Visteon Village Corporate Center

Van Buren Township, MI

Technical Assignment #2

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Structural Option
Advisor: Dr. Andres Lepage

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Executive Summary

This report is an analysis of alternate floor framing systems for the Visteon Village Corporate Headquarters in Van Buren, MI. In this study, four different floor systems were designed and analyzed, including the existing floor framing system. The existing design calls for a composite metal deck floor system on steel beams. The framing system has long spans that are typically heavily loaded, so although the current system in place meets the design criteria it is worthwhile to investigate other framing options. Once all alternate floor systems were designed, they were compared based on factors such as cost, fire rating, serviceability, and ease of construction. The following pages include preliminary analyses of the following alternate systems:

- Pre-Cast Hollow Core Slab on Steel
- Long Span Steel Joists
- Post-Tensioned Two Way Slab

Based upon my results, the best framing options are the existing composite slab system, and the post-tensioned two way slab. The composite slab system is a relatively quick and easy system to construct, and is able to handle the long spans while maintaining vibration criteria. The post-tensioned two way slab also handles the long spans very efficiently and has a smaller required floor depth than any of the other systems analyzed. Both systems seem like viable options for the framing system and will be further assessed in future reports.

Design Guides and Criteria

During the analysis of the existing and alternative floor systems, many design aids were consulted including:

The 2006 International Building Code (IBC 2006)

Building Code Requirements for Structural Concrete 2008, American Concrete Institute (ACI 318-08)

Steel Construction Manual, 13th Edition, American Institute of Steel Construction (AISC)

Minimum Design Loads for Buildings and Other Structures 2005, American Society of Civil Engineers (ASCE 7-05)

All floor systems were designed to meet 2 hour fire rating standards.

All floor systems were held to the following deflection criteria:

Live Load Deflection: L / 480

Total Load Deflection: L / 240

Existing Composite Steel Floor System

Foundation:

All of the foundation systems for the Visteon Village Corporate Center were designed based upon the findings of a geotechnical investigation performed by Somat Engineering on October 14, 2002**.** There is a deep foundation system to support all building columns, walls, grade beams and other foundation elements. The deep foundation elements are comprised of friction steel H-piles in native medium compact to compact sand. All H-piles consist of 75 foot long HP12x84 sections with concrete pile caps and are of ASTM A992 steel (Fy = 50 ksi). The number of piles for each foundation element range from 1 to 7 providing capacities of 100 kips to 1050 kips respectively. The concrete pile caps are of reinforced concrete construction with their top elevation at a minimum depth of 3'-6" below finished grade as to prevent frost heave. The dimensions of the caps range from 3'x3' for a single H-pile element up to 13'x11'-8" for a 7 H-pile element. All concrete used in the foundation systems has a minimum compressive strength of 3000 psi.

Columns:

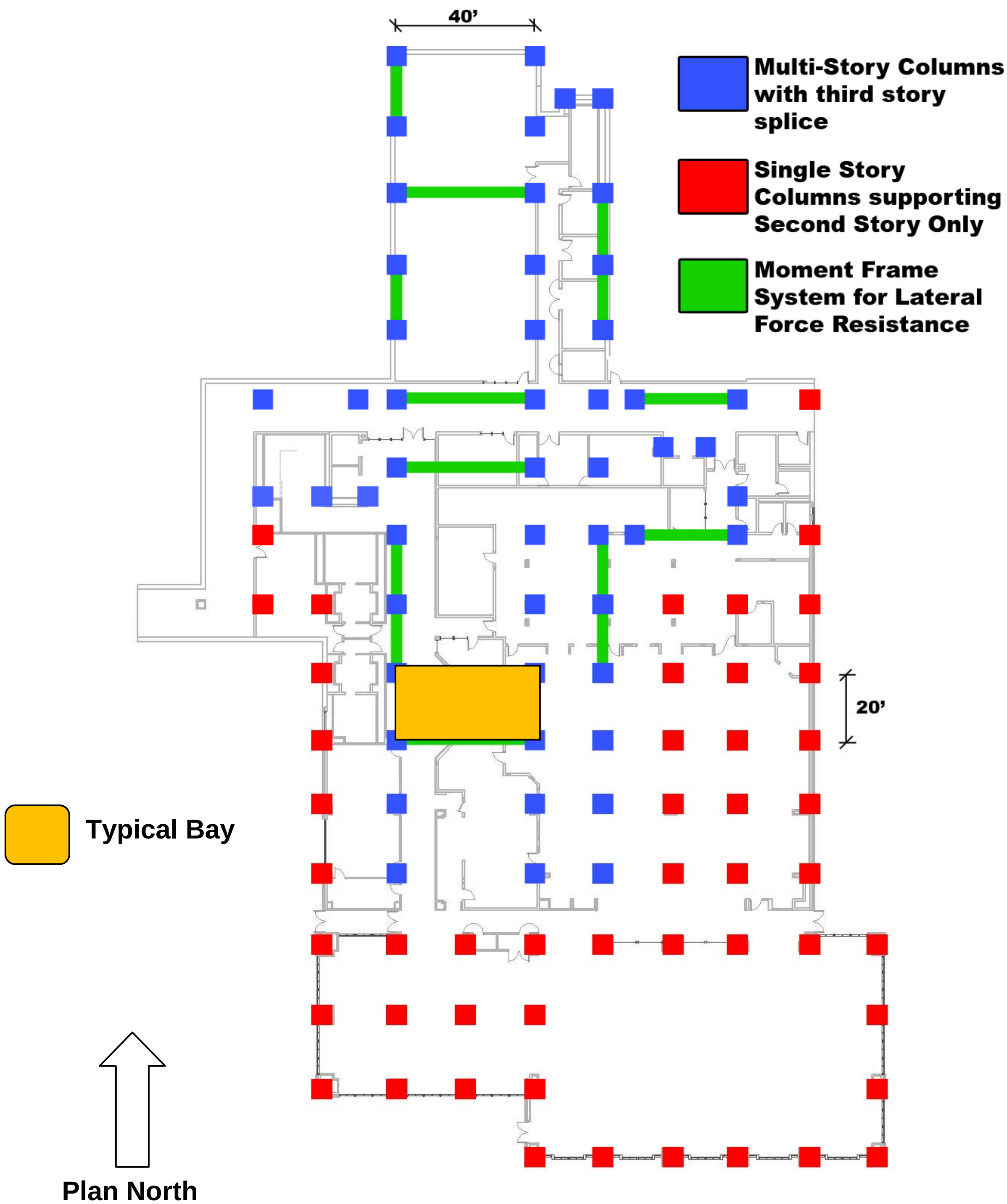
All of the columns of the building are composed of structural steel. The main column system is made up of ASTM A992 wide flange shapes ranging in size from W14x43 to W14x311. Typically, these columns rest upon the deep foundation system and extend 72 feet to the penthouse level with a column splice at an elevation of 52 feet (falling within the third story). These multistory columns are also part of the special moment frame system that resists lateral loading.

Floor and Roof Framing System:

The typical framing system for the Visteon Village Corporate Center is composed of structural steel composite beams and girders. The supported floor consists of 40 foot long ASTM A992 wide flange shapes spanning a column free space. The typical bay for each floor is 40'x20' with wide flange beams spaced at 10' on center supporting 3" composite metal floor deck with 3-1/4" light weight concrete fill providing a total slab depth of 6-1/4". All supporting materials for this system can be found in the appendix.

Lateral:

All lateral loads caused by wind and seismic forces are resisted by structural steel moment frames. There are five moment frames running in the North/South direction of analysis and six moment frames running in the East/West direction of analysis. Each moment frame consists of multistory wide flange columns and wide flange beams.

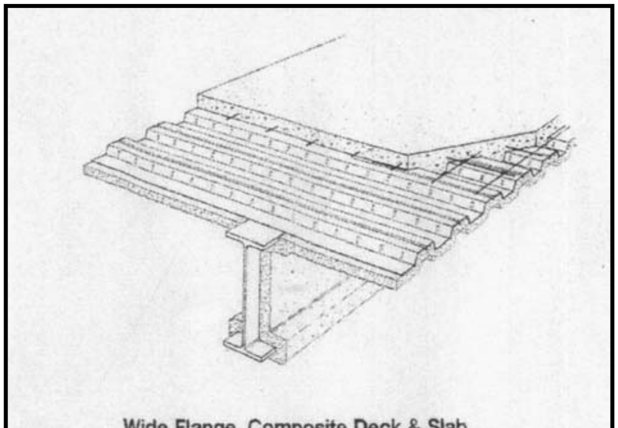


Pros and Cons: Existing Composite Steel Floor System

The system handles the structural requirements of the Visteon Village Corporate Center adequately. It is a very good system to use for long spans that have heavy distributed loads, which are present in each typical bay of the building. The combination of the steel deck and concrete slab also provides for a two hour fire rating. Deflection is minimized by the use of large steel sections, ensuring that this system meets the defined live load and total load deflection criteria. This system also meets vibration criteria as analyzed by Ram Structural System. The construction of the system is also relatively easy and very efficient. Formwork and shoring are not required with this method and there are minimal slab openings providing the opportunity for fast slab pouring. Erecting the supporting steel is also faster and more efficient than having to form and pour concrete beams and columns. Economically, the system is relatively cheap as well (about $28.00 per sq ft).

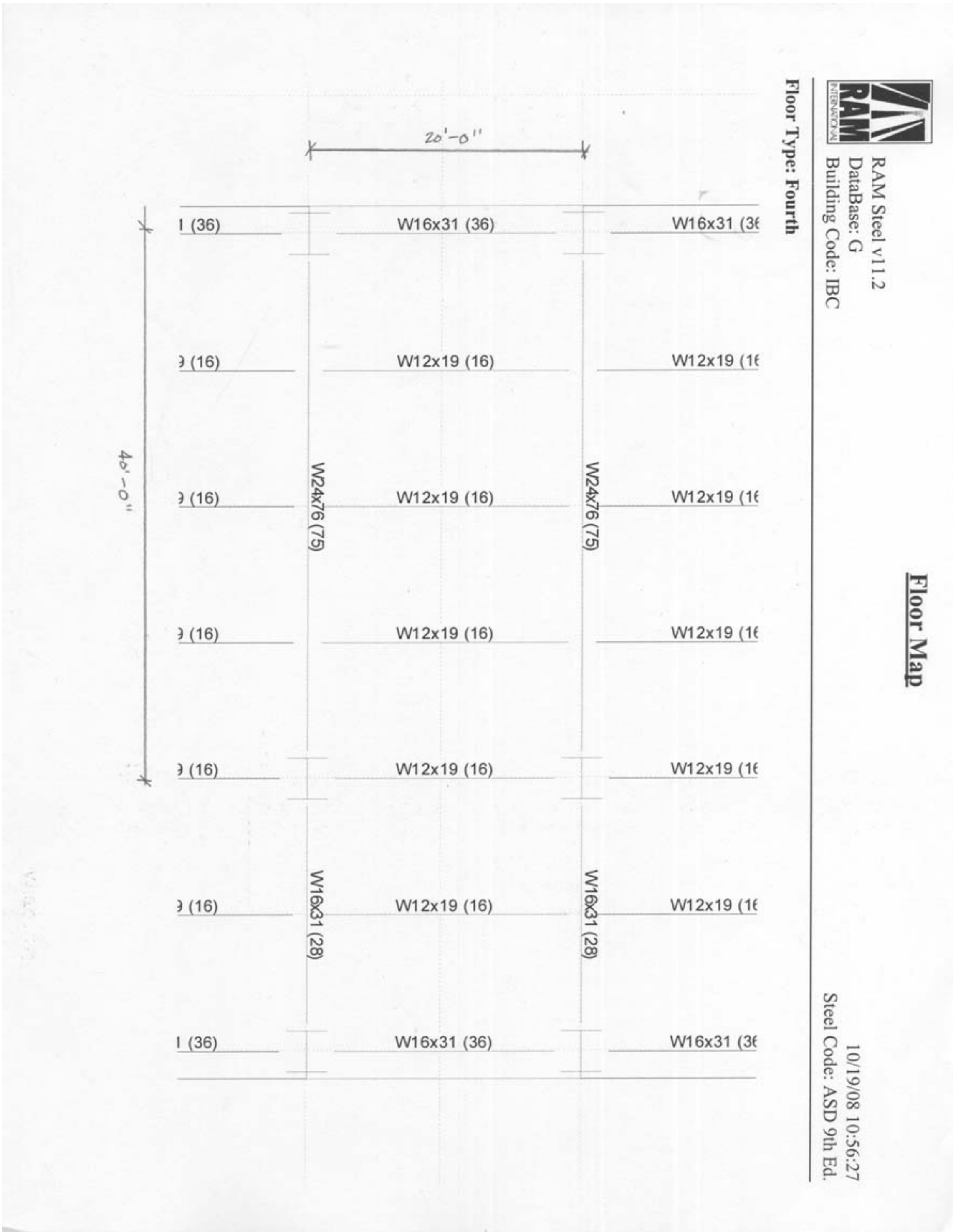
There are some drawbacks to the system however. The large steel sections and thick deck/slab combination provide a floor depth of about 30 inches, which could be difficult to work with from an architectural standpoint. The system also creates a large weight for the foundations to bear.

In conclusion, this system is an exceptionally good choice for the project as it meets all of the structural requirements and demands of the building.



Wide Flange, Composite Deck & Slab

Composite Steel System Typical Bay Framing:



Plan North

Precast Hollow Core Plank

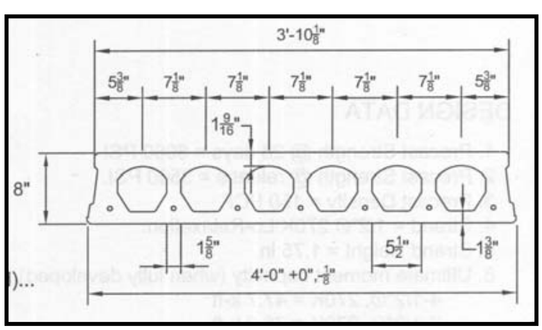
The first alternative floor system analyzed was the hollow core plank. This system was designed to be supported by wide flange steel section beams and girders for the typical 40' x 20' bay (see framing plan on next page). The hollow core plank was chosen based on a 20' span length, superimposed dead loading of 25 psf, and live loading of 100 psf. The concrete used in this system has an f'c = 5,000 psi with seven ½" Lo-Relaxation reinforcement strands of fpu = 270,000 psi. All supporting materials for this system can be found in the appendix.

Pros and Cons: Hollow Core Plank

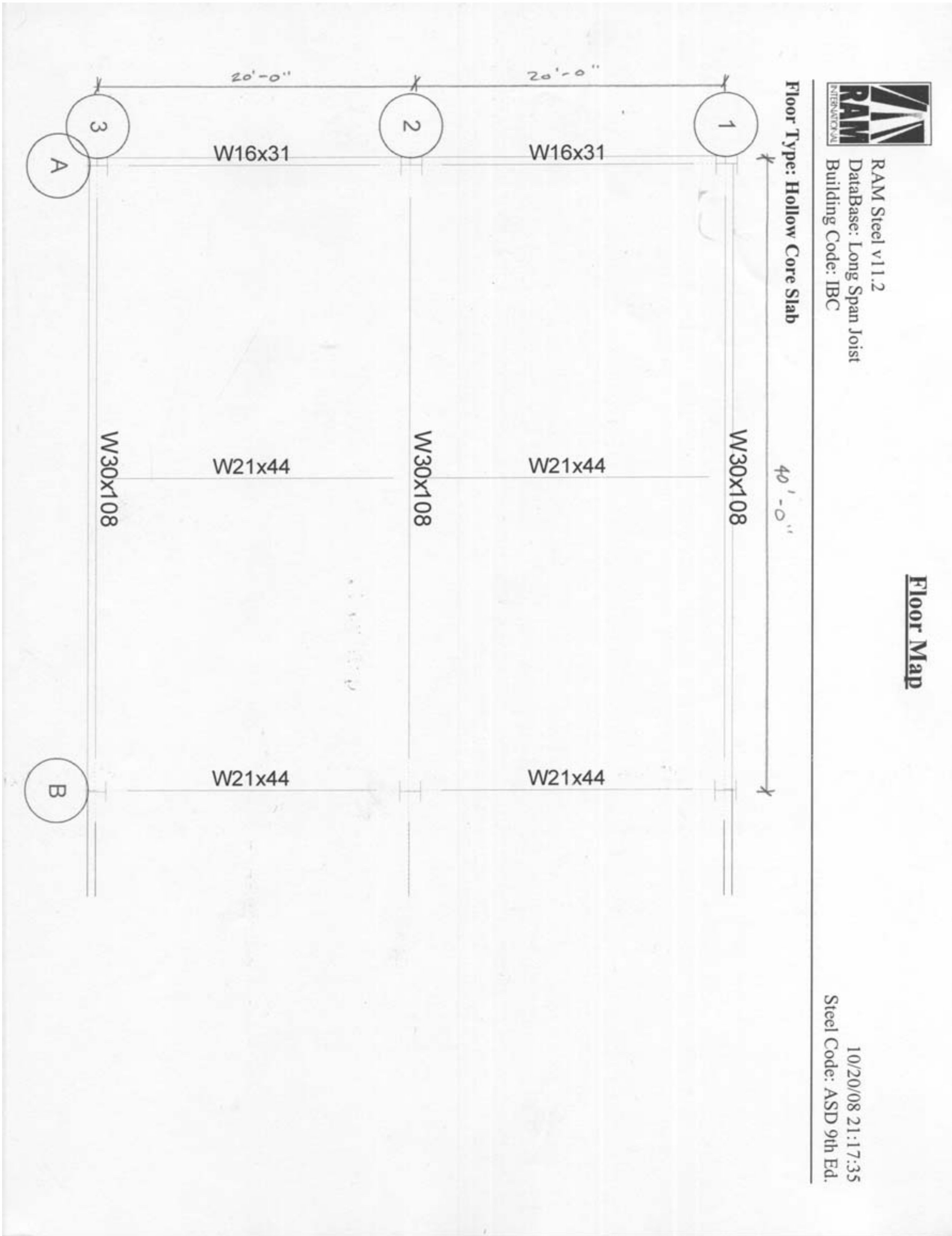
The system has the ability to adequately handle the spans of the typical bay while maintaining a slim slab thickness of only 8". The individual planks have a width of 4'-0" which fits very well into the existing bay size, meaning no alteration of the column grid would be needed to institute this system. The 8" hollow core plank provides a 2 hour fire rating as well. Each plank has a bit of camber to it, and when resting upon the steel framing the system easily meets all deflection criteria. The construction of the system, like most precast products, is relatively fast and efficient once all the materials are on site. The cost of this system is also the lowest of the systems analyzed.

A large lead time is needed when ordering the hollow core planks, which may slow the overall construction process. The deep girders needed to provide sound structural support for the system combined with the 8" plank itself provides an overall thickness of 38", which is quite large and can cause architectural problems. The vibration effects of this system are unknown and would require further analysis.

Overall, the system performs well structurally as it can handle the heavy loading over the long spans. The problems lie with the expensive nature of this system, and the extremely large floor depth required. Due to these reasons, this floor system does not seem like a viable option for the Visteon Village Corporate Center.



Hollow Core Plank Typical Bay Framing:



Plan North

Long Span Steel Joists

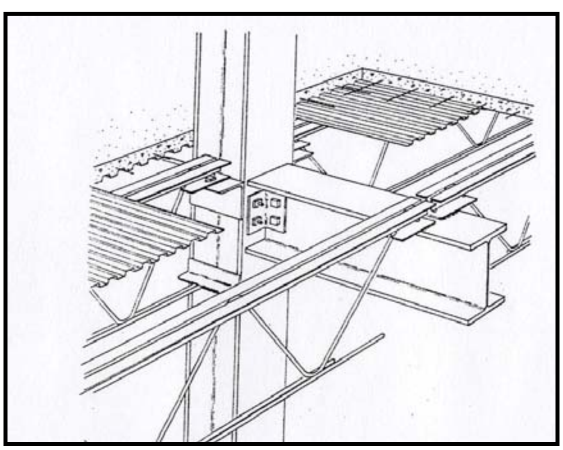
The long span steel joist system was used to span in the 40' direction of the typical 40' x 20' bay. A 20 gage 2" metal deck with 3-1/4" lightweight concrete slab was used to provide a total slab depth of 5-1/4". Since the maximum unshored span of this assembly was 9.39', two joists were needed along the 20' direction to provide adequate support for the system. Combining the 25 psf superimposed dead load with the 41 psf self weight dead load, a total dead load of 66 psf was used. The standard live load of 100 psf was also used. RAM Structural System was used for this analysis. All supporting materials for this system can be found in the appendix.

Pros and Cons: Long Span Steel Joists

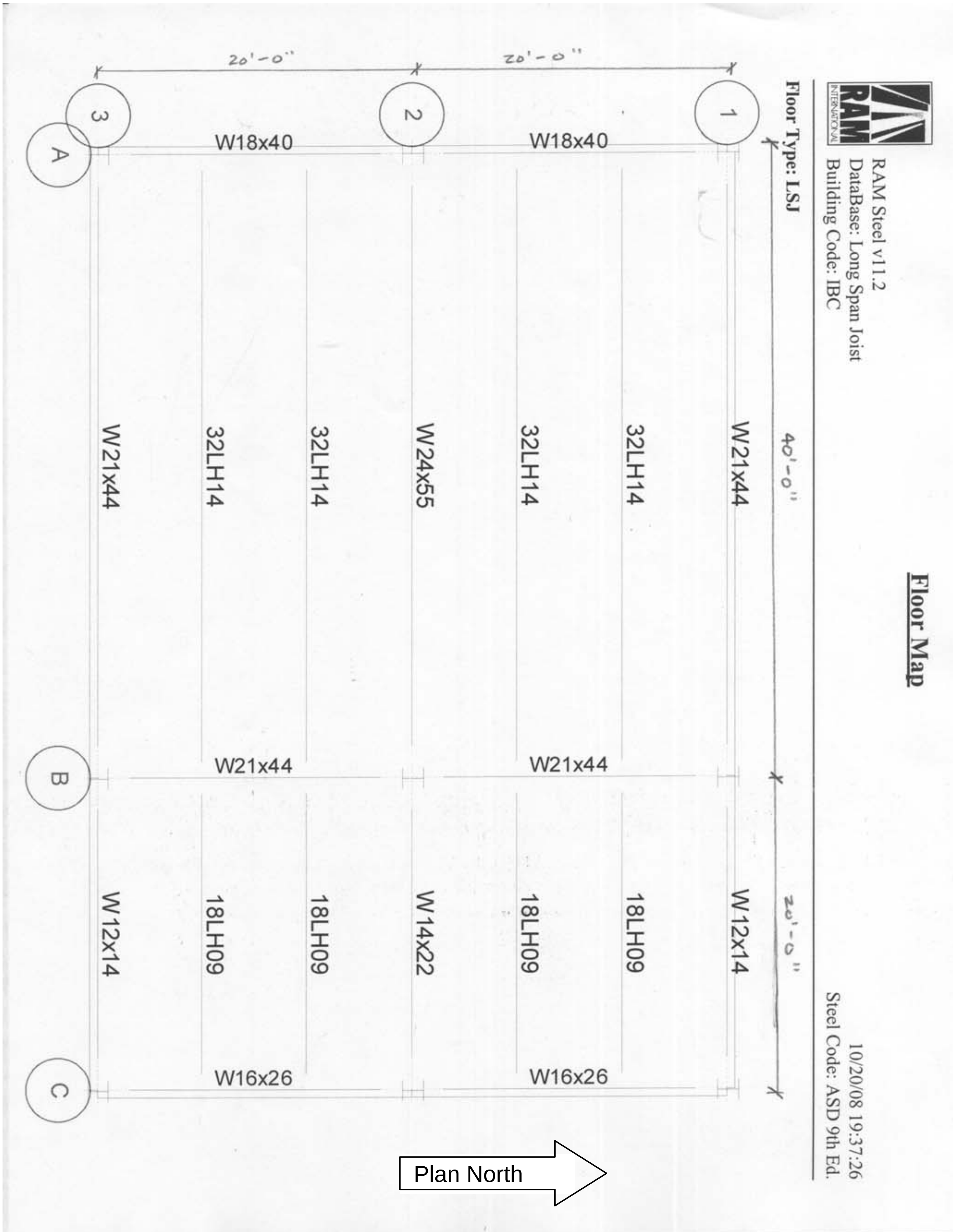
The long span steel joist system soundly supports the loads and structural demands that the Visteon Village Corporate Center provides. The system passes all the vibration and deflection criteria, as analyzed by RAM Structural System. The system is generally quick and easy to construct, and when spray on fireproofing is applied, both the joists and slab assembly achieve a 2 hour fire rating.

The heavy loading and the long spans cause the joists to have a 32" depth, and when combined with the 5-1/4" slab assembly the total floor depth totals 37-1/4". This can be an architectural problem as floor depth is an important factor in designing the building. Also, the cost of the joists in comparison with other systems is quite high and not economical.

While the long span joist system satisfies the structural conditions it was tested for, the cost and overall depth of the system prevent it from being a considerable option for the Visteon Village Corporate Center.



Long Span Steel Joist Typical Bay Framing:



Two Way Post-Tensioned Slab

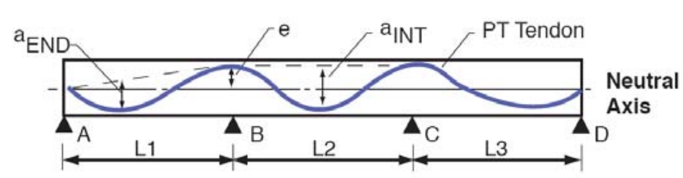
The last system I chose for my alternative floor system analysis was the two way post-tensioned slab system. For this system I used a quick, simplified design approach provided by the Portland Cement Association. The concrete used was normal weight concrete with a weight of 150 pcf, and f'c = 5000 psi. The rebar reinforcement used had fy = 60,000 psi and the post-tensioning consisted of unbounded tendons that were ½" diameter 7-wire strands. All supporting materials for this system can be found in the appendix.

Pros and Cons: Two Way Post-Tensioned Slab

The most beneficial aspect of using a two way post-tensioned slab system is that you are able to adequately support large, heavily loaded span lengths while keeping the floor depth relatively small. In my analysis I was able to meet design requirements using an 11" slab, which was much smaller than any other system analyzed for this report. This system was able to meet the requirements for a 2 hour fire rating as well. Economically, the cost is comparable to the other systems (about $27.00 per sq ft).

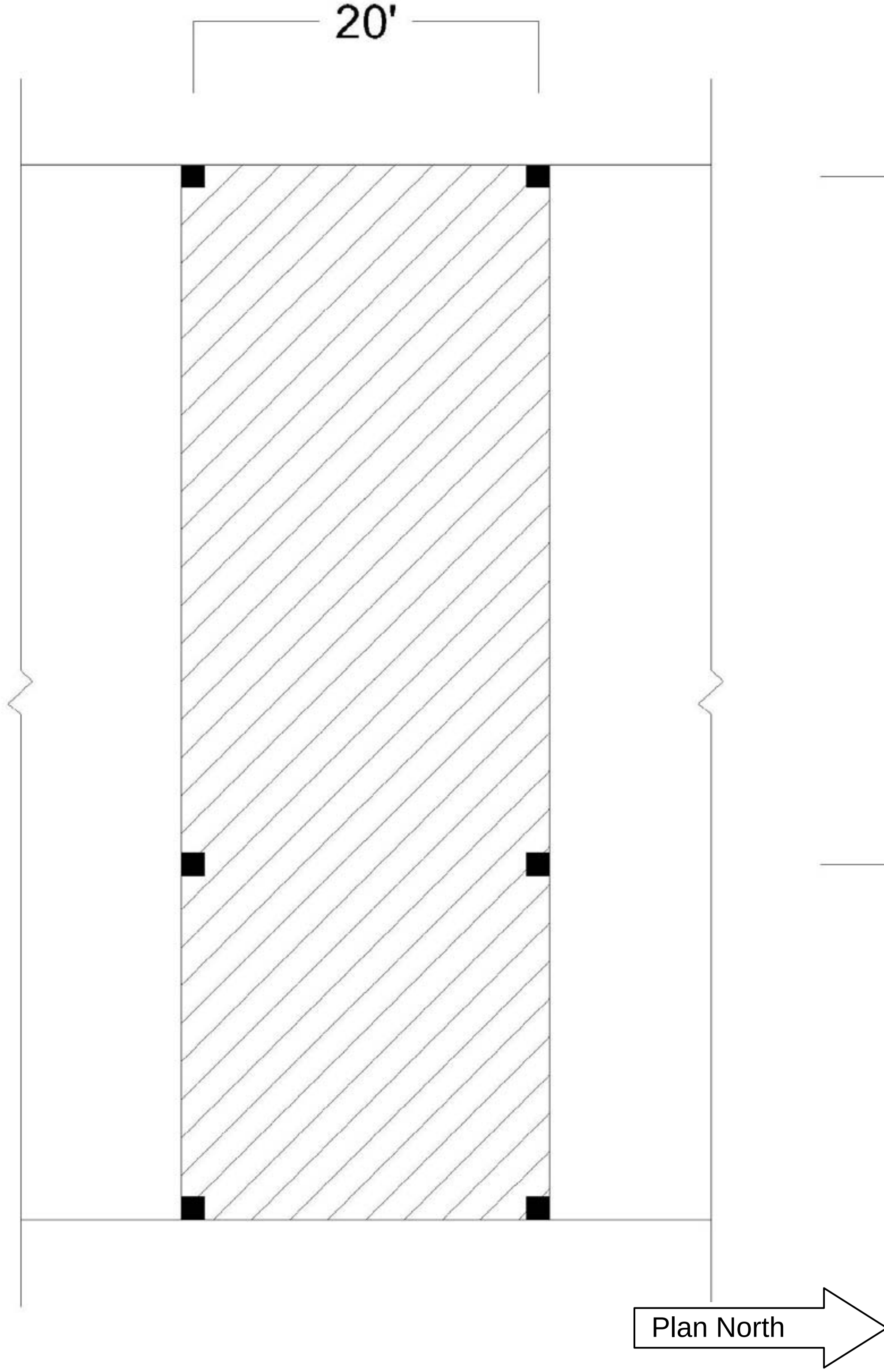
To properly execute the installation of a two way post-tensioned slab system, a specialized and very experienced design and construction team is needed. Also, supervision of the construction process is not only encouraged but mandatory, and due to specifications it may be required to have a testing agency on site to monitor construction. Due to these facts, the construction process can get complicated. Once the system is in place, there can be no additional openings added to the layout to minimize risk of severing a tendon. This could mean an increase in the planning stages of construction creating a longer lead time and overall longer duration of the construction process. The vibration effects of this system are unknown and will require further analyzation.

The two way post-tensioned slab system is definitely the best of the alternate floor systems analyzed for this building. When executed properly, its structural efficiency and minimum floor depth make this flooring system a viable option for further research.



Continuous Post-Tensioned Beam

Two Way Post-Tensioned Slab Typical Bay:



Floor System Direct Comparison

Category	Existing Composite Steel	Hollow Core Plank	Long Span Steel Joists	Two Way Post-Tensioned Slab
Slab Depth (in)	5.25	8.00	5.25	11.00
Total Floor Depth (in)	30.00	38.00	37.25	11.00
Weight (psf)	92	87	66	138
Ease of Construction	Medium	Medium	Easy	Hard
Fire Rating (hrs)	2	2	2	2
Material Cost (per sq ft)	$19.05	$14.55	$17.15	$17.50
Labor Cost (per sq ft)	$8.70	$7.95	$11.40	$9.45
Total Cost (per sq ft)	$27.75	$22.50	$28.55	$26.95
Viable Alternative	-	No	No	Yes
Additional Study	-	No	No	Yes

Color Key:

Architectural
Structural
Construction
Safety
Economical
Analysis

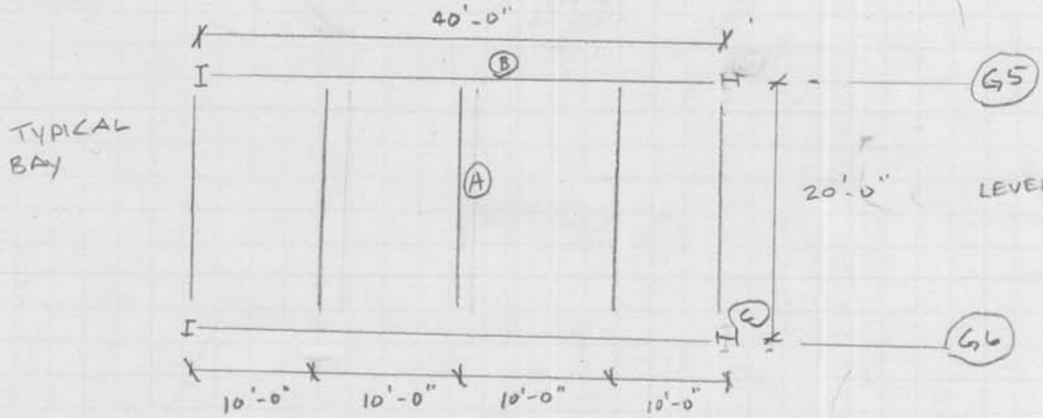
Conclusion

After all of the alternative floor systems were assessed using simplified design methods, only two systems stood out as viable options: the existing composite steel system, and the two way post-tensioned slab. While the other two systems turned out to be lighter, the additional floor depth that they would bring to the building was cause for concern. The building is used as a mixed office and laboratory space, where some equipment placed in the building requires a minimum floor to floor clear space. The existing system has a floor depth of 30”, which means any increase to this value would cause the overall building height to increase to match an identical floor to floor clear height. This change would potentially nullify any cost benefits of the hollow core plank system. The long span steel joists have a cost similar to the existing composite steel system, but due to the deeper required floor depth of 37.25”, this system was turned down as well. The two way post-tensioned slab has a larger load than the existing system, and utilizes a more advanced construction technique requiring a skilled team and on-site supervision. The overall cost of the system comes out to be less than the existing system however, and the total floor depth is reduced by close to two thirds (11”). This makes the post-tensioned system an option worth further research.

Appendix

Existing Floor System: Composite Steel

FLOOR SYSTEM SPOT CHECK



BEAM "A"

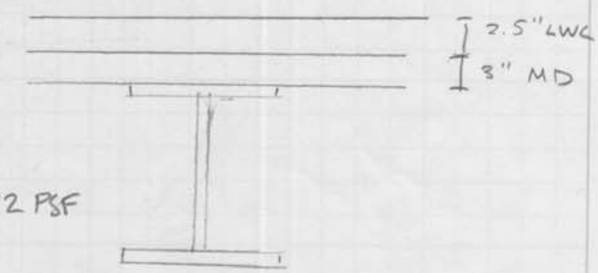
GIVEN: COMPOSITE W12×19 (FULL COMPOSITE ACTION) w/ 16 STUDS

SPAN = 20'-0"

SPACED @ 10'-0" O.C.

$A_s = 5.57 \text{ IN}^2$

$f'_c = 4000$ PSI



DL

FIRE RATED COMPOSITE FLR DECK: 92 PSF

LL

OFFICE = 100 PSF

b_{eff} $\begin{cases} 10' \text{ TRIB WIDTH} \\ 20'/4 = 5' \leftarrow \text{CONTROLS} \end{cases}$

COMPRESSION FORCES

$V'_c = 0.85(4)\underbrace{(5')(12)}_{60"}(2.5) = 510 \text{ K}$

$V'_s = 5.57(50) = 278 \text{ K} = V'_q = \Sigma Q_n$

STEEL CONTROLS SO PNA IS AT OR ABOVE FLANGE

DEPTH OF CONCRETE TO BALANCE V'_s

$$a = \frac{278}{(0.85)(4)(60)} = 1.36"$$

MOMENT ARM OF COMPRESSIVE FROM TOS

$$Y_2 = 2.5 - \frac{1.36}{2} = 1.82 \rightarrow 2" \text{ (MIN)}$$

TABLE 3-19

$\phi M_n = 169'\text{K}$ $\quad \Sigma Q_n = 279 \text{ K}$

2

MU

$w_{DEAD} = (10')(92\ PSF) = 920 \doteq 0.92\ K/FT$

$w_{LIVE} = (10')(100\ PSF) = 1000 \approx 1\ K/FT$

$W_u = 1.2(0.92) + 1.6(1.0) = 2.70\ K/FT$

$M_u = \frac{wL^2}{8} = 135'K < 167'K$ ✓

\# OF STUDS

$Q_n = 21.2^K$ PER STUD (3-21)

$\Sigma Q_n = \frac{279^K}{21.2} = 13.3 \rightarrow 14$ STUDS

[16] BY DESIGN > 14 ✓

GIRDER "B"

GIVEN:

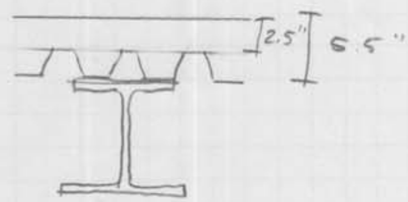
COMPOSITE 24 x 76 (FULL COMPOSITE ACTION) w/ 75 STUDS

SPAN: 40'

SPACING: 20' O.C.

$A_S = 22.4\ IN^2$

$f'_c = 4000$ PSI



DL:

FIRE RATED COMPOSITE FLR DECK: 92 PSF

LL:

OFFICE: 100 PSF

$b_{eff} \begin{cases} 20' \text{ TRIB WIDTH} \\ 40'/4 = 10' \leftarrow \text{CONTROLS} \end{cases}$

COMPRESSION FORCES

$V'_c = 0.85(4)\underbrace{(10')(12)}_{120''}(2.5) = 1020^K$

$V'_s = 22.4(50) = 1120^K$

$V'_s > V'_c$ SO PNA IN STEEL

3

WHERE IS PNA

$T_f = 0.68"(8.99")(50) = 305.7^k$

$T_w = 1120 - 2(305.7^k) = 508.6^k$

$814.3^k < 1020^k$

∴ PNA IN FLANGE

AREA OF STEEL IN COMPRESSION

$$A_{s-c} = \frac{1120 - 1020}{2(50)} = 1 \text{ in}^2$$

$$x = 1/8.99 = 0.111"$$

$$M_n = T_s\left(\frac{d}{2}\right) + C_c\left(\frac{t}{2}\right) - 2A_{sc}F_y\left(\frac{x}{2}\right)$$

$$M_n = 1120\left(\frac{23.7}{2}\right) + 1020\left(5.5 - \frac{2.5}{2}\right) - 2(1)(50)\left(\frac{0.111}{2}\right)$$

$$M_n = \frac{17713.5}{12} = 1476.12\ 'k$$

$$\phi M_n = 0.9(1476) = 1328\ 'k$$

$w_D = (20')(92\ PSF) = 1840 = 1.84\ k/ft$

$w_L = (20')(100\ PSF) = 2000 = 2.0\ k/ft$

$1.2(1.84) + 1.6(2.0) = 5.41\ k/ft$

$$M_u = \frac{wl^2}{8} = \frac{5.41(40')^2}{8} = 1082\ 'k < 1328\ 'k \checkmark$$

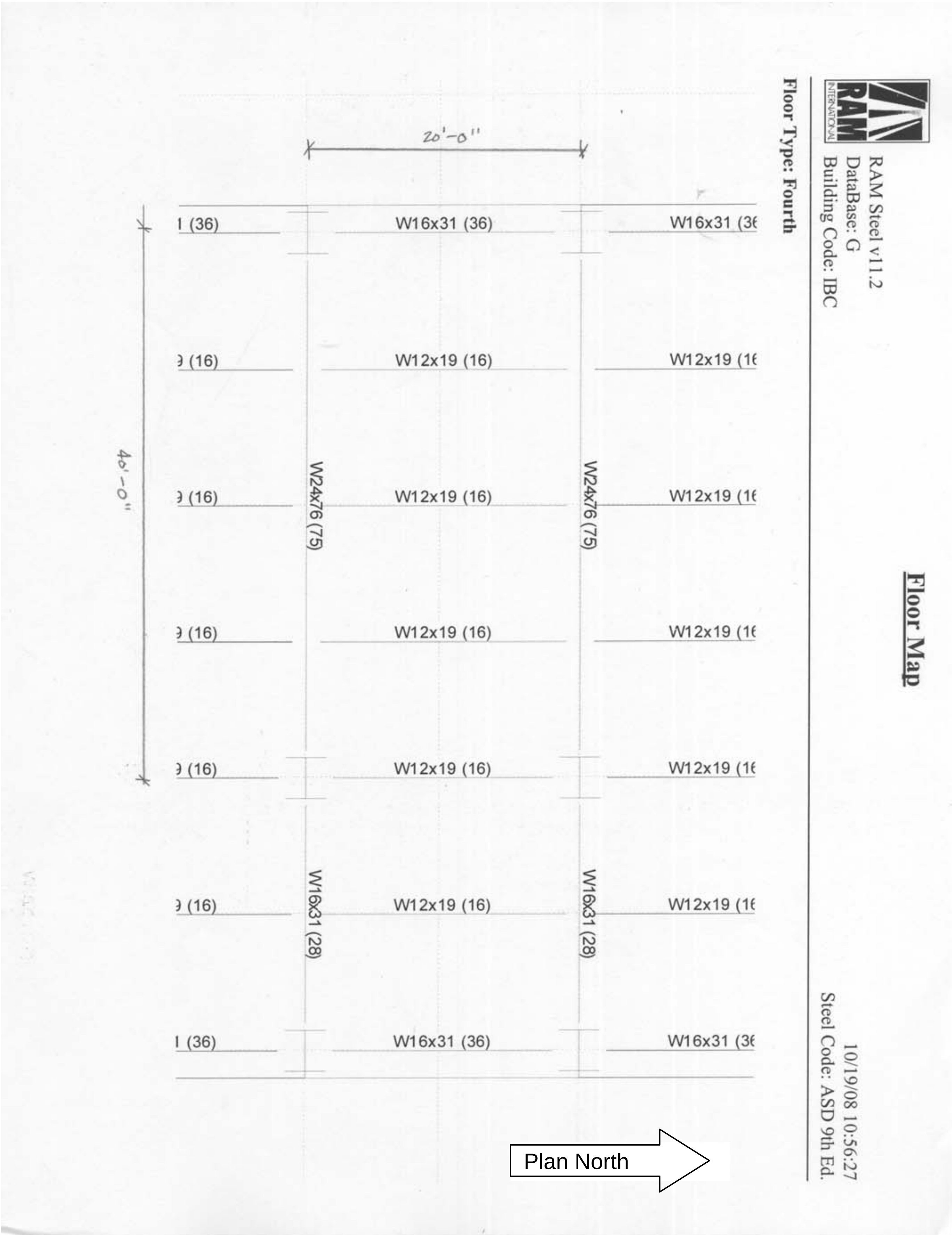
	ΣQ_n
0	1120
0.111	1019
0.170	966

$\Sigma Q_n = 1019$

$Q_n = 17.2$

$$\frac{1019}{17.2} = 59.2 \rightarrow 60 \text{ STUDS}$$

[75] AS DESIGNED ∴ ✓

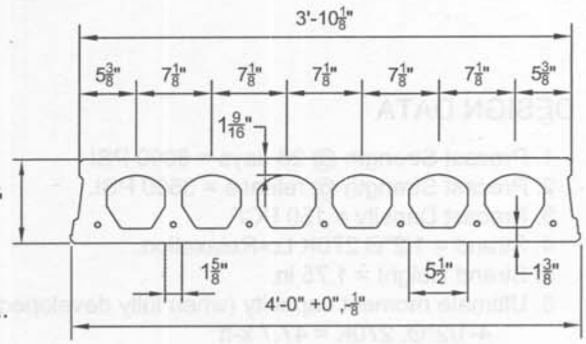

Floor Map
RAM Steel v11.2
DataBase: G
Building Code: IBC
10/19/08 10:56:27
Steel Code: ASD 9th Ed.
Floor Type: Fourth
20'-0"
40'-0"
W16x31 (36)
W12x19 (16)
W24x76 (75)
W16x31 (28)
Plan North

Alternative Floor System: Hollow Core Slab

Prestressed Concrete
8"x4'-0" Hollow Core Plank

2 Hour Fire Resistance Rating (Untopped)

PHYSICAL PROPERTIES Precast	
$A = 235$ in.^{2}	$S_b = 459$ in.^{3}
$I = 1838$ in.^{4}	$S_t = 459$ in.^{3}
$Y_b = 4.00$ in.	Wt.= 245 PLF
$Y_t = 4.00$ in.	Wt.= 61.25 PSF
$e = 2.25$ in.	



DESIGN DATA

1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI.
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
   4-1/2"Ø, 270K = 72.8 k-ft
   7-1/2"Ø, 270K = 119.8 k-ft
7. Maximum bottom tensile stress is $7.5\sqrt{f'c} = 580$ PSI
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. Load values to the left of the solid line are controlled by ultimate shear strength.
12. Load values to the right are controlled by ultimate flexural strength or structural fire endurance.
13. Load values may be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
14. Camber is inherent in all prestressed hollow core slabs and is a function of the amount of eccentric prestressing force needed to carry the superimposed design loads along with a number of other variables. Because prediction of camber is based on empirical formulas it is at best an estimate, with the actual camber usually higher than calculated values.

SAFE SUPERIMPOSED SERVICE LOADS — IBC 2003 & ACI 318-02 (1.2 D + 1.6 L)

Strand Pattern		SPAN (FEET) 17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35
4 - 1/2"ø	LOAD (PSF)	218	188	162	140	121	105	91	78	67	58	49	41	34						
7 - 1/2"ø	LOAD (PSF)	288	269	252	236	222	210	196	176	157	141	126	113	101	90	81	72	64	57	50

NITTERHOUSE
CONCRETE PRODUCTS

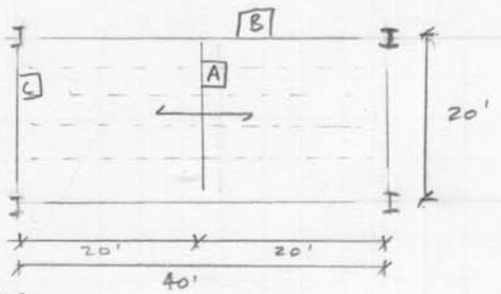
2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17201-0813
717-267-4505 Fax 717-267-4518

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. The allowable loads shown in this table reflect a 2 Hour & 0 Minute fire resistance rating.

08/10/07

8SF2.0

HOLLOW CORE PLANK CALCULATIONS



8" x 4'-0" HCP
2 HR FIRE RATING

LOADS

DL (PLANK): 61.25 PSF
DL (SUPERIMPOSED): 25 PSF
DL (STEEL): 8 PSF
LL = 100 PSF

SERVICE LOADING

$1.2(25) + 1.6(100) = 190 \text{ PSF} < 236 \text{ PSF}$

$w_u = 1.2(61.25 + 25 + 8) + 1.6(100) = 273.1 \approx 275 \text{ PSF}$

DEFLECTION LIMITS

$\Delta_{LL} = L/480$
$\Delta_{TL} = L/240$

DESIGN OF BEAM A

$\Delta_{LL} = 20(12)/480 = 0.5''$

$\Delta_{TL} = 20(12)/240 = 1.0''$

w_{LL} (UNFACTORED) = 100 PSF (20') = 2000 PLF = 2 KLF

$\Delta_{LL} \quad \dfrac{5 \; (2)(20')^4(1728)}{384 \; (29000)(I_{req})} \le 0.5'' \qquad I_{req} = 496 \text{ IN}^4$

$w_{TOT} = 0.275(20') = 5.5 \text{ KLF}$

$\Delta_{TL} \quad \dfrac{5 \; (5.5)(20')^4(1728)}{384 \; (29000)(I_{req})} = 1.0'' \qquad I_{req} = 683 \text{ IN}^4$ ← CRITICAL

$M_u = \dfrac{w\ell^2}{8} = \dfrac{(5.5)(20)^2}{8} = 275 \text{ 'K}$

$V_u = \dfrac{w\ell}{2} = \dfrac{(5.5)(20')}{2} = 55 \text{ K}$

$\boxed{\text{USE W21x44}}$

$\phi M_n = 358 \text{ 'K} > 275 \text{ 'K}$ ✓
$\phi V_n = 217 \text{ K} > 55 \text{ K}$ ✓
$I_x = 843 \text{ IN}^4 > 683 \text{ IN}^4$ ✓

DESIGN OF GIRDER B

$\Delta_{LL} = 40(12)/480 = 1''$

$\Delta_{TL} = 40(12)/240 = 2''$

$P_{LL} = (20')(20')(100\ PSF) = 40000\ lb = 40\ K$
(UNFACTORED)

$$\Delta_{LL} \quad \frac{(40\,K)(40')^3(1728)}{48\,(29000)\,I_{req}} = 1'' \qquad I_{req} = 3178\ IN^4$$

$P_{TL} = 2(55^K) = 110^K$

$$\Delta_{TL} = \frac{(110\,K)(40')^3(1728)}{48\,(29000)\,I_{req}} = 2'' \qquad I_{req} = 4370\ IN^4 \leftarrow \text{CONTROLS}$$

$$M_u = \frac{PL}{4} = \frac{110^K(40')}{4} = 1100'K$$

$$V_u = P/2 = \frac{110^K}{2} = 55\ K$$

$\boxed{\text{USE}\quad W30 \times 108}$

$\phi M_n = 1300'K > 1100'K$ ✓

$\phi V_n = 488\ K > 55\ K$ ✓

$I_x = 4470\ IN^4 > 4370\ IN^4$ ✓

DESIGN OF GIRDER C

$\Delta_{LL} = 0.5''$

$\Delta_{DL} = 1.0''$

w_{LL} (UNFACTORED) = 100 PSF (10') = 1000 PLF = 1 KLF

$$\Delta_{LL} = \frac{5(1)(20')^4(1728)}{384(29000)I_{req}} \qquad I_{req} = 248$$

$$\Delta_{TL} = \frac{5(2.75)(20')^4(1728)}{384(29000)I_{req}} \qquad I_{req} = 342 \leftarrow \text{CRITICAL}$$

$$M_u = \frac{wl^2}{8} = \frac{(2.75)(20')^2}{8} = 138'k$$

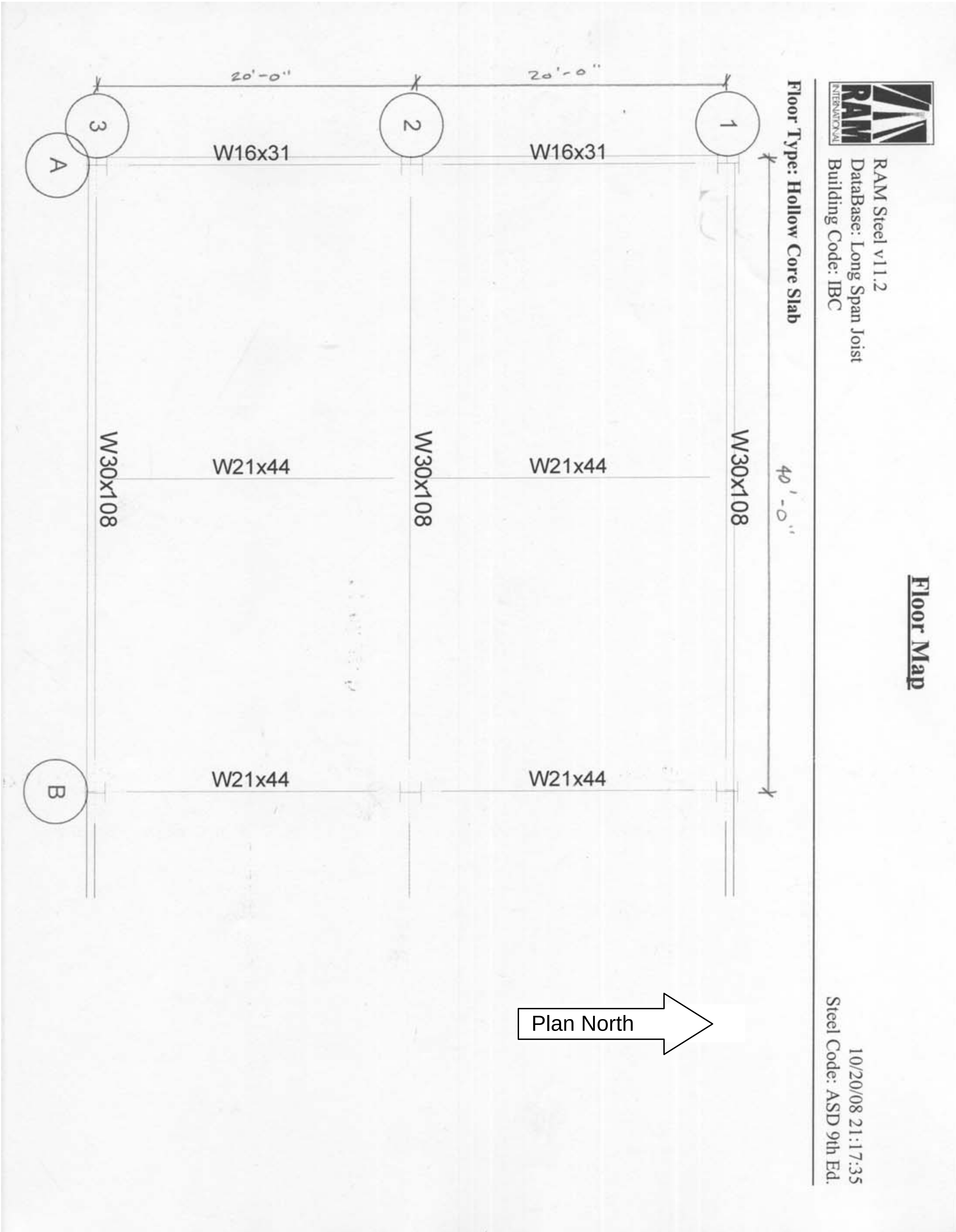
$$V_u = \frac{wl}{2} = \frac{(2.75)(20')}{2} = 27.5'k$$

$\boxed{\text{USE W16x31}}$

$\phi M_n = 203'k > 138'k$ ✓

$\phi V_n = 131'k > 27.5'k$ ✓

$I_y = 375\ in^4 > 342\ in^4$ ✓

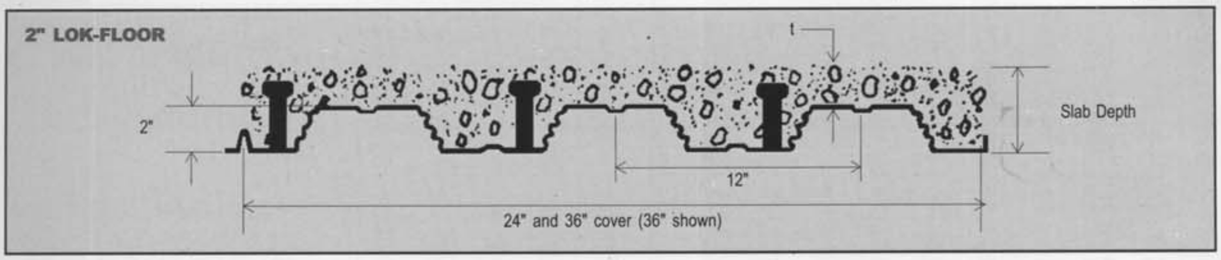

Floor Map
RAM Steel v11.2
DataBase: Long Span Joist
Building Code: IBC
10/20/08 21:17:35
Steel Code: ASD 9th Ed.
Floor Type: Hollow Core Slab
20'-0"
20'-0"
40'-0"
1
2
3
A
B
W16x31
W16x31
W21x44
W21x44
W21x44
W21x44
W30x108
W30x108
W30x108
Plan North

Alternative Floor System: Long Span Steel Joists

2 x 12" DECK F_y = 33ksi f'_c = 3 ksi 115 pcf concrete

USD United Steel Deck, Inc.

LRFD



The **Deck Section Properties** are per foot of width. The **I** value is for positive bending (in.^{4}); **t** is the gage thickness in inches; **w** is the weight in pounds per square foot; $\mathbf{S_p}$ and $\mathbf{S_n}$ are the section moduli for positive and negative bending (in.^{3}); $\mathbf{R_b}$ and $\phi \mathbf{V_n}$ are the interior reaction and the shear in pounds (per foot of width); studs is the number of studs required per foot in order to obtain the full resisting moment, $\phi \mathbf{M_{nf}}$.

DECK PROPERTIES									
Gage	t	w	As	I	S_p	S_n	R_b	ϕV_n	studs
22	0.0295	1.5	0.440	0.338	0.284	0.302	714	1990	0.43
20	0.0358	1.8	0.540	0.420	0.367	0.387	1010	2410	0.52
19	0.0418	2.1	0.630	0.490	0.445	0.458	1330	2810	0.61
18	0.0474	2.4	0.710	0.560	0.523	0.529	1680	3180	0.69
16	0.0598	3.1	0.900	0.700	0.654	0.654	2470	3990	0.87

The **Composite Properties** are a list of values for the composite slab. The **slab depth** is the distance from the bottom of the steel deck to the top of the slab in inches as shown on the sketch. U.L. ratings generally refer to the cover over the top of the deck so it is important to be aware of the difference in names. $\phi \mathbf{M_{nf}}$ is the factored resisting moment provided by the composite slab when the "full" number of studs as shown in the upper table are in place; inch kips (per foot of width). $\mathbf{A_c}$ is the area of concrete available to resist shear, in.^{2} per foot of width. **Vol.** is the volume of concrete in ft.^{3} per ft.^{2} needed to make up the slab; no allowance for frame or deck deflection is included. **W** is the concrete weight in pounds per ft.^{2}. $\mathbf{S_c}$ is the section modulus of the "cracked" concrete composite slab; in.^{3} per foot of width. $\mathbf{I_{av}}$ is the average of the "cracked" and "uncracked" moments of inertia of the transformed composite slab; in.^{4} per foot of width. The $\mathbf{I_{av}}$ transformed section analysis is based on steel; therefore, to calculate deflections the appropriate modulus of elasticity to use is 29.5 x 10^6 psi. $\phi \mathbf{M_{no}}$ is the factored resisting moment of the composite slab if there are no studs on the beams (the deck is attached to the beams or walls on which it is resting) inch kips (per foot of width). $\phi \mathbf{V_{nt}}$ is the factored vertical shear resistance of the composite system; it is the sum of the shear resistances of the steel deck and the concrete but is not allowed to exceed $\phi 4(f'_c)^{1/2} A_c$; pounds (per foot of width). The next three columns list the **maximum unshored spans** in feet; these values are obtained by using the construction loading requirements of the SDI; combined bending and shear, deflection, and interior reactions are considered in calculating these values. $\mathbf{A_{wwf}}$ is the minimum area of welded wire fabric recommended for temperature reinforcing in the composite slab; square inches per foot.

	COMPOSITE PROPERTIES												
	Slab Depth	ϕM_{nf} in.k	A_c in²	Vol. ft³/ft²	W psf	S_c in³	I_{av} in⁴	ϕM_{no} in.k	ϕV_{nt} lbs.	Max. unshored spans, ft. 1span	2span	3span	A_{wwf}
22 gage	4.50	40.27	32.6	0.292	34	1.00	4.4	28.13	4270	6.32	8.46	8.56	0.023
	5.00	46.44	37.5	0.333	38	1.18	6.0	33.12	4610	6.03	8.09	8.19	0.027
	5.25	49.53	40.0	0.354	41	1.27	6.9	35.69	4790	5.90	7.93	8.02	0.029
	5.50	52.61	42.6	0.375	43	1.36	7.9	38.29	4970	5.77	7.77	7.86	0.032
	6.00	58.78	48.0	0.417	48	1.55	10.1	43.58	5340	5.55	7.49	7.58	0.036
	6.25	61.87	50.8	0.438	50	1.65	11.3	46.26	5540	5.45	7.36	7.45	0.038
	6.50	64.95	53.6	0.458	53	1.75	12.7	48.97	5730	5.36	7.24	7.32	0.041
	7.00	71.12	59.5	0.500	58	1.94	15.7	54.44	6150	5.18	7.01	7.10	0.045
	7.25	74.21	61.9	0.521	60	2.04	17.4	57.20	6310	5.10	6.91	6.99	0.047
	7.50	77.29	64.3	0.542	62	2.14	19.2	59.97	6480	5.05	6.81	6.89	0.050
20 gage	4.50	48.60	32.6	0.292	34	1.20	4.8	33.77	4560	7.42	9.71	10.03	0.023
	5.00	56.18	37.5	0.333	38	1.42	6.5	39.80	5030	7.07	9.28	9.59	0.027
	5.25	59.96	40.0	0.354	41	1.53	7.4	42.91	5210	6.91	9.09	9.39	0.029
	5.50	63.75	42.6	0.375	43	1.64	8.5	46.05	5390	6.76	8.91	9.20	0.032
	6.00	71.32	48.0	0.417	48	1.87	10.9	52.47	5760	6.49	8.57	8.86	0.036
	6.25	75.11	50.8	0.438	50	1.99	12.2	55.73	5960	6.37	8.42	8.70	0.038
	6.50	78.90	53.6	0.458	53	2.10	13.7	59.02	6150	6.26	8.27	8.55	0.041
	7.00	86.47	59.5	0.500	58	2.34	16.9	65.67	6570	6.05	8.00	8.27	0.045
	7.25	90.26	61.9	0.521	60	2.46	18.7	69.03	6730	5.95	7.87	8.14	0.047
	7.50	94.05	64.3	0.542	62	2.58	20.6	72.41	6900	5.89	7.75	8.01	0.050
19 gage	4.50	55.85	32.6	0.292	34	1.38	5.1	38.67	4560	8.35	10.55	10.91	0.023
	5.00	64.68	37.5	0.333	38	1.63	6.9	45.61	5240	7.94	10.10	10.43	0.027
	5.25	69.10	40.0	0.354	41	1.75	7.9	49.19	5590	7.76	9.89	10.22	0.029
	5.50	73.52	42.6	0.375	43	1.88	9.0	52.83	5790	7.59	9.69	10.01	0.032
	6.00	82.35	48.0	0.417	48	2.15	11.6	60.25	6160	7.29	9.33	9.64	0.036
	6.25	86.77	50.8	0.438	50	2.28	13.0	64.02	6360	7.15	9.16	9.47	0.038
	6.50	91.19	53.6	0.458	53	2.42	14.5	67.83	6550	7.02	9.00	9.30	0.041
	7.00	100.03	59.5	0.500	58	2.69	17.9	75.53	6970	6.78	8.71	9.00	0.045
	7.25	104.44	61.9	0.521	60	2.83	19.8	79.42	7130	6.67	8.57	8.86	0.047
	7.50	108.86	64.3	0.542	62	2.97	21.8	83.33	7300	6.59	8.44	8.72	0.050
18 gage	4.50	62.08	32.6	0.292	34	1.53	5.4	42.99	4560	9.20	11.33	11.71	0.023
	5.00	72.04	37.5	0.333	38	1.81	7.3	50.72	5240	8.75	10.84	11.20	0.027
	5.25	77.02	40.0	0.354	41	1.95	8.3	54.72	5590	8.54	10.62	10.97	0.029
	5.50	82.00	42.6	0.375	43	2.10	9.5	58.78	5950	8.35	10.41	10.76	0.032
	6.00	91.95	48.0	0.417	48	2.39	12.1	67.07	6530	8.01	10.02	10.36	0.036
	6.25	96.93	50.8	0.438	50	2.54	13.6	71.29	6730	7.86	9.84	10.17	0.038
	6.50	101.91	53.6	0.458	53	2.69	15.2	75.55	6920	7.71	9.68	10.00	0.041
	7.00	111.87	59.5	0.500	58	3.00	18.8	84.17	7340	7.44	9.36	9.67	0.045
	7.25	116.85	61.9	0.521	60	3.16	20.7	88.52	7500	7.32	9.21	9.52	0.047
	7.50	121.83	64.3	0.542	62	3.31	22.8	92.91	7670	7.24	9.07	9.38	0.050
16 gage	4.50	62.08	32.6	0.292	34	1.88	6.0	42.99	4560	10.49	12.57	12.99	0.023
	5.00	72.04	37.5	0.333	38	2.22	8.0	50.72	5240	9.96	12.03	12.43	0.027
	5.25	77.02	40.0	0.354	41	2.40	9.2	54.72	5590	9.72	11.78	12.18	0.029
	5.50	82.00	42.6	0.375	43	2.58	10.5	58.78	5950	9.50	11.55	11.94	0.032
	6.00	91.95	48.0	0.417	48	2.94	13.4	67.07	6700	9.11	11.13	11.50	0.036
	6.25	96.93	50.8	0.438	50	3.13	15.0	71.29	7090	8.93	10.94	11.30	0.038
	6.50	101.91	53.6	0.458	53	3.32	16.8	75.55	7490	8.76	10.75	11.11	0.041
	7.00	111.87	59.5	0.500	58	3.71	20.6	84.17	8150	8.45	10.40	10.75	0.045
	7.25	116.85	61.9	0.521	60	3.90	22.8	88.52	8310	8.31	10.24	10.59	0.047
	7.50	121.83	64.3	0.542	62	4.10	25.1	92.91	8480	8.22	10.09	10.43	0.050

Light Weight

2" LOK-FLOOR

38

Load Diagram

RAM Steel v11.2
DataBase: Long Span Joist
Building Code: IBC

10/20/08 19:37:26

Floor Type: LSJ **Beam Number = 34**
Span information (ft): I-End (0.00,26.67) J-End (40.00,26.67)

Load	Dist ft	DL k/ft	LL+ k/ft	LL- k/ft	Max Tot k/ft
W1	0.000	0.440	0.600	0.000	1.040
W2	40.000	0.440	0.600	0.000	1.040

LOADING:

SELF WEIGHT DEAD LOAD: 41 PSF (TABLE)
SUPERIMPOSED DEAD LOAD: 25 PSF
TOTAL DEAD: 66 PSF
LIVE LOAD: 100 PSF

Standard Joist Selection

RAM Steel v11.2
DataBase: Long Span Joist
Building Code: IBC

10/20/08 19:37:26

Floor Type: LSJ **Beam Number = 35**

SPAN INFORMATION (ft): I-End (0.00,33.33) J-End (40.00,33.33)

Joist Size (Optimum) = 32LH14
Total Beam Length (ft) = 40.00

LINE LOADS (k/ft):

Load	Dist	DL	LL	Red%	Type
1	0.000	0.440	0.667	10.0%	Red
	40.000	0.440	0.667		
2	0.000	0.000	0.000	---	NonR
	40.000	0.000	0.000		

Maximum Total Unif. Load at any location (lbs/ft) : 1039.7

Allowable Stress Ratio: 1.00

	Design Loads	Allowable Loads (lbs/ft)
Dead:	440.0	
Live:	599.7	794.1
Total:	1039.7	1045.4

MOMENTS:

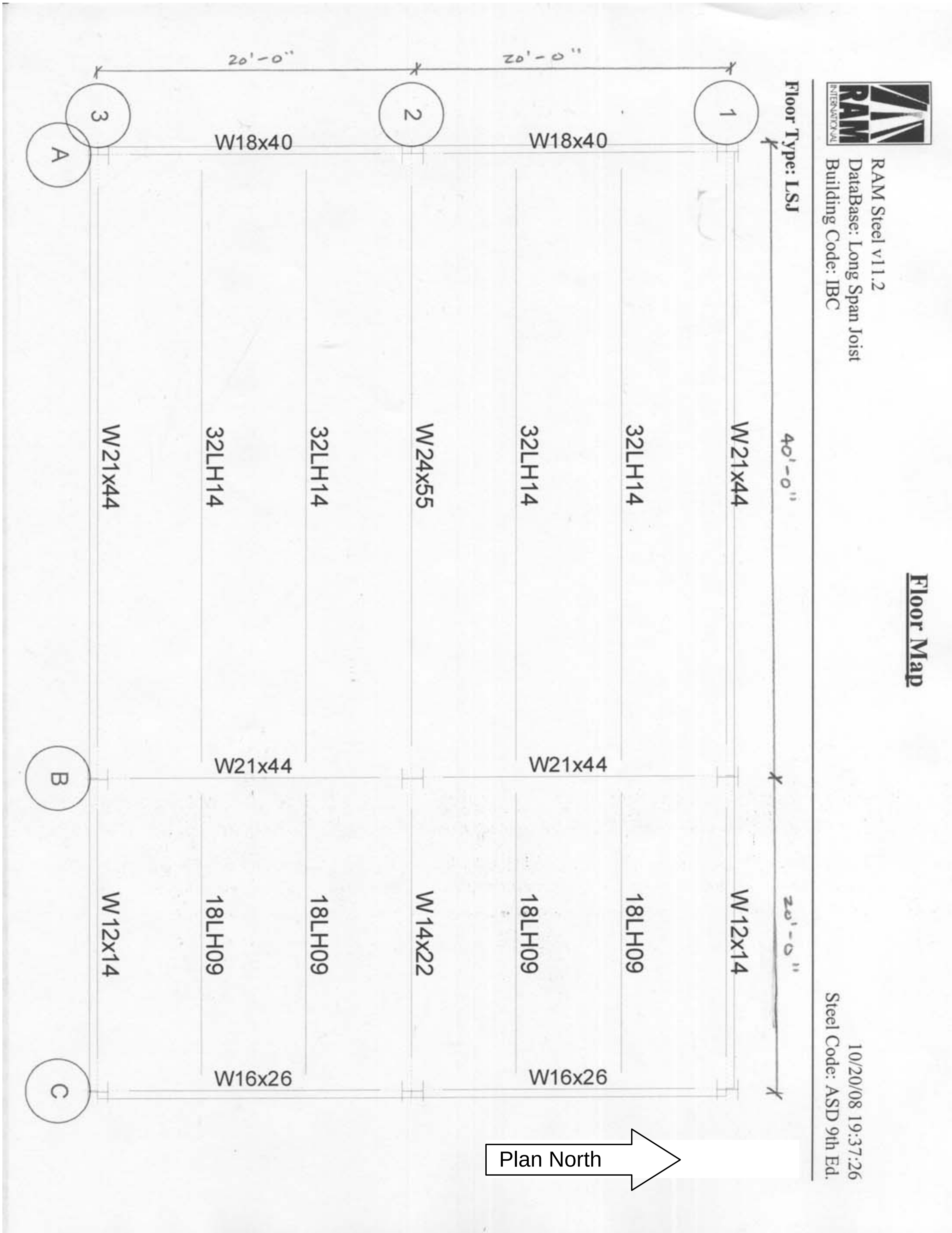
Span	Cond	Moment kip-ft	@ ft
Center	Max +	207.9	20.0

REACTIONS (kips):

	Left	Right
DL reaction	8.80	8.80
Max +LL reaction	11.99	11.99
Max +total reaction	20.79	20.79

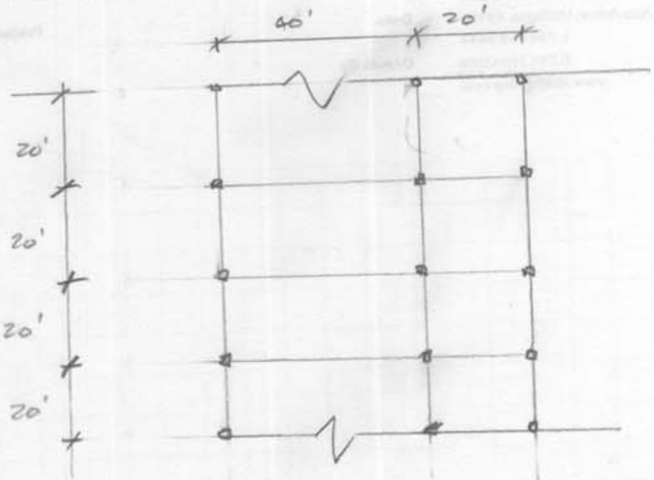
DEFLECTIONS:

Dead load (in)	=	0.739	L/D	=	650
Live load (in)	=	1.007	L/D	=	477
Total load (in)	=	1.746	L/D	=	275


Floor Map
RAM Steel v11.2
DataBase: Long Span Joist
Building Code: IBC
10/20/08 19:37:26
Steel Code: ASD 9th Ed.
Floor Type: LSJ
20'-0"
20'-0"
40'-0"
20'-0"
1
2
3
A
B
C
W18x40
W18x40
W21x44
W21x44
W16x26
W16x26
W21x44
32LH14
32LH14
W24x55
32LH14
32LH14
W21x44
W12x14
18LH09
18LH09
W14x22
18LH09
18LH09
W12x14
Plan North

Alternative Floor System:
Two Way Post-Tensioned Slab

TWO WAY POST-TENSIONED SLAB



2 HR FIRE RATING

CONCRETE

NORMAL WEIGHT: 150 PCF

$f'_c = 5{,}000$ psi

$f'_{ci} = 3{,}000$ psi

REBAR: $f_y = 60{,}000$ psi

PT: UNBONDED TENDONS

$\frac{1}{2}''\phi$ 7-WIRE STRANDS

$A = 0.153\ in^2$

$f_{pu} = 270$ ksi

ESTIMATED PRESTRESS LOSSES = 15 ksi (ACI 18.6)

$f_{se} = 0.7(270\text{ ksi}) - 15\text{ ksi} = 174\text{ ksi}$ (ACI 18.5.1)

$P_{eff} = A*f_{se} = (0.153)(174\text{ ksi}) = 26.6$ kips / TENDON

PRELIM SLAB THICKNESS

START WITH $L/h = 45$

LONGEST SPAN = 40'

$h = (40')(12)/45 = 10.67 = 11''$ SLAB

LOADING

DEAD LOAD:

SELFWEIGHT: $(11'')(150\text{ pcf}) = 138$ PSF

SUPERIMPOSED: 25 PSF

LL = 100 PSF

LIVE LOAD REDUCTION

$A_T = (40)(20) = 800\ ft^2$

$K_{LL} = 1$

$LL_{40x20} = \left(0.25 + \frac{15}{\sqrt{1(800)}}\right)(100) = 78$ PSF

$LL_{20x20} = \left(0.25 + \frac{15}{\sqrt{400}}\right)(100) = 100$ PSF (NO REDUCTION)

SECTION PROPERTIES (CLASS U)

$A = bh = (240 in)(11 in) = 2640\ in^2$

$S = \frac{bh^2}{6} = \frac{(240)(11)^2}{6} = 4840\ in^3$

DESIGN PARAMETERS (CLASS U)

AT TIME OF JACKING

$f'_{ci} = 3{,}000$ PSI

COMPRESSION $= 0.6(3000) = 1800$ PSI

TENSION $= 3\sqrt{f'_c} = 3\sqrt{3000} = 164$ PSI

AT SERVICE LOADS

$f'_c = 5000$ PSI

COMPRESSION $= 0.45 f'_c = 0.45(5000) = 2250$ PSI

TENSION $= 6\sqrt{f'_c} = 6\sqrt{5000} = 424$ PSI

AVERAGE PRECOMPRESSION LIMITS

$P/A = 125$ PSI MIN (ACI 18.12.4)

TARGET LOAD BALANCES

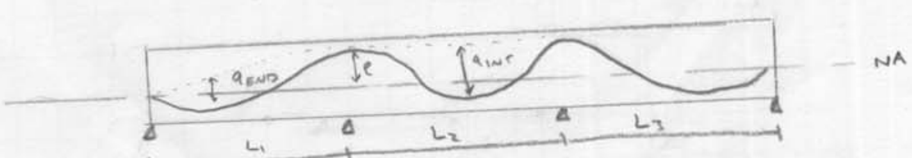
$0.75\ w_{DL} = 0.75(138) = 104$ PSF

COVER REQUIREMENTS (2 HR FIRE RATING)

RESTRAINED SLABS: 3/4" BOTTOM

UNRESTRAINED SLABS: 1/2" BOTTOM

3/4" TOP



TENDON ORDINATE	TENDON (CG) LOCATION
EXTERIOR SUPPORT - ANCHOR	5.5"
INTERIOR SUPPORT - TOP	10"
INTERIOR SPAN - BOTTOM	1"
END SPAN - BOTTOM	1.75"

(FROM BOTTOM OF SLAB)

$a_{INT} = 10" - 1" = 9"$

$a_{END} = (5.5 + 10)/2 - 1.75 = 6"$

PRESTRESS FORCE REQUIRED TO BALANCE 75% OF SELF WT DL

$w_b = 0.75\, w_{DL} = 0.75(138)(20') = 2070$ PLF $= 2.07$ KLF

FORCE NEEDED IN TENDONS

$P = w_b L^2 / 8\, a_{END}$

$= (2.07)(40')^2 / (8(6''/12)) = 828^K$

CHECK PRECOMPRESSION ALLOWANCE

\# OF TENDONS NEEDED

$828^K / 26.6^K/_{TENDON} = 31.12 \approx 31$ USE TENDONS

$P_{ACTUAL} = (31)(26.6^K) = 825^K$

$w_b = (.825/828)(2.07) = 2.06$ KLF

ACTUAL PRECOMPRESSION STRESS

$P_{ACTUAL} / A = 825^K (1000) / (2640\ in^2) = 312.5 > 125^K$ ✓

20' SPAN FORCE

$P = (2.07\ KLF)(20')^2 / (8(9/12)) = 138^K$

$138^K < 828^K$

LESS FORCE REQ'D IN 20'x20' BAY

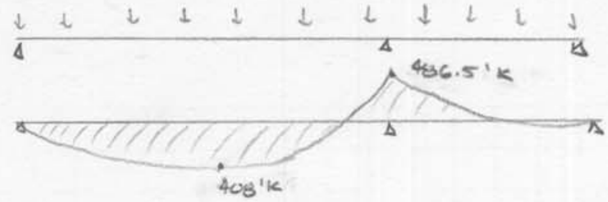
EFFECTIVE PRESTRESS FORCE, $P_{EFF} = 825^K$

3

SLAB STRESSES

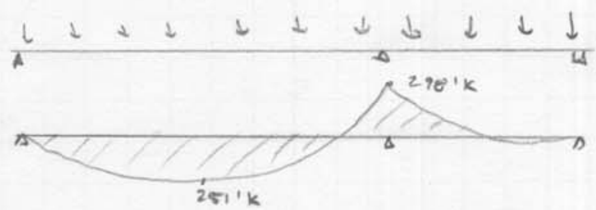
$w_{DL} = (163)(20')/1000 = 3.26$ K/FT

DL MOMENTS



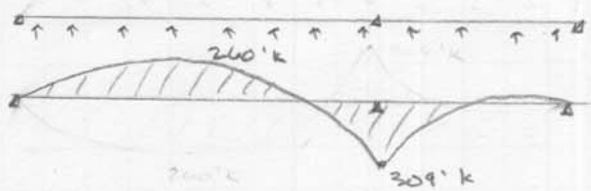
$w_{LL} = (100 PSF)(20')/1000 = 2$ K/FT

LIVE LOAD MOMENTS



$w_b = 2.07$ K/FT

TOTAL BALANCING MOMENT



* ALL MOMENT DIAGRAMS TAKEN FROM STAAD ANALYSIS

STAGE 1: STRESSES IMMEDIATELY AFTER JACKING

MIDSPAN STRESS

$f_{top} = [(-408 + 260)(12)(1000)]/4840 - 312.5$

$= -679$ psi (c) $< 0.6 f'_{ci} = 1800 \therefore$ ✓

$f_{bot} = [(408 - 260)(12)(1000)]/4840 - 312.5$

$= 54.4$ psi (t) $< 3\sqrt{f'_{ci}} = 164$ psi $\therefore$ ✓

SUPPORT STRESS

$f_{top} = [(486 - 309)(12)(1000)]/4840 - 312.5$

$= 126$ psi (t) $< 3\sqrt{f'_c} = 164$ psi $\therefore$ ✓

f_{bot} $[(-486 + 309)(12)(1000)]/4840 - 312.5$

$= -751.3$ psi (c) $< 0.6 f'_{ci} = 1800$ psi $\therefore$ ✓

STAGE 2:

MIDSPAN STRESS

$f_{top} = [(-408 - 251 + 260)(12)(1000)/4840 - 312.5$

$= -1302$ psi (c) $< 0.45 f'_c = 2250 \therefore$ ✓

$f_{bot} = [(408 + 251 - 260)(12)(1000)]/4840 - 312.5$

$= 677$ psi (t) $> 6\sqrt{f'_c} = 424$ psi ✗

CRACKS, NEEDS FURTHER ANALYSIS

SUPPORT STRESSES

$f_{top} = [(487 + 298 - 309)(12)(1000)]/4840 - 312.5$ ✗

$= 868$ psi (t) $> 6\sqrt{f'_c} = 424$ psi

CRACKS, NEEDS FURTHER ANALYSIS

ULTIMATE STRENGTH

FACTORED MOMENTS

PRIMARY POST-TENSIONING MOMENTS

$M_1 = P * e$

$e = 0$ @ EXT SUPPORT

$e = 4.5"$ @ INTERIOR SUPPORT

$M_1 = 825^k(3)/12 = 206'k$

SECONDARY POST-TENSIONING MOMENTS

$M_{sec} = M_{bal} - M_1$

$= 309 - 206 = 103'k$ @ INT SUPPORT

$M_u = 1.2 M_{DL} + 1.6 M_{LL} + 1.0 M_{SEC}$

@ MIDSPAN

$M_u = 1.2(408) + 1.6(251) + 1.0(51.5) = 943'k$

@ SUPPORT

$M_u = 1.2(487) + 1.6(298) + 1.0(103) = 1164'k$

MINIMUM BONDED REINFORCEMENT

POSITIVE MOMENT REGION

MID SPAN

$f_t = 677$ psi (t) $> 2\sqrt{f'_c} = 141$ psi

$y = f_t/(f_t + f_c)h = [677/(677+1302)](11) = 3.76"$

$N_c = M_{DL+LL}/S * 0.5 * y * l_2 = [(408+251)(12)/4840](0.5)(3.76)(20)(12)$

$N_c = 818k$

$A_{s,min} = N_c / 0.5 f_y = 818^k/(0.5(60ksi)) = 27.3 in^2$

$27.3 in^2 / 40' = 0.683 in^2/FT$

USE #8 BARS @ 12" OC @ BOTTOM

$= 0.79 in^2/FT > 0.683 in^2/FT$

6

NEGATIVE MOMENT REGION

INTERIOR SUPPORT

$A_{cf} = (11")(30')(12) = 3960 \text{ in}^2$

$A_{s,min} = 0.00075(3960) = 2.97 \text{ in}^2$

USE 10 - #5 = $3.10 \text{ in}^2 > 2.97 \text{ in}^2$

EXTERIOR SUPPORT

$A_{cf} = 11(20)(12) = 2640 \text{ in}^2$

$A_{s,min} = 0.00075(2640) = 1.98 \text{ in}^2$

USE 10 - #4 = $2.00 \text{ in}^2 > 1.98 \text{ in}^2$

+ MUST SPAN A MIN. OF 1/6 THE CLEAR SPAN ON EACH SIDE OF SUPPORT
+ AT LEAST 4 BARS REQUIRED IN EACH DIRECTION
+ PLACE BARS WITHIN 1.5(11") = 16.5" AWAY FROM THE SUPPORT ON EACH SIDE
+ MAX BAR SPACING IS 16.5"

MIN REINFORCEMENT CHECK FOR ULTIMATE STRENGTH

$A_{ps} = 0.153 \text{ in}^2 (31) = 4.74 \text{ in}^2$

$f_{ps} = 174000 + 10000 + (5000)(20 \times 12) d / (300(4.74))$

$f_{ps} = 184000 + 844d$

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

@ SUPPORTS

$d = 11" - 3/4" - 1/4" = 10"$

$f_{ps} = 184000 + 844(10) = 192440 \text{ psi}$

$a = [(3.10)(60 \text{ ksi}) + (4.74)(192 \text{ ksi})] / [(0.85)(5)(20)(12)] = 1.07$

$\phi M_n = 0.9[(3.10)(60) + (4.74)(192)][10" - 1.07/2] / 12 = 778'k$

$778'k < 1164'k$ SO REINFORCEMENT FOR ULTIMATE STRENGTH GOVERNS

10 - #5 TOP @ INT SUPPORT
10 - #4 TOP @ EXT SUPPORT

@ MIDSPAN

$d = 11" - 1.5" - 1/4" = 9\,1/4"$

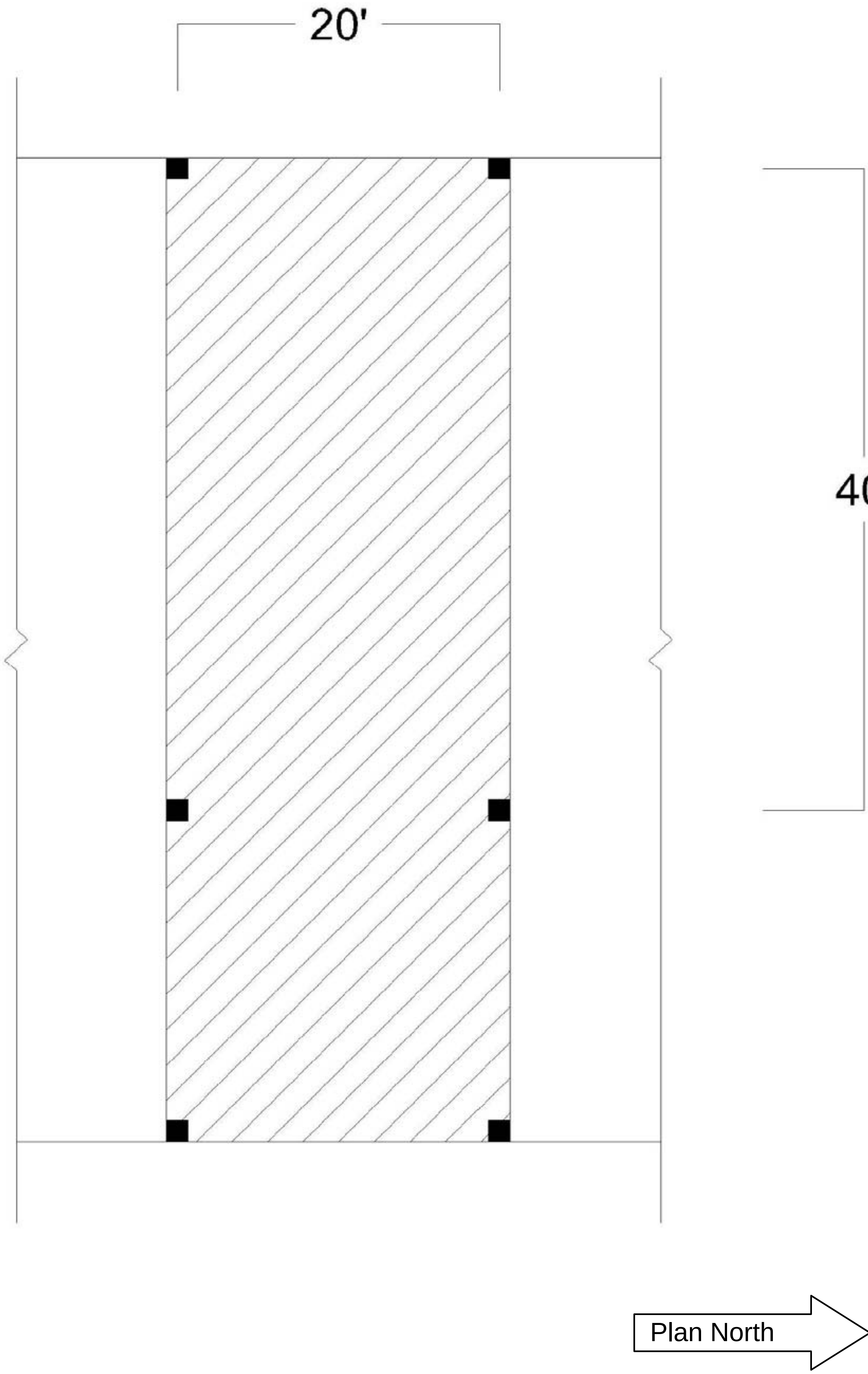
$f_{ps} = 184{,}000 + 844(9.25") = 191807 \text{ psi}$

$a = [(27.3 \text{ in}^2)(60 \text{ ksi}) + (4.74)(191.8)] / [0.85(5)(20)(12)] = 2.49$

$\phi M_n = 0.9[(27.3)(60) + (4.74)(191.8)][9.25 - 2.49/2] / 12 = 1529'k$

$1529'k > 963'k$ ∴ MIN REINFORCEMENT OK

#8 BARS @ 12" OC ON BOTTOM


20'
40'
Plan North