440 FIRST STREET, NW
WASHIGNTON, D.C.

TECHNICAL REPORT III

YEMI OSITELU
STRUCTURAL OPTION
ADVISOR | ALY SAID
16 OCTOBER 2015

Letter of Transmittal

October 15, 2015

Aly Said
Structural Thesis Advisor
The Pennsylvania State University
aly.said@engr.psu.edu

Dear Dr. Said,

The following technical report fulfills the requirements specified in the structural Technical Report II assigned by the faculty for senior thesis.

Technical Report III includes;

I. A detailed structural analysis of the loads used in the construction and renovation
II. Member spot checks for gravity loads in a typical bay
III. A detailed analysis of alternative framing systems which are;
    - Reinforced two-way flat-slab with edge beam
    - Structural steel framing w/ composite joists
    - Non-composite wide flange steel frame on composite deck

IV. Comparison of the existing and the alternate framing systems

Thank you for reviewing this report. I will kindly appreciate your feedback.

Sincerely,

Yemi A. Ositelu.

Enclosed: Technical Report III

EXECUTIVE SUMMARY

440 First Street is a mixed use building located in Washington, D.C. The existing 8-story building, constructed in the early 80's began renovation in 2012 and was completed in 2013. Three stories were added to the building, including a penthouse, resulting in a 20.6 foot increase in building height and a total gross square footage of about 142000 GSF. The new 10-story architectural design provided a seamless transformation of the existing building into a more modern, state-of-the-art building, well on its way to a platinum LEED certification.

The existing building, floors 1 to 7, comprises of a frame assembly of cast-in-place concrete structural slabs and column, with low story heights. The foundation system is mainly supported by the spread footings. The new, additional framing (8th floor and above) uses composite framing, with wide flange steel shapes used in the majority of the added structural system.

Building codes and design standards typically used in the project include the ASCE and the IBC, with gravity, lateral, and seismic loads all considered.

This report will cover the codes, design loads, existing framing, framing renovations and additional framing in more detail and in a wider perspective.

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440 FIRST STREET

YEMI A. OSITELU | STRUCTURAL OPTION
ADVISOR: DR. ALY SAID

GENERAL DESCRIPTION

LOCATION	WASHINGTON, D.C.
OCCUPANCY	OFFICE/ RETAIL
SIZE	141,929 SQUARE FT.
NUMBER OF STORIES	11 (ABOVE GRADE)
ACTUAL COST INFO.	$20,000,000 (RENO.)

PROJECT TEAM

NEW CONSTRUCTION

OWNER	FP FIRST STREET, LLC
GENERAL CONTRACTOR	SIGAL CONSTRUCTION
ARCHITECT	FOX ARCHITECTS
CIVIL ENGINEER	VIKA
STRUCTURAL ENGINEER	RGA
MEP ENGINEER	VANDERWEIL
LIGHTING CONSULTANT	C.M KLING & ASSOC.
SPECS. WRITER	BETHEL SPECS.
LEED CONSULTANT	LORAX
CODE CONSULTANT	AON RISK SOLUTIONS

EXISTING CONSTRUCTION

ARCHITECT	VLASTMIL KOUBEK, AIA
STRUCTURAL ENGINEER	BASKAM & JURCZYK
MECHANICAL & ELECTRICAL	THE OFFICE OF LEE KENDRICK

ARCHITECTURE

440 First Street, NW, is located between D and E Streets in downtown Washington, D.C. The existing 8-story building was constructed in 1982 and renovation was initiated in 2012. It has 10 stories + a mechanical penthouse, and there are two existing below grade parking garages, which were repaired and utilized as a valet parking facility. The new façade is a combined glass-and-metal curtain wall system, which allows for outstanding views and more importantly, natural daylighting.

STRUCTURAL SYSTEM

FRAMING SYSTEM

EXISTING	*Cast-in-place concrete with two-way structural concrete slabs and reinforced concrete columns and beams.*
NEW	*Composite steel framing with 5 1/4" slabs*

LATERAL SYSTEM

EXISTING	*Slab-Column Concrete Frames*
NEW	*Steel Moment Frames*

FOUNDATION

Walls and columns are supported by spread footings.

MECHANICAL SYSTEM

During the renovation of 440 First Street, the primary mechanical (DOAS) systems were replaced and resulted in a 25% reduction in energy usage. It consists of 3 mechanical rooms housed in the penthouse and 2 cooling towers on the penthouse roof.

Openings were created in the steel beams and girders

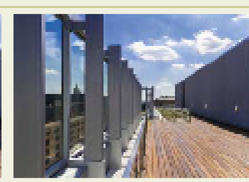
SUSTAINABILITY

- Majority of the building 's structural elements will be reused
- Green Roof will have local plants that require minimal watering and also reduces storm water overflow and minimizes "heat island" effect
- Recycled materials are used and are obtained regionally
- The building has achieved LEED Platinum Certification

LIGHTING/ELECTRICAL SYSYTEM

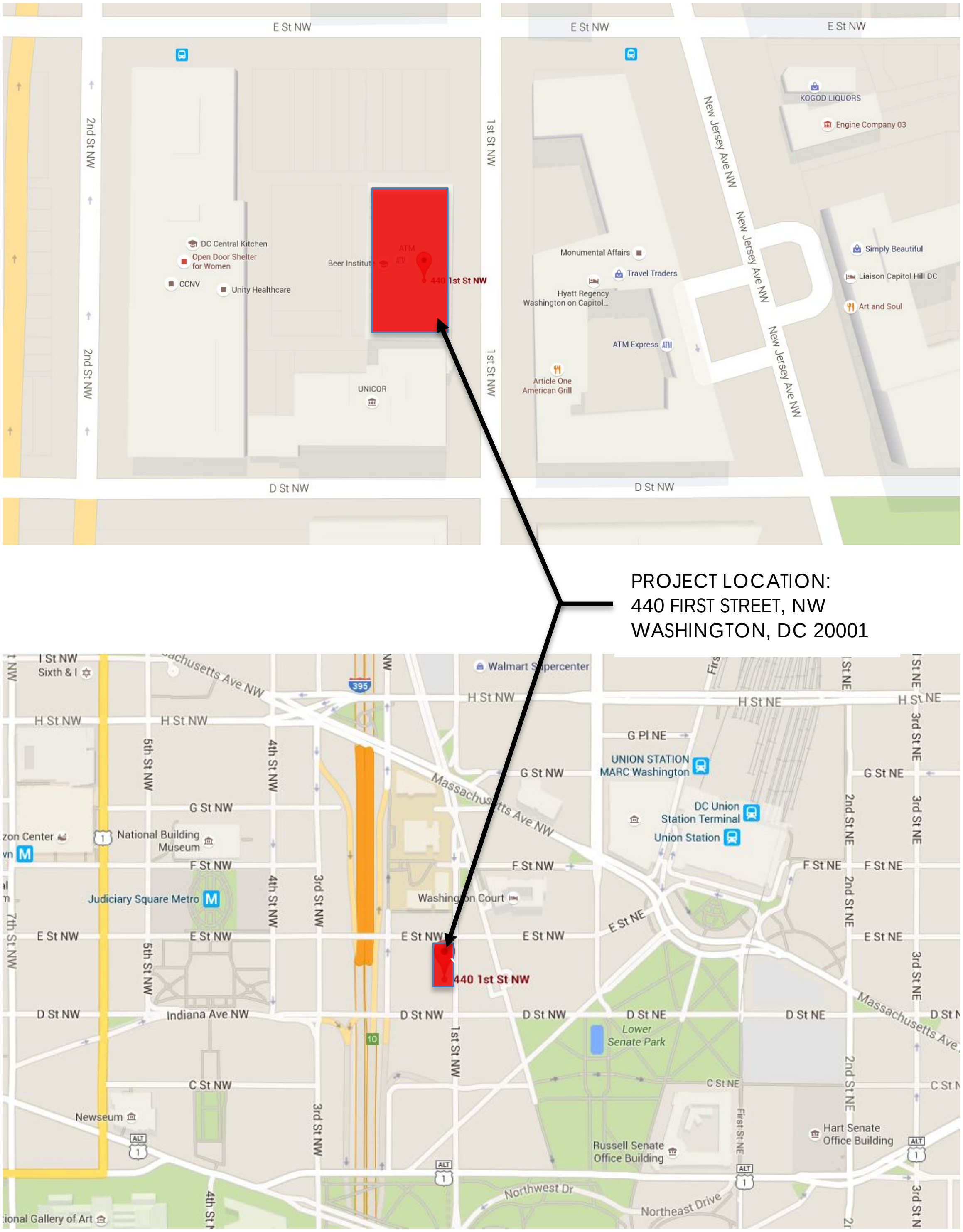
The curtain wall and the many windows in the façade provide the building with natural daylighting, improving energy efficiency.

The interiors are well lit with LED fixtures and other various energy efficient light fixtures



ALL IMAGES COURTESY OF JEFF GOLDBERG OF ESTO PHOTOGRAPHY FOR FOX ARCHITECTS

SITE AND LOCATION PLAN



DOCUMENTS USED DURING THE PREPARATION OF REPORT

The following is a list of the structural codes and design standards used in the structural analysis of 440 First Street, NW, Washington, D.C.

I. **International Code Council**
   - International Building Code 2006

II. **American Society of Civil Engineers**
   - ASCE 7-05 &10: Minimum Design Loads for Buildings and Other Structures

III. **American Concrete Institute**
   - ACI 318-11: Building Code Requirements for Structural Concrete

IV. **American Institute of Steel Construction**
   - AISC 14th Edition: Steel Construction Manual

V. **Vulcraft Deck Catalog**

VI. **First Edition, Standard Specification for Composite Steel Joists**

VII. **Reinforced Concrete Mechanics and Design Textbook**

VIII. **Previous AE Course Notes**

GRAVITY LOADS

Roof Loads

This section includes the calculations of the penthouse and main roof loads; dead, roof live and snow loads.

Figure 3 and Figure 4 show cross-sections through the main roof and penthouse roof respectively.

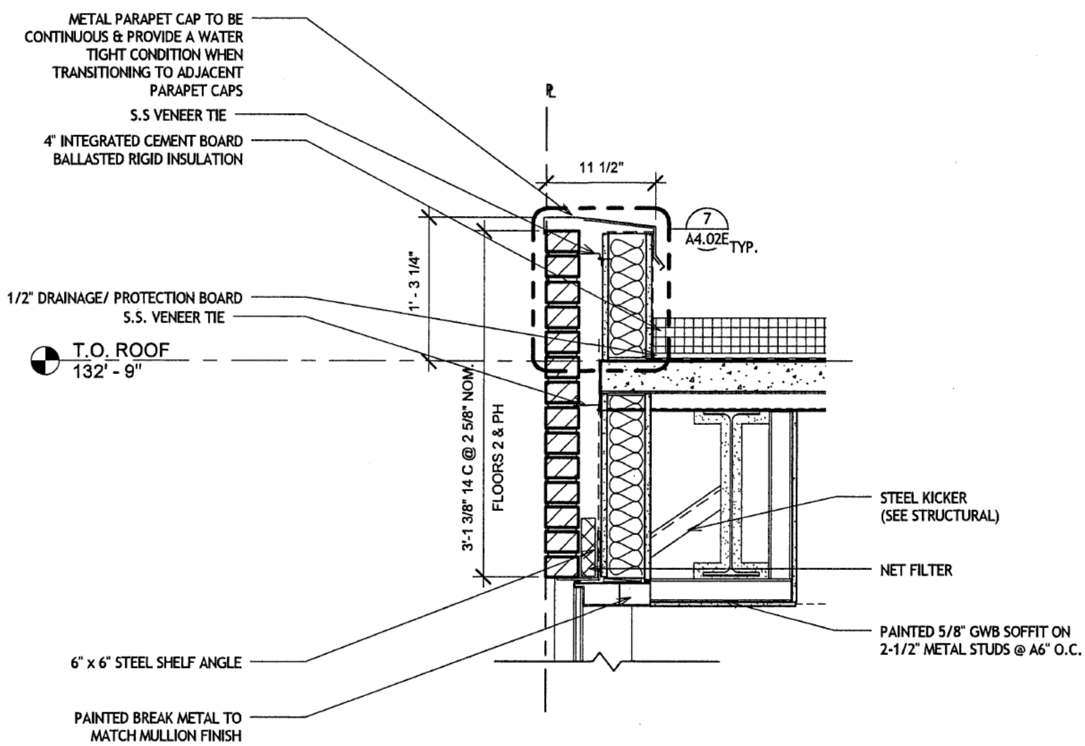


Figure 3: Section Detail At Main Roof Level

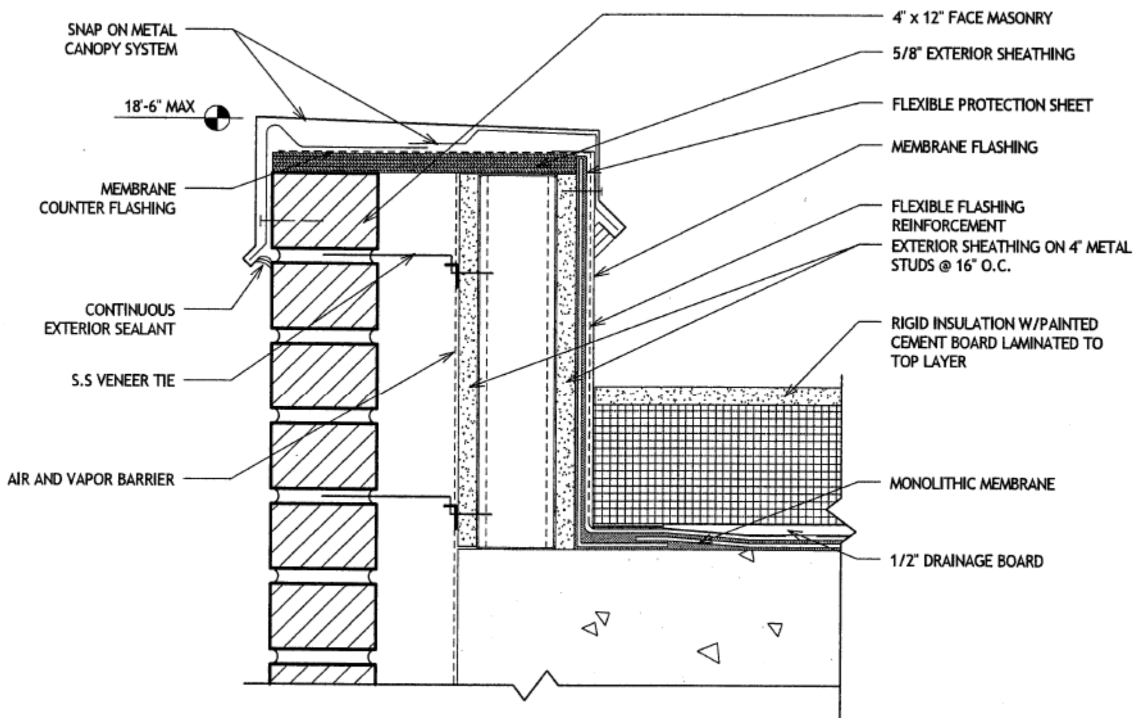


Figure 4: Section Detail At Penthouse Roof Level

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GRAVITY LOADS [ROOF]

TYPICAL ROOF BAY LOADING (DEAD)

PENTHOUSE ROOF	LOAD [PSF]
JOIST/BEAM ALLOWANCE	10
ROOFING SYSTEM	7
ROOF DECKING	10
	27

MAIN/PENTHOUSE FLOOR ROOF	LOAD [PSF]
3¼" LW CONC OVER 2" DEEP METAL DECK	42
JOIST/BEAM ALLOWANCE	10
4" RIGID INSULATION	3
CEILING	5
MEP	15
SPRINKLERS	3
ROOF TOP CONCRETE PAVERS	25
	103

TYPICAL ROOF BAY LOADING (LIVE)

PENTHOUSE ROOF — 30 PSF [DESIGN VALUE]
CODE MINIMUM IS (20 PSF) AS PER ASCE 7-05 TABLE 4-1
FOR ROOFS, ORDINARY FLAT

MAIN/PENTHOUSE FLOOR ROOF — 100 PSF [DESIGN VALUE]
CODE MINIMUM IS (100 PSF) AS PER ASCE 7-05 TABLE 4-1
FOR ROOFS USED FOR ROOF GARDENS OR ASSEMBLY PURPOSES

NOTE: SHEET S0-01 REQUIRES THAT SNOW LOAD SHOULD
BE USED FOR AREAS LARGER THAN 30 PSF

TYPICAL ROOF BAY LOADING (SNOW)

AS PER ASCE 7-05 CHAPTER 7, SECTION 7.3

$P_f = 0.7 C_e C_t I P_g$ WHERE,

$P_g = 25$ PSF

$C_e = 1.0$

$C_t = 1.0$

$I = 1.0$

$P_f = 0.7(1.0)(1.0)(1.0)(25) = 17.5$ PSF

MINIMUM P_f WHERE P_g EXCEEDS 20 PSF = 20 I

$\therefore P_f = 20 \times 1.0 =$ 20 PSF [DESIGN SNOW LOAD]

NOTE: DESIGN VALUE DOES NOT EXCEED 30 PSF, THEREFORE
$P_f = 20$ PSF SHOULD BE USED

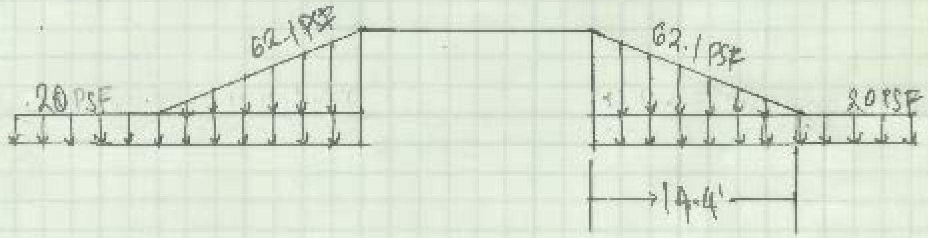
SNOW DRIFT [SECTION 7.7 - DRIFTS ON LOWER ROOFS]

$\gamma = 0.13 p_g + 14$ WHERE $p_g = 25$ PSF ; $0.13(25) + 14 = 17.25$ PCF
$h_b = P_s / \gamma$; WHERE $P_s = C_s P_f = 1.0 \times 20 = 20$; $h_b = 20/17.25 = 1.16$
$h_c = 18.5'$ $\therefore$ $h_c / h_b = \frac{18.5}{1.16} > 0.2$ [DRIFT REQD]

$l_u = 115'$ (WINDWARD); $h_d = 3.6$ [FIG 7-9] (USE LARGER)
$l_u = 55'$ (LEEWARD); $h_d = 2.5$ [FIG 7-9]

$h_d < h_c = 18.5$ $\therefore$ $W = 4 \times 3.6 = 14.4'$

$P_d = h_d \gamma = 3.6(17.25) = \boxed{62.1 \text{ PSF}}$



GRAVITY LOADS

Floor Loads

This section includes calculations of dead and live loads for the floors of the original cast-in-place concrete design and the new addition.

Figure 5 shows a section through a typical cast-in-place concrete slab in the existing building, and Figure 6 shows a section through a typical new floor.

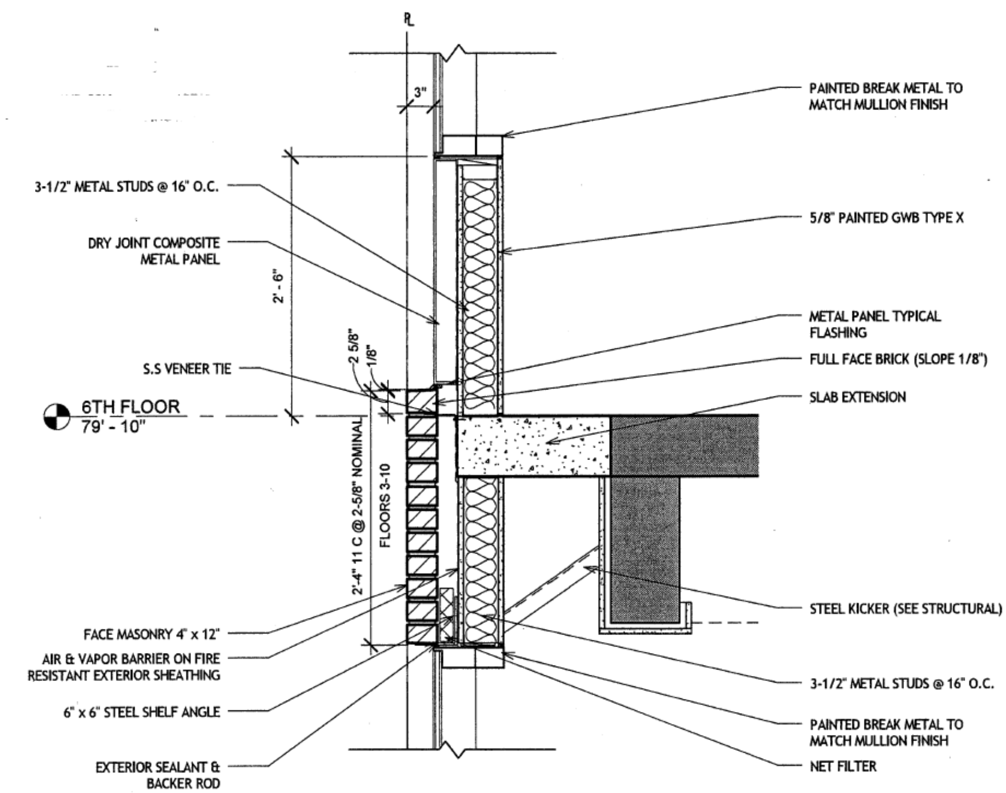


Figure 5: Section Detail Through Typical Existing Floor

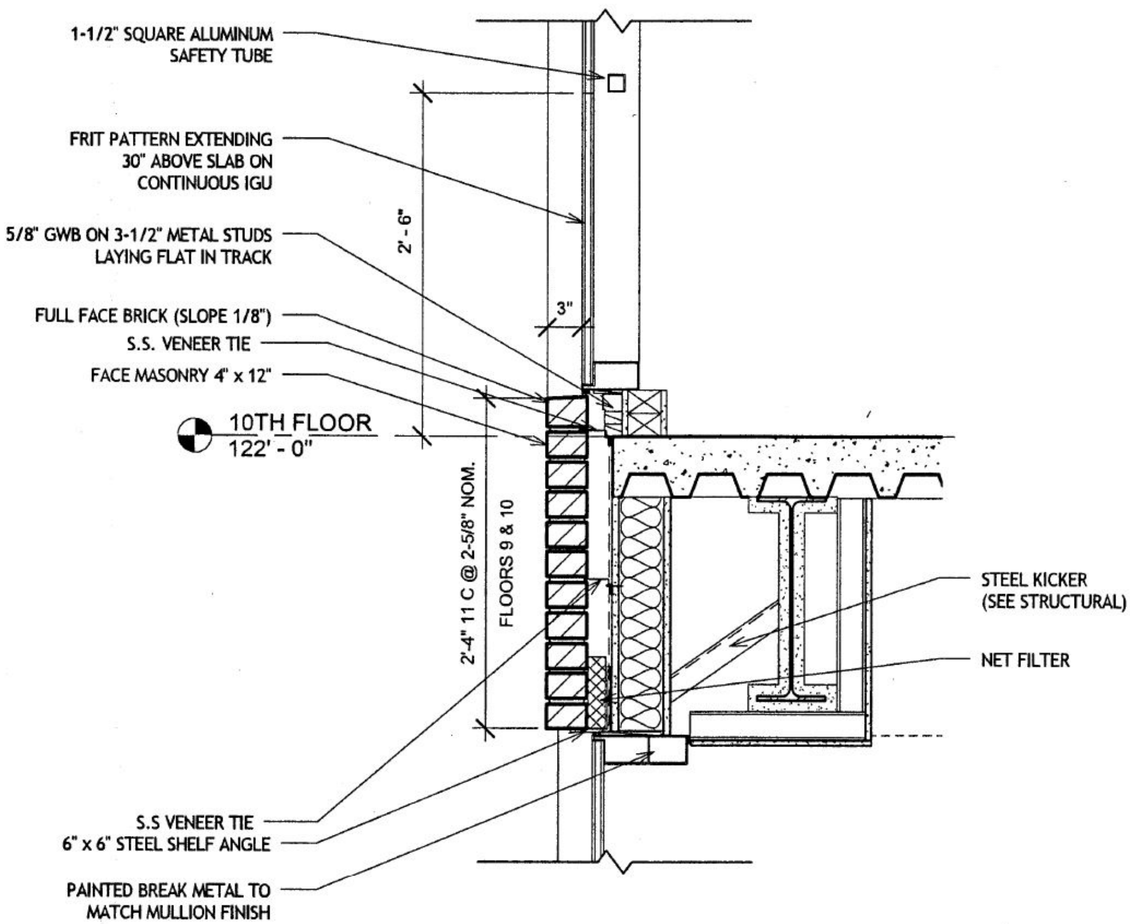


Figure 6: Section Detail Through Typical New Floor

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GRAVITY LOADS [FLOOR]

TYPICAL FLOOR BAY LOADING (DEAD)

CAST-IN-PLACE CONCRETE FLOOR	LOAD (PSF)
CONCRETE – 7" x 145/12	85
9½" x 145/12	115
CEILING	5
MEP	15
SPRINKLERS	3
TOTAL LOAD {7" SLAB} =	108
TOTAL LOAD {9½" SLAB} =	138 ← Controls

STRUCTURAL STEEL FRAMED FLOORS	LOAD (PSF)
3¼" LW CONC OVER 2" DEEP METAL DECK	42
BEAM/GIRDER ALLOWANCE	15
CEILING	5
MEP	15
SPRINKLERS	3
	80

TYPICAL FLOOR BAY LOADING (LIVE)

LIVE LOAD REDUCTION APPLIED AS PER ASCE 7-05

AREA	LOAD (PSF)	CODE MINIMUM
OFFICE + PARTITIONS	100	80
LOBBIES/STAIRS/EXITS	100	100
PENTHOUSE FLOOR	100	100
CORRIDORS ABOVE 1ST FLOOR	80	40
PARKING	50	40

EXTERIOR WALL LOADS

This section includes calculations of the exterior wall loads.

Figure 7 shows a cross-section of typical exterior wall detail, and Figure 8 shows a cross-section through the curtain wall on the east façade of the building.

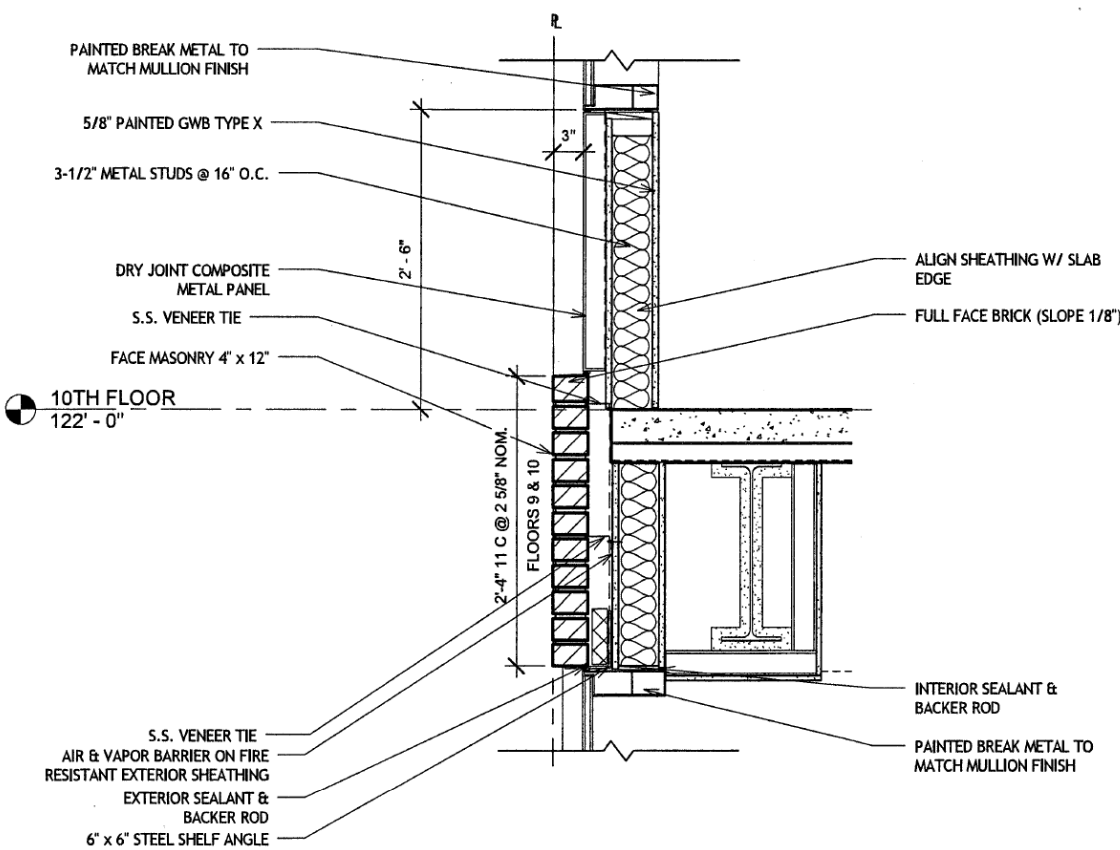

Figure 7: Section Detail Of A Typical Exterior Wall

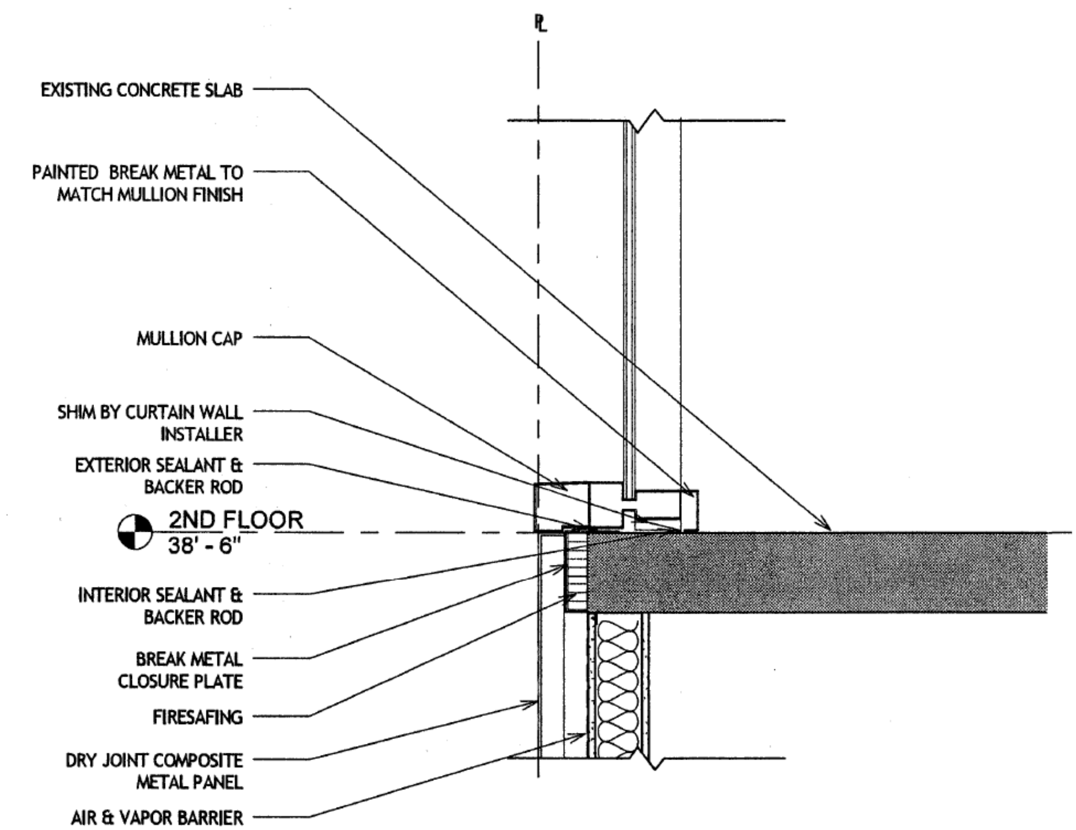

Figure 8: Section Detail Through The Curtain Wall

Notes

I. Weights of building materials shown in cross-section were assumed using typical weights of materials.
II. The north, south and west façades consist of windows as well as masonry, but the greatest wall load will occur through a fully face masonry section.

Load Path

Load is typically carried by the composite deck. The deck transfers load to the steel wide flange members and concrete beams, which then transfers the load to the steel/concrete columns. The load is ultimately transferred to the foundation

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EXTERIOR WALL LOADS

TYPICAL EXT MASONRY WALL DEAD LOAD

GYPSUM WALL BOARD – 5/8" × 4 PSF/in × 10.25' = 25.6 PLF
SEMI-RIGID INSULATION – 1.0 PSF × 10.25' = 10.25 PLF
FACE MASONRY – 39 PSF × 10.25' = 400 PLF
≈ 436 PLF

TYPICAL CURTAIN WALL DEAD LOAD

CURTAIN WALL SYSTEM – 10 PSF × 10.25' = 102.5 PLF

LATERAL LOADS

Wind Loads

This section includes wind load calculations for 440 First Street in the two orthogonal directions, according to ASCE 7-05: Chapter 6.5; Method 2.

Microsoft Excel was used in programming equations for optimum efficiency.

Notes

I. C_p values were calculated through interpolation of values in Figure 6.6 of the ASCE 7-05: Chapter 6.5

II. The velocity pressure exposure coefficients for the building at the different heights are shown in Table 1 below

   - K_z values are obtained through interpolation of values in Table 6-3 of ASCE 7-05: Chapter 6, using Exposure B – Case 2.

TABLE 1: Velocity Pressure Exposure Coefficients

Height (ft)	Kz	**qz or qh**
15	0.57	**10.05**
25.33	0.66	**11.63**
35.67	0.73	**12.87**
46	0.79	**13.92**
56.33	0.84	**14.81**
66.67	0.88	**15.51**
77	0.92	**16.22**
87.75	0.95	**16.74**
98.5	0.99	**17.45**
109.25	1.01	**17.8**
118.5	1.04	**18.33**
127.25	1.06	**18.68**

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LATERAL LOAD - WIND

ASCE 7-05: CHAPTER 6.5; METHOD 2 - ANALYTICAL DESIGN PROCEDURE FROM SECTION 6.5.3

WIND FORCE DETERMINATION - [N-S DIRECTION]

1. Building Information

$B = 87'$ $L = 160.25'$ $h = 118.5'$

2. Basic Wind Speed (V) — 90 MPH [FIG 6-1]

3. Directionality factor (K_d) — 0.85 [TABLE 6-4]

4. Determining the Importance Factor (I)

Occupancy Category — II [TABLE 1-1]
Importance Factor — 1 [TABLE 6-1]

5. Exposure Category — B [S0-01 OF DRAWINGS]

6. VELOCITY PRESSURE EXPOSURE COEFFICIENT

+ Using EXPOSURE B ; CASE 2 FOR MWFRS
+ K_z values obtained through interpolation
+ For Breakdown, See TABLE 1

7. TOPOGRAPHIC FACTOR (K_{zt}) - 1.0 [S0-01 OF DRAWINGS]

8. GUST EFFECT FACTOR (G_f) — 0.85 [SEC 6.5.8.1]

9. ENCLOSURE CLASSIFICATION — Enclosed [SEC 6.5.9]

10. Internal Pressure Coefficient

$GC_{pi} = +/- 0.18$ [FIG. 6-5]

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11. EXTERNAL PRESSURE COEFFICIENTS

Windward Wall	$C_p = 0.8$	[FIG 6-6]
Leeward Wall	$C_p = -0.325$	[FIG 6-6]
Side Wall	$C_p = -0.7$	[FIG 6.6]
Roof (0' to 59.25')	$C_p = -0.90$	[FIG 6.6]
Roof (59.25 to 118.5')	$C_p = -0.80$	[FIG 6.6]
Roof (118.5' to 180.25)	$C_p = -0.60$	[FIG 6.6]

TABLE 2: Wind Pressures in the North-South Direction

Wind Pressure Chart (N-S)								
Location	z	qz or qh	Cp	Gf	Gcpi	qiGCpi	Net Pressure (PSF)	
							qzGfCp-qi(+Gcpi)	qzGfCp-qi(-Gcpi)
Windward	15	10.05	0.8	0.85	0.18	1.809	5.03	**8.64**
	25.33	11.63	0.8	0.85	0.18	2.0934	5.82	**10.00**
	35.67	12.87	0.8	0.85	0.18	2.3166	6.44	**11.07**
	46	13.92	0.8	0.85	0.18	2.5056	6.96	**11.97**
	56.33	14.81	0.8	0.85	0.18	2.6658	7.41	**12.74**
	66.67	15.51	0.8	0.85	0.18	2.7918	7.76	**13.34**
	77	16.22	0.8	0.85	0.18	2.9196	8.11	**13.95**
	87.75	16.74	0.8	0.85	0.18	3.0132	8.37	**14.40**
	98.5	17.45	0.8	0.85	0.18	3.141	8.73	**15.01**
	109.25	17.8	0.8	0.85	0.18	3.204	8.90	**15.31**
	118.5	18.33	0.8	0.85	0.18	3.2994	9.17	**15.76**
Leeward	All	18.68	-0.325	0.85	0.18	3.3624	**-8.52**	-1.80
Side	All	18.68	-0.7	0.85	0.18	3.3624	**-14.48**	-7.75
Roof (0 to 59.25)	118.5	18.68	-0.98	0.85	0.18	3.3624	**-18.92**	-12.20
Roof (59.25 to 118.5)	118.5	18.68	-0.8	0.85	0.18	3.3624	**-16.06**	-9.34
Roof (118.5 to 160.25)	118.5	18.68	-0.6	0.85	0.18	3.3624	**-12.89**	-6.16
Low Parapet WW	110.5	17.98			1.5	26.97		**26.97**
Low Parapet LW	110.5	17.98			-1.0	-17.98		**-17.98**
High Parapet WW	127.25	18.68			1.5	28.02		**28.02**
High Parapet LW	127.25	18.68			-1.0	-18.68		**-18.68**

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

WIND FORCE DETERMINATION [E-W DIRECTION]

1. Building Information

$B = 160.25'$ $L = 87'$ $h = 118.5'$

2. Basic Wind Speed (V) — 90 MPH [FIG 6-1]
3. Directionality Factor (K_d) — 0.85 [TABLE 6-4]
4. Determining the Importance Factor (I)
   - Occupancy Category - II [TABLE 1-1]
   - Importance Factor - 1 [TABLE 6-1]
5. Exposure Category — B [S0-01 OF DRAWINGS]
6. Velocity Pressure Exposure Coefficient

   As Calculated Previously (Shown in TABLE 1)
7. Topographic Factor (K_{zt}) - 1 [S0-01 OF DRAWINGS]
8. Gust Effect Factor (G) — 0.85 [SEC 6.5.8.1]
9. Enclosure Classification - Enclosed [SEC 6.5.9]
10. Internal Pressure Coefficient

    $GC_{pi} = +/-0.18$ [FIG 6.5]
11. External Pressure Coefficient

Windward Wall	$C_p = 0.8$	[FIG 6-6]
Leeward Wall	$C_p = -0.5$	[FIG 6-6]
Side Wall	$C_p = -0.7$	[FIG 6-6]
Roof (0-59.25')	$C_p = -1.04$	[FIG 6-6]
Roof (59.25-87')	$C_p = -0.7$	[FIG 6-6]

Base shear calculations

The base shear was calculated for the two orthogonal directions and determined by multiplying the story height by the net wind pressure at that level and by the width of the building perpendicular to the direction of the wind.

The total base shear in both orthogonal directions are shown in Table 3.

Width (N-S) – 87’

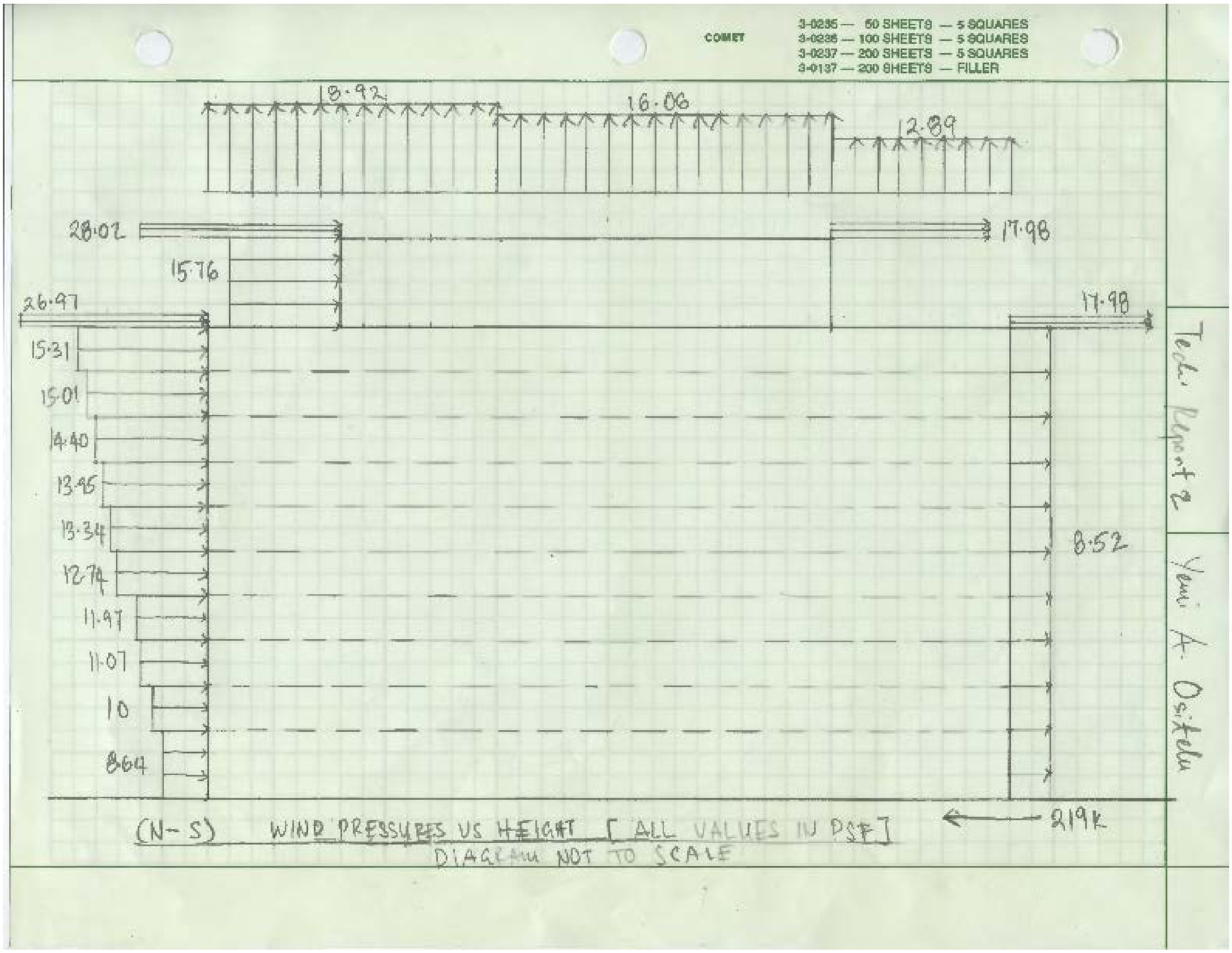
Width (E-W) – 160.25

TABLE 3: Base Shear Calculations

Story Height (ft)	Story Trib. Height x Net Pressure x Trib. Width	
	Wind (N-S)	**Wind (E-W)**
15	**22.39**	**41.25**
25.33	**16.64**	**30.66**
35.67	**17.61**	**32.45**
46	**18.42**	**33.94**
56.33	**19.11**	**35.19**
66.67	**19.65**	**36.19**
77	**20.19**	**37.20**
87.75	**20.60**	**37.94**
98.5	**21.15**	**38.95**
109.25	**21.42**	**39.45**
127.25	**21.82**	**40.19**
Base Shear	**219.00**	**403.40**

TABLE 4: Wind Pressures in the East-West Direction

Wind Pressure Chart (E-W)								
Location	z	qz or qh	Cp	Gf	Gcpi	qiGCpi	Net Pressure (PSF)	
							qzGfCp-qi(+Gcpi)	qzGfCp-qi(-Gcpi)
Windward	15	10.05	0.8	0.85	0.18	1.81	5.03	**8.64**
	25.33	11.63	0.8	0.85	0.18	2.09	5.82	**10.00**
	35.67	12.87	0.8	0.85	0.18	2.32	6.44	**11.07**
	46	13.92	0.8	0.85	0.18	2.51	6.96	**11.97**
	56.33	14.81	0.8	0.85	0.18	2.67	7.41	**12.74**
	66.67	15.51	0.8	0.85	0.18	2.79	7.76	**13.34**
	77	16.22	0.8	0.85	0.18	2.92	8.11	**13.95**
	87.75	16.74	0.8	0.85	0.18	3.01	8.37	**14.40**
	98.5	17.45	0.8	0.85	0.18	3.14	8.73	**15.01**
	109.25	17.8	0.8	0.85	0.18	3.20	8.90	**15.31**
	118.5	18.33	0.8	0.85	0.18	3.30	9.17	**15.76**
Leeward	All	18.68	-0.5	0.85	0.18	3.36	**-11.30**	-4.58
Side	All	18.68	-0.7	0.85	0.18	3.36	**-14.48**	-7.75
Roof (0 to 59.25)	118.5	18.68	-1.04	0.85	0.18	3.36	**-19.88**	-13.15
Roof (59.25 to 87)	118.5	18.68	-0.7	0.85	0.18	3.36	**-14.48**	-7.75
Low Parapet WW	110.5	17.98			1.5	26.97		**26.97**
Low Parapet LW	110.5	17.98			-1.0	-17.98		**-17.98**
High Parapet WW	127.25	18.68			1.5	28.02		**28.02**
High Parapet LW	127.25	18.68			-1.0	-18.68		**-18.68**


18.92
16.06
12.89
28.02
17.98
15.76
26.97
17.98
15.31
15.01
14.40
13.95
13.34
12.74
11.97
11.07
10
8.64
8.52
219k
(N-S) WIND PRESSURES VS HEIGHT [ALL VALUES IN PSF]
DIAGRAM NOT TO SCALE
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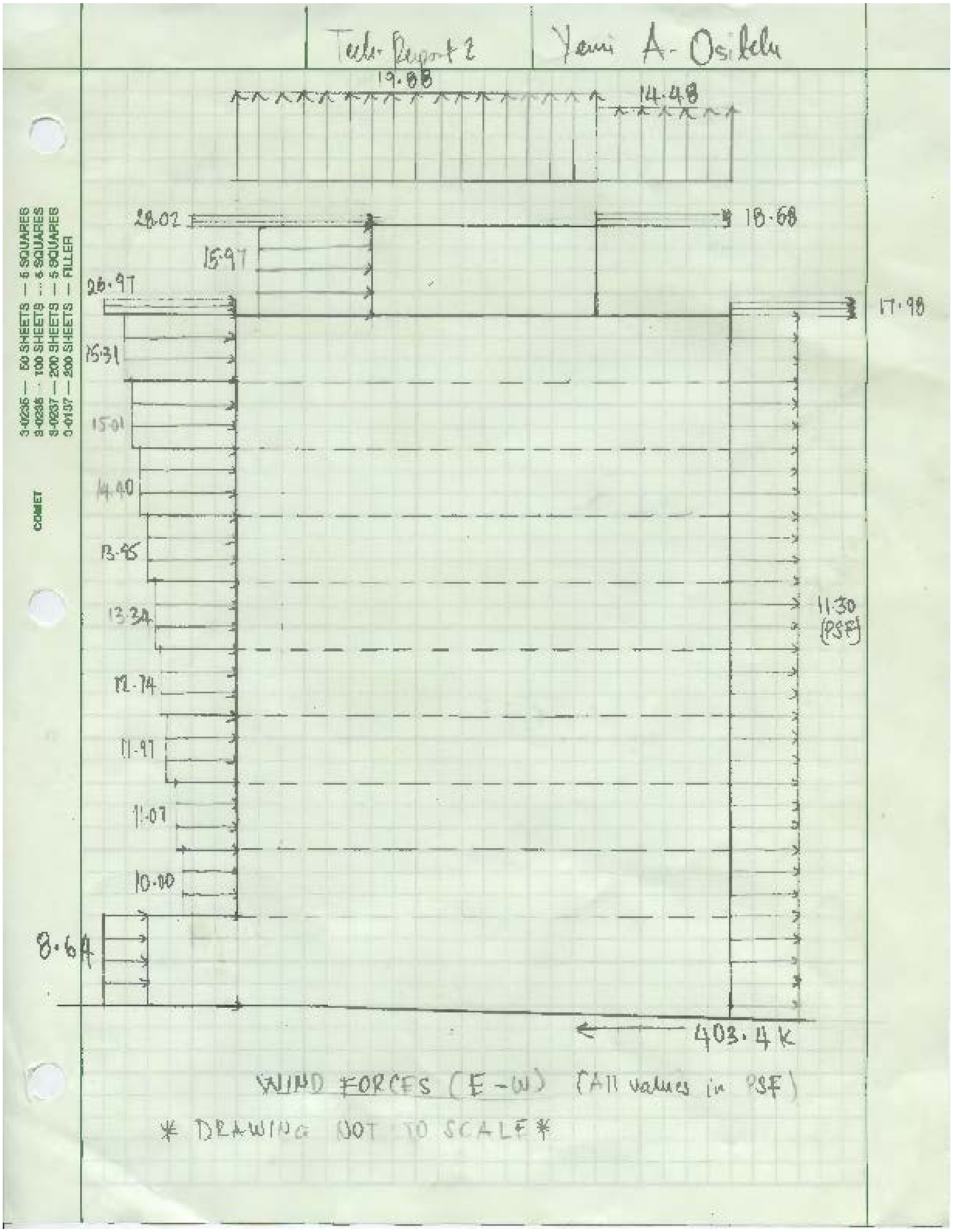
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19.88
14.48
28.02
18.68
15.97
26.97
17.98
15.31
15.01
14.40
13.85
13.34
11.30 (PSF)
12.74
11.97
11.07
10.00
8.64
403.4K
WIND FORCES (E-W) (All values in PSF)
* DRAWING NOT TO SCALE *

LATERAL LOAD

Seismic Loads

This sections outlines the seismic load calculations, in accordance to ASCE 7-05: Chapter 11 and 12.

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8-0235 — 50 SHEETS — 5 SQUARES
8-0236 — 100 SHEETS — 5 SQUARES
8-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

LATERAL LOAD – SEISMIC

AS PER ASCE 7-05, CHAP. 11 & 12 | SEISMIC DESIGN REQUIREMENTS FOR BUILDING STRUCTURES

1. EXEMPTIONS [SEC 11.1.2]
   BUILDING IS NOT EXEMPT

2. SITE CLASSIFICATION
   C [OBTAINED FROM SQ-01]

3. MAPPED ACCELERATION PARAMETERS [SEC 11.4.1, FIG 22.1 TO 22.6)
   $S_s = 0.154$
   $S_1 = 0.05$ [OBTAINED FROM S0-01]

4. SPECTRAL RESPONSE COEFFICIENTS CALC
   TABLE 11.4-1, $S_s \le 0.25$, $F_a = 1.2$
   TABLE 11.4-2, $S_1 \le 0.1$, $F_v = 1.7$

   $S_{MS} = F_a S_s = 1.2(0.154) = 0.185g$, EQN 11.4-1
   $S_{M1} = F_v S_1 = 1.7(0.050) = 0.085g$, EQN 11.4-2

   $S_{DS} = 2/3\ S_{MS} = (2/3)(0.185) = 0.123g$, EQN 11.4-3
   $S_{D1} = 2/3\ S_{M1} = (2/3)(0.085) = 0.057g$, EQN 11.4-4

NOTE: S_{DS} AND S_{D1} VALUES MATCH DESIGN VALUES IN S0-01.

5. SEISMIC DESIGN CATEGORY [SEC 11.6, TABLE 11.6-1, 2]

   $S_{DS} < 0.167$
   $S_{D1} < 0.067$ $\Rightarrow$ $\boxed{\text{SEISMIC DESIGN CATEGORY A}}$

6. OCCUPANCY CATEGORY [SEISMIC USE GROUP]
   II

7. SEISMIC IMPORTANCE FACTOR
   IE = 1.0

8. SEISMIC ANALYSIS PROCEDURE [SEC 11.7]

   $F_x = 0.01 W_x$ [FROM EQN 11.7-1]

   NOTE: BUILDING CAN USE ABOVE FORMULA BECAUSE IT IS IN SEISMIC DESIGN CATEGORY A

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9. DETERMINE THE EFFECTIVE TOTAL SEISMIC WEIGHT

- DL + 20% SL [ON ROOF]
- DL [ON FLOORS]

STRUCTURAL STEEL FLOORS

$$W = (160.25)(87)(80) + 2(160.25 + 87)(539)$$
$$= 1381875.5 \text{ POUNDS}$$
$$= \underline{1382 \text{ KIPS}}$$

CAST-IN-PLACE CONCRETE FLOORS

$$W = (160.25)(87)(138 \text{ PSF}) + 2(160.25 + 87)(539)$$
$$= 2190497$$
$$= \underline{2190 \text{ KIPS}}$$

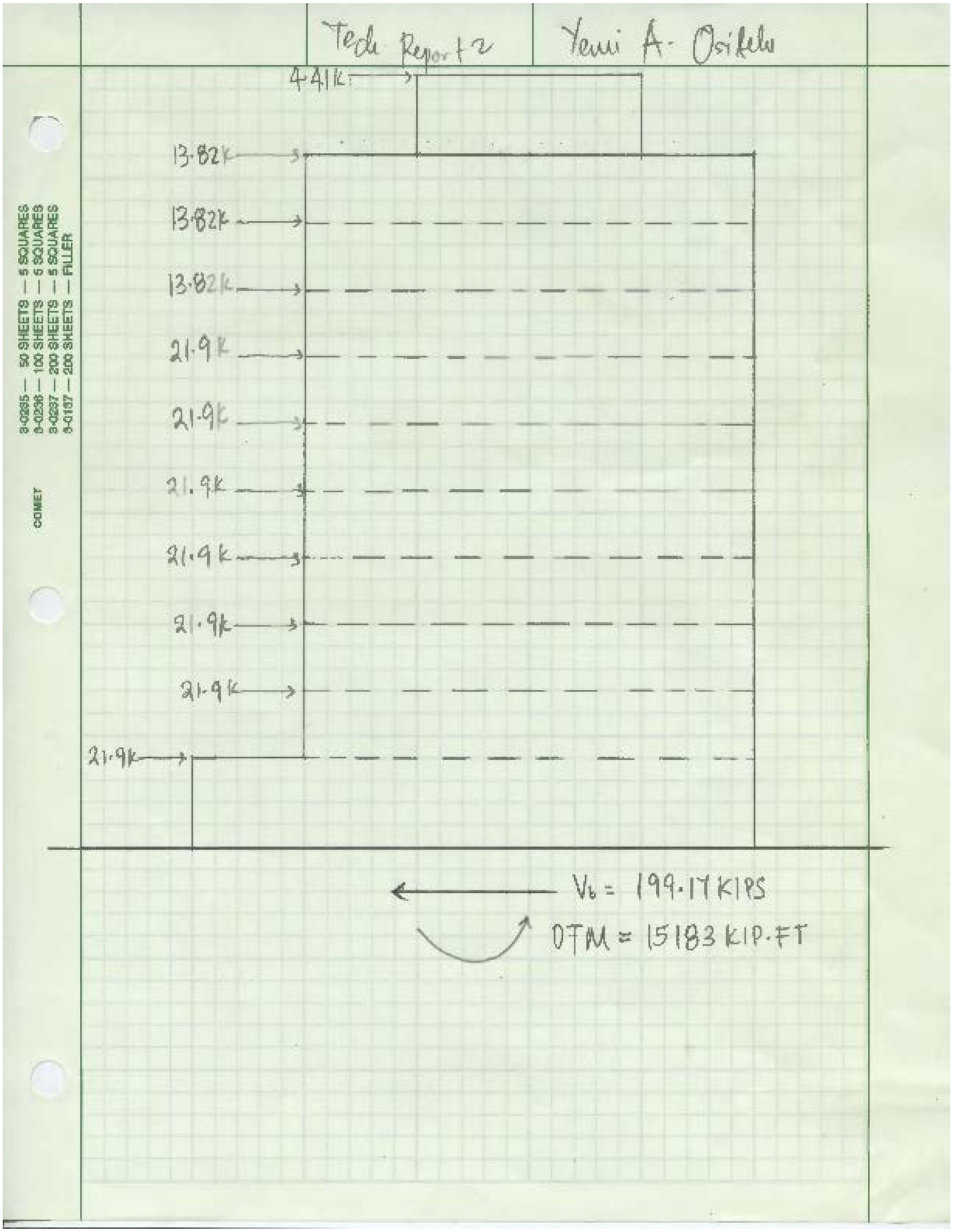
PENTHOUSE ROOF

$$W = (115.25)(54.8)(27 + 0.2(20)) + 2(115.25 + 54.8)(39 \times 18.5')$$
$$= 441168.85 \text{ POUNDS}$$
$$= \underline{441 \text{ KIPS}}$$

TOTAL LOAD:

$$W = 441 \text{ KIPS} + 7(2190) \text{ KIPS} + 3(1382) \text{ KIPS}$$

$$\boxed{W = 19917 \text{ KIPS}}$$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4.41k
13.82k
13.82k
13.82k
21.9k
21.9k
21.9k
21.9k
21.9k
21.9k
21.9k
Vb = 199.17 KIPS
OTM = 15183 KIP·FT

MEMBER SPOT CHECKS FOR GRAVITY LOADS

The members were analyzed for gravity loads in a typical floor bay shown in Figure 9. These evaluated members include; the infill beams, interior and exterior girders and, the interior and exterior column. After the analysis, it was determined that the composite framing system was adequate to carry the loads. Figure 10 shows an enlarged typical bay.

The structural slab was a 3 ¼" lightweight concrete on 2" x 18 gage metal, which is a total thickness of 5 ¼"

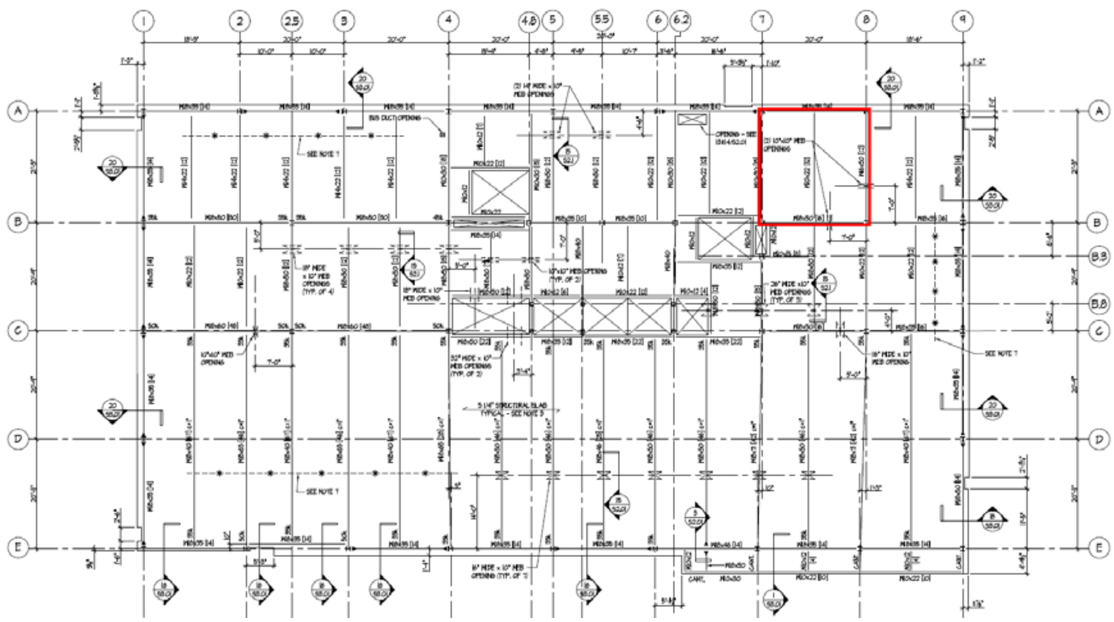

Figure 9: Typical Floor Bay

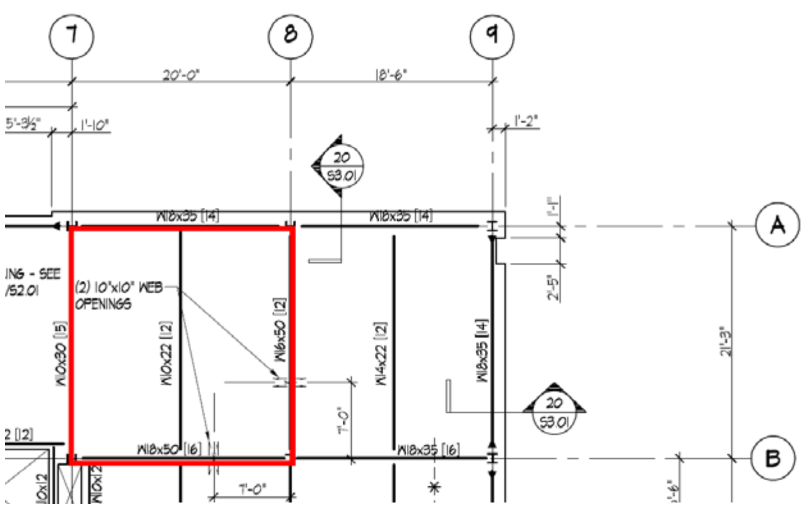

Figure 10: Enlarged Typical Floor Bay

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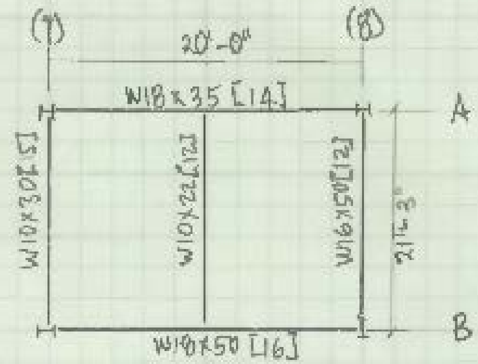
GRAVITY SPOT CHECKS [NEW ADDITION]

TYPICAL BAY (9TH AND 10TH FLOOR)



+ STRUCTURAL SLAB IS 3¼" L.W CONCRETE OVER 2" DEEP x 18GA GALVANIZED COMPOSITE METAL DECK (TOTAL TH. = 5¼") W/ 6x6 - W2.0 x 2.0 WWF

+ DEAD LOAD - 80 PSF [FROM GRAVITY CALCS.]
LIVE LOAD - 100 PSF [FRAMED FLOOR AREAS + PARTITIONS]

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DECKING - (2" x 18GA DECK, 5¼" LWC)

+ USING VULCRAFT TABLES FOR 2VLI, 18 GA +

1. DECK SPAN CHECK

→ SDI MAX UNSHORED CLEAR SPAN FOR 3/MORE SPANS – 12'-7"

12'-7" > 10'-0" ✓ GOOD!

2. CHECK SUPERIMPOSED LOAD

W_{LL} + SUPERIMPOSED DL ≤ SUPERIMPOSED LOAD

100. + (15 + 5 + 3) ≤ 205

123 ≤ 205 ✓ GOOD!

VERIFYING FLOOR BEAMS

W10x22 SPAN = 21'-3" SPACING = 10'-0" (TRIB. WIDTH)

DL = 42 + 15 + 5 + 3 + 2.2 PSF = 67.2 PSF

LL = 100 PSF

$W_u = 1.2DL + 1.6LL$

$= [1.2(67.2) + 1.6(100)] \times 10'$

$\approx 2.41\text{ KLF}$

$M_u = \frac{wl^2}{8} = \frac{2.41 \times 21.25^2}{8} = 136\text{ Kft}$

CHECK COMPOSITE STRENGTH

12. (3/4") DIAMETER STUDS EVENLY SPACED ALONG LENGTH

→ FOR 1 STUD/RIB → Q_n = 17.1 K [weak studs, f'c = 3 ksi (CONSERVATIVE)

∴ ΣQ_n = 6 x 17.1 = 102.6 KIPS

BEAM WT. = 22(21.25) + (12 x 10) = 588 POUNDS

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DETERMINE EFFECTIVE FLANGE WIDTH

$b_{eff} = 2 \times \min \begin{cases} 21.25 \times 12/8 \Rightarrow 32 \Rightarrow 64'' \\ 10 \times 12/2 \Rightarrow 60 \end{cases}$

$a = \dfrac{102.6}{0.85 \times 3.5 \times 64} = 0.54''$; $Y2 = 5.25 - \dfrac{0.54}{2} = 4.98$

$\left.\begin{array}{l} V_{s,max} = 6.49 \times 50 = 325k \\ V_{c,max} = 0.85 \times 3.5 \times 64 \times 3.25 = 619k \end{array}\right\} > \Sigma Q_n = 102.6K$

FROM TABLE 3-19; $\phi M_n = 149$ ftk [CONSERVATIVE]

$\phi M_n = 149$ ftk > 136 ftk ✓ GOOD!

CHECK UNSHORED STRENGTH

1.4DL and 1.2DL + 1.6(LL) → CONSTRUCTION

$DL = 42 + 22/10 = 44.2$ PSF

$LL = 20$ PSF (CONSTRUCTION LIVE LOAD)

$W_u = [1.4(44.2)(10)]/1000 = 0.62$ KLF

$W_u = [1.2(44.2) + 1.6(20)](10)/1000 = 0.85$ KLF

$\Rightarrow M_u = \dfrac{w_u l^2}{8} = \dfrac{0.85 \times 21.25^2}{8} = 48$ k.ft

$\phi M_p = 97.5$ kft > 48 k.ft ✓ GOOD!

CHECK DEFLECTIONS

· Wet Concrete Deflection

$W_{DL} = 44.2$ PSF

$\Delta_{DL(wet\ conc)} = \dfrac{5 W_{DL} L^4}{384 E I_x}$ where I_x

$= \dfrac{5 \times 0.0442 \times 10 \times 21.25^4 \times 1728}{384 \times 29000 \times 118} = 0.59''$

$\Delta_{TL} = \dfrac{L}{240} = \dfrac{21.25' \times 12}{240} = 1.06''$

$0.59'' < 1.06''$ ✓ GOOD!

Live Load Deflection

$W_{LL} = 100 PSF \times 10 = 1 KLF$

$I_{LB} = 250 in^4$ [CONSERVATIVE]

$$\Delta_{LL} = \frac{5 \times 1.0 \times 21.25^4 \times 1728}{384 \times 29000 \times 250} = 0.63''$$

$$\Delta_{LL,max} = \frac{L}{360} = \frac{21.25 \times 12}{360} = 0.71''$$

$0.63'' < 0.71''$ ✓ GOOD!

SUMMARY OF DESIGN

W10×22 WITH 12 EVENLY SPACED STUDS

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VERYFING FLOOR GIRDERS (INTERIOR)

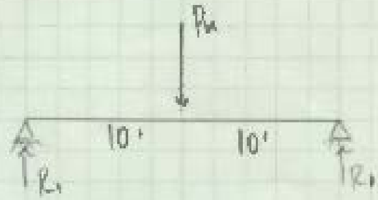
W18x50, SPAN: 20'-0" TRIB WIDTH = 21.25'

POINT LOAD CALC.

DL = 67.2 + 5%/135 = 69.6 PSF

$W_u = 1.2(69.6) + 1.6(100) = 244\text{PSF} \times 10' \times 21.25' = 51.8k = P_u$

$P_u = 51.8K$



$V_{max} = \frac{51.8}{2} = 25.9K$

$M_{max} = \frac{51.8 \times 20}{4} = 259 \text{ ft K}$

CHECK COMPOSITE STRENGTH

16(3/4") DIAMETER STUDS EVENLY SPACED ALONG LENGTH

→ Deck is parallel — $\frac{w_r}{h_r} = \frac{5}{2} = 2.5 \geq 1.5$

$Q_n = 17.1K$ [Conservative ($f'_c = 3ksi$)]

$\Sigma Q_n = 8 \times 17.1 = 136.8K$

DETERMINE EFFECTIVE FLANGE WIDTH

$b_{eff} = \min \begin{cases} (1.3 \times 12) + (21.25 \times 12/2) = 143.1" \\ 20 \times 12/8 \times 2 = 60" \end{cases}$

$a = \frac{136.8}{0.85 \times 3.5 \times 60} = 0.77\text{in}; \quad Y2 = 5.25 - \frac{0.77}{2} = 4.9\text{in}$

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$V_{s,max} = 14.7 \times 50 = 735$
$V_{c,max} = 0.85 \times 3.5 \times 60 \times 3.25 = 580k$ } $> \Sigma Q_n = 136.8k$

FROM TABLE 3-19, $\phi M_n = 537$ ftk [CONSERVATIVE]

$\phi M_n = 537$ ftk > 259 ftk ✓ GOOD!

CHECK UNSHORED STRENGTH

1.4 DL and 1.2 DL + 1.6 LL ← CONSTRUCTION

DL = 46.5 PSF LL = 20 PSF

$W_u = 1.4[46.5] = 65.1 \times 10 \times 21.25 = 13.8k = P_u$

$W_u = 1.2(46.5) + 1.6(20) = 87.8$ PSF
$P_u = 87.8 \times 10 \times 21.25 = 18.6k$

$M_u = \dfrac{18.6 \times 20}{4} = 93$ ftk

$\phi M_p = 379$ ftk > 93 ftk ✓ GOOD!

CHECK DEFLECTIONS

Wet Concrete Deflection

$W_{DL} = 46.5$ PSF ; $P_D = 46.5 \times 10 \times 21.25 = 9.9k$, $I_x = 800$ in^4

$\Delta_{DL,\,wet\,conc} = \dfrac{PL^3}{48EI} = \dfrac{9.9 \times 20^3 \times 1728}{48 \times 29000 \times 800} = 0.12$ in

$\Delta_{wet,max} = \dfrac{20 \times 12}{240} = 1"$

0.12" < 1" ✓ GOOD!

Live Load Deflection

$W_{LL} = 100$ PSF ; $P_L = 0.1 \times 10' \times 21.25' = 21.25k$
$I_{LB} = 1380$ in^4 [CONSERVATIVE]

$\Delta_{LL} = \dfrac{21.25 \times 20^3 \times 1728}{48 \times 29000 \times 1380} = 0.15 < \dfrac{20 \times 12}{360} = 0.67"$ ✓ GOOD!

SUMMARY: W18x50 WITH 16 EVENLY SPACED STUDS WORKS!
GIRDER WT = 50x(20) + (16x10) = 1060 lbs

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VERIFYING FLOOR GIRDERS (EXTERIOR)

W18x35 , SPAN: 20'-0" TRIB WIDTH = 10.625'

POINT LOAD CALC

DL = 67.2 + 35/10.625 = 70.5 PSF

$W_{u,D}$ (Exterior wall) = 436 PLF (unfactored)

P_u = [1.2 (70.5) + 1.6 (100)] x 10 x 10.625 = 25.9 K

$W_{u,D}$ = 1.2 (436) = 0.523 KLF

$M_u = \frac{25.9 \times 20}{4} + \frac{0.523 \times 20^2}{8}$

M_u = 156 ft k

CHECK COMPOSITE STRENGTH

14 (3/4") DIAMETER STUDS EVENLY SPACED ALONG LENGTH

→ Deck is parallel – w_r/h_r = 2.5 ≥ 1.5

Q_n = 17.1 k [Conservative (f'_c = 3 ksi)]

ΣQ_n = 7 x 17.1 = 120 k

DETERMINE EFFECTIVE FLANGE WIDTH

b_{eff} = 60 in ; $a = \frac{120}{0.85 \times 3.5 \times 60}$ = 0.67 ; $Y2 = 5.25 - \frac{0.67}{2}$ = 4.9 in

$V_{s,max}$ = 515 k
$V_{c,max}$ = 580 k > ΣQ_n = 120 k

From Table 3-19, ϕM_n = 363 ft k [Conservative]

ϕM_n = 363 ft k > 156 ft k ✓ GOOD

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Check Unshored Strength

1.4DL and 1.2DL + 1.6LL * CONSTRUCTION

DL = 47.5 PSF   LL = 20 PSF W_{DL} = 436 PLF

$P_u = 1.4[47.5] \times 10 \times 10.625 = 6.9K$

$W_{uo} = 1.4[436] = 0.61\ KLF$

$P_u = (1.2[47.5] + 1.6[20]) \times 10 \times 10.625 = 9.5K$

$W_u = 1.2(436) = 0.523\ KLF$

$$M_u = \frac{9.5 \times 20}{4} + \frac{0.523 \times 20^2}{8} = 73.6\ ftk$$

$\phi M_p = 249\ ftk > 73.6\ ftk = M_u$ ✓ GOOD

Check Deflections

Wet Concrete Deflection I_x =

$$\Delta_{DL,\text{wet conc}} = \frac{P_{DL} L^3}{48EI} + \frac{5 w_{DL} L^4}{384EI}$$

$$= \frac{5k \times 20^3 \times 1728}{48 \times 29000 \times 510} + \frac{5 \times 0.436 \times 20^4 \times 1728}{384 \times 29000 \times 510}$$

$\Delta_{DL\,\text{wet conc}} = 0.20\text{in} < L/360 = 0.67\text{in}$ ✓ GOOD!

Live Load Deflection

W_{LL} = 100 PSF ; P_{LL} = 10.625k , I_{LB} = 906 in^4 [Conservative]

$$\Delta_{LL} = \frac{10.625 \times 20^3 \times 1728}{48 \times 29000 \times 906} = 0.12'' < 0.67''$$ ✓ GOOD

SUMMARY: W18×35 WITH 14 EVENLY SPACED STUDS WORKS!

GIRDER WT = (35×20) + (14×10) = 840 POUNDS

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VERIFYING THE COLUMNS (INTERIOR)

Column B8 (W12x65)

⇒ The renovated building consists of 7 floors of concrete + 3 floors of structural steel. With that said, the column analysis was performed assuming wide flange columns were used on the lowest framed floor.

Worst Case Loading: $1.2DL + 1.6LL + 0.5SL$

FLOOR DL – 80PSF SNOW LOAD – 20PSF
FLOOR LL – 100PSF MAIN ROOF DL – 103 PSF
PENTHOUSE ROOF LL – 30 PSF PENTHOUSE ROOF DL – 27PSF
MAIN ROOF LIVE LOAD – 100PSF TRIB AREA = 425 ft^2

Live load Reduction
MAIN ROOF LIVE LOAD – $100 \cdot 0.5 = 50$ PSF

FLOOR LIVE LOAD – 50 PSF

$$P_u = ([1.2(80 \times 9) + 1.2(103) + 1.6(50 \times 9) + 1.6(50) + 0.5(20PSF)])$$

$= 763k$

From Table 4-1, Eff length = 10

ϕP_n for W12x65 = 766 k > 763k ✓ GOOD!

NOTE. The P_u load was over-designed due to the use of the worst case tributary area in areas where the trib. should be smaller.

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VERIFYING THE COLUMNS (EXTERIOR)

Column A8 (W10x49)

Same loads as for interior ; TRIB AREA = 212.5 ft^2

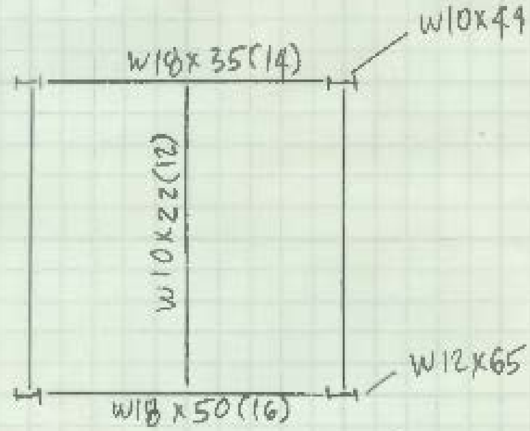
$$P_u = ([1.2(80\times9) + 1.6(50\times9) + 1.6(50)]) \times 212.5) + 1.2(\underbrace{.436\times 20}_{\text{Distributed Wall Load}})$$

P_u = 364 k

From Table 4-1, Eff. length = 10

ϕP_n = 550 k > 364 k ✓

CHECK SUMMARY



OVERVIEW OF ALTERNATIVE SYSTEMS

Three alternative systems were designed for the same typical bay analyzed for the existing framing system and a comparison between the three various systems was made. The three alternative systems include;

I. Reinforced two-way flat-slab with edge beam
II. Structural steel framing w/ composite joists
III. Non-composite wide flange steel frame on composite deck

These systems were selected on a structural efficiency and cost-saving basis. The following sections will highlight the advantages and disadvantages of each alternative system in greater detail.

ALTERNATIVE SYSTEM #1: REINFORCED TWO-WAY FLAT SLAB W/ EDGE BEAM

The reinforced two-way flat slab was designed for a typical 20’-0” x 21’-3” bay. 12” x 16” edge beams were incorporated into the design to reduce the moments at the exterior columns, thus distributing the reinforcement between the slab and the beams, which save cost. This, however, can be counter-productive when the cost of constructing an edge beam in to the two-way slab is taken into account. The typical column size used in the design is a 24” x 24” square column.

All calculations were performed using the Direct Design Method, taken into account the on-way shear and the two-way punching shear and followed the Building Code Requirements for Structural Concrete (ACI 318-11).

Advantages

- Reduced slab thickness thus reducing overall floor to floor height
- Inexpensive system
- Relatively easy to construct
-

Disadvantages

- Increased overall weight
- Increased labor costs due to use of formwork and placement and handling of concrete
- Lateral system has to be re-evaluated

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ALTERNATIVE SYSTEM 1

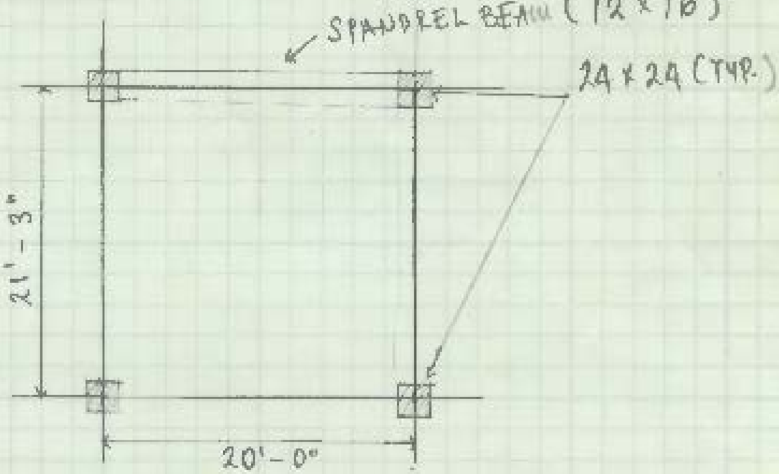
REINFORCED TWO-WAY FLAT SLAB

Assumptions:

$f'c = 4000$ PSI $fy = 60000$ PSI

SDL – 23 PSF LL – 100 PSF

TYPICAL BAY



DETERMINE SLAB THICKNESS

→ As per ACI Code Section 9.5.3, minimum thickness for slabs without interior beams spanning between the supports and having a ratio of long to short span ≤ 2

$$\frac{LS}{SS} = \frac{21.25}{20} = 1.06 \leq 2 \checkmark$$

→ Minimum slab thickness of slabs without interior beams without drop panels [TABLE 9.5(c) ACI 318-10]

$$\frac{ln}{33} = \frac{[(20'\text{-}0" \times 12) - 24]}{33} \simeq 7"$$

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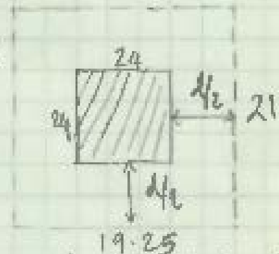
$$w_u = 1.2\left[\left(\frac{7}{12} \times 150\right) + 23\right) + 1.6(80)\right] = 261\,PSF$$

CHECK PUNCHING SHEAR (Interior Col.)

→ ASSUMING #5 BARS

$$d_{avg} = 7" - \underbrace{0.75}_{CC} - \underbrace{0.625}_{Reinf\ Dia} = 5.625"$$



→ length of critical perimeter $= (24 + 5.625) \times 4 = 118.5"$

→ Computing V_c for critical Section

$V_c = \min$

$$4\sqrt{f'_c}\, b_o d = 4\sqrt{4000}(118.5)(5.625) = 168.8k$$

$$\left(2 + \frac{4}{\beta_c}\right)\sqrt{f'_c}\, b_o d = \left(2 + \frac{4}{1}\right)\sqrt{4000}(118.5)(5.625) = 252.9k$$

$$\left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f'_c}\, b_o d = \left(\frac{40 \times 5.625}{118.5} + 2\right)\sqrt{4000} \times 118.5 \times 5.625 = 165k$$

Use $V_c = 168.8k$; $\phi V_c = 0.75 \times 169 = \boxed{124k}$

ACI 318, R11.11.2.1

V_u on the critical perimeter $= 261 \times [20 \times 21.25 - (29.625 \times 2/12)$

$\boxed{V_u = 109.6k}$

Because $\phi V_c = 124k$ exceeds $V_u = 109.6k$, the slab is OK in 2-WAY Punching Shear

USE DIRECT DESIGN METHOD TO DISTRIBUTE MOMENTS

As per Section 13.6 ACI-318-10

Limitations

1. Min. of three continuous spans → GOOD
2. Span lengths should not differ by more than 1/3 l_n

   $1/3(21.25) = 7.1'$ → GOOD
3. Unfactored live load shall not exceed 2 x the unfactored DL

   $L_u = 80 \le 2 \times 87.5 = D_u$ ✓ GOOD

CAN USE DIRECT DESIGN METHOD

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Before proceeding with DDM, Check One Way Shear

CHECK

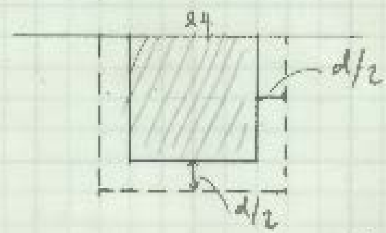
$V_u = (0.261)(21)(10 - 5.625/12) = 52.2K$

$V_c = 2 \times 1 \times \sqrt{4000} \times (20 \times 12) \times 5.625 = 170.7K$

$\phi V_c = 0.75 \times 170.7 = 128$ Kips

$\phi V_c = 128K > V_u = 52.2K$ ✓ GOOD!

CHECK PUNCHING SHEAR (Exterior Col.)



Assuming #5 Bars

$d_{avg} = 5.625''$

→ length of critical perimeter $= 2\left[(24 + 5.625) + (24 + 5.625/2)\right] = 113''$

→ Computing Vc for Critical Section

$$V_c = \min \begin{cases} 4\sqrt{f'_c}\, b_o d = \\ \left(2 + \frac{4}{\beta_c}\right)\sqrt{f'_c}\, b_o d \\ \left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f'_c}\, b_o d \end{cases}$$

Use $V_c = 4\sqrt{4000} \times 113 \times 5.625 = 161K$; $\phi V_c = 0.75 \times 161 = 121K$

$V_u = .261 \times \left[20 \times 10.625 - \left(29.625/12 \times 26.8125/12\right)\right] = 54K$

$\phi V_c = 121K > V_u = 54K$ ✓ GOOD.

Hence the slab is OK in 2-WAY Punching Shear

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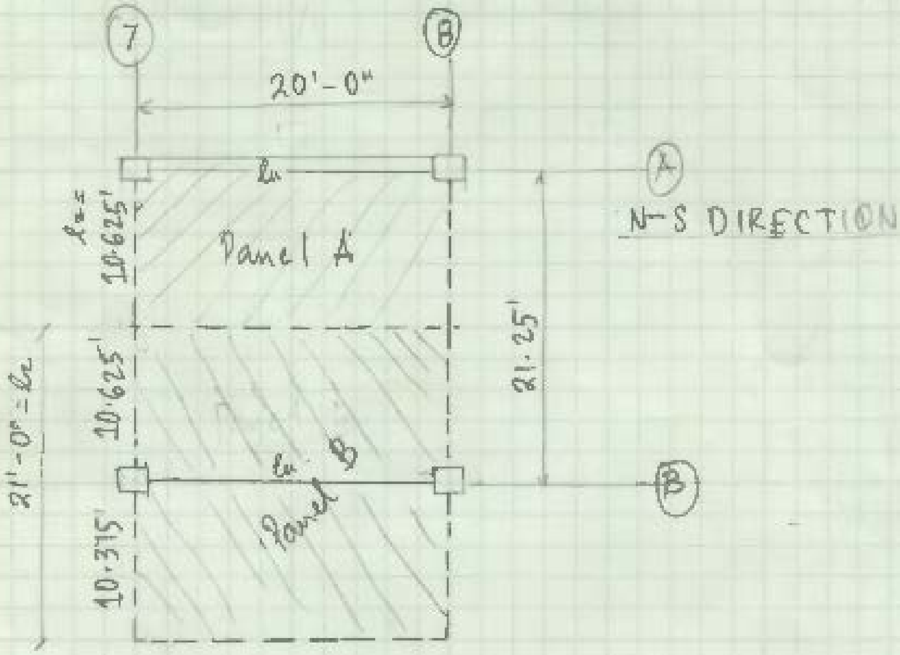
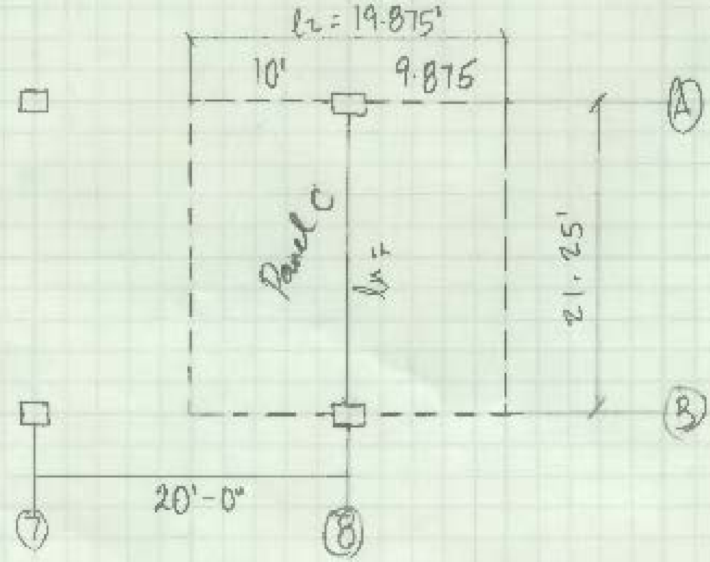
DIRECT DESIGN METHOD

$q_u = 261$ PSF Statistical Moment, $M_o = \frac{q_u l_2 l_n^2}{8}$





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Mo determination for the different methods

For Panel A
$$M_o = \frac{(0.261)(10.625)(20-2)^2}{8} = 112.3 \text{ ft k}$$

For Panel B
$$M_o = \frac{(0.261)(21)(20-2)^2}{8} = 221.9 \text{ ft k}$$

For Panel C
$$M_o = \frac{(0.261)(19.875)(21.25-2)^2}{8} = 240.2 \text{ ft k}$$

DISTRIBUTION OF MOMENTS

→ For Interior Spans (Panel B), as per ACI 318-10 13.6.3.2

-ve factored Moment [Support] = $-0.65 M_o$ = -144.2 k ft
+ve factored Moment [Midspan] = $0.35 M_o$ = 77.7 k ft

→ Dividing the moments between the column & middle strips

Negative Moments: As per ACI 318-11 13.6.4.1

$\alpha_{f1} l_2 / l_1 = 0$ [NO BEAMS BETWEEN COLUMNS]

Col. Strip -ve moment = $0.75 \times -144.2 = -108.2$ k ft
Middle Strip -ve moment = $0.25 \times -144.2 = -36.1$ k ft

Positive Moments: As per ACI 318-11 13.6.4.4
$\alpha_{f1} l_2 / l_1 = 0$

Column Strip +ve moment = $0.60 \times 77.7 = 46.6$ k ft
Middle Strip +ve moment = $0.40 \times 77.7 = 31.1$ k ft

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

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→ For Exterior Span (Panel A & Panel C) as per ACI 318-11 13.6.3.3

Panel C

Ext. −ve factored moment $= -0.30 M_o = -72.1$
+ve factored moment $= 0.50 M_o = 120.1$
Int. −ve factored moment $= -0.70 M_o = -168.1$ ft k

→ Dividing the moments between the column & middle strips

Interior −ve moments: As per ACI 318-11 13.6.4.1

$\alpha_{f1} \ell_2/\ell_1 = 0$ [NO BEAMS BETWEEN COLUMNS]

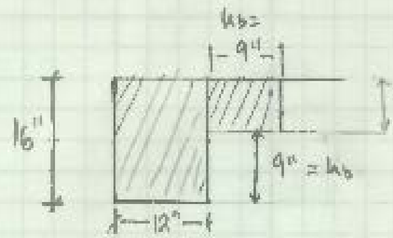
Interior Column-strip −ve moment $= 0.75 \times -168.1 = -126.1$ ft k
Interior Middle-strip −ve moment $= 0.25 \times -168.1 = -42$ ft k

+ve moments: As per ACI 318-11 13.6.4.4

Column-strip +ve moment $= 0.60 \times 120.1 = 72.1$ ft k
Middle-strip +ve moment $= 0.40 \times 120.1 = 48$ ft k

Exterior −ve moments

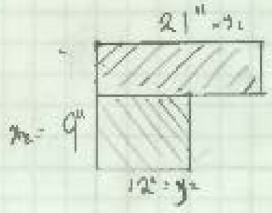
$\alpha_{f1} \ell_2/\ell_1 = 0$, $\beta_t = \dfrac{E_{cb} C}{2 E_{cs} I_s}$ For the attached torsional member shown below



limitations
$h_b \le 4 h_f$; $9 < 4(7)$ ✓

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

$$C = \frac{(1 - 0.63 \times 12/16)\, 12^3 \times 16}{3} + \frac{(1 - 0.63 \times 7/9)\, 7^3 \times 9}{3} = 2854 \text{ in}^4$$



$$C = \frac{(1 - 0.63 \times 7/21)\, 7^3 \times 21}{3} + \frac{(1 - 0.63 \times 9/12)\, 9^3 \times 12}{3}$$

$$C = 1105 \text{ in}^4$$

USE LARGER C ; $C = 2854 \text{ in}^4$

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$\Rightarrow I_s = \dfrac{b h^3}{12}$ where b = length of strip of slab being designed
b = l_2 and h = 7in

$I_s = \dfrac{(19.875 \times 12) \times 7^3}{12} = 6817 \text{ in}^4$

Assuming same f'c value in slab and beam, $E_{cb} = E_{cs}$

$\beta_t = \dfrac{2854 \text{ in}^4}{2 \times (6817) \text{ in}^4} = 0.209$

As per ACI 318-11 13.6.4.2, through linear interpolation

β_t	$l_2/l_1 = 1.0$
0	100
0.209	x
2.5	75

$\dfrac{2.5 - 0.209}{2.5 - 0} = \dfrac{75 - x}{75 - 100}$

$x = 97.9\%$ to column strip

Therefore,

Exterior column strip -ve moment = 0.979 (-72.1) = $\boxed{-70.6 \text{ k ft}}$
Exterior middle strip -ve moment = 0.021 (-72.1) = $\boxed{-1.5 \text{ k ft}}$

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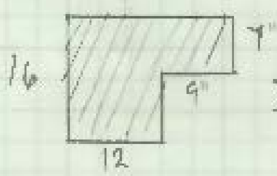
For Panel A as per ACI 318-11 13.6.3.3

Ext. -ve factored moment = $-0.30 M_o$ = -33.7 ft k
+ve factored moment = $0.50 M_o$ = 56.2 ft k

Calculation of α_f for an Edge Beam

Centroid is 6.9 in from the top of slab



$$I_b = \frac{12 \times 16^3}{12} + (12 \times 16) \times (8 - 6.9)^2 + \frac{9 \times 7^3}{12} + (9 \times 7) \times (6.9 - 3.5)^2 = 5314 \text{ in}^4$$

$$I_s = \frac{(11.125 \times 12) \times 7^3}{12} = 3816 \text{ in}^4, \quad l_2/l_1 = 0.53$$

$$\Rightarrow \alpha_f = \frac{5314}{3816} = 1.4 \quad ; \quad \alpha_f l_2/l_1 = 0.74$$

Distribution of moments to the column and middle strips

Exterior -ve moment

$B_t = 0.209$ $\quad \alpha_f l_2/l_1 = 0$ [CONSERVATIVE] $\quad l_2/l_1 \approx 0.5$

As per ACI 318 13.6.4.2, through interpolation

B_t	$l_2/l_1 = 0.5$
0	100
0.209	x
2.5	75

$x = 97.9\%$ to Col Strip

Column strip = $0.979 M_o$ = -32.99 ft k
Middle strip = $0.021 M_o$ = -0.707 ft k

* Column strip moments are divided between the beam and slab according to the value of $\alpha_f l_2/l_1$

→ As per ACI 318-11, 13.6.5.2, if $\alpha_f l_2/l_1 < 1.0$, a linear interpolation is performed between 85% and 0%

$\alpha_f l_2/l_1$	%
1.0	85
→ 0.74	x
0	0

$$\frac{1.0 - 0.74}{1.0 - 0} = \frac{85 - x}{85 - 0}$$

$x = 62.9\%$ of column strip moment to the beams.

Balance of 37.1% assigned to slab

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Beam = 0.629 × -32.99 = -20.75 kft
Slab Portion = 0.371 × -32.99 = -12.24 kft

Positive Moment

$\alpha_f l_2/l_1 = 0.74$ and As per ACI 318-11 13.6.4.4

Through Interpolation,

$\alpha_f l_2/l_1$	$l_2/l_1 = 0.5$
0	60
0.74	x
1.0	90

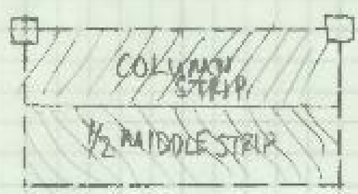
$$\frac{1-0.74}{1-0} = \frac{90-x}{90-60}$$

$x = 82.2\%$ to column strip

Column strip = 0.822 × 56.2 = 46.2 ft k
Middle strip = 0.178 × 56.2 = 10.0 ft k

⇒ As calculated earlier, 62.9% of col strip moment to beams and the balance to the rest (37.1%)

Beam = 0.629 × 46.2 = 29.1 kft
Slab Portion = 0.371 × 46.2 = 17.1 kft



Panel A

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DETERMINATION OF REINFORCEMENT

INTERIOR SPAN (Negative Reinforcement) – Panel B

→ Column Strip

$A_s = \frac{M_u}{\phi f_y j d} = \frac{-108.2 \times 12}{0.9 \times 60 \times 0.95 \times 5.625}$

$A_{s,req} = 4.5 in^2$

Minimum Reinforcing As per ACI 318-11 13.3.1

$A_{s,min} \geq 0.0018bh = 0.0018 \times 10.47 \times 7 \times 12 = 1.58 in^2$

$A_{s,req} > A_{s,min}$ ∴ GOOD

Hence, try (15) #5's

$\Rightarrow a = \frac{(4.65 in^2) \times 60}{0.85 \times 4 \times 10.47 \times 12} = 0.65$

$\phi M_n = [0.9(4.65)(5.625 - 0.65/2) \times 60]/12 = 111 ft k$

$\phi M_n = 111 ft k > M_u = 108.2 k$

Check max spacing

$S \leq 2h = 2 \times 7 = 14 in$

Therefore, $\boxed{\text{use (15) \#5's @ 12in}}$

→ Middle Strip

$A_s = \frac{-36.1 \times 12}{0.9 \times 60 \times 0.95 \times 5.625} = 1.5 in^2$

Check Minimum Reinforcement

$A_{s,min} = 1.58 in^2 > A_{s,req}$, Use $A_{s,min} = 1.58 in^2$

Try (6) #5's @ 12in ; $A_s = 1.86 in^2$

$\Rightarrow a = \frac{1.86 in^2 \times 60}{0.85 \times 4 \times 10.47 \times 12} = 0.26$

$\phi M_n = [0.9(1.86)60(5.625 - 0.26/2)]/12 = 46 ft k$

$\phi M_n = 46 ft k > M_u = 36.1 ft k$ ✓ GOOD

$\boxed{\text{Use (6) \#5's @ 12in}}$

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INTERIOR SPAN (+ve Reinforcement)

→ Column Strip

$$A_{s,req} = \frac{46.6 \times 12}{0.9 \times 60 \times 0.95 \times 5.625} = 1.94\ in^2$$

Check Min. Reinforcement

$A_{s,req} = 1.94\ in^2 > A_{s,min} = 1.58\ in^2$ ✓

Try (7) #5's @ 12 in, $A_s = 2.17\ in^2$

$$a = \frac{2.17 \times 60}{0.85 \times 4 \times 10.47 \times 12} = 0.304\ in$$

$$\phi M_n = \left[0.9 \times 2.17 \times 60 \times \left(5.625 - 0.304/2\right)\right] / 12 = 53.4\ ft\text{-}K$$

$\phi M_n > M_u$ ✓ GOOD!

Hence, $\boxed{\text{use (7) \#5's @ 12 in}}$

→ Middle Strip

$$A_{s,req} = \frac{31.1 \times 12}{0.9 \times 60 \times 0.95 \times 5.625} = 1.29\ in^2$$

Check min Reinforcement

$A_{s,min} = 1.58\ in^2 > A_{s,req} = 1.29\ in^2$

Use $A_s = 1.58\ in^2$

Try (6) #5's @ 12 in $A_s = 1.86\ in^2$

$$a = \frac{1.86 \times 60}{0.85 \times 4 \times 10.47 \times 12} = 0.261\ in$$

$$\phi M_n = \left[0.9 \times 1.86 \times 60 \times \left(5.625 - 0.261/2\right)\right] / 12 = 46\ ft\text{-}K$$

$\phi M_n > M_u$ ✓ GOOD!

Hence, $\boxed{\text{use (6) \#5's @ 12 in}}$

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EXTERIOR SPAN (-ve Reinforcement) [Panel C]

Column Strip

$$A_{s,req} = \frac{126.1 \times 12}{0.9 \times 60 \times 0.95 \times 5.625} = 5.24 \text{ in}^2$$

Check min. reinforcement

$A_{s,req} = 5.24 \text{ in}^2 \geq A_{s,min} = 1.50 \text{ in}^2$

Try (18) #5's, $A_s = 5.58 \text{ in}^2$

$$a = \frac{5.58 \times 60}{0.85 \times 4 \times 9.9375 \times 12} = 0.825 \text{ in}$$

$$\phi M_n = [0.9 \times 5.58 \times 60 \times (5.625 - 0.825/2)]/12 = 130.8 \text{ ft-k}$$

$\phi M_n > M_u$ ✓ GOOD!

Hence, $\boxed{\text{use (18) \#5's @ 12}}$

Middle Strip

$$A_{s,req} = \frac{42 \times 12}{0.9 \times 60 \times 0.95 \times 5.625} = 1.75 \text{ in}^2$$

$A_{s,req} = 1.75 \text{ in}^2 \geq A_{s,min} = 1.50 \text{ in}^2$ ✓ GOOD!

Try (6) #5's @ 12in $A_s = 1.86 \text{ in}^2$

$$a = \frac{1.86 \times 60}{0.85 \times 4 \times 9.9375 \times 12} = 0.275 \text{ in}$$

$$\phi M_n = [0.9 \times 1.86 \times 60 \times (5.625 - 0.275/2)]/12 = 45.9 \text{ ft-k}$$

$\phi M_n > M_u$ ✓ GOOD

$\boxed{\text{Use (6) \#5's @ 12in}}$

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EXTERIOR SPAN (+ve Reinforcement) [Panel C]

Column Strip

$$A_{s,req} = \frac{72.1 \times 12}{0.9 \times 60 \times 0.95 \times 5.625} = 2.99\ in^2$$

$A_{s,req} = 2.99\ in^2 > A_{s,min} = 1.50\ in^2$

Try (10) #5's @ 12 in , $A_s = 3.10\ in^2$

$$a = \frac{3.10 \times 60}{0.85 \times 4 \times 9.9375 \times 12} = 0.458\ in$$

$$\phi M_n = [0.9 \times 3.10 \times 60 \times (5.625 - 0.458/2)]/12 = 75.2\ ft\,k$$

$\phi M_n > M_u$ ✓ GOOD!

$\boxed{\text{USE (10) \#5's @ 12in}}$

Middle Strip

$$A_{s,req} = \frac{48 \times 12}{0.9 \times 60 \times 0.95 \times 5.625} = 1.996\ in^2$$

$A_{s,req} = 1.996\ in^2 > A_{s,min} = 1.50\ in^2$

Try (7) #5's @ 12in , $A_s = 2.17\ in^2$

$$a = \frac{2.17 \times 60}{0.85 \times 4 \times 9.9375 \times 12} = 0.32\ in$$

$$\phi M_n = [0.9 \times 2.17 \times 60 \times (5.625 - 0.32/2)]/12 = 53.4\ ft\,k$$

$\phi M_n > M_u$ ✓ GOOD!

$\boxed{\text{USE (7) \#5's @ 12in}}$

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

EXTERIOR SPAN (-ve Reinforcement) [Panel A]

Column Strip (Beam Portion) - Assumed #4's

$$A_{s,req} = \frac{20.75 \times 12}{0.9 \times 60 \times 0.95 \times 14.25} = 0.34 in^2$$

Try 2 #4's, $A_s = 0.40 in^2 > 0.34 in^2$ ✓

$$a = \frac{0.4 \times 60}{0.85 \times 4 \times 12} = 0.58 in$$

$$\phi M_n = [0.9 \times 0.4 \times 60 \times (14.25 - 0.58/2)]/12 = 25.1 kft$$

$\phi M_n > M_u$

USE $\boxed{(2) \#4's @ 6 in}$

Slab Portion (Assumed #5's)

$$A_{s,req} = \frac{12.24 \times 12}{0.9 \times 60 \times 0.95 \times 5.625} = 0.51 in^2$$

$A_{s,min} = 0.8 in^2 > 0.51 in^2$ Use $A_{s,min}$

Try (3) #5's @ 10 in, $A_s = 0.93 in^2$

$$a = \frac{0.93 in^2 \times 60}{0.85 \times 4 \times 5.31 \times 12} = 0.26 in$$

$$\phi M_n = \left[0.9 \times 60 \times 0.93 \times \left(5.625 - \frac{0.26}{2}\right)\right]/12 = 23 ftk > M_u \checkmark$$

$\boxed{USE (3) \#5's @ 10 in}$

Positive Reinforcement

Column Strip (Beam Portion) - Assumed #4's

$$A_{s,req} = \frac{29.1 \times 12}{0.9 \times 60 \times 0.95 \times 14.25} = 0.477 in^2$$

Try (3) #4's @ 8 in ; $A_s = 0.60 in^2$

$$a = \frac{0.6 \times 60}{0.85 \times 4 \times 12} = 0.88 in$$

$$\phi M_n = [0.9 \times 0.6 \times 60 \times (14.25 - 0.88/2)]/12 = 37.2 ftk > M_u$$

$\boxed{Use (3) \#4's @ 8 in}$

Slab Portion (Assumed #5's)

$$A_{s,req} = \frac{17.1 \times 12}{0.9 \times 60 \times 0.95 \times 5.625} = 0.71 in^2$$

$A_{s,min} = 0.8 in^2 > 0.71 in^2$ Use $A_{s,min} = 0.8 in^2$

Try (3) #5's $A_s = 0.93 in^2$

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$a = 0.20 in \quad \phi M_n = 23 ft k = M_u = 17.1 ft k$

$\boxed{\text{USE (3) \#5's @ 10 in}}$

Middle Strip (-ve) moment

$$A_s = \frac{0.707 \times 12}{0.9 \times 60 \times 0.95 \times 5.625} = 0.029 in^2$$

$A_{s,min} = 0.8 in^2 > 0.029 in^2$. Use $A_s = 0.8 in^2$

Try (3) #5's @ 10 in, $A_s = 0.93 in^2$

$\phi M_n = 23 ft k >> 0.707$ ✓

$\boxed{\text{USE (3) \#5's @ 10 in}}$

Middle Strip (+ve moment)

$$A_s = \frac{10 \times 12}{0.9 \times 60 \times 0.95 \times 5.625} = 0.42 in^2$$

$A_{s,min} > A_{s,req}$, Use $A_{s,min} = 0.8 in^2$

Try (3) #5's @ 10 in, $A_s = 0.93 in^2$

$\phi M_n = 23 ft k > 10 ft k$ ✓

$\boxed{\text{USE (3) \#5's @ 10 in}}$

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

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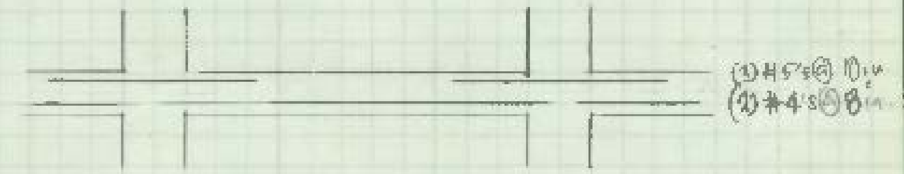
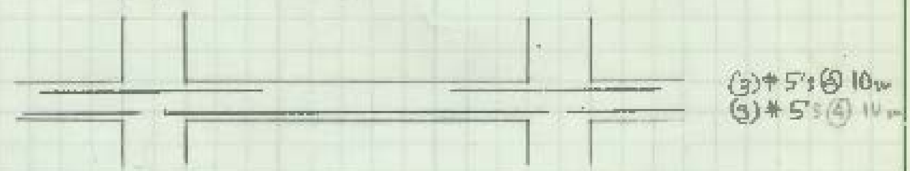
DESIGN SUMMARY

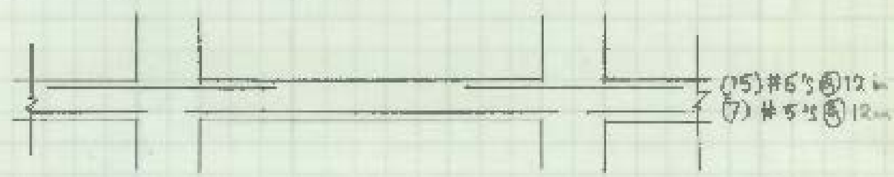
PANEL A COLUMN STRIP



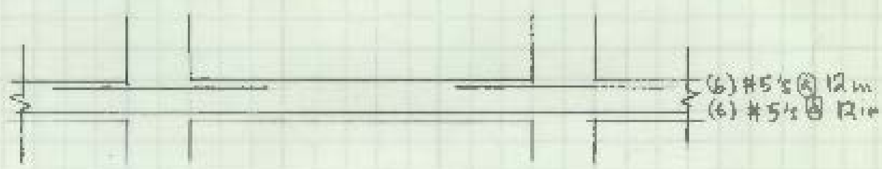
PANEL A MIDDLE STRIP



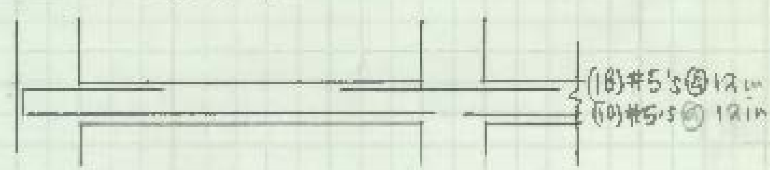
PANEL B COLUMN STRIP



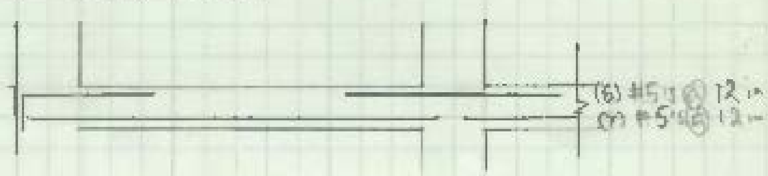
PANEL B MIDDLE STRIP



PANEL C COLUMN STRIP



PANEL C MIDDLE STRIP



ALTERNATIVE SYSTEM #2: STRUCTURAL STEEL FRAME W/ COMPOSITE JOISTS

The structural steel frame was system was designed for a modified 20'-0" x 20'-0" bay, due to considerations for use of the SJI Standard for Composite Joists. The composite joists were incorporated with the use of non-composite beam/girders on the column lines with the goal of adding extra stiffness to the structure. The composite joists were designed using a structural slab of 2 ½" lightweight concrete over a 2" x 18 gage metal deck

Although CJ-series joists were specified in this design, it will be more economical to use ECOSPAN composite joists. This creates a simple, lightweight, flexible and easily constructible framing system, which also saves costs.

Advantages

- Lightweight system which potentially translates to reduced foundation costs
- Non-combustible steel makes it good for fire protection
- Reductions in overall floor to floor height
- Easily constructible
- Efficient erection
- Inexpensive system

Disadvantages

- Not a typical system used for construction

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DETERMINATION OF VALUES USED IN CALCULATION

Decking – (2" x 18 GA DECK, 4½" LWC)

1. Span Check
SDI Max Unshored Span = 13'-1" > 10" ✓ GOOD!

2. Superimposed load deck
W_{LL} + Superimposed DL ≤ Superimposed load
123 ≤ 170 ✓ GOOD!

Construction Live Load
Construction live load should be estimated as follows:

$L_c = 20R_1$, where $12 \le L_c \le 20$ PSF

$A_t = 20 \times 20 = 400\ ft^2$

$R_1 = 1.2 - 0.001A_t$ for $200\ ft^2 < A_t < 600\ ft^2$

$R_1 = 0.8$; $L_L = 20\ PSF \times 0.8 = 16\ PSF$

Composite Live Load

Reduced as per ASCE 7-10, 4.7.2

$L_0 = 100$ PSF ; $L = 100 \times \max \begin{vmatrix} 0.5 \\ 0.25 + \frac{15}{\sqrt{2 \times 400}} = 0.78 \end{vmatrix}$

$L = 100 \times 0.78 = 78$ PSF

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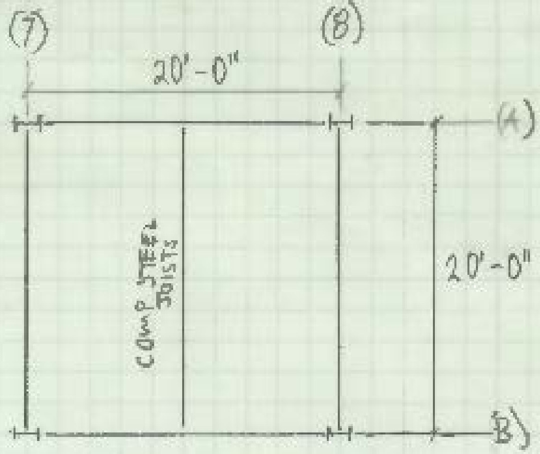
ALTERNATIVE SYSTEM 2

STRUCTURAL STEEL FRAME W/ COMPOSITE JOISTS

TYPICAL BAY



ASSUMPTIONS AND CALCULATIONS

JOIST GEOMETRY
1. DEPTH — TO BE DETERMINED
2. SPAN — 20 ft
3. ADJ. MEMBER SPACING — 10 ft

CONCRETE AND DECK
1. TYPE OF FLOOR DECK — 2VLI 18 GA
2. DEPTH OF FLOOR DECK — 4.5"
3. SLAB THICKNESS ABOVE DECK — 2.5"
4. CONCRETE UNIT WT — 115 PCF
5. CONCRETE COMP. STRENGTH — 4 KSI

NOMINAL LOADS:

1. NON-COMPOSITE CONSTRUCTION DEAD LOAD
a. Concrete + Deck 35 PSF
b. Joist and Bridging 5 PSF (Assumed)
40 PSF
⇒ 40 PSF × 10' = 400 PLF

2. CONSTRUCTION LIVE LOAD [REDUCED]
a) During Concrete Placement 16 PSF
⇒ 16 × 10 = 160 PLF

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3. COMPOSITE DEAD LOAD
(a) MEP 15 PSF
(b) CEILING 5 PSF
(c) SPRINKLERS 3 PSF
23 PSF
⇒ 23 x 10 = 230 PLF

4. COMPOSITE LIVE LOAD
(a) Live Load [Reduced] 78 PSF
⇒ 78 x 10 = 780 PLF

5. TOTAL FACTORED NON-COMPOSITE DEAD LOAD, 1.2 x (NCDL)
⇒ 1.2 x 40 = 48 PSF or 480 PLF

6. TOTAL FACTORED COMPOSITE DEAD LOAD, 1.2 x (CDL)
⇒ 1.2 x 23 = 27.6 PSF or 276 PLF

7. TOTAL FACTORED COMPOSITE LIVE LOAD, 1.6 x (CLL)
⇒ 1.6 x 78 = 124.8 PSF or 1248 PLF

8 TOTAL FACTORED COMPOSITE DESIGN LOAD,
⇒ 480 + 276 + 1248 = 2004 PLF

CAMBER AND DEFLECTION (Unfactored Load)

1. LOADS TO CAMBER FOR:
a) Non-composite Dead Load 40 x 100% = 40 PSF
b) Composite Dead Load 23 x 50% = 11.5 PSF
c) Composite Live Load 78 x 10% = 7.8 PSF

2. MAXIMUM ALLOWABLE LIVE LOAD DEFLECTION, L/360
⇒ (20 x 12)/360 = 0.67 in

3. MAXIMUM DEFLECTION, L/240
⇒ (20 x 12)/240 = 1 in

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→ DETERMINE JOIST WT/FT, QUANTITY AND SIZE OF SHEAR STUDS, ANTICIPATED FLOOR DEFLECTIONS, NUMBER OF BRIDGING ROWS RQD AND MAXIMUM CIRCULAR DUCT OPENING

1). Assumed Joist Depth – $\boxed{14\,in}$

2). Joist Selection

* The proper joist shall be selected from the Design Guide LRFD Weight Table for Composite Steel Joists CJ-Series – LWC for a 20ft Joist, depth of 14in, a total factored composite design load of 2004 PLF, and a composite live load of 1248 PLF *

For 14in joist depth selected;

(a) W_t = 13.6 PLF
(b) W360 = 1384 PLF > 1248 PLF ✓ GOOD!
(c) N–ds = 16 – 5/8"

where N = Quantity of Shear Studs
ds = Types of Shear Studs

3. BRIDGING AND NOMINAL HORIZONTAL TOP CHORD FORCE (P_{br}) SELECTION

→ From the Design Guide LRFD Weight Bridging Table for Composite Steel Joists, CJ-Series – LWC

+1 row of horizontal bridging (1H) is required

Using J_s = 10ft and Joist depth = 14in

Bridging Size – L 2.5 x 2.5 x 0.187 [Conservative Choice]

$\boxed{P_{br} = 750\,lbs}$

4. Non-Composite Effective Moment of Inertia Selection

→ From the Design Guide LRFD Weight Bridging Table for Composite Steel Joists, CJ-Series – LWC

Using TL = 2000 PLF and Joist depth = 14in

$I_{non\text{-}comp\,eff}$ = 108 in^4

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

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DEFLECTION

$$\Delta_{NCDL} = \frac{5(W_{non\text{-}composite\ DL})(\text{Design Length})^4(1728)}{384\, E_s\, I_{non\text{-}comp\ eff}}$$

where Design Length = Span − 4in
= 20 − 4/12 = 19.67ft

$$\Delta_{NCDL} = \frac{5(.400)(19.67)^4(1728)}{384(29000)(108)} = 0.43\text{in}$$

$$\Delta_{CDL} = \left[\frac{W_{comp.DL}}{W_{360}}\right]\left[\frac{L}{360}\right] = \left[\frac{230}{1384}\right]\left[\frac{19.67\times 12}{360}\right] = 0.11\text{in}$$

$$\Delta_{CLL} = \left[\frac{W_{comp\ LL}}{W_{360}}\right]\left[\frac{L}{360}\right] = \left[\frac{780}{1384}\right]\left[\frac{19.67\times 12}{360}\right] = 0.37\text{in}$$

$$\Delta_{TL} = \Delta_{non\text{-}composite\ DL} + \Delta_{composite\ DL} + \Delta_{composite\ LL}$$

$$\Delta_{TL} = 0.43\text{in} + 0.11\text{in} + 0.37\text{in} = 0.91\text{in}$$

CAMBER

Camber joist for 100% × $\Delta_{NON\text{-}COMPOSITE\ DL}$ +
50% × $\Delta_{COMPOSITE\ DL}$ +
10% × $\Delta_{COMPOSITE\ LL}$

Joist Camber = 1.0 × 0.43 +
0.5 × 0.11 +
0.1 × 0.37
= 0.52in

EFFECTIVE MOMENT OF INERTIA SELECTION

→ From the Design Guide LRFD Weight Table for Composite Steel Joists, CJ-Series – LWC

Using TL = 2000PLF and 14in joist depth

→ $I_{eff} = 258\text{in}^4$

Note: the published value of W360 takes into account the reductions in the effective transformed moment of inertia associated with web deformations and interfacial slippage. Hence, Ieff has been reduced be an assumed factor of 1.05 to account for these behaviors

⇒ $I_{e,\text{composite without slippage}} = 1.05\, I_{eff} = 1.05 \times 258\text{in}^4 = 271\text{in}^4$

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DESIGN SUMMARY

The composite steel joist used is $\boxed{14CJ\ 2004/1248/276}$

Designations

14	—	Depth (in)
CJ	—	Composite Joist Series
2004	—	Total Factored Composite Design Load (PLF)
1268	—	Total Factored Composite Live Load (PLF)
276	—	Total Factored Composite Dead Load (PLF)

BRIDGING

Use 1 row of 2L's 2.5×2.5×0.187

JOIST WT = 13.6 PLF

DEFLECTIONS

$\Delta_{\text{Non-composite DL}} = 0.43\text{ in}$

$\Delta_{\text{composite DL}} = 0.11\text{ in}$

$\Delta_{\text{composite LL}} = 0.37\text{ in}$

CAMBER = 0.52 in

QUANTITY AND TYPE OF SHEAR STUDS

N — ds = 16 — 5/8"

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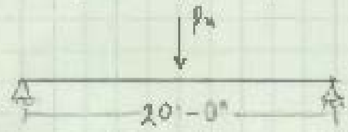
GIRDER DESIGN (Interior)

(WEIGHT OF COMP. JOISTS)

Span = 20'-0" $DL = 35 + 23 + 13.6/20 = 58.6$ PSF

$LL = 78.0$ PSF [Reduced]

$P_u = [1.2(58.6) + 1.6(78)]\ 10 \times 20 = 39k$



$M_u = \frac{P_u L}{4} = \frac{39 \times 20}{4} = 195$ ftk

Check Bending

Using Zx tables (Table 3-2), Try W16x31

$\phi M_n = 203$ ftk $> M_u = 195$ ft k ✓ GOOD

Check Deflection, $I_x = 375\ in^4$

$\Delta_{LL} = \frac{0.78 \times 20 \times 20^3 \times 1728}{48 \times 29000 \times 375} = 0.41in < \frac{L}{360} = 0.67in$ ✓

$\Delta_{DL;WET\ CONC} = \frac{0.357 \times 20 \times 20^3 \times 1728}{48 \times 29000 \times 375} = 0.19in < \frac{L}{360} = 0.67in$ ✓

NO CAMBER

Exterior Girder

Span = 20'-0" $DL = 58.6$ PSF + Distributed Load (Wall)

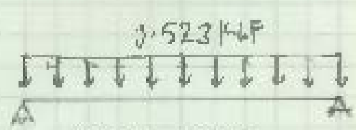
$LL = 100$ PSF

$P_u = [1.2(58.6) + 1.6(100)] \times 10 \times 10 = 23k$

$W_{u,wall} = 1.2(436\ PLF) = 0.523$ KLF

Super-position,

+



$M_u = \frac{23 \times 20}{4} + \frac{0.523 \times 20^2}{8}$

$M_u = 141.1$ ftk

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Check Bending

Using Zx tables (Table 3-2), $\boxed{\text{Try W14X26}}$

$$\phi M_n = 151 \text{ ft-k} > M_u = 141 \text{ ft-k}$$

Check Deflection, $I_x = 245 \text{ in}^4$

$$\Delta_{LL} = \frac{P_{LL} l^3}{48EI} = \frac{.1 \times 10 \times 10 \times 20^3 \times 1728}{48 \times 29000 \times 245} = 0.41\text{in} < 0.67\text{in} \checkmark \text{ GOOD}$$

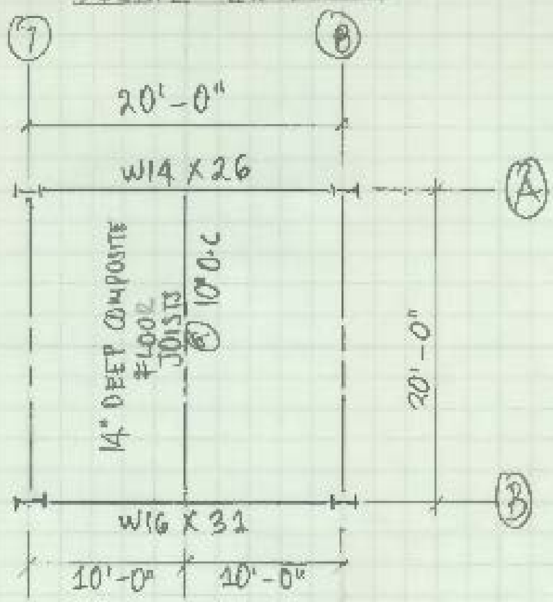
$$\Delta_{DL,\text{wet conc}} = \frac{P_{DL} l^3}{48EI} + \frac{5 w_{DL} l^4}{384EI}$$

$$= \frac{0.357 \times 10 \times 20^3 \times 1728}{48 \times 29000 \times 245} + \frac{5 \times 0.436 \times 20^4 \times 1728}{384 \times 29000 \times 245}$$

$$\Delta_{DL,\text{wet conc}} = 0.366\text{in} < 0.67\text{in} \checkmark \text{ GOOD!}$$

Hence, Interior Girder — W16 x 31
Exterior Girder — W14 x 26

DESIGN SUMMARY



ALTERNATIVE SYSTEM #3: NON-COMPOSITE WIDE FLANGE STEEL FRAME ON COMPOSITE DECK

The final alternative system was the design of a non-composite steel frame consisting of wide-flange members, which was evaluated for a 20'-0" x 21'-3" typical bay. The original structural slab + deck of 3 ¼" lightweight concrete over 2" x 18 gage metal deck (total structural slab depth = 5 ¼").

NOTE: The non-composite system was designed without considering the mechanical duct, which had to be passed through some members. This resulted in deeper and heavier wide flange shapes, thus increasing the weight of the system.

Advantages

- Lightweight system
- Works well with various lateral systems

Disadvantages

- Large overall depth
- Requires additional fire protection
- Expensive to construct

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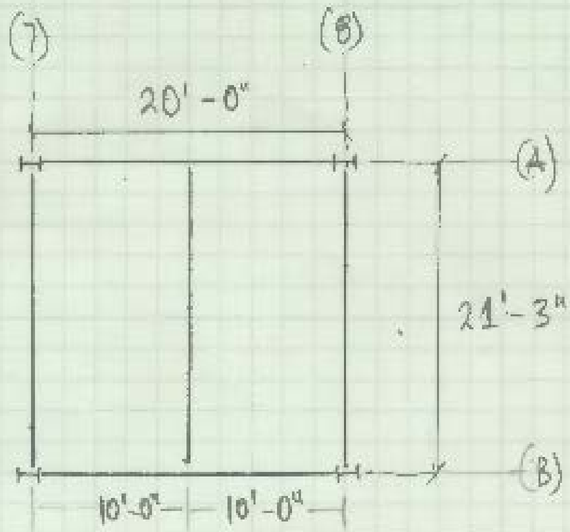
ALTERNATIVE SYSTEM 3
NON-COMPOSITE STEEL SYSTEM



Assumptions:

1. 2VLI x 18 GAGE DECK w/ 3 1/4 L.W CONCRETE
   (Achieves 2hr Fire Rating) – Composite Decking
2. SDL: 23 PSF   SLAB + DECK = 42 PSF   BEAM ALLOWANCE = 5 PSF
   LL = 100 PSF (+ PARTITIONS)

Span and Load Check for Deck

1. Span Check
   SDI Max Unshored Span = 12'-7" > 10" ✓ GOOD!
2. Superimposed Load Check
   W_{LL} + Superimposed DL ≤ Superimposed load
   123 ≤ 205 ✓ GOOD!

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CALCULATIONS FOR INFILL BEAM

Live Load Reduction: $L = 100 \times \begin{cases} 0.5 \\ 0.25 + \frac{15}{\sqrt{2 \times 425}} \end{cases} \; 0.764$

$\Rightarrow 100 \times 0.764 = 76.4\,PSF = LL$

$DL = 42 + 5 + 23 = 70\,PSF$

$W_u = 1.2DL + 1.6LL = [1.2(70) + 1.6(76.4)]10 = 2.06\,KLF$

$M_u = \frac{2.06 \times 21.25^2}{8} = 116.3\,ft\,k$

$V_u = \frac{2.06 \times 21.25}{2} = 21.9\,K$

Check Bending

Using Z_x tables (Table 3-2); $\boxed{\text{Try W14}\times 22}$

$\phi M_p = 125\,ft\,k > M_u = 116.3\,ft\,K$

Check Shear

$\phi_v V_{nx} = 94.5\,K > V_u = 21.9\,K$

Check Deflection $\quad I_x = 199\,in^4$

$\Delta_{LL} = \frac{5 w_{LL} L^4}{384 E I_x} < \frac{L}{360} = \frac{21.25 \times 12}{360} = 0.71$

$= \frac{5 \times .764 \times 21.25^4 \times 1728}{384 \times 29000 \times 199} = 0.607 < 0.71$ ✓ GOOD

$\Delta_{DL,\,wet\,conc} = \frac{5 W_{DL} L^4}{384 E I_x} < \frac{L}{360} = 0.71$

$= \frac{5 \times 0.47 \times 21.25^4 \times 1728}{384 \times 29000 \times 199} = 0.37 < 0.71$ ✓ GOOD

$\Delta_{TL} = \frac{L}{240} = 1.06 \qquad 0.607 < 1.06$ ✓ GOOD

Check Beam Wt. Assumptions:

Weight $= 22/10 = 2.2\,PSF < 5\,PSF$ ✓ GOOD

$\boxed{\rightarrow \text{USE W14} \times 22 \text{ INFILL BEAMS SPACED AT } 10'-0'' \leftarrow}$

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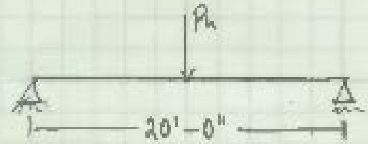
CALCULATIONS FOR INTERIOR GIRDER

Assumed Girder wt

Span = 20'-0" DL = 42 + 23 + 2.2 + 2 = 69.2
LL = 76.4 PSF [Reduced]

$P_u = [1.2(69.2) + 1.6(76.4)]\ 10' \times 21.25' = 43.6\text{K}$



$V_u = P_u/2 = 21.8\text{K}$ $M_{max,u} = \frac{P_u L}{4} = 218.0\ \text{ft K}$

Check Bending

Using Zx tables (Table 3-2). $\boxed{\text{Try W18x35}}$

$\phi M_n = 249\ \text{ftK} > M_u = 218.0\ \text{ftK}$ ✓ GOOD

Check Deflection $I_x = 510\ \text{in}^4$

$\Delta_{LL} = \frac{PL^3}{48EI} = \frac{0.764 \times 21.25 \times 20^3 \times 1728}{48 \times 29000 \times 510\ \text{in}^4} = 0.316\text{in}$

$0.316 < \frac{L}{360} = 0.67\text{in}$ ✓ GOOD

$\Delta_{DL,WET\ CONC} = \frac{0.462 \times 21.25 \times 20^3 \times 1728}{48 \times 29000 \times 510} = 0.19\text{in} < \frac{L}{360}$ ✓ GOOD!

$\Delta_{TL}: 0.316 < L/240$ ✓ GOOD

Check Girder Weight Assumption

Weight of Girder = 35/21.25 = 1.647 PSF < 2 PSF ✓ GOOD!

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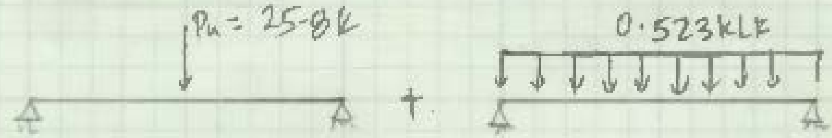
CALCULATIONS FOR EXTERIOR GIRDER

Span = 20'-0"   DL = 69.2 PSF + Distributed Wall Load
LL = 100 PSF

Wall Load = 436 PLF × 1.2 = 0.523 KLF = W_u

$P_u = [1.2(69.2) + 1.6(100)] \times 10 \times 10.625 = 25.8k$

Through superposition,



$V_u = \frac{25.8}{2} = 12.9k$ $V_u = \frac{0.523 \times 20}{2} = 5.23k$

$M_u = \frac{25.8 \times 20}{4} = 129k\,ft$ $M_u = \frac{0.523 \times 20^2}{8} = 26.15\,ft\,k$

$M_{u,TOT} = 129 + 26.15 = 155.1\,ft\,k$

Check Bending

Using Z_x tables (Table 3-2), $\boxed{\text{Try } W14 \times 30}$

$\phi M_n = 177\,ft\,k > M_u = 155.1\,ft\,k$

Check Deflection, $I_x = 291\,in^4$

$\Delta_{LL} = \frac{Pl^3}{48EI} < \frac{L}{360} = 0.67$

$= \frac{.1 \times 10 \times 10.625 \times 20^3 \times 1728}{48 \times 29000 \times 291} = 0.36in < 0.67in$ ✓ GOOD

$\Delta_{DL,WET\,CONC} = \frac{P_{DL} l^3}{48EI} + \frac{5 w_{DL} l^4}{384EI}$

$= \frac{0.462 \times 10.625 \times 20^3 \times 1728}{48 \times 29000 \times 291} + \frac{5 \times 0.436 \times 20^4 \times 1728}{384 \times 29000 \times 291}$

$0.167 + 0.186$

$\Delta_{DL,\,wet\,conc} = 0.353in < 0.67in$

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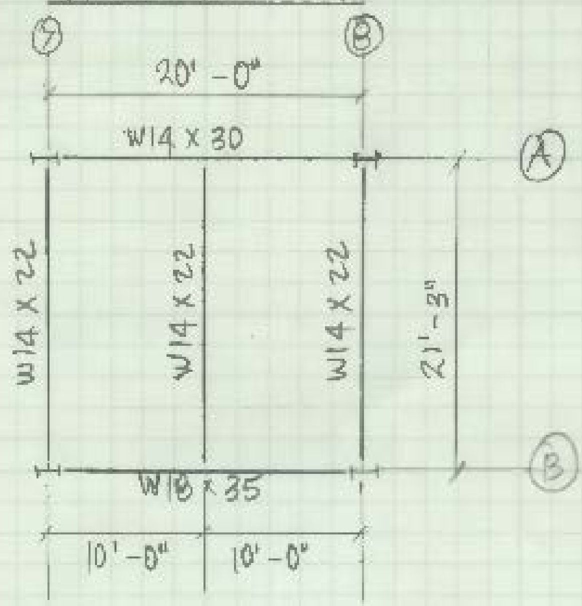
Check Girder Wt Assumption

Weight of Girder = $30/21.25 = 1.41 < 2$ ✓ GOOD

DESIGN SUMMARY



SYSTEM COMPARISONS

Criteria	Existing Composite Steel Framing	Reinforced Two-Way Flat Slab	Structural Steel Frame W/ Composite Joists	Non-Composite Structural Steel Frame
Weight (PSF)	57.8	87.5	43.2	53.1
Cost/SF	24.5	13.58	15.8	21.9
Depth (in)	23.25	16	18.5	23.25
Constructability	Medium	Medium	Easy	Medium
Fire Protection	NO	NO	NO	NO
Fire Rating	2 HR	2 HR	2 HR	2 HR
Future Considerations				
Lateral System Impact	N/A	YES	YES	YES
Additional Study Rqd?	N/A	YES	YES	NO
Possible Alternative	N/A	YES	YES	NO

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SYSTEM COMPARISON [WEIGHT CALCULATION]

EXISTING COMPOSITE STEEL

Deck = 42 PSF ; Beams: 3PSF + 2.2PSF + 5PSF = 10.2PSF
Girders: 2.35PSF + 3.29PSF = 5.6 PSF

TOTAL WT = 57.8 PSF

REINFORCED TWO-WAY FLAT SLAB

SLAB: 150 × 7/12 = 87.5 PSF

STRUCTURAL STEEL FRAME w/ COMPOSITE JOISTS

Deck: 35PSF JOISTS: 1.36 × 3 = 4.08PSF
Girders: 2.6PSF + 1.55 = 4.15PSF

TOTAL WT = 43.2 PSF

NON-COMPOSITE WIDE FLANGE STEEL FRAME

Deck: 42PSF Beams: 2.2 × 3 = 6.6PSF
Girders: 2.82 + 1.65 = 4.47PSF

TOTAL WT = 53.1 PSF

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COST ANALYSIS [RS MEANS ASSEMBLIES COST DATA] 2014

EXISTING SYSTEM

W Shape, Composite Deck and Slab, 20X20 Bay

TL = 177PSF   Cost = $24.5/sqft

REINFORCED 2-WAY FLAT SLAB

Cast-in-Place Flat Slab, 20X20 Bay   Slab th: 7in

Cost = $13.58/sqft

STRUCTURAL STEEL FRAME W/ COMPOSITE JOISTS

Steel Joists, Beams and Slab on Columns
20X20 Bay, TL = 119 PSF

Cost = $15.8/sqft

NON-COMPOSITE STRUCTURAL STEEL FRAME

W Shape, Composite Deck, 20X20 Bay, TL = 126PSF

Cost = $21.90/sqft