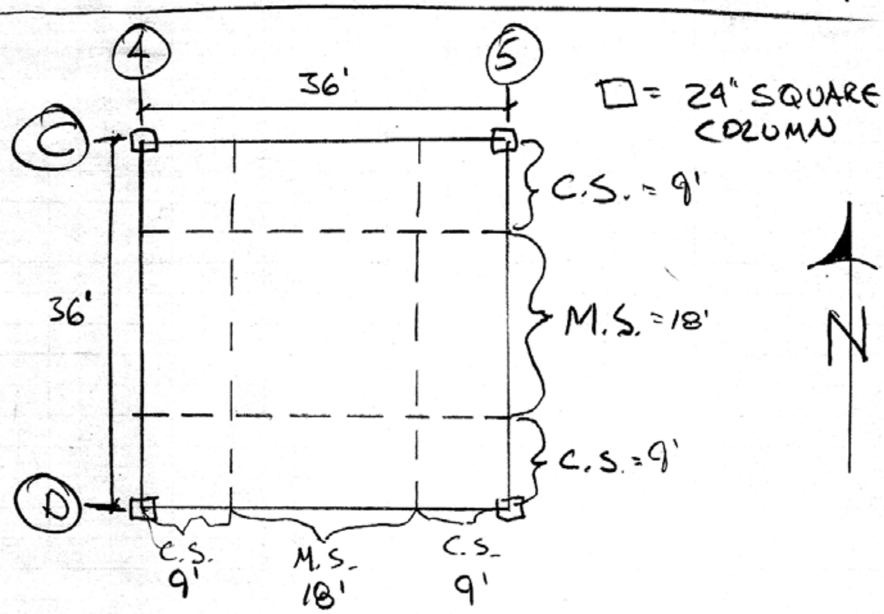
POST-TENSIONED SLAB DESIGN



C.S. = COLUMN STRIP = ½ SHORT SPAN = $\frac{36}{2}$ = 18'

THICKNESS OF SLAB: SPAN/DEPTH RATIO MAX ≈ 45

$$\frac{36'(12)}{45} \approx 9.5''$$

USE 9.5" OF CONCRETE.

LOADS:

LL = 100 PSF

DL = 10 + 145($\frac{9.5}{12}$) = 125 PSF

ω_U = .225 KSF — NO FACTORING

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P-T- (2)

N-S DESIGN = E-W DESIGN

$\ell_1 = 36'$

$\ell_2 = 36'$

$\ell_n = 36' - 24" = 32'$

$w_u = .285$ KSF

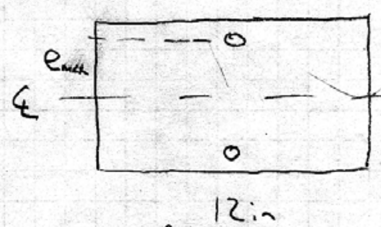
$M_o = \frac{w_u \ell_2 \ell_n^2}{8} = \frac{.262(36)(32)^2}{8} = 1037^{1K}$

	DESIGN M_o	TOTAL WIDTH	MOMENT/FT OF WIDTH
INT SUPPORT 65% of M_o 674[1K]	C.S. 75% 505	18'	28.1 1K/FT
	M.S 25% 169	18'	9.4 1K/FT
MID SPAN 55% of M_o 362[1K]	C.S. 60% 218	18'	12.1 1K/FT
	M.S. 40% 145	18'	8.1 1K/FT

INT. SUPPORT

C.S. $M_o = 28.1^{1K/FT}$

$S = \frac{bd^2}{6} = \frac{(12)(9.5)^2}{6} = 181 \text{ in}^3$



$_{max} f_t = 7.5\sqrt{f'_c} = 474$ psi

$e_{max} = \frac{9.5"}{2} - 1" = 3.75"$

$f'_c = 4$ ksi

$A = (12\text{in})(9.5\text{in}) = 114\text{in}^2$

$e = 3.75"$

$f_c = .6 f'_c = 2400$ psi

$f_{top} = f_t = -\frac{M}{S} + \frac{P_e}{A} + \frac{P_e e}{S} =$

P-T (3)

INT SUPPORT $M = 505^{'K}$

$$f_{t_{max}} = \frac{M}{S} - \frac{P_e}{A} - \frac{P_e e}{S}$$

$$.474 = \frac{505(12)}{3258} - \frac{P_e}{2052} - \frac{P_e(3.75)}{3258}$$

$P_e = 846^K$ (MIN) ⟵ CONTROLS

$$f_b = -\frac{M}{S} - \frac{P_e}{A} + \frac{P_e e}{S}$$

$$-2.4 = -\frac{505(12)}{3258} - \frac{P_e}{2052} + \frac{P_e(3.75)}{3258}$$

$P_e = -814^{K}$ (WILL NOT CRUSH FROM COMPRESSION)

MIDSPAN $M = 218^{'K}$

$$f_t = -\frac{M}{S} - \frac{P_e}{A} + \frac{P_e e}{S}$$

$$-2.4 = -\frac{218(12)}{3258} - \frac{P_e}{2052} + \frac{P_e(3.75)}{3258}$$

$P_{e_{min}} = -2406$ → (WILL NOT CRUSH FROM COMPRESSION)

$$f_b = \frac{M}{S} - \frac{P_e}{A} - \frac{P_e e}{S}$$

$$.474 = \frac{218(12)}{3258} - \frac{P_e}{2052} - \frac{P_e(3.75)}{3258}$$

$f_b = 201^K$

P-T (4)

$P_{e\,min} = 846^k$

WANT TO USE $P_{e\,min}$ ⇒ CHEAPEST

P_{e1} (1 STRAND) = (.216)(270)(.6) = 35^k

$\frac{P_e}{P_{e1}}$ = # OF STRANDS = $\frac{846}{35}$ = 24.17 ⇒ 25 STRANDS

P_e = 25(35) = 875^k

18 FEET – 25 STRANDS

SPACING = 8.6" ⇒

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