Executive Summary

The purpose of this report is to provide an analysis of the existing floor system of the Earth and Engineering Sciences Building at University Park, Pennsylvania, as well as investigating four additional floor systems that will compare and contrast with the original. A typical bay, which will be defined in the report, was used to provide a small scale design for each floor system. All loads were computed in accordance to the International Building Code with the exception of the loading used with the CRSI Handbook. Each design includes a design of the main floor system, accompanied by suggestions for column and girder sizing. The existing system, a steel frame with concrete on metal decking, was analyzed by hand and by RAM structural analysis software.

A comparison of four systems will also be detailed in this report. The four alternate systems considered are as follows:

1. A992 (50 ksi) Grade Steel, w/concrete on metal deck
2. Hollow core plank, w/ steel framing members
3. One way concrete pan joists, w/concrete framing members
4. Open web steel joists, w/steel framing members

Each alternate system involves slightly altered spans, loadings, and directional properties. All of which are defined in the section that explains their design. Various methods were used for each system, as well as multiple references. After analysis each system was compared and contrasted to each other and the existing system in order to determine a suitable alternate. The first alternative is similar to the existing system in design but still provides added benefits. The other three systems are significantly different and added many aspects that needed to be considered.

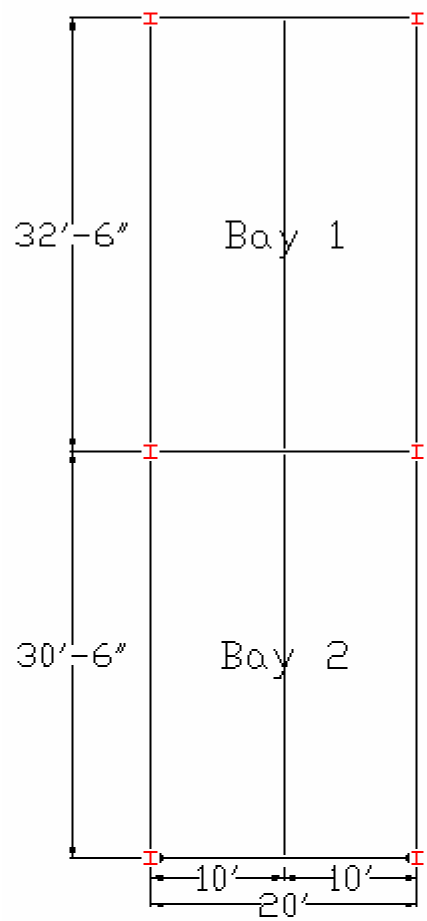
After evaluating all the pros and cons of all the systems a recommendation for an appropriate alternate system will be made. The A992 grade steel system was discarded due to it's likeness to the original system in depth. It does have the benefit of smaller members and a reduction in weight but does not provide many additional benefits. The open web steel joist system was not considered an option after it was found to be a deeper system than the existing one and would be difficult to fireproof. The two remaining systems were both concrete based systems. The edge in the recommendation went to the hollow core system. The hollow core slabs provide a smaller self weight and a more shallow depth than the one way pan joist. This summarization is provided in greater detail at the end of the report and can be viewed in both written and tabular format. The existing system was an efficient and cost effective system, but upon further investigation it can be determined that both of the alternate concrete based systems can be viable options. The hollow core plank system should be looked at in greater detail as a new option for design of the EES Building at Penn State.

Existing Structural Floor System
Composite 36 ksi Steel Beams w/Concrete on Metal Deck

The existing floor system is composed of composite action steel beams that work in conjunction with the floor slab system. The steel used in the system is primarily 36 ksi grade steel. There are a few exceptions where 50 ksi grade steel was used, but predominantly the 36 ksi steel is found. The slab system has three deck types, however the most common deck a 3", 20 GA galvanized composite metal deck with 3 ¼ " light weight concrete topping reinforced with 6 x 6, W2.1 x W2.1 welded wire fabric will be used in this study. This deck, commonly referred to as Slab 1, is used in most floor spaces with the exception of some high live load areas such as stairwells. The beams frame into steel girders which help transfer the load to the steel columns. The columns then transfer the load to the concrete piers and footings that they rest on.

A typical bay was chosen to compare and contrast alternate systems to the existing system. Most bays in the Earth and Engineering Sciences building are fairly uniform spanning either 30'-6" or 32'-6". The widths of the bays are 20' with beams spaced midway at 10' on center. Two typical bays spanning in the North-South Direction are shown to the right. (Fig. 1)

Fig. 1



The bay spanning 30'-6" (Bay 2) will be the focus of the analysis and comparison of the existing and alternate floor systems. The unfactored service loads to be used will be as follows:

- Dead load (excluding self weight of floor system) – 25 psf
- Live Load (For Northern most bays, i.e. Bay 1) – 125 psf
- Live Load (For Southern most bays, i.e. Bay 2) – 80 psf

The plan for the existing structural system specifies a number of beam and girder sizes throughout the building, however there is repetition and a common bay does exist. The most common configuration is shown below (Fig. 2a, Fig. 2b). As can be seen below, the beam sizes in Bay 1 are larger than Bay 2. This is due to the higher live load on this bay which was noted earlier. A previous spot check was performed on these bays and found that member sizes are accurate for the assumptions and loadings that will be used to perform further analysis of alternate systems. This spot check can be found in Technical Assignment 1 in Justin Strauser's e-portfolio. A second check was performed using RAM structural design software. The results of this analysis also yielded beam and girder sizes that were close to that of the original plans, the output of this program can be seen in Appendix A.

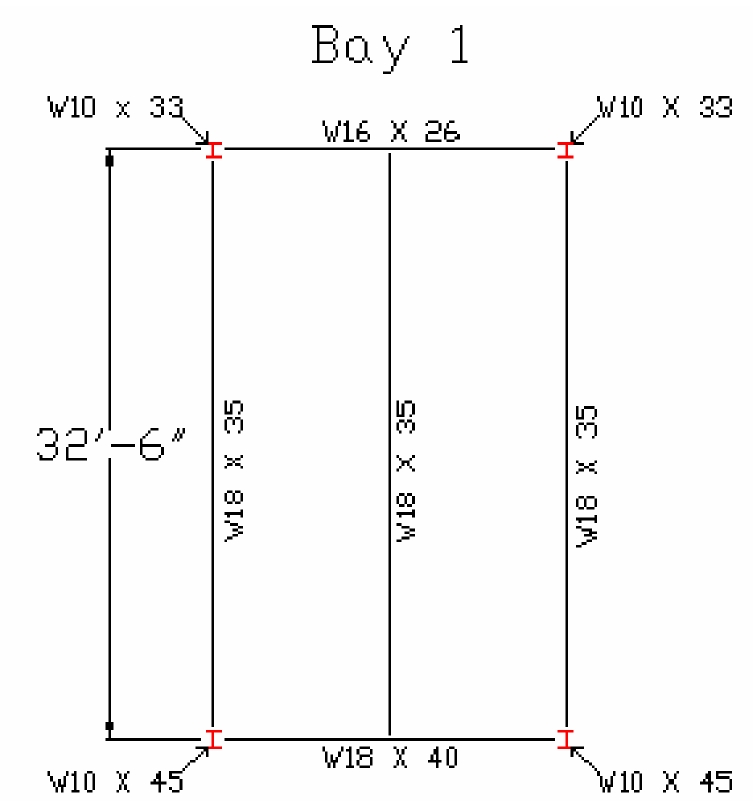


Fig. 2A

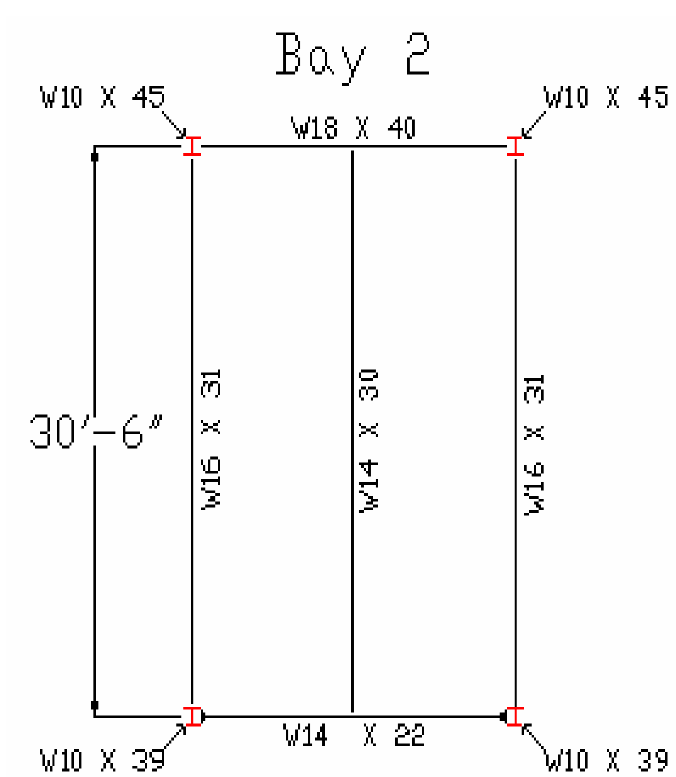
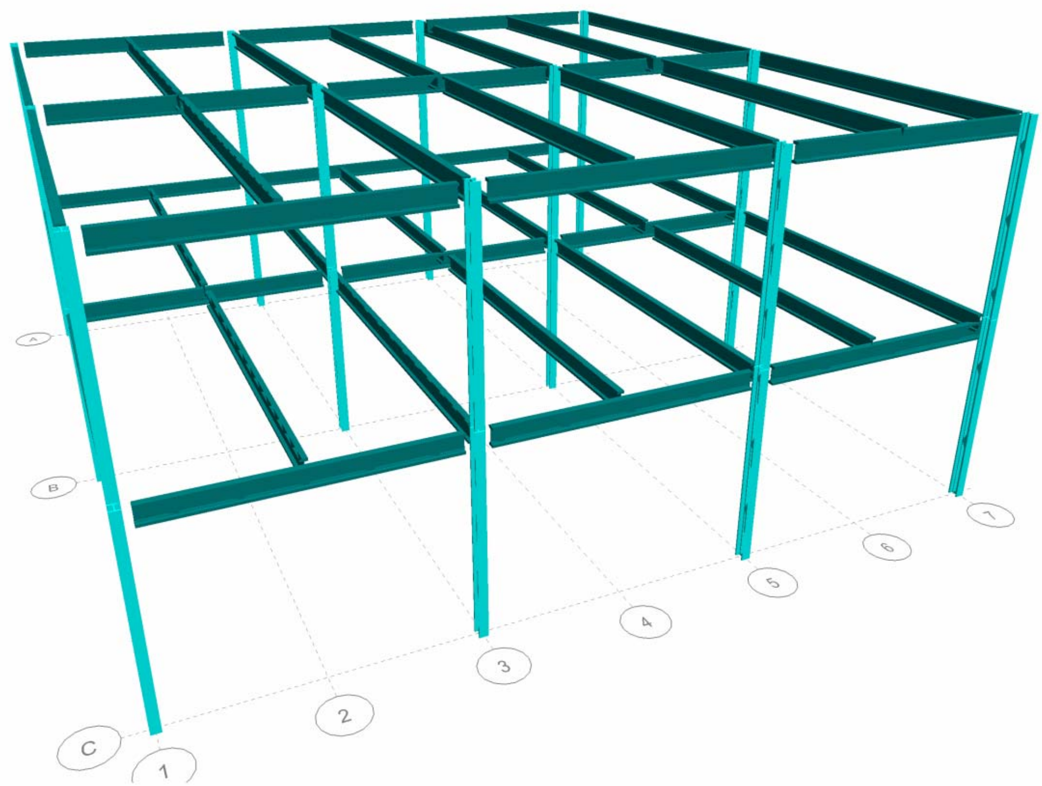


Fig. 2B



Typical Frame of Existing System

Advantages:

- Easily connected
- Erection is fast
- Relatively light in weight
- Composite action improves strength and easy to connect
- Slab provides fire rated barrier between floors

Disadvantages:

- Deep floor system that does not provide much room for supplementary systems (i.e. mechanical, electrical)
- Steel corrodes and rusts
  → Requires protective layer
- Steel fails under high temperatures
  → Needs to be fireproofed

Alternate Structural Floor System 1

Composite and Non-Composite 50 ksi Steel Beams w/Concrete on Metal Deck

The first alternative system was to change the steel from 36 ksi steel to 50 ksi steel. This change could be more efficient in that smaller beams could be used reducing the amount of steel used throughout the building. Smaller beams could also reduce the amount of space wasted between floors. This system's performance will be evaluated as both composite and non-composite. This system will also be designed using repetitive members with no changes in beams. A 2", 18 gauge, unshored LOK floor, topped with 3" light weight concrete providing a 2 hour fire rating will be used in the design.

Service Loads

- Dead load (excluding self weight of floor system) – 25 psf
- Live Load (For Northern most bays, i.e. Bay 1) – 125 psf
- Live Load (For Southern most bays, i.e. Bay 2) – 80 psf

Material Properties

f'_c = 3 ksi
f_y = 50 ksi

Composite Design

w_u = 1.2(25 psf) + 1.6(80 psf) = 158 psf
M_u = (.158 ksf)(10 ft)(30.5 ft)^2/8 = 183.7 ft-k

Assume a = 1.0"
Y_2 = 5" – (1.0"/2) = 4.5 "
Using Table 5-14 from AISC Manual for Steel Construction with Y_2 = 4.5"

Try W14 x 22 with Y_1 @ location 6

b_{eff} = lesser of spacing or L/4, 120" and 91.5" respectively
b_{eff} = 91.5"

Check assumption:
a = 119 k/.85(3 ksi)(91.5") = .51"
Y_2 = 5" – (.51"/2) = 4.74 " => O.K.
Check for efficiency:

Beam Size	ΦM_p (ft-k)	ΦM_n (ft-k)	$\sum Q_n$ (k)	*# of Studs	Total Weight (lbs)
W14x22	125	191	119	12.0	780
W14x26	151	208	96.1	10.0	880
W12x30	162	219	110	12.0	1020
W12x26	140	189	95.6	10.0	880
W12x22	110	187	153	16.0	820
W12x19	92.6	186	208	20.0	770
W10x26	117	197	190	18.0	960

*Based on shear capacity of studs = 21 k per stud

The most efficient beam size for this design would be the W14 x 22. It may not be the lightest selection, but the 12 shear studs for the W14 x 22, as opposed to the 20 shear studs for the W12 x 19, would take less time and money to connect. The W14 x 22 would also provide a little more bending strength than the W12 x 19.

Check deflection criteria:

ΔLL = 5(.8 klf)(30.5 ft)^4(1728)/384(29,000 ksi)(423 in^4) = 1.27 " > L/360 = 1.02 "
< L/180 = 2.03"
Will work!

ΔDL = 5(.43 klf)(30.5 ft)^4(1728)/384(29,000 ksi)(199 in^4) = 1.45 " > L/360 = 1.02 "
< L/180 = 2.03"
Will work!

Note: The inertia values for deflection were taken form Table 5-15 in the AISC Manual.

Non-Composite Design

The beam no longer has composite action from slab and must take into account an added dead load from the slab when using the AISC beam design tables.

M_u = 214 ft-k
Assume unbraced length L_b = 30.5 ft

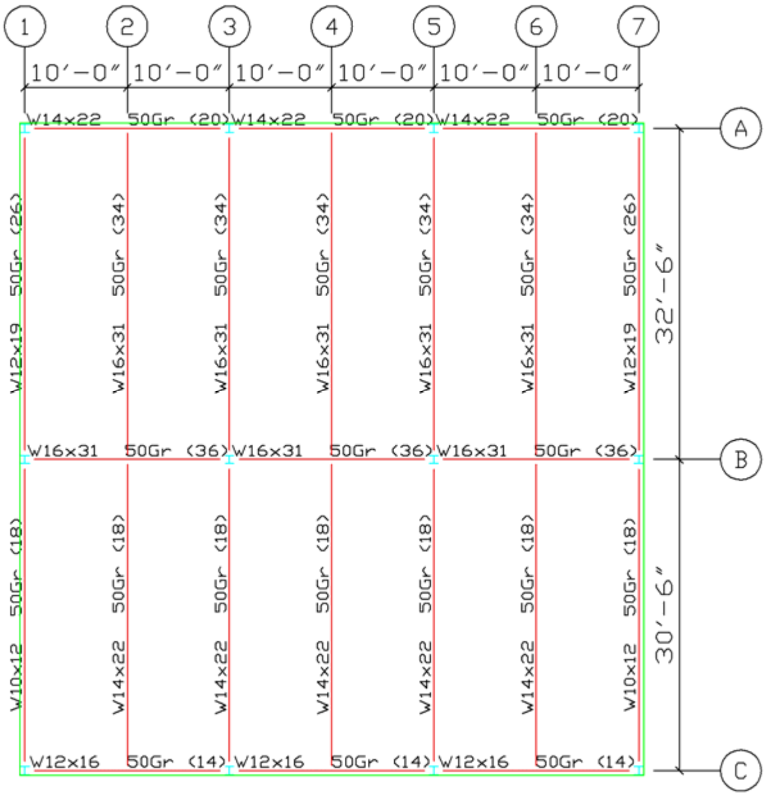
From Beam Selection Table pg. 5-97 of the AISC Manual
Use a W12 x 65

The use of a non-composite system would not be a good choice here. To obtain the same amount of strength as the composite system a much heavier beam would need to be used. When comparing the amount of steel alone of the W14 x 22 with 12 shear studs to that of the W12 x 65, it can be seen the disadvantage of this system. The W12 x 65 would be almost twice as heavy as its composite counterpart. This large amount of steel would increase the price of erection and fabrication costs. Therefore only the Composite 50 ksi system will be considered as an alternative to the existing system.

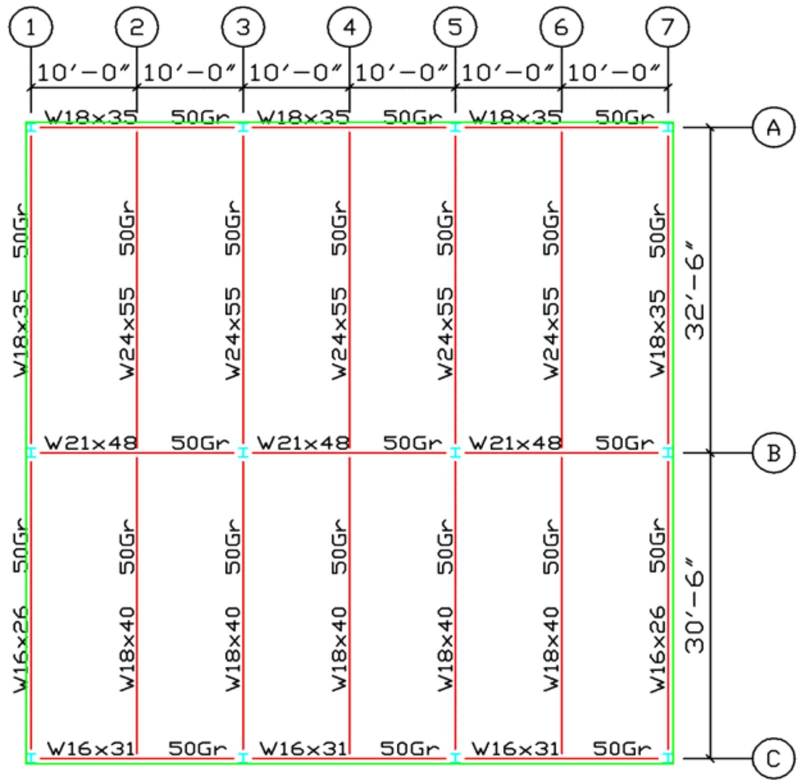
A secondary analysis was done in RAM to calculate further member sizes. A typical layout can be seen below. Column sizes can be found in the summary located in Appendix B. Column sizes are W10 x 33, which did not change from the RAM selection for the existing system.

Girder Consideration

The supporting girders for this alternate system will be taken from the RAM output file which can be found on the next page. The composite design provides smaller members than the existing system while the noncomposite suggests larger members,



50 Ksi Composite



50 Ksi Non-Composite

Advantages:

- Easily connected
- Erection is fast
- Relatively light in weight, will be lighter than existing system (smaller beams/columns)
- Composite action improves strength and easy to connect
- Slab provides fire rated barrier between floors

Disadvantages:

- Deep floor system that does not provide much room for supplementary systems (i.e. mechanical, electrical), however will be more shallow than existing system
- Steel corrodes and rusts
  → Requires protective layer
- Steel fails under high temperatures
  → Needs to be fireproofed

Alternate Structural Floor System 2
Prestressed Hollow Core Plank

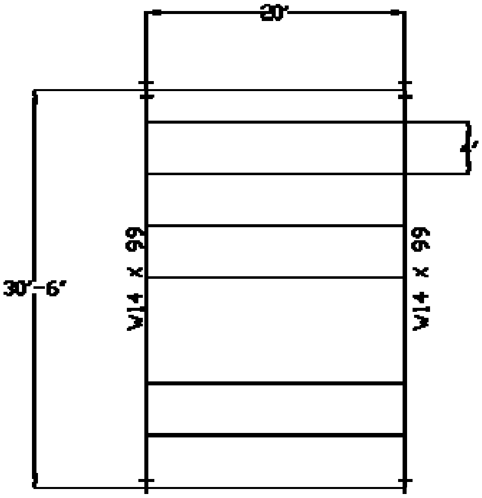
The second alternative system is a prestressed hollow core plank floor system. Hollow core planks are concrete planks fabricated off site and brought onto the job when needed. The prefabrication alone has its own disadvantages and advantages. Scheduling the delivery sequence is a major factor in the cost and efficiency of hollow core construction. However, when the planks are brought on site they are placed on girders or load bearing walls by a crane. The planks in this design will rest on steel girders that will frame into the columns. In order to keep the same bay configuration the planks have been selected to span the short direction. The same loading patterns used previously will be used in the design of this system.

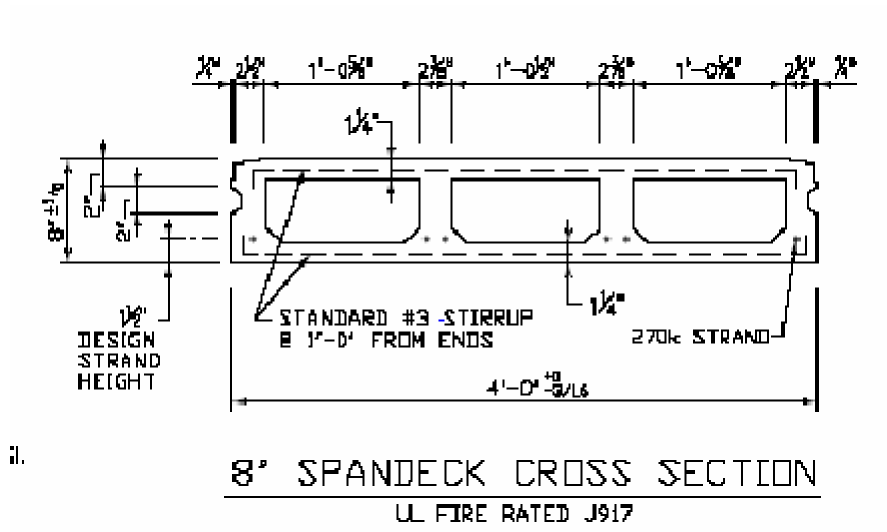
Design

w_u = 1.2(25 psf) + 1.6(80 psf) = 158 psf

From PCI and Nitterhouse Tables using a span of 20'
(Table can be found in Appendix B)

Use 8" x 4' Spandeck – U.L. – J917 without topping
Self weight 57.5psf
4 – ½ "Φ, 270 K, Low-Lax Strands
f'_c = 5000 psi





8" Hollowcore planks
Framing Plan

Girder Considerations

w_u = 1.2(82.5 psf) + 1.6(80 psf) = 227 psf
Tributary Width = 20 ft
227 psf (20') = 4.54 klf
M_u = (4.54 klf)(30.5 ft)^2/8 = 528 ft-k

Steel: From beam selection tables
W14 x 99 – 99 plf selfweight 14" depth

Concrete Inverted T Beam from PCI Design Handbook(See Appendix E)
28IT32 – 600 plf selfweight 32" depth

Concrete Rectangular Beam from PCI Design Handbook(See Appendix E)
12RB36 – 450 plf selfweight 36" depth

Girder selected: The steel W14 x 99 will be used with this design. Its reduced size and weight make it the optimum choice for this system. It can also be constructed the quickest and easiest.

Other Considerations

The self-weight of the planks will increase the dead load seen be the columns by about 10 psf. This change will slightly increase the size of the columns needed to support the frame.

Advantages:

- Easily erected
- Fabricated Off Site reducing lag time
- Thin Slab
- Lighter than most concrete systems
- No forming to be done
- Easier to construct supplementary systems (i.e. electrical)
- Prefabrication assures consistency in properties (i.e. strength, quality, durability)

Disadvantages:

- Scheduling issues most be worked out due to prefabrication
- Fireproofing needed
- Needs slightly larger beam sizes or bearing walls to support
- Not as cost effective

Alternate Structural Floor System 3

One Way Pan Joist

The third alternate system is a concrete one way pan joist system. This system will be designed to span the short direction of 20 feet. This is a cast in place system that utilizes metal pans to create ribs that act as joists. Steel strands are placed to reinforce the top of the slab and in the bottom of the rib. When compared to previous systems, a pan joist system will be heavier due to the weight of the concrete. When contemplating the use of a pan joist floor system, scheduling must be taken under consideration and can create issues. Concrete becomes more difficult to pour at lower temperatures, and the pans need to be placed for any pours to be made. In addition, if a concrete plant is not located near the site, it can be difficult to continuously pour the system. A batch plant may need to be located on site. The benefits of this system will need to be weighted against the ability to work with constructability issues.

Design

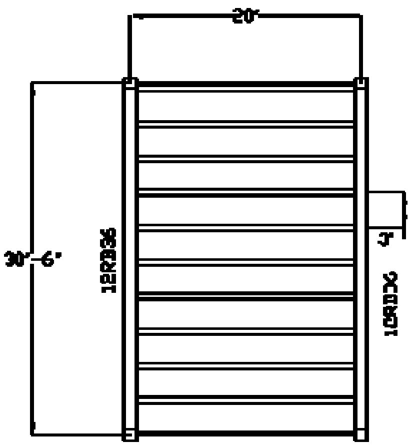
w_u = 1.4(25 psf) + 1.7(80 psf) = 171 psf

From CRSI Tables for one way pan joists using a span of 20'
(Table can be found in Appendix D)

Use pan joist with total width = 30" Forms + 6" Ribs @ 36" c.-c.
Total Depth = 10" Rib + 3" slab = 13"
Self weight 61.5psf
f'_c = 4000 psi
f_y = 60,000 psi
Steel reinforcing (.86 psf)

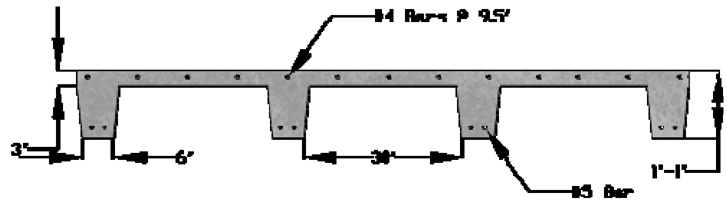
1. Top - #4's @ 9.5 "
2. Bottom – #5, #5

Provided 209 psf > 171 psf needed



One Way Pan Joist

Framing Plan



Cross Section of Pan Joist

Girder Considerations

Selfweight of pan joist = .41 CF/SF (610 SF) = 250.1 CF => 250.1 CF (150 PCF) = 37.52 k
Selfweight (psf) = 61.5 psf
w_u = 1.2(86.5 psf) + 1.6(80 psf) = 231.8 psf
Tributary Width = 20 ft
231.8 psf (20') = 4.64 klf
M_u = (4.64 klf)(30.5 ft)^2/8 = 539.55 ft-k

Steel: From beam selection tables
W21 x 101 – 101 plf selfweight   21" depth

Concrete Inverted T Beam from PCI Design Handbook(See Appendix E)
28IT32 – 600 plf selfweight   32" depth

Concrete Rectangular Beam from PCI Design Handbook(See Appendix E)
12RB36 – 450 plf selfweight   36" depth

Girder selected: The 12RB36 will be used with this floor design. A concrete beam would be the most reasonable choice with a pan joist system as it can be

poured and constructed along with the joists. The rectangular beam is the simpler of the two concrete beams and contributes the smaller dead load to the structural system.

Other Considerations

Columns for this system will be the largest of all the systems as they will be required the largest load of 231.8 psf.

Advantages:

- Reduced depth
- Uniform properties aid in construction
- Does not require fireproofing
- Increases resistance to shear failure
- Reusable formwork

Disadvantages:

- Takes longer to construct because it is a cast in place system
- Self weight of system is higher
- Needs a larger framing system to support

Alternate Structural Floor System 4

Open Web Steel Joists

The final system being considered will be open web steel joists. Steel joists are fabricated off site and are available in different truss configurations. The configuration of this design will be a K-joist. A reinforced concrete slab will be placed on the joists. The concrete will be held on a metal deck to allow it to cure. The depth of this system can vary depending on the joist selected, however in this analysis it will be chosen to remain fairly uniform. A two hour fire rating has been selected for determining slab characteristics. The joist will span the long direction of 30'-6".

Design

2" reinforced concrete topping
3" LOK Floor deck

w_u = 1.2(55 psf) + 1.6(80 psf) = 194 psf
*Weight of concrete and metal deck included in 55 psf dead load

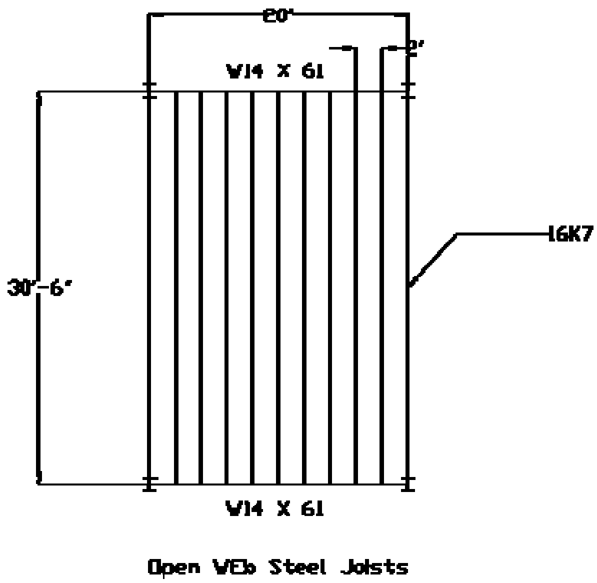
w_{ji} = 194 psf /(1.65)(.9) = 130 psf

span = 31'

Using NCJ Design tables pgs. 18, 32-35 (see Appendix F)

Spacing (ft)	Load (plf)	Joist from Table	Allowable Load (plf)	Selfweight (plf)	Rows of Bridging
5	650	NA	NA	NA	NA
4	520	26K9	550	12.2	2
3	390	24K7	424	10.1	2
2	260	16K7	277	8.6	2

The most efficient joist is the 16K7 spaced at 2' on center. The 5 ft spacing does produces a load that cannot be supported by any K-joist listed in the table and is not practical to use. The 16K7 will require two rows of bridging and with the concrete slab and decking will have a total depth of 21".



Framing Plan

Girder Considerations

Selfweight of steel joists = 8.6 plf (30.5 ft) = 262.3 lbs
9 joists (262.3 lbs/joist) = 2360.7 lbs --- Assume uniformly distributed along beam length
Selfweight (plf) = 118.035 plf
Tributary Width = 30.5 ft
w_u = 1.2(118.035 plf) + 1.2(30.5 ft)(55 psf) + 1.6(30.5 ft)(80 psf) = 6.06 klf
M_u = (6.06 klf)(20 ft)^2/8 = 303 ft-k

Steel: From beam selection tables
W14 x 61 – 61 plf selfweight 14” depth

Concrete Inverted T Beam from PCI Design Handbook(See Appendix E)
28IT28 – 550 plf selfweight 28” depth

Concrete Rectangular Beam from PCI Design Handbook(See Appendix E)
12RB28 – 350 plf selfweight 28” depth

Girder selected: The steel W14 x 61 will be used in this. The weight, depth, and constructability of this girder makes this the best selection.

Other Considerations

The columns for this system will actually be reduced in size from the existing and alternate steel designs. The steel joist system transfers the smallest loading to the framing elements than each system presented.

Advantages:

- Excess room for supplementary systems especially mechanical
- Lightweight
- Short erection time

Disadvantages:

- Deep floor system
- Hard to fireproof
- More tightly spaced
- Lateral loads will be increased
- Need lead time for fabrication
- Produces excess vibrations

Summary

System	Depth*	Weight (psf)	Advantages	Disadvantages	Consideration for future system
Existing Steel	22 ¼"	212	Easily connected Erection is fast Relatively light in weight	Deep floor system Steel corrodes and rusts Steel fails under high temperatures	N/A
50 ksi steel	21"	210	Easily connected Erection is fast Relatively light in weight[1]	Deep floor system[2] Steel corrodes and rusts Steel fails under high temperatures	NO
Hollowcore Planks	10"	227	Easily erected Lighter than most concrete systems Thin Slab	Fireproofing needed Not as cost effective Needs slightly larger beam sizes or bearing walls to support Scheduling issues most be worked out due to prefabrication	YES
One Way Pan Joist	15"	231.8	Reduced depth Uniform properties aid in construction Does not require fireproofing Increases resistance to shear failure Reusable formwork	Takes longer to construct Self weight of system is higher Needs a larger framing system to support	YES
Open Web Steel Joists	24"	198.7	Excess room for supplementary systems especially mechanical Lightweight Short erection time	Deep floor system Hard to fireproof More tightly spaced Lateral loads will be increased Need lead time for fabrication Produces excess vibrations	NO

*2" tolerance added for floor and ceiling finishes

[1]Lighter than existing system
[2]Not as deep as existing

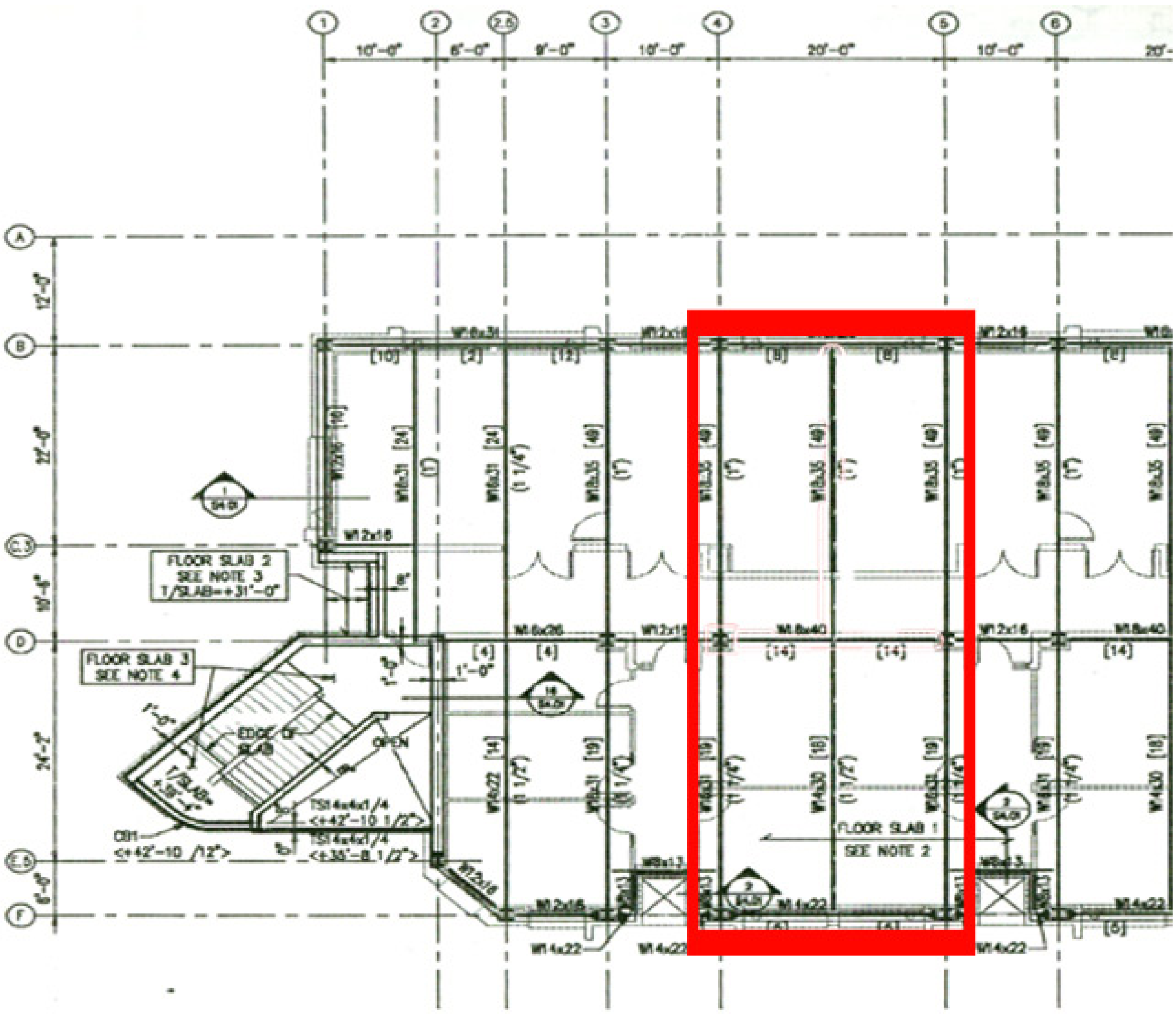
Every system has multiple advantages and disadvantages to consider. Changing the grade of steel helped reduce the size of steel used, but the effects weren't drastic and the depth was still relatively large. The alternate steel system will not be considered as a viable alternative.

The concrete systems both become viable options as a system replacement. These systems both add weight to the overall structure, but the depth of the floor is greatly reduced. The added weight should not be a limiting criterion as the structure will only be four stories. The hollow core can be efficiently erected with proper planning and does not require a minimum temperature for placement. The hollow core can also rest on both steel or concrete framing members and load bearing walls. The pan joist will require planning as well in order to prevent pouring concrete in low temperatures and having the forms ready in proper sequence. One great advantage to the pan joist is the increased resistance to shear failure. Both of these systems will be considered as alternates to the current system.

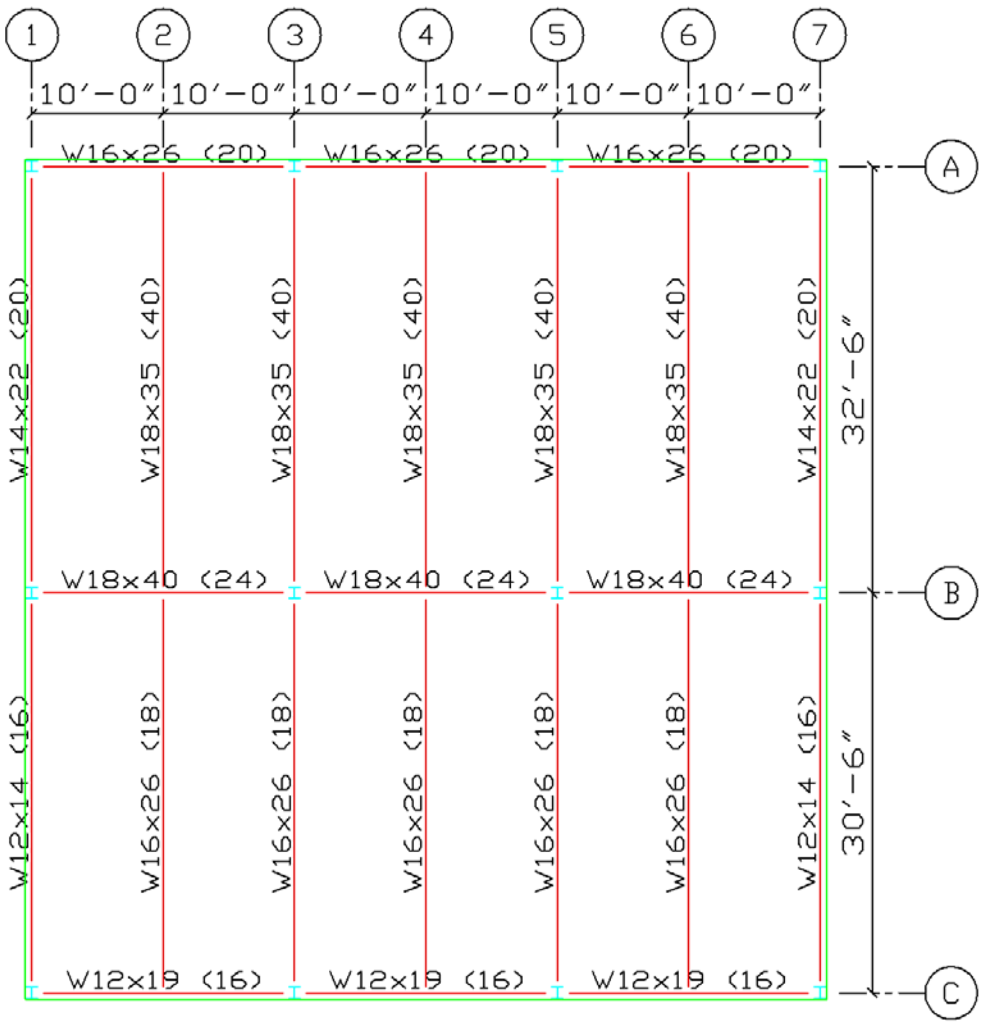
The final system considered was the open web steel joists. This system was discarded for a number of reasons. The fireproofing would be tough to apply. Lateral loads will be increased as well as additional vibrations in the floor. The fact that this system is lighter than the original system is one of its greatest advantages but this is overshadowed by its depth which is almost equivalent to the existing system.

In conclusion the existing system was a good selection for the Earth and Engineering Sciences Building. However, upon further design and consideration it may be found that the two alternate concrete systems may be as good or better than the existing system.

APPENDIX A – Floor Layout and Ram Output for existing floor system



THIRD FLOOR FRAMING PLAN — WEST WING



1
2
3
4
5
6
7
10'−0"
10'−0"
10'−0"
10'−0"
10'−0"
10'−0"
W16x26 (20)
W16x26 (20)
W16x26 (20)
A
W14x22 (20)
W18x35 (40)
W18x35 (40)
W18x35 (40)
W18x35 (40)
W18x35 (40)
W14x22 (20)
32'−6"
W18x40 (24)
W18x40 (24)
W18x40 (24)
B
W12x14 (16)
W16x26 (18)
W16x26 (18)
W16x26 (18)
W16x26 (18)
W16x26 (18)
W12x14 (16)
30'−6"
W12x19 (16)
W12x19 (16)
W12x19 (16)
C

Ram Output

APPENDIX B – Column Summary 36 ksi and 50 ksi

Gravity Column Design Summary

RAM Steel v8.1
tech
DataBase: tech 10/28/05 14:13:12
Building Code: IBC Steel Code: ASD 9th Ed.

Column Line 1 - C

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
third	15.5	4.5	4.1	1	0.38 Eq H1-3	90.0	36	W10X33
third	26.5	1.9	1.7	1	0.22 Eq H1-1	90.0	50	W10X33

Column Line 1 - B

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
third	27.6	4.4	8.5	8	0.61 Eq H1-2	90.0	36	W10X33
third	55.1	0.8	3.7	1	0.46 Eq H1-1	90.0	50	W10X33

Column Line 1 - A

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
third	22.6	6.7	6.0	1	0.51 Eq H1-2	90.0	36	W10X33
third	38.0	2.8	2.5	1	0.33 Eq H1-1	90.0	50	W10X33

Column Line 3 - C

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
third	21.9	7.7	2.2	6	0.33 Eq H1-2	90.0	36	W10X33
third	42.4	3.3	0.9	6	0.31 Eq H1-1	90.0	50	W10X33

Column Line 3 - B

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
third	45.7	3.0	4.8	6	0.63 Eq H1-1	90.0	36	W10X33
third	90.0	1.3	2.0	6	0.65 Eq H1-1	90.0	50	W10X33

Column Line 3 - A

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
third	30.8	11.1	3.7	6	0.50 Eq H1-2	90.0	36	W10X33
third	59.4	4.8	1.5	6	0.45 Eq H1-1	90.0	50	W10X33

Column Line 5 - C

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
third	21.9	7.7	2.2	6	0.33 Eq H1-2	90.0	36	W10X33
third	42.4	3.3	0.9	6	0.31 Eq H1-1	90.0	50	W10X33

Column Line 5 - B

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
third	45.7	3.0	4.8	11	0.63 Eq H1-1	90.0	36	W10X33
third	90.0	1.3	2.0	6	0.65 Eq H1-1	90.0	50	W10X33

Column Line 5 - A

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size

Gravity Column Design Summary

RAM Steel v8.1 Page 2/2
tech
DataBase: tech 10/28/05 14:13:12
Building Code: IBC Steel Code: ASD 9th Ed.

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
third	30.8	11.1	3.7	6	0.50 Eq H1-2	90.0	36	W10X33
third	59.4	4.8	1.5	6	0.45 Eq H1-1	90.0	50	W10X33

Column Line 7 - C

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
third	15.5	4.5	4.1	1	0.38 Eq H1-3	90.0	36	W10X33
third	26.5	1.9	1.7	1	0.22 Eq H1-1	90.0	50	W10X33

Column Line 7 - B

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
third	27.6	4.4	8.5	8	0.61 Eq H1-2	90.0	36	W10X33
third	55.1	0.8	3.7	1	0.46 Eq H1-1	90.0	50	W10X33

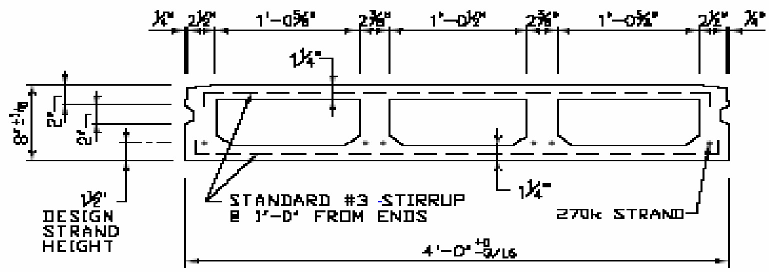
Column Line 7 - A

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
third	22.6	6.7	6.0	1	0.51 Eq H1-2	90.0	36	W10X33
third	38.0	2.8	2.5	1	0.33 Eq H1-1	90.0	50	W10X33

APPENDIX C – Hollow core Selection

Prestressed Concrete
8"x4' SpanDeck—U.L.—J917
(NO TOPPING)

PHYSICAL PROPERTIES Precast			
A = 180 in.^{2}		S_b = 397 in.^{3}	
I = 1543 in.^{4}		S_t = 375 in.^{3}	
Y_b = 3.89 in.		Wt. = 230 PLF	
Y_t = 4.11 in.		Wt. = 57.5 PSF	
e = 2.39 in.			



8" SPANDECK CROSS SECTION
UL FIRE RATED J917

DESIGN DATA
1. Precast Strength @ 28 days = 5000 PSI.
2. Precast Density = 150 PCF.
3. Strand = 1/2"ϕ, 270 K Lo-Relaxation.
4. Strand Height = 1.50 in.
5. Ultimate moment capacities (when fully developed)...
   4 — 1/2"ϕ, 270K = 74.3'K
   6 — 1/2"ϕ, 270K = 105.6'K
6. Maximum bottom tensile stress is $6\sqrt{f'c}$ =424 PSI.
7. All superimposed load is treated as live load in the strength analysis of flexure and shear.
8. Flexural strength capacity is based on stress/strain strand relationships.
9. All values in this table are based on ultimate strength and are not governed by service stress.
10. Shear values are the maximum allowable before shear reinforcement is required.
11. Deflection limits were not considered when determing allowable loads in this table.

8" SPANDECK W/O TOPPING	ALLOWABLE SUPERIMPOSED LOAD (PSF)																						
STRAND PATTERN	SPAN (FEET)																						
	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32
Flexure 4 — 1/2"ϕ	610	550	499	457	399	341	294	255	222	195	171	151	133	117	103	92	82	72	66	56	49	43	X
Shear 4 — 1/2"ϕ	441	393	354	321	294	270	249	231	215	201	188	177	160	145	132	120	110	101	95	90	82	75	X
Flexure 6 — 1/2"ϕ	885	800	726	667	586	509	437	382	334	296	263	234	208	187	168	151	136	122	111	100	90	81	73
Shear 6 — 1/2"ϕ	459	411	370	337	308	283	262	243	226	211	197	185	174	164	155	147	139	131	120	111	102	94	87

NITTERHOUSE
CONCRETE PRODUCTS

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.

2655 MOLLY PITCHER HWY. SOUTH, BOX N
CHAMBERSBURG, PA 17201-0813
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REVISED 12/93

APPENDIX D – CRSI Handbook Excerpt

STANDARD ONE-WAY JOISTS (1) MULTIPLE SPANS		30" Forms + 6" Rib @ 36" c.-c. (2) FACTORED USABLE SUPERIMPOSED LOAD (PSF)											f'_c = 4,000 psi f_y = 60,000 psi
	10" Deep Rib + 3.0" Top Slab = 13.0" Total Depth												
TOP BARS	Size @	# 4 12	# 4 12	# 4 9.5	# 5 12	# 5 10	End Span Defl. Coeff. (3)	# 4 12	# 4 11.5	# 4 8.5	# 4 7	# 5 8.5	Int. Span Defl. Coeff. (3)
BOTTOM BARS	# #	# 4 # 4	# 4 # 5	# 5 # 5	# 5 # 6	# 6 # 6		# 3 # 4	# 4 # 4	# 4 # 5	# 5 # 5	# 5 # 6	
Steel (psf)		.58	.69	.86	1.05	1.25		.63	.74	.95	1.18	1.46	
CLEAR SPAN		END SPAN						INTERIOR SPAN					
17'-0"		180 0	251 0	309* 322	317* 404	328* 478*	1.006	215 0	301 0	358* 404	365* 508	374* 570*	.619
18'-0"		151 0	214 0	278 0	288* 351	297* 425	1.264	182 0	259 0	328* 351	334* 444	342* 525*	.778
19'-0"		127 0	184 0	241 0	263* 306	271* 373	1.569	155 0	224 0	301* 306	307* 390	314* 481*	.966
20'-0"		106 0	157 0	209 0	241* 268	248* 328	1.926	131 0	194 0	268 0	284* 343	289* 429	1.185
21'-0"		88 0	135 0	182 0	221* 235	228* 290	2.342	111 0	168 0	235 0	263* 303	267* 381	1.441
22'-0"		73 0	115 0	158 0	204* 207	210* 256	2.820	94 0	145 0	207 0	244* 269	248* 340	1.736
23'-0"		59 0	98 0	137 0	182 0	194* 227	3.369	78 0	126 0	182 0	227* 239	231* 303	2.073
24'-0"		48 0	83 0	119 0	160 0	180* 202	3.995	65 0	108 0	160 0	212* 0	215* 272	2.458
25'-0"			70 0	103 0	141 0	167* 179	4.703	53 0	93 0	141 0	189 0	201* 244	2.894
26'-0"			58 0	89 0	124 0	156* 159	5.502	43 0	80 0	123 0	168 0	188* 219	3.386
27'-0"			48 0	76 0	108 0	141 0	6.398	68 0	108 0	150 0	176* 197		3.938
28'-0"				65 0	95 0	125 0	7.400		57 0	95 0	133 0	166* 177	4.554
29'-0"				54 0	82 0	111 0	8.516		47 0	82 0	118 0	156* 159	5.240
30'-0"				45 0	71 0	98 0	9.752			71 0	105 0	143 0	6.001

(1) For gross section properties, see Table 8-1.
(2) First load is for standard square joist ends; second load is for special tapered joist ends.
(3) Computation of deflection is not required above horizonal line (thickness ≥ $\ell_n/18.5$ for end spans, $\ell_n/21$ for interior spans).
(4) Exclusive of bridging joists and tapered ends.
*Controlled by shear capacity. +Capacity at elastic deflection = $\ell_n/360$.

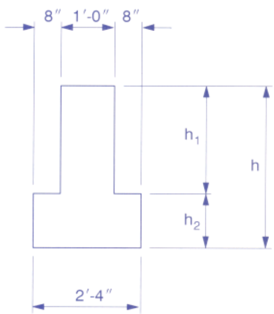
PROPERTIES FOR DESIGN (CONCRETE .41 CF/SF) (4)

NEGATIVE MOMENT											
STEEL AREA (SQ. IN.)	.60	.60	.76	.93	1.12		.60	.63	.85	1.03	1.31
STEEL % (UNIFORM)	.73	.73	.92	1.14	1.37		.73	.76	1.03	1.25	1.61
(TAPERED)	.43	.43	.54	.66	.80		.43	.44	.60	.73	.94
EFF. DEPTH, IN.	11.8	11.8	11.8	11.7	11.7		11.8	11.8	11.8	11.8	11.7
– ICR/IGR	.179	.179	.214	.246	.280		.179	.185	.232	.268	.314
POSITIVE MOMENT											
STEEL AREA (SQ. IN.)	.40	.51	.62	.75	.88		.31	.40	.51	.62	.75
STEEL %	.09	.12	.15	.18	.21		.07	.09	.12	.15	.18
EFF. DEPTH, IN.	11.8	11.7	11.7	11.6	11.6		11.8	11.8	11.7	11.7	11.6
+ICR/IGR	.162	.200	.239	.280	.323		.128	.162	.200	.239	.280

APPENDIX E – PCI Design Handbook Beam Tables

INVERTED TEE BEAMS

Normal Weight Concrete



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

½ in. diameter
low-relaxation strand

Key
6,929 — Safe superimposed service load, plf
0.3 — Estimated camber at erection, in.
0.1 — Estimated long-time camber, in.

Section Properties

Designation	h in.	h_1/h_2 in.	A in^2	I in^4	y_b in.	S_b in^3	S_t in^3	wt plf
28IT20	20	12/8	368	11,688	7.91	1,478	967	383
28IT24	24	12/12	480	20,275	9.60	2,112	1,408	500
28IT28	28	16/12	528	32,076	11.09	2,892	1,897	550
28IT32	32	20/12	576	47,872	12.67	3,778	2,477	600
28IT36	36	24/12	624	68,101	14.31	4,759	3,140	650
28IT40	40	24/16	736	93,503	15.83	5,907	3,869	767
28IT44	44	28/16	784	124,437	17.43	7,139	4,683	817
28IT48	48	32/16	832	161,424	19.08	8,460	5,582	867
28IT52	52	36/16	880	204,884	20.76	9,869	6,558	917
28IT56	56	40/16	928	255,229	22.48	11,354	7,614	967
28IT60	60	44/16	976	312,866	24.23	12,912	8,747	1,017

1. Check local area for availability of other sizes.

2. Safe loads shown include 50% superimposed dead load and 50% live load. 800 psi top tension has been allowed, therefore additional top reinforcement is required.

3. Safe loads can be significantly increased by use of structural composite topping.

Table of safe superimposed service load (plf) and cambers

Desig-nation	No. Strand	e	Span, ft 16	18	20	22	24	26	28	30	32	34	36	38	40	42	44	46	48	50
28IT20	9	5.82	6929 0.3 0.1	5402 0.3 0.1	4310 0.4 0.1	3502 0.4 0.1	2887 0.5 0.1	2409 0.6 0.1	2029 0.6 0.0	1723 0.7 0.0	1473 0.7 0.0	1265 0.8 −0.1	1091 0.8 −0.1							
28IT24	11	6.77	9714 0.2 0.1	7580 0.3 0.1	6054 0.3 0.1	4925 0.4 0.1	4066 0.4 0.1	3398 0.5 0.1	2868 0.6 0.1	2440 0.6 0.1	2090 0.7 0.1	1799 0.7 0.0	1556 0.7 0.0	1351 0.8 0.0	1175 0.8 −0.1	1024 0.8 −0.2				
28IT28	13	8.44			8505 0.3 0.1	6951 0.4 0.1	5768 0.5 0.1	4848 0.5 0.2	4118 0.6 0.2	3529 0.7 0.2	3047 0.7 0.2	2648 0.8 0.2	2313 0.9 0.2	2030 0.9 0.2	1788 1.0 0.2	1579 1.0 0.1	1399 1.1 0.1	1242 1.1 0.1	1103 1.1 0.0	981 1.1 −0.1
28IT32	15	9.17				9202 0.3 0.1	7646 0.4 0.1	6435 0.4 0.1	5474 0.5 0.1	4698 0.5 0.1	4064 0.6 0.1	3538 0.6 0.1	3097 0.7 0.1	2724 0.7 0.1	2406 0.8 0.1	2132 0.8 0.1	1894 0.9 0.0	1687 0.9 0.0	1505 0.9 0.0	1345 0.9 −0.1
28IT36	16	10.81						8485 0.4 0.1	7236 0.4 0.1	6227 0.5 0.1	5402 0.5 0.1	4718 0.6 0.1	4145 0.6 0.1	3660 0.7 0.1	3246 0.7 0.1	2890 0.8 0.1	2581 0.8 0.0	2311 0.9 0.0	2075 0.9 0.0	1866 0.9 −0.1
28IT40	19	11.28							8615 0.4 0.1	7415 0.4 0.1	6433 0.5 0.1	5620 0.5 0.1	4938 0.6 0.1	4361 0.6 0.1	3868 0.7 0.1	3444 0.7 0.1	3077 0.8 0.1	2756 0.8 0.1	2475 0.8 0.1	2226 0.9 0.0
28IT44	20	12.89								9308 0.4 0.1	8092 0.5 0.1	7083 0.5 0.1	6239 0.5 0.1	5524 0.6 0.1	4913 0.6 0.1	4388 0.7 0.1	3932 0.7 0.1	3535 0.8 0.1	3186 0.8 0.1	2879 0.8 0.0
28IT48	22	14.16									9741 0.4 0.1	8539 0.5 0.1	7532 0.5 0.1	6680 0.6 0.1	5952 0.6 0.1	5326 0.7 0.1	4783 0.7 0.1	4310 0.8 0.1	3894 0.8 0.1	3528 0.8 0.1
28IT52	24	15.44											8935 0.5 0.1	7934 0.5 0.1	7080 0.6 0.1	6345 0.6 0.1	5707 0.7 0.1	5151 0.7 0.1	4664 0.8 0.1	4233 0.8 0.1
28IT56	26	16.74												9284 0.5 0.1	8294 0.6 0.1	7442 0.6 0.1	6703 0.7 0.1	6059 0.7 0.1	5493 0.8 0.1	4994 0.8 0.1
28IT60	28	18.04													9590 0.6 0.1	8613 0.6 0.2	7766 0.6 0.2	7027 0.7 0.2	6379 0.7 0.2	5807 0.8 0.2

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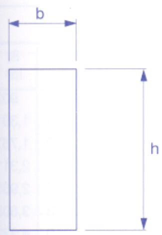
RECTANGULAR BEAMS

Normal Weight Concrete



f'_c = 5,000 psi
f_{pu} = 270,000 psi
½ in. diameter
low-relaxation strand

Key
3,344 — Safe superimposed service load, plf
0.4 — Estimated camber at erection, in.
0.1 — Estimated long-time camber, in.

Section Properties							
Designation	b in.	h in.	A in²	I in⁴	y_b in.	S in³	wt plf
12RB16	12	16	192	4,096	8.00	512	200
12RB20	12	20	240	8,000	10.00	800	250
12RB24	12	24	288	13,824	12.00	1,152	300
12RB28	12	28	336	21,952	14.00	1,568	350
12RB32	12	32	384	32,768	16.00	2,048	400
12RB36	12	36	432	46,656	18.00	2,592	450
16RB24	16	24	384	18,432	12.00	1,536	400
16RB28	16	28	448	29,269	14.00	2,091	467
16RB32	16	32	512	43,691	16.00	2,731	533
16RB36	16	36	576	62,208	18.00	3,456	600
16RB40	16	40	640	85,333	20.00	4,267	667

1. Check local area for availability of other sizes.

2. Safe loads shown include 50% superimposed dead load and 50% live load. 800 psi top tension has been allowed, therefore additional top reinforcement is required.

3. Safe loads can be significantly increased by use of structural composite topping.

Table of safe superimposed service load (plf) and cambers

Desig-nation	No. Strand	e	Span, ft																	
			16	18	20	22	24	26	28	30	32	34	36	38	40	42	44	46	48	50
12RB16	5	5.67	3344	2605	2075	1684	1386	1154	970											
			0.4	0.5	0.6	0.7	0.8	0.9	1.0											
			0.1	0.2	0.2	0.2	0.2	0.2	0.2											
12RB20	8	6.60	6101	4773	3823	3121	2585	2166	1833	1565	1345	1163	1010							
			0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2	1.3	1.4							
			0.1	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3							
12RB24	10	7.76	8884	6957	5578	4558	3782	3178	2699	2312	1996	1734	1514	1328	1170	1033				
			0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2	1.3	1.4	1.5	1.6				
			0.1	0.1	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.4	0.4				
12RB28	12	8.89		9502	7630	6245	5192	4372	3721	3197	2767	2411	2113	1861	1645	1460	1299	1159	1035	
				0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2	1.3	1.4	1.5	1.5	1.6	1.7	
				0.1	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	
12RB32	13	10.48				8238	6859	5785	4933	4246	3683	3217	2826	2495	2213	1970	1760	1576	1415	1272
						0.4	0.5	0.6	0.7	0.8	0.9	0.9	1.0	1.1	1.2	1.3	1.4	1.5	1.5	1.6
						0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.3	0.3
12RB36	15	11.64					8734	7376	6298	5428	4716	4126	3632	3214	2856	2549	2283	2050	1846	1666
							0.5	0.5	0.6	0.7	0.8	0.9	1.0	1.0	1.1	1.2	1.3	1.4	1.5	1.5
							0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.4	0.4	0.4
16RB24	13	7.86		9278	7439	6079	5044	4239	3600	3084	2662	2313	2020	1772	1560	1378	1220	1082	961	
				0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2	1.3	1.4	1.5	1.6	1.6	1.7	1.8	
				0.1	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.3	0.3	0.3	0.2	
16RB28	13	8.89			9022	7383	6137	5167	4397	3776	3267	2846	2493	2194	1939	1720	1530	1364	1218	1089
					0.4	0.4	0.5	0.6	0.6	0.7	0.8	0.9	1.0	1.0	1.1	1.2	1.2	1.3	1.3	1.3
					0.1	0.1	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	0.0
16RB32	18	10.29					9145	7713	6577	5661	4911	4289	3768	3327	2951	2627	2346	2101	1886	1697
							0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.1	1.2	1.3	1.4	1.5	1.6	1.7
							0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.4	0.4	0.4	0.4
16RB36	20	11.64						9834	8397	7237	6288	5502	4843	4285	3809	3399	3043	2733	2461	2221
								0.5	0.6	0.7	0.8	0.9	1.0	1.0	1.1	1.2	1.3	1.4	1.5	1.5
								0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.4	0.4	0.4
16RB40	22	13.00								9010	7839	6867	6054	5365	4777	4271	3832	3449	3113	2817
										0.6	0.7	0.8	0.9	1.0	1.0	1.1	1.2	1.3	1.4	1.4
										0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.4

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APPENDIX F – SJI K-Series Selection Table

STANDARD LOAD TABLE/OPEN WEB STEEL JOISTS, K-SERIES
Based on a Maximum Allowable Tensile Stress of 30 ksi

Joist Designation	8K1	10K1	12K1	12K3	12K5	14K1	14K3	14K4	14K6	16K2	16K3	16K4	16K5	16K6	16K7	16K9
Depth (in.)	8	10	12	12	12	14	14	14	14	16	16	16	16	16	16	16
Approx. Wt (lbs./ft.)	5.1	5.0	5.0	5.7	7.1	5.2	6.0	6.7	7.7	5.5	6.3	7.0	7.5	8.1	8.6	10.0
Span (ft.) ↓																
8	550 550															
9	550 550															
10	550 480	550 550														
11	532 377	550 542														
12	444 288	550 455	550 550	550 550	550 550											
13	377 225	479 363	550 510	550 510	550 510											
14	324 179	412 289	500 425	550 463	550 463	550 550	550 550	550 550	550 550							
15	281 145	358 234	434 344	543 428	550 434	511 475	550 507	550 507	550 507							
16	246 119	313 192	380 282	476 351	550 396	448 390	550 467	550 467	550 467	550 550	550 550	550 550	550 550	550 550	550 550	550 550
17		277 159	336 234	420 291	550 366	395 324	495 404	550 443	550 443	512 488	550 526	550 526	550 526	550 526	550 526	550 526
18		246 134	299 197	374 245	507 317	352 272	441 339	530 397	550 408	456 409	508 456	550 490	550 490	550 490	550 490	550 490
19		221 113	268 167	335 207	454 269	315 230	395 287	475 336	550 383	408 347	455 386	547 452	550 455	550 455	550 455	550 455
20		199 97	241 142	302 177	409 230	284 197	356 246	428 287	525 347	368 297	410 330	493 386	550 426	550 426	550 426	550 426
21			218 123	273 153	370 198	257 170	322 212	388 248	475 299	333 255	371 285	447 333	503 373	548 405	550 406	550 406
22			199 106	249 132	337 172	234 147	293 184	353 215	432 259	303 222	337 247	406 289	458 323	498 351	550 385	550 385
23			181 93	227 116	308 150	214 128	268 160	322 188	395 226	277 194	308 216	371 252	418 282	455 307	507 339	550 363
24			166 81	208 101	282 132	196 113	245 141	295 165	362 199	254 170	283 189	340 221	384 248	418 269	465 298	550 346
25						180 100	226 124	272 145	334 175	234 150	260 167	313 195	353 219	384 238	428 263	514 311
26						166 88	209 110	251 129	308 156	216 133	240 148	289 173	326 194	355 211	395 233	474 276
27						154 79	193 98	233 115	285 139	200 119	223 132	268 155	302 173	329 188	366 208	439 246
28						143 70	180 88	216 103	265 124	186 106	207 118	249 138	281 155	306 168	340 186	408 220
29										173 95	193 106	232 124	261 139	285 151	317 167	380 198
30										161 86	180 96	216 112	244 126	266 137	296 151	355 178
31										151 78	168 87	203 101	228 114	249 124	277 137	332 161
32										142 71	158 79	190 92	214 103	233 112	259 124	311 147

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APPENDIX G – References

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