NOTEBOOK SUBMISSION A

THE HEALTH CENTRE

LOCATION | SOUTHEASTERN US

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STRUCTURAL OPTION | ADVISOR: DR. HANAGAN

THE HEALTH CENTRE

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GENERAL INFORMATION

Location	Southeastern US
Occupancy	Healthcare
Height	166 ft
Total Levels	14 (above + below grade)
Size	450,000 SF of program space
Cost	$168-203 million
Construction	January 2012-2016 (projected)
Project Delivery	CM At-Risk

PROJECT TEAM

Architecture	SmithGroupJJR
Structural	Walter P. Moore
Lighting/Mechanical	ccrd
Construction	McCarthy Building Construction
Civil/Site	Kimley-Horn and Associates, Inc
Wind Consultant	RWDI Consulting Engineers

ARCHITECTURE

The Health Centre is a new "core-and-shell" university hospital expansion project featuring a nine-story hospital bed tower and state-of-the-art technical facilities. Inspired by the concept of lifelines, it takes architectural cues from surrounding classical campus buildings. A variety of health facilities are offered in the building, including operating rooms, an intensive care unit, emergency department, clinical facilities, and med-surg patient rooms.

STRUCTURAL SYSTEMS

Framing.......... Cast-in place concrete with one-way floor slabs are used for framing above grade. Post-tensioned two-way concrete slabs are used in the parking garage.

Foundations.....Slab on grade is connected by grade beams. Below grade are cast-in-place spread footings & drilled piers.

Lateral .... Concrete moment frames resist wind lateral loads. Parking garage shear walls resist seismic/soil loads.

MECHANICAL SYSTEMS

Two mechanical rooms service the building on the 5th and penthouse floors. Three large cooling towers go up to the roof. Fan coil units are used to heat the building. Custom central-station air-handling units utilizing split system air conditioners are used to cool the building.

LIGHTING/ELECTRICAL SYSTEMS

Interiors are lit with linear T8 and T5 LEDs fixtures, and energy efficient lamps. Surge protective devices were installed for low-voltage equipment. Both photoelectric switches and daylight-harvesting switching controls contribute to energy savings.

CONSTRUCTION

Special efforts have been made to ensure a sustainable construction process. Dirt and filling material from digging the foundations was used to build a new soccer field in the community. All trees removed during the building process are scheduled to be replanted.

RENDERINGS AND INFORMATION COURTESY OF:

SMITHGROUPJJR

WALTER P MOORE

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2 | General Information

2.1 Executive Summary

The Health Centre is a 450,000 square foot university hospital expansion project located in the southeastern United States. Located adjacent to existing hospital facility 'Clinic B,' this nine story L-shaped building is connected by two bridges to the surrounding campus. Demand for new, state-of-the-art medical technology, additional research space, and extra hospital beds prompted the design and construction the Health Centre. At a height of 163 feet, the Health Centre will be by far the tallest building in the surrounding area when its construction is complete in 2016.

As a nod to the heritage and character of the surrounding university campus, The Health Centre takes its architectural cues from classical Italian and contemporary sources. Façade materials used on the building include stucco, metal panels, and a glass curtain wall. A green roof and four story underground parking garage contribute towards its goal of LEED silver certification. This building was designed as a "core-and shell," necessitating a structural consideration for flexibility of spaces and future expansion.

The structure of the Health Centre is mainly cast-in-place concrete on drilled piers and spread footings. Its floor system in the hospital bed tower consists of cast-in-place one-way concrete slabs and beams. Concrete moment frames spread throughout the structure resist the building's lateral loads. Below grade, parking garage floor slabs consist of two-way post-tensioned concrete slabs. The parking garage has its own lateral system of concrete shear walls. Some structural steel components exist in the building, including roofing and bridges connecting to other buildings on campus.

Governing codes for the design of the Health Centre required the use of IBC 2012. However, an exemption was obtained to allow the structural design to use IBC 2006 requirements. ASCE 7-05 provides the minimum design loads for live, snow, wind, and seismic considerations. Due to the life safety importance associated with hospital structures, a conservative approach was used to determine building loads.

2.2 Site Plan

The Health Centre is located on a university campus in the southeastern United States. Adjacent to the site is 'Clinic B,' the existing hospital building. Bridges connect the hospital facilities to the surrounding campus. A new entry drive allows patients and emergency vehicles direct access to the new Health Centre. Figure 1 shows the site plans from SmithGroupJJR documents.

Terrain around the site is extremely flat. As the tallest building in the immediate area, The Health Centre will be fully impacted by wind loads.

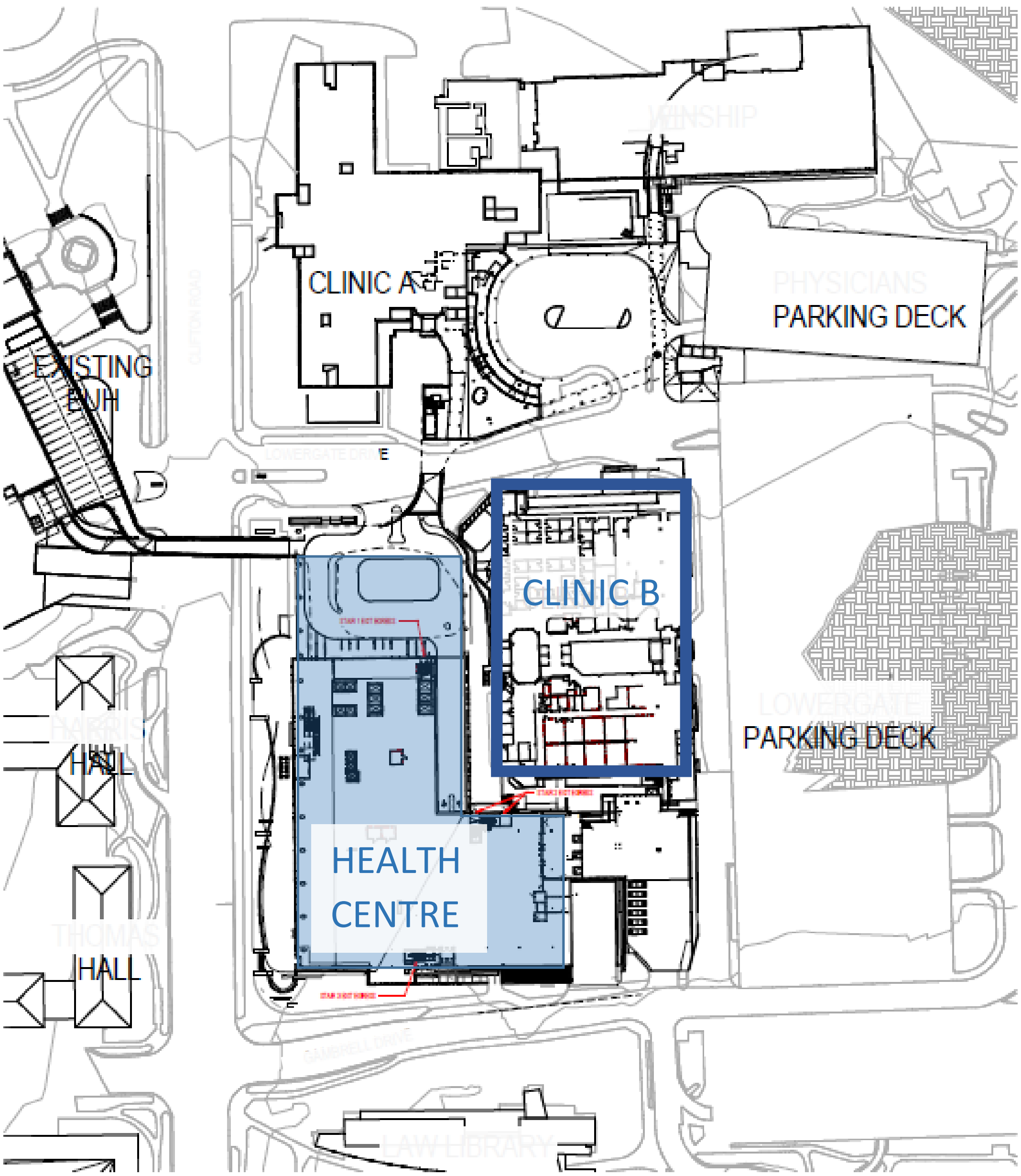


Figure 1 | Site Plan of Surrounding Area

2.3 References

The following table is a list of documents referenced during the preparation of Notebook A to determine building loads.

Organization	Reference
International Building Code	2006 International Building Code
American Society of Civil Engineers	ASCE 7-05 | Minimum Design Loads for Buildings and Other Structures
American Concrete Institute	ACI 318 | Building Code Requirements for Structural Concrete
American Institute of Steel Construction	Steel Construction Manual, 14^{th} Edition
United States Geological Survey	Seismic Design Maps
Penn State	Architectural Engineering Course Notes
Vulcraft	Deck Catalog
Walter P. Moore	Health Centre General Notes Sheet

Table 1 | Notebook A References

3 | Gravity Loads

This section details the building gravity loads due to dead, live, and snow, and perimeter loads. Loads were determined using structural documentation from Walter P. Moore and the references listed in the previous section. A full list of design gravity loads used by the original structural engineer may be found in Appendix A.

3.1 Roof Loads

Three roof gravity load cases exist for this building: typical concrete roof, penthouse roof, and green roof. Figures 2-4 depict the roof sections that correspond with each load case.

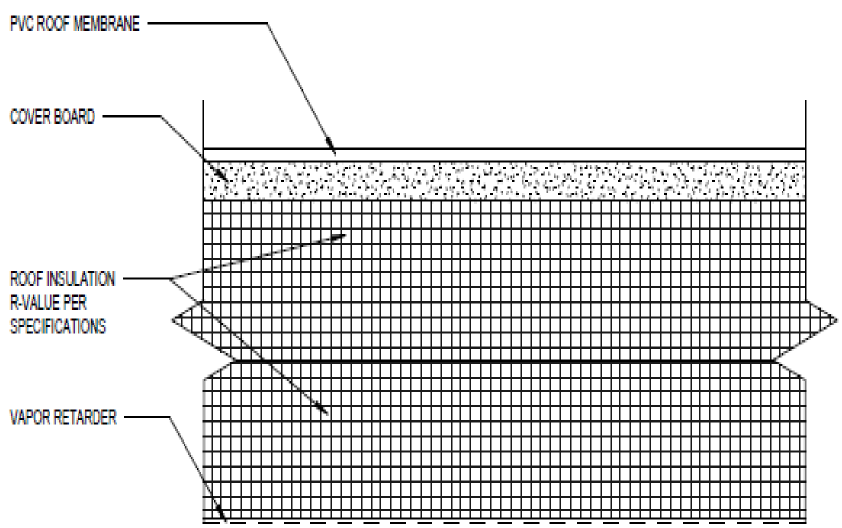


Figure 2 | Typical Concrete Roof Section (SmithGroupJJR)

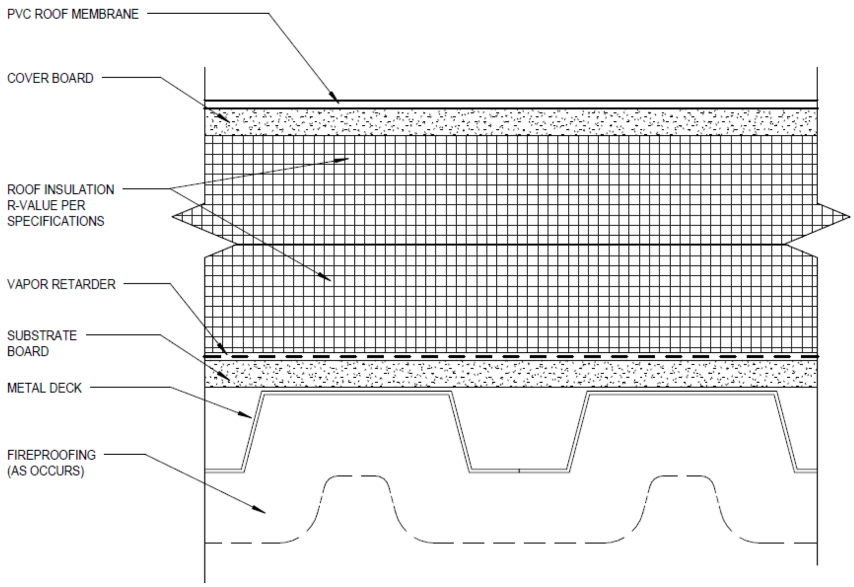


Figure 3 | Penthouse Roof Section (SmithGroupJJR)

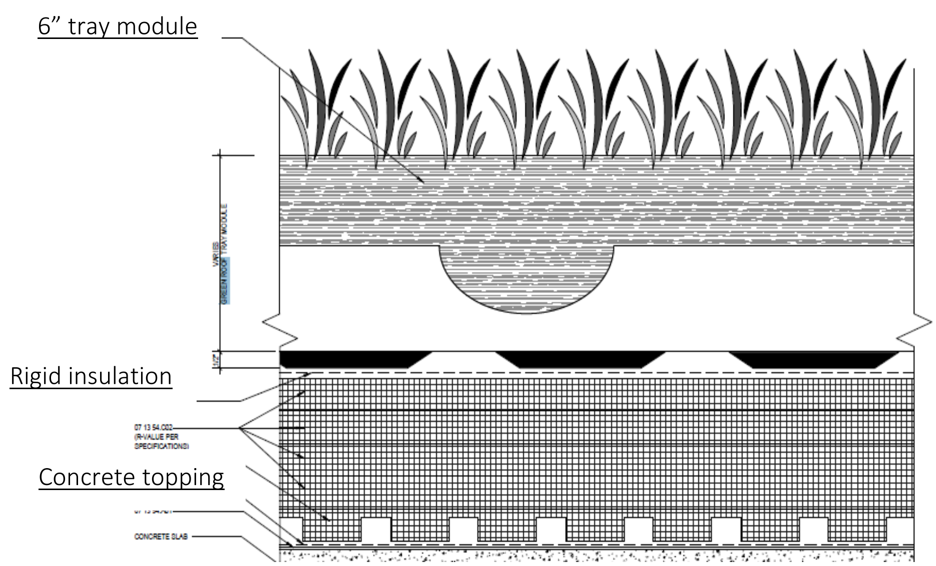


Figure 4 | Green Roof Section (SmithGroupJJR)

Gravity Loads - Roof

Dead Loads:

1) Green Roof System

6" Green Roof Tray Module	30 psf
Landscaping Stone	5 psf
Insulation	5 psf
Concrete Slab *(see note below)	63 psf
	103 psf

* See floor loads for 5" conc slab ca

2) Typical Concrete Roof

Roof Finishes + Insulation	5 psf
Concrete Slab + Framing	63 psf
MEP	15 psf
	83 psf

3) Penthouse Roof

Roof Finishes + Insulation	5 psf
Steel Framing	15 psf
MEP	15 psf
Deck - 1.5" 22GA Type B	1.78 psf
Misc	3 psf
	40 psf

Live Loads:

1) Green Roof System — 100 psf
Yards + Terraces, pedestrian - Not Reduced

2) Typical Roof — 20 psf
Ordinary Flat Roof - Not Reduced

3) Penthouse Roof — 20 psf
Ordinary Flat Roof - Not Reduced

Live Loads from ASCE 7-05 Table 4-1

Below is a summary of the roof gravity dead and live load values determined in this section.

Load Type	Dead	Live
Typical Roof	115 psf	20 psf Not reduced
Penthouse Roof	40 psf (50 psf SDL from structural drawings)	20 psf Not reduced
Green Roof	135 psf	100 psf Not reduced

Table 2 | Roof Gravity Load Summary

A flat roof snow load for the building is calculated below, but will not control design. Snow drift will be considered for the green roof and lower 6th level roof due to the large height difference between these levels and the penthouse roof. The 6th level is designed for future expansion and may become an enclosed floor in the future. Floor live loads for the 6th level roof will likely control.

Snow Loads:

By inspection, snow loads will not control.

$p_f = 0.7 C_e C_t I p_g$ (7-1)

$p_g = 5$ psf (Fig. 7-1)

$C_e = 0.9$ (Table 7-2)

$C_t = 1.0$ (Table 7-3)

$I = 1.2$ (Table 7-4)

$p_f = 0.7(0.9)(1.0)(1.2)(5) = 4$ psf

Section 7-7 – Drifts on Lower Roofs

1) Snow Drift on 4th Floor Green Roof From Penthouse Level (117 ft)

$$h_d = \max \begin{cases} 2 \text{ ft (leeward)} \\ <1 \text{ ft (windward)} \end{cases} \text{(fig 7-9)} = 2 \text{ ft} \quad \left(\begin{matrix}\text{Section}\\ 7\text{-}7\end{matrix}\right)$$

$$\gamma = 0.13p_g + 14 = 0.13(5) + 14 = 14.65 \quad (7\text{-}2)$$

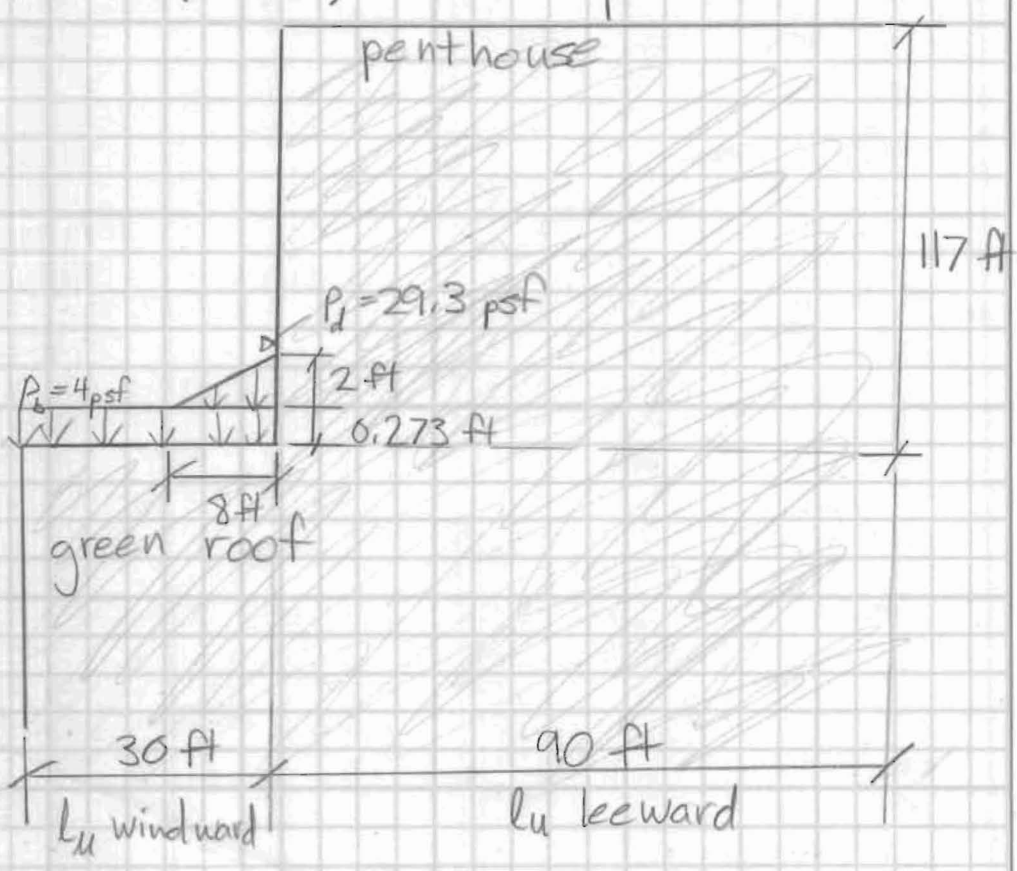
$$h_b = P_o/_{14.65} = 4/_{14.65} = 0.273 \text{ ft}$$

$$h_c \approx 117 \text{ ft}$$

$$h_c/h_b > 0.2 \therefore \text{Must calculate drift}$$

$$h_d < h_c \therefore W = 4h_d = 4(2) = 8 \text{ ft}$$

$$P_d = h_d \gamma = 2(14.65) = 29.3 \text{ psf}$$



2) Snow Drift on 6^{th} Floor From Penthouse Roof Level (68 ft)

$\gamma = 14.65$

$h_b = 0.273$

$h_c \approx 68$ ft

$h_c/h_b > 0.2$ ∴ Must calculate drift

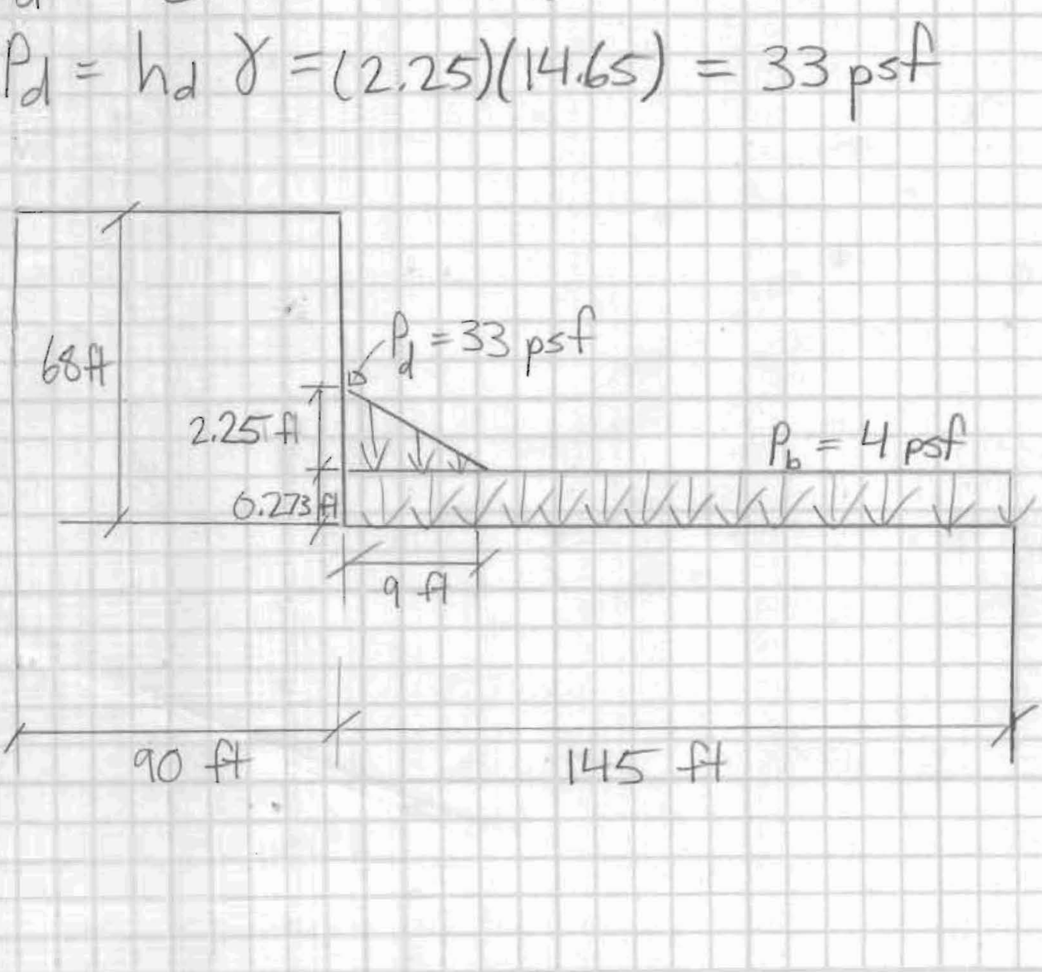
$h_d = \Big|_{max} \begin{matrix} 2 \\ 3/4(3) = \underline{2.25 \text{ ft}} \end{matrix}$ 145 ft

$l_u = 90$ ft leeward

$l_u = 145$ ft windward

$h_d < h_c$ ∴ $w = 4h_d = 4(2.25) = 9$ ft

$P_d = h_d \gamma = (2.25)(14.65) = 33$ psf



3.2 Floor Loads

Floor dead and live loads will be determined for both the bed tower and parking garage floor systems in this report. On the following page, Figures 6 shows typical details for the floor slabs under consideration for this report. Concrete floor slabs in the bed tower are typically 5 or 7 inches. All dead load values are based on the typical bay pictured in Figure 5.

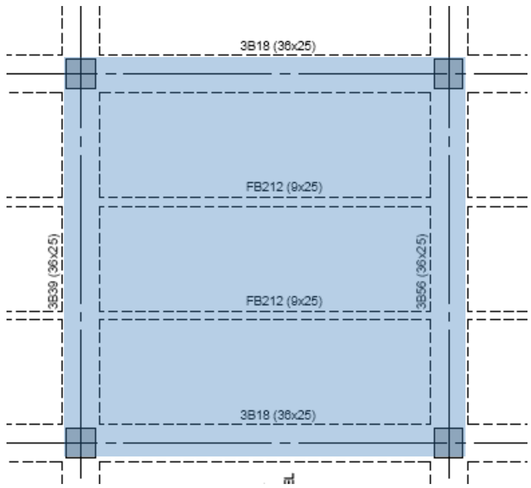


Figure 5 | Typical Bay from Third Floor Area D Floor Plan (Walter P. Moore)

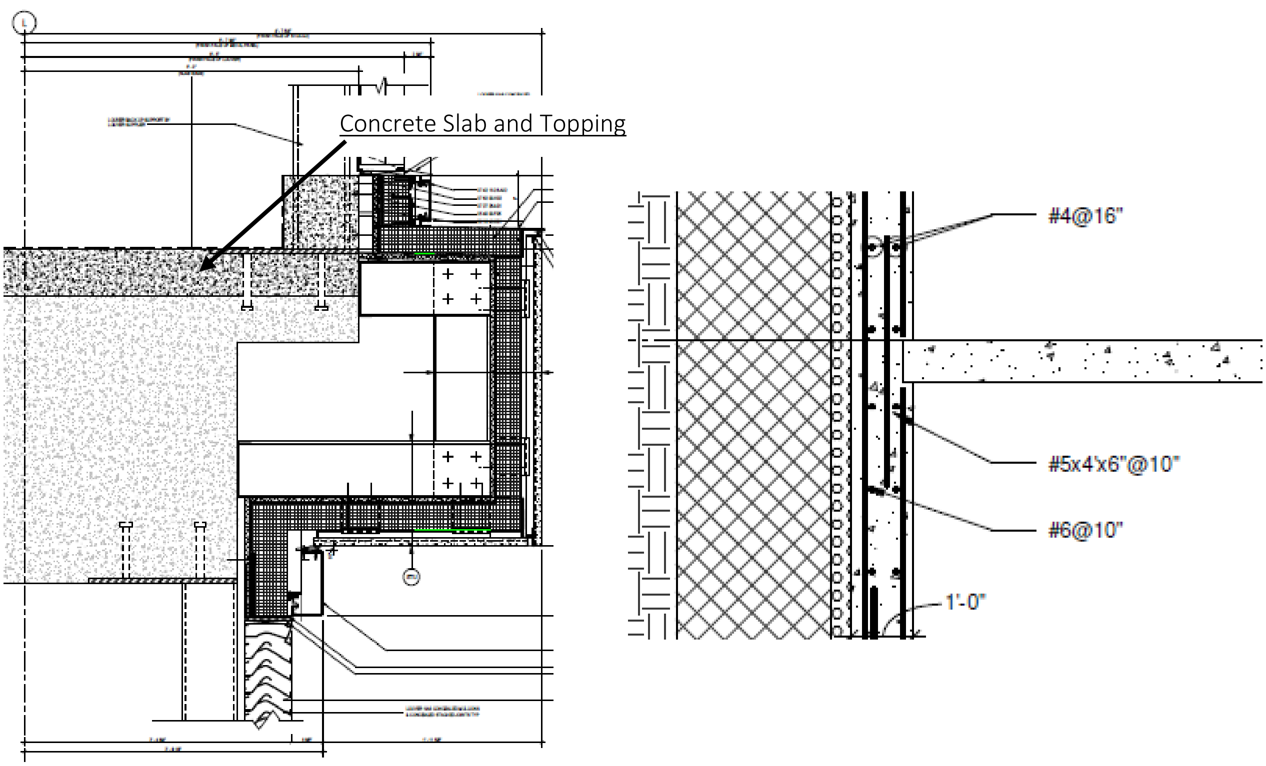


Figure 6 | Floor Section in Bed Tower (left, SmithGroupJJR) and Parking Garage (right, Walter P. Moore)

Floor Use	Dead		Live
Typical Hospital Areas	5” slab – 120 psf 7” slab – 145 psf		100 psf – reduced (design value)
Corridors + Lobbies	5” slab – 120 psf 7” slab – 145 psf		100 psf
Stairs	5” slab – 120 psf 7” slab – 145 psf		100 psf
Mechanical Rooms	5” slab – 120 psf 7” slab – 145 psf	+ 200 K mech. equip	150 psf
Diagnostics + Imaging	5” slab – 120 psf 7” slab – 145 psf	+ 80 K diagnostic equip.	350 psf – not reduced (design value)
Patient Rooms (Designed as Hospital – Corridor)	5” slab – 120 psf 7” slab – 145 psf		80 psf
Parking Garage	5” slab – 120 psf 7” slab – 145 psf		40 psf

Table 3 | Floor Gravity Loads Summary

Bed Tower Dead Loads – Concrete

Thickness	Area + Floors	Slab Wt	Slab + Floor Loads Total Wt.	Seismic Wt (total + framing)
5"	2B,C,D 3B,C,D 4A,B,C,D 5A,B,C,D 6A,B,C,D 7(all), 8(all) 9(all), penthouse	$5'' \cdot \frac{1ft}{12''} \times 150 = 62.5$ psf	86 psf	117.5 psf ≈ 120 psf
7"	2B, 3B, 4B 5A/B, 6A/B 7A/B, 8(all) 9(all)	$7'' \cdot \frac{1ft}{12''} \times 150 = 87.5$ psf	111 psf	142.5 psf ≈ 145 psf

Occurances of all other slab thicknesses are infrequent and in small areas

5000 psi NWC → 150 pcf

Typical Bay: $4 \times ((36''\times(25-5)'' \times 1\,ft^2/144\,in^2 \times 30') = 150\,ft^3$

$2 \times ((9''\times(25-5)'' \times 1\,ft^2/144\,in^2 \times 30') = 37.5\,ft^3$

Beam Wt $(150 + 37.5\,ft^3)(150\,pcf) = 28125$ lbs

$\frac{28125\ lbs}{30' \times 30'\ bay} = 32$ psf

Floor Loads

Finishes + Ceiling	5 psf
Concrete Slab	see above
MEP	15 psf
Misc.	3 psf
	23 psf + slab

Bed Tower Dead Loads - Elevator Machine Room

3 1/2" LWC on 2" Deep 20GA Composite Deck
Vulcraft Value: 57 psf

Steel Framing	15 psf
Finishes + Ceiling	5 psf
Metal Deck	57 psf
MEP	15 psf
Elev. Equip	20 psf
Misc.	3 psf
	$\boxed{115 \text{ psf}}$

Parking Garage Dead Loads

Typical Depth = 8" NWC (structural plan notes)

Slab Wt = $8'' \cdot \frac{1}{12} \cdot 150$ pcf = 100 psf

Concrete Slab	100 psf
Misc. (lighting, etc)	5 psf
	105 psf

Floor Live Loads - ASCE 7-05 Table 4-1

Bed Tower

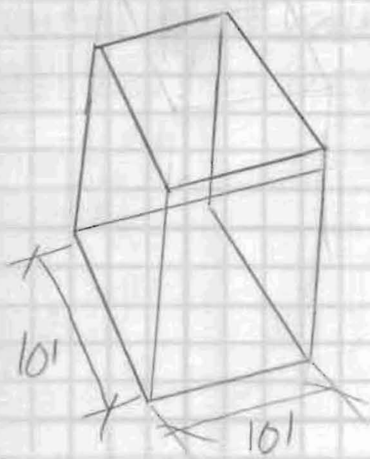
Typical Hospital Areas (restroom, etc)	100 RED (design value)
Corridors + Lobbies	100 psf
Stairs	100 psf
Mechanical Rooms	150 psf (design value)
Diagnostics + Imaging	350 psf NR (design value)
Patient Rooms (Hospital-Corridor)	80 psf
Light Storage	125 psf NR

Parking Garage - 40 psf

Non-typical Loads Considered:

- Green roof (see roof loads)
- Diagnostic Equipment
  - 350 psf design live load in some diagnostic areas.
  - Equip. weight unknown from specs
  - Assume 10'x10' machine...

$$350 \text{ psf} \times (10 \text{ ft})^2 = \underline{35 \text{ k}}$$



3.3 Perimeter Loads

The building perimeter enclosure produces a linear dead load through its attachment to the main building structure. The Health Centre has three main enclosure systems: curtain wall, stucco panels, and metal panels. Figures 7-8 depict the methods of attachment for each system.

Each system has a different load path that is dependent on its connection to the structure. The curtain wall's framing system is connected to the main structure by a structural steel plate and embedded metal stud.

Loads transfer from the stucco wall via continuous light gauge angles attached to continuous light gauge zees. The light gauge zees are connected by a fiberglass thermal spacer clip to gypsum sheathing, which takes the load to the main structure via another light gauge zee.

A light gauge zee connects the metal wall panels to the main structure, and load is transferred through steel bolts.

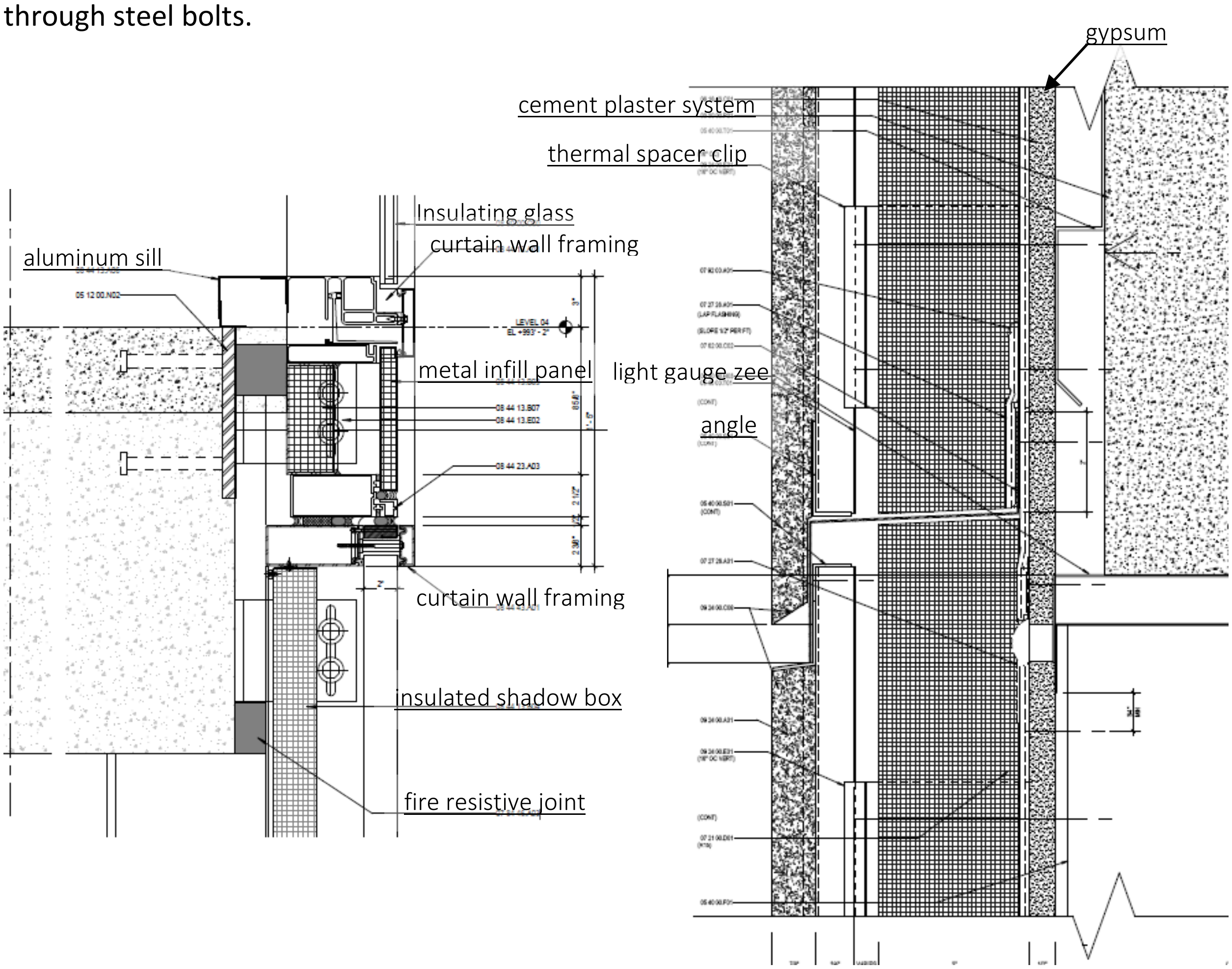


Figure 7 | Curtain Wall Connection Detail (left) and Stucco Panel Wall Envelope (right) from SmithGroupJJR

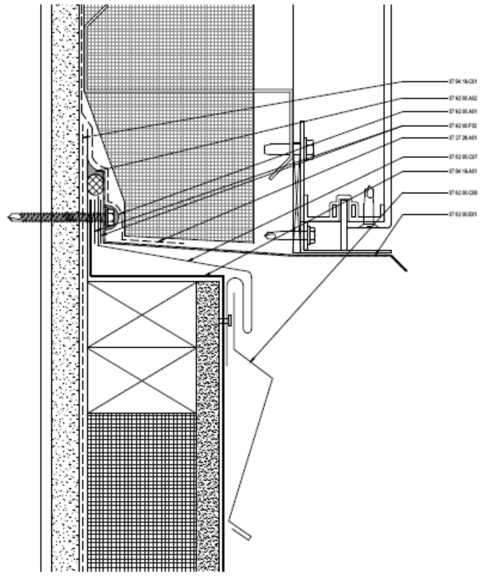


Figure 8 | Metal Panel Connection Detail (SmithGroupJJR)

Perimeter Enclosure Dead Loads

Curtain Wall

2" Insulating Glass	1 psf
Curtain Wall Framing	4 psf
Insulating Shadow Box / 3" Rigid Insulation	5 psf
Interior Sheathing ½"	2 psf
Misc. (sealants for joints, etc)	4 psf
	16 psf

Panel Systems

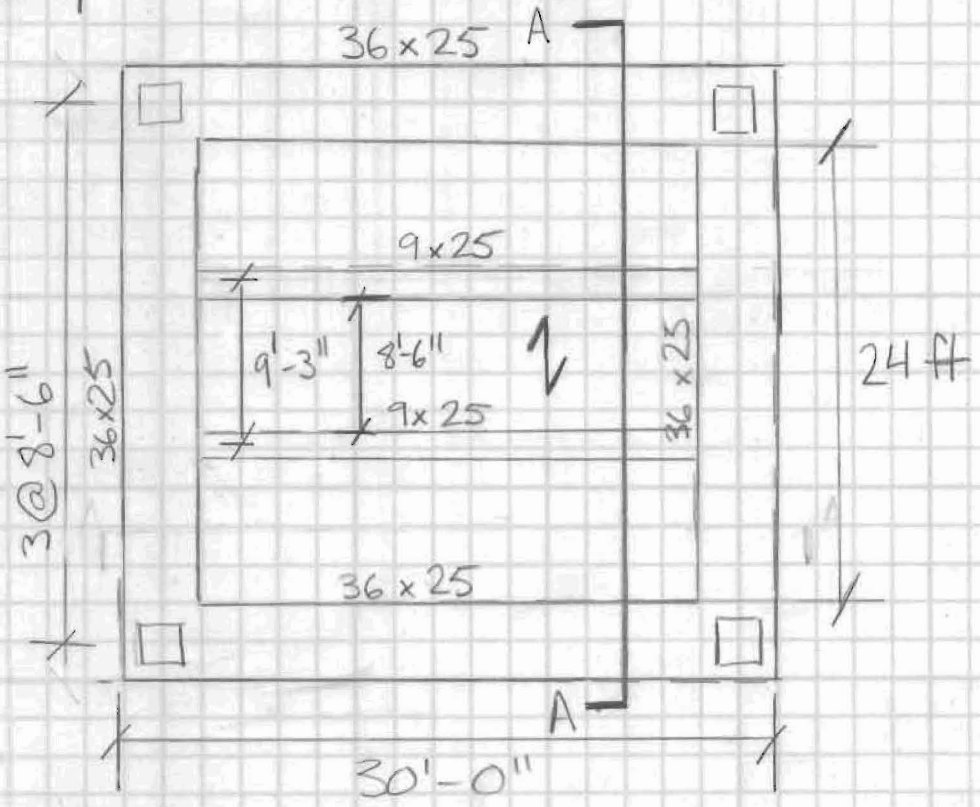
Stucco/Metal Panels (7/8")	10 psf
3" Rigid Insulation	4 psf
½" Gypsum Interior	2 psf
Metal Framing / Air Gap	2 psf
	0 psf
	18 psf

* Building material weights via AISC Steel Manual, 14th edition

4 | Typical Spot Checks for Gravity Loads

The following section analyzes the existing gravity framing system of the Health Centre. The existing system is a one-way cast-in-place concrete slab with intermediate concrete beams. Framing members and slab were analyzed for flexural and shear capacity.

One-Way Slab w/ Intermediate Beams



One-way Slab
- 5" thick
- Top Bars: #3 @ 12"
- Bottom Bars: #3 @ 12"
- Temp Shrinkage: #4 @ 18"

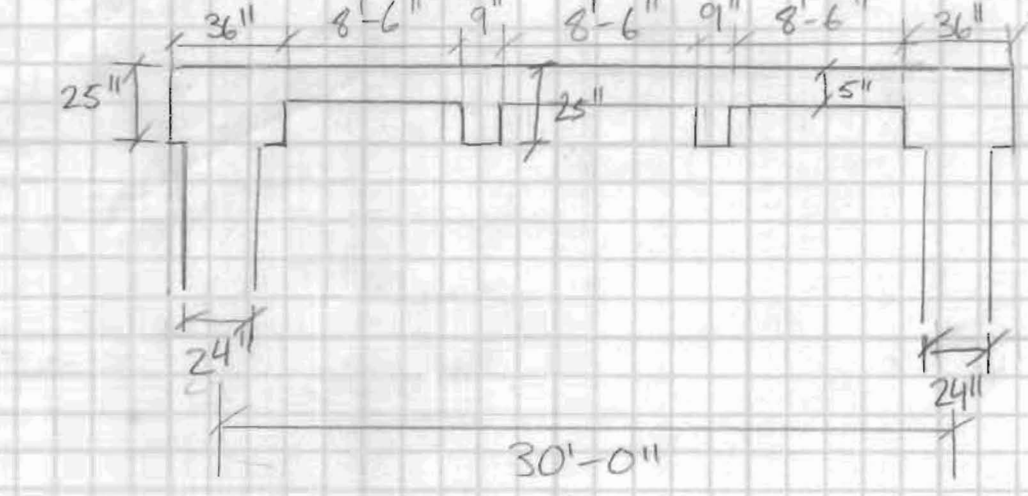
(Top and bottom bars: 3/4" cover)

Floor Dead: 86 psf (includes slab self-weight)
Floor Live: 80 psf

Spot Check: 1-Way Slab

- Min thickness $= l/28 = \frac{30' \times 12 \text{ in/ft}}{28} = 12.85" > 5"$

$\therefore$ Must calculate deflections (Table 9.5(a) ACI)



One bay Cross Section

Load Combinations

$1.4D = 1.4(86\text{ psf}) = 120.6\text{ psf}$

$1.2D + 1.6L = 1.2(86\text{ psf}) + 1.6(80\text{ psf}) = \boxed{231\text{ psf}}$

For 1ft of slab section: $w = 231$ plf

Max Moment will be @ exterior face of first interior support: $M_u = \dfrac{w_u l_n^2}{10}$ (ACI 8.3.3)

$$M_u = \frac{(231\text{ plf})(8.5\text{ ft})^2}{10} = \underline{1.67\text{ ft-k/ft}}$$

$$d = 5" - 0.75" - \frac{0.375"}{2} = 4.06"$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.11\text{ in}^2)(60)}{0.85(5)(12\text{ in})} = 0.13\text{ in}$$

Moment Capacity

$$\phi M_n = \phi f_y A_s \left(d - \frac{a}{2}\right) = 0.9(60)(0.11\text{ in}^2)(4.06 - 0.13)$$
$$= 23.3\text{ in-k} = \underline{1.94\text{ ft-k}}$$

$1.94\text{ ft-k/ft} > 1.67\text{ ft-k/ft}$ <u>OK</u>

Shear Capacity:

$$V_u = \frac{1.15 w_u l_n}{2} = \frac{1.15(231\text{ lb/ft})(8.5\text{ ft})}{2} = 1.13\text{K}$$

(ACI 8.3.3)

$V_c = 2\lambda\sqrt{f'_c}\, b_w d = 2(1)\sqrt{5000}(12\text{ in})(4.06") = 6.89\text{ k}$

$\phi = 0.75,\ \phi V_c = 5.17\text{ k} > 1.13\text{ k}$ OK

Min Steel Check:

$A_{s,min} = 0.0018bh = 0.0018(12")(5") = 0.108\text{ in}^2 < 0.11\text{ in}^2$ OK

Max Spacing Check:

$$S_{max} = \min \begin{cases} 15\left(\frac{40{,}000}{40{,}000}\right) - 2.5(0.75) = 13.1\text{ in} \\ 12\left(\frac{40{,}000}{40{,}000}\right) = \boxed{12\text{ in}} \\ 18" \end{cases}$$

(ACI 10.6.4)

$f_s = 2/3 f_y = 40{,}000\text{ psi}$

$\boxed{\therefore \#3 @ 12"\text{ top + bottom flexural reinf. checks}}$

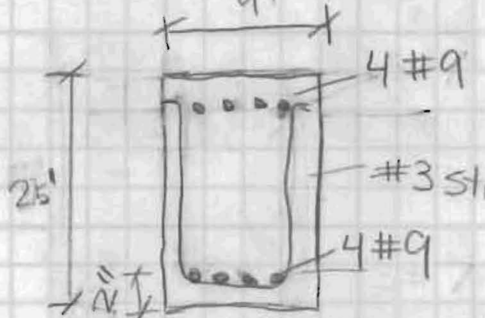
Shrinkage Reinf:

$A_s = 0.0018bh = 0.108\text{ in}^2$

$$S_{max} = \min \begin{cases} 5h = 25" \\ \boxed{18"} \end{cases}$$

$\boxed{\therefore \#4 @ 18"\text{ temp shrinkage reinf. checks}}$

Spot Check – 9x25 Interior Beam FB231

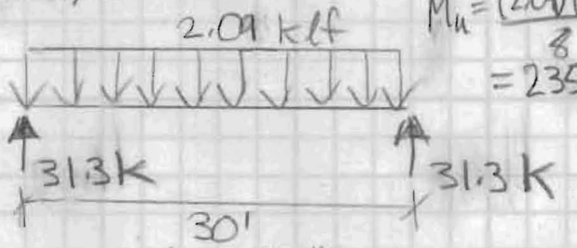


Live = 80 psf
Dead = 86 psf + bm wt.

$W_u = 1.2[8.5(86) + 150 \cdot \frac{(20.9)}{144}] + 1.6(72.8)(8.5)]$

$W_u = 2.09$ klf

$d = 25" - 2 - \left(\frac{1.128}{2}\right) = 22.5"$



$M_u = \frac{(2.09)(30)^2}{8} = 235$ ft-k

Can reduce LL?

$K_{LL}A_T = 2(8.5)(30) = 510 \text{ ft}^2 > 400 \text{ ft}^2$ ∴ Yes

$L_{red} = \max \begin{cases} 0.5L_o \\ 0.25 + \frac{15}{\sqrt{K_{LL}A_T}} = 0.91(80) = 72.8 \end{cases}$

$l_n = 8'\text{-}6"$, $l = 9.3$ ft

Flexural Strength:

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(60)(4 \text{ in}^2)}{0.85(5)(9")} = 6.27"$ $c = \frac{6.27}{0.8} = 7.84"$

$\beta_1 = 0.8$

$\phi M_n = \phi A_s f_y (d - \frac{a}{2}) / 12"/\text{ft}$

$= 0.9(4 \text{ in}^2)(60)(22.5 - \frac{6.27}{2}) / 12 = 348 \text{ ft-k} > 235 \text{ ft-k}$ OK

Steel yielding?

$\varepsilon_s = \varepsilon_c \frac{d-c}{c} = \frac{0.003}{7.84}(22.5 - 7.84) = 0.0056 > 0.00207$ ✓

∴ Beam ok for flexure

Steel Area Req.

$A_{s,min} = \frac{3\sqrt{f'_c}\, b_w d}{f_y} \geq \frac{200 b_w d}{f_y} = \frac{200(9)(22.5)}{60{,}000} = 0.675 \text{ in}^2$

$= \frac{3(\sqrt{5000})(9)(22.5)}{60{,}000} = 0.0795 \text{ in}^2$

$4 \text{ in}^2 > 0.675 \text{ in}^2$ OK

$A_{s,max}$: $\rho = \frac{A_s}{A_{conc}} = \frac{4 in^2}{9 \times 25} = 0.01778$

$\rho_{max} = 0.0213$ for $\beta_1 = 0.80$, $f'_c = 5000$, $f_y = 60$ ksi

$0.0213 > 0.01778$ Ok

Min Spacing

$S_{min} = \max \begin{cases} d_b = 1.128'' \\ 1'' = 1'' \\ 4/3\, S_a = 4/3'' \text{ controls} \end{cases}$

$\text{Spacing} = \frac{9 - 2(1.5) + 2(0.5) - 4(1.128)}{3} = 0.163''$

∴ Does not meet min spacing req.

Shear Strength:

Note: All beams sized for lateral + gravity loads by Walter P. Moore. This will affect design values matching calculated values.

V_u @ distance d from support

$V_u = 31.3k \left[-\left(22.5/2 \; /12in/ft\right) \right] = 29.3\ k$

$V_c = 2\lambda\sqrt{f'_c}\, b_w d = 2(1)\sqrt{5000}\,(9)(22.5) = 28.6k$

$\frac{V_u}{\phi} = \frac{29.3}{0.75} = 39.1\ k > 28.6k$

∴ Shear stirrups needed

$V_{s,req} = 39.1 - 28.6 = 10.5\ k$

$V_s = 8\sqrt{f'_c}\, b_w d = 8\sqrt{5000}\,(9)(22.5) = 114.5k \geq 10.5k$

Ok

$V_s \leq 4\sqrt{f'_c}\, b_w d$

$10.5 \leq 4\sqrt{5000}\,(9)(22.5) = 57.3\text{k}$ ✓

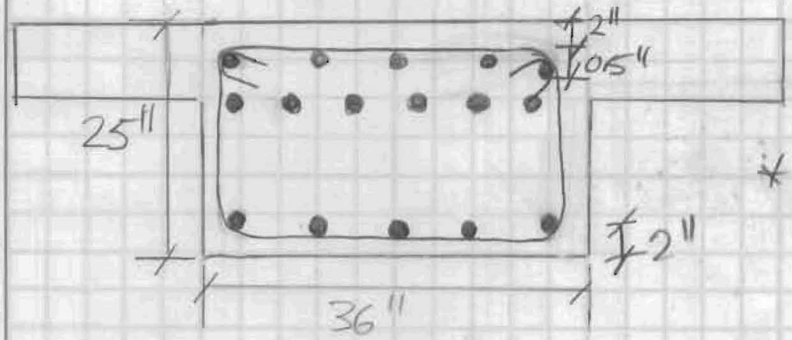
$S_{max} = \min \begin{cases} d/2 = 22.5/2 = 11.25'' > 11'' \ \underline{\text{OK}} \\ 24'' \end{cases}$

Minimum Shear Reinforcing

$A_{v,min} = \max \begin{cases} 0.75\sqrt{5000}\,(9)(11)/60000 = \underline{0.087 \text{ in}^2} \\ 50(9)(11)/60000 = 0.082 \end{cases}$

#3 stirrups at 11" Checks ✓

Spot Check - 36 x 25 Interior Girder 3B39

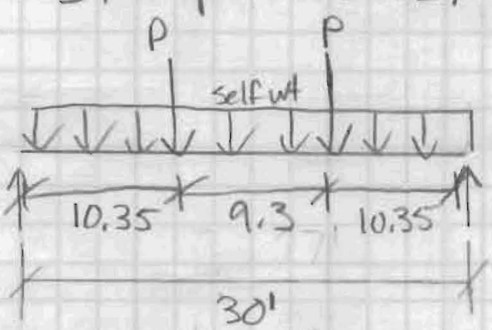


* Note: All concrete beams have extra reinf. because designed for lateral and gravity.

Top Bars: 5-#6, 6-#8 $\quad A_s = 6(0.79\,in^2) + 5(0.44\,in^2) = 6.94\,in^2$

Bottom Bars: 5-#8 $\quad A_s = 5(0.79\,in^2) = 3.95\,in^2$

Stirrups: #3 S27 $\quad$ 1@2/10@5/R@10



$P_D = \left(8.5(86) + \frac{150}{\frac{(20.9)}{144}}\right)30 = 25.5\,k$

$P_L = (8.5)(30)(48) = 12.2\,k$

$l_n = (30 \times 12) - 36" = 324" = 27'$

$L_{red} \begin{vmatrix} 0.5 \\ 0.25 + \frac{15}{\sqrt{2(30)^2}} = 0.6(80) = 48 \end{vmatrix} k$
max

Flexural Strength:

$P_u = 1.2(25.5\,k) + 1.6(12.2\,k) \quad W_D = 150\,\frac{lb}{ft^3} \times \left(\frac{36 \cdot 20}{144}\right) = 0.750\,klf$

$P_u = 50.1\,k$

$w_u = 1.2(0.750\,klf) = 0.9\,klf$

See following pages for M_u and V_u value determination

$M_{u,max} = 619.8\,ft\text{-}k$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(3.95\,in^2)(60)}{0.85(5)(36)} = 1.55" \quad c = \frac{1.55}{0.8} = 1.94"$

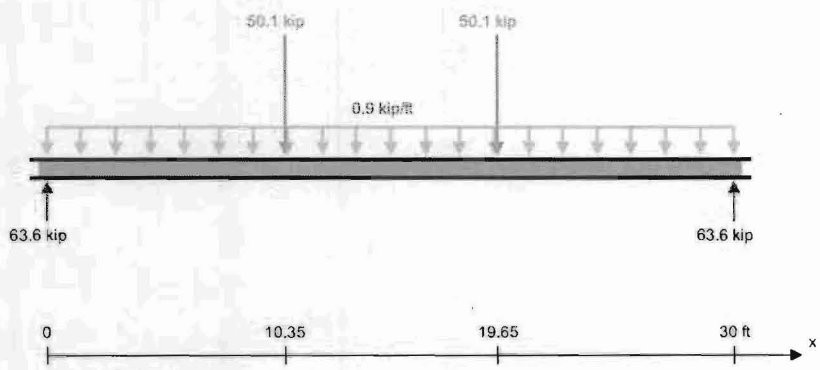
$\phi M_{n_1} = 0.9(3.95)\left(22.5 - \frac{1.55"}{2}\right)(60) = 386\,ft\text{-}k$

$\therefore$ Look at doubly reinf. section...

8

The FREE Shear & Moment Calculator has solved 748,407 bending moment diagrams and counting...

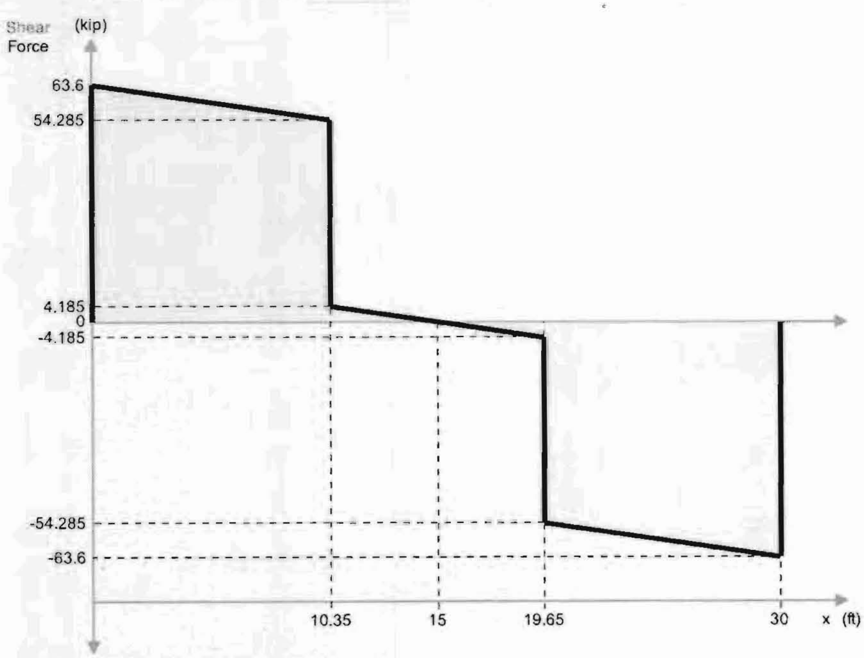
Free Body Diagram (FBD)



Full Working/Hand Solution for Reaction Forces

Shear Force Diagram (SFD)

The Shear Force (V) at ft along the beam is: (No Location Entered)

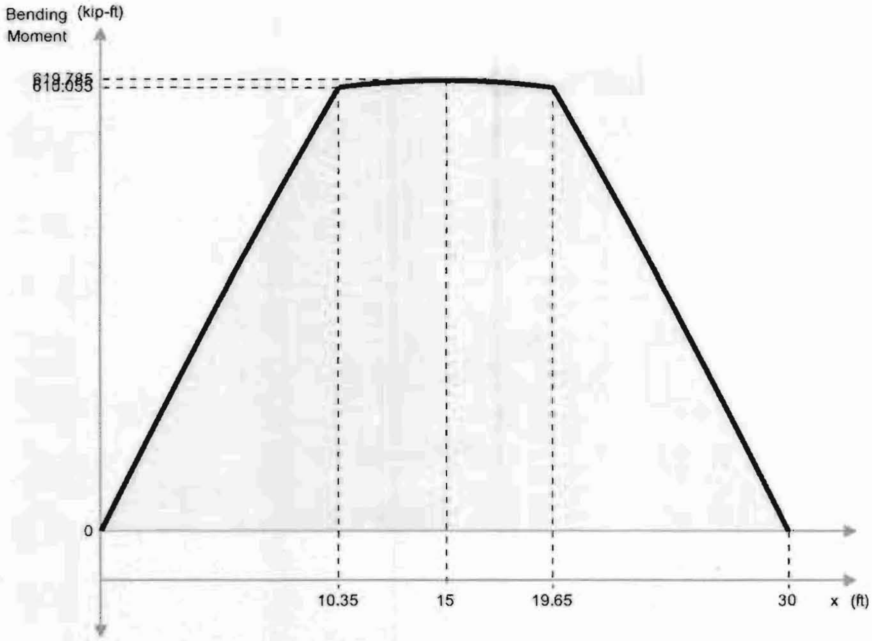


Full Working/Hand Solution for Shear Force Diagram

Bending Moment Diagram (BMD) Reverse BMD Sign Convention

The Bending Moment (M) at ft along the beam is: (No Location Entered)

9



Full Working/Hand Solution for Bending Moment Diagram

$$d' = 2'' + 0.75'' + \frac{0.5''}{2} = 3''$$

$$A_{s2} = 6.94 \text{ in}^2$$

$$M_{n2} = (6.94 \text{ in}^2)(60)(22.5 - 3 \cdot \tfrac{1}{12}) = 676.7 \text{ ft-k}$$

$$M_n = 386 \text{ ft-k} + 676.7 \text{ ft-k} = 1063 \text{ ft-k}$$

$$\phi M_n = 0.9\,(1063) = 956 \text{ ft-k}$$
$$0.75(1063) = 797 \text{ ft-k} > 619.8 \text{ ft-k}$$

Check Strains...

$$\varepsilon_s = \frac{0.003(22.5 - 1.94)}{1.94} = 0.0318$$

$$\varepsilon_s' = \frac{0.003(3 - 1.94)}{1.94} = 0.002$$

Steel close to
not yielding, some reinf. not flexural

Steel Area Req'

$$A_{s,min} = \frac{3\sqrt{f'_c}\, b_w d}{f_y} \geq \frac{200\, b_w d}{f_y}$$

$$= \frac{3\sqrt{5000}(36)(25)}{60{,}000} \geq \frac{200(36)(25)}{60{,}000}$$

$$= 3.18 \text{ in}^2 \geq 3 \text{ in}^2$$

OK

$$A_{s,max}: \; \rho = \frac{A_s}{A_c} = \frac{6.94 + 3.95}{36 \times 25} = 0.0121$$

$$\rho_{max} = 0.0213 > 0.0121 \quad \underline{\text{OK}}$$

Min Spacing

$$S_{min} = \begin{cases} d_b = 1'' \\ 1'' = 1'' \\ 4/3\, S_a = 4/3'' \end{cases}$$

$$\text{Smallest Spacing} = \frac{36 - 2(1.5) - 2(0.5) - 6(1'')}{5} = 5.2''$$

OK

Shear Strength:

V_u @ distance d from support

Slope of shear from previous diag...

$$\frac{63.6 - 54.285}{10.35} = 0.89, \quad d = 22.5 \text{ in} = 1.875 \text{ ft}$$

$$V_u = 63.6 - (0.89)(1.875 \text{ ft}) = \underline{61.9 \text{ k}}$$

$$V_s = 8\sqrt{f'_c}\, b_w d = 8\sqrt{5000}(36)(22.5) = 458.2 \text{ k} > 61.9 \text{ k}$$

∴ Shear stirrups not required, will check spacing of #3 stirrups

$V_s \leq 4\sqrt{f'_c}\, b_w d$?

$$\leq 4\sqrt{5000}(36)(22.5) = 229 \text{ k} \checkmark$$

$$S_{max} = \begin{cases} d/2 = 22.5/2 = 11.25'' > 10'' \ \underline{\text{Ok}} \\ 24 \end{cases}$$

#3 stirrups w/ max spacing @ 10'' checks ✓

Typical Column D12

2 lower parking levels: 28x38 w/ 22#9
2 upper parking levels: 28x38 w/ 14#8
levels 1-4: 28x32 w/ 12#8
levels 4-9: 24x24 w/ 8#8
level 9 - pent roof: 24x24 w/ 12#9

Column Loads

See following excel. Note that roof, penthouse, and parking live loads are not reduced.

Live load reduction calc: 9th floor

$$L = 80 \times \left| \begin{matrix} 0.4 \\ 0.25 + \frac{15}{\sqrt{4(900)}} \end{matrix} \right._{max} = 40 \text{ psf}$$

8th floor

$$L = 80 \times \left| \begin{matrix} 0.4 \\ 0.25 + \frac{15}{\sqrt{4\underbrace{(2 \cdot 900)}_{2 \text{ floors}}}} \end{matrix} \right._{max} = 32 \text{ psf}$$

See excel for all values. No reducible LL floor has column supporting 1 floor, $0.5L_0$ not a limit.

Snow Loads do not control for typical column.

Axial loads $= (1.2D + 1.6L)(900 \text{ ft}^2)$ per floor

Roof trib area $= (15 + 8.333)(30) = 700 \text{ ft}^2$

See excel for all axial values

$$\text{Self-Weight} = \frac{150 \text{ pcf}}{1000}\left(\frac{28 \times 38}{144}(47.3) + \frac{28 \times 32}{144}(49) + \frac{24 \times 24}{144}(117.8)\right) = 169 \text{k}$$

Controlling Case $= 1.2D + 1.6L + 0.5L_r$

$$P_u = 1.2(1188 + 169) + 1.6(576.13) + 0.5(14) = \boxed{2557.2 \text{ k}}$$

TOTAL VALUES FROM EXCEL

Column Strength Check

$$\phi P_n = 0.8\phi[0.85 f'_c(A_g - A_s) + f_y A_s)]$$

$f'_c = 5000$ psi

$\phi = 0.65$

$A_g = 28" \times 38" = 1064 \text{ in}^2$

$A_s = 22(1 \text{in}^2) = 22 \text{ in}^2$

$$\phi P_n = 0.8(0.65)[0.85(5000)(1064-22) + 60{,}000(22)]$$

$$\phi P_n = 2989 K > 2557.2 K \quad \underline{OK}$$

* Note: All columns were designed for lateral loads and gravity loads, and should be over capacity.

Interior Column D12

Level	Dead (psf)	Live (psf)	Red. Live (psf)	Total Axial Load (K)		
				Dead	L or L_r	1.2D+1.6L+.5L_r
Penthouse Roof	40	20	20	36	13.9998	50.200
Penthouse	86	150	150	113.4	135.000	359.080
Level 9	86	80	40	190.8	171.000	509.560
Level 8	86	80	34.142	268.200	201.728	651.605
Level 7	86	80	32	345.6	230.528	790.565
Level 6	86	80	32	423	259.328	929.525
Level 5	86	80	32	500.4	288.128	1068.485
Level 4	86	100	40	577.8	324.128	1218.965
Level 3	86	100	40	655.2	360.128	1369.445
Level 2	86	100	40	732.6	396.128	1519.925
Level 1	86	100	40	810	432.128	1670.405
Parking 1	105	40	40	904.5	468.128	1841.405
Parking 2	105	40	40	999	504.128	2012.405
Parking 3	105	40	40	1093.5	540.128	2183.405
Parking 4	105	40	40	1188	576.128	2354.405
					Axial + 1.2 Self Wt.	**2557.205**

Trib Area =	900	ft^2
K_{LL} =	4	
Roof Trib=	700	ft^2
Self-Weight=	169	K

Note: L_r excluded from total axial live load total and added as .5 L_r to third column.

Exterior Column C13.5

4 lower parking levels: 28 x 38 w/ 14 #8
level 1-2: 24 x 24 w/ 16 #9
levels 2-9: 24 x 24 w/ 8 #8
levels 9- pent. roof: 24 x 24 w/ 12 #9

$f'_c = 5000$ psi

Column Loads:

Trib Area varies by level

Pent roof: $15' \times 15' = 225\ ft^2$

Penthouse: $(15' \times 15') + \left(\frac{7.5' \times 15'}{2}\right) = 281.25\ ft^2$

Level 9-5: $281.25\ ft^2$

Level 4-1: $(15 \times 15) + (7.5 \times 15) = 337.5\ ft^2$

Parking 4: $(5' + 7.5') \times 30' = 375\ ft^2$

Parking 3-1: $375 - (15' \times 15') = 150\ ft^2$

Live Load Reduction:

$K_{LL} = 3$ due to cantilevered slabs

Can reduce LL for all levels $K_{LL} A_T = 3(150) > 400\ ft^2$

↳ Will not reduce roof, penthouse, parking

At level 9

$$L = 80 \times \left|_{max} \begin{matrix} 0.4 \\ 0.25 + \frac{15}{\sqrt{3(281.25)}} \end{matrix}\right. = 0.766(80) = 61.3\ psf$$

See excel for all values

Perimeter Dead Load:

Panel System = 16 psf

Perim. Levels PR-5: $15 + 7 + 6 + 7.5 = 35.5$ ft
4-1: 22.5 ft

panel nominal thickness

$$DL = 16\left(6(35.5) + 3(22.5)\right)\left(\frac{36''}{12''/ft}\right) = 13.46\ k$$

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

Self-Weight

$$\frac{150\text{ pcf}}{1000}\left(\frac{28\times38}{144}(47.3)+\frac{24\times24}{144}(49+117.8)\right)=152.5\text{K}$$

Controlling Case $= 1.2D + 1.6L + 0.5L_r$

$$P_u = 1.2(356.85+13.46+152.5)+1.6(198)+0.5(45)$$

$$=\boxed{946.4\text{ k}}$$

Column Strength Check

$$\phi P_n = 0.8\,\phi\left[0.85 f'_c(A_g - A_s) + f_y A_s\right]$$

$A_g = 28\times38 = 1064\text{ in}^2$

$A_s = 14\times0.79 = 11.06\text{ in}^2$

$$\phi P_n = 0.8(0.65)\left[0.85(5000)(1064-11.06)+(60)(11.06)\right]$$

$$\phi P_n = 2327\text{ K} > 946.4\text{ k} \quad \underline{\text{ok}}$$

Exterior Column C13.5

Level	Dead	Live	Reduced Live	Trib Area	Axial Load (K)		
					Dead - Area	L or L_r	$1.2D+1.6L+.5L_r$
Penthouse Roof	40	20	20	225	9	4.500	13.050
Penthouse	86	150	150	281.25	33.1875	42.188	109.575
Level 9	86	80	61.312	281.25	57.375	59.431	166.190
Level 8	86	80	49.212	281.25	81.5625	73.272	217.361
Level 7	86	80	43.851	281.25	105.75	85.605	266.119
Level 6	86	80	40.656	281.25	129.9375	97.040	313.439
Level 5	86	80	38.475	281.25	154.125	107.861	359.778
Level 4	86	100	44.245	337.5	183.15	122.794	418.500
Level 3	86	100	42.817	337.5	212.175	137.245	476.452
Level 2	86	100	41.667	337.5	241.2	151.307	533.782
Level 1	86	100	40.713	337.5	270.225	165.048	590.597
Parking 1	105	40	40	375	309.6	180.048	661.847
Parking 2	105	40	40	150	325.35	186.048	690.347
Parking 3	105	40	40	150	341.1	192.048	718.847
Parking 4	105	40	40	150	356.85	198.048	747.347
						Axial + 1.2(Perim + SW)	**747.347**

Perimeter DL=	13.46	K
K_{LL} =	3	
Self-Weight=	153	K

Note: L_r excluded from total axial live load total and added as .5 L_r to third column.

5 | Alternative Framing Systems for Gravity Loads

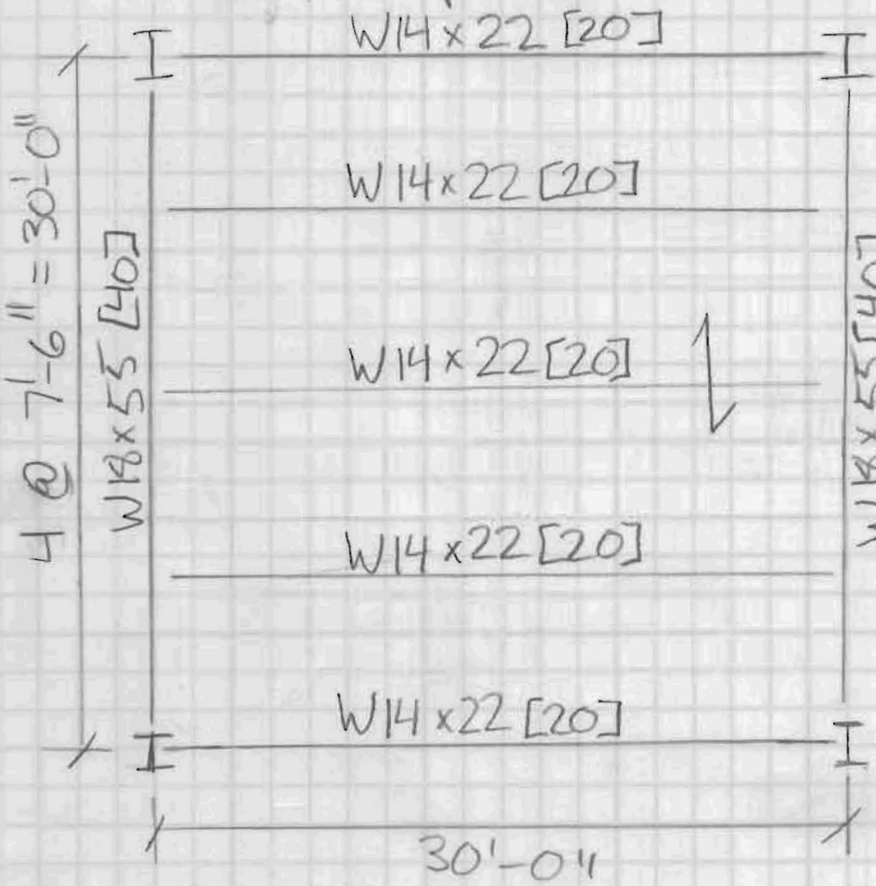
Three alternative gravity systems were explored for the existing building loads. The systems under consideration in this report are:

- Alternative 1: Composite Wide-Flange Steel
- Alternative 2: Two-way Flat Slab
- Alternative 3: Non-Composite Steel Joists

All gravity framing systems maintained the 30’-0” x 30’-0” bay size of the existing structure. Structural systems were evaluated based on strength and serviceability.

5.1 Alternative 1: Composite Wide-Flange Steel

Alternative 1: Composite Steel



<u>4" Deck</u>: 1.5VLI20 with 2½ NWC topping

1) Composite Decking

- 2 hr fire-rating req'd
- Super Imposed Dead:

Finishes + Ceiling	5 psf
MEP	15 psf
Misc	3 psf
	23 psf

- Live Load: 100 psf

W_{Total} = <u>123 psf</u>

$\boxed{\text{Try 1.5VLI20 deck w/ 2½ NWC topping}}$

- Max3 span unshored: 7'-11" > 7'-6" ∴ OK
- SDL = 217 psf < 123 psf ∴ OK
- Slab weight = 39 psf

2) Infill Beams

Dead: 23 + 39 + 5 (self-weight) = 67 psf
Live: 100 psf

- Unshored Strength

$1.4D = 1.4(67 \text{ psf}) = 93.8 \text{ psf}$

$1.2D + 1.6L = 1.2(67) + 1.6(20) = 112.4 \text{ psf}$

$W = 7.5\text{ft}(112.4 \text{ psf}) = 843 \text{ plf}$

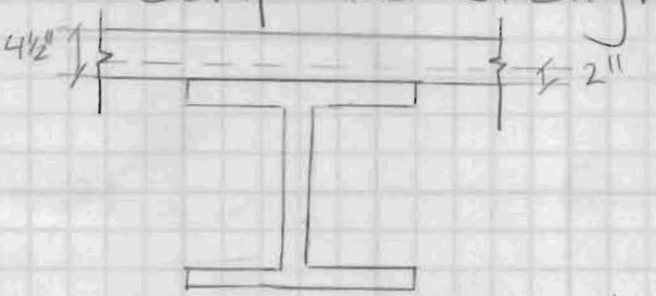
$$M = \frac{wl^2}{8} = \frac{(0.843)(30)^2}{8} = 94.8 \text{ k-ft}$$

W14x22: $\phi M_p = 125$ ft-k

W12x30: $\phi M_p = 162$ ft-k > 94.8 ft-k

W16x26: $\phi M_p = 166$ ft-k

- Composite Strength



Live Load Reduction

$K_{LL} = 2$

$A_T = (7.5)(30) = 225 \text{ ft}^2$

(eq 4-1) $L = 100 \times \left| \begin{matrix} 0.5 \\ 0.25 + \frac{15}{\sqrt{2(225)}} \end{matrix} \right|_{max} = 0.9571$

$L = 95.7$ psf

Load Combos

$1.4D = 93.8$ psf

$1.2D + 1.6L = 1.2(67 \text{ psf}) + 1.6(95.7 \text{ psf}) = \underline{233.5 \text{ psf}}$

$w = (233.5 \text{ psf})(7.5 \text{ ft}) = 1.751 \text{ klf}$

$$M = \frac{(1.751)(30)^2}{8} = \underline{197 \text{ k-ft}}$$

$$b_{eff} = \min \left| \begin{array}{l} 3.75\ ft \times 12\ in/ft \\ 30\ ft \times 12\ in/ft\ /\ 8 \end{array} \right. \times 2 = 90''$$

Assume $a = 1''$

$Y2 = 4'' - \frac{1}{2}'' = 3.5''$

From Table 3-19:

	Economy	
W14x22: $\Sigma Q_n = 157k$	$\frac{157\,k}{17.2} = 9.13$	$2(10 \times 10) + 22(30) = 860\ lbs$
W12x30: $\Sigma Q_n = 136\,k$	$\frac{136}{17.2} = 7.91$	$2(8 \times 10) + 30(30) = 1060\ lbs$
W14x26: $\Sigma Q_n = 96\ k$	$\frac{96}{17.2} = 5.58$	$2(6 \times 10) + 26(30) = 900\ lbs$

$\boxed{\therefore \text{ Use W14x22 with 20 studs/beam}}$

- Check a Assumption

$V_c = (3.5'')(90'')(4\ ksi)(0.85) = 1071\ k$

$V_s = (50\ ksi)(6.49\ in^2) = 324.5\ k$

20 studs $\times$ 17.2 k/stud = 344 k

$$a = \frac{344}{0.85(3.5)(90)}$$

$a = 1.28'' > 1''$ $\therefore$ a assumption wrong

- Assume $a = 2''$, $Y2 = 3''$ $\phi M_n = 200\ ft\text{-}k$

$200\ ft\text{-}k > 197\ ft\text{-}k$

$\Sigma Q_n = 157\ k$ still

a assumption <u>OK</u>

- Wet Conc Deflection

$W = (39\ psf)(7.5\ ft) = 292.5\ plf$

$$\Delta = \frac{5(0.2925)(30)^4(1728)}{384(29000)(199\ in^4)} = 0.924$$

- Live Load Deflection

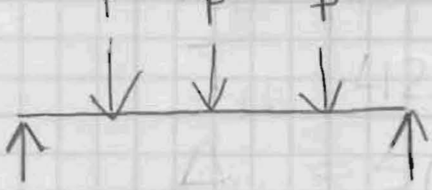
$W_{LL} = (95.7\ psf)(7.5\ ft) = 717.8\ plf$

$I_{LB} = 426\ in^4$

$$\Delta_{LL} = \frac{5(0.7178)(30)^4(1728)}{384(29000)(426)} = 1''$$

$$\frac{l}{360} = \frac{30 \times 12}{360} = 1'' = 1''\ \underline{OK}$$

3) Girders



$P_D = (23+39+7\,psf)(7.5\,ft)(30\,ft) = 15.5\,k$

$P_L = (60.4\,psf)(7.5\,ft)(30\,ft) = 13.6\,k$

LL Reduction

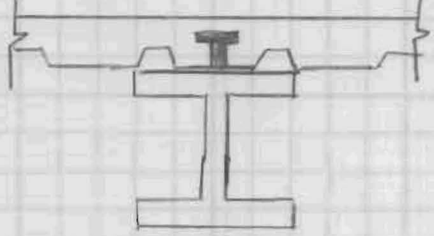
$L_{red} = 100 \times \begin{vmatrix} 0.5 \\ 0.25 + \frac{15}{\sqrt{2(30)^2}} = 0.604 \end{vmatrix} = 60.4\,psf$

Load Combos

$1.4P_D = 1.4(15.5\,K) = 21.7\,k$

$1.2P_D + 1.6P_L = 1.2(15.5) + 1.6(13.6\,k) = 40.4\,K$

$M_{max} = \frac{Pl}{4} + Pa = \frac{40.4(30)}{4} + 40.4(7.5) = 606\,ft\text{-}k$



If had assumed distributed load...

$w = 30(1.2(23+39+7) + 1.6(60.4))$

$w = 5.38\,klf$

$M = \frac{wl^2}{8} = \frac{(5.38)(30)^2}{8}$

$M = 605.3\,ft\text{-}k$

∴ Can assume a distrib. load for further calc.

Flexural Design

$b_{eff} = \min \begin{vmatrix} 15\,ft \times 12\,in/ft \\ \frac{30\,ft \times 12\,in/ft}{8} \end{vmatrix} \times 2 = \underline{90''}$

$Y1 = 0.285$ Assume $a = 1''$, $Y2 = 3.5''$

	Economy
W18x50 : $\Sigma Q_n = 521\,k$	$\frac{521}{17.2} = 30.3$ $2(31 \times 10) + 50(30) = 2120$ lbs
BFL $\boxed{W18x55}$: $\Sigma Q_n = 336\,k$	$\frac{336}{17.2} = 19.5$ $2(20)(10) + 55(30) = 2050$ lbs

- Unshored Strength

$1.4D = 1.4(69\text{ psf}) = 96.6\text{ psf}$

$1.2D + 1.6L = 1.2(69) + 1.6(20) = 114.8\text{ psf}$

$w = (114.8\text{ psf})(30\text{ ft}) - 3.44\text{ klf}$ & $L_b = 7.5\text{ ft}$ (Table 3-10 AISC)

$M = \dfrac{wl^2}{8} = \dfrac{(3.44\text{ klf})(30\text{ ft})^2}{8} = 387\text{ k-ft}$

W18x50 : $\phi M_n = 379\text{ k-ft} < 387\text{ k-ft}$ <u>NG</u>

$\boxed{\text{W18x35}}$: $\phi M_n = 420\text{ k-ft} > 387\text{ k-ft}$ <u>OK</u>

- Wet Concrete Deflection

$w = (39\text{ psf})(30\text{ ft}) = 1.17\text{ klf}$

$\Delta = \dfrac{5(1.17)(30)^4(1728)}{384(29000)(890\text{ in}^4)} = 0.826\text{ in}$

$\boxed{\therefore \text{ Camber } 0.5'' - 0.75''}$

- Live Load Deflection

$w_{LL} = (60.4\text{ psf})(30\text{ ft}) = 1.812\text{ klf}$

$I_{LB} = 2140\text{ in}^4$

$\Delta_{LL} = \dfrac{5(1.812)(30)^4(1728)}{384(2140\text{ in}^4)(29000\text{ ksi})} = 0.532\text{ in}$

- Check a assumption

$V_c = 1071\text{ k}$ (see previous)

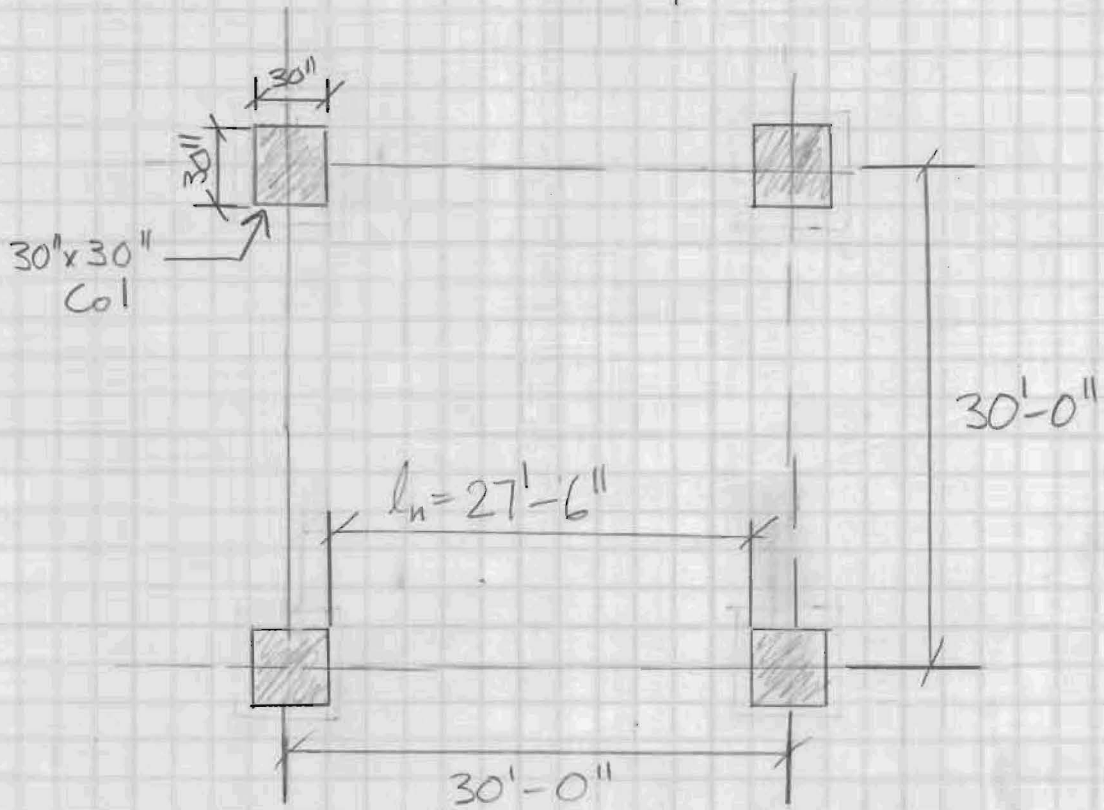
$V_s = 50\text{ ksi}(16.2\text{ in}^2) = 810\text{ k}$

15 studs x 17.2 k/stud = 258 k

$a = \dfrac{258}{0.85(3.5)(90)} = 0.9636'' < 1''$ <u>OK</u>

5.2 Alternative 2: Two-Way Flat Slab

Alternative 2: Two-Way Flat Slab



Deflections

Table 9.5(c) in ACI 318-11

- $f_y = 60{,}000$ psi

Design - $f_y = 60{,}000$	Interior t_{min}	Exterior t_{min}
W/O drop panels + w/ edge beams	$l_n/33 = \frac{27.5 ft}{33} = 10"$	$l_n/33 = 10"$
W/ drop panels + W/ edge beams	$l_n/36 = \frac{27.5 ft}{36} = 9.2$ in	$l_n/36 = \frac{27.5'}{36} = 9.2$ in

∴ Start w/ 10" thick slab w/o drop panels

$d_{(avg)} = 10" - 0.75" - 0.5" = 8.75"$

Assuming #4 bars

One-Way Shear:

$LL = 100\ psf$

$DL = 10''/12^{in}/ft \times 150\ pcf = 125\ psf$

$w = 1.2(125) + 1.6(100) = 310\ psf$

$V_u = 0.310\ ksf \times 22.5ft \times 30ft = 209\ K$

middle strip width, see moment design

$\phi V_c = 0.75\ 2\lambda\sqrt{f'_c}\ bd$

$= 0.75(2)(1)\sqrt{5000_{psi}}(30 \times \frac{12\ in}{ft})(8.75\ in)$

$\phi V_c = 334.1\ k > 209\ K$ OK

Two-Way Shear/Punching Shear:

-Critical location @ $d/2 = 8.75''/2 = 4.38''$

$V_u = (0.310\ ksf)\left[30' \times 30' - \left(\frac{30'' + 2(4.38'')}{12^{in}/ft}\right)^2\right]$

$V_u\ 275K$

Critical perim. $b_o = 4(34.38'') = 137.5''$

$\phi V_c\ \phi 4\lambda\sqrt{f'_c}\ b_o d = 4(1)\sqrt{5000}(137.5'')(8.75'')(0.75) = \boxed{255K}$

* ACI eq. 11-31 for highly rect. column not applicable w/ square columns

$\frac{b_o}{d} = \frac{137.5''}{4.38''} = 31.4$ ∴ ACI eq 11-32 applies

Interior Col: $\phi V_c = \phi\left(\frac{\alpha_s d}{b_o} + 2\right)\lambda\sqrt{f'_c}\ b_o d$

$= 0.75\left(\frac{40 \cdot 8.75}{137.5} + 2\right)(1)\sqrt{5000}(137.5)(8.75)$

$= 290\ K$

Corner Col: $\phi V_c = 0.75\left(\frac{20 \cdot 8.75}{137.5} + 2\right)(1)\sqrt{5000}(137.5)(8.75)$

$= \boxed{209K}$

Edge Col: $\phi V_c = 0.75\left(\frac{30 \cdot 8.75}{137.5} + 2\right)(1)\sqrt{5000}(137.5)(8.75)$

$\phi V_c = \boxed{249\text{ K}}$

Eqn 11-32 ϕV_c values control $< V_u = 275\text{ K}$

$\therefore$ Use drop panels and $t = 9.5''$

One-Way Shear w/ $t = 9.5''$

$d_{avg} = 9.5'' - 0.75'' - 0.5'' = 8.25''$

$LL = 100$ psf

$DL = 9.5/12 \times 150 = 119$ psf

$w = 1.2(119) + 1.6(100) = 303$ psf

$V_u = 0.303\text{ ksf} \times 22.5' \times 30' = 204.5\text{ k}$

$\phi V_c = 0.75(2)(1)\sqrt{5000}\,(30 \times 12\text{ in/ft})(8.25)$

$= 315\text{ k} > 204.5\text{ k}$ OK

Two-Way Shear / Punching Shear

$d/2 = 8.25/2 = 4.125''$, $b_o = 4(34.125) = 136.5''$

$V_u = (0.303)\left[30^2 - \left(\frac{30 + 2(4.125)}{12}\right)^2\right] = 270\text{ k}$

$\phi V_c = \phi\left(\frac{\alpha_s d}{b_o} + 2\right)\lambda\sqrt{f'_c}\, b_o d$

Interior: $\left[\left(\frac{40 \cdot 8.25''}{136.5''} + 2\right)(1)\sqrt{5000}(136.5)(8.25)\right] \times 0.75$

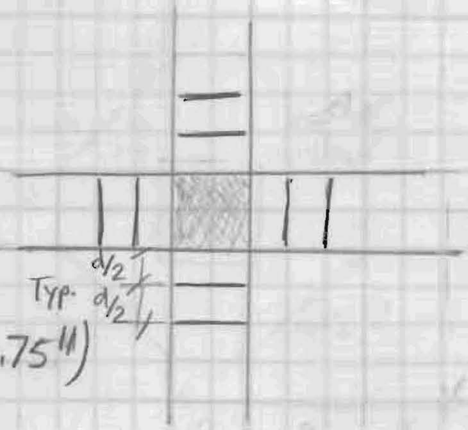
$\phi V_c = 264\text{ K}$

Edge: $\alpha_s = 30$, $\phi V_c = 228\text{ K}$

Corner: $\alpha_s = 20$, $\phi V_c = 191\text{ K}$

∴ Use Shear Reinforcement

Stirrup Cage Design



$V_n = V_c + V_s$

$V_c = 2\lambda\sqrt{f'_c}\, b_o d$

$V_c = 2(1)\sqrt{5000}\,(137.5'')(8.75'')$

$V_c = 170k$

$V_u = \phi(V_c + V_s)$

<u>Stirrup Req'd:</u>

$275k = 0.75(170 + V_s)$

$V_{s,req} = 197\ k$

$V_s = A_v f_{yt} \dfrac{d}{s}$

$d = 8.75''$ $\quad f_{yt} = 60$ ksi

$s = \dfrac{d}{2} = 4.38''$

$197 = A_v(60)\dfrac{8.75}{4.38}$

$A_v = 1.64\ in^2$

Try #4 bars

$\dfrac{1.64\ in^2}{0.5\ in^2} = 4$ bars on a shear plane

<u>Check Max Shear</u>

$\phi V_n = \phi 6\lambda\sqrt{f'_c}\, b_o d = 0.75(6)\sqrt{5000}\,(137.5)(8.75'')$

$= 383\ K$

$V_s = (0.5\ in^2)(4)(60)\left(\frac{8.75}{4.38}\right) = 240K$

$\phi V_n = 0.75(170 + 240) = 307\ K < 383K$ <u>OK</u>

Slab Moments

Square Bays = same in both dir.

Can use direct design? (13.6.1 ACI)

- Min of 3 cont. spans ✓
- longer : shorter = 1:1 < 2 ✓
- Successive span lengths ok ✓
- Max col. offset = 10% span
- Will only consider gravity loads
- LL vs. DL

$LL = 100$ psf

$DL = 10/12 \times 150 = 125$ psf

$100 \text{ psf} \not> 2(125 \text{ psf})$ OK

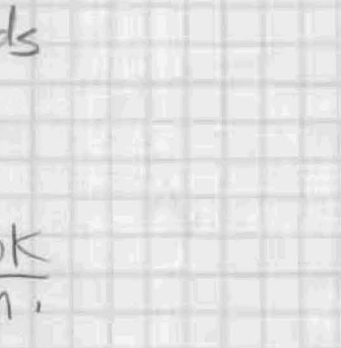
$\therefore$ Yes, can use direct design.

$$M_o = \frac{w_u \ell_2 \ell_n^2}{8} = \frac{(0.310 \text{ ksf})(30 \text{ ft})(27.5 \text{ ft})^2}{8} = \underline{879 \text{ ft-k}}$$

$\ell_1 = \ell_2 = 30$ ft + $\ell_n = 27.5$ ft (see pg 1 layout)

Col strip Dimension: $\frac{30 \text{ ft}}{4} = 7.5'$

Interior $\alpha_f = 0$ (no beams)



Exterior Span w/ Edge Beam

ACI 13.6.3.3 Slab w/o Interior Beams but w/ Edge Beams

Interior $-M_u = -0.70 M_o = -0.7(879) = -615$ ft-k

$+M_u = 0.50 M_o = 0.50(879) = 440$ ft-k

Exterior $-M_u = -0.30 M_o = -0.3(879) = -264$ ft-k

Interior Neg. Moments (ACI 13.6.4.1)

$\Rightarrow$ Col Strip: $0.75(-615) = -461$ ft-k

$-461 \text{ ft-k} / 7.5 \text{ ft} = 61.5$ ft-k/ft of mid. strip

Middle Strip: $(-615 \text{ ft-k})(0.25) = -154$ ft-k

Positive Moments - ACI 13.6.4.4

$\Rightarrow$ Col Strip: $0.60(440 \text{ ft-k}) = 264 \text{ ft-k}$

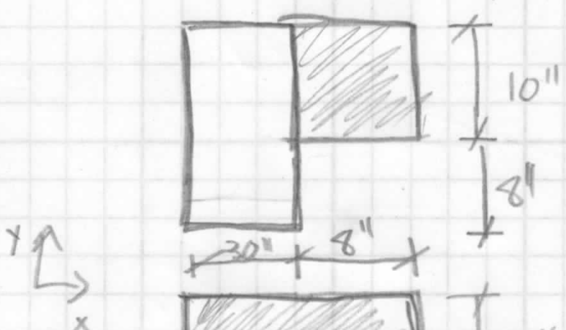
Middle Strip: $0.40(440 \text{ ft-k}) = 176 \text{ ft-k}$

Exterior Neg. Moment - ACI 13.6.4.2

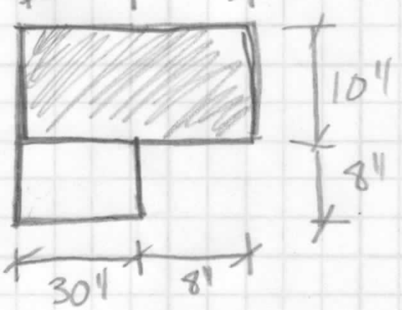
$$\beta_t = \frac{\cancel{E_{cb}}\, C}{2\cancel{E_{cs}}\, I_s}$$

$E_{cb} = E_{cs}$

$$C = \Sigma\left(1 - 0.63\frac{x}{y}\right)\frac{x^3 y}{3}$$



$$C = \left[\left(1 - 0.63\left(\frac{18}{30}\right)\right)\frac{(18)^3(30)}{3}\right] + \left[\left(1 - 0.63\left(\frac{8}{10}\right)\right)\frac{(8)^3(10)}{3}\right] = \underline{37122 \text{ in}^4}$$



$$C = \left[\left(1 - 0.63\left(\frac{10}{38}\right)\right)\frac{(10)^3(38)}{3}\right] + \left[\left(1 - 0.63\left(\frac{8}{30}\right)\right)\frac{(8)^3(30)}{3}\right]$$

$C = 14827 \text{ in}^4$

* 18" trial beam depth

$$I_s = \frac{(30 \text{ ft} \times 12 \text{ in/ft})(10 \text{ in})^3}{12} = 30{,}000 \text{ in}^4$$

$$\beta_t = \frac{37122}{2(30000)} = 0.62$$

Interpolating between $\beta_t = 0$ and $\beta_t = 2.5$ in table,

93.8% of Ext. Neg. M_u to Column Strip

$\Rightarrow$ Col strip $= 0.938(-264) = -247.6 \text{ ft-k}$

Middle Strip $= -16.4 \text{ ft-k}$

Interior Span

$-M_u = 0.65 M_o = 0.65(879) = -571$ ft-k
$+M_u = 0.35 M_o = 0.35(879) = 308$ ft-k

Negative Moments (ACI 13.6.4.1)

Col Strip: $0.75(-571$ ft-k$) = \underline{-428 \text{ ft-k}}$
Middle Strip: $0.25(-571$ ft-k$) = \underline{-143 \text{ ft-k}}$

Positive Moments (ACI 13.6.4.4)

Col Strip: $0.60(308$ ft-k$) = 185$ ft-k
Middle Strip: $0.40(308$ ft-k$) = 123$ ft-k

Typical Interior Bay Flexural Reinf:

-428 ft-k 185 ft-k -428 ft-k Column Strip

-143 ft-k 123 ft-k -143 ft-k Middle Strip

-428 ft-k 185 ft-k -428 ft-k Column Strip

Approximate d:
long bars: $d \approx h - 1.1 = 10 - 1.1 = 8.9$ in
short bars: $d \approx h - 1.7 = 10 - 1.7 = \underline{8.3 \text{ in}}$ use

$A_{s,min} = 0.0018 bh$
$= 0.0018(12'')(10'') = 0.216$ in^2/ft

$$A_{s,req} = \frac{M_u (\times 12^{in}/_{ft})}{\phi f'_y jd} \text{ where } \phi = 0.9, \quad f'_y = 60 \text{ ksi}, \quad jd = 0.95d = 0.95(8.3\text{in}) = 7.89 \text{ in}$$

Location	M_u (ft-k)	$A_{s,req}$ (in²)	Use...	Spacing	$A_{s,actual}$ (in²)	$A_{s\,min}$/ft	ε_s
-col	-428	12.05	16 #8	12" o.c.	12.64	12.64/12 = 1.05	
+col	185	5.21	12 #6	12" o.c.	5.28	0.44	
-mid	-143	4.03	13 #5	12"	4.03	0.336	
+mid	123	3.46	12 #5	12"	3.72	0.31	
						ok	

Check that steel is in tension...

$$\varepsilon_s = \frac{d-c}{c}\varepsilon_{cu}$$

$$A_s f'_y = 0.85 f'_c ba$$

$$\varepsilon_s = \frac{d-c}{c}\varepsilon_{cu}, \text{ where}$$

$$a = \frac{(12.64)(60)}{0.85(5)(15\times 12)} = 0.991$$

$$\varepsilon_s = \frac{7.89 - 1.24}{1.24}(0.003)$$

$$c = \frac{a}{0.80} = \frac{0.991}{0.80} = 1.24$$

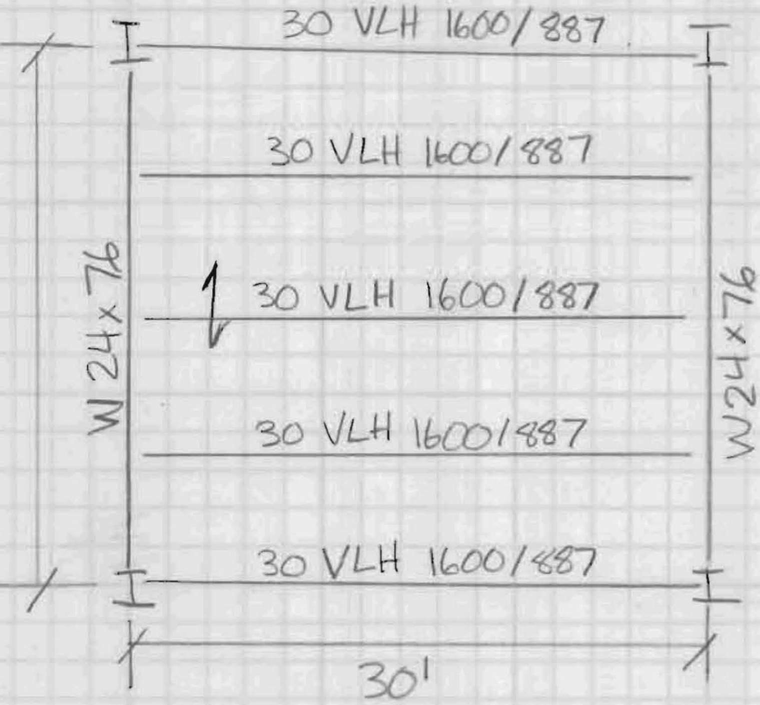
$$\varepsilon_s = 0.0161 > 0.00208$$

$\beta_1 = 0.8$ for 5000 psi conc

OK

5.3 Alternative 3: Non-Composite Steel Joists

Alternative 3: Non-composite Steel Joists



Deck: 5" deck
2C18 with
3" LW topping

1) Non-Composite Steel Deck
- LW conc to decrease weight of deck on joists
- 2 hr fire-rating required
- SDL = 23 psf
  LL = 80 psf
  W_{total} = 103 psf

Try 2C18 with 3" LW conc. topping

Fire-Rating Check – Will need Cementitious or Sprayed Fiber

Max Construction Span:

for LW conc, 3-span = 11'-11"
∴ Try 10' or 7'-6" joist spacing

Allowable Uniform Load:

10'-0" clear span: 123 psf
7'-6" clear span: 215 psf > 103 psf OK

* for 3-span construction

W_{TL} = L/180 for Roof

- 20 + 23 = 43 psf for design
  10'-0" clear span: 92 psf
  7'-6" clear span: 218 psf > 43 psf OK

W_{LL} = L/240 for floor live:

10'-0" clear span: 69 psf ≤ 80 psf
7'-6" clear span: 164 psf > 80 psf
∴ Can't use 10'-0" clear span. Decrease gage of deck for 7'-6" span...

Try 2C22 w/ 3" LW conc. topping

Max Construction Span: 8'-11" > 7'-6" OK
Uniform Load: 113 psf > 103 psf OK
W_{TL} = L/180 : 126 psf > 43 psf OK
W_{LL} = L/240 : 95 psf > 80 psf OK
↳ Some areas have 100 psf LL. Use 2C20 with W_{LL} of 119 psf

∴ Use 2C20 deck with 3" LW conc. topping

2) Steel Joists

Live: 80 psf
Dead: 23+39 psf = 62 psf
$W_u = 1.2(62 \text{ psf}) + 1.6(80 \text{ psf}) = 202 \text{ psf}$
$W = D + L = 62 + 80 = 142 \text{ psf}$

Try VLH Series Non-Composite Floor Joists

Span = 30 ft
$W_u = (202 \text{ psf})(7.5 \text{ ft}) = 1515 \text{ plf}$
$W_L = (80 \text{ psf})(7.5 \text{ ft}) = 600 \text{ plf}$ (for LL Δ)

Try 30 VLH 1600 / 887 with 18" depth
- 5" deep bearings
- 1 bridging

Checks:
W_{TL}: 1600 > 1515 + 1.2(38) = 1561 plf ✓
Δ_{LL}: 887 > 600 plf ✓

Use 18" floor joists: 30 VLH 1600/887 from Vulcraft Composite + Noncomposite Floor Joists Catalog checks.

3) Wide-Flange Girders – Non-Composite

Live: 80 psf

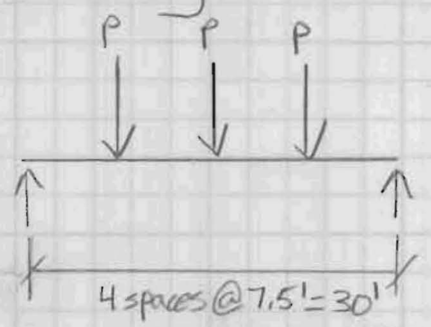
Dead: $62\ psf + \left(\frac{38\ plf}{30\ ft}\right) = 63.3\ psf$

Live Load Reduction

$$L = L_o \left|\begin{array}{l} 0.4 \\ 0.25 + \frac{15}{\sqrt{K_{LL}A_T}} \end{array}\right._{max} = 0.25 + \frac{15}{\sqrt{2(30\times 50)}} = 0.6$$

$$= 80 \times 0.6 = 40\ psf$$

Loading:



$P_{u,TL} = (1561\ plf)(30\ ft) = 46.8\ K$

$P_{u,LL} = (600\ plf)(30\ ft) = 18\ K$

$$M_{max} = \frac{Pl}{4} + Pa = \frac{46.8(30)}{4} + 46.8(7.5) = 702\ ft\text{-}k$$

(superposition from AISC tables)

W24x76: $\phi M_p = 750\ ft\text{-}k > 702\ ft\text{-}k$

LL Deflection

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

$$= \frac{(18\ K)(30' \times 12)^3}{48(29000)(2100\ in^4)} + \frac{(18)(7.5)(12\ in/ft)}{24(29000)(2100)}\left(3(30\times 12)^2 - 4(7.5\times 12)^2\right)$$

$$= 0.286" + 0.395" = 0.681" < 1"\ \text{OK}$$

$$\frac{l}{360} = \frac{30\times 12}{360} = 1"$$

Camber Check

Weight of wet conc: $\frac{39 \times 7.5' \times 30'}{1000} = 8.78k$

$$\Delta_{DL(wet\ conc)} = \frac{(8.78)(30\times12)^3}{48(29000)(2100)} + \frac{(8.78)(7.5)(12)}{24(29000)(2100)}\left(3(30\times12)^2 - 4(7.5\times12)^2\right)$$

$$= 0.14 + 0.19 = 0.33''$$

Don't camber.....

Use W24 x 76 for girders

6 | System Comparisons

	Existing: One-Way Slab	Composite Steel	Two-Way Slab	Non-Composite Joists
Architectural Coordination				
Depth	25"	22"	10"	29"
Fire Rating	> 2 hr	2 hr	> 2 hr	2 hr
Fire Protection Type	None	Cementitious/Sprayed	None	Cementitious/Sprayed
Construction Statistics				
Cost	$21.45 / SF	$28.41 / SF	$16.70 / SF	$23.90 / SF
Durability	High	Acceptable	High	Acceptable
Structural Considerations				
Weight	175.1 psf	48.3 psf	125 psf	50.4 psf
Servicability	N/A	Vibrations	N/A	Vibrations
Lateral Systems				
Concrete Shear Wall	Yes	Yes	Yes	No
Steel Moment Frame	No	Yes	No	Yes
Steel Braced Frame	No	Yes	No	Yes
Moving Forward?	N/A	YES	YES	NO

Further consideration will be given to the composite steel and two-way slab gravity systems for a more in-depth investigation. Both systems offer the fire-rating and durability required for a healthcare facility. The two-way slab system is a cheaper and thinner gravity system, but the composite steel system is much lighter. Additional investigation will also be required to determine if the vibration requirements of hospital equipment are better suited to concrete construction. Non-composite joists were not as light-weight as expected, and are likely to have vibration issues. Therefore, this system has been ruled out for future consideration.

1) Weight per bay

Existing - One Way Slab w/ Intermediate Beams

- Slab: $(150 pcf)(5/12)(30)(30) = 56.3k$
- 9x25 bms $2(150 pcf)\left(\frac{9\times20}{144}\right)(30') = 11.25k$
- 36x25 bms $4(150 pcf)\left(\frac{36\times20}{144}\right)(30') = 90.00k$

$157.6k$

- $\frac{157.6k \times 1000}{30' \times 30'} = \boxed{175.1 psf}$

Alternative 1 - Composite Steel

- Deck: $(39 psf)(30')(30') = 35.1k$
- Beams: $5(22 plf)(30') = 3.3k$
- Girders: $2(55 plf)(30') = 3.3k$
- Studs: $5(20\times10) + 2(40\times10) = 1.8k$

$43.5k$

- $\frac{43.5k \times 1000}{30' \times 30'} = \boxed{48.3 psf}$

Alternative 2 - Flat Plate Two-Way Slab

- Slab: $(150 pcf)(10/12)(30)(30) = 112.5k$
- $\frac{112.5k \times 1000}{30' \times 30'} = \boxed{125 psf}$

Alternative 3 - Non-Composite Steel Joists

- Deck: $(39 psf)(30')(30') = 35.1k$
- Joists: $5(38 plf)(30') = 5.7k$
- Girders: $2(76 plf)(30') = 4.56k$

$45.36k$

- $\frac{45.36 \times 1000}{30' \times 30'} = \boxed{50.4 psf}$

2) Cost per bay

— Existing - One Way Slab w/ Intermediate Beams

⇒ RSMeans B10B10 (2015 RSMeans Assemblies)

⇒ Location Factor:

Values for table: - B1010 219 7300 Solid Conc one way slab monolithically cast w/ reinf. concrete support, beams + girders

- Bay Size 30 x 30
- SDL ≈ 125 psf
- Min col size 20"

∴ Use Assembly 7300

Total Base Cost/ SF = 21.45 $/SF

- Composite Steel Slab

Use B1010 254 2300 - composite gage steel deck

- Bay Size 30x30
- SDL ≈ 125 psf (100 psf LL)
- Depth: 4" + 18" = 22"

Total Base Cost/ SF = $28.40/SF

- Two-Way Flat Slab

Use B1010 223 7600 - Flat Plate 10" thick w/ 125 SDL

Total Base Cost/ SF = $16.70 /SF

- Non-Composite Steel Joists

6900

Use B1010 250 7000 - Steel joists, beams, + slab on column w/ column add

Total Base Cost /SF = 22.10 + 1.80 = $23.9/SF

7 | Appendix A

Design building loads from the load key plan on structural documentation are listed in Table 5. The table includes superimposed dead loads, live loads, and concentrated live loads. Superimposed dead loads do not account for the total dead load of the structure.

LOAD KEY TABLE SUPERIMPOSED DEAD AND LIVE LOAD				SENSITIVE EQUIPMENT VIBRATION CRITERIA
OCCUPANCY	SUPER IMPOSED DEAD LOAD (PSF)	UNIFORM LIVE LOAD (PSF)	CONCENTRATED LIVE LOAD (LB)	
AHU 1 = AIR HANDLING UNIT	15CMEP + 60TOPPING = 75	150 NR	15,000 LBS	-
CEPR1 = CENTRAL ENERGY PLANT ROOF	80CMEP	40 NR	3,000	-
CEPR2 = CENTRAL ENERGY PLANT ROOF	80CMEP + 80RAMP = 160	40 NR	3,000	-
CT = COOLING TOWER	50	150 NR	52,800 LBS (EACH CELL) THREE CELL LOCATIONS	-
DROP = DROP OFF AREA	300	100 NR	2,000	-
EXT1 = EXTERIOR PLAZA ALONG CLIFTON RD	230	100 NR	2,000	-
EXT2 = EXTERIOR SOIL ALONG GRID 13.5	1,180	200 NR	2,000	-
HOS1 = TYPICAL HOSPITAL AREAS	15	100 RED	2,000	8,000 MIPS @ 85 PPM
HOS2 = HOSPITAL DIAGNOSTICS AND IMAGING	15CMEP + 60TOP = 75	350 NR	106 KIPS OR EQUIP. WGT	1000 MIPS @ 100 PPM
HOS 3 = HOSPITAL DIAGNOSTICS AND SURGICAL SUITES	15	100 RED	2,000 OR EQUIP. WGT	4,000 MIPS @ 85 PPM
KIT = KITCHEN	15CMEP + 40TOP + 40CMU = 95	150 NR	2,000	-
LD = LOADING DOCK	15	250 NR	-	-
MEC = MECHANICAL/ELECTRICAL ROOMS	15CMEP + 60TOPPING = 75	150 NR	2,000 OR EQUIP. WGT	60 TOPPING NOT APPLIED AT LEVEL 3 PNEUMATIC TUBE ROOM
MRI ACC	TYPICAL SDL FLOOR LOADS PER FINAL USE	150 NR	-	-
MU1 = MIXED USE 1	15CMEP + 40CMU = 55	100 RED	2,000	-
MU2 = MIXED USE 2	15CMEP + 5 FIN = 20	100 RED	2,000	-
MU3 = MIXED USE 3	15CMEP + 40 TOP + 40CMU = 95	100 RED	2,000	-
PAT = TYPICAL PATIENT ROOMS	15	80 RED (+)	1,000	-
PK1 = TYPICAL PARKING	5	40 NR	3,000	-
PK2 = PARKING WITH CURB ALLOWANCE	5CMEP + 40TOP = 45	40 NR	3,000	-
PUB1 = PUBLIC AREAS, LOBBIES, AND CORRIDORS	15	100 NR	2,000	-
PUB2 = PUBLIC AREAS, LOBBIES, AND CORRIDORS w/ THICK SET TILE/TOPPING	15CMEP + 25TILE = 40	100 NR	2,000	-
RF1 = ROOF WITH INSULATED CONCRETE TOPPING	15CMEP + 25TOP + 10ROOF = 50	20 NR	-	-
RF2 = GREEN ROOF/OUTDOOR PUBLIC AREA	15CMEP + 25TOP + 10ROOF + 50GREEN ROOF/PAVERS = 100	100 NR	-	-
RF3 = TYPICAL ROOF	25	20 NR	-	-
RFPH = PENTHOUSE ROOF	25CMEP + 25 ROOF = 50	20 RED	-	-
RR = RESTROOM	15CMEP + 25FIN = 40	100 RED	-	-
STA = METAL STAIR	60	100 NR	-	-
STO = LIGHT STORAGE	15	125 NR	2,000	-
TVROOF = TRANSFORMER VAULT ROOF	-	-	-	-

Table 5 | Load Key Plan Values

Figures 9-11 show different floorplate shapes typical for The Health Centre.

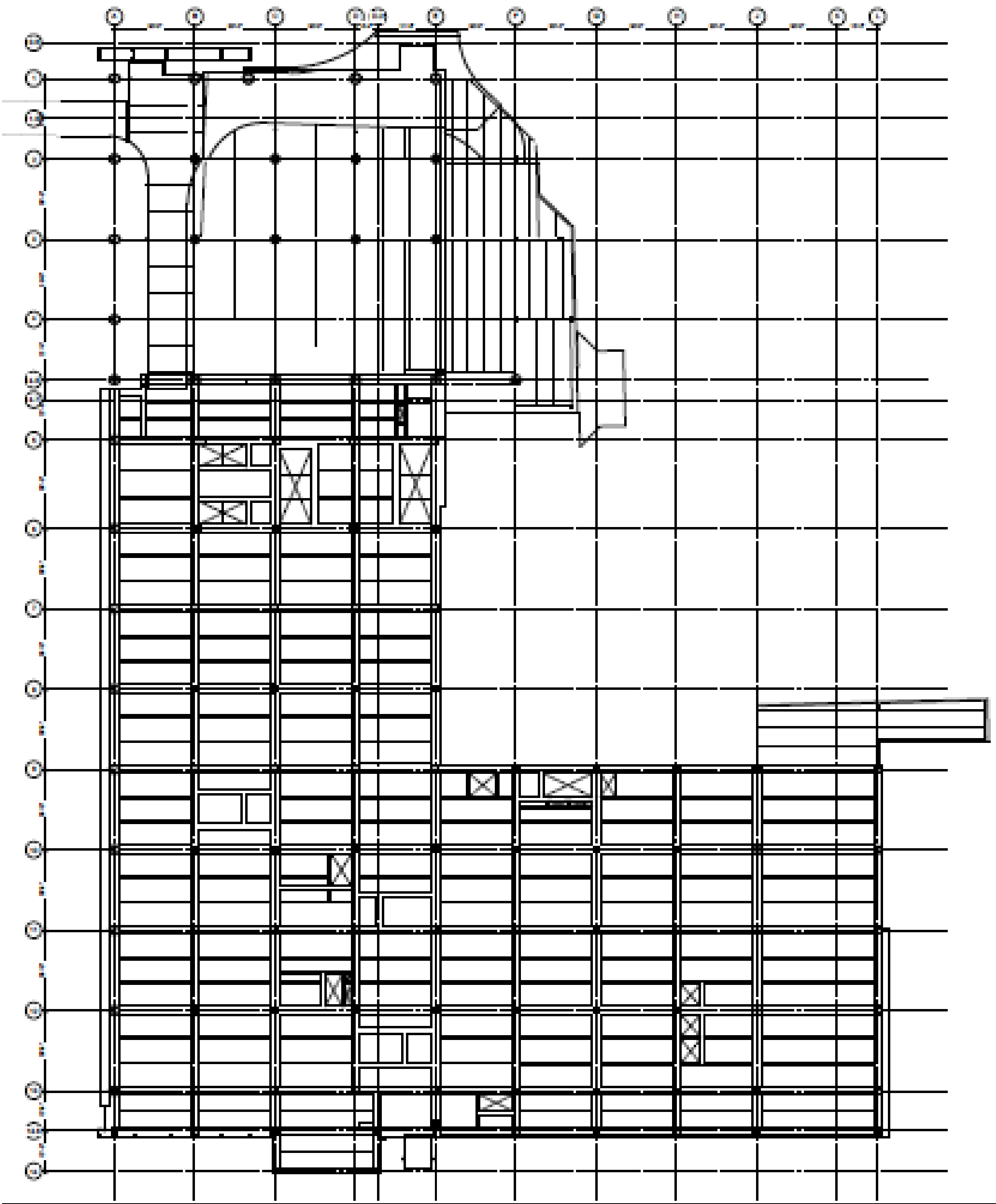


Figure 9 | Typical Structural Floor Plan for Floors 1-3 (Walter P. Moore)

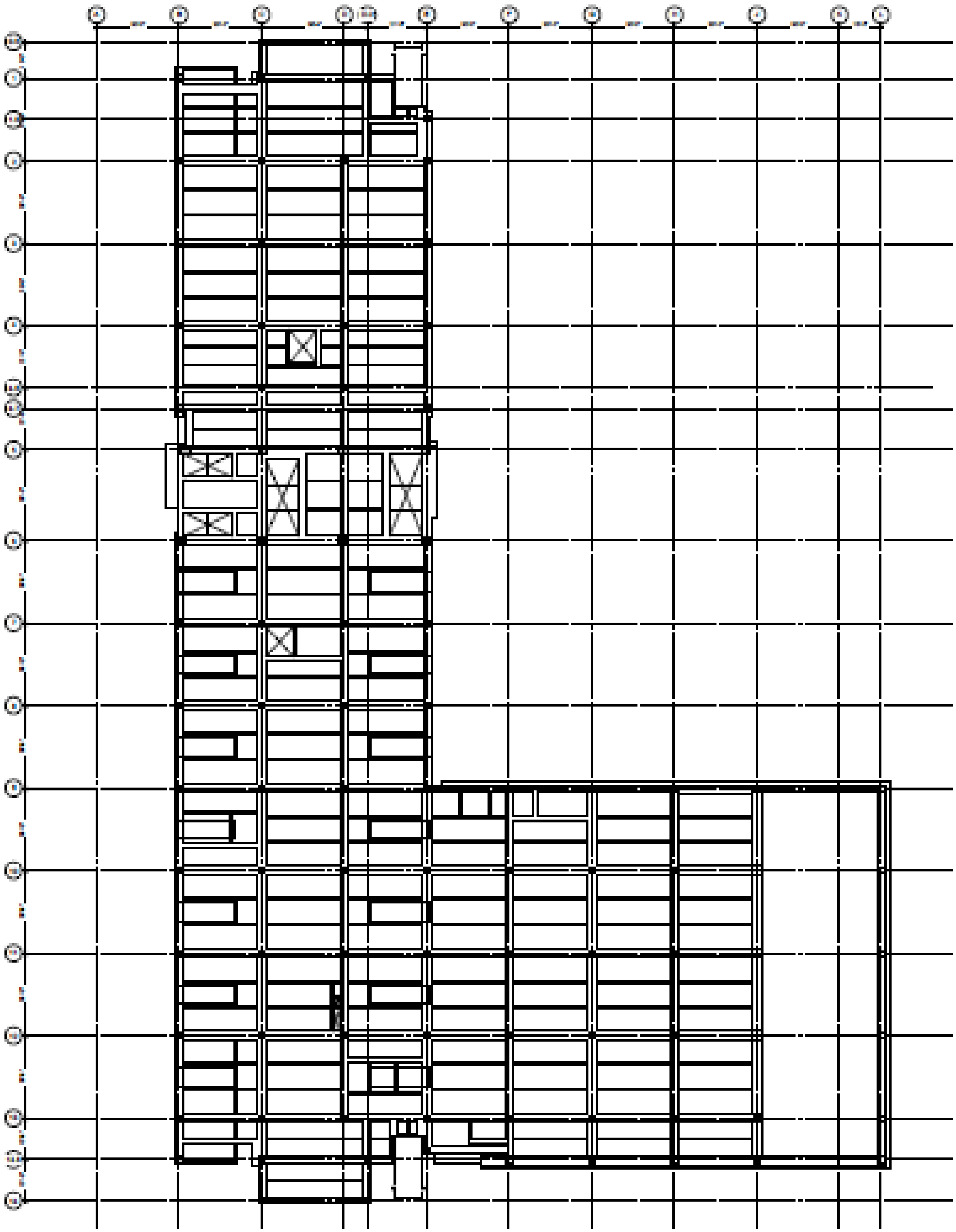


Figure 10 | Typical Structural Floor Plan for Floors 5-6 (Walter P. Moore)

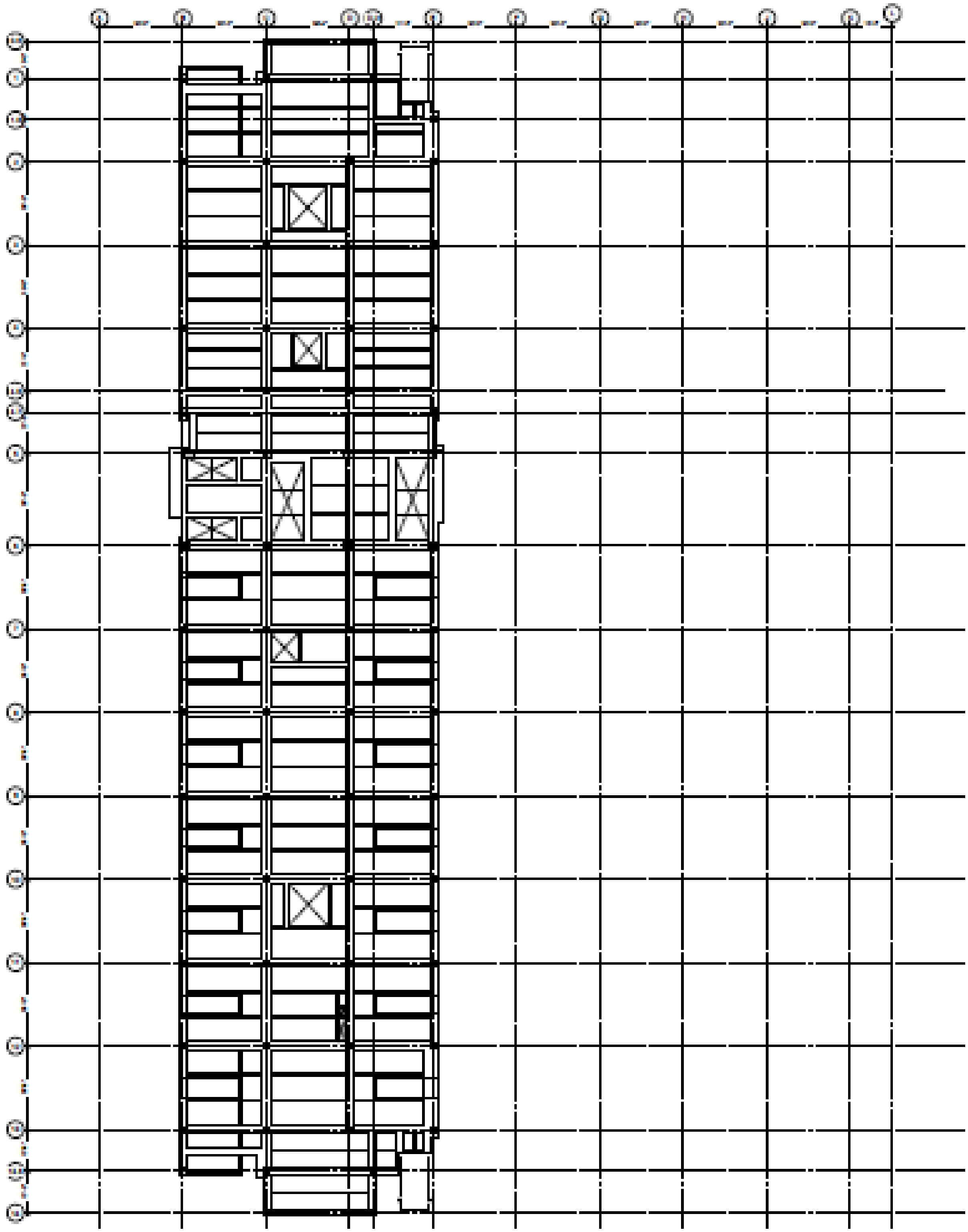


Figure 11 | Typical Structural Floor Plan for Floors 7-9 (Walter P. Moore)