Letter of Transmittal

October 17, 2014

Ali Said
Structural Thesis Advisor
The Pennsylvania State University
aus59@psu.edu

Dear Doctor Said,

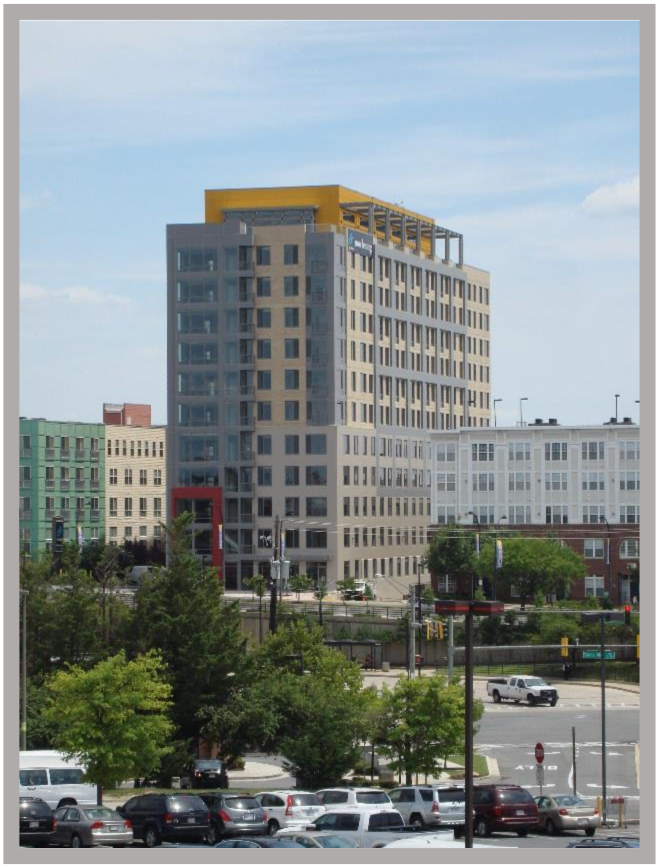
The following technical report fulfills the third Technical Report assigned by the structural faculty for senior thesis. Technical Report 3 includes the following in relation to 11141 Georgia Ave in Wheaton, MD:

- Gravity load calculations from tech report 2
- Codes and documents used to compile this report
- Typical bay member spot checks for gravity loads
- Alternative Framing Types:
    - Alternate #1: Non-Composite wide-flange steel
    - Alternate #2: Two-way slab with drop panels and perimeter beams
    - Alternate #3: One-way slab with girders
- System Comparisons

Thank you for your time in reviewing this report. I look forward to hearing your feedback and discussing it with you.

Sincerely,
Samantha deVries

Enclosed: Technical Report 3



11141 Georgia Avenue

Located in Wheaton, MD

Technical Report 3

Samantha deVries

Structural Option

Advisor: Ali Said

October 17, 2014

Table of Contents

Executive Summary

11141 Georgia Avenue, located in Wheaton, MD, is a 1960's concrete office building on which a 7-story steel addition was completed in August 2014 for $20 million. The building is a high rise apartment building with one and two bedroom studios, a rooftop terrace and penthouse, and is conveniently located next to the metro station.

The Foundations are spread footings with piers and a foundation retaining wall where the building steps from the lowest basement level to the next. Modifications were required to the foundations and slab on grade only where a new elevator pit was added and the old pit was removed.

The structure of the original building is reinforced concrete with typical two-way concrete slab bays that are approximately 22' by 21'. Again, the slabs in the original building only required modifications where new stairwells and elevators were added and the original ones were removed. The addition's structure is framed in structural steel with rolled W-shapes for the columns, girders, and beams, and composites joists for the bays in the floors and on the roof. Each floor has metal deck with a concrete topping.

The lateral system consists of concrete moment frames in the original structure, and steel moment frames in the new structure. Some columns were expanded for additional stiffness to resist an increase in lateral loads due to an increased building height.

There are many joints and connections that involved tying the new columns, beams, and other structural elements into the original building through drilling a hole to embed and grout rebar, anchors, or other connections.

The loads used in the structural design on the project all followed IBC 2009, which allows the use of ASCE 7-05. Due to a change in building use which allows a smaller reduced live load, the removal of the original penthouse, and the use of steel rather than concrete for the addition, the total loads reaching the foundations were close to the original 1960's design loads.

Purpose

The Purpose of this report is to identify and quantify the structural design loads used in the design of the building 11141 Georgia Avenue located in Wheaton, MD.

The report will identify all building codes, specifications, and other relevant documents used in the design loads of the building. A code analysis was completed using thesis documents to provide a site-specific and building-specific determination of the design loads to be used in the design of the building. Gravity, wind, and seismic loads will be determined and summarized in this report. Because the loads determined will be used for further evaluation of the existing design, codes used for the original design have been used. Redesigns in the spring semester may include an update to a more current code.

11141 Georgia Avenue: High Rise Residential Apartments Located in: Wheaton, MD

Building Statistics

Full Height: 158 Feet
Number of Stories: 14
Size: 179,760 GSF Square Feet
Cost: $44 Million (for the addition)
Construction Dates: February 2013 - August 2014
Project Delivery Method: Contractor at Risk

Project Team

Owner: ML Wheaton, LLC c/o Lower Enterprises
General Contractor and CM: Whiting-Turner
Architect: Bonstra Haresign Architects, LLP
Structure: Rathgeber/Goss Associates
Mechanical: Brothers Ductwork HVAC, Inc.
Plumbing: KNI Engineering, Inc.
Lighting Design: Gilmore Lighting Design

Photo of building from nearby parking garage roof: Photo taken by Samantha deVries

Structural Systems

- Original Concrete Building
  - Concrete moment frames
  - Concrete floor slabs
  - Spread footings and retaining walls
- New Addition
  - Steel moment frames
  - Lightweight composite floor joists with deck
- Loads
  - Original loads for office building
  - New live loads smaller for residential
- Renovation Work
  - New stairwell and elevator locations
  - New utility openings
  - Façade modifications

Photo of typical apartment: Photo courtesy of The George (Apartment)

Architecture

- 5 story 1960's office building
- 7 story addition
- High rise apartment building with one and two bedroom studios.

Construction

- Underpin Foundations
- Renovations work in existing building
- Construct addition directly above existing

Mechanical

- Cooling by rooftop chiller condensing units
- Units have occupant operable windows
- Heating by electrical heaters and heat pumps.

Electrical/Lighting

- Recessed lighting in apartments
- Pendant and wall mounted fixtures in lobbies
- 2 Main Power Distributers fed from a transformer
- One 1400 KVA and one 1750 KVA

Photo of rooftop terrace: Photo courtesy of The George (Apartment)

Samantha deVries: Structural Option
Advisor: Ali Said

Project Sponsor: Rathgeger/Goss Associates
https://www.engr.psu.edu/ae/thesis/portfolios/2015/sjd5225/deVries_AE_Thesis/Home.html

Site Plan and Location of Building

11141 Georgia Ave is Located in Wheaton Maryland near the Wheaton Metro Station. To the west of the site is a mainly commercial zone, while to the east is a residential zone. The site itself is combined commercial-residential. Figures 1 and 2 below illustrate the building's location.

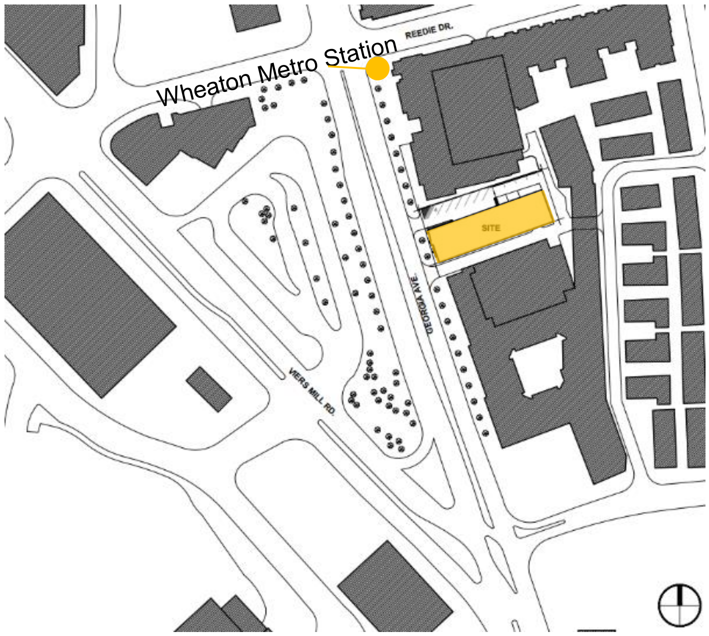


Figure 1: Building Location on Site, Courtesy of Bonstra Haresign Architects

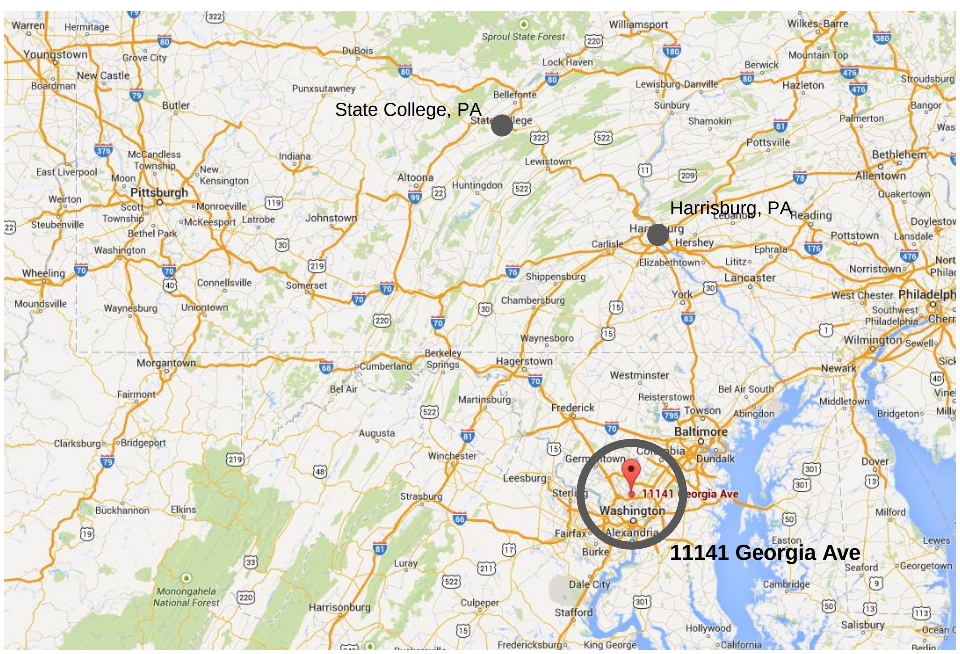


Figure 2: Map showing building location relative to State College and Harrisburg

Documents used during preparation of report

The following is a list of the structural codes used on the project. The codes used in the original 1962 drawings were not available. The codes used on the new addition to and renovation of the original building will be the referenced codes in this and future technical. The following codes will be used to determine current loads on the structure.

International Code Council

International Building Code 2009

American Society of Civil Engineers

ASCE 7-05: Minimum Design Loads for Buildings and Other Structures

American Concrete Institute

ACI 318-11

American Institute of Steel Construction

AISC Steel Manual 14th Edition

Vulcraft Deck Catalog

Steel Joist Institute

Standard Specifications for Composite Steel Joists

Previous Course Notes

Concrete Design (AE 402)

Advanced Concrete Design (AE 431)

Advanced Steel Design (AE 403)

Roof Loads

The roof loads calculation includes the roof dead loads, roof live loads, and snow loads. The loads calculated will also be compared to the loads used in the design of the building. Figure 3 and 4 below shows the layers of roofing considered in the dead load calculations.

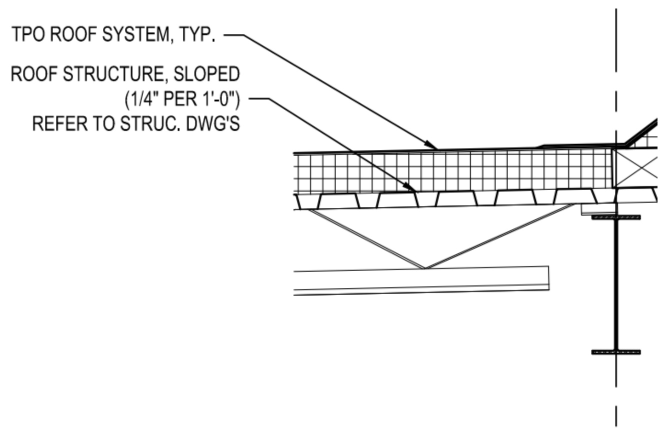


Figure 3: Section through penthouse roof. From 1/A4.09

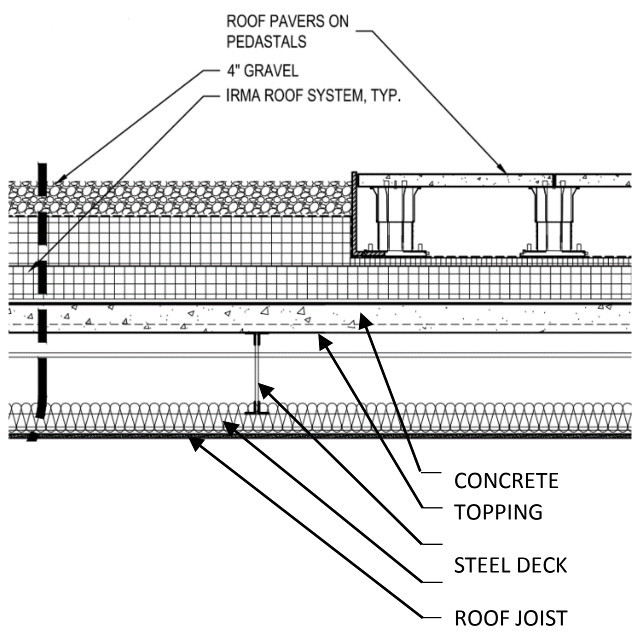


Figure 4: Section through roof at the 12th floor terrace level. From 3/A4.09.

Note: IRMA (Inverted Roof Membrane Assembly) roof system includes a membrane layer and rigid insulation

Tech Report 2 | Roof Loads | Samantha deVries

Roof Dead Load

Penthouse Roof: Load (psf)

Joist/Beam Allowance	10
Roof Decking	10
Roofing System	7
	$\boxed{27\text{ psf}}$

12th Floor Terrace:

Concrete/Deck	37
Joist/Beam Allowance	10
4" rigid insulation	3
Drop Ceiling	5
MEP	15
Sprinklers	3
Pavers or Tiles	25
	$\boxed{98\text{ psf}}$

Roof Live Load

Penthouse Roof:

Code minimum is 20 psf
(Table 4-1: Ordinary flat roofs)

Use $\boxed{30\text{ psf}}$ (value used in design)

12th Floor Terrace:

Table 4-1: Roofs used for assemby purposes

Use $\boxed{100\text{ psf}}$ (same as design value)

* Note: drawings indicate that snow load must be used instead as the live load where it is the larger value

Tech Report 2 | Snow Load | Samantha deVries

ASCE 7-05 : Chapter 7

Section 7.3 : Flat Roof Snow Loads

$p_f = 0.7 C_e C_t I p_g$

$p_g = 25$ psf (Figure 7-1)

$C_e = 0.9$ (Table 7-2) Terrain Category B
Roof Fully Exposed

$C_t = 1.0$ (Table 7-3)

$I = 1.0$ (Table 7-4) Use w/ importance Category II

$p_f = 0.7(0.9)(1.0)(1.0)(25) = 15.8$ psf

min. where $p_g > 20$ psf
$p_f = 20(I) = 20(1.0)$

$\boxed{p_f = 20 \text{ psf}}$ (Design snow load = 20 psf)
< 30 psf LL on Penthouse Roof

Snow Drift Section 7.7: Drifts on Lower Roofs

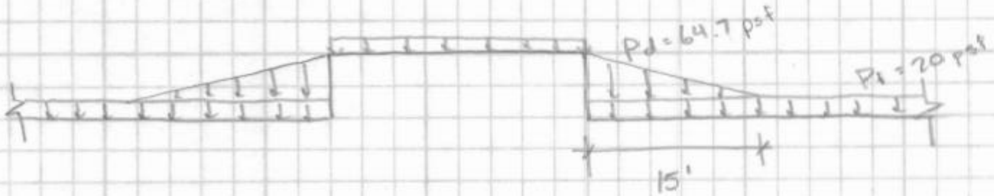
$\gamma = 0.13 p_g + 14 = 0.13(25) + 14 = 17.25$
$h_b = p_s / \gamma = 15.8 / 17.25 = 0.916$
$h_c = 15' \rightarrow h_c / h_b = 16.4 > 0.2$ (must calc. drift)

$l_{upper\ roof} = 128'$ $l_{lower\ roof} = 40'$

leeward drift (Fig. 7-9 w/ 128')
$h_d = 3.75' \rightarrow$ use larger value
windward drift (Fig. 7-9 w/ 40')
$h_d = 2.0'$

$h_d < h_c = 15$, so $w = 4h_d = 4(3.75) = 15'$

$p_d = h_d \gamma = 3.75(17.25) = \boxed{64.7 \text{ psf}}$ < 100 psf LL on level 12



Floor Loads

The floor load calculations will include both the dead and live loads for both the original concrete floors and the new addition's floors. Figure 5 below shows a section through a typical concrete slab in the original building, and figure 6 shows a section through a typical floor of the addition.

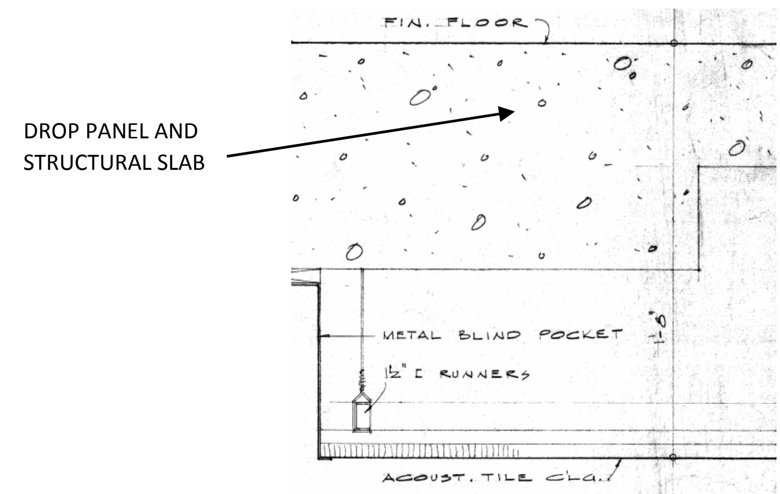


Figure 5: Section through typical floor in existing building. From A.12: Window & Wall Sections

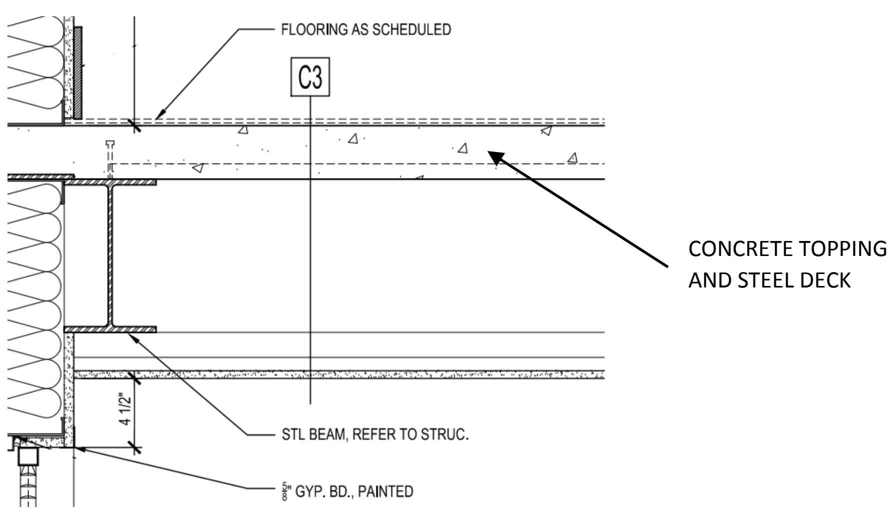


Figure 6: Section through typical floor in addition. From 10/A4.20

Tech Report 2 | Floor Loads | Samantha deVries

Floor Dead Loads

Concrete Floor	Load (psf)
Drop Ceiling	5
MEP	15
Sprinklers	3
Concrete 6 1/2"	81.25
or 8" × 150 pcf	100

6 1/2" slab : 105 psf
8" slab : 123 psf

Steel Framed Floors	
Ceiling	5
MEP	15
Sprinklers	3
Beam / Joist Allowance	15
Concrete / Deck	37

75 psf

Floor Live Loads

Area	Code Min. (psf)	Design Value
Residential	40	40
Lobbies / Stairs / Exits	100	100
Penthouse Floor	100	100
Lobby Floor	100	100
Corridors above 1st Floor	40	40
12th Floor Corridors	40	100
Parking	40	40

Note: Residential Areas also receive a 20 psf partition Allowance

Perimeter and Exterior Wall Loads

The exterior wall load calculations will produce a line load around the perimeter of the building for the original façade and the new façades. Figure 7 is a typical section through the exterior wall in the original building, and figure 8 is a section through a typical exterior wall in the addition.

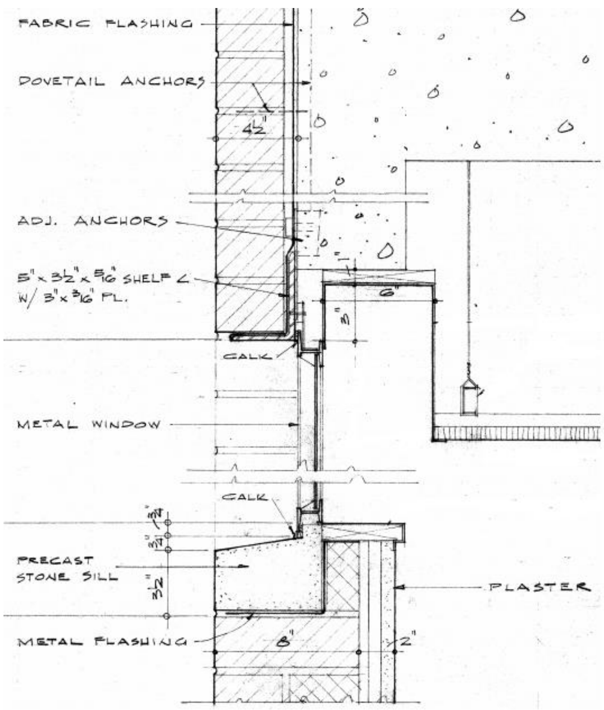


Figure 7: Section through typical exterior wall in existing building. From A.12: Window & Wall Sections

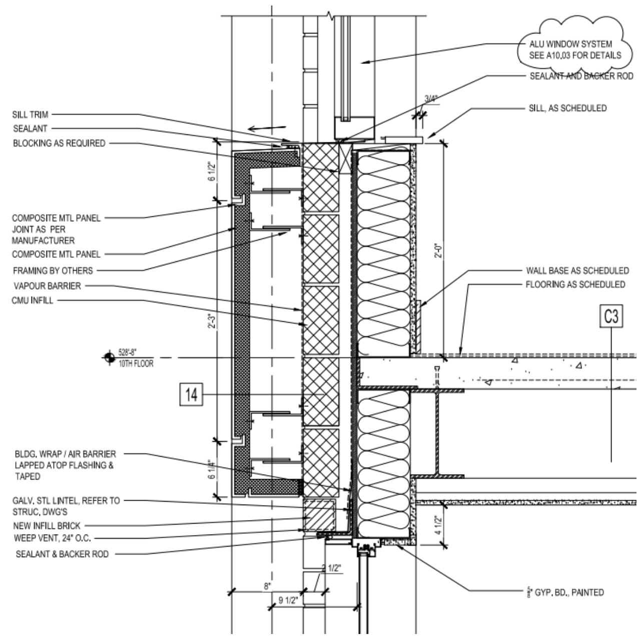


Figure 8: Section through typical exterior wall in addition. From 4A.21.

Wall Load Path

The exterior façade components, such as the brick or metal panels, rest on a steel angle at each level, and the gypsum board and insulation rests on the framed interior wall, which is attached to the brick or CMU. Therefore, the exterior wall loads acts as a line load at each floor slab around the perimeter of the building. The load on the slab edge is then carried by the slab to the exterior columns, which then carry the load down to the foundations, followed by the soil.

Tech Report 2 | Exterior Wall Loads | Samantha deVries

Typical Existing Building Wall Dead Load:

Applied as a line load at the edge of the slab

8" Brick Layer (assume hard brick)

130 pcf × 8/12 = 87 psf × 11' typ. = 957 plf

3/4" layer gypsum board

50 pcf × 0.75/12 × 11' = 34.4 plf

Total = $\boxed{992 \text{ plf}}$

Typical Addition Wall Dead Load:

Composite Metal Panel

5 psf × 11' = 55 plf

CMU Infill (or Brick facade w/out metal panel)

29 psf (CMU) × 11' = 319 plf or 38 psf (brick medium weight) × 11' = 418 plf

Water Membrane

2 psf × 11' = 22 plf

3/4" gypsum board = 34.4 plf

Fibrous glass insulation

1.1 psf × 11 = 12.1 plf

Total: at metal panels = $\boxed{443 \text{ plf}}$

at brick faces = $\boxed{487 \text{ plf}}$

Tech Report 2 | Gravity Loads | Samantha deVries

Non-Typical Dead Loads

Floors & Roofs:

At 3/4" drop panels (7' x 7') existing building

3/4" x 150 pcf = $\boxed{9 \text{ psf}}$

Existing Building Perimeter Beams

12" x 150 pcf x 12" width (avg.) = $\boxed{150 \text{ plf}}$

16" depth = $\boxed{200 \text{ plf}}$

18" = $\boxed{225 \text{ plf}}$

24" = $\boxed{300 \text{ plf}}$

30" = $\boxed{375 \text{ plf}}$

(Note: there is a large variety of perimeter beam sizes, so this is a sample to provide a range of additional load)

Typical Member Spot Checks for Gravity Loads

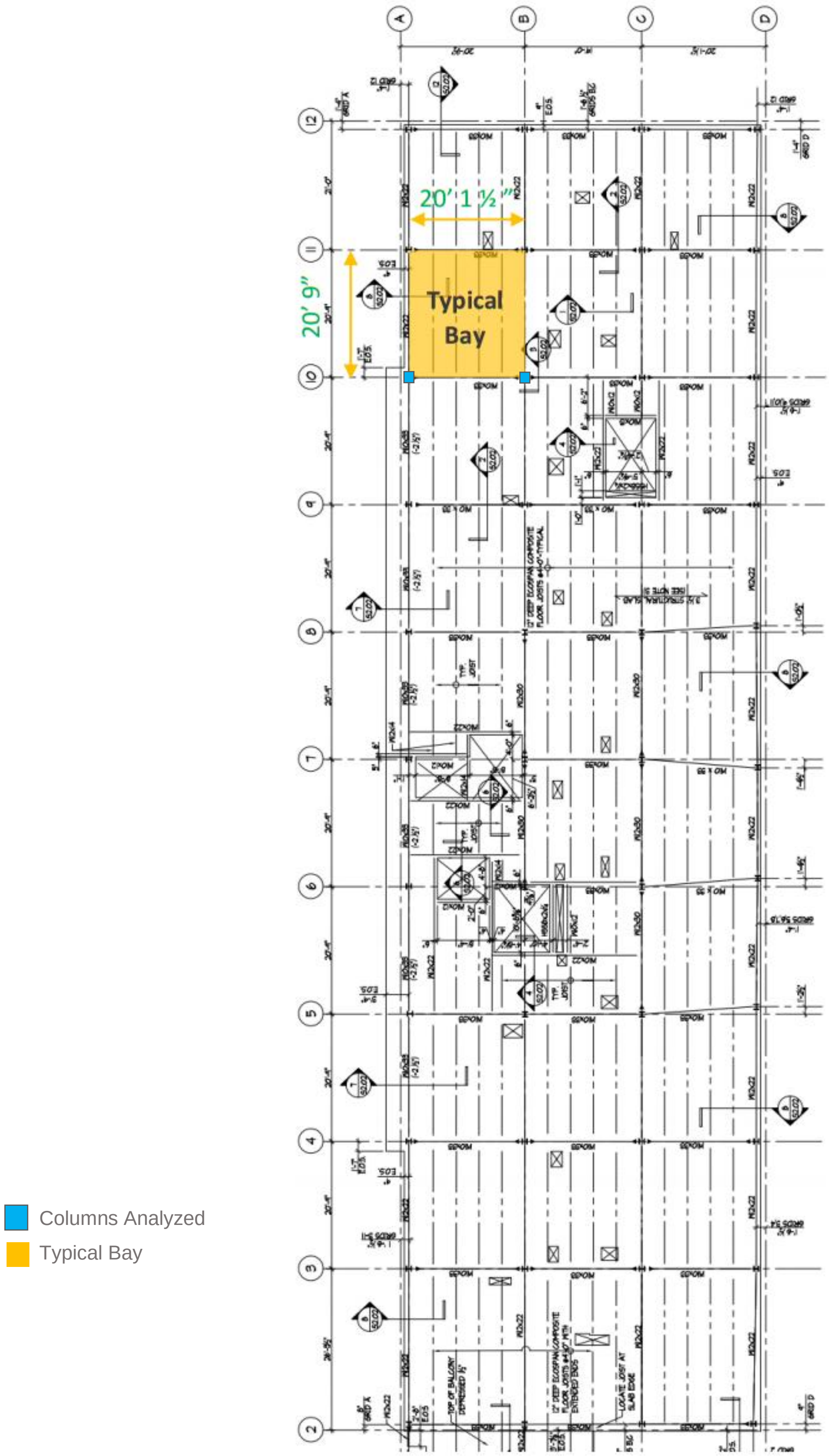
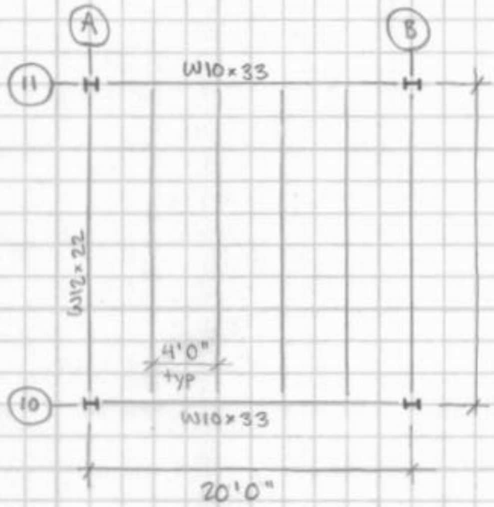


Figure 9: Typical Bay and Columns Analyzed for Gravity Loads, S1.07

Tech Report 3 | Spot Checks | Samantha deVries

Gravity Check - Typical Bay in Existing Addition



- Bay typical for levels 7 through 11.
- Bay infill members are 12" deep composite floor joists.
- Slab: 2 1/2" NW concrete 1" deep, 24 gage galv. form deck, w/ 6x6 - W2.1xW2.1 WWF

Dead Load from Tech 2: 75 psf
Live Load from Tech 2: Residential → 40 psf
Corridors above 1st floor → 40 psf

Spot Check Deck:

using 1.0 C, CSV conform, 24 gage

Max Construction Clear Span

for NW concrete, 3 span = 5'10" > 4' ✓

Check $W_{TL} \leq$ allowable total

$W_{TL} = 75 + 40 = 115$ psf

1.0C24, 3span, $F_b = 36{,}000$

Allowable uniform load = 191 psf > 115 psf ✓

$W_{TL} \leq L/180$ for roof total load

57 psf ≤ 149 psf ✓

$W_{LL} \leq L/240$ for floor live load: 40 psf ≤ 112 psf ✓

Tech Report 3 | Spot Checks | Samantha deVries

Spot Check Floor Joists

12" deep ecospan composite floor joists.

Use Steel Joist Institute:
standard specifications for Composite Steel Joists

DL: 75 psf × 4 ft trib width = 300 plf
LL: 40 psf × 4 ft trib. = 160 plf

Factored Uniform Load:

$1.2(300) + 1.6(160) = 616$ plf

Using 25' joist span, 12" joist depth:

for joist that is 8.0 plf,
total safe factored unif. dist. load = 700 plf > 616 plf ✓

Live Load Reduction for girders:

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right)$$

$L_o = 40$
$K_{LL} = 2$
$A_T = 415$

$$L = 40\left(0.25 + \frac{15}{\sqrt{2(415)}}\right) = 30.8 \text{ psf}$$

30.8 × 4 ft trib = 123 plf on joist

Tech Report 3 | Spot Check | Samantha deVries

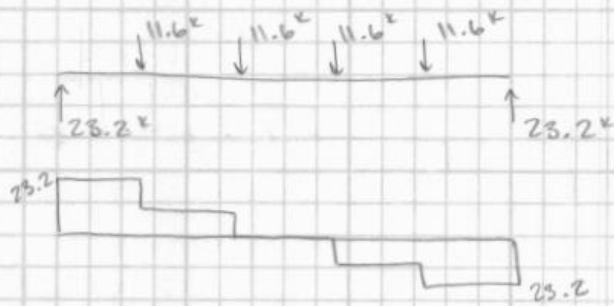
Girder Spot Check

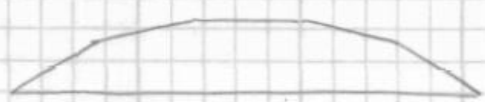
Span = 20'0"
Spacing = 20'9"
W10x33

Loads from beams

$P_{DL} = 300\text{ plf }(20.75') = 6.23\text{ kip}$
$P_{LL} = 123\text{ plf }(20.75') = 2.56\text{ kip}$

$1.2(6.23) + 1.6(2.56) = 11.6\text{ kip}$





$M_{max} = 23.2(4') + 11.6(4')$

$M_{max} = 139.2\text{ ft}\cdot\text{k}$

W10x33:
$\phi M_n = 146\text{ ft-k} > M_u = 139.2\text{ ft-k}$ ✓ Strength Check

Check Deflections: (eq. from steel manual)

$$\Delta_{LL} = \frac{0.063 P l^3}{EI} = \frac{0.063(2.56)(20)^3(1728)}{(29{,}000)(171\text{ in}^4)} = 0.45\text{ in.}$$

$$\frac{l}{360} = \frac{20\cdot 12}{360} = 0.67 > 0.45\text{ in}$$ ✓

Check Camber

$$\Delta_{DL\,(\text{wet conc})} = \frac{0.063(37\text{psf}\cdot 4'\cdot 20.75'/1000)(20)^3(1728)}{29000(171)}$$

$= 0.54"(0.8) = 0.43 < 0.75" \rightarrow$ no camber ✓

Summary

typical W10x33 passes for strength & deflections

Tech Report 3 | Spot Checks | Samantha deVries

Column Spot Checks

Interior Column B10 & Exterior Column A8

Interior

W10x49 (Level 6 to Penthouse)
18x18 conc (Level 1 to 5)
20x20 conc. (B2 & B1)

Check steel W10x49 at base (worst-case loading)
$1.2D + 1.6L + 1.0S$

Floor DL = 75 psf
Floor LL = 40 psf
12th level DL = 98 psf
12th level LL = 100 psf
Roof DL = 27 psf
Roof LL = 30 psf
Roof snow load = 20 psf

LL reduction

Floor: $L = 40\left(0.25 + \frac{15}{\sqrt{4(415)}}\right) = 24.7$ psf

12th level: $L = 100\left(0.25 + \frac{15}{\sqrt{4(415)}}\right) = 61.8$ psf

$P_u = \left(1.2[(75 \cdot 6) + 98 + 27] + 1.6[(24.7 \cdot 6) + 61.8 + 30] + 20\right)(415)/1000 = 454$ kip

+ weight of columns = $80'(49\ plf)/1000 + 454$

$P_u = 458$ kip

Steel Manual table 4-1:

Effective Length = 10'4"

$\phi P_n = 544$ kip $> P_u = 458$ kip ✓

column passes strength check

Tech Report 3 | Spot Checks | Samantha deVries

Exterior

W10x33 (Level 6 to Penthouse)
18x14 conc (Level 1 to 5)
18x16 conc (B1) 18 x 27 1/2 conc (B2)

Check steel W10x33 at base (worst-case)

Uniform Loads same as previous interior

LL Reduction:

Floor: $L = 40\left(0.25 + \frac{15}{\sqrt{4(207.5)}}\right) = 30.8$ psf

12^{th} level: $L = 100\left(0.25 + \frac{15}{\sqrt{4(207.5)}}\right) = 77.1$ psf

Roof is unreducible

$P_u = \left(1.2[(75\cdot6)+98+27] + 1.6[(30.8\cdot6)+77.1+30] + 20\right)(207.5)/1000 = 244$ kip

+ Column self-weight + ext. wall weight

= 244 kip + 80' (33 plf)/1000 + 487 plf (20.75')/1000

P_u = 257 kip

Table 4-1:

Effective length = 10'4"

$\phi P_n = 323$ kip > $\phi P_u = 257$ kip ✓

column passes strength check

Alternative Framing Systems for Gravity Loads

Three alternate framing systems were explored:

Alternate #1: Non-Composite wide-flange steel

Alternate #2: Two-way slab with drop panels and perimeter beams

Alternate #3: One-way slab with girders

Alternate #1 notes:

Same layout used for comparison purposes. However, if this system were chosen, it would allow for larger spans in the layout and the removal of some columns.

Alternate #2 notes:

Initial calculations determined that drop panels were ultimately not necessary. However, this is based on a started column size of 24 inches square. In the case that this framing system is further explored and the column size decreases, shear must be reconsidered.

Not all code checks were performed. The calculations completed in this report were only intended to get some initial sizes and check that the required reinforcement seems reasonable at an initial stage of design.

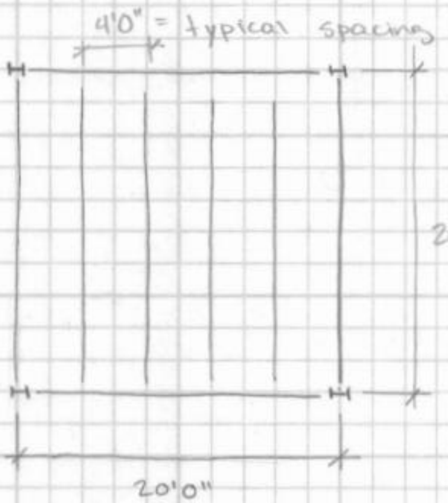
Perimeter beams may be controlled by their contribution to lateral system moment frames, which will be further explored should the system be kept for future use.

Alternate #3 notes:

As in alternate #2, not all concrete code checks were performed. The calculations completed in this report were only intended to get some initial sizes and check that the required reinforcement seems reasonable at an initial stage of design.

Tech Report 3 | Alternate #1 | Samantha deVries

Framing Alternate #1 – Non-Composite Steel



Same beam spacing as existing system → use same decking

note: beam allowance in dead load calc was conservative enough to use in beam alternate.

Beam

Factored Uniform Load from existing system = 616 plf

$$M_u = \frac{w_u l^2}{8} = \frac{0.616\,(20.75)^2}{8} = 33.2 \text{ ft·k}$$

Deflections:

$l/240$ (for total unfactored load) = $(20.75 \times 12)/240$ = 1.037 in.

$l/360$ (for live load unfactored) = $(20.75 \times 12)/360$ = 0.691 in

D+L = 115 psf (4') = 460 plf
L = 40 psf (4') = 160 plf

$$\Delta_{max} = \frac{5wl^4}{384EI} \rightarrow I = \frac{5wl^4}{384E\Delta_{max}}$$

$$I_{D+L} = \frac{5\,(0.460)(20.75)^4(1728)}{384\,(29000)(1.037)} = 63.8 \text{ in}^4$$

$$I_L = \frac{5\,(0.160)(20.75)^4(1728)}{384\,(29000)(0.691)} = 33.3 \text{ in}^4$$

$\boxed{W12\times14}$ → $I = 88.6 \text{ in}^4 > 63.8 \text{ in}^4$ ✓
$\phi M_n = 65.3 \text{ ft·k} > 33.2 \text{ ft·k}$ ✓
ϕM_n @ unbraced length = 4' = 56 ft·k ✓

Tech Report 3 | Alternate #1 | Samantha deVries

Girder

Load based on tributary area approximation:

$[1.2(75) + 1.6(40)] \times 20.75' = 3.2$ klf

$M_u = \frac{w_u l^2}{8} = \frac{3.2(20)^2}{8} = 160$ ft·k

Deflections:

$L/240 = (20 \times 12)/240 = 1.0$ in

$L/360 = (20 \times 12)/360 = 0.67$ in

D+L = 115 psf (20.75) = 2.39 klf
L = 40 psf (20.75) = 0.83 klf

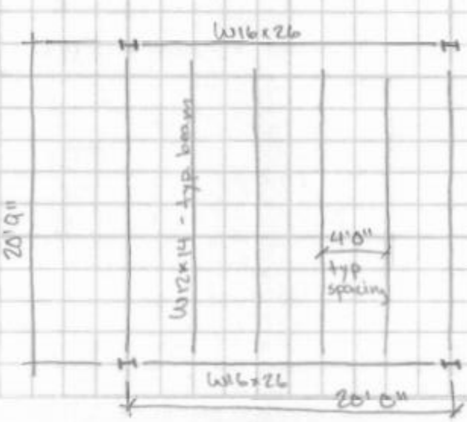
$I = \frac{5 w l^4}{384 E \Delta_{max}}$

$I_{240} = \frac{5(2.39)(20)^4(1728)}{384(29000)(1.0)} = \underline{296.7 \text{ in}^4}$

$I_{360} = \frac{5(0.83)(20)^4(1728)}{384(29000)(0.83)} = 124.1 \text{ in}^4$

$\boxed{\text{W16x26}}$ → $I = 301$ in^4 > 296.7 in^4 ✓
$\phi M_n = 166$ ft·k < 160 ft·k ✓
ϕM_n @ unbraced length = 165 ft·k ✓

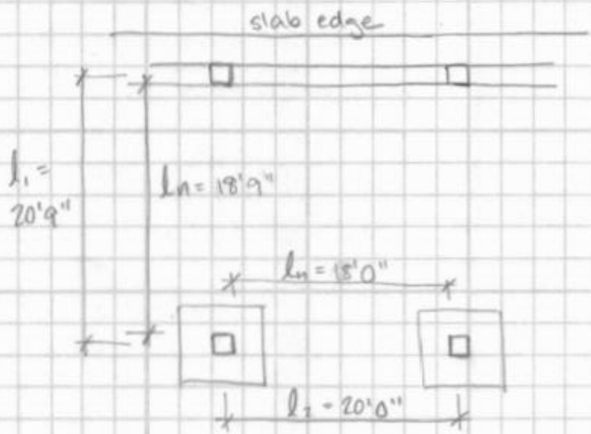
Non Composite Steel Summary:



Deck: 2½" NW concrete
1" deep ribs, 24 gage
w/ 6x6 -W2.1xW2.1 WWF

Tech Report 3 | Alternate #2 | Samantha deVries

Framing Alternate #2 – Two-way slab with Drop Panels and Perimeter Beams



Start w/ 24" columns

Deflections

No interior beams → use table 9.5(c) in ACI 318-11

f_y = 60,000 psi w/ drop panels

ext. panel w/ edge beams: $l_n/36$
int. panels: $l_n/36$

min thickness of slab without interior beams

$l_n/36 = (18'9")(12)/36 = 6.25$

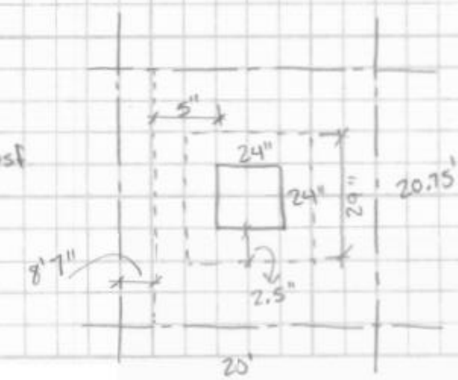
Therefore, start with a <u>6 ½"</u> slab

Drop Panels

Live Load = 40 psf
Dead Load = 105 psf

$q_u = 1.2(105) + 1.6(40) = 190$ psf

$d \approx 5"$



Tech Report 3 | Alternate #2 | Samantha deVries

Check one-way shear:

$V_u = 0.190\ (8'7")(20.75') = 33.8\ k$

$V_c = 2\sqrt{f'c}\ b_w d = \dfrac{2\sqrt{4000}\ (20.75' \times 12)(5.0)}{1000} = 157.5\ k$

$\phi V_c = 0.75(157.5) = 118.1\ k > 33.8\ k$ ✓

Check two-way shear:

$V_u = 0.190\ \left(20(20.75) - (29/12)^2\right) = 77.7\ k$

$\alpha_s = 40$ for interior
$\beta = 20.75/20 = 1.04$
$b_o = 4(29") = 116"$

$$V_c = \frac{\sqrt{4000}\ (116)(5)}{1000} \begin{bmatrix} \left(2 + \frac{4}{1.04}\right) = 5.85 \\ \left(\frac{40(5)}{116} + 2\right) = \underline{3.72} \\ \text{min} \quad 4 \end{bmatrix} = 136.5\ k$$

$\phi V_c = 0.75(136.5) = \underline{102.3\ k > 77.7\ k}$

→ slab works for shear
<u>without drop panels</u>

Recheck slab minimum thickness without drop panels:

back to table 9.5 (c)

ext. panel w/ edge beams: $l_n/33$
int. panels: $l_n/33$

$l_n/33 = (18'9")(12)/33 = 6.82$ → use <u>7" slab</u> w/out drop panels

Tech Report 3 | Alternate #2 | Samantha deVries

Determine Slab Moments

Dead Load w/ 7" slab = $105 + \frac{0.5}{12}(150) = 111.3$ psf

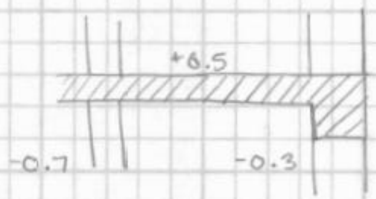
$q_u = 1.2(111.3) + 1.6(40) = 197.6$ psf

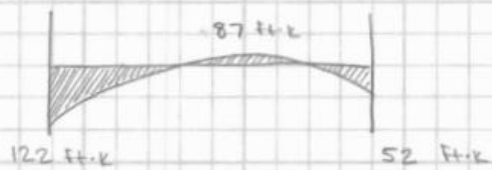
$M_{o_{\text{short dir}}} = \frac{q_u l_1 l_n^2}{8} = \frac{197.6\,(20.75')(18)^2}{8} = 166.1$ ft·k

$M_{o_{\text{long dir}}} = \frac{q_u l_2 l_n^2}{8} = \frac{197.6\,(20)(18.75)^2}{8} = 173.7$ ft·k

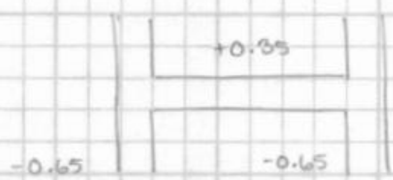
Longitudinal Moment: Bay meets requirements to use DDM

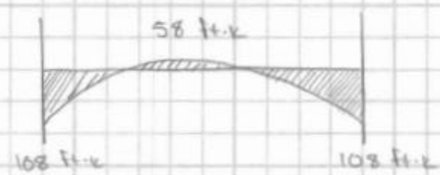
Exterior beam, no interior beams (Exterior Span)
Long Direction





No beams (Interior Span) Short Direction





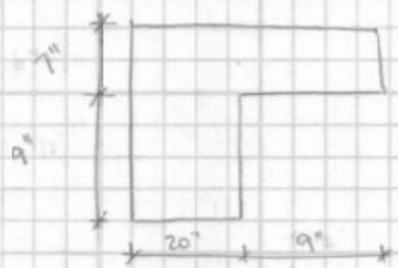
Tech Report 3 | Alternate # 2 | Samantha deVries

Column & Middle Strip Moments

α_f : α_{f_1} (along long direction) = 0 (no beams)

$\alpha_{f_1} = \dfrac{E_{cb} I_b}{E_{cs} I_s} \rightarrow E_{cb} = E_{cs} \rightarrow \alpha_{f_1} = \dfrac{I_b}{I_s}$ (along short direction)

For now, try 16" deep beam & width of 20" columns



$\bar{y} = \dfrac{8(16 \cdot 20) + 12.5(7 \cdot 29)}{(16)(20) + (7)(29)}$

$\bar{y} = 9.75"$

$I_b = \dfrac{20(16)^3}{12} + (20)(16)(1.75")^2 + \dfrac{(9)(7)^3}{12} + (9)(7)(2.75)^2$

$I_b = 8540 \text{ in}^4$

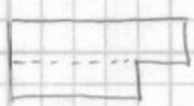
$I_s = \dfrac{(12 \times 10.4)(7)^3}{12} = 3567 \text{ in}^4$

$\alpha_{f_1} = \dfrac{8540}{3567} = 2.4$

$l_2/l_1 = \dfrac{20}{20.75} = 0.96$

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

$C = \left(1 - 0.63\left(\frac{16}{20}\right)\right)\left(\frac{16^3 20}{3}\right) + \left(1 - 0.63\left(\frac{7}{9}\right)\right)\left(\frac{7^3 9}{3}\right) = 14069$

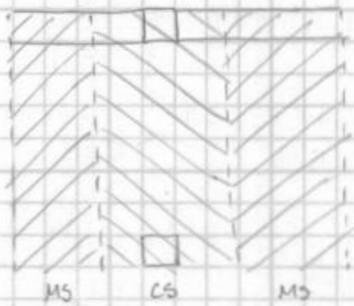


$C = \left(1 - 0.63\left(\frac{7}{29}\right)\right)\left(\frac{7^3 29}{3}\right) + \left(1 - 0.63\left(\frac{9}{20}\right)\right)\left(\frac{9^3 20}{3}\right) = 6294$

$\beta_T = \dfrac{\cancel{E_{cb}} C}{2 \cancel{E_{cs}} I_s} = \dfrac{14069}{2(3567)} = 1.97$

Tech Report 3 | Alternate #2 | Samantha deVries

Long Direction:



$\alpha_{f_1} = 0$

% of neg. moment @ int. support

$\alpha_{f_1} \, l_2/l_1 = 0 \rightarrow 75\%$

% of positive moment

$\alpha_{f_1} \, l_2/l_1 = 0 \rightarrow 60\%$

% of neg. moment @ ext. support

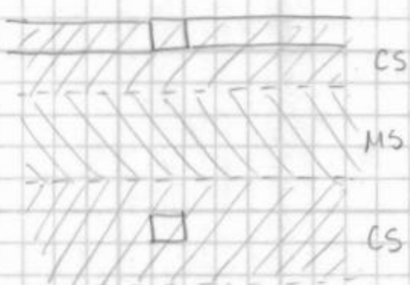
$\beta_T \approx 2.0 \rightarrow 80\%$

52 ft·k → 75% CS = 39 ft·k
→ 25% MS = 13 ft·k

87 ft·k → 60% CS = 52.2 ft·k
→ 40% MS = 34.8 ft·k

122 ft·k → 80% CS = 97.6 ft·k
→ 20% MS = 24.4 ft·k

Short Direction:



$\alpha_{f_1} \approx 2.4 \quad \beta_T \approx 2.0$

% neg. moment @ int. support
interpolate → 78.5%

% positive moment
interpolate → 72%

108 (interior) → 78.5% CS = 84.8 ft·k
→ 21.5% MS = 23.2 ft·k

58 ft·k → 72% CS = 41.8 ft·k
→ 28% MS = 16.2 ft·k

108 (ext.) → 78.5% CS = 84.8 ft·k [85% to beam = 72.1 ft·k
15% to slab = 12.7 ft·k
→ 21.5% MS = 23.2 ft·k

Tech Report 3 | Alternate #2 | Samantha deVries

Summary of 2-way slab moments: (ft·k)

Direction	Strip	Left Side	Middle	Right Side
Long	Column	97.6	52.2	39
	Middle	24.4	34.8	13
Short	Col. (slab)	12.7	41.8	84.8
	(beam)	72.1	N/A	N/A
	Middle	23.2	16.2	23.2

Flexural

long bars: $d \approx h-1.1 = 5.9"$
short bars: $d \approx h-1.7 = 5.3"$

$A_{s_{min}} \geq 0.0018bh = 0.0018(12")(7") = 0.152 \text{ in}^2/\text{ft}$

$A_{s_{req}} = \dfrac{M_u \cdot 12^{"/'}}{\phi f'_y jd}$ $\phi = 0.9$
$f'_y = 60$ ksi
per ft $- A_{s_{req}}/10'$ $jd = 0.95d$
$= 0.95(5.9") = 5.61$ (long)
$= 0.95(5.3") = 5.04$ (short)

Left Side (Top)

Direction	Strip	Moment	$A_{s_{req}}$	Bar Size	Steel Area
Long	Column	97.6	0.386	#6	0.44
	Middle	24.4	0.097	#3	0.11
Short	Column	12.7	0.057	#3	0.11
	Middle	23.2	0.102	#3	0.11

Middle (Bottom)

Direction	Strip	Moment	$A_{s_{req}}$	Bar Size	Steel Area
Long	Column	52.2	0.206	#4	0.2
	Middle	34.8	0.138	#4	0.2
Short	Column	41.8	0.185	#4	0.2
	Middle	16.2	0.072	#3	0.11

Right Side (Top)

Direction	Strip	Moment	$A_{s_{req}}$	Bar Size	Steel Area
Long	Column	39	0.155	#4	0.2
	Middle	13	0.052	#3	0.11
Short	Column	84.8	0.374	#6	0.44
	Middle	23.2	0.102	#3	0.11

$S_{max} = 2h = 14"$
use 12" for initial bar selection

Tech Report 3 | Aternate #2 | Samantha deVries

Perimeter Beam Design for Gravity

Choose depth based on deflections

Table 9.5(a) in ACI - minimum beam thickness
beams → both ends continuous (interior bay in direction of beam)

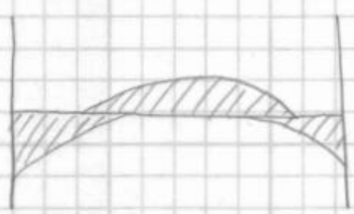
$$l/21 = (20' \times 12''/') / 21 = 11.4 \text{ in.} \rightarrow \text{use } 12''$$

A 16" beam was used in slab design. Using a 12" beam would affect the distribution of moment. The perimeter beams are also intended to create moment frames for the lateral system. For this report, a 12" beam will be used as an initial size to check gravity effects only.

Strength Check: since columns are 24", use 24" wide beam for this report

Moment in beam due to distribution of slab loads from previous portion: 72.1 ft·k

Beam weight = 150 pcf × 12" × 24" = 300 plf
Ext. wall load = 487 plf
Total Line Load = 787 (1.2) = 945 plf factored



Interior Span:
Moments from ACI coefficients

$$\text{Neg.} = \frac{wl^2}{11} = \frac{0.945(20)^2}{11} = 34.4 \text{ ft·k}$$

$$\text{Pos.} = \frac{wl^2}{16} = \frac{0.945(20)^2}{16} = 23.6 \text{ ft·k}$$

Total Conservative M_u = 72.1 + 34.4 = 106.5 ft·k

Approximate d → if #9 bars used
d = 12 - 1.5 clear cover - 0.5" #4 stirrup - $\frac{1}{2}$(0.5) = 9.75"

$$A_{s,req} = \frac{M_u}{\phi f_y j d} = \frac{106.5 \cdot 12}{0.9(60)(0.95)(9.75)} = 2.56 \text{ in}^2$$

↳ reasonable

Tech Report 3 | Alternate #2 | Samantha deVries

Beam Shear Check

Line Load = 945 plf

w_u from uniform loads:

$[1.2(23) + 1.6(40)] \cdot 2'$ beam = 1.83 k

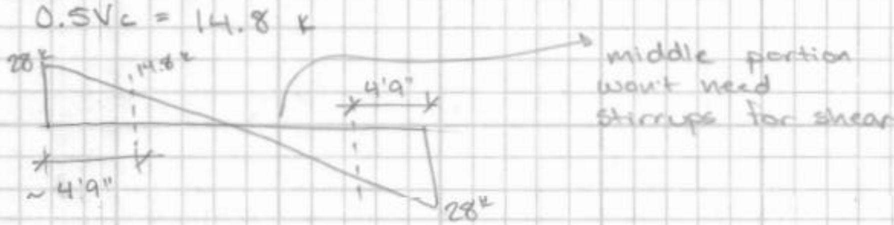
Total factored w_u = 2.78 klf

$V_c = 2\sqrt{f'c}\, b_w d = \dfrac{2\sqrt{4000}\,(24'')(9.75'')}{1000} = 29.6^k$

$V_u \le \phi V_c$

$28 \le 0.75(29.6) = 22.2$, need stirrups

$0.5V_c = 14.8$ k



V_u @ d from support $= 28 - 2.8(9.75/12) = 25.7^k$

$V_s = \dfrac{V_u}{\phi} - V_c = \dfrac{25.7}{0.75} - 29.6 = 4.67^k$

$V_s \le \dfrac{8\sqrt{4000}\,(24)(9.75)}{1000} = 118.4 > 5$ ✓

$S_{max} = \min \begin{cases} d/2 = 4.875 \rightarrow \text{use } 4.5'' \\ 24'' \end{cases}$

$S_{req} = \dfrac{A_v f_{yt} d}{V_s} = \dfrac{0.11(60)(9.75)}{4.67} = 13.8''$

space at 4.5"

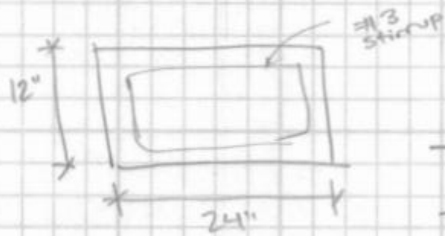
Tech Report # 3 | Alternate # 2 | Samantha deVries

Beam Torsion Check

$M_u = 34.8$ ft·k (use largest middle strip moment for approximation – long span)

$M_u = 34.8 / 20' = 1.74$ ft·k / 1' strip width

$T_u = 1.74\left(\frac{1}{2}\right)(\underset{l_n}{18'}) = 15.7$ ft·k



$A_{cp} = 12''(24'') = 288\ in^2$
$P_{cp} = 2(12) + 2(24) = 72 in$

$T_{threshold} = 0.75\sqrt{4000}\left(\frac{288^2}{72}\right) = 54.6$ in·k

$T_u = 15.7 \cdot 12 = 188.4$ in·k

torsion reinforcement required

check section size:

$x_o = 24 - 2(1.5) - 0.375 = 20.6$
$y_o = 12 - 2(1.5) - 0.375 = 8.6$

$A_{oh} = 20.6''(8.6)'' = 178\ in^2$
$P_h = 2(20.6 + 8.6) = 58.4$ in

$V_c = \frac{2\sqrt{4000}(24'')(12'')}{1000} = 36.4$ k

$$\sqrt{\left[\frac{28000 \text{ lbs}}{(24)(9.75)}\right]^2 + \left[\frac{188400(58.4)}{1.7(178)^2}\right]^2} \le 0.75\left(\frac{36400}{(24)(9.75)} + 8\sqrt{4000}\right)$$

237 psi ≤ 496 ✓

Based on initial calculations, the beam will work for strength, shear, torsion, and deflections due to gravity loads.

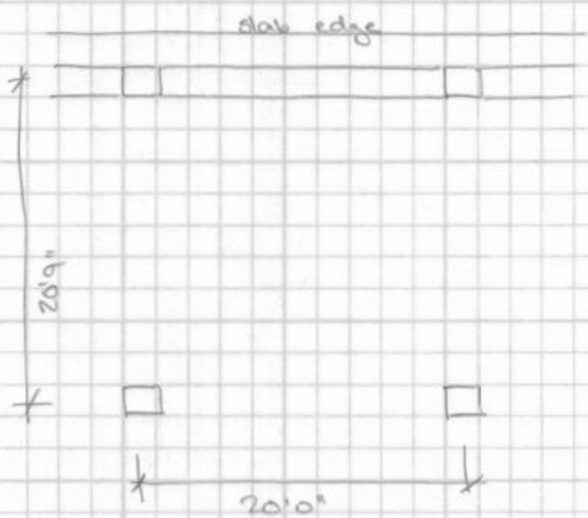
Tech Report #3 | Alternate #2 | Samantha deVries

Two-Way Slab with Perimeter Beams Summary

f'_c = 4000 psi
f_y = 60 ksi
NW concrete

Columns: 24" x 24"
Perimeter Beam: 24" wide
12" deep

Slab: 7" deep



Reinforcement:

Bottom: #4 @ 12" o.c.
Each way

Top:

Long Direction:

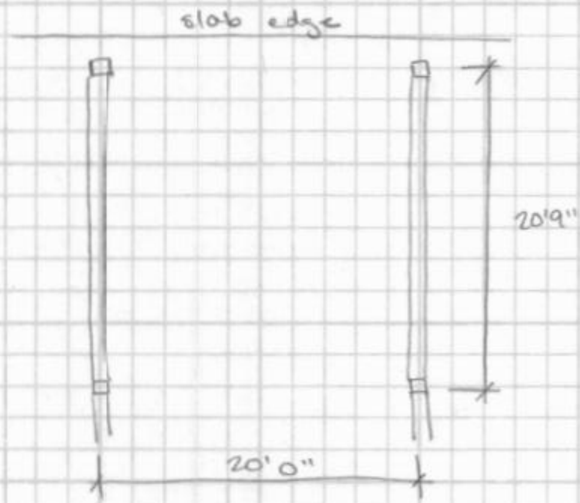
@ column: #6 @ 12"
middle: #3 @ 12

Short Direction:

@ column: #6 @ 12"
middle: #3 @ 12"

Tech Report 3 | Alternate #3 | Samantha deVries

Framing Alternate #3 – One Way Slab with Girders



Slab Design

From AE 402 notes – start w/ L/20 for initial slab thickness: $20' - 2' = l_n \rightarrow (18' \times 12)/20 = 10.8"$ → use 11"

Live Load = 40 psf
Dead Load = (Misc.) = 23 psf
11" slab = 138 psf
Total Dead Load = 161 psf

w_u (for a 1' strip) $= [1.2(161) + 1.6(40)] \cdot 1' = 257.2$ plf

$$M_u^+ = \frac{w_u l_n^2}{16} = \frac{0.2852(18)^2}{16} = +5.78 \text{ ft-k}$$

$$M_u^- = \frac{w_u l_n^2}{11} = \frac{0.2852(18)^2}{11} = -8.4 \text{ ft-k}$$

$$A_{s\,req} = \frac{M_u}{\phi f_y j d} \rightarrow d = 11 - 1.5 - \tfrac{1}{2}(0.375) = 9.3"$$

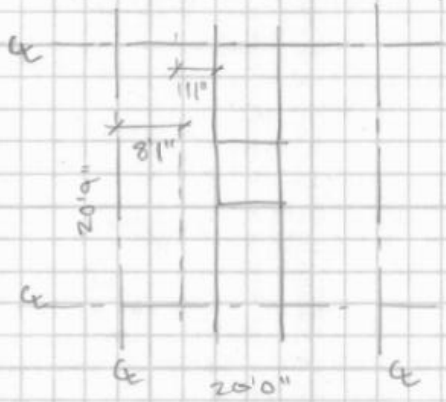
$$= \frac{8.4 \text{ ft-k} \cdot 12"/'}{0.9(60)(0.95)(9.3)} = 0.211 \text{ in}^2/\text{ft}$$

$$S_{max} = 2h = 22" \text{ or } \underline{18"} \rightarrow 0.211 \times \left(\frac{10" \text{ spacing}}{12"/'}\right) = 0.18 \text{ in}^2$$

use #4 @ 10" spacing

Tech Report 3 | Alternate #3 | Samantha deVries

Check Slab One-Way Shear



$$V_u = 0.2572 \text{ ksf } (8.083')(20.75') = 43.1 \text{ k}$$

$$V_c = \frac{2\sqrt{4000}\,(20.75' \times 12"/')(11")}{1000} = 346 \text{ k}$$

$$V_u \le \phi V_c$$

$$43.1 \text{ k} \le 0.75(346) = 260\text{ k} \checkmark$$

Slab works for One-way shear without additional shear reinforcing

Tech Report #3 | Alternate #3 | Samantha deVries

Girder Design

Initial Size for Deflection

(note: 24" columns → 24" wide girders to start)

Live Load = 40 psf
Dead Load = 161 psf

Use Table 9.5 (a) to choose approximate size for deflections

Beam → one end continuous (exterior bay)
$l/18.2 = (20.75 \cdot 12)/18.2 = 13.7 \rightarrow$ use 14"

Beam self weight $= 150 \text{ pcf } \frac{(24")(14")}{144} = 350$ plf

Total factored load $= 350 \text{ plf} + [1.2(161) + 1.6(40)] \cdot 20'$
$= \underline{5.5 \text{ klf}}$

Strength Check

ACI moment coefficients

largest: $\frac{w_u l_n^2}{10} = \frac{5.5(18.75)^2}{10} = 193.4$ ft·k

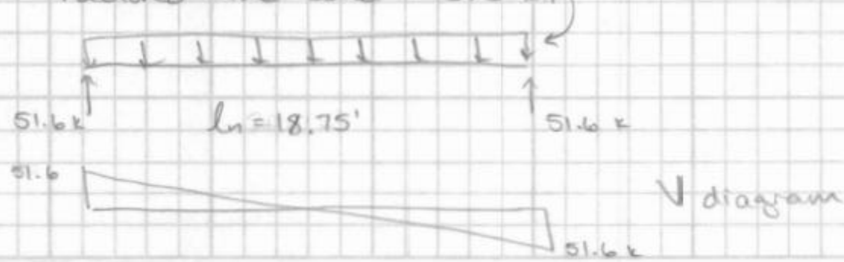
Approximate d → $14 - 1.5 - 0.5 - \frac{1}{2}(1.0) = 11.5"$
#4 stirrup #9

$A_{s\,req} = \frac{M_u}{\phi f_y jd} = \frac{193.4 \cdot 12}{0.9(60)(0.95)(11.5)} = 3.94 \text{ in}^2$

use 4 #9's → this is reasonable

Shear Check

Factored Line Load = 5.5 klf

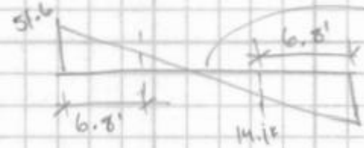


Tech Report 3 | Alternate #3 | Samantha deVries

$$V_c = 2\sqrt{f'_c}\, b_w d = \frac{2\sqrt{4000}\,(24)(9.3")}{1000} = 28.2^k$$

$$V_u \le \phi V_c \rightarrow 51.6 \le 0.75(28.2) \quad \times \rightarrow \text{need shear reinf.}$$

$$0.5V_c = 0.5(28.2) = 14.1^k$$



no stirrups for shear required here in middle portion of beam

$$V_u \text{ @ d from support} = 51.6 - 5.5\left(\tfrac{9.3}{12}\right) = 47.4^k$$

$$V_s = \frac{V_u}{\phi} - V_c = \frac{47.4}{0.75} - 28.2 = 35^k$$

$$V_s \le \frac{8\sqrt{4000}\,(24)(9.3)}{1000} = 113^k > 35^k \checkmark$$

$$S_{max} = \min\begin{cases} d/2 = 4.65 \rightarrow \text{use } 4.5" \\ 24" \end{cases}$$

$$S_{req} = \frac{A_v f_{yt} d}{V_s} = \frac{(0.20)(60)(9.3)}{35} = 3.19 \text{ in}$$

$\rightarrow$ use #4 @ 3"

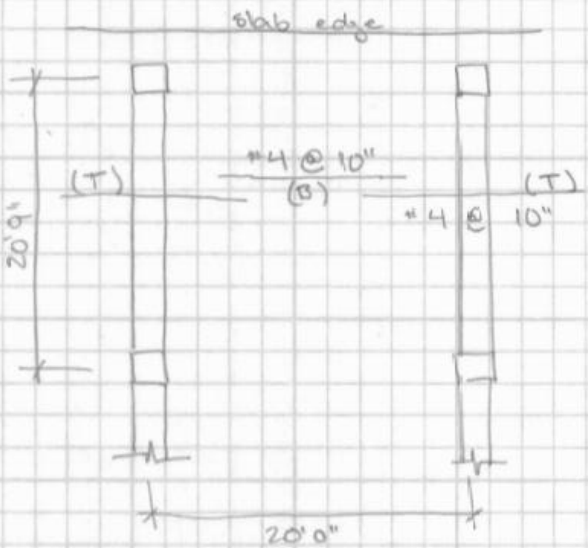
Girder in typical bay is an interior beam, so torsion would only occur in the event of uneven loading, which will be small compared to the Dead Load. Therefore, torsion of the girder in this system will not be explored in this report.

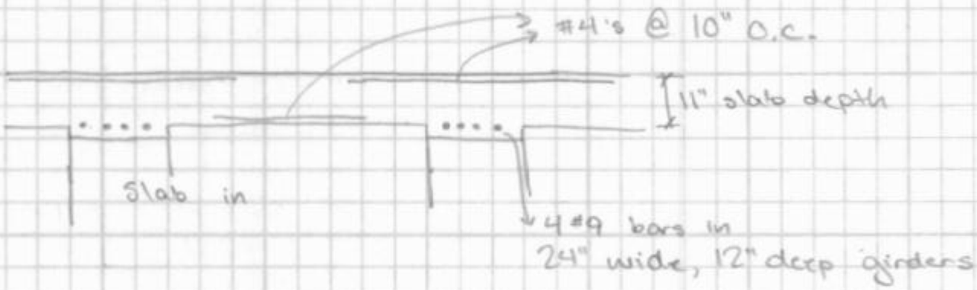
Based on initial calculations, the slab and girders will work for strength, shear, and deflections.

Tech Report 3 | Alternate #3 | Samantha deVries

One-Way Slab with Girders Summary

$f'_c = 4000$ psi
$f_y = 60$ ksi
NW concrete





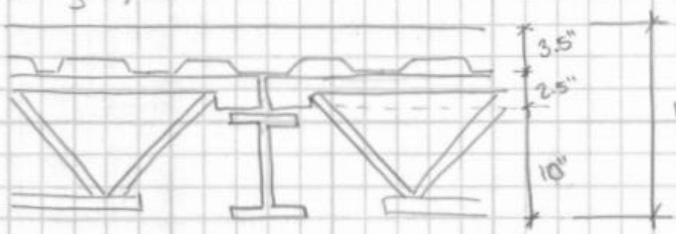
System Comparisons

Considerations		Existing Steel	Two-Way Slab	One-Way Slab	Non-Composite Steel
Architectural Considerations					
Total System Depth		16"	12"	12"	19.5"
Fire Rating		2 hr	3 hr +	3 hr +	2 hr
2 hr Fire Rating?		yes	yes	yes	yes
System Statistics					
Durability		acceptable	high durability	high durability	acceptable
Weight		40.7 psf	87.5 psf	138.8 psf	41.8 psf
Cost per square foot		$15.80	$13.61	$18.90	$21.90
Future Design Considerations					
Lateral System Options	Concrete Shear Walls	No	No	No	Yes
	Concrete Moment Frame	Yes	Yes	Yes	No
	Steel Moment Frame	Yes	No	No	Yes
	Steel Braced Frame	No	No	No	No
Advantages		-Lightweight -Relatively inexpensive	-Least Expensive System -Small slab depth -No interior beams	-Small total depth	-Lightweight -More layout flexibility
Disadvantages		-Not a typical system for new construction	-None	-Heaviest System -Relatively Expensive	-Most expensive system -Largest total system depth
Future Use?		N/A	Yes	No	Yes

Tech Report 3 | System Comparisons | Samantha deVries

Depth

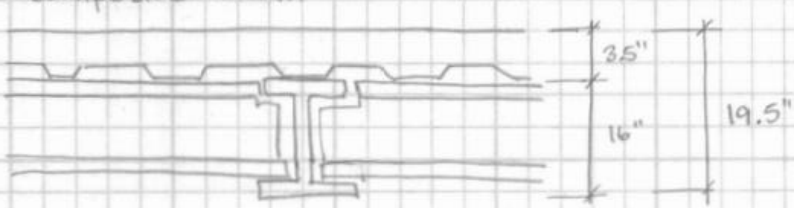
Exisiting System



Two-Way Slab

One Way Slab

Non-Composite Beam



Tech Report 3 | System Comparison | Samantha deVries

Weight

Existing System

Deck: 37 psf
Joist: 8 plf / 4' ft spacing = 2 psf
Girders: 33plf / 20' spacing = 1.65 psf

Total Weight = 40.7 psf

Two-Way Slab

Slab: 150 pcf × (7"/12) = 87.5 psf
No Drop Panels for now

Total Weight = 87.5 psf

One-Way Slab

Slab: 150 pcf × (11"/12) = 137.5 psf
Girders: 150 pcf × (1"/12) × 2'/20' = 1.25 psf

Total Weight = 138.8 psf

Non-Composite Steel

Slab: 37 psf
Beam: 14 plf / 4' sp. = 3.5 psf
Girder: 26 plf / 20' sp. = 1.3 psf

Total Weight = 41.8 psf

Tech Report 3 | System Comparisons | Samantha deVries

Cost (Using RS Means Square Foot Assemblies 2014)

Existing System

Steel Joists, Beams, & Slab on Columns
Total Load = 115 psf, use 20×20 bay w/ 119 psf

Cost: $15.80 / sq. ft.

Two-Way Slab

Cast in place flat plate, 20×20 bay
7" slab thickness

Cost: $13.61 / sq. ft.

One-Way Slab

Cast in place Beam & Slab, one way, 20×25 bay
total load = 215

Cost: $18.90 / sq. ft.

Non-Composite Beam

Wide flange, composite deck & slab, 20×20 bay
total load = 126 psf

Cost: $21.90 / sq. ft

Appendix

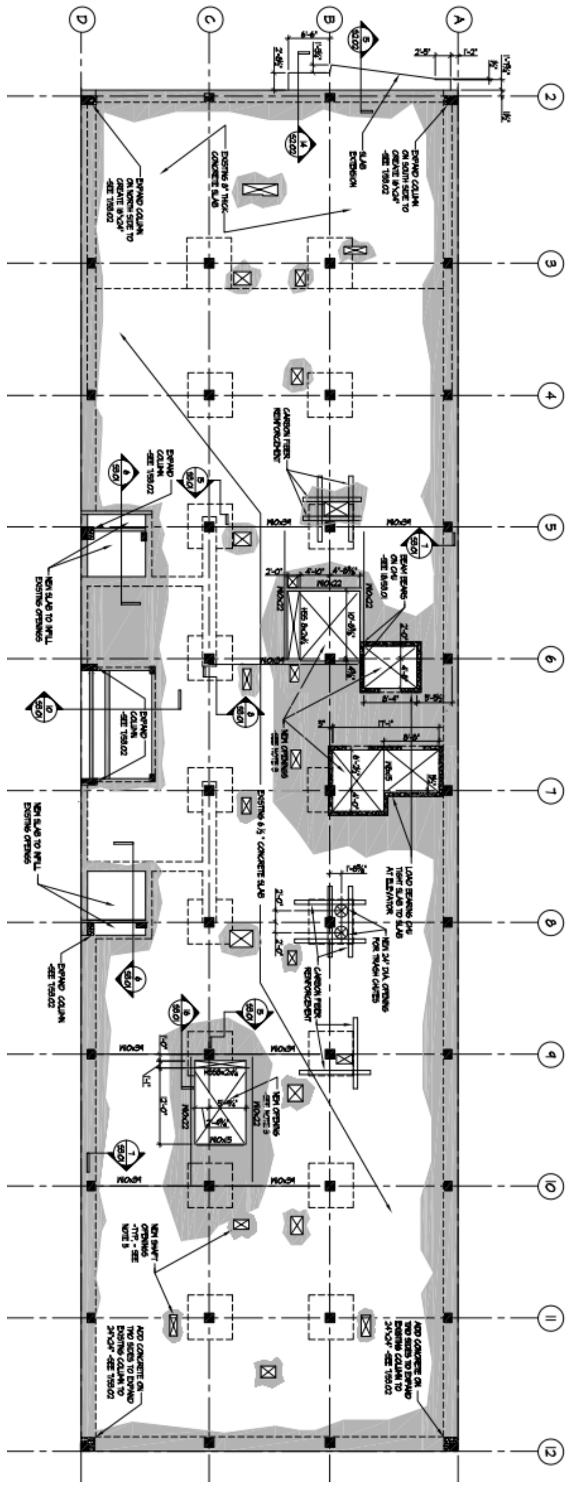


Figure 10: Typical Floor Plan in Original Building, S1.04

Note: Building Drawing sets and images pulled from those sets which appear in this report are courtesy of Rathgeber Goss Association and Bonstra Haresign Architects.