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Hershey Academic Support Center 10/31/05

Technical Report 2 – Alternate Floor Systems

Executive Summary

This report is an analysis of possible alternate floor systems for The Hershey Academic Support Center found in Hershey, PA. After analyzing the existing system, feasible alternate systems were chosen and considered for the building's floor system. Advantages and disadvantages were contrived and studied to determine if each system was worth further investigation.

Existing Floor System – Composite Steel Beams and Girders
~Advantages: Quick Erection with Lightweight Construction
~Disadvantages: Needs Fireproofing and Possible Vibration Issues

Floor System #1 – Composite Steel Beams and Girders (No Camber)
~Advantages: Existing System Advantages and Easier to Fabricate
~Disadvantages: Existing System Disadvantages and a Possibly Higher Cost

Floor System #2 – Non-Composite Steel Beams and Girders
~Advantages: Cheaper System with a Shorter Construction Time
~Disadvantages: Increased System Weight and Possible Lateral Effects
~Further Investigation: YES

Floor System #3 – One-way Multiple Span Concrete Joists
~Advantages: Decreased Overall Depth and Less Vibration Issues
~Disadvantages: Longer Construction Time and a Heavier System
~Further Investigation: YES

Floor System #4 – Two-way Flat Slab with Drop Panels
~Advantages: No Fireproofing or Vibration Concerns
~Disadvantages: Thick Slab with Drops and Very Heavy Weight

Floor System #5 – Two-way Waffle Flat Slab
~Advantages: Very Thin Floor System with Little Vibration Effects
~Disadvantages: More Expensive System and Longer Construction Time
~Further Investigation: YES

Floor System #6 – Open Web Steel Joists
~Advantages: Lightweight System with a Faster Build Time
~Disadvantages: Increased Floor Depth with Difficult Fireproofing

General Introduction

The Hershey Academic Support Center is part of the Hershey Medical Center complex and is owned by The Pennsylvania State University. Constructed from March 1999 to August 2000, The Penn State Geisinger Health System was designed as the primary occupant, but was dissolved before the building was occupied. Currently the building is used for auxiliary purposes of the Hershey Medical Center and accommodates 680 people. The building itself can be considered in two sections, an East and a West wing. The wings are structurally identical with the only difference between them found in the center section. The building footprint encompasses a total area of 150,000 square feet. The total height of the building over 5 stories is measured as 56'-0" with the height to top of the roof including the Mechanical Penthouse being 69'-0". The building consists of a conventional structural steel system with composite beam floor framing and a precast concrete and glass facade. Moment connections placed at the columns as well as braced steel frames help to resist the wind and lateral loads throughout the building.

Dead Load (Office Floor):

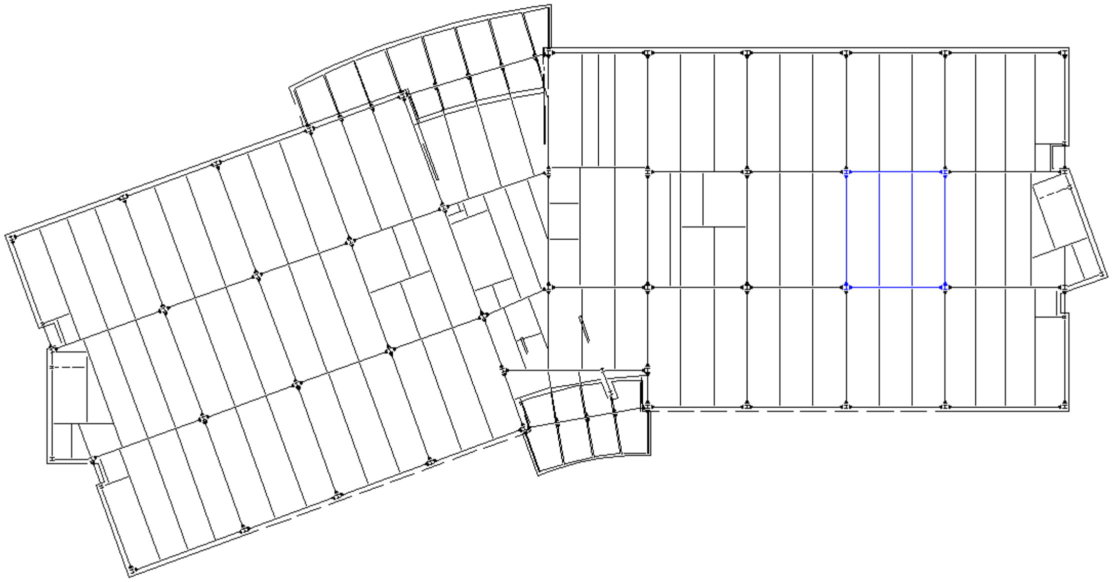
~Decking = 2 psf
~2.5" Lightweight Concrete = 40 psf
~Steel = 5 psf
~MEP = 10 psf
~Plenum Air System = 10 psf
~Finishing = 3 psf
~Total Office Dead Load = 70 psf

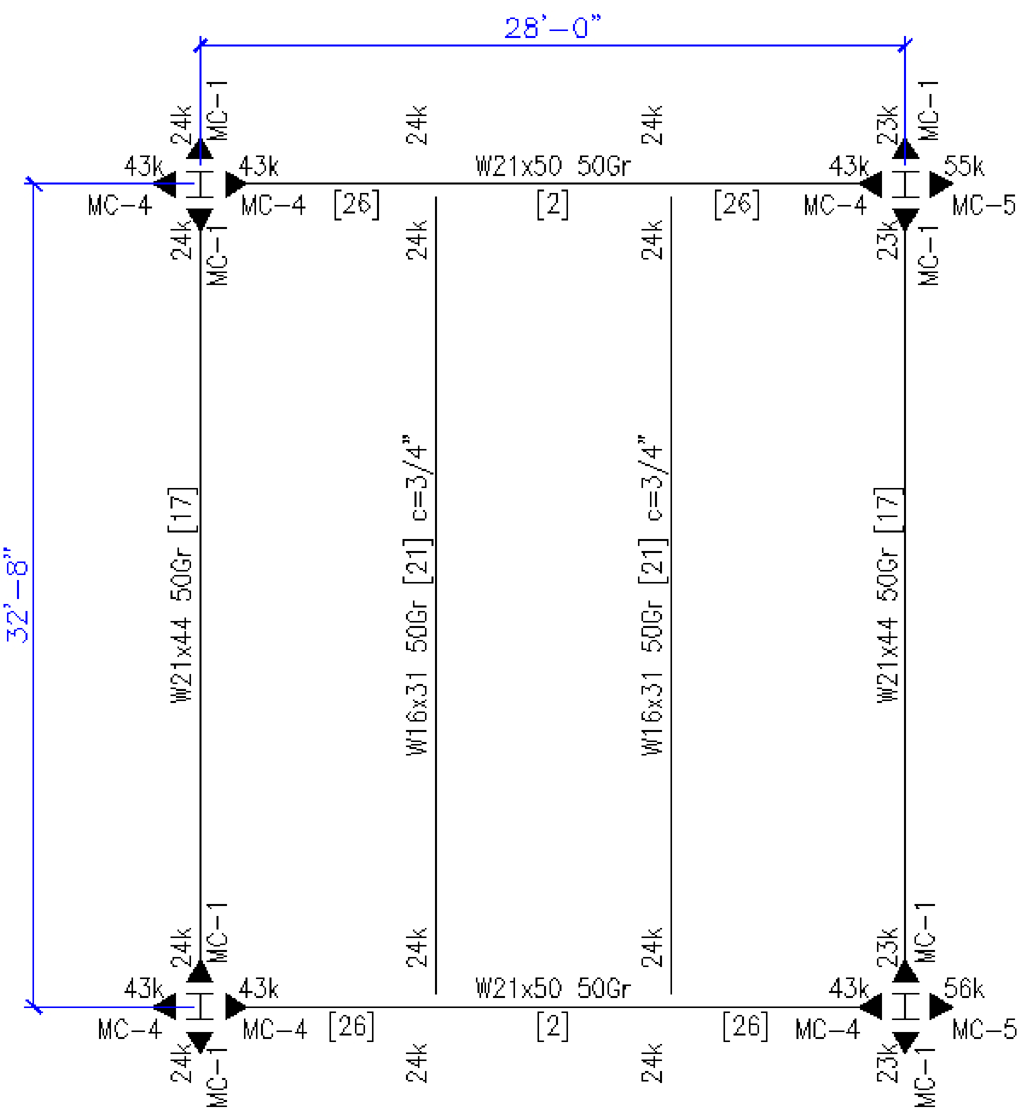
Live Load:

Main Floor = 100 psf (with corridors and partitions)

Existing Floor System – Composite Steel Beams and Girders

The floor system at the Hershey Academic Support Center utilizes a composite beam floor framing system with 3" 20 gage Vulcraft galvanized steel metal decking and 6x6 W1.4XW1.4 Welded Wire Fabric between the steel members and the concrete. The 2.5" Lightweight concrete along with the decking give an overall slab thickness of 5.5" and a total system depth at the girder of 26.5". To hold together the decking and concrete slab, 0.75" ø x 4.5" long headed steel studs were used. The most common connection used in a typical bay between the beams and columns is a L6 x 4 x 7/8 x 0'7" steel angle moment connection with 4 bolts to a beam and 2 bolts to a column. Each typical bay is 28' by 32'-8" and consists of W21x50 and W21x44 girders with W16x31 interior beams that have a ¾" camber. Material strength is given as 4000 psi for the concrete slab and Fy = 50 ksi ASTM A-572 steel in the beams and girders. Spray on cementitious fireproofing was used to meet the fire rating required for the building. The floor framing plan and a typical interior bay are shown below in blue.





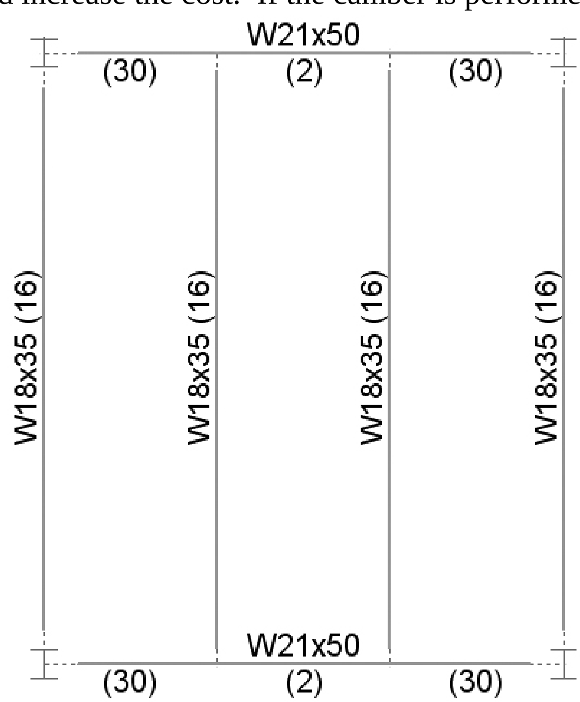
28'-0"
32'-8"
24k
MC-1
23k
43k
55k
56k
MC-4
MC-5
W21x50 50Gr
[26]
[2]
W21x44 50Gr [17]
W16x31 50Gr [21] c=3/4"

One inherent advantages of the current system are that it can be quickly erected and at a generally low cost. Another advantage of the existing system is that the floor depth is comparatively shallow and can accommodate most building height restrictions. The structure for this system is lightweight, allowing smaller column sizes as well as a smaller foundation. One disadvantage of the system is that it requires fireproofing to meet the 2 hour fire-rating necessary. Another disadvantage is the labor required to weld the shear studs, which can slow down time and be costly. Vibration issues may come into play with such a light system, but overall, this system works well for the Hershey Academic Support Center.

System #1 – Composite Steel Beams and Girders (No Camber)

Using my existing system and the RAM steel software, I changed the design to pin connections at the columns with no camber allowed on the beams. The spacing of 28' by 32'-8" was kept the same as well as the 4000 psi concrete strength and A-572 beam and girder steel. The input RAM data was a 2.5" Lightweight slab (115 pcf), 4.5" long 3/4"ø studs, Fy = 50 ksi, and Vulcraft 20 gage 3" steel decking. With the camber removed and the new connections, the beams changed from a W16x31 with ¾" camber to a W18x35 without camber. This new beam would be enough to support the pre-construction weight of the building without needing the ¾" camber previously used.

The new beam would make the floor system 2" deeper and 4 psf heavier, but would save the camber in the system. In terms of a cost comparison, the transportation of beams that are 16" is a lot easier than 18", so it would increase the cost. If the camber is performed in shop, it is not that expensive to have it done. The extra weight from the change in system would have to be factored into the foundation, but an increase of only 4 psf is not likely to make major changes. Overall, this system is still a valid option, but the existing system is probably more cost efficient.



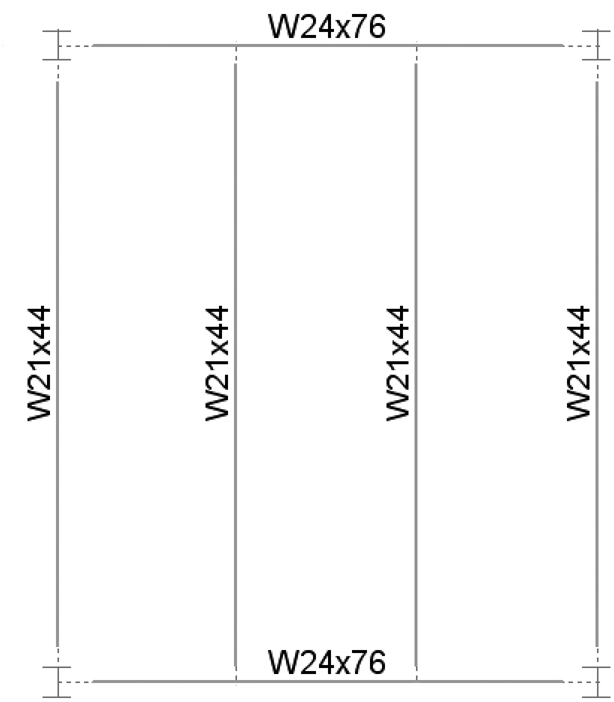
Beams:
W18x35
øMn = 249'k
Mu = 198.6'k

Girders:
W21x50
øMn = 413'k
Mu = 408.1'k

System #2 – Non-Composite Steel Beams and Girders

The next system used for the floor is a non-composite floor system that was analyzed for my building with RAM Steel. The new dead load on the floor system was recalculated to be 47 psf for the 28’ by 32’-8” bay. The decking stayed at Vulcraft 20 gage 3” galvanized steel decking at Fy = 50 ksi strength steel. The RAM results yielded larger girders and larger beams for each typical bay. The girder size increased to W24x76 in both directions and W21x44 interior beams.

This new system will increase the total weight in the building, so this might impact both the column sizes as well as the foundation. The depth of the overall system increased by 3” but the 2.5” of slab was removed so there should not be any problems with increased building height. The main advantage to this system is the cost saved on the labor and materials from the shear studs. Building time will also be decreased as a result of the lower labor necessary which is another advantage of this system. The loss of the composite system may have a negative effect on the lateral building system, but with all the moment connections in place, this effect would not be huge. Vibration control for this system would be better overall than the original composite system as well. All in all, this system is a viable choice but more in-depth studies would need to be done to see the total weight effects on the columns and the foundation drastically.



Beams:
W21x44
øMn = 358’k
Mu = 199.8’k

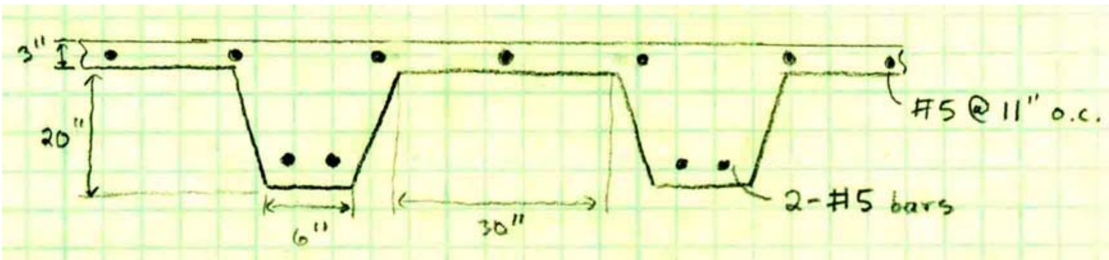
Girders:
W24x76
øMn = 750’k
Mu = 413.5’k

System #3 – One-way Multiple Span Concrete Joists

The next system I chose to analyze is a one-way multiple span joist system that was compiled from the 2002 CRSI Handbook Tables. The total load calculated for the system was 161.82 psf over a span of 33’. Using page 8-25 from CRSI, I obtained a design of 20” Deep Ribs with a 3” Top Slab giving my system a total depth of 23”. The system would contain 30” forms with 6” ribs @ 36” center to center for the interior span.

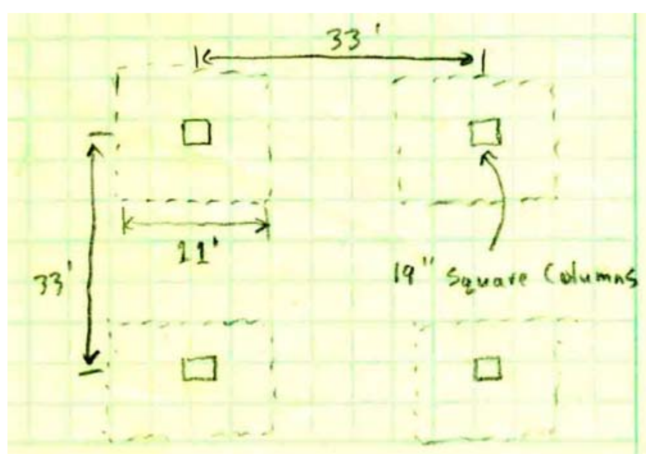
Reinforcement for the system is designed as #5@11" o.c. for the top bars and 2-#5 bottom bars. The total weight of the system is calculated to be 91 psf.

One of the first advantages to this system is that the overall depth is decreased by 3.5" from the original system, so there would be more ceiling space overall. There would also not be any spray on fireproofing required with this system as another cost saver. In terms of building time, this system would likely take longer than the existing system to construct. Another issue to consider is that the overall weight in this system is more than the existing system. This could cause an increase in column size as well as in the foundation, which must be considered.



System #4 – Two-way Flat Slab with Drop Panels

Next, I chose to analyze a two-way flat slab with drop panels for extra support over the 33' span. Again, I used the 2002 CRSI Handbook Tables to solve for the system. The total calculated load is the same as the one-way system at 161.82 psf. Assume that the drop panel weight is equally distributed and that there are square spans, panels, and column sizes to use the tables conservatively. For the loading and spans given, you get a slab design of 11" thick and 11' x 11' x 11" drop panels. The reason the system is designed like this is because the drop panel fits within the distance of the column strip and that punching shear is a huge contributor to the overall design. The reinforcement in the column strip is 15-#7 top bars and 11-#7 bottom bars with the middle strip reinforcement at 11-#7 top bars and 18-#5 top bars. The overall self weight of the system was calculated to be 156.91 psf.

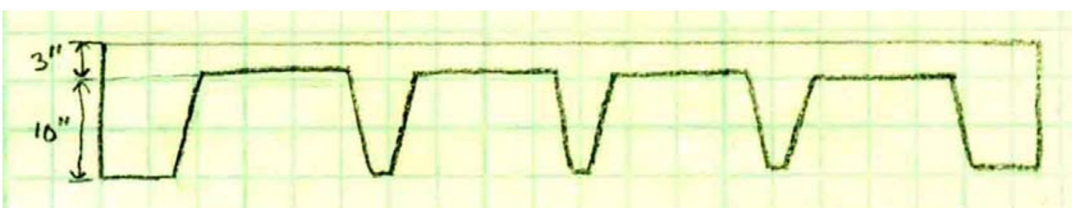


The main issue with this system is that the very thick slab and drop panels would cause a lot of concrete cost as well as slow down the overall building construction time. Another big issue is that my building bay is not perfectly square so the system could possibly get even bigger than it is now. The overall weight of the system is double the original weight, which does stop vibrational issues but will definitely affect the columns and foundation negatively. The system would save space on the overall thickness (22" instead of 26.5") and there is no fireproofing necessary, but these advantages do not outweigh the disadvantages for this system, so it will not be considered.

System #5 – Two-way Waffle Flat Slab

The following selected system was a two-way waffle flat slab system to see if a two-way system could be possible for my building. Consider a load of 161.82 psf, a 33' clear span, and square interior panels and use page 11-20 from the 2002 CRSI Handbook to find the appropriate system. The specifications selected were a 30" x 30" system with 6" voids and ribs at 36". The rib depth would be 10" and the slab depth at 3" for a total system depth of only 13". Basic reinforcement is required for the column strip and middle strip as well as a #3 Stirrup at 4" that extends over two modules. The column strip would need 5 ribs with 2-#7 bars per rib on the bottom as well as 30-#6 bars for the top interior. The middle strip requires 6 ribs with both #5 long bars and #6 short bars on the bottom. The top interior uses 10-#6 bars in this system. The overall weight of the system was calculated to be 94.98 psf.

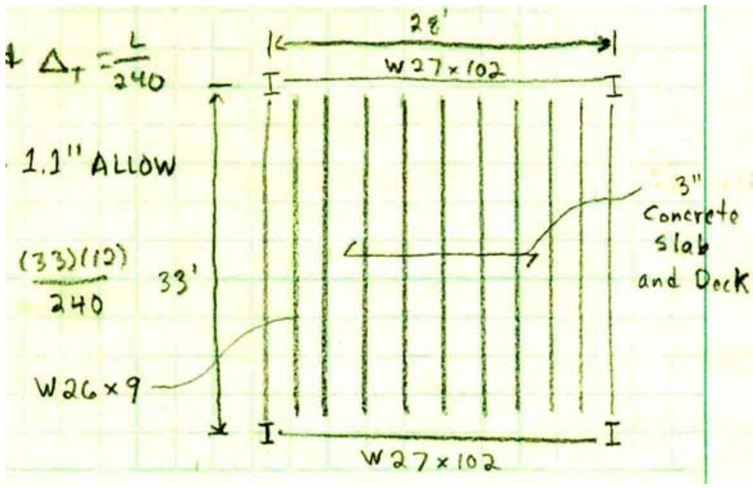
The biggest advantage of the two-way waffle system is that the overall floor depth is cut in half from 26.5" to 13". This reduction in thickness would allow more building flexibility in the ceiling area for electrical and mechanical utilities. This type of flooring system tends to be more expensive in both labor and formwork. This extra formwork also leads to a longer construction time overall. As with the other concrete systems, spray on fireproofing would not be required. The overall weight of the system provides extra vibration support at the cost of increased member sizes below. All in all, this could be a viable system depending on what factor was most important in design.



System #6 – Open Web Steel Joists

For my last system, I chose to model an open web steel joist system for the structure. This system would be modeled to obtain a 2-hour fire rating, so a 3" Normal weight concrete slab will be used. For a total superimposed dead load of 110.74 psf, the Vulcraft Decking manual suggests 3" .6C26 Decking as well as 4x4 W2.9 x W2.9 WWF. For a total slab thickness of 3.5" and a self-weight of 41 psf, the total uniform load is 469.22 plf and the total live load is 257.22 plf. The NCJ Joist Manual's Economy Table suggests the use of 26K9 Joists at 3' o.c. over the 33' span. For the Girders, LRFD specifies that for an 1112.96'k load, a W27x102 girder would be used.

One of the first advantages of this system is that the overall system is lightweight so it doesn't need big columns or foundations. Large girders are needed to support the systems so the overall thickness at the girders will increase from the existing system by 4". This system can be easily erected, so it has a short building construction time as a result. In terms of fireproofing, open web steel joist systems are known to be hard to fireproof with spray on fireproofing, so extra cost and labor can be accrued there. All in all, this system is workable for my building but provides no clear advantages to the current system, so it will not be considered.



Conclusions

Existing Floor System – Composite Steel Beams and Girders

Advantages
~ Quick Erection
~ Low Cost
~ Shallow Floor Depth
~ Lightweight

Disadvantages
~ Requires Fireproofing
~ Shear Stud Labor and Cost
~ Possible Vibration Issues

This system works well for the Hershey Academic Support Center. There are no major disadvantages in the system and quite a few advantages to be seen.

System #1 – Composite Steel Beams and Girders (No Camber)

Advantages
~ Same Initial Advantages
~ Easier to Fabricate

Disadvantages
~ Same Initial Disadvantages
~ Members Slightly Larger
~ Possible Higher Cost

This system is possible for the Hershey Academic Support Building but does not provide any major advantages to the previous system. The transportation of W18x35 beams would offset the savings from a W16x31 beam with camber. Consideration: NO

System #2 – Non-Composite Steel Beams and Girders

Advantages
~ Cheaper Cost
~ Faster Build Time
~ Lesser Vibration Effects

Disadvantages
~ Heavier System
~ Possible Lateral Effects

The Non-Composite system shares a lot in common with the original system in terms of advantages and disadvantages. While this is true, it does have a few extra advantages and disadvantages than the original system as seen above. The heavier system requirements will have to be checked to make sure the column size and foundation size aren't greatly affected, but overall, this would be a viable system to use. Consideration: YES

System #3 – One-way Multiple Span Concrete Joists

Advantages
~ Decreased Depth
~ No Required Fireproofing
~ Lower Vibration Issues

Disadvantages
~ Longer Construction Time
~ Heavier System

This One-way system could be considered as an option for the floor framing system in the Hershey Academic Support Center. This system contains one major advantage (decreased total depth) and one major disadvantage (heavier total weight), so it is worth the time to design and see how this system matches up. Consideration: YES

System #4 – Two-way Flat Slab with Drop Panels

Advantages
~ Lower Overall Thickness
~ No Fireproofing
~ Vibration Not an Issue

Disadvantages
~ Thick Slab and Panels
~ Floor System Not Square
~ Very Heavy System

This Two-way system should not be considered as a flooring option for my building. The disadvantages of this system are all major concerns especially that the system is twice as heavy as it was previously. This new system weight will definitely cause the columns and foundations to be increased in size, causing many cost issues. Consideration: NO

System #5 – Two-way Waffle Flat Slab

Advantages
~ Half Total Floor Depth
~ No Fireproofing
~ Small Vibration Effects

Disadvantages
~ More Expensive Overall
~ Longer Construction Time
~ Heavy System

The Waffle slab system plays off the one major advantage that the floor depth is very thin comparatively to other systems. This works particularly well for places that have a rigid height requirement or that need lots of space between floors. The Waffle system can be more expensive as well as longer to construct, which is why it is often overlooked in this kind of construction. All in all, this could be a viable system and should be analyzed further. Consideration: YES

System #6 – Open Web Steel Joists

Advantages
~ Lightweight
~ Short Construction Time

Disadvantages
~ Increased Floor Depth
~ Difficult Fireproofing

This system resembles the existing system in the Hershey Academic Support Center. The advantages and disadvantages outweigh each other over and that leaves nothing special to be found in this system. Since there are no major positive changes, there is no reason to complete further work on this problem. Consideration: NO

Appendix – Existing System

AE Senior Thesis | Spot Check | Technical Assignment 1

First Solve the loading for a typical Office Floor:

Dead Load = 70 psf (See Loading for specifics)

Live Load = 100 psf (Main Floor)

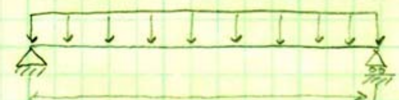
Use Live Load Reduction → $L = L_o\left(0.25 + \frac{15}{\sqrt{A_I}}\right)$ where

$A_{TOT} = (9.\bar{3}')(32.\bar{6}') = 304.\bar{8}\ ft^2$, $A_I = 2A_T = 2(304.\bar{8}) = 609.\bar{7}\ ft^2$

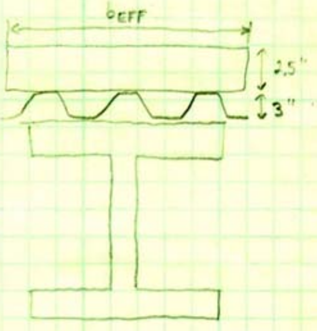
$L = (100)\left(0.25 + \frac{15}{\sqrt{609.\bar{7}}}\right) = 85.74$ psf

Using Load Factors: $1.2\,DL + 1.6\,LL = 1.2(70) + 1.6(85.74) = 221.84_{psf}$

$P_u = 221.84$ psf, $\omega_u = (9.\bar{3}')(221.84) = 2.06$ klf



Typical Beam Calculation



Lightweight (115pcf)

Given: $f'_c = 4$ksi & $f_y = 50$ksi

From Above, $\omega_u = 2.06$ klf

$M_u = \frac{\omega_u l^2}{8} = \frac{(2.06)(32.\bar{6}')^2}{8} = 274.78$ 'k

Assume $a = 1''$, $b_{EFF} = \begin{cases} l_n = 112'' \\ \frac{(32.\bar{6})}{4}(12) = \boxed{98''} \end{cases}$

$Y_2 = 5.5'' - \frac{a}{2} = 5''$

Use LRFD Table 5-14, Try W18×40 where $\phi_b M_p = 294$'k

PNA @7 for $Y_2 = 5''$ → 400 so $\phi_b M_p = 400$'k & $\Sigma Q_n = 148$k

$\Sigma Q_n = 0.85 f'_c b a \rightarrow a = \frac{\Sigma Q_n}{0.85 f'_c b} = \frac{148}{0.85(4)(98)} = 0.444$

$Y_2 = 5.5'' - \frac{0.444}{2} = 5.28'' \rightarrow \phi_b M_N = 403\text{'k}$ by interpolation

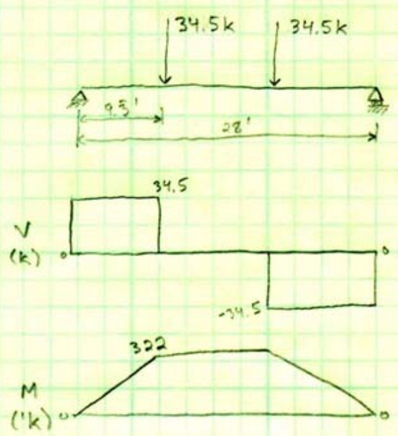
From the Structural notes, Minimum stud capacity is 9.7k

$\frac{\Sigma Q_n}{\text{stud capacity}} = \frac{148}{9.7} = 15.26 \rightarrow$ Use 32 shear studs

Overall Design W18×40 with 32 shear studs

The beam specified in the drawings is W16×31. I believe my beam is larger because an older code was used for factoring and there is a perscribed camber of $c = \frac{3}{4}''$ for the given design.

Typical Girder Calculation



$P = \frac{\omega_u l}{2} = \frac{(2.06)(32.\bar{6})}{2} = 33.6\text{k}$

$M_u = (33.6)(9.\bar{3}) = 313.6\text{'k}$

Assume $a = 1''$, $b_{EFF} = \begin{cases} l_n = 112'' \\ \frac{(28)}{4}(12) = \boxed{84''} \end{cases}$

$Y_2 = 5.5'' - \frac{a}{2} = 5''$, LRFD Table 5-14

Try W21×44 where $\phi_b M_p = 358\text{'k}$

$A_s = 13.0\text{ in}^2$, $\Sigma Q_n = A_s f_y = (50)(13) = 650\text{k}$

Find $a = \frac{\Sigma Q_n}{0.85 f'_c b} = \frac{650}{0.85(4)(84)} = 2.28$, $Y_2 = 5.5'' - \frac{2.28}{2} = 4.36''$

Appendix – Existing System

For PNA @ 7 + $Y_2 = 4.36"$ → $\phi_b M_p = 481'k$ by interpolation

For shear studs: $\Sigma Q_n = 163k$ & capacity is given as 33k

$$\frac{\Sigma Q_n}{\text{stud capacity}} = \frac{163}{33} = 4.9 \rightarrow \text{Use 10 shear studs}$$

Overall Design W21×44 with 10 studs

The girder specified in the drawings is given as W21×50. The design I proposed is close in size and the slight variance can be attributed from the beam load being slightly different.

Typical Column Design

Dead Load = $70psf + \underset{\text{self weight}}{5psf} = 75psf$ (Office Design)

Live Load = 100psf (Main Floor)

Use Live Load Reduction → $L = L_o\left(0.25 + \frac{15}{\sqrt{A_I}}\right)$ where

Column Tributary Area = $(28')(32.\bar{6}') = 914.\bar{6}\ ft^2$

$A_T = (4\ \text{floors})(914.\bar{6}) = 3658.\bar{6}\ ft^2$, $A_I = 4A_T = 4(3658.\bar{6}) = 14{,}634.\bar{6}\ ft^2$

Reduction Factor = $\left(0.25 + \frac{15}{\sqrt{14{,}634.\bar{6}}}\right) = 0.374 \not> 0.4$ for multiple story buildings, $L = (100)(0.4) = 40psf$, Factor the loading

$1.2DL + 1.6LL = 1.2(75) + 1.6(40) = 154psf$, $P_u = 154\ psf$,

$P_{Floor} = (154)(3658.\bar{6}) = 563.43K$, $\omega_u = 2.06 + \underset{\text{Self Weight}}{(0.05)}(1.2) = 2.12\ klf$

$M_u = \frac{\omega l^2}{12} = \frac{(2.12)(28)^2}{12} = 138.51'k$, Calculate for the wall

Appendix – Existing System

Wall Load = (28')(55') = 1540 ft^2 where wall height = 69'-14' = 55'

Assume Wall Load of 30 psf, $P_{wall} = (1540)(30) = 46.2\text{k}$

$P_u = P_{floors} + P_{wall} = 563.43\text{k} + 42.6\text{k} = 606.03\text{k}$

Using $M_u = 138.51'\text{k}$ & $P_u = 606.03\text{k}$, Try W14 shape

$P_{eff} = P_u + \alpha M_u$ where $\alpha = \frac{24}{d} = \frac{24}{14} = 1.71$, $P_{EFF} = 606.03 + (1.71)(138.51)$

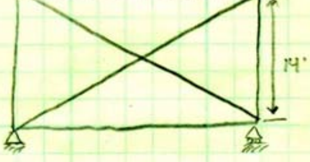
$P_{EFF} = 842.88\text{k}$ Assume Pinned connection, $K = 1$ (conservative)

$KL = 14'$, From LRFD Table 4-2, Try W14x90

969 k > 842.88k ALLOW, Overall Design is W14x90

The column specified in the drawings is listed as a W14x120. The differences can be attributed possibily to not considering a heavy enough exterior wall load or a smaller live load reduction used by the designer.

Lateral Force Check



Use the Given member, $L6 \times 6 \times \frac{5}{16}$

$A_g = \frac{425}{50} = 8.5\text{in}^2$, Use LRFD Table 1-7

Choose $L6 \times 6 \times \frac{7}{8} \rightarrow A_g = 9.75\text{in}^2$

The designed member is not adequate due to the large calculated Seismic Force (Critical Force)

Appendix – Alternate System

AE Senior Thesis | STANDARD ONE-WAY JOISTS | Tech Assignment #2

Use CRSI Handbook 2002 to design the system

Find the loading using the CRSI factors

Superimposed Dead Load = 25 psf (See Loading for specifics)

Live Load = 100 psf (Main Floor)

Use Live Load Reduction → $L = L_o \left(0.25 + \frac{15}{\sqrt{A_I}}\right)$ where

$A_{TOT} = (28')(32.\bar{6}') = 914.\bar{6}\ ft^2$, For One-way slabs $A_{TOT} = A_I$

$L = (100)\left(0.25 + \frac{15}{\sqrt{914.\bar{6}}}\right) = 74.60\ psf$

CRSI Load Factors: $1.4DL + 1.7LL = 1.4(25) + 1.7(74.6) = 161.82\ psf$

The One-Way Joists - Multiple Span System will span over the long direction giving a clear span of about 33'

For an Interior Span, look at page 8-25 from CRSI

Try 20" Deep Rib with 3" Top Slab for a 23" Total Depth

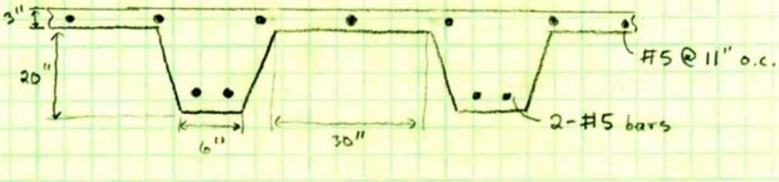
For 30" forms & 6" rib @ 36" c-c, 33' Interior span

Allowable Load = 166psf > 161.82psf ALLOW

For Reinforcement, Use #5 @ 11" o.c Top Bars & #5, #5 Bottom Bars

Check Deflection → $\frac{\ell_n}{21} = \frac{(33)(12)}{21} = 18.86" < 23"$ ALLOW

From Table 8-1 in CRSI, Total weight = 91psf



Appendix – Alternate System

AE Senior Thesis | Two-way Flat Slab with Drop Panels | Tech Assignment #2

Use CRSI Handbook 2002 to design the system

Using CRSI Load Factors: $P_u = 161.82$ (see One-way System)

The Two-way Flat Slab with Drop Panels will span over the long direction giving a clear span of about 33'

For Square Interior Panels, look at page 10-27 from CRSI

Assume Square Spans, Panels, and Column Sizes for the bay

Also assume Drop Panel Weight is equally distributed

Try a Total slab thickness of 11" with 11' x 11' x 11" Drop Panels and 19" Square Columns

Allowable Load = 200 psf > 161.82 psf ALLOW

Column Strip Reinforcement

Top: 15-#7 bars & Bottom: 11-#7 bars

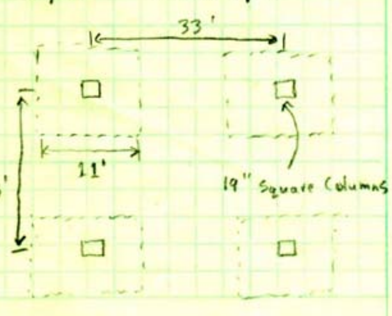
Middle Strip Reinforcement

Top: 11-#7 bars & Bottom: 18-#5 bars

Find Self Weight: $(1.019\ \frac{cf}{sf})(150\ pcf) + 4.06\ psf = 156.91\ psf$

Check Deflection $\rightarrow \frac{\ell}{36}$ for interior slab > 4", $\frac{\ell}{36} = \frac{(33)(12)}{36} = 11" \leq 11"$

ALLOW



Appendix – Alternate System

AE Senior Thesis	TWO-WAY WAFFLE FLAT SLAB SYSTEM	Tech Assignment #2

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

Use CRSI Handbook 2002 to design the system

Using CRSI Load Factors: $P_u = 161.82$ (See One-Way System)

The Two-Way Waffle Flat Slab System will span over the long direction giving a clear span of about 33'

For Square Interior Panels, look at page 11-20 from CRSI

Try a 30"x30" System with 6" Voids and Ribs @ 36"

Rib Depth = 10" & slab Depth = 3" → Total Depth = 13"

D = 12.5 with the Rib on the Column Line, Steel = 3.63 psf

Allowable Load = 200 psf > 161.82 psf ALLOW

Column Strip Reinforcement

5 Ribs with 2-#7 bars per rib on the bottom

30-#6 bars for the top interior

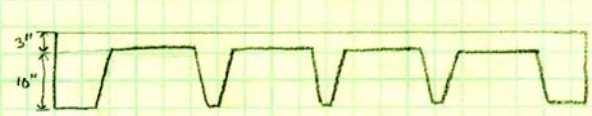
Middle Strip Reinforcement

6 Ribs with #5 Long Bars & #6 Short bars on the bottom

10-#6 bars for the top interior

Waffle Slab needs a #3 Stirrup @4" that extends two modules

Find Self weight: $(0.609\ CF/SF)(150\ pcf) + 3.63\ psf = 94.98\ psf$



Appendix – Alternate System

AE Senior Thesis | OPEN WEB STEEL JOISTS | Tech Assignment #2

For this system, a 2-hour fire rating will be obtained by increasing the concrete to 3" and normal weight

Superimosed Dead Load = 25 psf (See Loading for Specifics)

Reduced Live Load = 85.74 psf (See Existing Calculations)

For Total Load = 110.74 psf, Use 3" Vulcraft .6C26 Decking and use 4x4 W2.9 x W2.9 WWF

This system will use a 3.5" Slab with a self-weight of 41psf

Find the Joists using 3' o.c. for a 33' span

$\omega_{DEAD} = (25+41)(3') + 14\text{plf}$ self weight $= 212\text{ plf}$

$\omega_{LIVE} = (85.74)(3') = 257.22\text{plf}$, $\omega_{TOTAL} = 469.22\text{plf}$

From the NCJ Joist Manual, Use 26K9 Joists

Check: Total → 488 plf > 469.22 plf & Live 370 plf > 257.22 plf ALLOW

Find the Girder with a tributary width of 33'

$\omega_{DEAD} = (25+67)(33') = 3.04\text{ klf}$ $\omega_{LIVE} = (85.74)(33') = 2.83\text{ klf}$

$\omega_{TOTAL} = 1.2(3.04) + 1.6(2.83) = 8.176\text{plf}$, $M_u = \frac{(8.176)(33')^2}{8} = 1112.96\text{'k}$

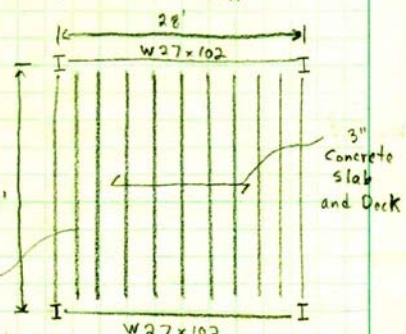
Use W27×102 where $\phi M_N = 1140\text{'k}$ and $I = 3620\text{ in}^4$

Check deflections for $\Delta_L = \frac{L}{360}$ & $\Delta_T = \frac{L}{240}$

$$\Delta_L = \frac{5(2830)(33)^4}{384(29000)(3620)} = 0.42" < \frac{(33)(12)}{360} = 1.1" \text{ ALLOW}$$

$$\Delta_T = \frac{(0.42)(3040+2830+102)}{2830} = 0.88" < \frac{(33)(12)}{240}$$

0.88" < 1.65" ALLOW



2002 CRSI Handbook Design Table

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

20" Deep Rib + 3.0" Top Slab = 23.0" Total Depth

	END SPAN						INTERIOR SPAN					
TOP BARS Size @	# 4 10	# 4 8	# 4 7	# 5 9	# 6 11	End Span Defl. Coeff.	# 4 9	# 5 11	# 5 9.5	# 5 8	# 6 9	Int. Span Defl. Coeff.
BOTTOM BARS # #	# 5 # 5	# 5 # 6	# 6 # 6	# 6 # 7	# 7 # 7		# 4 # 5	# 5 # 5	# 5 # 6	# 6 # 6	# 6 # 7	
Steel (psf)	.85	1.04	1.23	1.44	1.69	(3)	.93	1.18	1.38	1.66	2.01	(3)
CLEAR SPAN												
30'-0"	117	167	218	278	334*	9.752	165	228	301	375	395*	6.001
	0	0	0	0	340		0	0	0	0	458*	
31'-0"	101	148	196	253	310	11.119	147	206	274	343	374*	6.842
	0	0	0	0	0		0	0	0	0	426	
32'-0"	87	131	176	229	283	12.625	130	185	249	314	355*	7.769
	0	0	0	0	0		0	0	0	0	391	
33'-0"	74	16	158	208	259	14.278	114	166	226	287	337*	8.787
	0	0	0	0	0		0	0	0	0	360	
34'-0"	63	102	141	168	236	16.089	100	149	206	263	320*	9.901
	0	0	0	0	0		0	0	0	0	332	
35'-0"	52	89	126	171	216	18.067	88	134	187	241	305*	11.118
	0	0	0	0	0		0	0	0	0	306	
36'-0"	42	77	112	154	197	20.222	76	119	170	221	282	12.445
	0	0	0	0	0		0	0	0	0	0	
37'-0"		66	99	139	180	22.565	65	106	154	202	261	13.886
		0	0	0	0		0	0	0	0	0	
38'-0"		56	87	125	164	25.105	55	94	139	185	240	15.449
		0	0	0	0		0	0	0	0	0	
39'-0"		47	76	112	149	27.853	46	83	126	169	222	17.141
		0	0	0	0		0	0	0	0	0	
40'-0"			66	101	135	30.822		72	113	155	204	18.967
			0	0	0			0	0	0	0	
41'-0"			57	90	123	34.022		63	102	141	188	20.936
			0	0	0			0	0	0	0	
42'-0"			48	79	111	37.464		54	91	128	174	23.055
			0	0	0			0	0	0	0	
43'-0"			40	70	100	41.162		45	81	117	160	25.330
			0	0	0			0	0	0	0	

(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 .60 CF/SF) (4)

NEGATIVE MOMENT												
STEEL AREA (SQ. IN.)	.72	.90	1.03	1.24	1.44		.80	1.01	1.17	1.39	1.76	
STEEL % (UNIFORM)	.42	.53	.61	.73	.85		.47	.60	.69	.82	1.04	
(TAPERED)	.26	.32	.37	.45	.52		.29	.37	.42	.50	.64	
EFF. DEPTH, IN.	21.8	21.8	21.8	21.7	21.6		21.8	21.7	21.7	21.7	21.6	
−ICR/IGR	.149	.179	.199	.228	.254		.162	.195	.219	.250	.295	
POSITIVE MOMENT												
STEEL AREA (SQ. IN.)	.62	.75	.88	1.04	1.20		.51	.62	.75	.88	1.04	
STEEL %	.08	.10	.11	.13	.15		.07	.08	.10	.11	.13	
EFF. DEPTH, IN.	21.7	21.6	21.6	21.6	21.6		21.7	21.7	21.6	21.6	21.6	
+ICR/IGR	.160	.190	.220	.255	.290		.134	.160	.190	.220	.255	

STANDARD ONE-WAY JOISTS (1) MULTIPLE SPANS — 30" Forms + 7" Rib @ 37" c.-c. (2)

FACTORED USABLE SUPERIMPOSED LOAD (PSF) — f'_c = 4,000 psi, f_y = 60,000 psi

20" Deep Rib + 3.0" Top Slab = 23.0" Total Depth

	END SPAN						INTERIOR SPAN					
TOP BARS Size @	# 5 12	# 5 11	# 5 9.5	# 5 8	# 6 9.5	End Span Defl. Coeff.	# 5 12	# 5 9.5	# 5 8	# 5 7	# 6 8	Int. Span Defl. Coeff.
BOTTOM BARS # #	# 5 # 6	# 6 # 6	# 6 # 7	# 7 # 7	# 7 # 8		# 5 # 5	# 5 # 6	# 6 # 6	# 6 # 7	# 7 # 7	
Steel (psf)	1.03	1.20	1.40	1.63	1.89	(3)	1.11	1.36	1.63	1.88	2.29	(3)
CLEAR SPAN												
30'-0"	152	201	261	321	376*	10.023	212	282	354	433*	440*	6.168
	0	0	0	0	390		0	0	0	440	510*	
31'-0"	134	180	235	292	354*	11.428	190	256	323	404	417*	7.033
	0	0	0	0	357		0	0	0	0	481*	
32'-0"	117	161	213	265	326	12.975	170	232	295	371	396*	7.985
	0	0	0	0	0		0	0	0	0	447	
33'-0"	102	143	192	241	299	14.675	151	210	269	340	376*	9.031
	0	0	0	0	0		0	0	0	0	413	
34'-0"	88	127	173	220	274	16.536	135	190	246	313	358*	10.176
	0	0	0	0	0		0	0	0	0	381	
35'-0"	76	112	156	200	251	18.569	120	172	224	288	341*	11.427
	0	0	0	0	0		0	0	0	0	352	
36'-0"	64	99	140	181	230	20.784	106	155	205	265	325	12.790
	0	0	0	0	0		0	0	0	0	0	
37'-0"	54	86	125	165	210	23.191	93	139	187	243	301	14.272
	0	0	0	0	0		0	0	0	0	0	
38'-0"	44	75	112	149	192	25.802	81	125	170	224	278	15.878
	0	0	0	0	0		0	0	0	0	0	
39'-0"		64	99	135	176	28.627	70	112	154	205	257	17.617
		0	0	0	0		0	0	0	0	0	
40'-0"		54	88	121	160	31.678	60	100	140	189	238	19.494
		0	0	0	0		0	0	0	0	0	
41'-0"		45	77	109	146	34.967	51	89	127	173	220	21.518
		0	0	0	0		0	0	0	0	0	
42'-0"			67	97	133	38.505	42	78	115	159	203	23.695
			0	0	0		0	0	0	0	0	
43'-0"			58	87	121	42.305		68	103	145	188	26.034
			0	0	0			0	0	0	0	

(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 .64 CF/SF) (4)

NEGATIVE MOMENT												
STEEL AREA (SQ. IN.)	.96	1.04	1.21	1.43	1.71		.96	1.21	1.43	1.64	2.04	
STEEL % (UNIFORM)	.50	.55	.63	.75	.90		.50	.63	.75	.86	1.07	
(TAPERED)	.32	.35	.40	.48	.57		.32	.40	.48	.55	.68	
EFF. DEPTH, IN.	21.7	21.7	21.7	21.7	21.6		21.7	21.7	21.7	21.7	21.6	
−ICR/IGR	.174	.186	.209	.239	.272		.174	.209	.239	.265	.309	
POSITIVE MOMENT												
STEEL AREA (SQ. IN.)	.75	.88	1.04	1.20	1.39		.62	.75	.88	1.04	1.20	
STEEL %	.09	.11	.13	.15	.17		.08	.09	.11	.13	.15	
EFF. DEPTH, IN.	21.6	21.6	21.6	21.6	21.5		21.7	21.6	21.6	21.6	21.6	
+ICR/IGR	.173	.200	.231	.263	.298		.146	.173	.200	.231	.263	

2002 CRSI Handbook Design Table

f'_c = 4,000 psi
Grade 60 Bars

FLAT SLAB SYSTEM
SQUARE EDGE PANEL — With Drop Panels — No Beams

SQUARE INTERIOR PANEL
With Drop Panels(2)
No Beams

SPAN c-c $\ell_1 = \ell_2$ (ft)	Factored Superimposed Load (psf)	Square Drop Panel Depth (in.)	Square Drop Panel Width (ft)	(3) Square Column Size (in.)	(3) Square Column γ_f	Edge Panel Reinforcing Bars (E. W.) Column Strip (1) Top Ext. +	Edge Panel Column Strip (1) Bottom	Edge Panel Column Strip (1) Top Int.	Edge Panel Middle Strip Bottom	Edge Panel Middle Strip Top Int.	Edge Panel Total Steel (psf)	Moments Edge (−) (ft-k)	Moments Bot. (+) (ft-k)	Moments Int. (−) (ft-k)	Interior Panel Factored Superimposed Load (psf)	(3) Square Column Size (in.)	Interior Panel Reinforcing Bars (E. W.) Column Strip Top	Interior Column Strip Bottom	Interior Middle Strip Top	Interior Middle Strip Bottom	Interior Total Steel (psf)	Concrete (cu. ft / sq. ft)
h = 11 in. = TOTAL SLAB DEPTH BETWEEN DROP PANELS															h = 11 in. = TOTAL SLAB DEPTH BETWEEN DROP PANELS							
28	100	7.00	9.33	12	0.761	13-#5 2	19-#5	18-#5	13-#5	11-#5	2.77	196.6	393.2	529.4	100	12	17-#5	13-#5	11-#5	11-#5	2.53	0.981
28	200	7.00	9.33	16	0.777	13-#5 4	10-#8	23-#5	16-#5	10-#6	3.42	253.9	507.8	683.6	200	19	15-#6	16-#5	13-#5	11-#5	2.98	0.981
28	300	9.00	9.33	18	0.643	13-#5 1	16-#7	18-#6	14-#6	12-#6	4.07	313.6	627.2	844.3	300	22	23-#5	14-#6	15-#5	13-#5	3.51	1.000
28	400	11.00	9.33	20	0.634	14-#5 1	12-#9	27-#5	10-#8	14-#6	4.94	372.5	744.9	1002.8	400	24	13-#7	17-#6	10-#7	11-#6	4.23	1.019
28	500	11.00	11.20	22	0.696	16-#5 3	14-#9	13-#8	12-#8	10-#8	5.76	430.5	861.0	1159.1	500	27	15-#7	12-#8	11-#7	18-#5	4.91	1.063
29	100	7.00	9.67	12	0.797	13-#5 3	11-#7	14-#6	10-#6	12-#5	2.94	219.0	438.0	589.7	100	12	13-#6	14-#5	12-#5	12-#5	2.64	0.981
29	200	9.00	9.67	16	0.688	13-#5 3	15-#7	23-#5	10-#7	15-#5	3.64	283.9	567.9	764.5	200	19	15-#6	18-#5	10-#6	12-#5	3.08	1.000
29	300	9.00	9.67	18	0.776	14-#5 5	14-#8	15-#7	12-#7	10-#7	4.52	350.0	699.9	942.2	300	22	26-#5	12-#7	12-#6	15-#5	3.91	1.000
29	400	11.00	11.60	21	0.698	15-#5 4	17-#8	16-#7	11-#8	12-#7	5.28	416.4	832.8	1121.1	400	25	15-#7	11-#8	11-#7	18-#5	4.58	1.063
29	500	11.00	11.60	23	0.740	17-#5 4	16-#9	14-#8	13-#8	11-#8	6.17	479.4	958.9	1290.8	500	27	13-#8	10-#9	10-#8	20-#5	5.28	1.063
30	100	9.00	10.00	12	0.710	14-#5 1	17-#6	14-#6	11-#6	13-#5	3.03	244.0	488.0	656.9	100	12	18-#5	11-#6	12-#5	12-#5	2.63	1.000
30	200	9.00	10.00	16	0.751	14-#5 4	16-#7	18-#6	20-#5	12-#6	3.81	315.5	631.0	849.4	200	19	17-#6	20-#5	11-#6	13-#5	3.30	1.000
30	300	11.00	10.00	19	0.638	14-#5 3	13-#9	15-#7	10-#8	11-#7	4.79	388.7	777.3	1046.4	300	22	26-#5	10-#8	10-#7	16-#5	4.04	1.019
30	400	11.00	12.00	21	0.747	17-#5 4	19-#8	14-#8	10-#9	18-#6	5.72	463.2	926.5	1247.2	400	25	22-#6	10-#9	12-#7	14-#6	4.93	1.063
30	500	11.00	12.00	26	0.735	19-#5 5	18-#9	16-#8	14-#8	12-#8	6.64	528.7	1057.4	1423.4	500	27	15-#8	14-#8	11-#8	12-#7	5.75	1.063
31	100	9.00	10.33	12	0.767	14-#5 3	11-#8	16-#6	12-#6	14-#5	3.20	269.8	539.7	726.5	100	12	20-#5	17-#5	13-#5	12-#5	2.69	1.000
31	200	11.00	10.33	16	0.639	14-#5 2	14-#8	18-#6	12-#7	19-#5	4.08	350.2	700.4	942.8	200	19	17-#6	22-#5	12-#6	15-#5	3.46	1.019
31	300	11.00	10.33	19	0.693	16-#5 3	14-#9	17-#7	12-#8	13-#7	5.21	430.5	861.1	1159.2	300	22	15-#7	15-#7	15-#6	18-#5	4.39	1.019
31	400	11.00	12.40	24	0.739	18-#5 6	17-#9	15-#8	11-#9	12-#8	6.20	508.9	1017.8	1370.1	400	25	14-#8	11-#9	18-#6	12-#7	5.35	1.063
31	500	11.00	12.40	29	0.737	15-#6 4	20-#9	14-#9	13-#9	13-#8	7.23	577.8	1155.5	1555.5	500	29	16-#8	13-#9	12-#8	13-#7	6.19	1.063
32	100	9.00	10.67	12	0.811	15-#5 5	15-#7	17-#6	19-#5	11-#6	3.38	297.4	594.9	800.8	100	12	16-#6	19-#5	15-#5	13-#5	2.93	1.000
32	200	11.00	10.67	16	0.688	15-#5 3	20-#7	15-#7	13-#7	11-#7	4.34	386.4	772.9	1040.4	200	19	26-#5	13-#7	19-#5	16-#5	3.70	1.019
32	300	11.00	10.67	20	0.756	17-#5 6	16-#9	18-#7	16-#7	18-#6	5.51	474.0	948.0	1276.2	300	22	17-#7	22-#6	13-#7	20-#5	4.70	1.019
32	400	11.00	12.80	27	0.694	20-#5 4	23-#8	17-#8	12-#9	16-#7	6.57	557.5	1115.0	1500.9	400	25	16-#8	12-#9	12-#8	13-#7	5.81	1.063
33	100	11.00	11.00	12	0.753	15-#5 4	13-#8	17-#6	11-#7	13-#6	3.61	328.1	656.2	883.4	100	12	16-#6	11-#7	16-#5	14-#5	3.07	1.019
33	200	11.00	11.00	16	0.765	15-#5 5	17-#8	22-#6	11-#8	12-#7	4.68	425.1	850.1	1144.4	200	19	15-#7	11-#8	11-#7	18-#5	4.06	1.019
33	300	11.00	11.00	23	0.721	19-#5 5	17-#9	16-#8	11-#9	12-#8	6.04	517.4	1034.9	1393.1	300	22	15-#8	11-#9	11-#8	12-#7	5.30	1.019
33	400	11.00	13.20	30	0.689	22-#5 5	21-#9	18-#8	13-#9	11-#9	7.02	604.0	1207.9	1626.1	400	28	17-#8	13-#9	16-#7	11-#8	6.08	1.063

NOTES: (1) 50 percent of these bars may be placed in the middle third of column strip. (2) Drop panels same size as for edge panels. (3) Same column size above and below slab.

2002 CRSI Handbook Design Table

WAFFLE FLAT SLAB SYSTEM 30″ X 30″ Voids: 6″ Ribs @ 36″

f'_c = 4,000 psi
Grade 60 Bars

SQUARE EDGE PANELS

Total Depth = 13 in. Rib Depth = 10 in. Total Slab Depth = 3 in.

Span c.-c. Columns $\ell_1 = \ell_2$ (ft)	Factored Super-imposed Load (psf)	(1) Steel (psf)	Square Edge Column: $c_1 = c_2$ (in.)	Square Edge Column: γ_f	Square Edge Column: (2) Stirrups	Column Strip: Top Edge No.-size +	Column Strip: Bottom No. Ribs	Column Strip: Bottom Bars per Rib	Column Strip: Top Interior No.-size	Middle Strip: Bottom No. Ribs	Middle Strip: Bottom Long Bars	Middle Strip: Bottom Short Bars	Middle Strip: Top Interior No.-size	Moments: −M Edge (ft-k)	Moments: +M Bot. (ft-k)	Moments: −M Int. (ft-k)
18'- 0" D= 6.500 RIB ON COLUMN LINE 0.597 CF/SF	50	1.86	12	0.664		13-#5+ 0	3	2-#4	13-#5	3	#4	#4	5-#5	28	56	76
	100	1.86	12	0.687		13-#5+ 0	3	2-#4	13-#5	3	#4	#4	5-#5	37	77	98
	150	1.92	12	0.711		13-#5+ 0	3	1-#4 and 1-#5	13-#5	3	#4	#4	5-#5	45	106	121
	200	1.98	12	0.735		13-#5+ 0	3	2-#5	13-#5	3	#4	#4	5-#5	53	132	144
	300	2.16	12	0.782		13-#5+ 0	3	1-#5 and 1-#6	13-#5	3	#4	#5	5-#5	70	185	189
	400	2.61	12	0.830		13-#5+ 0	3	1-#6 and 1-#7	13-#5	3	#5	#6	5-#5	87	236	235
21'- 0" D= 9.500 RIB NOT ON COLUMN LINE 0.637 CF/SF	50	1.84	12	0.735		15-#5+ 0	4	2-#4	15-#5	3	#4	#4	6-#5	47	93	125
	100	1.84	12	0.763		15-#5+ 0	4	2-#4	15-#5	3	#4	#4	6-#5	60	128	162
	150	2.01	12	0.791		15-#5+ 0	4	2-#5	15-#5	3	#4	#5	6-#5	74	174	199
	200	2.19	12	0.818		15-#5+ 0	4	1-#5 and 1-#6	15-#5	3	#5	#5	6-#5	87	218	236
	300	2.63	12 *	0.874		15-#5+ 0	4	1-#6 and 1-#7	15-#5	3	#6	#6	6-#5	115	302	309
	400	3.13	12 *	0.634		15-#5+ 0	4	2-#7	18-#5	3	#7	#7	6-#5	142	385	383
24'- 0" D= 9.500 RIB NOT ON COLUMN LINE 0.613 CF/SF	50	1.92	12	0.801		18-#5+ 0	4	1-#4 and 1-#5	18-#5	4	#4	#4	7-#5	69	138	186
	100	1.98	12	0.831		18-#5+ 0	4	2-#5	18-#5	4	#4	#4	7-#5	90	190	242
	150	2.32	12	0.862		18-#5+ 0	4	2-#6	18-#5	4	#5	#5	7-#5	111	260	298
	200	2.57	12 *	0.892		18-#5+ 0	4	1-#6 and 1-#7	18-#5	4	#5	#6	7-#5	131	325	353
	300	3.32	12 *	0.634		18-#5+ 0	4	2-#8	16-#6	4	#6	#7	7-#5	172	453	464
	400	4.07	16 *	0.628		18-#5+ 1	4	2-#9	20-#6	4	#7	#7	9-#5	210	564	565
27'- 0" D= 9.500 RIB NOT RIB NOT ON COLUMN LINE 0.597 CF/SF	50	1.96	13	0.824		20-#5+ 0	4	2-#5	20-#5	5	#4	#4	8-#5	98	196	264
	100	2.22	13	0.860		20-#5+ 0	4	2-#6	20-#5	5	#4	#5	8-#5	127	263	343
	150	2.49	13 *	0.896		20-#5+ 0	4	2-#7	20-#5	5	#5	#5	8-#5	157	363	422
	200	3.03	13 *	0.632		20-#5+ 0	4	2-#8	25-#5	5	#5	#6	8-#5	186	455	502
	300	3.99	18 *	0.626		20-#5+ 1	4	2-#9	23-#6	5	#6	#7	8-#6	240	617	647
30'- 0" D=12.500 RIB ON COLUMN LINE 0.624 CF/SF	50	2.12	15	0.829		22-#5+ 0	5	1-#5 and 1-#6	22-#5	5	#4	#5	9-#5	136	272	367
	100	2.31	15 *	0.887		22-#5+ 3	5	2-#6	23-#5	5	#5	#5	9-#5	177	353	475
	150	2.95	15 *	0.933		22-#5+ 6	5	2-#7	28-#5	5	#6	#6	9-#5	217	483	584
	200	3.53	18 *	0.626		22-#5+ 3	5	2-#8	24-#6	5	#6	#7	11-#5	255	604	686
33'- 0" D=12.500 RIB ON COLUMN LINE 0.609 CF/SF	50	2.25	16 *	0.867		25-#5+ 2	5	2-#6	25-#5	6	#4	#5	10-#5	180	360	485
	100	2.71	16 *	0.922		25-#5+ 6	5	2-#7	31-#5	6	#5	#5	10-#5	234	467	629
	150	3.42	19 *	0.931		25-#5+10	5	2-#8	27-#6	6	#6	#6	13-#5	285	614	768
	200	4.05	25 *	0.621	3 S 4 1	25-#5+ 3	5	2-#9	32-#6	6	#6	#7	15-#5	331	711	892
36'- 0" D=12.500 RIB ON COLUMN LINE 0.597 CF/SF	50	2.52	18 *	0.875		27-#5+ 3	5	2-#7	31-#5	7	#5	#5	11-#5	231	463	623
	100	3.15	19 *	0.926		27-#5+ 9	5	2-#8	40-#5	7	#5	#6	13-#5	300	601	808
	150	3.93	27 *	0.620	3 S 4 1	27-#5+ 5	5	2-#9	35-#6	7	#6	#6	11-#6	362	725	976

SQUARE INTERIOR PANELS

Total Depth = 13 in. Rib Depth = 10 in. Total Slab Depth = 3 in.

Span c.-c. Columns $\ell_1 = \ell_2$ (ft)	Factored Super-imposed Load (psf)	(1) Steel (psf)	Square Interior Column: $c_1 = c_2$ (in.)	Square Interior Column: (2) Stirrups	Column Strip: Bottom No. Ribs	Column Strip: Bottom Bars per Rib	Column Strip: Top Interior No.-size	Middle Strip: Bottom No. Ribs	Middle Strip: Bottom Long Bars	Middle Strip: Bottom Short Bars	Middle Strip: Top Interior No.-size
18'- 0"	50	1.84	12		3	2-#4	13-#5	3	#4	#4	5-#5
	100	1.84	12		3	2-#4	13-#5	3	#4	#4	5-#5
	150	1.84	12		3	2-#4	13-#5	3	#4	#4	5-#5
	200	1.84	12		3	2-#4	13-#5	3	#4	#4	5-#5
	300	1.95	12	3 S 4 1	3	1-#4 and 1-#5	13-#5	3	#4	#4	5-#5
	400	2.28	12	3 S 4 1	3	1-#5 and 1-#6	13-#5	3	#4	#5	5-#5
21'- 0"	50	1.82	12		4	2-#4	15-#5	3	#4	#4	6-#5
	100	1.82	12		4	2-#4	15-#5	3	#4	#4	6-#5
	150	1.82	12		4	2-#4	15-#5	3	#4	#4	6-#5
	200	1.93	12		4	1-#4 and 1-#5	15-#5	3	#4	#4	6-#5
	300	2.22	12	3 S 4 1	4	2-#5	15-#5	3	#5	#5	6-#5
	400	2.69	12 *	3 S 4 1	4	2-#6	17-#5	3	#5	#6	6-#5
24'- 0"	50	1.86	12		4	2-#4	18-#5	4	#4	#4	7-#5
	100	1.86	12		4	2-#4	18-#5	4	#4	#4	7-#5
	150	2.05	12		4	2-#5	18-#5	4	#4	#4	7-#5
	200	2.25	12	3 S 4 1	4	1-#5 and 1-#6	18-#5	4	#4	#5	7-#5
	300	2.78	12 *	3 S 4 1	4	1-#6 and 1-#7	21-#5	4	#5	#5	7-#5
	400	3.49	13 *	3 S 4 1	4	2-#7	19-#6	4	#6	#6	9-#5
27'- 0"	50	1.86	13		4	2-#4	20-#5	5	#4	#4	8-#5
	100	2.02	13		4	2-#5	20-#5	5	#4	#4	8-#5
	150	2.13	13	3 S 4 1	4	1-#5 and 1-#6	20-#5	5	#4	#4	8-#5
	200	2.61	13 *	3 S 4 1	4	1-#6 and 1-#7	23-#5	5	#4	#5	8-#5
	300	3.55	14 *	3 S 4 2	4	1-#7 and 1-#8	21-#6	5	#5	#6	10-#5
30'- 0"	50	1.94	15		5	1-#4 and 1-#5	22-#5	5	#4	#4	9-#5
	100	2.03	15		5	2-#5	22-#5	5	#4	#4	9-#5
	150	2.63	15 *	3 S 4 1	5	2-#6	26-#5	5	#5	#5	9-#5
	200	3.12	15 *	3 S 4 1	5	1-#6 and 1-#7	22-#6	5	#5	#6	10-#5
33'- 0"	50	2.06	16		5	2-#5	25-#5	6	#4	#4	10-#5
	100	2.46	16 *	3 S 4 1	5	2-#6	28-#5	6	#4	#5	10-#5
	150	3.07	16 *	3 S 4 1	5	1-#6 and 1-#7	25-#6	6	#5	#5	12-#5
	200	3.63	18 *	3 S 4 2	5	2-#7	30-#6	6	#5	#6	10-#6
36'- 0"	50	2.16	18	3 S 4 1	5	1-#5 and 1-#6	28-#5	7	#4	#4	11-#5
	100	2.79	18 *	3 S 4 1	5	1-#6 and 1-#7	37-#5	7	#4	#5	12-#5
	150	3.62	18 *	3 S 4 2	5	1-#7 and 1-#8	33-#6	7	#5	#6	15-#5

See the notes on page 11-19.