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OCTOBER 27, 2006

555 12TH STREET
OAKLAND, CALIFORNIA

Technical Report #2:
Pro-Con Structural Study of Alternate Floor Systems

Executive Summary

The purpose of this report is to analyze alternate floor framing systems and determine if they are a viable alternative to the existing system. The existing floor system consists of 2 ½" concrete slab over 3" deck that act compositely with the steel frame. Beams sized as W18's, span 43.5' in the critical bay and are supported by W24-W27 girders. Alternative floor systems were compared to one another based on several criteria, and are listed below. Most important were weight of the system, ability to span long distances, ease of construction, and cost. Secondary were floor system depth, fireproofing, and vibration control.

- Concrete Pan Joist
- Precast Hollowcore
- Non-Composite Slab/Steel
- Composite Joists

After analysis, concrete and precast systems were determined to be the least efficient. While they are inherently fire resistant, and do not require fireproofing, they are inefficient in spanning long distances. This long span is needed for the column free floor plan. Also, the weight of the system is unfavorable in the high seismic region. Foundation and lateral system redesign would be required, which can be very costly. There benefits were in vibration control and relative cost. Non-composite steel joists offer long spans, lighter weight than concrete, and ease of construction. No shear studs are required in this system which saves cost and time over the existing. The members are larger which increases foundation loads, and costs, but is worth further investigation. Lastly, were the composite joists systems which offer a light weight and efficient system, and is worth further investigation. Joists are able to span long distances and carry the floor load. These are relatively expensive compared to the other systems, because of the need for shear studs. However, reduction in overall weight of the building will reduce foundation and lateral system costs.

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Introduction

555 City Center is a 21 story office building located in the heart of downtown Oakland. It was designed using the Uniform Building Code, but I have analyzed it with the International Building Code 2003. This code references the ANSI's minimum design loads for buildings, which are provided below for live loads. The dead loads were calculated by myself, and are used for this technical assignment. Combined, these gravity loads will be used in determining alternate floor systems for a critical bay on a typical floor in the building. The entire typical floor framing is shown in appendix.

Live Loads: Taken from table 1607.1 from IBC 2003
- Office Floor: 80 psf – Assume any spot could be a corridor at some point
- Partitions: 20 psf – Assume 10 psf for seismic calculations

Dead Loads: (Assumed) waiting for confirmation
- Metal Deck: 2 psf
- Reinforced Concrete: 150 pcf (includes reinforcing)
- Steel Structural Members: 15 psf
- MEP: 10 psf
- Collateral: 5 psf

Design References Used:

- Nitterhouse Precast Concrete Literature
- CRSI – Concrete Reinforcing Steel Institute
- LRFD Manual of Steel Construction – Third Edition
- Vulcraft Steel Joist Catalogs
- RS Means Assemblies Catalog 2002

Existing Structural System

The elevated floors, starting from level 2, are composite metal deck systems. Typical floors 3-21 are 3" 18 gage composite deck with 2 ½" of normal weight concrete cover. The slabs are reinforced by either #6 @ 13" EW or WWF6x6 W1.9. The majority of the structural system is steel framing. All structural framing steel is designated as ASTM A992, Gr 50, unless otherwise noted. The building takes advantage of two lines of symmetry, one in the N-S direction, and the other in the E-W direction. The typical floors, 2-21, have the same framing, unless otherwise noted.

On a typical floor, composite steel deck spans the beams and girders. Beams are sized as W18x55 and W18x60 and span 43'-6". The girders are sized W18 – W27, and span 30 – 35', depending on location in the bay analyzed. The overall max depth of the floor system is 26.7" for a W27, plus an additional 5.5" for the composite deck, for a total of 32.2". For this report, one critical bay was looked at between column lines X4-D2 and 5-6. It is not a square bay, but it is analyzed as a 30'x 43.5' bay for simplicity. Beams are spaced 10' on center in the long span. The original and analyzed bay layout is shown below.
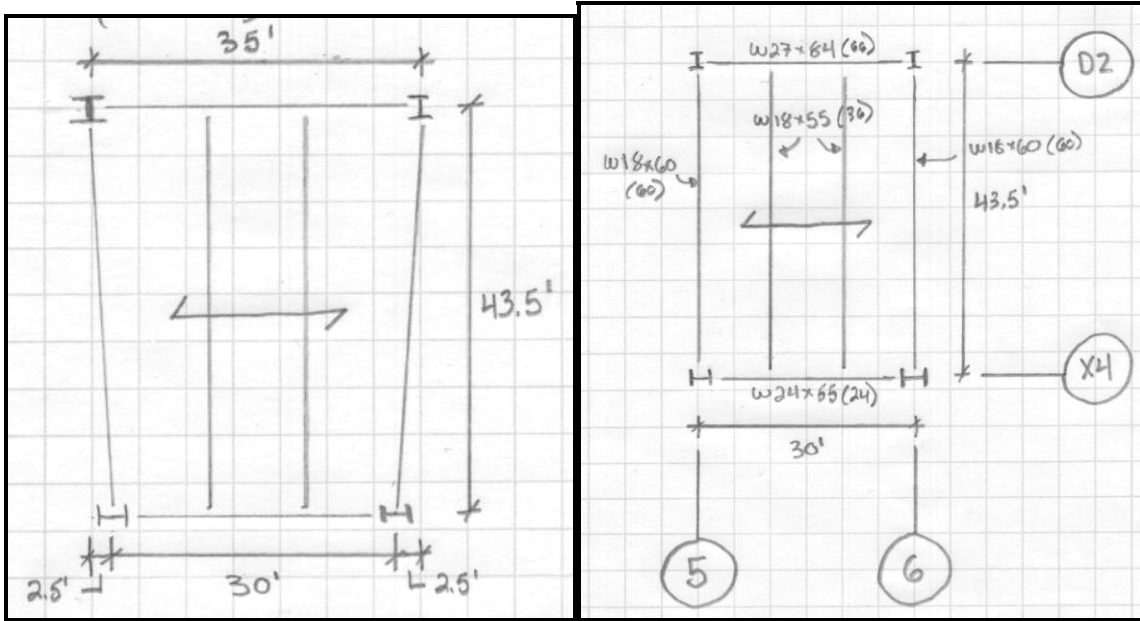
Existing Bay   **Analyzed Bay**

Design Considerations

When identifying possible alternative floor systems, several factors were taken into account
- Must be able to span long distances to keep column free spaces
- Weight of system
- Fire rating
- Depth of system
- Ease of construction
- Price

Alternative Floor Systems
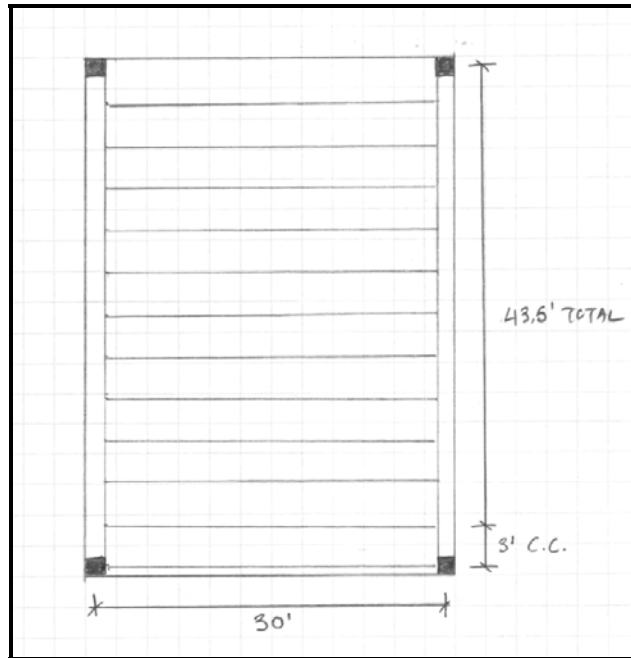
Concrete Pan Joists

Concrete pan joist, also known as one-way joist floor systems, consist of evenly spaced concrete joists spanning in one direction. A reinforced concrete slab is cast integral with the joists to form a monolithic floor system. The one way joists frame into beams that span between the columns, perpendicular to the joists. These systems are effective for column layouts short span in one direction and long span in the orthogonal. It is preferable to span the joists in the short direction in rectangular bays, to achieve maximum economy.

Fire resistance is inherently provided by the concrete, so no additional fireproofing is required for the system. The joists also provide adequate stiffness over long spans and minimize structural depth of the system from 32" to 20.5". The formwork for the system is the majority of the cost at 58%. However, because of the redundancy of the system, the formwork can be reused over and over again to reduce construction costs. These costs become an issue because of the need to set up formwork, set time for bars, waiting for concrete to cure, shoring, and teardown of formwork. The increased system weight is both good and bad. While the added weight helps to reduce floor vibrations and noise, it also requires the columns to become larger, and ultimately the foundations. Seismic issues also arise because the diaphragm has changed. Also, the extra weight of the concrete will increase the base shear on the building, and require a new lateral system analysis.

The system was designed, using the CRSI Design Guide for standard one-way joists, to span the 30' distance. A 30" form with 6" ribs @ 36" c.c, with a total depth oh 20.5" was selected. The ribs are 16" deep and covered with a 4.5" top slab. Number 5 bars @ 9" on top, and (1) #6 and (1) #7 in the bottom provide the adequate reinforcing. A joist band beam was not designed, but one would be required to span perpendicular to the joists along the column line.
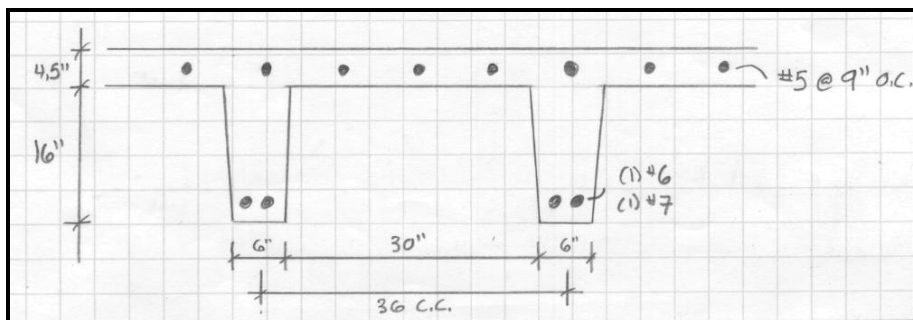
Pan Joist Cross Section


Pan Joist Framing

Precast Hollowcore Plank

Precast hollowcore planks were chosen for another alternative floor system from the Nitterhouse design literature. Precast concrete offers several advantages over the original floor system. They are cast in a controlled environment which increases their quality over cast in place concrete. They are also prestressed which gives them a higher capacity and span length than normal weight concrete. Also, being concrete, the system is inherently fire resistant, but the wide flange beams they rest on, are not. They must be fireproofed, or replaced with inverted tee beams as the structural support. The planks also offer superior durability, natural sound attenuation, and vibration control.

There are several disadvantages of precast, including transportation cost and lead time. Precast must be ordered in advance, and transported to site. Transportation costs can become relatively expensive if the manufacturer does not operate locally. Because the planks are very heavy, 102.5 psf for those selected, this limits the amount that can be carried to site in one load. This will also increase transportation costs and project schedule. However, once on site, the construction is extremely fast. Only a 2" topping is needed to be placed before it is complete. There is no curing time, shoring, and formwork needed which will lead to a decrease in project schedule. The weight of the system will also impact the design of the lateral system and increase foundation size, which increases project cost.

The hollowcore selected comes in 4' pieces and spans 30' in the bay. A 12" x 4' Spandeck-UL-J917 with a 2" cast in place topping, with a capacity of 187 psf was

selected. The reinforcing pattern consists of 6 – 1/2" diameter, 270 K Lo-Relaxation strands. The planks span between two girders, both sized to be W40x149. However, a W24x192 may be selected as an alternate size. It is heavier but reduced floor system depth. The overall depth of the system including the plank is 52.2" for W40 and 39.5" with the W24. Realistically, another girder, or column line should be added in order to decrease the size and weight of the beams used to carry the planks.
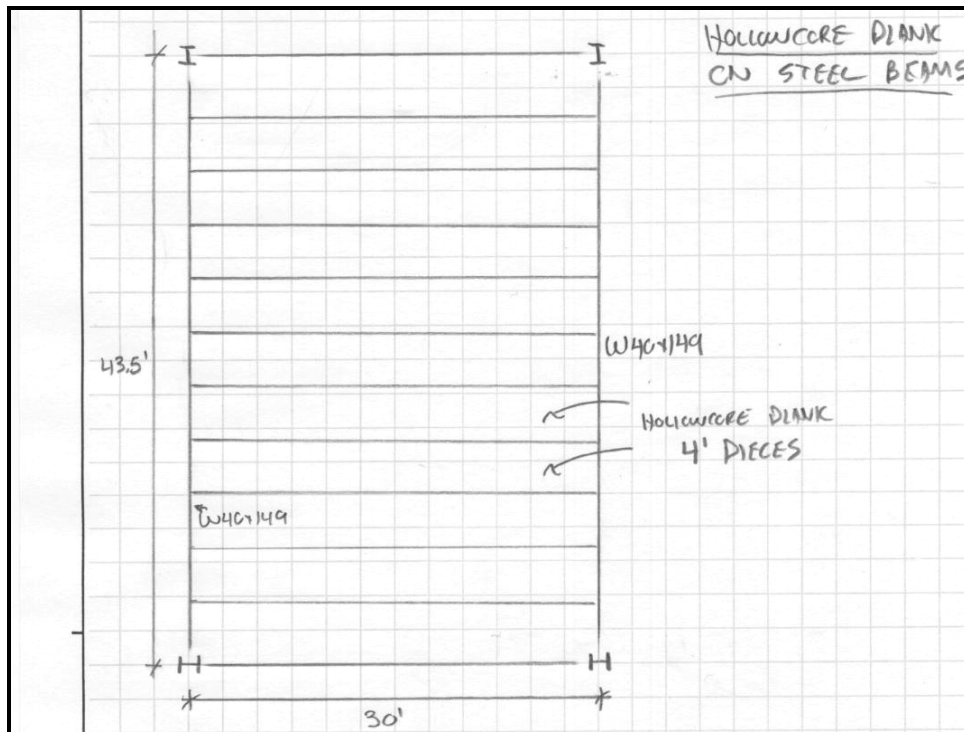
**Precast Hollowcore Framing**
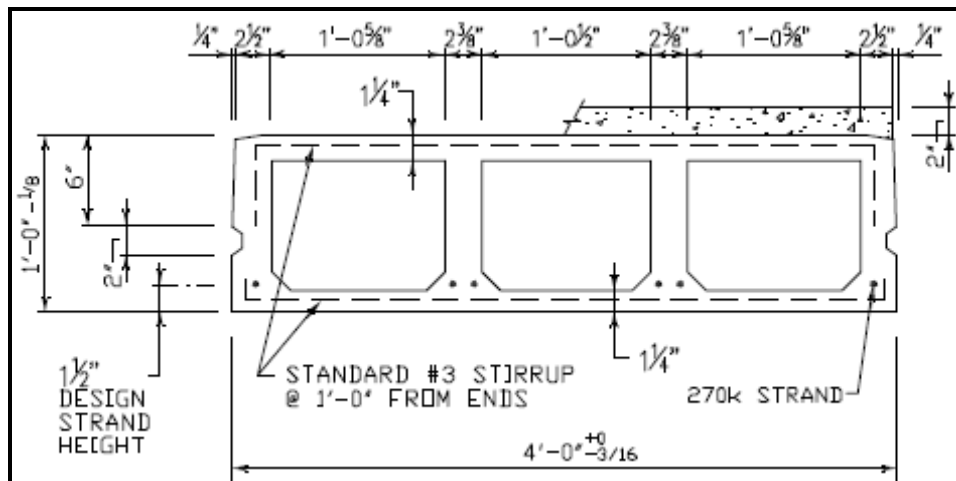
Hollowcore Plank Cross Section

Non-Composite Slab and Steel

This system is just like the original, except that the beams and girders do not have composite action with the slab. This means that the beam will take all the moment, and not transfer any to the slab. The locations of the beams are every 10', and span the long direction. A 3" deck with 2 ½" concrete cover spans perpendicular to the beams. The benefit of this system is its ease of construction. Shear studs are no longer needed to help transfer load, so there is a material and labor cost savings. Welders are not needed to be on site as often, and no time is needed to install. Schedule decreases and floors go up quicker. Another benefit of steel is its reduction of dead load in the building compared to concrete.

Steel systems require spray on fireproofing or encasements to achieve necessary fire ratings. This is an expensive added cost. Also, because the beams act alone from the concrete, they will become larger to accommodate the added moment. This increased weight increases column size, which increases foundations and cost. It also affects seismic issues with the lateral system negatively. Vibration control is more of a problem with steel than concrete because of weight. Steel is more susceptible to floor vibrations and sound attenuation.

Beam sizes showed an increase in size as expected. They jumped from W18x55 to W27x84, controlled by total load deflection. The girders, however, remained relatively unchanged from a W24x55 to a W24x68. More calculations would need to be done to determine the appropriate size of the girder compared to the beam it is supporting.
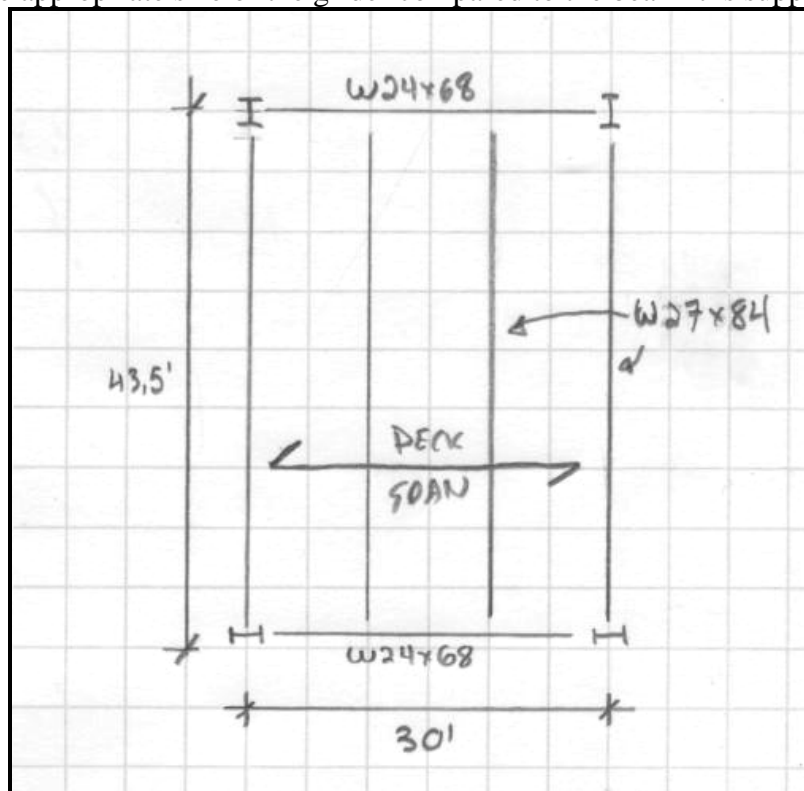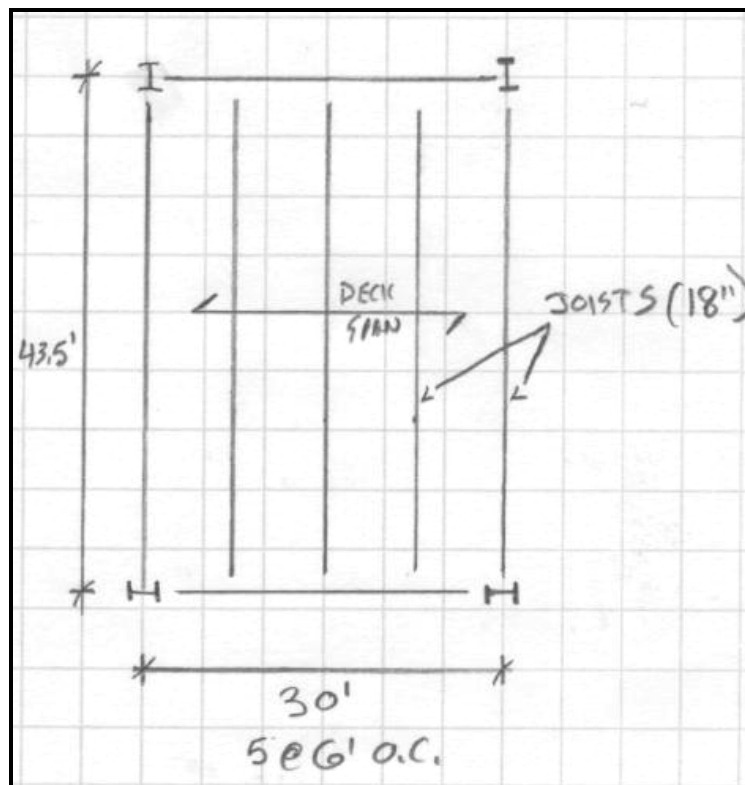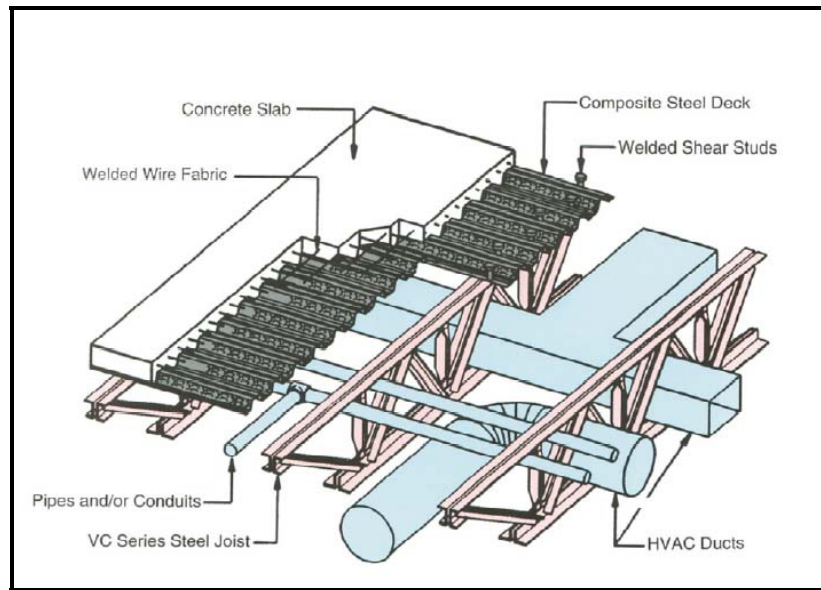

Non-Composite Framing

Composite Steel Joists

This system is ideally suited for commercial construction where large column free areas are desired. This system takes advantage of the interaction between concrete slab and steel joists through shear studs like the original steel system, resulting in smaller members. Floor to floor height is reduced because mechanical systems can be routed through the open webs of the joists. However, shear stud installation is labor intensive and expensive to the project. Erection and connections are simplified due to standard end bearing seats which mean faster construction. Weight savings over large W-shapes reduces building weight, which reduces foundation size and cost. It also reduces the overall base shear and seismic reaction on the lateral system. Floor vibrations can be a problem with steel structures, but over the long spans provided by the joists, it is reduced.

Joists were designed and chosen from the Vulcraft Composite Joist and Decking Catalogs online. In the analyzed bay, the joists are spaced every 6' o.c. and span 43.5'. The existing decking, 2 ½" concrete over 3" deck, was used to determine a correct joist size. However, once picked from the chart it was given that the decking could be made smaller to a 2" deck, for a total of 4.5". The joist designation found was an 18VC 1200/600/450 with 32- ¾" diameter shear studs per span. This is an 18" joist with 4.5" decking, for a total of only 22.5" depth. Two rows of bridging between joists is needed to prevent movement and rotation. The loads from the joists were then used to determine a girder size to support them. A W24x55 was calculated to adequately carry the joists.

Composite Steel Joist Framing

Composite Joist with MEP through web

Comparison of Alternative Floor Systems

| Framing System	Advantages	Disadvantages	Cost/ Sq. foot	System Weight	Further Investigation
Existing Composite Beams	Column free space Lightweight Beam Layout Flexible	Cost + Labor Fireproofing	$20	Light	Y
Concrete Pan Joist	Small depth Vibration Control	Weight - Seismic Trouble spanning bay Foundation redesign	$15-16	Heavy	N
Precast Hollowcore	Cost Quality Product	Weight - Seismic Large Depth Foundation redesign	$10	Heaviest	N
Noncomposite Slab/Steel	Ease of Construction Beam Layout Flexible	Large Members Vibration	$15	Medium	Y
Composite Joists	Column free space Lightweight Ease of Construction	Cost + Labor Fireproofing	$18-19	Lightest	Y

Conclusions

After analysis, it was found that any of the alternative systems could work as floor systems in the building. However, some are far more efficient and better suited for a high rise office building in a high seismic region. The concrete pan joist system can be ruled out as an alternative and further investigation. While it does offer small system depth, it is not enough to make up for the shear weight of the system, and the foundation and lateral system redesign that comes with it. Also, it would be difficult to span a band beam 43.5' under the presented loading. An extra set of columns would need to be installed, which would go against the column free floor plan. Precast hollowcore plank can be ruled out as well. While it is relatively cheap and easy to construct compared to the other systems, its weight disqualifies it as a viable alternative. Foundation redesign and lateral system redesign would result, like with pan joists. Non-composite slab system with steel beams provides the column free spaces the building needs. Although the beams are larger than the composite system, they do not require shear studs. Shear stud installation is costly and labor intensive. Schedule of the project would be shortened, and in turn, the cost. Lastly, is the option of using composite open web steel joists. The joists are relatively lightweight compared to the existing system and still span the required 43.5'. They provide the column free space needed, and are fairly easy to construct. While vibration can be an issue in steel forms, the long span of the joist helps to minimize the effects. For these reasons, it is a viable alternative floor system.

Appendices

A. Existing Framing Plan

B. Composite Beams Calcs/Charts

C. Concrete Pan Joists Calcs/Charts

D. Precast Hollowcore Calcs/Charts

E. Non-Composite Slab and Steel Calcs/Charts

F. Composite Steel Joists Calcs/Charts

A. Existing Framing Plan (Bay chosen shaded)
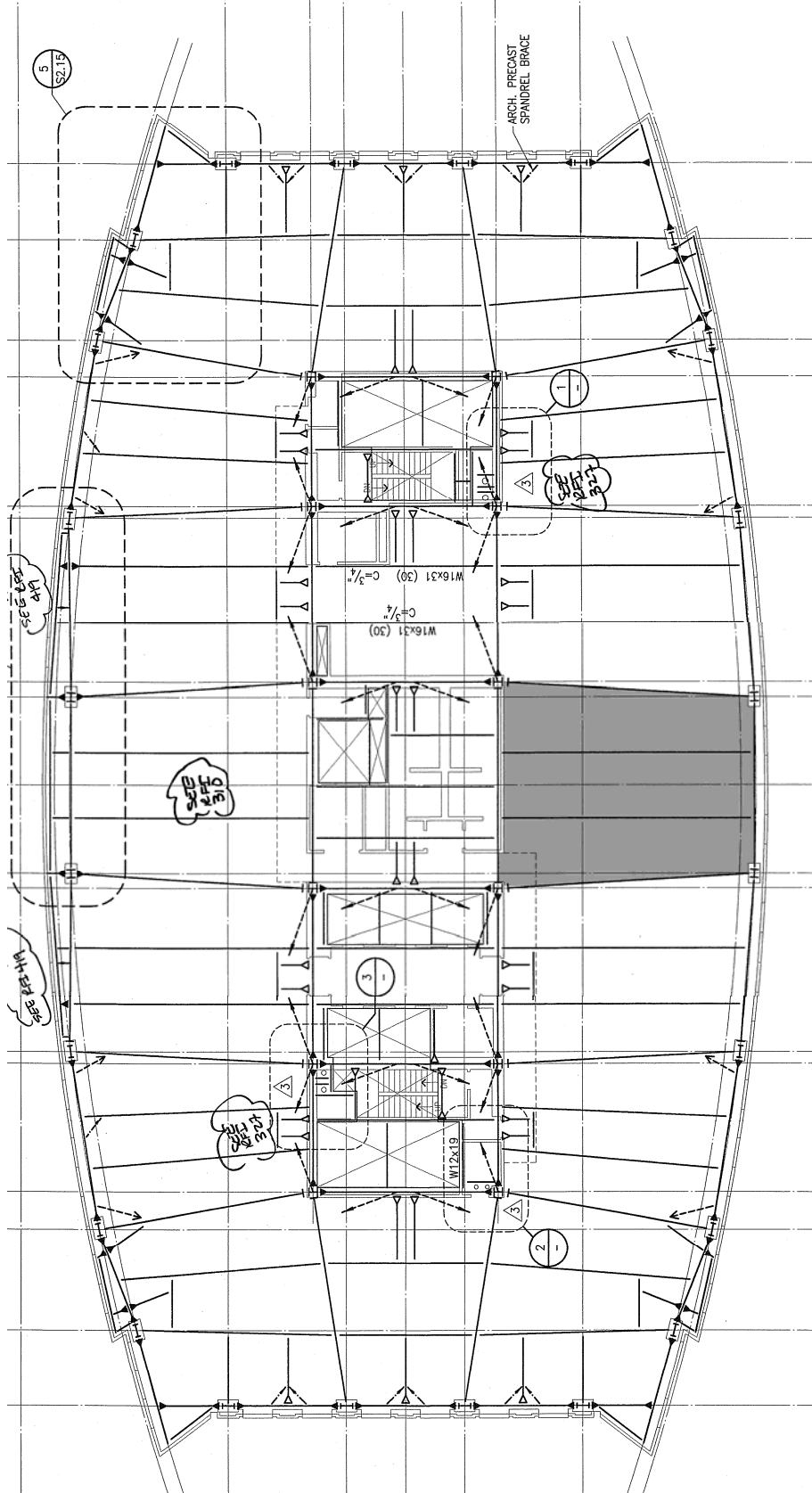
B. Composite Beams (Current System)

CURRENT SYSTEM - COMPOSITE DECK ON STEEL FRAME

30' x 43.5'

* BAY WAS CHECKED IN TECH #1 REPORT

DESIGNED:
B1: W18x55 (36) SHEAR STUDS ($3/4''$ d)
G1: W24x55 (24) SHEAR STUDS ($3/4''$ d)

RECHECK:

BEAM B1: DL = 100 PSF $\Rightarrow$ $w_u = (1.2(100) + 1.6(100)) 10' = 2.8$ KLF $= w_u$
LL = 100 PSF

$$M_u = \frac{2.8 \text{ KLF} (43.5)^2}{8} = 662'k$$

$b_{eff} = 1/4$ SPAN $= 1/4 (43.5) = 130.5''$
$=$ spacing $= 10'(12) = 120'' \leftarrow$ CONTROLS

USE LRFD MANUAL (TABLE 5-14) - ASSUME W18x55

$C_c = .85(5.5)(120)(3) = 1683$ KSI   STEEL CONTROLS
$T_s = (16.2 in^2)(50 KSI) = 810$ KSI

$a = \frac{810}{(.85)(5.5)(120)} = 1.44 = Y_1$ $Y_2 = 5.5 - \frac{1.44}{2} = 4.78$

TABLE 5-14 pg 5-142 $\Rightarrow$ @ $Y_2 = 4.5$ (CONSERVATIVE)   PNA @ 3
$\phi M_n = 699'k > 662'k$ $\therefore$ 'OK
$\Sigma Q_n = \frac{573}{21^k/STUD} = 27 \times 2 = 54$ SHEAR STUDS

OR $Y_2 = 5.0$, PNA @ 4 $\phi M_n = 674 > 662'k$ $\therefore$ OK
$\Sigma Q_n = \frac{454^k}{21^k/STUD} = 22$ STUDS $\times 2 = 44$ SHEAR STUDS

$\therefore$ W18x55 WORKS   OK FOR BOTH

30 by 38 blocks at .25 inches

13

CHECK GIRDER G1: $w_u = 1.2(100) + 1.6(100) = 280$ PSF

W24×55 $w_u = \frac{43.5}{2}(280 \text{ PSF}) = 6.1$ KLF

$$M_u = \frac{6.1 \text{ KLF}(30')^2}{8} = 686' \text{k}$$

$b_{eff} = 1/4 \text{ SPAN} = 1/4(12)(30) = 90 \leftarrow$ CONTROLS

$= \text{SPACING} = \frac{43.5}{2}(12) = 261$

$C_c = 0.85(3)(5.5)(90) = 1262$ KSI   STEEL CONTROLS

$T_s = 16.3 \text{in}^2(50) = \quad 815$ KSI

$a = \frac{815}{.85(5.5)(90)} = 1.94 = 4,$ $Y_2 = 5.5 - \frac{1.94}{2} = 4.53$

TABLE 5-14 (pg. 5-141) $\Rightarrow$ @ $Y_2 = 4.5$, PNA @ 6 (CONSERVATIVE)

$\phi M_n = 765' \text{k} > 686' \text{k} \therefore$ OK

$\Sigma Q_n = \frac{332^k}{21^k/\text{STUD}} = 16 \times 2 = \underline{\underline{32 \text{ STUDS}}}$

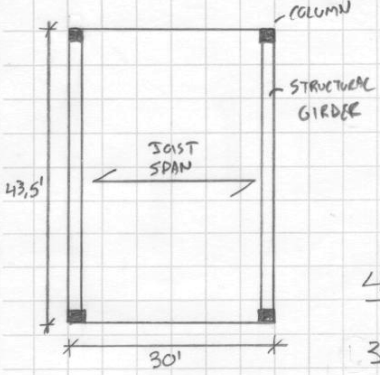SAME STEEL, MORE STUDS (CONSERVATIVE WITH Y_2) $\therefore$ OK

COMPOSITE SLAB/DECK AND BEAMS/GIRDERS ARE
COMPARABLE TO IN-SITU DESIGN. ONLY DIFFERENCE
IS SLIGHTLY MORE SHEAR STUDS BECAUSE I WAS
CONSERVATIVE.

30 by 38 blocks at .25 inches

C. Concrete Pan Joists

CONCRETE PAN JOIST GIRDERS (USE CRSI CHAPTER 8)
- STANDARD ONE-WAY JOISTS

LOADING: Dead Load = 30 PSF   > 1.4 D + 1.7L = 1.4(30) + 1.7(100) = 212 PSF
         Live Load = 100 PSF

TRY SPANNING FROM SHORT DIRECTION (30')

- COLUMN
- STRUCTURAL GIRDER

JOIST SPAN

43.5'

30'

f'_c = 4000 PSI
CRSI: END SPAN F_y = 60,000 PSI

*PICK ABOVE HORIZONTAL LINE, AND
DEFLECTION CALCULATION NOT REQUIRED.
(thickness $\geq \ell n/18.5$)
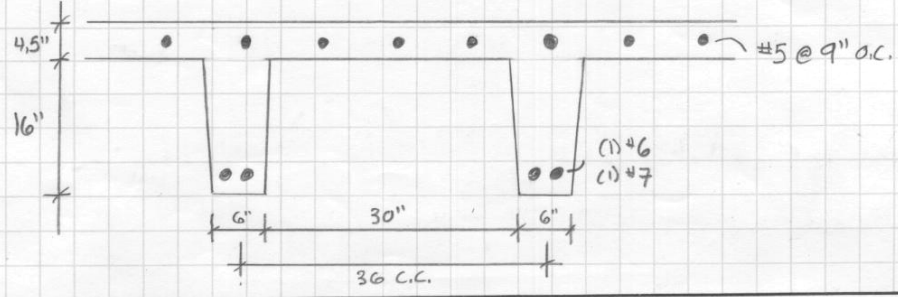
4 CHOICES WILL WORK

30+6 w/ Depth 20+3
      w/ Depth 16+4.5
30+7 w/ Depth 20+3
      w/ Depth 16+4.5

* CHOOSE MINIMAL DEPTH AND LEAST STEEL
  - 30" FORM = 6" RIB @ 36" C-C.   DESIGN LOAD = 222 PSF > 212 PSF ∴ OK
  Depth = 16" RIBS + 4.5" TOP SLAB = 20.5" TOTAL DEPTH
  REINFORCING: TOP = #5 @ 9"
               BOT = (1) #6 AND (1) #7 A_s = 1.44 PSF

4.5"

16"

#5 @ 9" O.C.

(1) #6
(1) #7

6"   30"   6"

36 C.C.

30 by 38 blocks at 25 inches

DESIGN FOR END SPAN - JOIST-BAND BEAM

$\omega_u = 212\,PSF$ FIND DISTRIBUTED WEIGHT FOR JOIST SYSTEM

(TABLE 8-1 OF CRSI)

JOIST-SLAB SYSTEM WEIGHT = 97 PSF ← FROM TABLE

$1.4(97) = 135.8\,PSF$

TOTAL WEIGHT = $212 + 135.8 = 347.8\,PSF$

$\omega = 347.8\,PSF\,(30') = 10.4\,KLF$

CHECK CRSI CH.12 FOR JOIST-BAND BEAM SIZE
FOR 30' SPAN WITH LOAD = 10.4 KLF

| STANDARD (1) ONE-WAY JOISTS MULTIPLE SPANS | | 30" Forms = 6" rib @ 36" c.-c. FACTORED USABLE SUPERIMPOSED LOAD (PSF)(2) | | | | | | | f'_c = 4,000 psi f_y = 60,000 psi | | | |

Depth: 16" Deep Rib + 4.5" Top Slab = 20.5" Total Depth

		END SPAN						INTERIOR SPAN				
TOP BARS Size @		#4 10	#4 8	#5 10.5	#5 9	#6 10.5	End Span Defl. Coeff. (3)	#4 9	#5 11.5	#5 9	#6 11	#6 9	Int. Span Defl. Coeff. (3)
BOTTOM BARS # #		#5 #5	#5 #6	#6 #6	#6 #7	#7 #7		#4 #4	#5 #5	#5 #6	#6 #6	#6 #7	
Steel (psf)		.85	1.04	1.24	1.44	1.71		.93	1.15	1.42	1.70	2.01	
CLEAR SPAN													
27'-0"		131 / 0	185 / 0	240 / 0	305* / 306	314* / 373	6.398	184 / 0	252 / 0	331 / 0	359* / 412	368* / 454*	3.938
28'-0"		112 / 0	163 / 0	214 / 0	275 / 0	293* / 338	7.400	161 / 0	225 / 0	299 / 0	338* / 373	346* / 423*	4.554
29'-0"		95 / 0	142 / 0	190 / 0	247 / 0	274* / 306	8.516	141 / 0	201 / 0	269 / 0	318* / 339	325* / 395*	5.240
30'-0"		80 / 0	124 / 0	169 / 0	222 / 0	256* / 277	9.752	123 / 0	178 / 0	243 / 0	300* / 307	306* / 370*	6.001
31'-0"		66 / 0	108 / 0	149 / 0	200 / 0	240* / 250	11.119	106 / 0	158 / 0	218 / 0	279 / 0	289* / 347*	6.842
32'-0"		54 / 0	93 / 0	132 / 0	179 / 0	226* / 227	12.625	92 / 0	140 / 0	197 / 0	254 / 0	273* / 322	7.769
33'-0"		43 / 0	79 / 0	116 / 0	160 / 0	205 / 0	14.278	78 / 0	124 / 0	177 / 0	230 / 0	258* / 295	8.787
34'-0"			66 / 0	101 / 0	143 / 0	185 / 0	16.089	66 / 0	109 / 0	159 / 0	209 / 0	244* / 270	9.901
35'-0"			55 / 0	88 / 0	127 / 0	167 / 0	18.067	54 / 0	95 / 0	142 / 0	190 / 0	231* / 247	11.118

D. Precast Hollowcore

HOLLOW CORE PLANK (SPAN SHORT DIMENSION)

LOADS: DEAD LOAD = 15 PSF   TOTAL LOAD = 115 PSF
       LIVE LOAD = 100 PSF

★ NITTERHOUSE PRESTRESSED HOLLOWCORE PLANK (CATALOG ONLINE)

- NEEDS TO SPAN 30' FROM GIRDER TO GIRDER

    12"×4' Spandeck - UL - J917 w/ 2" CIP TOPPING

      STRAND PATTERN - 6 - 1/2" φ, 270 K Lo-Relaxation

        ALLOWABLE SUPERIMPOSED LOAD = 187 PSF > 115 PSF (FLEXURE) OK

CHECK SUPPORTING GIRDERS FOR APPROPRIATE SIZE

LIVE LOAD = 100 PSF   TOTAL FACTORED LOAD
DEAD LOAD = 15 PSF   1.2D + 1.6L
SELF WEIGHT = 102.5 PSF FROM CATALOG   1.2(15 + 102.5) + 1.6(100)
                                = 301 PSF

$$M_U = \frac{\omega \ell^2}{8} = \frac{(.301\ KSF \times 30')(43.5)^2}{8} = 2135.91\ k$$

$$Z_{req} = \frac{M_U}{\phi F_y} = \frac{2135\ k (12)}{0.9\ (50\ ksi)} = 569.6\ in^3$$

CHECK LRFD STEEL MANUAL
- TABLE 5-3   A $\boxed{W\ 40 \times 149}$ w/ $Z_y = 598\ in^3 > 569.6\ in^3$ IS OK

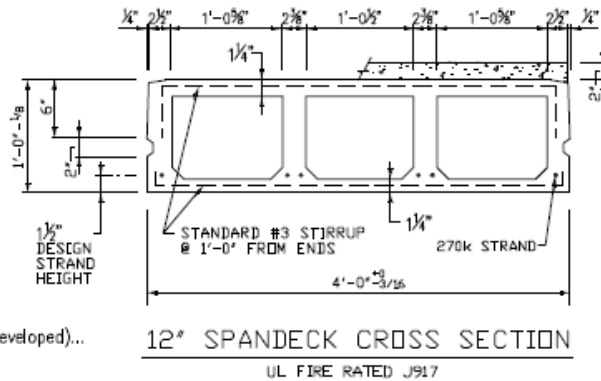   - SEE DETAILS FOR CONNECTION INFORMATION OF PLANKS TO STEEL

            - CHECK OUT PCI 2-27

30 by 38 blocks at .25 inches

Prestressed Concrete
12"x4' SpanDeck—U.L.—J917
(2" C.I.P. TOPPING)

PHYSICAL PROPERTIES
Composite

$A' = 293 \ in.^2$
$I' = 7164 \ in.^4$
$Y'_b = 7.65 \ in.$
$Y'_t = 4.35 \ in.$ (To Top of SpanDeck)
$Y'_{tt} = 6.35 \ in.$ (To Top of Topping)

$S'_b = 936 \ in.^3$
$S'_t = 1649 \ in.^3$ (At Top of SpanDeck)
$S'_{tt} = 1129 \ in.^3$ (At Top of Topping)
Wt. = 410 PLF
Wt. = 102.5 PSF

12" SPANDECK CROSS SECTION
UL FIRE RATED J917

STANDARD #3 STIRRUP @ 1'-0" FROM ENDS
270k STRAND
1½" DESIGN STRAND HEIGHT
4'-0"

DESIGN DATA
1. Precast Strength @ 28 days = 5000 PSI.
2. Precast Density = 150 PCF.
3. Strand = 1/2"∅, 270 K Lo-Relaxation.
4. Composite Strength = 3000 PSI.
5. Composite Density = 150 PCF.
6. Strand Height = 1.5 in.
7. Ultimate moment capacities (when fully developed)...
   4 – 1/2"∅, 270K = 146.2'K
   6 – 1/2"∅, 270K = 208.1'K
8. Maximum bottom tensile stress is $6\sqrt{f'c}$ =424 PSI.
9. All superimposed load is treated as live load in the strength analysis of flexure and shear.
10. Flexural strength capacity is based on stress/strain strand relationships.
11. Load values to the left of the solid line are controlled by ultimate strength. Load values to the right are controlled by service stress.
12. Shear values are the maximum allowable before shear reinforcement is required.
13. Deflection limits were not considered when determing allowable loads in this table.
14. All loads shown refer to allowable loads applied after the topping has hardened.

12" SPANDECK W/2" TOPPING			ALLOWABLE SUPERIMPOSED LOAD (PSF)																					
STRAND PATTERN			SPAN (FEET)																					
			18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
Flexure	4 – 1/2"∅		446	392	345	305	271	240	214	190	170	151	134	120	106	94	83	73	64						
Shear	4 – 1/2"∅		436	408	379	357	335	309	281	257	234	214	195	181	172	162	149	136	125						
Flexure	6 – 1/2"∅		671	593	527	470	421	378	340	307	277	251	227	206	187	170	154	140	127	115	104	94	85	76	66
Shear	6 – 1/2"∅		453	423	397	373	351	331	313	297	282	268	255	237	218	201	186	171	158	146	136	124	118	113	108

NITTERHOUSE
CONCRETE N 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.

REVISED 12/93

E. Non-Composite Slab and Steel

NON-COMPOSITE SLAB ON METAL DECK

- USE SAME FRAMING AND CONCRETE DEPTH AS EXISTS

LOADS: DEAD: $2\frac{1}{2}$ CONCRETE OVER 3" METAL DECK = 68.8 PSF

$$
\begin{aligned}
MEP &= 10 \\
DECK &= 2 \\
STRUCTURAL &= 15 \\
COLLATERAL &= 5 \\
&\approx 100 \text{ PSF}
\end{aligned}
$$

LIVE: 100 PSF FOR OFFICE LOADING

BEAMS: TRIB WIDTH = 10' < 120"

$$\omega_u = 1.2D + 1.6L = 1.2(100) + 1.6(100) = 280 \text{ PSF}$$
$$W_u = 280 \text{ PSF}(10') = 2.8 \text{ KLF}$$
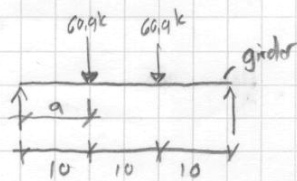$$M_u = \frac{2.8(43.5')^2}{8} = 662.3 \text{'k}$$

SIZE MEMBER: LRFD MANUAL (TABLE 5-3)

$$Z_x = \frac{M_u}{\phi_b F_y} = \frac{662.3(12)}{0.9(50)} = 176.6 \text{ in}^3$$

$\Rightarrow$ TRY A $\boxed{W24 \times 68}$ w/ $Z_y = 177 \text{in}^3 > 176.6 \text{in}^3$

$$\phi M_n = 664 > 662.3 \text{'k} \quad \therefore \text{ OK}$$

CHECK GIRDERS

$$P = \frac{2.8(43.5)}{2} \quad 60.9^k$$

$$M_{MAX} = P_a = 60.9(10') = 609 \text{'k}$$

$$Z_y = \frac{609(12)}{0.9(50)} = 162.4 \Rightarrow W24 \times 68 \text{ w/ } Z_y = 177 \quad \therefore \text{ OK}$$

$$\phi M_n = 664 > 609 \text{'k} \quad \therefore \text{ OK}$$

30 by 38"blocks at 25 inches

CHECK BEAMS FOR DEFLECTION   SERVICE LOADS
                               $W_{TL} = 200 PSF$
                               $W_{LL} = 100 PSF$

LIVE LOAD: $\Delta_{LL} = \dfrac{5\omega L^4}{384EI} \Rightarrow \dfrac{43.5(12)}{360} = \dfrac{5(100)(43.5)^4(1728)(10)}{384(29 \times 10^6) I_{req}}$

$$I_{req} = 1920 \, in^4$$

$\angle BFD \, MANUAL \Rightarrow \boxed{W24 \times 76} \quad w/ \quad I_x = 2100 \, in^4$

TOTAL LOAD: $\Delta_{TL} = \dfrac{5\omega L^4}{384EI} \Rightarrow \dfrac{43.5(12)}{240} = \dfrac{5(200)(43.5)^4(1728)(10)}{384(29 \times 10^6) I_{req}}$

$$I_{req} = 2560$$

$\angle BFD \, MANUAL \Rightarrow \boxed{W \, 27 \times 84} \quad w/ \quad I_x = 2850$

☆ $\underline{W27 \times 84}$ IS SIZE NEEDED DUE TO SERVICE LOADS

CHECK GIRDER FOR DEFLECTION: UNDER SERVICE LOADS

$P_{TOTAL} = \dfrac{200 PSF (10)(43.5)}{4} = 21.8^k \Rightarrow w_{TL} = \dfrac{21.8(2)}{30} = 1.45 KLF = 1450 PLF$

$P_{LIVE} = 10.9^k \qquad w_{LL} = \dfrac{10.9(2)}{30} = 0.726 KLF = 726 PLF$

$\Delta_{LL} \Rightarrow \dfrac{30(12)}{360} = \dfrac{5(726)(30)^4(1728)}{384(29 \times 10^6) I} \Rightarrow I_{req} = 456 \, in^4$

$\Delta_{TL} \Rightarrow \dfrac{30(12)}{240} = \dfrac{5(1450)(30)^4(1728)}{384(29 \times 10^6) I} \Rightarrow I_{req} = 607 \, in^4 \quad \times 2 = 1214 \, in^4 \; \text{for interior}$

A W18×40 FOR GIRDER ON EXTERIOR
A W24×55 FOR INTERIOR GIRDER

F. Composite Steel Joists

COMPOSITE DECK + OPEN WEB STEEL JOISTS

GIVEN: $f'c = 3000psi$, NORMAL WT. CONCRETE, ORIGINAL DECKING DESIGNED
$2\frac{1}{2}''$ CONCRETE OVER $3''$ RIB

Assume Joists will be placed every 6 feet spacing $43.5' \approx 44'$

Dead load = concrete = $5.5/12 (150) = 68.8$
joist = $41.5 PSF$ $D_{TOTAL} = 90.3 (6') = 542 PLF$
deck = $2 PSF$
MEP = $10 PSF$ *LIVE LOAD REDUCTIONS NOT TAKEN INTO
collateral = $5 PSF$ ACCOUNT

LIVE LOAD = $100 PSF (6') = 600 PLF$ TOTAL UNFACTORED = $\boxed{1142 PLF}$

CHECK JOIST CATALOGS - VULCRAFT - COMPOSITE JOISTS CATALOG

→ A 2.5" COVER OVE 2" DECK, JOISTS SPACED 8', CAN HOLD 1200 PLF
$W_{Lj} = 32 PLF = $ Weight of JOIST
$W_{360} = 850 PLF = W_L/360$
$N-ds = 32 - \frac{3}{4}'' \phi$ Shear Studs/span
Bridging required = 2 Rows

(COMPOSITE LIVE LOAD = $100 PSF(6) = 600 PLF < 850 PLF$ ∴ OK IN DEFLECTION

COMPOSITE DEAD LOAD = $68.8 + 2 + 41.5 = 75.3 PSF(6') = 452 \approx 450 PLF$

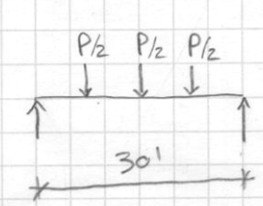USE: 18VC 1200/600/450   - JOIST DESIGNATION
$32 - \frac{3}{4}'' \phi$ Shear studs/span
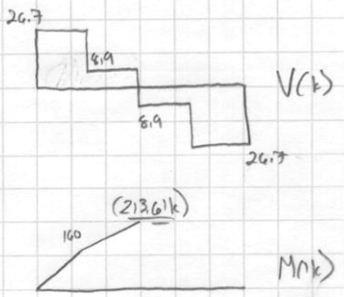
JOIST WEIGHT = 32 PLF
JOIST BEARING = 5"
2 ROWS OF BRIDGING

2" DECK w/ $2\frac{1}{2}''$ Concrete cover
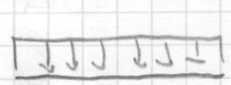
CHECK GIRDERS THAT SUPPORT JOISTS (SPAN=30')

DEAD LOAD

JOIST WT= 32 PLF (44)= 1408 lb

CONCRETE= $\frac{5.5}{12}$(150)= 68.8 PSF (6')(44)= 18163

MEP/COLLATERAL = 17 PSF (6')(44)= 4488

LIVE=100 PSF (44)(6)= 26400

$P = \frac{1.2(1408+18163+4488) + 1.6(26400)}{2}$

$P = 35.6^k/_2 = 17.8^k$

$V(k)$

LRFD (TABLE 5-3)

$M_{MAX}= 214^{'k}$

$Z_x = \frac{214(12)}{0.9(50)} = 57 \Rightarrow \boxed{W18\times35}$

w/ $Z_x=66.5 in^3$
$I=510 in^4$

$\phi M_n = 510^{'k} > 214^{'k}$ ∴ OK

CONVERT POINT LOADS TO UNIFORM + CHECK DEFLECTION

UNFACTORED DEAD LOAD= 24059 lb= $P/4 = 6.0^k$

LIVE LOAD= 26400 lb= $P/4 = 6.6^k$

$w_{TL}= \frac{6+6.6}{30}= 420 PLF$

$w_{LL}= \frac{6.6}{30}= 220 PLF$

$\Delta_{TL}= \frac{5wl^4}{384EI} \Rightarrow \frac{30(12)}{240} = \frac{5(420)(30)^4(1728)}{384(29\times10^6)I}$

$I_{req}= 176 in^4 \Rightarrow LRFD \Rightarrow W14\times22$ w/ $I= 199 in^4$
MANUAL

$\Delta_{LL}= \frac{30(12)}{360} = \frac{5(220)(30)^4(1728)}{384(29\times10^6)I}$

$I_{req}= 138 \Rightarrow W12\times22$

← STRENGTH GOVERNS, BUT INTERIOR GIRDER WOULD HOLD
TWICE THE LOAD ⇒ $Z_x= 133$, $\boxed{W24\times55}$ WOULD BE NEEDED

	Slab Design																		
WEIGHT TABLE AND DESIGN GUIDE
VULCRAFT COMPOSITE STEEL JOISTS, VC SERIES — Based on Allowable Tensile Stress of 30,000 psi

Joist Span (ft)	Joist Depth (in)		Normal Weight Concrete (145 pcf) f'c = 3.0 ksi																	
		tc (in)	2.00	2.00	2.00	2.00	2.50	2.50	2.50	2.50	2.50	2.50	2.50	2.50	2.50	2.50	3.00	3.00	3.00	3.00
		hr (in)	1.0	1.0	1.0	1.0	1.5	1.5	1.5	1.5	2.0	2.0	2.0	2.0	2.0	2.0	3.0	3.0	3.0	3.0
		Js (ft)	3.5	4.0	4.0	4.5	5.0	6.0	6.5	7.0	8.0	8.5	9.0	10.0	10.0	10.0	11.0	12.0	12.0	12.0
			Total Uniformly Distributed Joist Load in Pounds Per Linear Foot																	
		TL	400	500	600	700	800	900	1000	1100	1200	1300	1400	1500	1600	1800	2000	2200	2400	2700	3000
44	18	Wtj (plf)	12	15	17	19	22	24	27	30	32	36	37	40	41	47	55	55	66	74	86
		W360 (plf)	277	350	413	461	560	614	712	796	850	968	979	1103	1111	1176	1302	1554	1796	1873	1999
		N-ds	28-1/2	30-1/2	36-1/2	40-1/2	44-1/2	30-5/8	36-5/8	42-5/8	32-3/4	38-3/4	38-3/4	42-3/4	44-3/4	52-3/4	62-3/4	62-3/4	80-3/4	84-3/4	84-3/4