TECHNICAL REPORT III

Daniel Goff
Structural Option

Faculty Advisor
Linda M. Hanagan

Letter of Transmittal

Daniel Goff
Structural Option
October 17, 2014

Dr. Linda Hanagan
Advisor
The Pennsylvania State University

Dear Dr. Hanagan,

The following technical report was prepared to meet requirements from AE 481W. The report includes an analysis of one typical bay of existing framing including checks on the floor deck, floor joists, girders, and interior and exterior columns. Three alternative gravity systems were proposed as design solutions to the typical bay and subsequently explored. The alternative systems included non-composite steel, composite steel, and two-way flat plate slab framing. Alternative lateral force resisting systems were also discussed, but not explored in detail. The gravity systems were compared to determine the most viable alternative.

Thank you for taking the time to review this report, I look forward to reviewing your feedback.

Sincerely,

Daniel E. Goff

Executive Summary

The Primary Health Networks Medical Office Building is located in Sharon, Pa in between Pitt and E Silver streets next to the Shenango River. It will be a 5 story structure rising 85 feet, having four elevated floors and a roof. The building offers 78,000 square feet of occupiable space and will cost approximately $10 million.

The site soil was found to have a bearing capacity of 2500psi allowing for concrete spread and mat footings to serve as a foundation for the building. The building is primarily a steel framed structure with steel columns supporting wide flange steel girders and steel bar joists. Typical sizes for floor joists and girders range from 10 inch to a maximum depth of 24 inches. The floor structure is concrete on metal deck for all four elevated floors, whereas the first floor is concrete slab on grade. Typical bay sizes range from 30'x26' to 33'-10"x30'.

The building's lateral force resisting system is comprised of three Ivany block shear walls. Ivany block is a concrete masonry unit with pre-determined locations for the rebar and having an f'm of 3000psi. The shearwalls are located around stairwells throughout the building.

Typical shear and moment connections are to be designed by the steel fabricator. Other connections typical to this building discussed in detail include joist to ivany block wall connections and concrete slab on metal deck to ivany block to wall connections.

The building was designed using the International Building code (IBC) edition 2009 which references the American Society of Civil Engineers (ASCE) document 7-05. The exception to this is the lateral loads on the building, which were determined with and designed to the IBC 2012 -edition which adopts ASCE 7-10.

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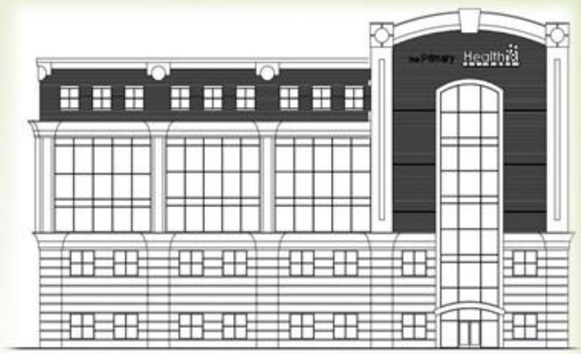
Building Abstract

The Primary Health Network's Medical Office Building
Sharon, PA



General Information

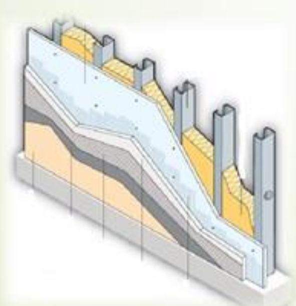
Height: 82ft
Size: 78,000 sq. ft.
Cost: $10 million
Construction: November 2014-January 2016
Project Delivery Method: Design-Build

Project Team

Owner: The Primary Health Network
Architect: John N Guitza Associates, Inc.
Structural Engineer: Taylor Structural Engineers
MEP Engineer: BDA Engineering
Construction Manager: Hudson Construction
Civil Engineer: Professional Service Industries, Inc.

Architecture

The primary architectural goal was to create a modern look with a strong focus on economy. This was accomplished by methods such as incorporating an exterior finish/insulation system (E.I.F.S. shown below).



Mechanical System

Variable Air Volume system comprised of (2) 65 ton units and (1) 30 ton unit

Lighting and Electrical Systems

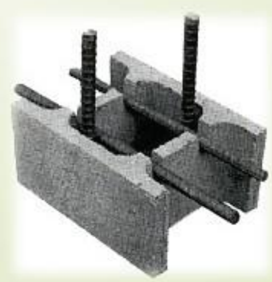
(5) 120/208V 3 Phase panel boards
(6) 480/277V 3 Phase panel boards

Low voltage dual technology occupancy sensors are used to increase efficiency

Structural System

Foundation: Concrete spread and Mat footings
Gravity: Steel columns and wide flange girders, steel bar joists, and normal weight concrete on metal deck floors

Lateral: 3 Ivany block shear walls
(Ivany Block Pictured below)



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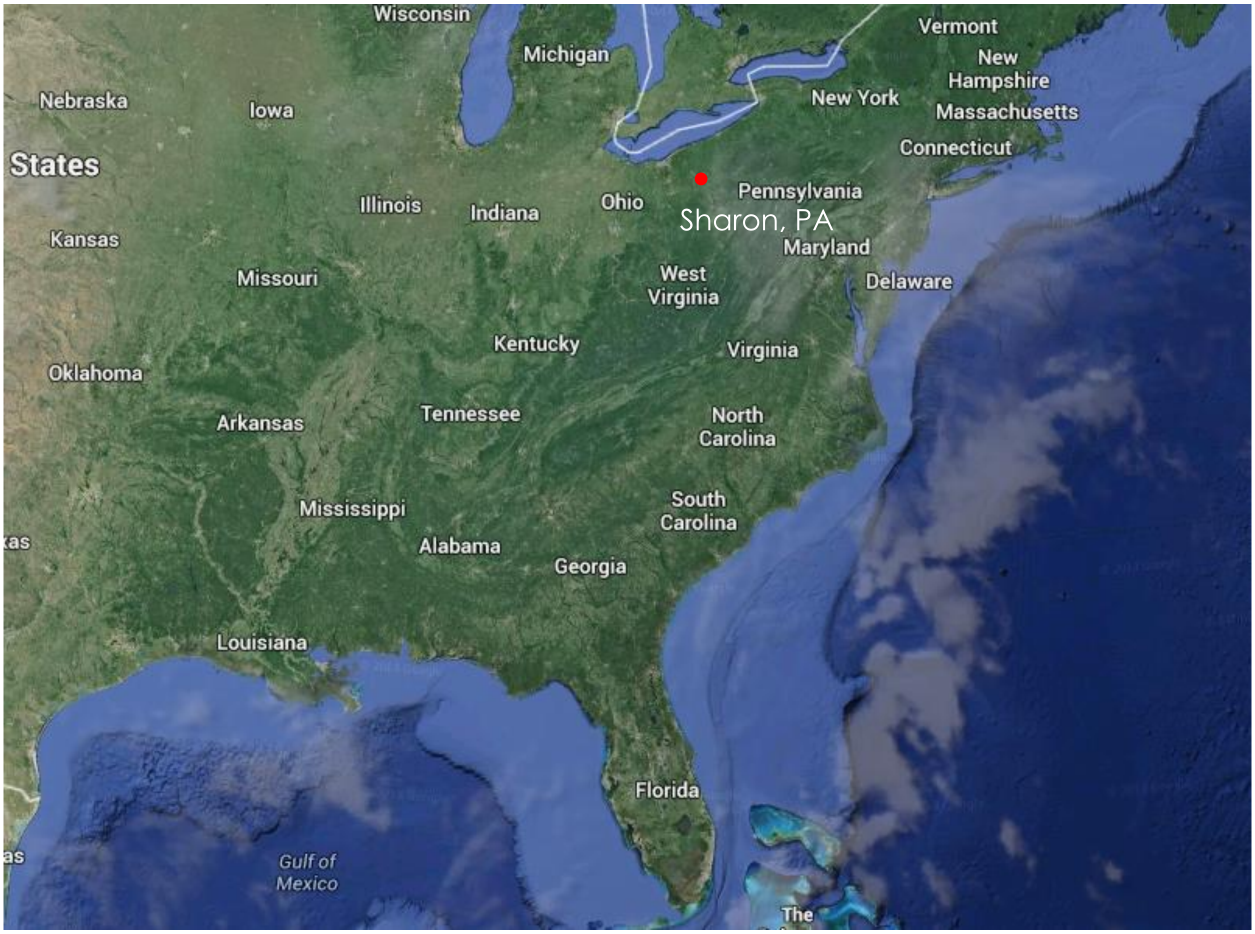
Site Plan

Location Plan

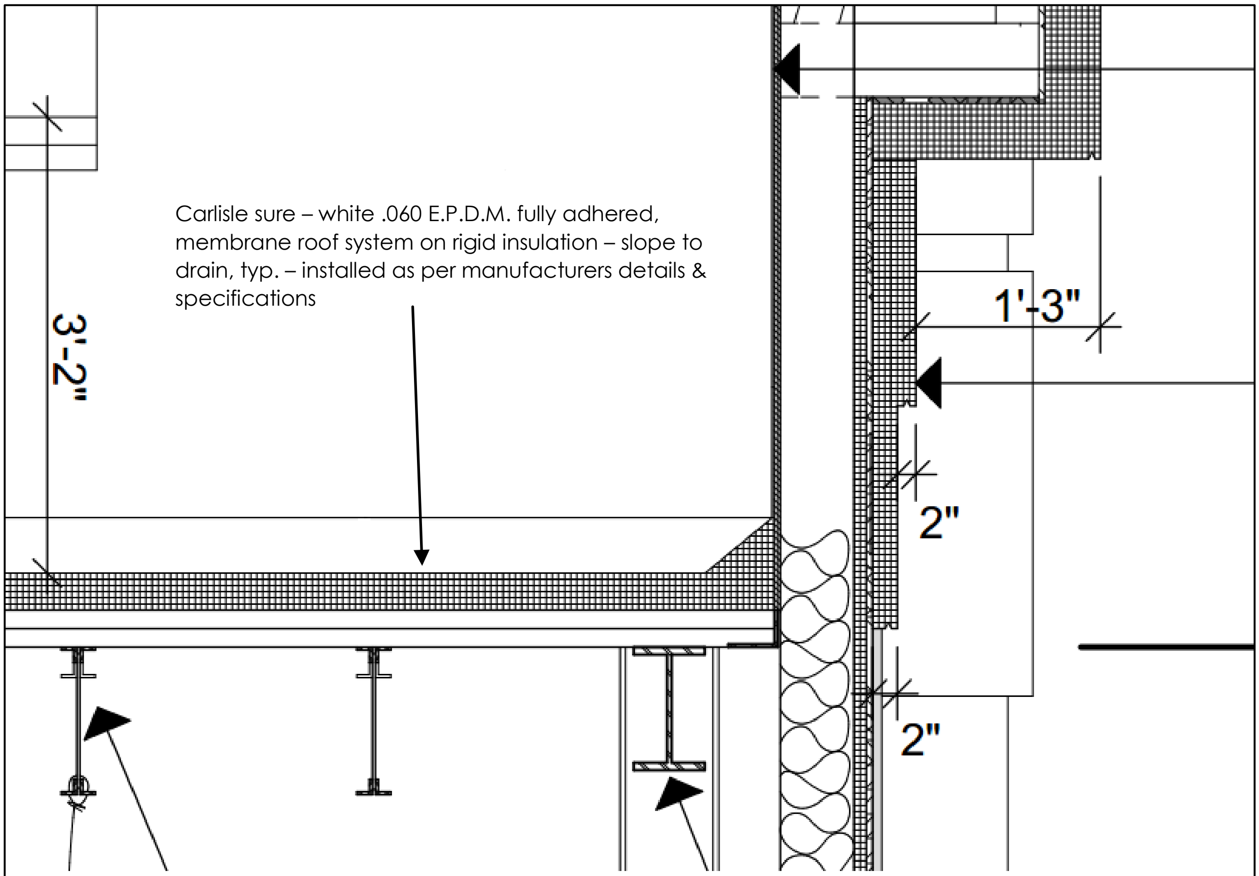


Preparatory Documents

Building Code:	2012 International Building Code (IBC)
Steel:	American Institute of Steel Construction (AISC)
Welding:	American Welding Society
Concrete:	American Concrete Institute (ACI)
Concrete Masonry:	American Concrete Institute (ACI)
	American Society of Civil Engineers (ASCE)
	ASCE 7-05
	ASCE 7-10 (for lateral loads only)

Gravity Loads

Typical Roof Loading



Typical roof section from section 1 drawing A-31

<u>Roof Dead Loads:</u>

Roofing/Membrane	1 psf	
Insulation:	6 psf	
Deck:	2 psf	from vulcraft
Steel:	5 psf	
Miscellaneous/MEP:	10 psf	
Total roof dead load:	24 psf (20psf was used in design)	

Roof Live Loads:

Basic roof live load: 20 psf per table 4-1 in ASCE 7-05
(30 psf was used in design)

Roof snow load: 21 psf
(21 psf was used in design)

Design snow load = $0.7*C_c*C_t*I*P_g$

$C_c=1.0$
$C_t=1.0$
$I=1.0$
$P_g=30psf$

Snow drift load:

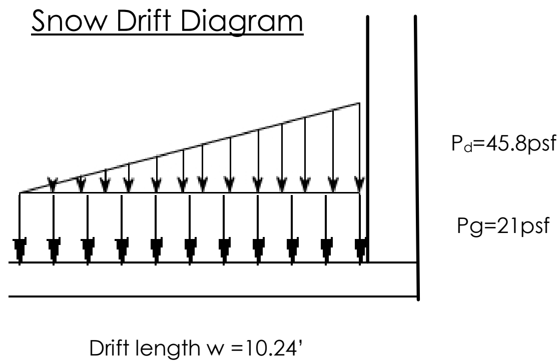
$\gamma=0.13*30+14=17.9$

hd=2.56' from eq. in figure 7-9

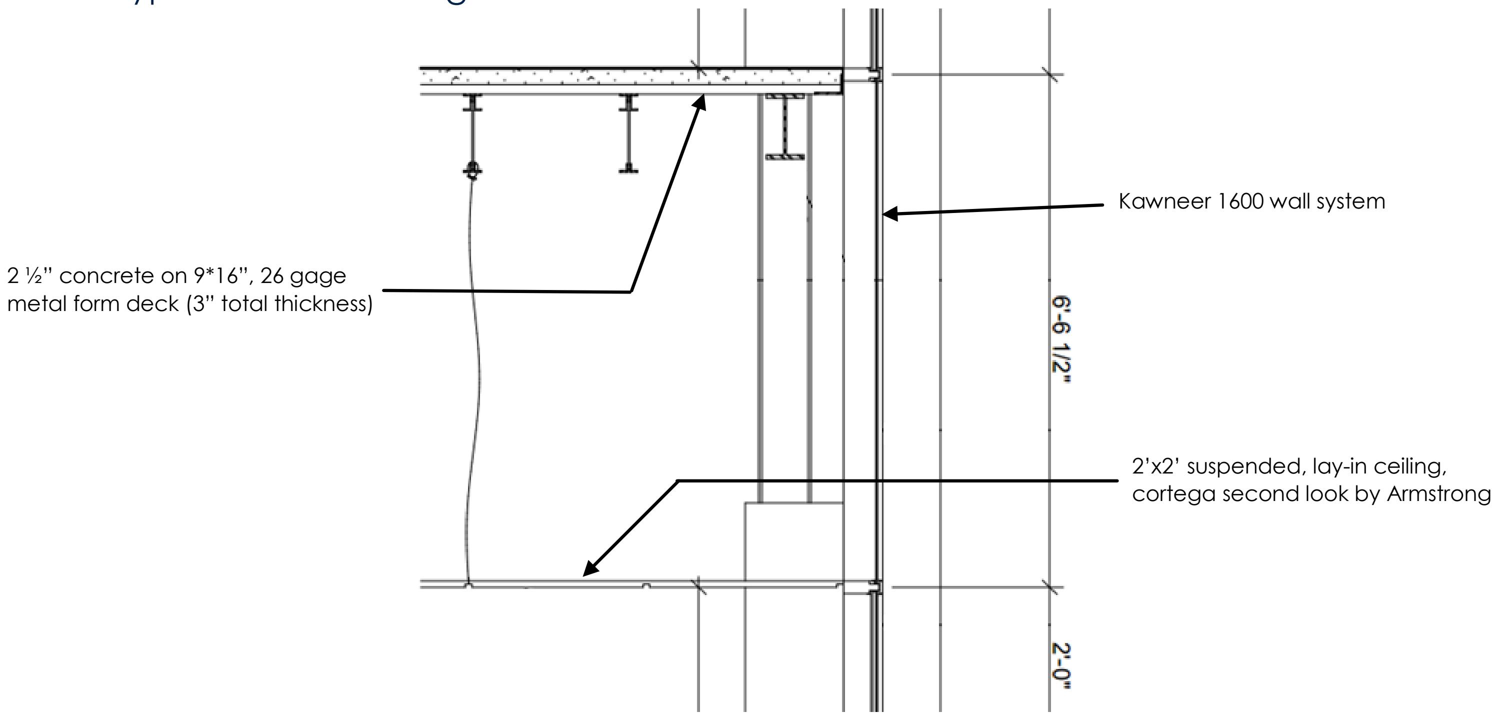
w=4*2.56=10.24'

$P_d=2.56*17.9=45.8psf$

Snow Drift Diagram



Typical Floor Loadings



Floor Dead Load

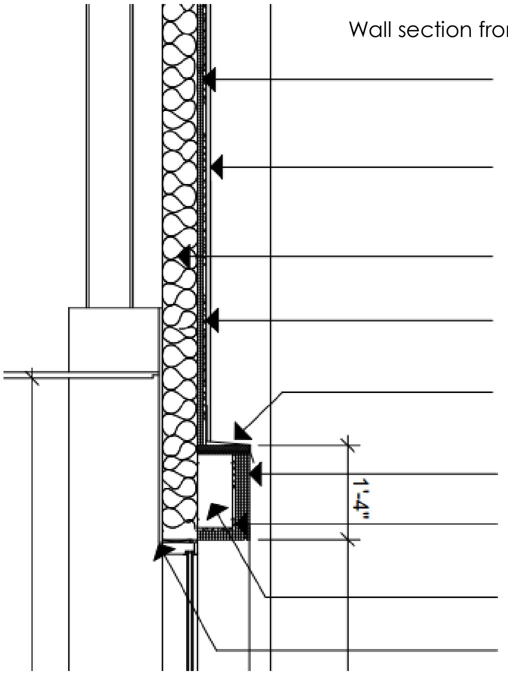
Flooring:	1 psf	
Slab-on-deck	35 psf	- from vulcraft
Steel:	8 psf	(includes joists & girders)
Miscellaneous/MEP:	10 psf	
Total floor dead load:	53 psf	

Floor Live Load (Table 4-1 ASCE 7-05)

Area	As Designed (psf)	ASCE 7-05 (psf)
Office	80	50
First Floor Corridors	100	100
Corridors above first floor	80	80
Stairs	100	100
Partitions	15	15

Non-typical Loadings

Wall section from section 1 drawing A-31



Insulation:	1 psf
Cement Finish:	6 psf
Densglass Sheathing:	2 psf
Studs:	2 psf
Total	11 psf

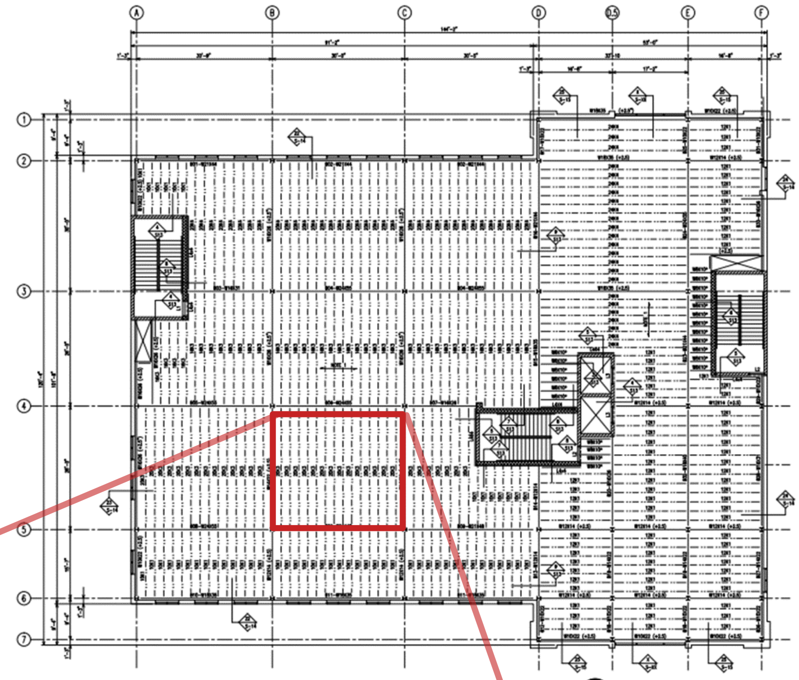
The load of the cement finish, sheathing and insulation is transferred into the light gage steel studs. These in turn send the load into steel angles which transfer it into the columns and finally to the foundations.

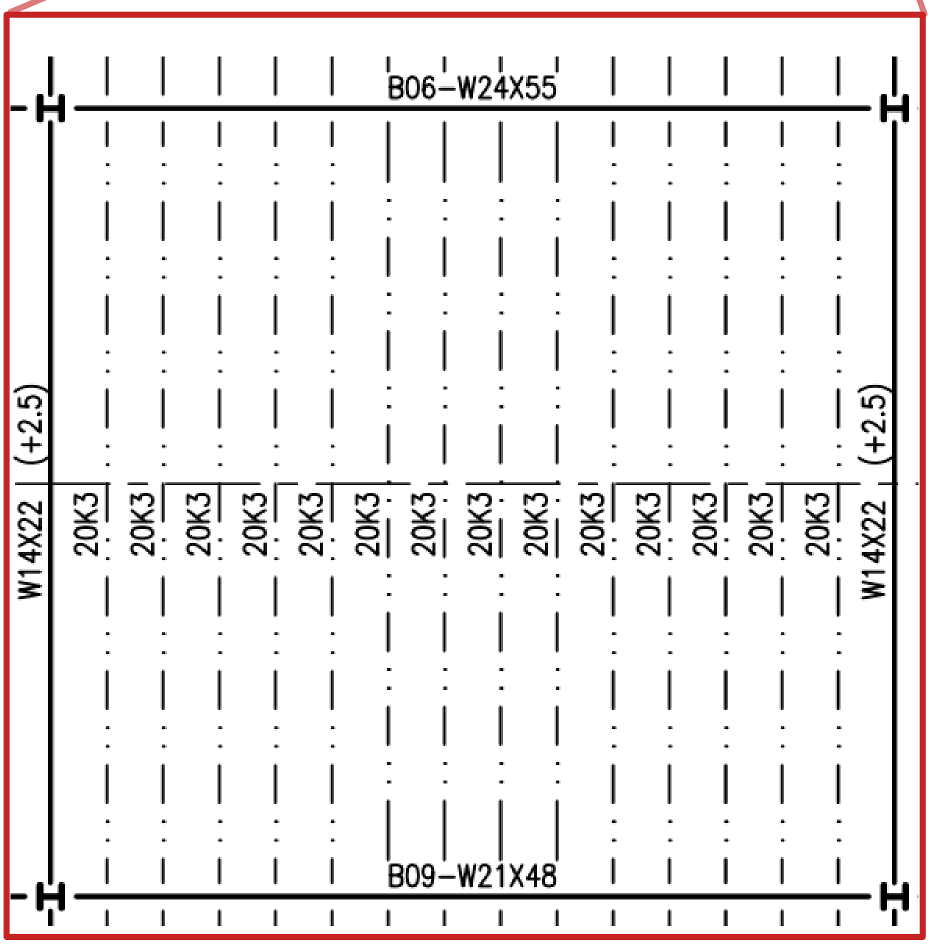
Other Non-typical loadings

There are there roof top units on The Primary Health Networks Medical Office building. The worst case of these being a 11,000lb unit occupying a 33ft. by 9ft. space. This essentially superimposes a 37psf dead load on all other loads already being applied to this space.

Gravity Spot Check

A typical bay, outlined in red on the second floor plan shown to the right, was analyzed for gravity loadings. This bay consists of 2 ½" normal weight concrete on 9/16", 26 gage metal form deck reinforced with 6x6 W1.4xW1.4 W.W.R. per note 1. The deck is supported by 20k3 joists spaced at 24 inches O.C. The joists are in turn supported by wide flange steel sections as shown in the enlarged view below.





Analysis of concrete on metal deck

0.6C26 - Per Vulcraft catalog

Check if shoring is required

3 span condition - 3'-2" > 2'-0"

2 span condition - 3'-2" > 2'-0"

1 span condition - 2'-5" > 2'-0"

No shoring is necessary

Check for strength

Live load = 80psf

Superimposed dead load = 11psf

Flooring = 1psf

Misc./MEP = 10psf

Total weight = 91psf

Clear span = 2'-0"

Allowable load = 342psf per Vulcraft catalog

Deck has sufficient strength

Analysis of 16K3 steel bar joists (ASD)

Dead load = 45psf

Live load = 80psf

Tributary width = 2'-0"

Span = 28ft

Total load (W) = (45psf+80psf)2' = 250plf

Live load (W_L) = 80psf(2') = 160plf

Check 20K3 joist capacity

Total allowable load from Vulcraft

261plf > 250plf

Allowable load causing deflections of 1/360 from Vulcraft

189plf > 160plf

Joists have sufficient strength to carry load

Analysis of W24x55 wide flange Girder (LRFD)

Span = 30 ft

Tributary width = $\frac{26\text{ft}}{2} + \frac{28\text{ft}}{2} = 27\text{ft}$

Influence Area = $(26\text{ft} + 28\text{ft})(30\text{ft}) = 1620\ \text{ft}^2$

Live load Reduction = $\left[0.25 + \left(\frac{15}{\sqrt{1620}}\right)\right] = 0.62$

Live load = $80\text{psf}(0.62) = 49.6\ \text{psf}$

Dead Load = 53 psf

$W = (53\text{psf} + 49.6\text{psf})(27\text{ft}) = 2770\ \text{plf}$

$W_{Live} = (49.6\text{psf})(27\text{ft}) = 1339\ \text{plf}$

$W_u = [1.2(53\text{psf}) + 1.6(49.6\text{psf})](27\text{ft}) = 3860\ \text{plf}$

Strength

$M_u = \frac{3.86\ \text{klf}\ (30\text{ft})^2}{8} = 434.25^{\text{k-ft}}$

Deflections

L/240 $I_{req'd} = \frac{5(240)(2.77)(30\text{ft} \times \frac{12\text{in}}{\text{ft}})^3}{(384)(29{,}000)(12\text{in/ft})} = 1160.5\ \text{in}^4$

L/360 $I_{req'd} = \frac{5(360)(1.34)(30 \times \frac{12\text{in}}{\text{ft}})^3}{(384)(29{,}000)(12\text{in/ft})} = 842\ \text{in}^4$

From Steel Construction Manual.

W24x55 $\phi M_p = 503^{\text{k-ft}} > 434^{\text{k-ft}}$ ✓

$I_x = 1350\ \text{in}^4 > 1160.5\ \text{in}^4$ ✓

W24x55 sufficient for Strength + Deflections

Analysis of Column B-4 (W10x49) (Interior)

Influence Area = 2 x (26' x 30') + 2 x (28' x 30')
= 3240 ft² x 4 floors = 12,960 ft²

Live load Reduction = $0.25 + \frac{15}{\sqrt{12{,}960\ ft^2}}$ = 0.38 → use 0.40

Floor Live load = 80 psf (0.40) = 32 psf

Floor Dead Load = 53 psf

Floor Tributary area = $\frac{12{,}960\ ft^2}{4}$ = 3240 ft²

Roof live load = 21 psf

Roof Dead load = 24 psf

Roof tributary area = 3240 ft²/4 = 810 ft²

Total Dead load = 53 psf (3240 ft²) + 24 psf (810 ft²)
= 191.2 psf

Total Live load = 32 psf (3240 ft²) + 21 psf (810 ft²)
= 104ᵏ + 17 kips

P_u = 1.2 (191.2ᵏ) + 1.6 (104 kip) + 0.5 (17 kip) = 404 kip

W10x49 from Steel Construction Manual

ϕP_n = 427ᵏ @ KL = 16'   427ᵏ > 404ᵏ ✓

Column has sufficient strength

Analysis of Column A-4 (W10x39) (Exterior)

Influence Area = (26ft + 28ft)(30.75ft) = 1660.5 ft²

1660.5ft x 4 floors = 6642 ft²

Live Load Reduction = $0.25 + \frac{15}{\sqrt{6642}} = 0.43$

Floor Live load = 80psf(0.43) = 35psf

Floor Dead Load = 53psf

Floor tributary area = $\frac{6642 ft^2}{4} = 1660.5 ft^2$

Roof Live Load = 21psf

Roof Dead Load = 24psf

Roof tributary area = $\frac{1660.5 ft^2}{4} = 415 ft^2$

Total Floor Dead Load = 53psf(1660.5) = 88k

Total Roof Dead Load = 24psf(415ft²) = 10k

Total Floor Live load = 35psf(1660.5) = 58k

Total Roof Live load = 21psf(415ft²) = 9k

Exterior wall load

Wall weight = 11 psf (Avg)

Total wall load = $11psf \left(\frac{26+28}{2}\right)(70ft)^{*} = 21^{kip}$

* 70ft height includes all non-self supporting wall to max total building height

$P_u = 1.2(88+10+21) + 1.6(58) + 0.5(9^k) = 240^k$

$\phi P_u = 282^k$ ✓

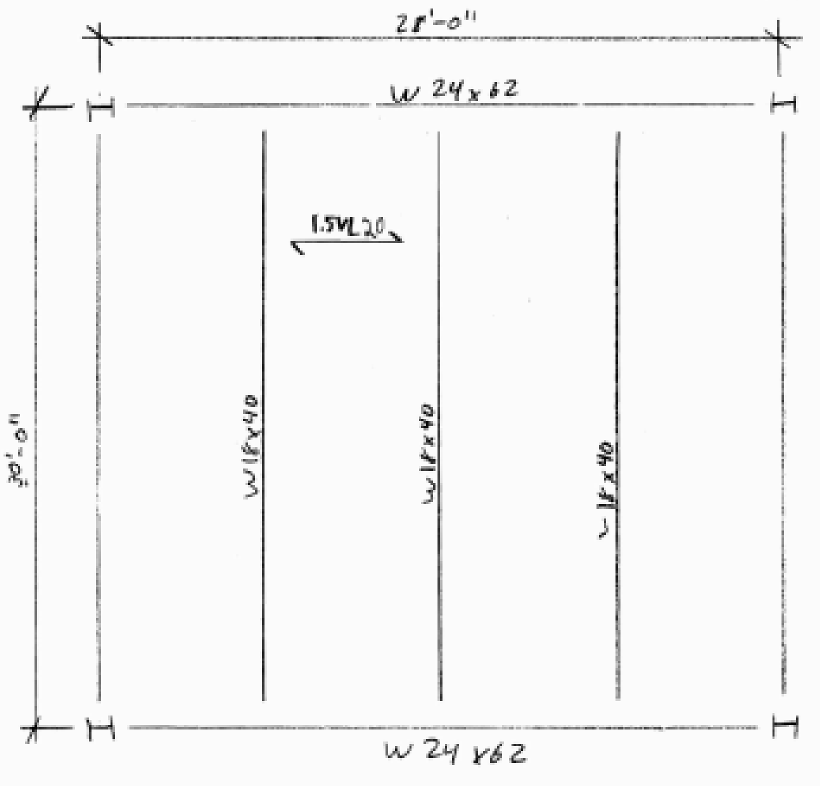
Column has sufficient strength

Alternate Framing System 1

Non-Composite Steel

Non-composite Steel

A typical 30'x28' bay was redesigned with beams spaced at 7-0" O.C. spanning in the long direction as shown below.



Select Deck

Per previous calculations req'd capacity = 91 psf

Span = 7'-0"

2 hour fire rating

Select 1.5VL20 → Load capacity = 187 psf

normal weight 3" total

Design Steel Beam

Dead Load = 51 psf (Includes 10psf framing allowance)

Live load = 80 psf

$W = (51+80)(7') = 0.92$ klf

$W_{Live} = 80\,(7') = 0.6$ klf

$$W_u = [1.2(51) + 1.6(80)](7') = 1.32 \text{ klf}$$

$$M_u = \frac{1.32(30^2)}{8} = 149 \text{ k-ft}$$

required I_x to limit deflections to L/240

$$I_{req} = \frac{5(240)(0.92)(30 \times 12)^3}{384(29{,}000)(12 \text{ in/ft})} = 578 \text{ in}^4$$

required I_x to limit deflections due to live loads to L/360

$$I_{req} = \frac{5(360)(0.6)(30' \times 12)^3}{384(29{,}000)(12'\text{in/ft})} = 377 \text{ in}^4$$

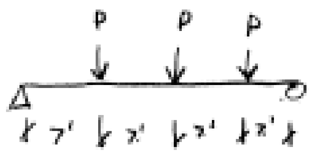
Select W18x40 $\quad \phi M_p = 294 > 149$ k-ft ✓

$I_x = 612 \text{ in}^4 > 578 \text{ in}^4$ ✓

Design Steel Girder

Dead Load = 51psf (Includes a 6psf framing allowance for girders)

Live load = 80psf

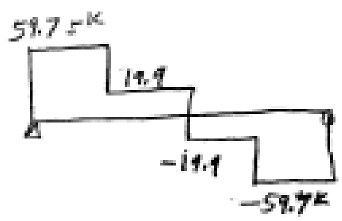


$P = [(51+80)(7' \times 30')] = 27.6$ kips

$P_{Live} = (80)(7' \times 30') = 1.68$ kips

$P_u = [(51 \times 1.2 + 80 \times 1.6)(7' \times 30')] = 39.8^{kips}$

V_x



$M_u = 557.2^k$

required I_x to limit deflections to L/240

$$I_{req} = \frac{27.6^k(7 \times 12)}{24(29000)(1.4)}\left(3(28 \times 12)^2 - 4(7 \times 12)^2\right) + \frac{27.6^k(28 \times 12)^3}{48(29,000)(1.4)} = 1276 in^4$$

equation from Steel manual Table 3-23 using super position

Select W 24x62

$\phi M_p = 574^{k\text{-}ft} > 557.2^{k\text{-}ft}$ ✓

$I_x = 1550\ in^4 > 1276 in^4$ ✓

Design Steel Column

Dead Load = 48psf
Live load = 80psf

Influence area = 2 x (26'x30') + 2x(28'x30')
= 3240 ft² x 4 floors = 12,960 ft²

Live load Reduction = $0.25 + \frac{15}{\sqrt{12,960}}$ = 0.38 → use .40

floor live load = 80(0.40) = 32psf

floor tributary area = $\frac{12,960 \text{ ft}^2}{4 \text{ floors}}$ = 3240 ft²

Roof live load = 21psf
Roof dead load = 24psf
Roof tributary area = $\frac{3240 \text{ ft}^2}{4}$ = 810 ft²

total Dead load = 48psf (3240) + 24psf (810)
= 175 kips

total live load = 32psf (3240) + 21 (810)
= 104 k + 17 k

$P_u = 1.2(175) + 1.6(104) + 0.5(17) = 385$ kips

Select W10x49 from Steel Construction Manual

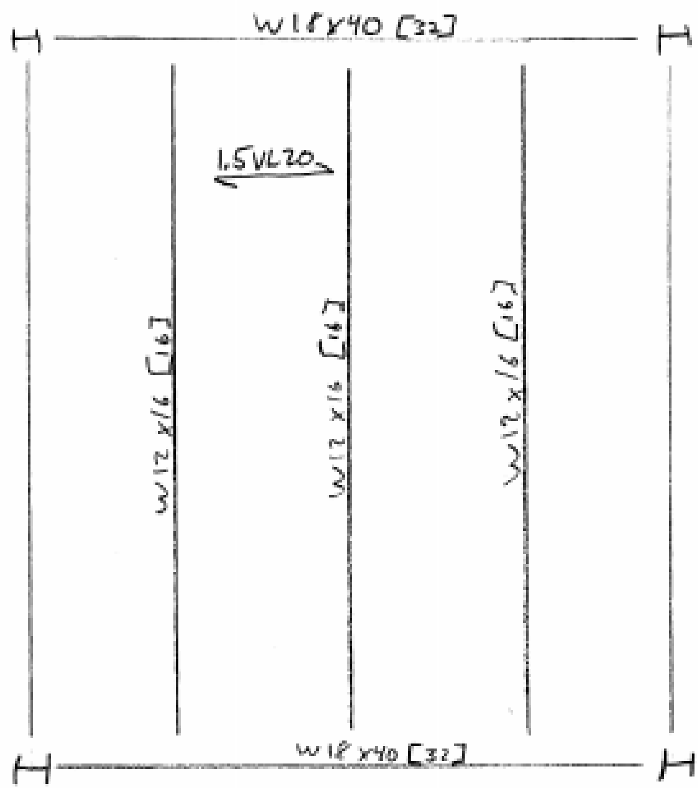
$\phi P_n = 427^k$ @ KL = 16' $427^k > 385^k$ ✓

Alternate Framing System 2

Composite-Steel Beams

Composite Steel

A typical 30'x28' bay was redesigned using composite steel beams spaced at 7'-0" O.C. spanning in the long direction as shown below



Select Deck

Per previous calculations req'd Capacity = 91psf
Span = 7'-8"
2hr fire rating

Select 1.5VL 20 L.W.
Capacity = 233psf >> 91 psf per Vulcraft
4" total

Design Composite Beam

Dead load = 48 psf (Includes 10 psf framing allowance)

Live load = 80 psf

Live load reduction

$$LL = 0.25 + \frac{15}{\sqrt{2(7)(30)}} = 0.77$$

$$LL = 80(0.77) = 61.4\,psf$$

$$w_u = 1.2(48) + 1.6(61.4) = 156\,psf$$

$$w_u = 156(7') = 1.09\,klf$$

$$M_u = \frac{1.09(30)^2}{8} = 123^{k\text{-}ft}$$

assume $a = 1''$ → $Y_2 = 4'' - \frac{1}{2}'' = 3.5''$

From steel construction manual

W12x16 → $\Sigma Q_n = 130$ $\phi M_n = 139^{k\text{-}ft} > 123^{k\text{-}ft}$ ✓

$\#studs = \frac{130}{17.2} = 7.5 \rightarrow 8 \times 2 = 16$ studs/Beam

W10x17 → $\Sigma Q_n = 150$ $\phi M_n = 133^{k\text{-}ft} > 123^{k\text{-}ft}$ ✓

$\#studs = \frac{150}{17.2} = 8.7 \rightarrow 9 \times 2 = 18$ studs/Beam

W10x15 → $\Sigma Q_n = 167$ $\phi M_n = 126^{k\text{-}ft} > 123^{k\text{-}ft}$ ✓

$\#studs = \frac{167}{17.2} = 9.7 \rightarrow 10 \times 2 = 20$ studs/Beam

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

$B_{eff} = 2x \; \min \begin{cases} 30/8 = 3.75' \\ 7/2 = 3.5' \leftarrow \text{controls} \end{cases}$

$b_{eff} = 84"$

Check economy

1 stud = 10 lb steel

W12x16 [16]

$16 \text{ lb/ft} \times 30 \text{ ft} + 10(16) = 640 \text{ lbs}$

W10x17 [18]

$17 \text{ lb/ft} \times 30 \text{ ft} + 10(18) = 690 \text{ lbs}$

W10x15 [20]

$15 \text{ lb/ft} \times 30 \text{ ft} + 10(20) = 650 \text{ lbs}$

W12x16 [16] is most economical

try W12x16

Check a assumption

$$a = \frac{.9(17.2)}{0.85(4)(84)} = 0.48'' < 1'' \checkmark$$

Check unshored strength

W12x16 $\phi M_p = 75.4^{ft\text{-}k}$ $I_x = 103\ in^4$

$w_u = 1.2(48 \times 7') + 1.6(20 \times 7') = 0.627\ klf$ (Includes framing allowance)

$$M_u = \frac{(0.627)(30^2)}{8} = 70.6^{k\text{-}ft} < 75.4^{k\text{-}ft} \checkmark$$

Check wet concrete deflection

$$W_{wet} = (30psf)(7') + 16 = 226\ plf$$

$$\Delta_{wc} = \frac{5(.226)(30^4)}{384(29{,}000)(103)}(1728) = 1.38''$$

$$\Delta_{max} = \frac{30 \times 12}{240} = 1.5'' > 1.38'' \checkmark$$

Check LL deflection

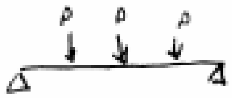
$$W_{LL} = 61.4psf(7') = 0.430\ klf$$

$I_{LB} = 254\ in^4$ from table 3-20 steel construction manual

$$\Delta_{LL} = \frac{5(0.43)(30^4)1728}{384(29{,}000)(254)} = 1.06''$$

$L/360 = 1''$ 1.06'' > 1.00'' → Acceptable by Engineering Judgement

Composite Girder design



Determine P_u

$P_u = 1.2(48)(7') + 1.6(66.4)(7') = 1091$ plf

$(1091)(30') = 32.7^k$ (Includes 8psf girder allowance)

$M_u = \frac{PL}{2} = \frac{32.7^k(28)}{2} = 458$ k-ft

assume $a = 1''$ $y_2 = 4 - \frac{1}{2} = 3.5''$

Determine b_{eff}

$b_{eff} = 2 \times \min \begin{cases} \frac{28 \times 12}{8} = 42'' \leftarrow \text{controls} \\ \frac{30 \times 12}{2} = 180'' \end{cases}$ $b_{eff} = 84''$

try W18x40

$\Sigma Q_n = 274$ $\phi M_n = 462$ k-ft

#studs: $\frac{274}{17.2} = 15.9 \rightarrow 16 \times 2 = 32$ studs

Check assumption

$a = \frac{16(17.2)}{0.85(4)(84)} = 0.96'' < 1''$ ✓

Check unshored Strength

W18x40 $\quad \phi M_n = 294^{k\text{-}ft}$

$w_u = 1.2(48 \times 7 + 16) + 1.6(20 \times 7) = 646.4\ plf$

$P_u = (646.4)(30) = 19.4^k$

$M_{max} = \frac{19.4(28)}{2} + \frac{1.2\left(\frac{40}{1000}\right)(28)^2}{8} = 276.3^{k\text{-}ft}$

$294^{k\text{-}ft} > 276.3^{k\text{-}ft}$ ✓

Check wet concrete deflection

$w = 48(7) + 16 = 352\ plf$

$P = (.352)(28') = 9.86^k$

$\Delta_{wc} = \frac{Pa}{24EI}(3l^2 - 4a^2) + \frac{Pl^3}{48EI}$

$= \frac{(9.86)(7' \times 12)}{24(29{,}000)(612)}\left(3(28 \times 12)^2 - 4(7 \times 12)^2\right) + \frac{9.86\ (28 \times 12)^3}{48\ (29{,}000)(612)}$

$= 1.04''$

$L/240 = \frac{28 \times 12}{240} = 1.4''$ $\quad 1.4'' > 1.04''$ ✓

Check Live load deflection

$I_{LB} = 1240$

$w_L = (61.4)(7') = 430\,plf$

$P_L = (0.43)(30') = 12.89^k$

$$\Delta_{LL} = \frac{12.89^k (7\times12)}{24(29{,}000)(1240)}\left(3(28\times12)^2 - 4(7\times12)^2\right) + \frac{12.89(28\times12)^3}{48(29{,}000)(1240)}$$

$$= 0.67''$$

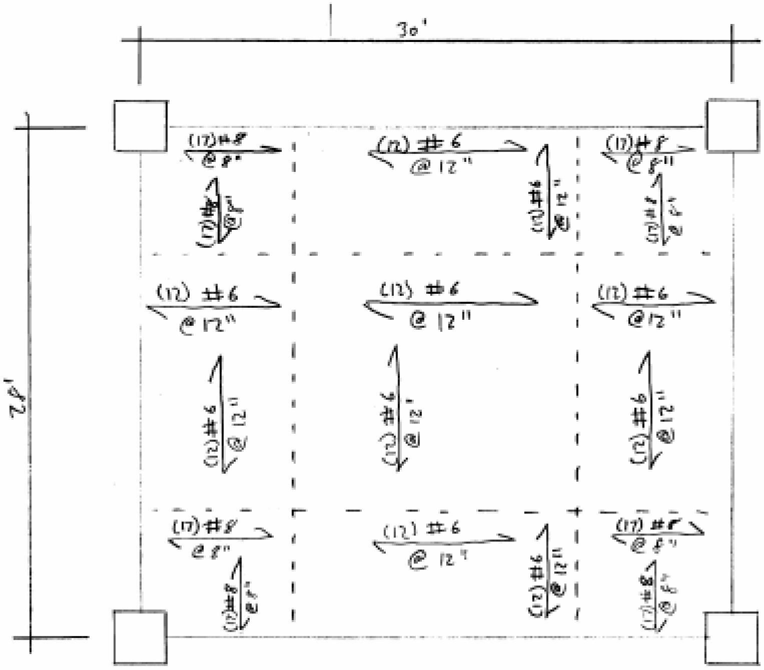
$$L/360 = \frac{28\times12}{360} = 0.93'' > 0.67'' \checkmark$$

Alternate Framing System 3

Two-Way Flat Plate Slab

Two way slab

A flat plate system was selected using the CRSI Design Handbook. A minimum depth of 10" is required for the 30'-0" span whether or not drop panels are utilized. Minimum square column size for an Interior panel in this design is 36."



Loading on slab

Live load = 80 psf

Miscellaneous Deadload = 20 psf

Slab self weight = 150 pcf ($\frac{10 in}{12 in}$ · 1 ft) = 120 psf

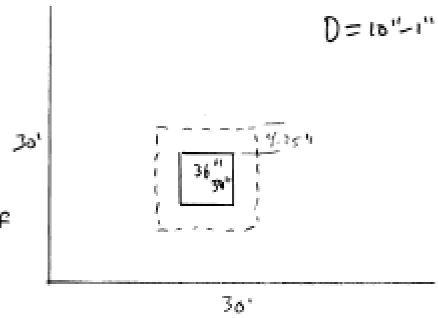
Check if Shear reinforcing is necessary

For this shear analysis the bay will conservatively be considered as 30'x30'

$D = 10'' - 1'' = 9''$

$f'_c = 4\,ksi$

$w = 1.2(120) + 1.6(100) = 304\,psf$



One way Shear

$V_n = 2\sqrt{4000}\,(30 \times 12)(9) = 409^k$

$\phi V_n = 307^k$

$V_u = 0.304\,(12.75' \times 30') = 116^k$

$307^k > 116^k$ ✓

Check two way shear

$V_u = (30^2 - 3.75^2)(0.304) = 269^k$

$V_c = \min \begin{cases} 2 + \frac{4}{1} = 6 \\ \frac{40(9)}{180} + 2 = 4.25 \\ 4 \leftarrow \text{controls} \end{cases}$

$\phi V_c = 0.75(4)\sqrt{4000}\,(180)(9) = 307^k$

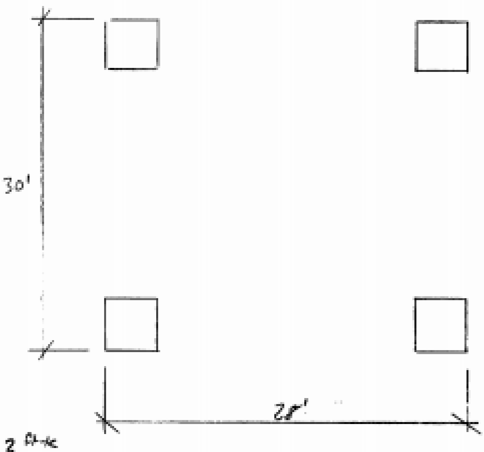
$307^k > 269^k$

No Shear reinforcing required

Flexural Strength

Interior span using
ACI Direct Design Method

Evaluate long direction first.



$q_u = 0.288$ klf

$$M_o = \frac{0.288(28')(30')^2}{8} = 907.2 \text{ ft-k}$$

Distribute moment

$M_u^- = 0.65(907.2) = 589.68$ ft-k

$M_u^+ = 0.35(907.2) = 317.52$ ft-k

Transverse Distribution $\alpha = 0$

Negative moment at Interior column strip = (0.75)(589.7) = 442.3 k-ft
Negative moment at Interior middle strip = (0.25)(589.7) = 147.4 k-ft

Positive moment at Interior column strip = (0.60)(312.5) = 187.5 k-ft
Positive Moment at Interior middle strip = (0.40)(312.5) = 125 k-ft

COMET

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER

Determine Reinforcing

Column Strip – negative moments

$$A_{s\,min}^{-} = \frac{M_u^{-}}{4d} = \frac{442.3}{(4)(9'')} = 12.29\,in^2 \rightarrow (17)\#8 \rightarrow 13.43\,in^2$$

Flexural strength

$$a = \frac{(13.43)(60)}{0.85(4)(14\times12)} = 1.41 \qquad c = \frac{1.41}{0.85} = 1.66$$

$$M_n = \frac{13.43(60)(9 - \frac{1.41}{2})}{12} = 557\ ft\text{-}k$$

$$\varepsilon_s = \frac{0.003}{1.66}(9 - 1.66) = .013 > .00207$$

$$\phi M_n = 0.9(557) = 501\ ft\text{-}k > 442.3\ ft\text{-}k \checkmark$$

min. Reinforcing

$$A_{s\,min} \geq \frac{200(14\times12)(9)}{60{,}000} = 5.04\,in^2 < 13.43\,in^2 \checkmark$$

Max Reinforcing

$$A_{s\,max} = (0.85)(0.85)\left(\frac{4}{60}\right)\left(\frac{.003}{.003+.004}\right)(14\times12)(9) = 31.2\,in^2$$

$$31.2\,in^2 > 13.43\,in^2 \checkmark$$

min spacing

min | 1"
 | 1" ← controls

$$\text{actual spacing} = \frac{(14\times12) - 17(1)}{19} = 7.95'' > 1'' \checkmark$$

Middle Strip - negative moments

$$As^-_{min} = \frac{147.4}{4\,(9'')} = 4.09\ in^2 \rightarrow (12)\ \#6 = 5.28\ in^2$$

flexural strength

$$a = \frac{(5.28)(60)}{0.85(4)(14\times12)} = 0.55 \qquad c = \frac{0.55}{0.85} = 0.65$$

$$M_n = \frac{(5.28)(60)(9 - \frac{.55}{2})}{12} = 230\ ^{k\text{-}ft}$$

$$\varepsilon_s = \frac{0.003}{.65}(9 - 0.65) = 0.016 > .00207\ \checkmark$$

$$\phi M_n = 0.9(230) = 207\ ^{k\text{-}ft} > 147\ ^{k\text{-}ft}\ \checkmark$$

min reinforcing

$$As_{min} \geq \frac{200(14\times12)(4)}{60{,}000} = 5.04\ in^2 < 5.28\ in^2\ \checkmark$$

max reinforcing

$$As_{max} = 0.85^2\left(\frac{4}{60}\right)\left(\frac{.003}{.007}\right)(14\times12)(9) = 31.2\ in^2$$

$$5.28 < 31.2\ in^2\ \checkmark$$

min spacing = 1"

$$\text{actual spacing} = \frac{14\times12 - 12(.75)}{14} = 11.36\ in > 1''\ \checkmark$$

Column Strip – positive moment

$As^{+}_{min} = \frac{187.5}{4(9)} = 5.21 \rightarrow (12)\ \#6 = 5.28\ in^2$

flexural strength

From – middle strip negative moment

$a = 0.55$ $c = 0.65$ $M_n = 230^{k\text{-}ft}$

$\varepsilon = 0.016$

$\phi M_n = 207^{k\text{-}ft} > 187.5^{k\text{-}ft}$ ✓

all requirements met

spacing = 11.76 in

middle strip – positive moment

$As^{+}_{min} = \frac{125}{4(9)} = 3.47 \rightarrow$ use (12) #6 to meet As_{min} requirements

From middle Strip negative moment

$a = 0.55$ $c = 0.65$ $M_n = 230^{k\text{-}ft}$

$\varepsilon = 0.016$

$\phi M_n = 207^{k\text{-}ft} > 125^{k\text{-}ft}$ ✓

all requirements met

spacing = 11.36 in

Check to see if short span direction design is necessary

q_u=0.288klf

M_o=(0.288)(30)(28^2)/8 = 847 kip-feet

Distribute Moment

M_u^-=550 k-ft

M_u^+=296k-ft

Transverse distribution to interior negative column strip

0.75(550)=412.5k-ft

As_{min}=(412.5)/(4*8)=12.89in^2 Use (17) #8 bars: As=13.43in^2

No short direction design needed by inspection. The same reinforcement can be used in both directions.

Cost Comparison

All cost estimates were completed using RSMeans Online version 5.0.6 with a location of New Castle, PA. Interpolation was used to find values between bay sizes. The corrected total cost per square foot value is outlined in red in each systems respective table.

Existing Steel Joist System

Bay size: 28'x30'

Total Load: 53psf + 80psf = 133psf

Bay size (S.F.)	Total Load (psf)	Total Cost per S.F.
750	120	$16.91
840	133	$18.79
900	145	$20.04

Non-Composite Steel System

Bay size: 28'x30'

Total Load: 48psf + 80psf = 128psf

Bay size (S.F.)	Total Load (psf)	Total Cost per S.F.
750	125	$15.07
840	128	$16.29
900	125	$17.11

Composite Steel System

Bay size: 28'x30'

Total Load: 42psf + 80psf = 122 psf

Bay size (S.F.)	Total Load (psf)	Total Cost per S.F.
750	119	$17.64
840	122	$19.93
900	168	$21.46

Two-Way Flat Plate Slab

Bay size: 28'x30'

Total Load: 120psf + 100psf = 220psf

Bay size (S.F.)	Total Load (psf)	Total Cost per S.F.
750	250	$14.58
840	220	$15.59
900	269	$16.94

Floor System Design Comparisons

	Steel Joists	Non-Composite Steel	Composite Steel	Two-Way Flat Plate Slab
Cost	$18.71/S.F.	$16.29/S.F.	$19.91/S.F.	$15.59/S.F.
Weight	133psf	128psf	122psf	220psf
Max. Depth	24"	24"	18"	10"
Passive Fire Proofing	No	Yes	Yes	No
Active Fire Proofing	Yes	No	No	No
Fire Rating	1 hr.	2 hr.	2 hr.	4 hr.
Lateral System	Ivany Blockwall	Concrete Shearwall	Concrete Shearwall	Concrete Shearwall
Advantages	constructability	Lower square foot cost, higher fire rating	Lower weight, lower max. depth, higher fire rating	Lowest cost, lowest max. depth, higher fire rating
Disadvantages	High cost, high max. depth, low fire rating	Large max. depth	Highest cost	Highest weight, formwork required, low durability, low aesthetics
Feasible Redesign	N/A	Yes	Yes	Yes

Conclusions

A typical bay of the existing framing system was analyzed for gravity loads and determined to be sufficient to carry the loads. Three alternative framing systems were proposed and then implemented over the same bay. These systems included; concrete on metal deck supported by non-composite steel wide flange beams and girders, with steel wide flange columns, concrete on metal deck supported by composite wide flange beams and girders, with steel wide flange columns, and a two-way flat plate concrete slab supported by concrete columns. The three alternate systems all proved to be viable alternatives to the existing floor structure, however one system clearly proved to be the most sensible solution. The two-way flat plate concrete slab had the lowest estimated construction cost, lowest maximum and overall floor depths and highest fire rating out all the proposed alternatives.

The existing system has a lateral force resisting system comprised of Ivany block shear walls. All three potential redesigns were considered with the intent of utilizing a traditional concrete shear wall system to resist lateral forces. In all three alternative systems lateral loads would be transferred to the shear walls via the floor diaphragm.