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

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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

Samantha deVries: Structural Option Advisor: Ali Said



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 courlesy of The George (Apartment)

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.

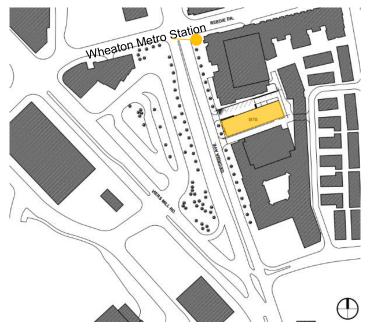


Figure 1: Building Location on Site, Courtesy of Bonstra Haresign Architects



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

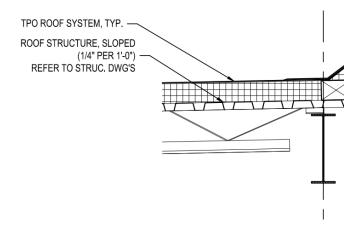
Standard Specifications for Composite Steel Joists

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.





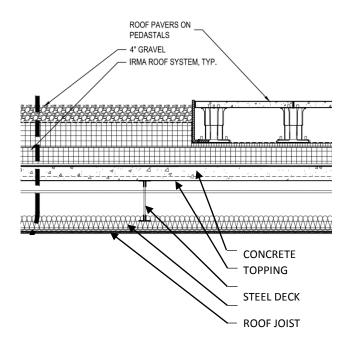


Figure 4: Section through roof at the 12th floor terrace level. From 3/A4.09.

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

Part	Deal Load	
1-001	Dead Load	
	Penemouse Root:	Load (pst)
		and chait
	Joist / Beam Allowance	10
	Roof Decking	10
	Roofing System	6
		27 pst
	12th Floor Terrace:	
		22
	Concrete Dece	37
	Joist / Beam Allowance	2
	4" rigid insulation Drop Ceiling	3
	MEP	15
	Sprinklers	3
	Payers or Tiles	25
	에 코의 바일 한 밤 밤 밤 밤 안 좀 좀 좀 할 것 .	[98 pst]
0		
Koo	F Live Load	
111	Penthouse Roof:	
	Controose coort	
	Code minimum is 20 pst	
	(table 4-1: Ordinary FL	at roote)
	Use 30 pet (value u	ned in design)
		2
	adda da la	
	12th Floor Terrace:	
	Table 4-1: Roots used for	
	TUBIC TT KDOIS USED YO	assembly porposes
	Use 100 pst) (same a	(autor mater a
	the second secon	
	* Note: drawing indicate that s	nous load must
	be used instead as the live	load must
	* Note: drawing indicate that so be used instead as the live it is the longer value	how load must load where
	be used instead as the live	load must
	be used instead as the live	load must

h Report 2	Snow Load	Samantha	deVries
ASCE 7-05 : 1	Chapter 7		
Section 7.3:	Flat Root Snow	Loads	
Pt = 0.7 C	e Ce I PS		
Po= 2:	5 pst (Figure 7	-1)	
Ce = 0	.9 (Table 7-2)) Terrain Categor Roof Fully Exp	y B losed
Ct = 1	1.0 (Table 7-3)		
I - 1.0) (табле 7-4)	Use w/ importan Category II	L-2
P1 = 0.7 ((0.9) (1.0) (1.0) (25) = 15.8 pst	
whin.	where p = > 7	20 pst 20 (1.0)	
(bt =		esign snow load = ? 30 pst LL on Pent	
Snow Drif	A Section 7.7.	Drifts on Lower R	coîs .
LE = Po	P3+14 = 0.13 (25) /8 = 15.8/17.25 5' -> he/hb =	+ H = 17.25 = 0.916 16.4 = 0.2 (must ca	re. drift)
	1 '851 = 1000		
100	eward drift (Fi	g. 7-9 w/128') " use larger valu (Fig. 7-9 w/40')	e
M9 <	he= 15, so w=	442 = 4 (3.75) = 15	,
pa = ha	aγ = 3.75 (17.25)		O pot LL on lever 12
	T	2 Pd = 64 7 pot	P1 = 20 P51
Jerrer		+ 15° +	1

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.

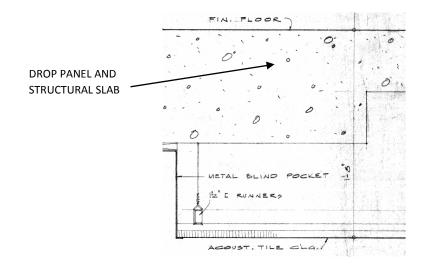


Figure 5: Section through typical floor in existing building. From A.12: Window & Wall Sections

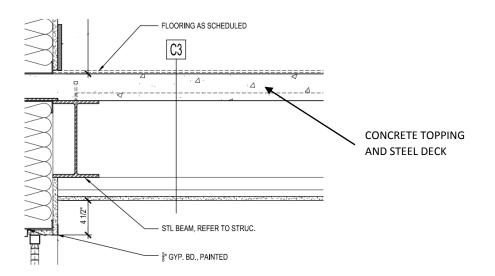


Figure 6: Section through typical floor in addition. From 10/A4.20

ch Repe	ort 2 Floor Loads	Samantha	delices
Floor	Dead Loads		
	Concrete Floor	Load (psf)	
	Drop Ceiling	5	
	MEP		
	Sprinklers	3	
++++	Concrete 6 12"	81.25	
	or 8" x150 pct	100	
++++			
++++	6 1/2" 6100: 105	PST	
	8" slab : 123	PST 1	
	Steel Framed Floors		
	Ceiting	5	
	MEP	15	
	Sprinklers	3	
	Beam/Joist Allowance	15	
++++	Concrete / Deck	37	
		120 1	
		75 pst	
111			1 1 1 1 1
Floor	Live Loads		
	Area	Code Min. (pst)	Design Valu
	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	-10	-10
	Note: Residential Areas also	o receive	
	a 20 pet portition		

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.

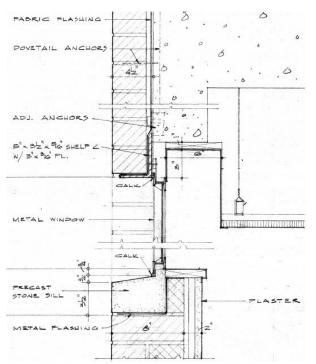


Figure 7: Section through typical exterior wall in existing building. From A.12: Window & Wall Sections

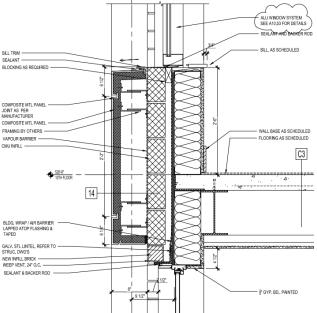


Figure 8: Section through typical exterior wall in addition. From 4A.21.

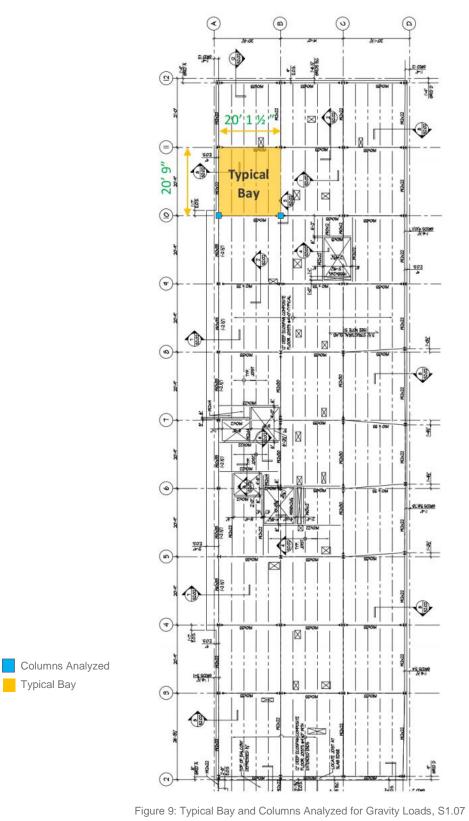
Wall Load Path

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.

lech	Report 2	Exterior	Wall	Loads	Samanthe	a delines
	Typical Existin	Building	Wall	Dead	Lood	
	Applied o	is a line	1000	4 64 4	he edge of	the slab
_	8" Brick	Layer (c	LOSUM	e hard	brick)	
	130	pct v 8	12 =	87	pet XII' typ.	= .957 pH
	3/4" laye	r gypsum	boord	4		
	50	pcf - 0.75/	12 *	11' =	34.4 pl	6
		Т	otas	= [99	2 pif	
	Typical Ad	dition W	all D	read	Lood:	
	Composi	e Melar F	Panel			
	5	58 × 11 ' =	55	P1F		
	CMU I	ntill (or	Brick	facad	e w/out me	lar panel)
	29	pot (chu) or	38	pst (brick) me	diwm weight)
		319 p19		418	p1F	
	Water N	rembrane				
	2.	11 × 709	= 23	- p1F		
	3/4" 3.	ypsum boo	bre	= 34	4 p/f	
		glass in	_			
		11 × 209			7	
		Total :	40	metal	panels = [443 pif
-			at	brick	faces = [487 pir]

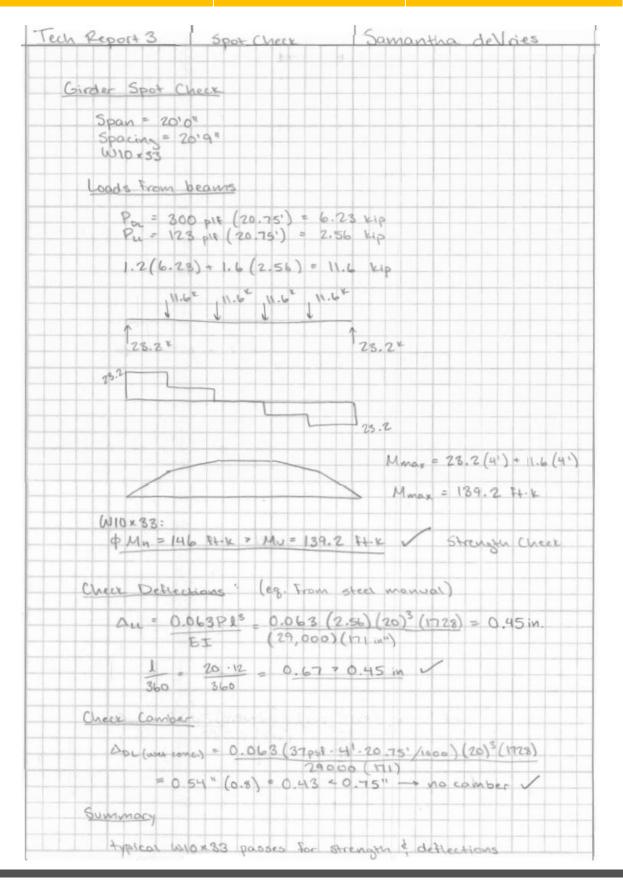
	TIT	Grawity Lo			L'ATTES
Non	- Typic	al Dead Load	15		
111	Floor	& Roofs:			
	At	3/4 " drop f			sting building
		3/4" × 150 p			
	Ex	isting Building	Perimeter	Beams	
		12" × 150 pct	× 12" width (avg.)	= [150 p)	£
		16" depth		= [200 F	91
		18"		= [225]	110
		24"		= [300 p'	7
111		30"		= [375	710
			beam siz	cs, so this	15
		of additi	onal Load	de a range	
+++	++++				
	111				
+++					





	+ - 1 - 1	ot Checks	Jaw	antha devries
	-			-
Gravity	, Check	Typico	1 Bay in	Existing Addition
A		B	e Ba	Maria C. S. Cari I
(I)-H-	W10×33		4	Typical for levels Through 11.
				· Bay infill members
c)			20'9"	are 12" deep composite theor
22×2114				joists.
	4'0",			· EVOL : 21/2" NW concret
(io)-H-	W10×33		-	1" deep, 24 gaze gab from deex, w/
1	20'0"			6×6-W2.1×W2.1 WWF
Live Lo	ad from	Tech 2: Tech 2:	Residential	-+ 40 pst
		Co	midors abi	ove 1st Floor - 40 pst
194044	State the			
Spot Chec				
Spot Chec	* Deck :	conform, 21	l gaze	
Spot Chec Using	x Deck : 1.0 C , CSV	Clear Span	l gaze	
Spot Chec Using Max C	<u>x Deck</u> : 1.0 C, CSV Construction			
Spot Chec Using Max C te Check	x Deck : 1.0 C, CSV Construction or NW con Wre 4	Clear Span crete, 3 span allowable to	= 5'10'	
Spot Chec Using Max C te Check	$\frac{x Deck}{1.0 \ c}, \frac{1.0 \ c}{0.000}$ Construction or NW con Wre = Wre = 75 +	Clear Span crete, 3 span allowable to 40 = 115 p	= 5'10'	
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Spot Chec Using Max C te Check	x Deck : 1.0 C, CSV ionstruction or NW con Wre = Jre = 75 + .0 C24, 39 Allowable w	Clear Span crete, 3 span allowable to 40 = 115 p pan, Fb= 36,1 wittoon load	= 5'10' $= 5'10'$ $= 191 psf$	
Spot Chec Using Max C te Check U	x Deck: 1.0 C, CSV Construction or NW con $W_{TL} =$ $W_{TL} =$ $1_{TL} =$ 75 + 0_{C2M} , 39 Allowable w $\leq c/180_{0}$	Clear Span crete, 3 span allowable to 40 = 115 p. pan, Eb=36,1 mitorm load or roat total	= 5'10' $= 5'10'$ $= 191 psf$	
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in ry	port 3 Spot Checks Samantha delices
501	Check Floor Joists
apor	CNEER FLOOR JOINTS
12	"deep ecospon composite Floor juists.
0	se Steel Joist Institute: standard specifications for Composite Steel Joists
DI	: 75 pst = 4 Ft tab width = 300 pit
LL	HO pot x u the trib. = 160 p/F
F	acrored Unitom Load:
	1.2 (300) + 1.6 (160) = 616 pif
1111	
	Using 25' joist span, 12" joist depth:
	for joist that is 8.0 p.4,
	total safe factored unit. dist. load = 700 pif = 616 pif
Live 1	load Reduction for girders:
	15
L	= Lo (0.25 + 15)
	이번도 왜 해야 했지만 좀 좀 잘 해야 할 것 하는 것 같이 잘 하는 것 같이 많이?
	Lo = 40
	$\begin{array}{c} L_{0} = 40 \\ K_{u} = 2 \\ A_{T} = 40 \\ \end{array} \left(0.25 + \sqrt{2(415)} \right) = 30.8 \text{ psf} \end{array}$
	30.8 ×4 14 trip = 123 pit on joist



Column Spot Checks Enterior Column Bio & Exterior Column A8 Interior WIOX49 (Level 6 to Penthouse) 18x18 cone (Level 1 to 5) 20x20 cone. (B2 \$ B1) Check oreal WIOX49 at base (worm-cose loading) Floor DL = 75 per Floor DL = 75 per 12th Level LL = 100 pr Poot bL = 27 per Poot bL = 27 per Poot Strow load = 20 per (L reduction) Hoor: L = 40 (0.25 + $\sqrt{4(415)})$ = 24.7 per 12th Level LL = 100 (0.25 + $\sqrt{4(415)})$ = 61.8 per Poot Strow load = 20 per Poot (L2[(75.6)+98.427) + 126[(24.7.6)+61.8+30] + 20) (415) / 1000 = 454 kip 4 = 0 (415) / 1000 = 454 kip	ch Report 3	Spot Checks	Samantha devices
Therior Column BIO & Exterior Column A8 Interior WIOX49 (Level 6 to Penthouse) 18x18 cone (Level 1 to 5) 20x20 cone, (82 \$ BI) Check steel WIOX49 at base (worst-case loading) Floor DL = 75 psf Floor LL = 40 psf 12th Level LL = 100 psf 12th Level LL = 100 psf Root DL = 27 psf Root DL = 27 psf Root Strow load = 20 psf UL reduction Floor : L = 40 (0.25 + $\frac{15}{\sqrt{4}(415)}$) = 24.7 psf Root strow load = 20 psf UL reduction Floor : L = 40 (0.25 + $\frac{15}{\sqrt{4}(415)}$) = 61.8 psf Pw (L2[(75.6) + 98.427] + Ub [(24.7.6) + 61.8.30] + 20) (415) / 1000 = 454 kip + weight of columns = 80' (49 prf)/1000 + 454 Pv = 458 kip Steel Manual table 4-1: Effective Length = 10'4"	0		all the second s
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WIDREG WIDREG 18x18 conc. (Level 1 to 5) 20x20 conc. (82 \$ B1) Check steel WIDREG at base (worst-case loading) Floor DL = 75 pst Floor DL = 75 pst 12th Level DL = 98 pst 12th Level DL = 98 pst 12th Level DL = 98 pst 12th Level DL = 100 pst 2 cot DL = 27 pst 12th Level DL = 30 pst 12th reduction: Ploor: L = 40 (0.25 + $\frac{15}{14(415)}$) = 24.7 pst 12th reduction: Ploor: L = 40 (0.25 + $\frac{15}{14(415)}$) = 61.8 pst 12th Level: L = 100 (0.25 + $\frac{15}{14(415)}$) = 61.8 pst Pu = (1.2[(75.6) + 98 + 27] + 1.6[(24.7.6) + 61.8 + 30] + 20) (415) / 1000 = 459 kip + 20) (415) / 1000 = 459 kip Pu = 958 kip Steel Manual table 4-1: Elterive Length = 10'4" dPh = 5441 kip > Pu = 458 kip /			
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Check steel WKX49 at base (worst-case loading) Floor DL = 75 pst Floor LL = 40 pst 12th Level LL = 40 pst 12th Level LL = 100 pst Pool DL = 27 pst Root DL = 27 pst Root Snow load = 20 pst LL reduction Hoor: L = 40 (0.25 + $\sqrt{4(415)}$) = 24.7 pst 12th Level: L = 100 (0.25 + $\sqrt{4(415)}$) = 61.8 pst 12th Level: L = 100 (0.25 + $\sqrt{4(415)}$) = 61.8 pst 12th Level: L = 100 (0.25 + $\sqrt{4(415)}$) = 61.8 pst Pu = (1.2[(75.6) + 98 + 27]) + 1.6[(24.7.6) + 61.8 + 30] + 20) (415) / 1000 = 454 kip + weight of columns = 80' (49 pit)/1000 + 454 Pu = 458 kip Steel Manual table 4-1: Effective Length = 10'9"	18×18 con	e (lever 1 to	5)
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Floor DL = 75 pst Floor UL = 40 pst 12th level DL = 98 pst 12th level UL = 100 pst Root DL = 27 pst Root LL = 30 pst Root Snow load = 20 pst UL reduction ' Floor : L = 40 (0.25 + $\sqrt{4}(415)$) = 24.7 pst 12th level : L = 100 (0.25 + $\sqrt{15}$) = 61.8 pst 12th level : L = 100 (0.25 + $\sqrt{15}$) = 61.8 pst Pw = (1.2[(75.6) + 98 + 27] + 1.6[(24.7.6) + 61.8 + 30] + 20) (415) / 1000 = 454 kip + weight of columns = 80' (49 prf)/1000 + 454 Pv = 458 kip Steel Manual table 4-1: Effective Length = 10'9"	C1		
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Floor U = 40 pst 12th level DL = 98 pst 12th level U = 100 pst Poot DL = 27 pst Root DL = 30 pst Root snow load = 20 pst U reduction Floor : L = 40 (0.25 + $\sqrt{4}(415)$) = 24.7 pst 12th level: L = 100 (0.25 + $\sqrt{15}$) = 61.8 pst 12th level: L = 100 (0.25 + $\sqrt{15}$) = 61.8 pst 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 61.8 pst 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 61.8 pst 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 61.8 pst 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 61.8 pst 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 61.8 pst 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 61.8 pst 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 61.8 pst 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 61.8 pst 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 61.8 pst 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 61.8 pst 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 61.8 pst 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 61.8 pst 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 61.8 pst 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 61.8 pst 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 61.8 pst 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 61.8 pst 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 61.8 pst 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 61.8 pst 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 61.8 pst 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 61.8 pst 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 61.8 pst 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 61.8 pst 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 61.8 pst 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 61.8 pst 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 7000 + 454 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 7000 + 454 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 7000 + 454 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 7000 + 454 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 7000 + 454 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 7000 + 454 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 7000 + 454 12th level: L = 100 (0.25 + $\sqrt{14}(415)$) = 7000 + 454 12th level: L =	C 1	D1 . 25	1.2 D+1.66 + 1.05
12th level DL = 98 pst 12th level LL = 100 pst Poot DL = 27 pst Root DL = 30 pst Root snow load = 20 pst LL reduction Floor: L = 40 (0.25 + $\sqrt{4(415)}$) = 24.7 pst 12th level: L = 100 (0.25 + $\sqrt{4(415)}$) = 61.8 pst 12th level: L = 100 (0.25 + $\sqrt{4(415)}$) = 61.8 pst Pu = (1.2[(75.6) + 98 + 27]) + 1.6[(24.7.6) + 61.8 + 30] + 20) (415) / 1000 = 454 kip + Weight at columns = 80' (49 pit)/1000 + 454 Pu = 458 kip Steel Manual table 4-1: Effective Length = 10'4" d Ph = 5444 kip > Pu = 458 kip	Floor	LL = 40 = t	
$\frac{12^{m}}{P_{00}t} = \frac{100}{P_{0}t} p_{0}t$ $\frac{P_{00}t}{P_{00}t} = \frac{27}{P_{0}t} p_{0}t$ $\frac{P_{00}t}{P_{00}t} = \frac{30}{P_{0}t} p_{0}t$ $\frac{P_{00}t}{P_{00}t} = \frac{30}{P_{0}t} p_{0}t$ $\frac{P_{00}t}{P_{0}t} = \frac{1}{12} = \frac{100}{P_{0}t} \left(0.25 + \frac{15}{P_{0}(u_{1}s_{1})}\right) = \frac{24.7}{P_{0}s_{1}} p_{0}s_{1}^{2}$ $\frac{P_{0}t}{P_{0}t} = \frac{100}{P_{0}t} \left(0.25 + \frac{15}{P_{0}(u_{1}s_{1})}\right) = \frac{61.8}{P_{0}s_{1}} p_{0}s_{1}^{2}$ $\frac{P_{0}t}{P_{0}t} = \frac{1.2[(75.6) + 98 + 27] + 1.6[(24.7.6) + 61.8 + 30]}{P_{0}t} + \frac{20}{P_{0}t} \left(\frac{49}{P_{0}t}\right) / 1000 = \frac{454}{P_{0}t} kip$ $\frac{P_{0}t}{P_{0}t} = \frac{458}{P_{0}t} kip$ $\frac{P_{0}t}{P_{0}t} = \frac{458}{P_{0}t} kip$ $\frac{P_{0}t}{P_{0}t} = \frac{458}{P_{0}t} kip$ $\frac{P_{0}t}{P_{0}t} = \frac{458}{P_{0}t} kip$ $\frac{P_{0}t}{P_{0}t} = \frac{10^{14}}{P_{0}t}$	FLOOR	LL = JU PSI	
Poot DL = 27 pst Root UL = 30 pst Root snow load = 20 pst UL reduction: Floor: L = 40 (0.25 + $\sqrt{4}(415)$) = 24.7 pst 12 ^m level: L = 100 (0.25 + $\sqrt{15}$) = 61.8 pst 12 ^m level: L = 100 (0.25 + $\sqrt{15}$) = 61.8 pst Pu = (1.2[(75.6) + 98 + 27] + 1.6[(24.7.6) + 61.8 + 30] + 20) (415) / 1000 = 454 kip + weight of columns = 80' (49 prf)/1000 + 454 Pu = 458 kip Steel Manual table 4-1: Effective Length = 10'4" dp Pn = 544 kip > Pu = 458 kip	12. 1	ever bus 18 pst	
Pool III = 30 pst Root Snow load = 20 pst II reduction Floor: L = 40 (0.25 + $\sqrt{4(415)}$) = 24.7 pst N ^m level: L = 100 (0.25 + $\sqrt{14(415)}$) = 61.8 pst Pw = (1.2[(75.6) + 98 + 27] + 1.6[(24.7.6) + 61.8 + 30] + 20) (415) / 1000 = 454 kip + weight of columns = 80' (49 pir)/1000 + 454 Pw = 458 kip Steel Manual table 4-1: Effective Length = 10'4" $d_{Ph} = 5444$ kip > Pw = 458 kip /	D- B	NL = 22 - 100 pst	
Roof snow load = 20 psf LL reduction: Floor: L = 40 (0.25 + $\sqrt{4(415)}$) = 24.7 psf N th level: L = 100 (0.25 + $\frac{15}{74(415)}$) = 61.8 psf Pw = (1.2[(75.6) + 98 + 27] + 1.6[(24.7.6) + 61.8 + 30] + 20) (415) / 1000 = 454 kip + weight of columns = 80' (49 psf)/1000 + 454 Pw = 458 kip Steel Manual table 4-1: Effective Length = 10'4" $P_{N} = 5444$ kip > Pw = 458 kip /	Root	11 = 30 -1	
U reduction Floor: L = 40 (0.25 + $\sqrt{4(415)}$) = 24.7 pst 12 th level: L = 100 (0.25 + $\frac{15}{\sqrt{4(415)}}$) = 61.8 psf Pv = (1.2[(75.6) + 98 + 27] + 1.6[(24.7.6) + 61.8 + 30] + 20) (415) / 1000 = 454 kip + weight of columns = 80' (49 pir)/1000 + 454 Pv = 458 kip Steel Manual table 4-1: Effective Length = 10'4" 4Ph = 544 kip > Pv = 458 kip /	Real	Synu land = 2	0 -14
$Floor: L = 40 \left(0.25 + \sqrt{4(415)}\right) = 24.7 \text{ psf}$ $12^{44} \text{ level}: L = 100 \left(0.25 + \frac{15}{\sqrt{4(415)}}\right) = 61.8 \text{ psf}$ $P_{u} = \left(1.2[(75.6) + 98 + 27] + 1.6[(24.7.6) + 61.8 + 30] + 20\right) (415) / 1000 = 454 \text{ kip}$ $+ 20 \left(415\right) / 1000 = 454 \text{ kip}$ $+ \text{ weight of columns} = 80' (49 \text{ pif}) / 1000 + 454$ $P_{u} = 458 \text{ kip}$ $Steel Manual table 4-1:$ $Effective Length = 10'4''$ $\Phi P_{u} = 544 \text{ kip} > P_{u} = 458 \text{ kip}$	1-001	021000 1000 - 2	o par
$Floor: L = 40 \left(0.25 + \sqrt{4(415)}\right) = 24.7 \text{ psf}$ $12^{44} \text{ level}: L = 100 \left(0.25 + \frac{15}{\sqrt{4(415)}}\right) = 61.8 \text{ psf}$ $P_{u} = \left(1.2[(75.6) + 98 + 27] + 1.6[(24.7.6) + 61.8 + 30] + 20\right) (415) / 1000 = 454 \text{ kip}$ $+ 20 \left(415\right) / 1000 = 454 \text{ kip}$ $+ \text{ weight of columns} = 80' (49 \text{ pif}) / 1000 + 454$ $P_{u} = 458 \text{ kip}$ $Steel Manual table 4-1:$ $Effective Length = 10'4''$ $\Phi P_{u} = 544 \text{ kip} > P_{u} = 458 \text{ kip}$	LL reduc	han'	
$\frac{12^{m} \text{ kevel}}{12^{m} \text{ kevel}} = \frac{100}{12^{n} (225 + \sqrt{4(415)})} = 61.8 \text{ psf}}$ $P_{v} = (1.2[(75.6) + 98 + 27] + 1.6[(24.7.6) + 61.8 + 30]$ $+ 20) (415) / 1000 = 454 \text{ kip}$ $+ \text{ weight of columns} = 80^{\circ} (49 \text{ pnf})/1000 + 454$ $P_{v} = 458 \text{ kip}$ $5teel Manual table 4-1:$ $Effective Length = 10^{\circ}4^{m}$ $\Phi P_{n} = 544 \text{ kip} > P_{v} = 458 \text{ kip} \checkmark$			15
$\frac{12^{m} \text{ kevel}}{12^{m} \text{ kevel}} = \frac{100}{12^{n} (225 + \sqrt{4(415)})} = 61.8 \text{ psf}}$ $P_{v} = (1.2[(75.6) + 98 + 27] + 1.6[(24.7.6) + 61.8 + 30]$ $+ 20) (415) / 1000 = 454 \text{ kip}$ $+ \text{ weight of columns} = 80^{\circ} (49 \text{ pnf})/1000 + 454$ $P_{v} = 458 \text{ kip}$ $5teel Manual table 4-1:$ $Effective Length = 10^{\circ}4^{m}$ $\Phi P_{n} = 544 \text{ kip} > P_{v} = 458 \text{ kip} \checkmark$	Floor :	L= 40 0.25+	= 74.7 pst
$P_{v} = (1.2[(75.6) + 98.427] + 1.6[(24.7.6) + 61.8+30] + 20) (415) / 1000 = 454 kip + weight of columns = 80' (49 prf)/1000 + 454 P_{v} = 458 kip Steel Manual table 4-1:Effective Length = 10'4"\Phi P_{n} = 5444 kip \ge P_{v} = 458 kip \sqrt{2}$			
$P_{v} = (1.2[(75.6) + 98.427] + 1.6[(24.7.6) + 61.8+30] + 20) (415) / 1000 = 454 kip + weight of columns = 80' (49 prf)/1000 + 454 P_{v} = 458 kip Steel Manual table 4-1:Effective Length = 10'4"\Phi P_{n} = 5444 kip \ge P_{v} = 458 kip \sqrt{2}$	12th level	· L= 100 0.25	+ 15) = 61.8 psf
+ 20) (415) / 1000 = 454 kip + weight of columns = 80' (49 pir)/1000 + 454 Pu = 458 kip Steel Manual table 4-1: Effective Length = 10'4" $\Phi P_{M} = 5444$ kip $P = P_{M} = 458$ kip V		-	
+ 20) (415) / 1000 = 454 kip + weight of columns = 80' (49 pir)/1000 + 454 Pu = 458 kip Steel Manual table 4-1: Effective Length = 10'4" $\Phi P_{M} = 5444$ kip $P = P_{M} = 458$ kip V	Pu= (1.2	(75.6) + 98 - 27]	+1.6 (24.7.6)+61.8+30]
+ weight of columns = 80° (49 prf)/1000 + 454 Pu = 458 kip Steel Manual table 4-1: Effective Length = $10^{\circ}4^{\circ}$ $\Phi P_{n} = 5444$ kip > $P_{u} = 458$ kip V			
Pu = 458 kip Steel Manual table 4-1: Effective Length = 10'4"		+ 20) (415) /	1000 = 454 kip
Pu = 458 kip Steel Manual table 4-1: Effective Length = 10'4"			
Steel Manual table 4-1: Effective Length = $10'4''$	+ W3	sight of columns	= 80' (49 pir)/1000 + 454
Steel Manual table 4-1: Effective Length = $10'4''$			
Effective Length = 10'4" & Ph = 544 kip > Ph = 458 kip V	FUE	158 Kip	
Effective Length = 10'4" & Ph = 544 kip > Ph = 458 kip V	E		
¢ Pn = 544 kip > PN = 458 kip √	oteel Mai	huas table 4-1	
¢ Pn = 544 kip > PN = 458 kip √	Ettery	1 - 1 - 10141	
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	d.P.	= 544 Vin 7 D	12458 40 1
column passes strength check	- in	our rip is	3-130 FIP V
	COLUMAN	passes strange	check
		, July and Stor	

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5.		
Exterior		
(A)10×33	(Level 6 to Per	thouse
18×14 com	(Level) to 5	
18×16 (mar	c (Level 1 to 5) (B1) 18×271	2 (04) (82)
	(51)	
Check ste	et wiax 33 as bas	e (worst-case)
Uniform	n Loods same o	is previous interior
IL Reduce	1	
Floor:	L= 40 (0.25+ F	15 1(207.5)) = 30.8 pst
17441	1 1 - 100/	$\frac{15}{\sqrt{4}(207.5)} = 77.1 \text{ pot}$
10.100	er · L = 100 (0.25	Tu (apra e) = 11.1 pet
12	is unreducible	1-1 (201.3) 1
P-COF)	15 UNTEQUENDER	
0=11251	1751)+00-22 +1	6[(30.8.6)+77.1+30]
10 (1.66	12.2) + JStril 1	6[100.8 6] - 17.13 30]
+7	0) (207.5) /100	0 = 744 kin
	-)	er op
+ (0)	unn self-weight +	ext. wall weight
	3	
= 244	Kip + 801 (33 p))	1000 + 487 pif (2075) / 100
Pu = 257	Kip	
Table 4-1	:	
Effectiv	re length = 10'4'	· · · · · · · · · · · · · · · · · · ·
¢Pn=	323 Kip > 0Pu:	= 257 kip
column po	asses strength che	<u>.x</u>

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.

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	DeFie	1240 1360	(70) (70)	r totel	unte logd	uniau	l lood	s) = (20 = (. = (20.7 = 0.	.75×1 037 5×12)	1/31	
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	DeFie	1240 1360 D:	(70) (70) (70) - L =	live live lise pst (unte load st (u '4') :	uniael 41) = 161	l lood (leno 460 Ng C	s) = (20 = (. = (20.7) = 0. plf	.75×1 037 5×12)	1/31	
	DeFie	1240 1360 D:	(to. (to. - 40	live live pst (unte load sp (u (4')	uniou uniou t') = llou T =	(1000 (0000) 460 2 p19 5 W	(20) = (20) = (1) = (1) = (20) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (.75×1 037 5×12)	1/31	
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	DeFie	1240 1360 D: D: D:	(to. (to. = 40 = 5 89	live live lise pot (wly SY ET	unte load sf (c 4')	1) = 161 T =	460 0 pl 5 w 3848	e) = (20 = (. = (20.7 = 0. = 0. p)F = 0.	-75×1 037 5×12 691	in. / 31	
	DeFie	1240 1360 D: D: D:	(10) (10) - 40 = 40 = 59 = 40	Live Live LIS p pst (Live LIS p st (Live LIS p S (0.41	unte load sf (c ('4')	untael +1) = = 160 T = 0.75)"	H60 D plf 5 W 384 E (1728	(20) = (20) = (1) = (1) = (20) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (1) = (-75×1 037 5×12 691	in. / 31	
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	DeFie	1/240 1/240 1/360 D: L: Omex I on	(to (to = 40 = 5 37 = 3	115 p 115 p psf (wl 54 EI 5 (0.41 84 (29)	unte load st (u (4)) (2)(2 000)	uniou 1') = 1') = 1'') = 1'') = (1.03:	460 2 pl 3848 (1728)	s) = (20 = (. = (20.7 = 0. p)F = 0. p)F = 0.	-75×1 037 5×12 691	1731 1731	
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	DeFu J	1/240 1/240 1/360 D: L: Omex I on	: (to) (to) = 40 = 5 = 3 = 5	115 p live 115 p pof (wl 4 5 (0.41 84 (29) (0.160 384 (2	unte load (1) 12 (4)) (20) (20) (20) (20) (20) (20) (20) (2	uniael 1') = 1') = 1'') = (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03: (1.03:	460 0 pl 5 u 3848 (1728) 7) 1728) 21) 1728)	s) = (20 = (. = (20.7 = 0. p)F = 0. p)F = 0.		18. 18. 18. 19. 19. 19. 19. 19. 19. 19. 19	

ECN 1	Kepper 3	Alternate #1 Samantha delives
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	Mu= wal	2 = 3.2 (20)2 = 160 H-K
		8
	Deflecti	
	Dencen	
	L/24	10 = (20×12)/240 = 1.0 in
	L/36	$0 = (20 \times \sqrt{2})/360 = 0.67 \text{ in}$
	D.	1 = 115 est (2075) = 2.39 410
	L	-L = 115 por (20.75) = 2.89 KIR = 40 per (20.75) = 0.83 KIR
		5w1"
		SHEDMAX
	Ta	= 5 (2.39) (20)" (1728) = 296.7 in"
	- 140	384 (29000) (1.0)
-	I 360	$= 5(0.83)(20)^{(1728)} = 124.1 \text{ in}^{(1728)}$
		384 (29000) (0.83)
	W16×26	- I = 301 in" > 296.7 in"
		= 4Mn= 166 Ft.K 4 160 H.X V
		thin @ unbraced length = 165 Hix
No	n Composi	te Etect Summary:
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1	1-1- Wiex	1.400
		Deck: 21/2" NW concrete
	pene	1" deep ribs, 24 gage
	a	w/6x6 -W2.1x202.1 WWF
5		
3		4.0"
	h1×210	typ spoking
	3	
+		
	031982	20' ON 1

lech	Report	3	Altery	nate 1	+2	50	amant	ha	delies	
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	The	ctore	stad	HIM	n a	6 1/2"	slab			
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	++++		-			0	11	1	.5	+
		+++								-
								25		

- Cin	Report 3 Alternate #2 Samantha devries
С	here one-way shear:
	Nu = 0,190 (8'7")(20.75') = 33.8 K
	$V_{c} = 2 \int f_{c} b_{w} d = \frac{2}{2} \int \frac{4000}{1000} (20.75' \times 12)(5.0) = 157.5 \times 1000$
	+Vc=0.75(157.5) = 118.1 K > 33.8 K
С	heer two-way shear:
1	$N_{0} = 0.190 (20(20.75) - (29/12)^2) = 77.7 \kappa$
	$X_6 = 40$ for interior B = 20.75 / 20 = 1.04 $B_0 = 4(29") = 116"$
	$N_{c} = \sqrt{4000} (116)(5) \left(2 + \frac{4}{1.04} \right) = 5.85$ $\frac{40(5)}{1000} \left(\frac{40(5)}{116} + 2 \right) = \frac{3.72}{1.72} = 136.5$
	win 4
	4Vc + 0.75(136.5) = 102.3 k > 77.7 k
	- stab works for shear without drop pavels
Rei	check slob minimum thickness without drop panels:
	back to table 9.5 (c)
	ext. panel w/ edge beams : ln/33 int. panels: ln/33
	ln/33 = (18'9")(12)/33 = 6.82 -> USE 7" slab W/out drop
	panels
tt	

Determine \$100 Moments Dead Load $u/7"$ state = 105 + $\frac{0.5}{12}(150) = 111.3 \text{ ps}$ $g_{u}=1.2(111.3)+1.6(90) = 197.6 \text{ psf}$ $M_{0} \frac{1}{100} = \frac{9}{8}ul_{11}l_{11} = 197.6(20.75)(18)^{2} = 166.1 \text{ ft} \text{ k}$ $\frac{1}{41} = \frac{8}{8} = \frac{197.6(20)(18.75)^{2}}{8} = 173.7 \text{ ft} \text{ k}$ $\frac{1}{41} = \frac{9}{8}ul_{11}l_{11} = 197.6(20)(18.75)^{2} = 173.7 \text{ ft} \text{ k}$ $\frac{1}{41} = \frac{9}{8}ul_{11}l_{11} = 197.6(20)(18.75)^{2} = 173.7 \text{ ft} \text{ k}$ $\frac{1}{617} = \frac{9}{8}ul_{11}l_{11} = \frac{197.6(20)(18.75)^{2}}{8} = 173.7 \text{ ft} \text{ k}$ $\frac{1}{607} = \frac{9}{8}ul_{11}l_{11} = \frac{197.6(20)(18.75)^{2}}{8} = 173.7 \text{ ft} \text{ k}$ $\frac{1}{607} = \frac{9}{8}ul_{11}l_{11} = \frac{197.6(20)(18.75)^{2}}{8} = 173.7 \text{ ft} \text{ k}$ $\frac{1}{607} = \frac{9}{8}ul_{11}l_{11} = \frac{197.6(20)(18.75)^{2}}{8} = 173.7 \text{ ft} \text{ k}$ $\frac{1}{607} = \frac{9}{8}ul_{11}l_{11} = \frac{197.6(20)(18.75)^{2}}{8} = 173.7 \text{ ft} \text{ k}$ $\frac{1}{607} = \frac{9}{8}ul_{11}l_{11} = \frac{197.6(20)(18.75)^{2}}{8} = 173.7 \text{ ft} \text{ k}$ $\frac{1}{607} = \frac{9}{8}ul_{11}l_{11} = \frac{197.6(20)(18.75)^{2}}{1007} = 173.7 \text{ ft} \text{ k}$ $\frac{1}{607} = \frac{9}{8}ul_{11}l_{11} = \frac{197.6(20)(18.75)^{2}}{1007} = 173.7 \text{ ft} \text{ k}$ $\frac{1}{607} = \frac{9}{8}ul_{11}l_{11} = \frac{197.6(20)(18.75)^{2}}{1007} = 173.7 \text{ ft} \text{ k}$ $\frac{1}{607} = \frac{9}{8}ul_{11}l_{11} = \frac{197.6(20)(18.75)^{2}}{1007} = 173.7 \text{ ft} \text{ k}$ $\frac{1}{607} = \frac{9}{8}ul_{11}l_{11} = \frac{197.6(20)(18.75)^{2}}{1007} = 1007 \text{ ft} \text{ k}$ $\frac{1}{607} = \frac{9}{6}ul_{11}l_{11} = \frac{197.6(20)(18.75)^{2}}{1007} = 1007 \text{ ft} \text{ k}$ $\frac{1}{607} = \frac{9}{6}ul_{11}l_{11} = \frac{197.6(20)(18.75)^{2}}{1007} = 1007 \text{ ft} \text{ k}$ $\frac{1}{607} = \frac{9}{6}ul_{11}l_{11} = \frac{197.6(20)(18.75)^{2}}{1007} = \frac{1007}{10} \text{ ft} \text{ k}$ $\frac{1}{607} = \frac{1000}{10} \frac{1000}{10} \frac{1000}{10} = \frac{1000}{10} \frac{1000}{10} = \frac{1000}{10} \frac{1000}{10} = \frac{1000}{10} \frac{1000}{10} \frac{1000}{10} = \frac{1000}{10} 10$	ch Report 3	Alternate #	2 Sama	ntha delines
Dead Load w/7" stab = $105 + \frac{0.5}{12}(150) = 111.3$ ps $g_0 = 1.2(111.3) + 1.6(40) = 197.6 \text{ psf}$ $M_{0}_{\text{short}} = \frac{g_0 l_1 l_n^2}{8} = \frac{197.6(20.75')(18)^2}{8} = 166.1 \text{ ft.k}$ $M_{0}_{\text{torn}} = \frac{g_0 l_2 l_n^2}{8} = \frac{197.6(20)(18.75)^2}{8} = 173.7 \text{ ft.k}$ $dir = \frac{197.6(20)(18.75)^2}{8} = 173.7 \text{ ft.k}$ Longitudinal Moment: Bay meets requirements to use Exterior beam, no interior beams (Exterior Span Long Direction 11 + 8.5 122 ft.k $52 ftNo beams (Interior Span) Short Direction10.35$ $58 ft.k$	Determine SI	ab Moments		
$\begin{array}{c} 9u = 1.2(111.3) + 1.6(40) = 197.6(20.75)(18)^{2} = 166.1 + 1.k \\ Mo_{short} = 9ul_{1}l_{1}^{2} = 197.6(20.75)(18)^{2} = 166.1 + 1.k \\ Mo_{ton} = 9ul_{2}l_{1}^{2} = 197.6(20)(18.75)^{2} = 173.7 + 1.k \\ dic = 8 \\ & 8 \\ & 8 \\ & & 8 \\ & & & 8 \\ \hline \\ Longitudinal Moment : Bay meets requirements to use \\ & & & & \\ \hline \\ Exterior beam, no interior beams (Exterior Span long Direction \\ & & & & \\ \hline \\ & & & & & \\ \hline \\ & & & &$			0.5/	
$Mo_{short} = g_{u}l_{1}l_{n}^{2} = \frac{197.6(20.75^{\circ})(18)^{2}}{8} = 166.1 \text{ Ft.k}$ $Mo_{air} = g_{u}l_{2}l_{n}^{2} = \frac{197.6(20)(18.75)^{2}}{8} = 173.7 \text{ Ft.k}$ $Mo_{air} = g_{u}l_{2}l_{n}^{2} = \frac{197.6(20)(18.75)^{2}}{8} = 173.7 \text{ Ft.k}$ $Longitudinal Moment: Bay meets requirements to use$ $Exterior beam, vio interior beams (Exterior Span long Direction)$ $= \frac{1}{1000} \frac{160.5}{122} \frac{122 \text{ Ft.k}}{122} \frac{122 \text{ Ft.k}}{122} \frac{52 \text{ Ft.k}}{1000}$ $= \frac{1}{1000} \frac{1000}{1000} \frac{1000}{$	Dead Loo	d w/7" slab	= 105 + ((20) = 111.3 pot
Maton = gulila ² = 197.6 (20) (18.75) ² = 173.7 H+x dic 8 Longitudinal Moment: Bay meets requirements to use Exterior beam, via interior beams (Exterior Span Long Direction 11 + 6.5 11 + 6.5 122 H-x 52 H No beams (Interior Span) Short Direction + 0.35 10.35 58 H-x - 0.35 58 H-x	V			
Longitudinal Moment: Bay meets requirements to use Exterior beam, no interior beams (Exterior Span Long Direction 46.5 -0.7 -0.3 122 ttik No beams (Interior Span) Short Direction +0.35 58 ttik			0	
Exterior beam, no interior beams (Exterior Span Long Direction -0.7 1 -0.3 122 Hik 52 H No beams (Interior Span) Short Direction +0.35 58 Hik	Maton = dic	Julila - 197	.6 (20)(18.75) ² 8	= 113.7 ft.k
Long Direction +8.5 -0.7 L -0.3 122 Hill No beams (Interior Span) Short Direction +0.35 58 Hill -0.35 58 Hill	Longitudino	Moment: 8	bay meets requir	ements to use
-0.7 -0.3 122 Hix 52 H No beams (Interior Span) Short Direction	Exterio	r beam, no int	rerior beams	(Exterior Span)
+6.5 -0.7 -0.3 122 H.K 52 H No beams (Interior Span) Short Direction +0.35 58 H.K	Lons	DIRECTION		
-0.7 -0.3 122 H.K 52 H No beams (Interior Span) Short Direction +0.35 58 H.K		+6.5		87 ++ 2
No beams (Interior Span) Short Direction	1/1/1/		100000	
+0.35 58 H.x	-0.711	-0.3	122 \$t+.K	52 F+1
+0.35 58 H.x		1 1		
	No bea	ms (Interior S	pan) Short Di	rection
and the second sec		+0.35	58	4.12
-0.65 -0.65 108 Hit 108 Hit 108 Hit			and the second	
	-0.65	-0.65	108 Ft-12	108 41-2
	-0.65	-0.65	108 Frie	10'8 \$4.2
	-0.65	-0.65	108 Fr.12	10'8 84.2
	-0.65	-0.65	108 Fr.16	10'8 \$4.2
	-0.65	-0.65	108 Frie	103 4.2
	-0.65	-0.65	108 Fr. K	103 84.2
	-0.65	-0.65	108 Fr. 12	10'8 14.2
	-0.65	-0.65	108 Fr. K	10'8 84.2
	-0.65	-0.65	108 Fire	10 8 84.2

Colomn	ž Middle	e Stri	p Mov	nents			
de:	Xs, Lavos	Ston	5 directi	= (noi	0 (n	o bea	ms)
0/ ₁ =	Eco Ib Eco Is	->	Ecb = E	45 - >	× F1 =	Ib cI	(along she direction
For vi	ous, try	16" d	eep b	eam i	k width	5 to	O" columns
The part of the pa		1	Ţ		·20) + 1		
9				ý = 9.	75"		
+	20" 1 9	r	Ţ	1:	(9)(7)	3 .	1.75") ² (9)(7)(2.75
					12	Tb	= 8540 in4
15=	(12×10.4	(7)		356	in 4		
×F'	= <u>8540</u> 3567	- 2.	4				
l 2/1	- 20		0.96	24 J.			
C = 2	(1-0.63	$\left(\frac{x}{y}\right)^{2}$	x*y 3				
		C =	(1-0.6	3(20))	$\left(\frac{16}{3}, \frac{20}{3}\right)$	3)	= 14069
		C+	(1-0.+(63 (7)	$\left(\frac{-7^{5}20}{3}\right)$ $3\left(\frac{9}{20}\right)$	9 3 20) = 6294
β.=	Edo C ZEKS IS						

ech Ke	port 3	Alternas	e #2	Saman	tha d	elvics
			part of the last			
1	Direction	3				
Cons	Ducence					
			XF, = 0			
17771	XIV7	82	Vit, U			
1/1	A A A	11			0	
VVV	NN		10 of Me	3. moment	- Co INF	. support
	NY		0.			
VA	Nil		as xc/	2,=0 ->	15°/0	
1/1A	Nik					
1/43	NI					
1AA	XXX		% & P!	ositive mon	nent	- top of the
1/i	TY VC					
MS	CS	MS	de l	1/2,=0-	60%	
			4-4A	121		
			0/0 5 00	y - moment	123 0	SNOP
			10 01 10	2 montene	C SAT	anthora
			0 0	2.0 - 5	20 9/	
			pr	2.0	00 10	
= 1		7501	0.0	P. I.		
54	++·K -	- 75°/0 (> 25°/0	5 - 57	++ 1 16		
		\$ 25%	MS = 13	Ft·K		
8	7 H.K	> 60% ce	= 52.2	ft.k		
	6	· 40% /	15 = 34.8	ft-k		
12	2 H·K	> 80% c: > 20% x	5 = 97.6	Ft-K		
		> 20% 1	15 = 24.4	4+·K		
Sland	Directio					
Colora			00 0 2	H BTE	2.0	
			un - 2	PT		
1111	1AXV	771	0/		1	
11/1	1810	V.CS	10 neg	moment @	De Fal	pport
-1-1-1-	5 5 5 5 5	13811	INTER	polate -	18.5%	
(VXX	XXXX	KAXLE	0/			
	11111	V/M5	To posi	tive moment	14	
	1111		inter	polate	2010	
- V//V	1/1	KA				
(AA	AAX	KA CS	108 (int	enor) I 2	8.5% CS	= 84.8 44
1///	XXVV	KA T		\rightarrow 2	1.5% M	5 = 23.2 ++. 1
58	4-K -++	72º10 CS	= 41.8	FL-K		
		28º/0 MS				
100	1000 1	785 .1	15 = 94 95	× 85% to b		12.1 4.4
100	(10.5 18	0.5 01.04	15% to	slah = 1	2744
			and the second second second	1 3 10 10	010.0	LAN TO A PA
	6	21.5% M	6 = 72 7 1			

A Report 3					dellies
Summary	of 2-way	slab w	concents :	(ft.k)	
Direction	Strip	Left Sic	te Mid	die Rie	sut Side
Long	Column	97.6		2.2	39 39
covis	Middle	24.4	34	.8	13
	Col. (6106)	12.7	41.		84.8
Short	(beam)	72.1	N/	A	N/A
	Middle	23.2	16	.2	23.2
Flexural					
rexoral					
lone bo	ah : an	h-1.1 =	59"		
shart 1	oors dæ	h-1.7 =	5.3"		
As min >	0.001864	≈ 0.00\	ซ (เ2')(า") = 0.15	2 in²/\$\$
Δ. =	Mu .12 + f'y jd	\$ 14	= 0.9		
1000	df'wid	t,	y = 60 KBi		
	1.270		1 = 0 95 d		
per Ft	- Abres /1	5' V	= 0.951	(5.9") = =	5.61 (10m) 5.04 (shore)
			= 0.95	(5,3") = 4	5.04 (short)
		4 Side			
Direction	Strip	Mornent 97.6	Asreg	Bor Size	Steel Area
Long	Column	97.6	0.386	#6	0.44
	Middle	29.4	0.097	#3	11.0
Short	Column		0.057	*3	0.11
	Middle	23.2	0.002	= 3	0.11
	M	ddie (F	Sottom)		
Direction	Strip	Moment	Asrez	Bor Size	Steel Are
Lons	Column	52.2	0.200	44	0.2
	Middle	34.9	0.188	# 4	0.2
Short	Column	41.8	0.185	#4	0.2
	Middle	16.2	0.072	#3	0.11
	Ric	M Side	(TOP)		
			Asrea	Ba Size	Sheel Are
Direction	Strip	Morment	12121		
	Column	39	0.155	#4	0,2
Direction	Strip Column Middle		0.155	* 3	0.11
	Strip Column Middle Column	39 13 84,8	0.155	* 3	0.11
Long	Strip Column Middle	39	0.155	* 3	0.11
Long Short	Strip Column Middle Column Middle	39 13 84.8 23.2	0.155	* 3	0.11
Long Short	Strip Column Middle Column	39 13 84.8 23.2	0.155 0.052 0.374 0.102	* 3	0.11

in Report 3	Atemate #2		mantha dellies
+ + + + + + + + + + + + + + + + + + + +			
00	2	1	
Perimeter Beau	in Design tor	Caronista	
Cher des	Va haard	1. 24 . 22	
Choose acp	th based on	dementions.	
Taul Q 5/	A. NOT -	here here	M.R. Mused
	a) in ACI - mi		
in direct	both ends c tion of begin)	ananana 1	incor ouy
2/21 =	(20' . 12")/2	= 11.4 1	use 12"
A 16" beam	, was used in	alab destav	Using a 12"
beam way	ild affect the	distribution	of moment.
			led to create
whoment to	ames for the 1	ateral system	m. For this
report, al	2" beam will	be used as	an initial
size to ch	neck gravity eff	let's only.	ا کا
	0 1		
Strength	Check: Since	e columns	are 24", use 24" wid
	be	and For the	is report
1 inch	and the state	1 1 1	
Mon	hent in beam	due to a	high ribution
of e	slab loods fro	in previous	portion = 72.1 Hik
1 payof 1			
Bea	un weight = 1	50 pcf * 12" *	24" = 300 pif
Ext.	moll lood =	487 plf	2) = 945 pit factored
lot	at live Load	1 = 181 (1-	2) = 173 pit factored
		+++++++	
		I Inter	ior Span:
			rents From ACI coefficient
	MIM	-1044	the the the
VIIIA	and the second	Ales	= wl = 0.945 (20)2
		1 meg.	TI TI
			= 244 4.4
			= 34.4 H.K
		Pos.	
		Bos.	
		Pos.	
		Pos.	= 84.4 H·K = $\frac{WL^2}{1b} = \frac{0.945(70)^2}{1b}$ = 23.6 H·K
Total C	proservative Mu		$= \frac{\omega l^2}{1b} = \frac{0.945(20)^2}{1b}$ = 23.6 H.4
		, = 72.1 + 34	$= \frac{WL^{2}}{1b} = \frac{0.945(70)^{2}}{1b}$ = 23.6 H.V .4= 106.5 H.K
A	the d - it	= 72.1 + 34	$= \underbrace{wl^{2}}_{1b} = \underbrace{0.945(20)^{2}}_{1b}$ $= 23.66 \text{ H} \cdot \text{K}$ $= \underbrace{106.5}_{15} \text{ H} \cdot \text{K}$
A	the d - it	= 72.1 + 34	$= \frac{WL^{2}}{1b} = \frac{0.945(70)^{2}}{1b}$ = 23.6 H.V .4= 106.5 H.K
Approxive	ate d - if 12-1.5 dear c	= 72.1 + 34 = 9 bars ouer - 0.5	= wl ² = 0.945 (20) ² 1b 23.6 H.4 4= 106.5 H.K Used = = + 65 m.p - 2(0.5)= 9.
Approxive	ate d - if 12-1.5 dear c	= 72.1 + 34 = 9 bars ouer - 0.5	= wl ² = 0.945 (20) ² 1b 23.6 H.4 4= 106.5 H.K Used = = + 65 m.p - 2(0.5)= 9.
Approxive	ate d - if 12-1.5 dear c	= 72.1 + 34 = 9 bars	$= \underbrace{wl^{2}}_{1b} = \underbrace{0.945(20)^{2}}_{1b}$ $= 23.66 \text{ H} \cdot \text{K}$ $= \underbrace{106.5}_{15} \text{ H} \cdot \text{K}$

La report o	Alterno	ARE 6	TI	aman-	Ma arel	nes
		h harden				
Beam 5	hear Check					
Lane	Load = 945	PIF				
W. F	nom unitorm	loads				
	[1.2(23)+1-6	(40)]	· 2'bi	eam =	1.83 L	
Taxa	factored		0 70	214		
10TQ	Tactoreo	000 -	6.10	<u>1</u>		
	1 1 3	PIJ B.S				
1	111	11	11			
1			1	8 kip		
1 28	Kip		23	8 kip		
10 -	2.Fre bus	1 - 21	4000	(201)10	751)-	2964
NC	C 11 C 0 (1) C	<u>cv</u>	100	0	and the	6110
V	U = AVC	1				
	28 = 0.75	(29.6)	- 27	2.2,	need o	Himps
0	.5VC = 14.	x v				
		0 4		0	middle	portion
20 -	141.6 2		4'9" y		wou't n	end
	1	1 *	Y			for shear
	1	1				
~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~	4'9" *	1	Ja	84		
			10	0		
N	ued From	n supp		28- 2	.8 (9.75)	12) = 25.7
	Vs = Nu	- Ne =	25.7	_ 29.	6=4.	67 4
	4		0.75	5		
	15 = 8 J400	17421	975	114	4 > 5	1
		000	115)	- 110	3	
	Smax =	0/2	= 4.9	575 -	Use 4.5'	
	min	( 24"	1			
	Sev EA.	L		1601	9.75) =	12 01
	Sreg = Au	Fyr d	- 0.	4.67	(15) e	0.01
		VS		1.61		
	space at	4.6"				
			+++			
			TIT			

lech F	eport # 3	Alterno	ste #2	Samani	ha dellies	-
			belt - to			++
P	eam Torsion	Check				
5						
+++	Mu = 34	8 A.K	(use la	gest middle	e ship now	ent
+++			for a	noitenvixaga	- long span	)
	11 - 34	\$ /	5 1 70 3	H-k / 1' stri		++
				· · · · · · · · · · · · · · · · · · ·	pwidth	
	Tu = 1:	74 (2)	18) = 1	5.7 A.L		-
		- In	T			++-
	15.7					+++
	1			5		
				J15.7 H.K	torsion	
×	1	A Shid	JP A	1011 (0.12)	044.2	
12"	15	1	Acp =	12" (24") = 2(12) + 2(24	400 in	-
10						
1	10	E1	Threater	= 0.75 400	$\frac{1}{100}\left(\frac{288^2}{72}\right) = 5$	4.6
	×	V				101-1
	7 24"		Tu = 15.	7 . 12 = 188.	4 in·k	
			torelow	reinforceme	nt required	
					b	
	check sect	ion siz	e.:			
+++-	V = 74	- 2/15	)-0.315	201		
	N. = 12	- 2115	)-0.375 =	8.6		
	Ash = 7	0.6" (8	.6)" = 8.6) = 5	178 in2		
+++	$P_{n} = 7$	(20.6 -	8.6) = 5	8.4 in		++
	Nr = 2	Junon 12	4")(12-)	- 36.4 K		tt
		1000				
			0		1	
		00 10572	+ 188401	(58.4) €	0.75 / 36400	+ 8 40
	(24)	(9.75)	(1.7(	178)2	((24) (9.75)	1
		237	psi 5	496		
-	2					
				s, the bea	nd deflecti	en er
	due to ge			, locaron, a	ND ORTHERM	CING-
	3	1 1 1				

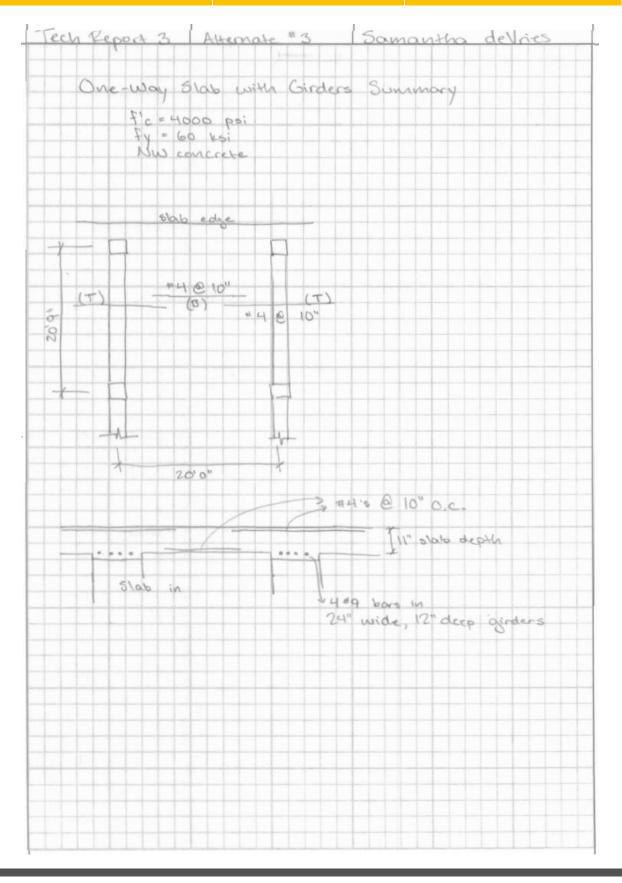
Diab with P Do psi xsi ncrete	Perimeter Beams Summary Columns: 24" x 24" Perimeter Beam: 24" wide 12" deep Stab: 7" deep Reinforcement: Bottom: #4@ 12" O.C. Each way Top: Cons Direction: @ column: #6@ 12" middle: #3@ 12 Short Direction: @ column: #6@ 12" middle : #3@ 12"
20 psi xsi ncrete	Columns: 24" x 24" Perimeter Beam: 24" wide 12" deep Slab: 7" deep Reinforcement: Bottom: #4@ 12" O.C. Each way Top: Column: #6@ 12" middle: #3@ 12 Short Direction:
xsi Accete	Perimeter Beam: 24" wide 12" deep Slab: 7" deep Reinforcement: Bottom: #4@ 12" O.C. Each way Top: Cong Direction: Middle: #3@ 12 Short Direction:
xsi Accete	Perimeter Beam: 24" wide 12" deep Slab: 7" deep Reinforcement: Bottom: #4@ 12" O.C. Each way Top: Cong Direction: Middle: #3@ 12 Short Direction:
Accese	Slob: 7" deep Slob: 7" deep Reinforcement: Bottom: #4@ 12" O.C. Each way Top: Long Direction: @ column: #6@12" middle: #3@12 Short Direction:
ab edge	Stab : 7" deep Reinforcement: Bottom: #4@ 12" O.C. Each way Top: Long Direction: @ column: #6@12" middle: #3@12 Short Direction:
	Bottom: #4@ 12" O.C. Each way Top: Long Direction: @ columni # 6@ 12* middle: # 3@ 12 Short Direction:
	Bottom: #4@ 12" O.C. Each way Top: Long Direction: @ columni # 6@ 12* middle: # 3@ 12 Short Direction:
	Each way Top: Long Direction: @ column: # 6@12* middle: # 3@12 Short Direction:
	Each way Top: Long Direction: @ column: # 6@12* middle: # 3@12 Short Direction:
	Top: Long Direction: @ columni # 6@12* middle: # 3@12 Short Direction:
0'0"	Cons Direction: @ columni # 6@ 12* middle: # 3@ 12 Short Direction:
0 ¹ 0 [*]	@ columni # 6@12* middle: # 3@12 Short Direction:
0'0"	Short Direction.
0'0"	Short Direction.
0'0"	
0'0"	@ column: "6 @ 12" middle : "3@ 12"
0'0"	middle: "3@ n"
0.0	maore 30 M

	A D	Alternas	e - 3	Jav	nanth	a del	053
Fromin	3 Altern	ate #3 -	One	Way	Slab	with G	irders
	61010	edge					
8		9	1				
				+++			
			2010				
		H	+				
11							1111
1	20	'o" }					
Slab	Design		+++	+++			
F	man AC	402 males	- shall	1. L/	20 10	in the second	
1110	slab this	402 notes kiness: 2	20' - 2' =	ln -	(18'212	1/20 =	10.8"
	1	N-1-1-1-1-1-1-1-1-1-1-1-1-1-1-1-1-1-1-1			+-+		se ll"
	Live 1	load = 40 p	10				
	Dead	Load = (M	(51.) = 23	est			
	Dead	Load = (M	1(sc.) = 23	1			
	Dead 11" S' Total	Load = (M 106 = 138 p Dead Loo	use.) = 23 sl xd = 161	feq	~1		
	Dead 11" S' Total	Load = (M	use.) = 23 sl xd = 161	feq	40)]-1	· = 2*	119 S. 2. FI
	Dead 11" 5' Total W. (Tor	Load = (M lob = 138 = Dead Loo a 1° strip)	use.) = 23 sl xd = 161	psf 1)+1.6(	-	1 = 2= 8 H-x	119 S. R.
	Dead 11" s' Total W. (Bc Mu	Load = (M lob = 138 p Dead Loo a l' strip) = wuln =	$u(s_{L}) = 23$ 31 ad = 161 = [1.2(46) 0.2852( 16	p=f 1)+1.6( 18) ² =	+5.7	8 ft-k	57.2 plf
	Dead 11" 5' Total W. (Tor	Load = (M lob = 138 = Dead Loo a 1° strip) = wuln = 16 = yull =	$u(s_{L}) = 23$ rad = 161 = [1.2(46) 0.2852( 16 0.2852(	p=f 1)+1.6( 18) ² =	-	8 ft-k	57.2 p/f
	Dead 11" s' Total W. (Bc Mu	Load = (M lob = 138 p Dead Loo a l' strip) = wuln =	$u(s_{L}) = 73$ $rac{1}{2}$ $rac{1}{2} = 161$ $ ac{1}{2} = [1.2(16)$ $ ac{0.2852(}{16}$ $ ac{16}{2}$ $ ac{16}{2}$ $ ac{1}{2}$ $ ac{1}{2}$	p=f )+1.6( 18) ² =	+5.7	8 H-K	
	Dead 11" s' Total W. (Bc Mu	Load = (M lab = 138 p Dead Loo a l' strip) = Wuln = 16 Utel = 11 H	$u(s_{L}) = 73$ $rac{1}{2}$ $rac{1}{2} = 161$ $ ac{1}{2} = [1.2(16)$ $ ac{0.2852(}{16}$ $ ac{16}{2}$ $ ac{16}{2}$ $ ac{1}{2}$ $ ac{1}{2}$	p = 1 $(18)^2 = (1)^2$	+5.7	8 ft-k	
	Dead II" S' Total W. (Soc Mu +	Load = (M lab = 138 p Dead Loo a l' strip) = wuhr = 16 = 4 b 16 = 16 = 14 b 16 = 14 b 16 = 14 b 16 = 14 b 16 = 14 b 16 = 16 = 16 = 16 b 16 b 16 b 16 b 16 b 16 b 16 b 16 b	$u_{5(2)} = 73$ 31 ad = 161 = [1.2(46) 0.2852( 16 0.2852( 11 11	p = 1 $(18)^2 = 11$ $(18)^2 = 11$ $(18)^2 = 11$ = 0	+5.7 - 8.4 - 1.5 -	8 H-K H-K 2(0.379	
	Dead 11" 5' Total W. (Joc Mu + Mu = As reg =	Load = (M lob = 138 = Dead Loo a 1 strip) = wuln = Hb= Hb= H Hu Alu Alu Alyd = 8.4 H	$u(s_{L}) = 23$ rad = 161 = [1.2(46) 0.2852( 16 0.2852( 16 0.2852( 11 rad = 161 rad = 161 ad = 161	p = 1 $(18)^2 = 11$ $(18)^2 = 11$ $(18)^2 = 11$ = 0	+5.7	8 H-K H-K 2(0.379	
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	Dead 11" 5' Total W. (For Mu + Mu = As reg =	Load = (M lob = 138 = Dead Loo a 1 strip) = wuln = Hb= Hb= H Hu Alu Alu Alyd = 8.4 H	$\frac{(6L)}{2} = \frac{23}{21}$ $\frac{161}{2} = \frac{1.2(16)}{1.2(16)}$ $\frac{0.2852(16)}{16}$ $\frac{0.2852(16)}{11}$ $\frac{11}{2}$ $\frac{11}{2}$ $\frac{11}{2}$ $\frac{11}{2}$	f = 1 f	+5.7 - 8.4 - 1.5 - .3"	8 H-K H-K 2(0.379	5)

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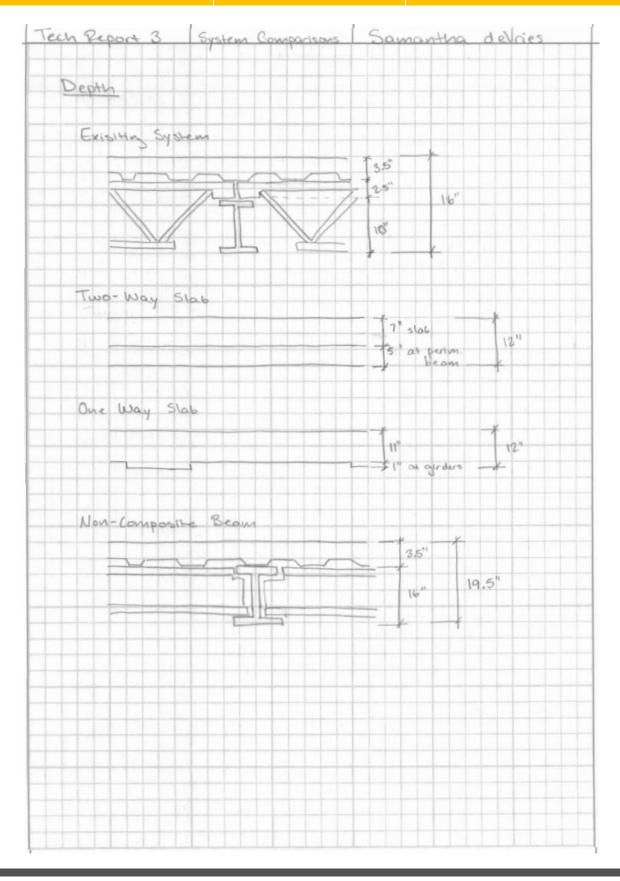
ch Report #3	Alternate # 3	Samantha	dellies
Girder Desig	n	Stra Provent	- e 16
Initial Size	for Deflection		
C. (note: C	24" columns - 24	" wide girders	to start)
	d = 40 psf ad = 161 pof		
Use Tabl. size for	e 9.5 (a) to choose deflections	approximate	
Bean	/ 18.2 = (20.75.	12)/18.2 = 13.7	r bay) 7 - use 14"
Beam of	elf weight = 150	PCF (24")(14")	= 350 pif
Total Fac	tored load = 350 = 5.5	p1F + [1.2 (161) + 1 K1F	.6 (40)]. 20'
Strength Che	ex.		
ACI MON	ment coefficients		
lorgest	$= \frac{\omega_{\rm o} l_{\rm o}^2}{10} = 5.5$ (1)	8.75)2 = 193.4	I Ft·K
Approxim	are d -> 14-1.5 445	- 0.5 - 2 (1.0) =	11.5"
Asry	· Mu · 193	5.4 · 12 =	3.94 in ²
USE	4 #915 - Hais i	6 reasonable.	
Shear Cheel	£		
Factored	Line Load = 5.5	KIF,	
E F I	TTTTT	12	
51.62	Ln=18.75'	1 51-6 4	
51.6	~~~~~		
		N dias	

Tech Repo	A 3 Alte	mate #3	Samant	na dellines
Vc = 2	Jf'c bud =	254000 (	24)(9.3") =	28.2 4
	N= + Vc	-> 51.1	= = 0.75 (28.	2) X - need shear reint.
5		6.8'	no stimups	for shee- end in action of beam
	Vo@d from	support +	51.6 - 5.5 (9.3/0	
	No= No	Ne = 47.4	1 _ 28.2 = 3	54
	and the second se	0 (24)(9.3)	= 1134 > 35*	/
	Smax = [	d/z = 4.65 24"	- upe 4.5"	
	S ref = A	No Vo	(0.20) (60) (9.3) 35	3.19 in - Use #4@ 3"
so tor of Uni compo torsion	sion would when loading and to the	only occur , which Dead Loc der in H	Nerior beam, in the event will be small ad. Therefore, is system will	
Base Will	d on initia work for	l colculation strength, s	ons, the slak hear, and de	and girders. Hections.



# System Comparisons

Considerations		<b>Existing Steel</b>	Two-Way Slab	One-Way Slab	Non-Composite Steel
Architect	ural Consideratio	ns			
Total System Depth		16"	12"	12"	19.5"
Fire Ratin	g	2 hr	3 hr +	3 hr +	2 hr
2 hr Fire F	Rating?	yes	yes	yes	yes
System St	tatistics	_			_
Durability		acceptable	high durability	high durability	acceptable
Weight		40.7 psf	87.5 psf	138.8 psf	41.8 psf
Cost per s	quare foot	\$15.80	\$13.61	\$18.90	\$21.90
Future De	esign Consideratio	ns			
	Concrete Shear Walls	No	No	No	Yes
Lateral	Concrete Moment Frame	Yes	Yes	Yes	No
System Options	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
Disadvant		-Not a typical system for new construction	-None	-Heaviest System -Relatively Expensive	-Most expensive system -Largest total system depth
Future Us	e?	N/A	Yes	No	Yes



Uleight Existing System Dece: 37 psF Joron: 8 psF / 20 spacing = ZpsF Conders: 35pt / 20 spacing = 1.65 psF Total Weight = 40.7 psF Two-Way Stab Stab: 150 pcF x (1"/12) = 87.5 psF No Drop Pomers for now Total Weight = 87.5 psF One-Way Stab Stab: 150 pcf × (1"/12) = 137.5 psF Girders: 150 pcf × (1"/12) = 137.5 psF Total Weight = 138.8 psF Non-Composite Street Stab: 37 psF Beam: 14 ptF /41 sp. = 3.5 psF Girder : 26 ptF /20 sp. = 1.3 psF Total Weight = 41.8 psF		Report 3 System Compassion Samantha dellies
Deck: 37 psf Joist: $7 pif/4' + crocing = Zpsf Coinders: 33pif/20' + 20' + crocing = 1.65 pefTotar Weight = 40.7 psfTwo-Way SlabSlab: 150 pct × (7"/12) = 87.5 psfNo Drop Panels For nowTotar Weight = 87.5 psfOne-Way SlabSlab: 150 pcf × (1"/12) = 137.5 psfCoinders: 150 pcf × (1"/12) = 137.5 psfTotar Weight = 138.8 psfTotar Weight = 138.8 psfNor-Composite SteelSlab: 37 psfBeam: 14 pif/4' sp. = 3.5 psfGirder 26 pif/20' sp. = 1.3 psf$	Nei	ght_
Joint: $7 \text{ pit}/41$ is gracing = 2 pet Girders: $33\text{ pit}/20' \text{ spacing} = 1.65 \text{ pet}$ Total Weight = 40.7 pet Two-Way Slab Slab: 150 pet × (7"/12) = 87.5 pet No Drop Poincis for now Total Weight = 87.5 pet One-Way Slab Slab: 150 pet × (1"/12) = 137.5 pet Girders: 150 pet × (1"/12) = 137.5 pet Total Weight = 138.8 pet Total Weight = 138.8 pet Nor-Composite Steel Slab: 37 pet Beam: 14 pit/41 sp. = 3.5 pet Girders: 26 pit/20' sp. = 1.3 pet	4	Existing System
Two-Way Slab Slab: 150 pct × $(7^{*}/12) = 87.5$ pst No Drop Panels for now Total Weight = 87.5 pst One-Way Slab Slab: 150 pct × $(7^{*}/12) = 137.5$ pst Girders: 150 pct × $(7^{*}/12) = 137.5$ pst Total Weight = 138.8 pst Non-Composite Steel Slab: 37 pst Beam: 14 plf /4' sp. = 3.5 pst Girder: 26 ptt / 20' sp. = 1.3 pst		Deck: 37 pof Joist: 8 pif /4' # opacing = 2 pof Girders: 33pit / 20' spacing = 1.65 pof
Stab: 150 pct × $(7"/12) = 87.5$ pst No Drop Painets for now Total Weight = $87.5$ pst One-Way Stab Stab: 150 pct × $(""/12) = 137.5$ pst Girders: 150 pct × $(1"/12) \times 2'/20" = 1.25$ pst Total Weight = $138.8$ pst Nar-Composite Steel Stab: 87 pst Beam: 14 ptf /4" sp. = $3.5$ pst Girder: 26 ptf /20" sp. = $1.3$ pst		Total Weight = 40.7 p=f
No Drop Poincis for now Total Weight = 87.5 pst One-Way Slab Slab: 150 pct × (""/12) = 137.5 pst Girders: 150 pct × (1*/12) × 21/20' = 1.25 pst Total Weight = 138.8 pst Non-Composite Steel Slab: 87 pst Beam: 14 pit /4' sp. = 3.5 pst Girder: 26 pit / 20' sp. = 1.3 pst	-	Two-Way Slab
<u>One-Way Slab</u> Slab: 150 pct × (""/12) = 137.5 pst Girders: 150 pct × (1"/12) × 2'/20" = 1.25 pst Total Weight = 138.8 pst <u>Non-Composite Steel</u> Slab: 87 pst Beam: 14 pit /4' sp. = 3.5 pst Girder: 26 ptt / 20' sp. = 1.3 pst		Slab: 150 pct × (7"/12) = 87.5 pot No Drop Panels for now
Slab: 150 pct × $(1^{"}/12) = 137.5 \text{ pst}$ Girders: 150 pct × $(1^{"}/12) \times 2^{'}/20^{"} = 1.25 \text{ pst}$ Total Weight = <u>138.8 pst</u> <u>Non-Composite Steel</u> Slab: 37 pst Beam: 14 plf/4' sp. = <u>3.5 pst</u> Girder: 26 plf/20' sp. = <u>1.3 pst</u>		Total Weight = 87.5 pst
Cirders: 150 pcf x (1°/12) x 21/20° = 1.25 pst Total Weight = 1 <u>38.8 pst</u> Non-Composite Steel Stab: 87 pst Beam: 14 pit /4' sp. = 3.5 pst Girder: 26 ptt / 20' sp. = 1.3 pst	(	Dine-Way Slab
Non-Composite Steel Slab: 87 psf Beam: 14 plF/4' sp. = 3.5 psf Girder: 26 plf/20' sp. = 1.3 psf		Slab: 150 pct × (""/12) = 137.5 pot Girders: 150 pct × (1"/12) × 21/20" = 1.25 pot
Stab: 37 psf Beam: 14 pif /4' sp. = 3.5 pst Girder: 26 ptf / 20' sp. = 1.3 pst		Total Weight = 138.8 pst
Beam: 14 pif /4' sp. = 3.5 pst Girder: 26 pit / 20' sp. = 1.3 pst		Van-Composite Steel
Total Weight = 41.8 pst		Beam: 14 pif /4' sp. = 3.5 psl
		Tatal Weight = 41.8 pst
	$\square$	
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can re	port 3	System	Comparison	s Samantha	devices
Cost	(Using	25 Meo	ins Squar	Foox Assembly	(05 2014)
Exi	sting Syst	em			
	9				
+++	Total Lo	aal = 115	came, 2 0 Epst, úse	lab on Column 20×20 bay w/	ind bet
	Cost:	\$ 15.8	0/ 5g- Ft		
Twi	s-Way SI	ab			
11	Cast in	F assig	stale tal	20×20 bay	
	7" 5126	thicknes	s	cay	33.0
	Cost :	\$ 13.6	or / 5g. tt.		
Ov	e-way 5	106			
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No	n - Compos	ite Bee	a.m.		
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Appendix

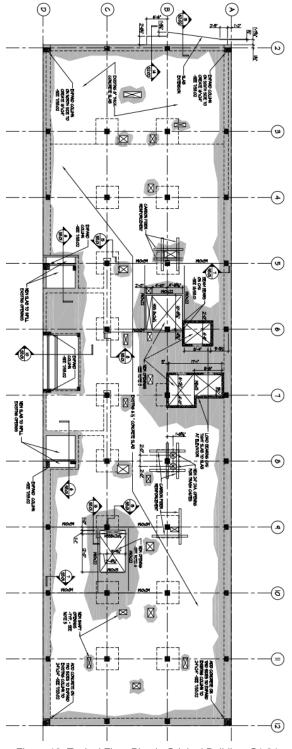


Figure 10: Typical Floor Plan in Original Building, S1.04

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.