Presentation Outline

- Welcome
- Introduction
- Architecture
- Existing Structure
- Problem Statement
- Arch. Breadth
- Structural Depth
- Conclusion
- Questions

Global Village
Rochester Institute of Technology
Rochester, New York

Christopher VandeLogt
B.A.E., Structural Option

Senior Thesis Final Project 2012
Advisor: Dr. Linda Hanagan

global village introduction

- $57.5 million project
- 122,000 square feet
- residential / commercial
- 4 stories + mech. penthouse
- 62.5 feet tall
- march 2009 – sept. 2009
- LEED Gold certified

project team

- **owner |** rochester institute of technology
- **architect |** arc
- **cm at risk |** the pike company
- **civil |** erdman anthony
- **structural |** lemessurier consultants
- **mechanical |** ibc engineering
- **lighting / electrical |** lam partners

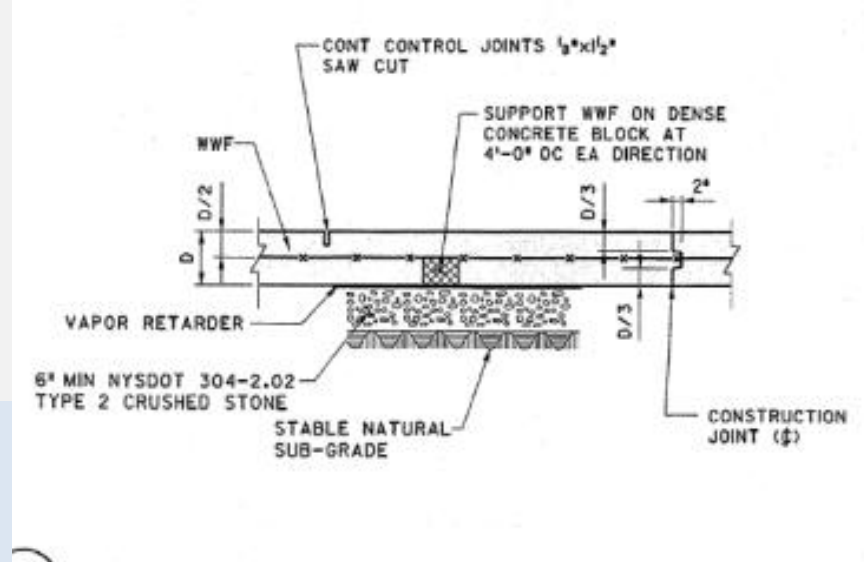
Architecture

- european-inspired complex
- incorporates different themes and materials to represent different regions from around the world
- unique shape creates an outdoor heated courtyard
- façade made up of cement fiber board, brick masonry veneer, and aluminum clad windows

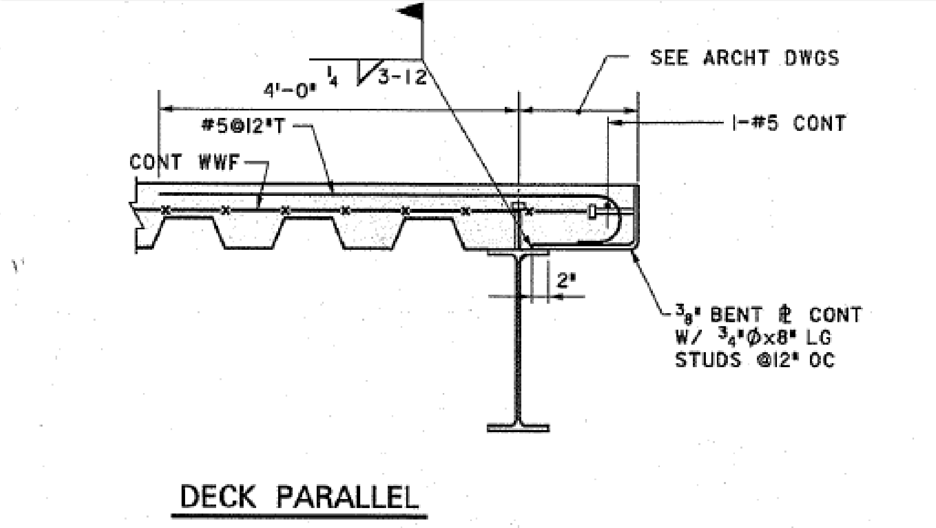
Existing Structure – Gravity

ground – 3rd floor

- 3¼" lightweight concrete on 3" composite metal decking
- typ. beams: W16x31
- typ. girders: W21x44
- typ. bay size: 29' x 32'
- 6" concrete on-grade slab
- isolated spread and continuous strip footings



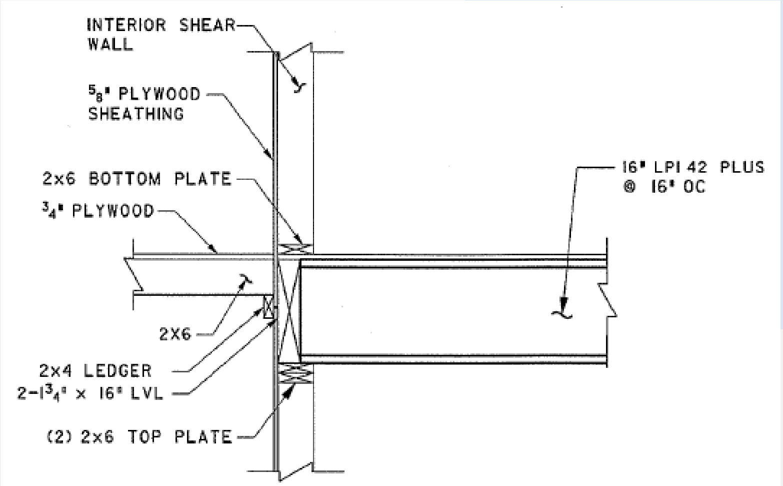
6 SLAB ON GRADE DETAIL



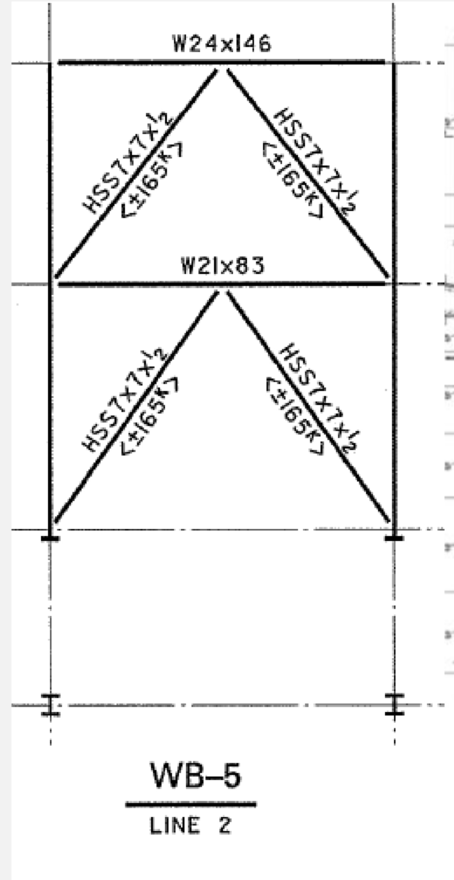
9 TYPICAL EXTERIOR COMPOSITE SLAB EDGES

4th floor - roof

- wood framing: 2-2x6 @ 16" wood studs typ.
- wood floor construction: 16" LPI Plus 42 @16" typ.



Existing Structure – Lateral

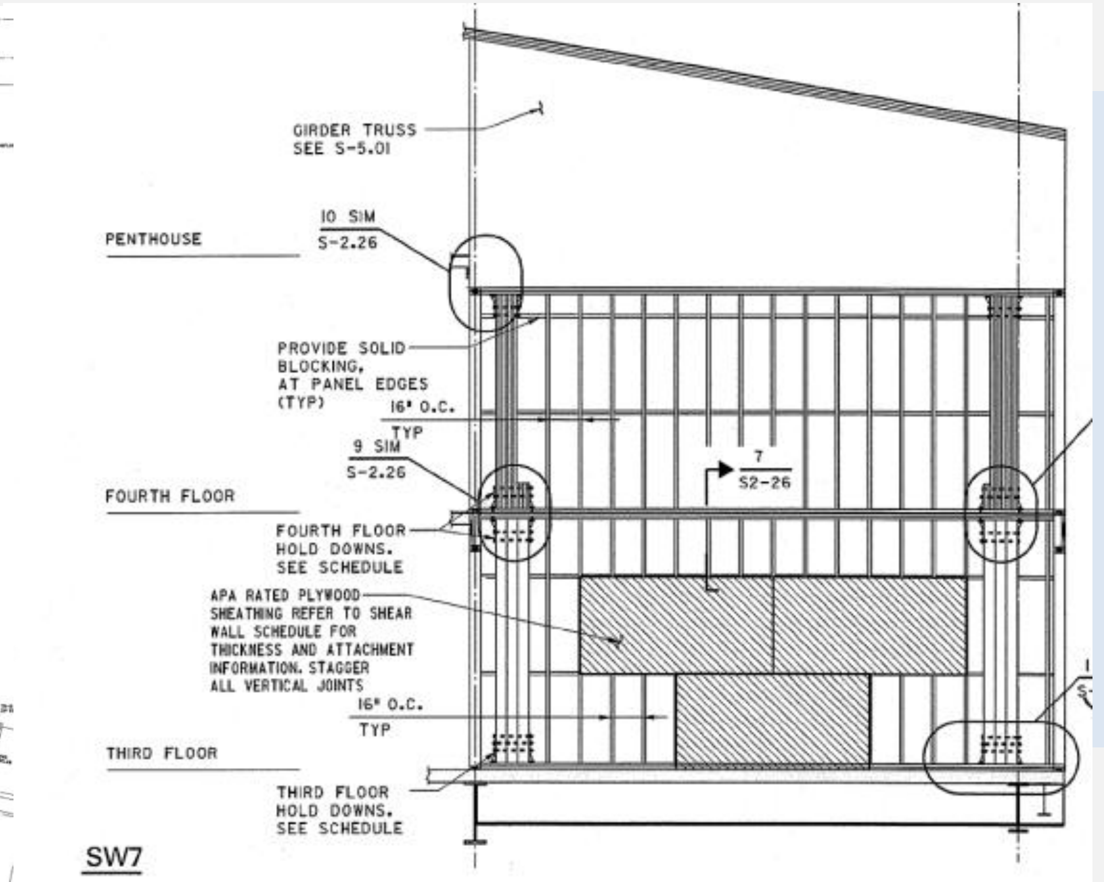


ground – 3rd floor

- concentrically braced frames in both directions
- HSS7x7x½ typ.

4th floor - roof

- wood stud shear walls
- 2x4 @ 16” wood studs and sheathing typ.



problem statement

- two different structural systems produces a complicated design and more firms need to be involved
- two different lateral systems used

proposed solution

- structure building entirely out of reinforced concrete through the use of a flat plate system
- use moment connections to resist lateral loads
- configure structural system as best as possible not to alter existing architecture or floor plans

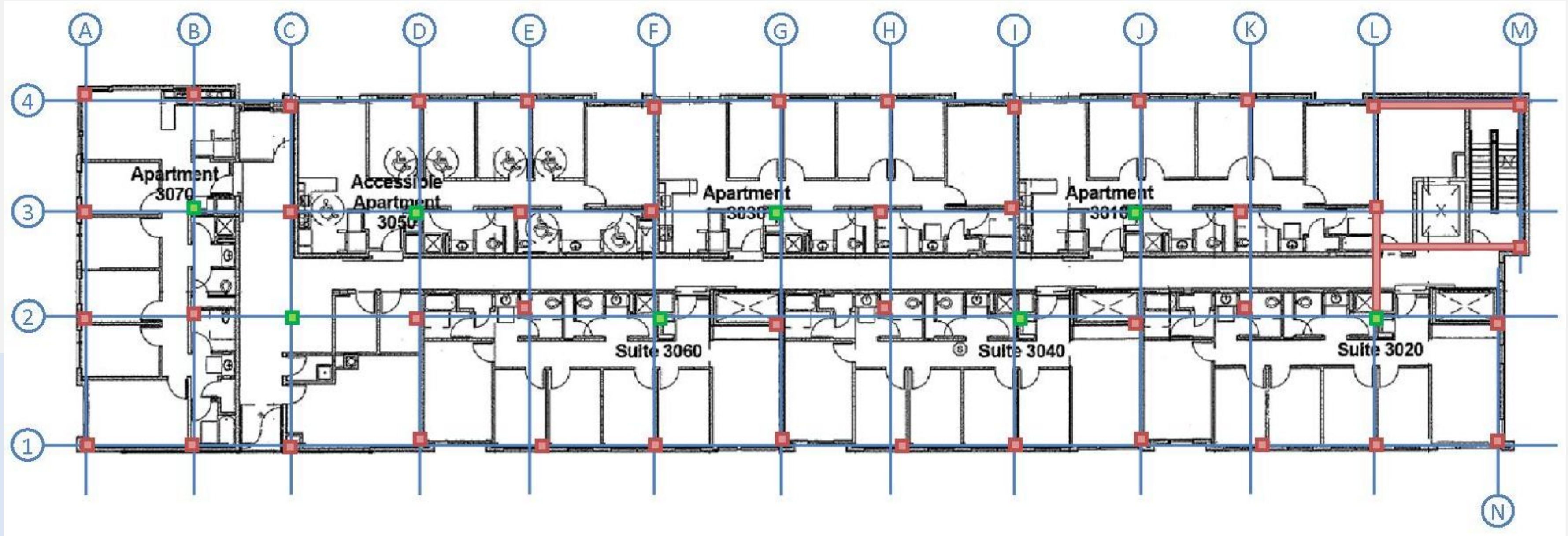
breadth topics

- architecture – analyze changes due to new structural system and create new plans if necessary
- construction management – study of on-campus residential buildings and cost analysis

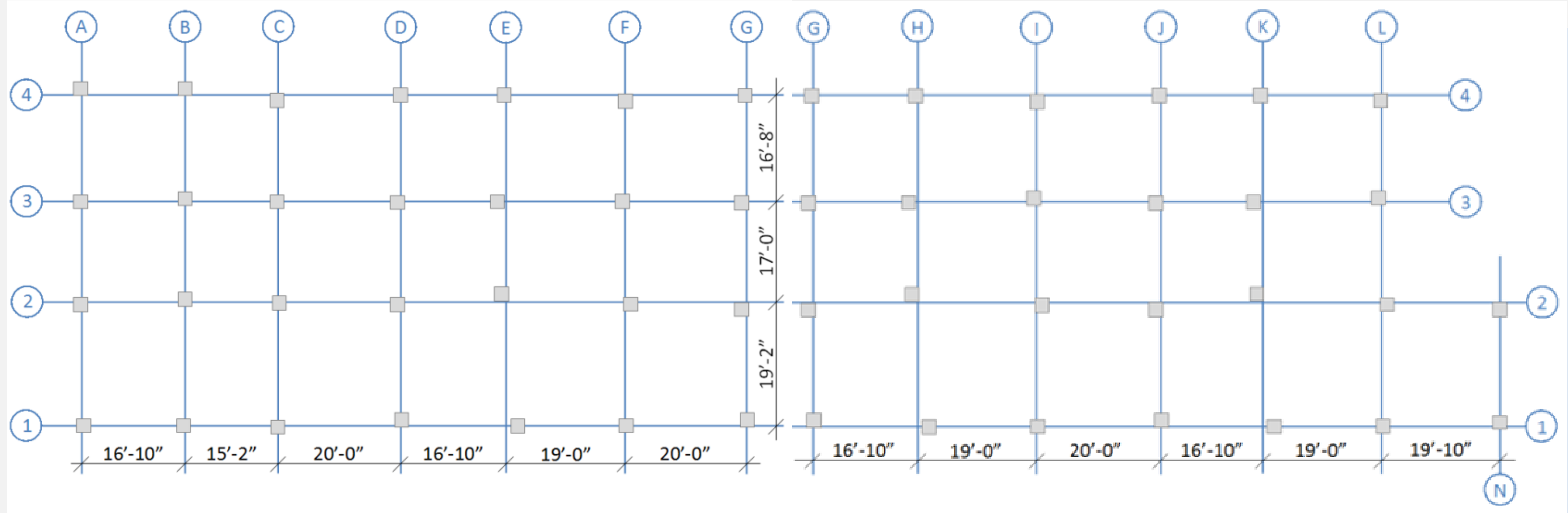
Architecture Breadth

column layout

- column placements chosen based on 3rd floor as to not affect residential floor plan best as possible
- bay widths in between 15' and 25' to be efficient for a flat plate system
- column centers vary a distance no more than 10% of span from column line



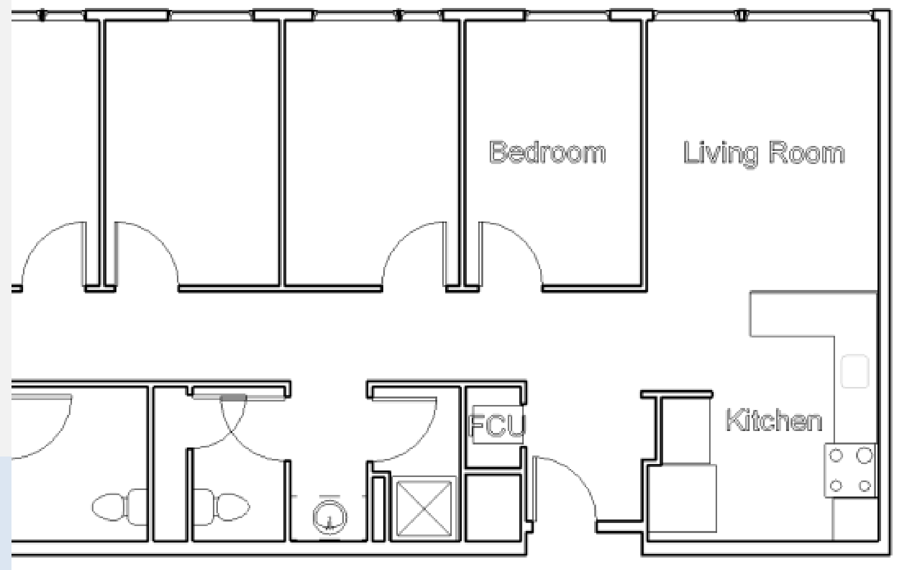
*columns highlighted in green denote a required change in floor plan or architecture



Architecture Breadth

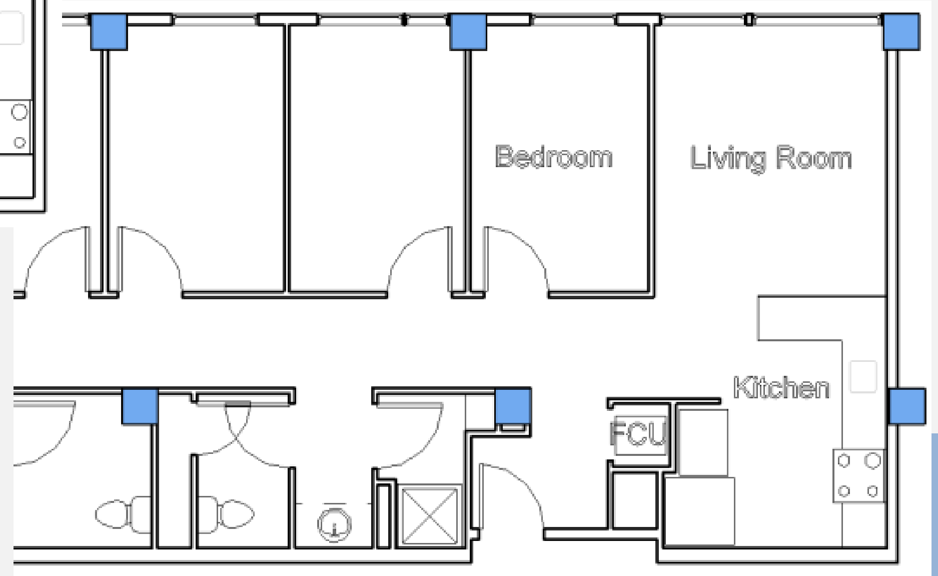
affected spaces

- 15 areas in total located on 3rd floor, 8 columns affect fan coil unit areas, 6 columns affect bathroom spaces, and 1 column affects corridor space
- entire second floor altered

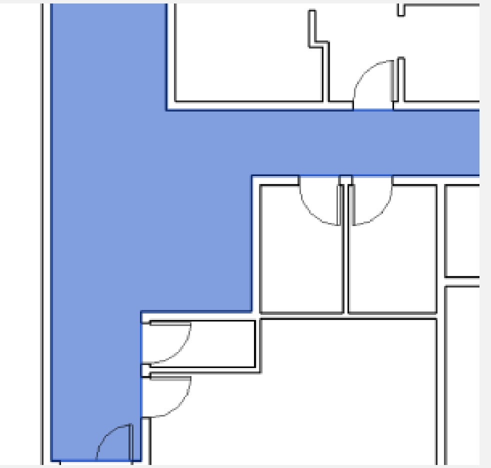


before

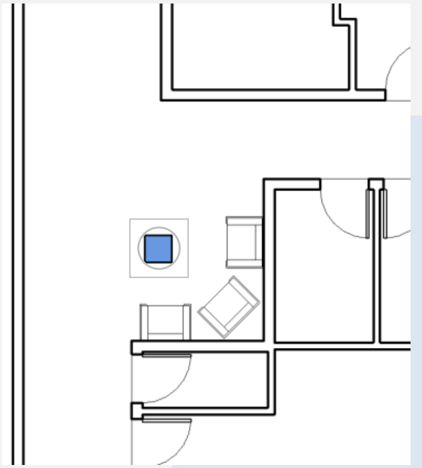
after



before



after



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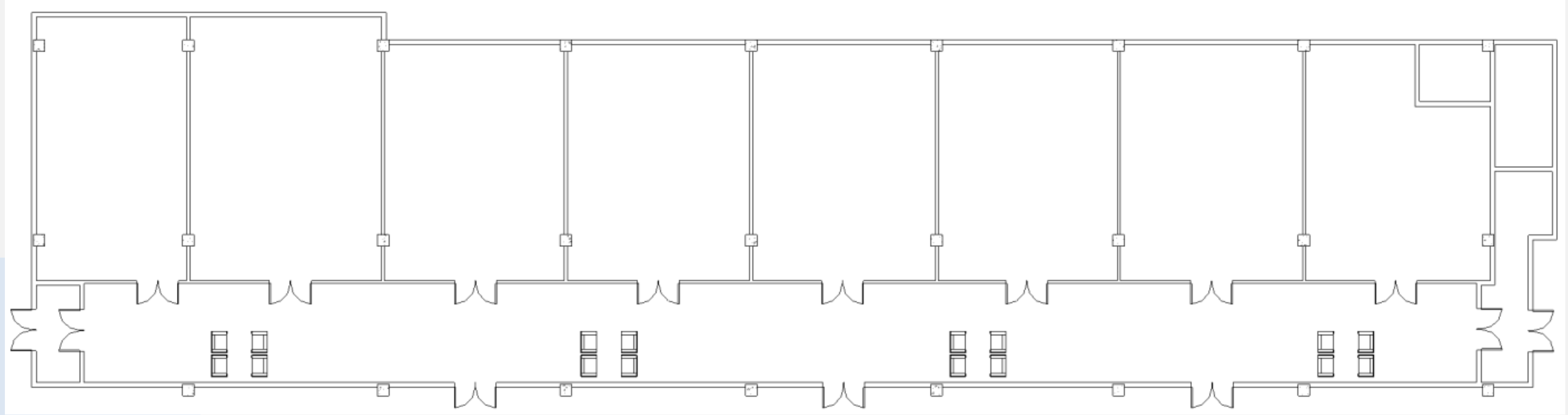
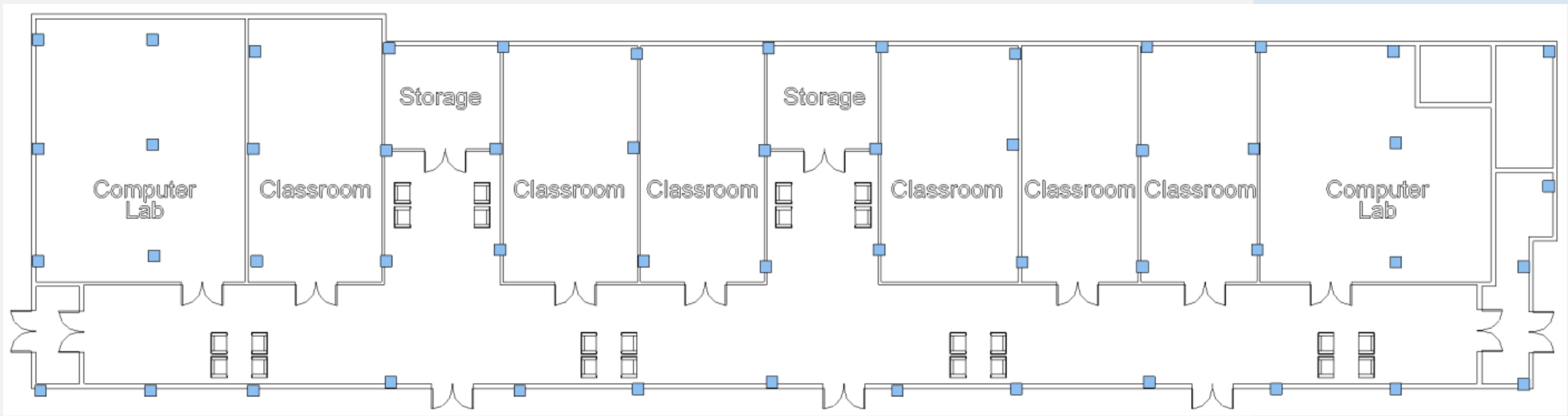


after



Depth – Gravity Redesign

super-imposed dead loads

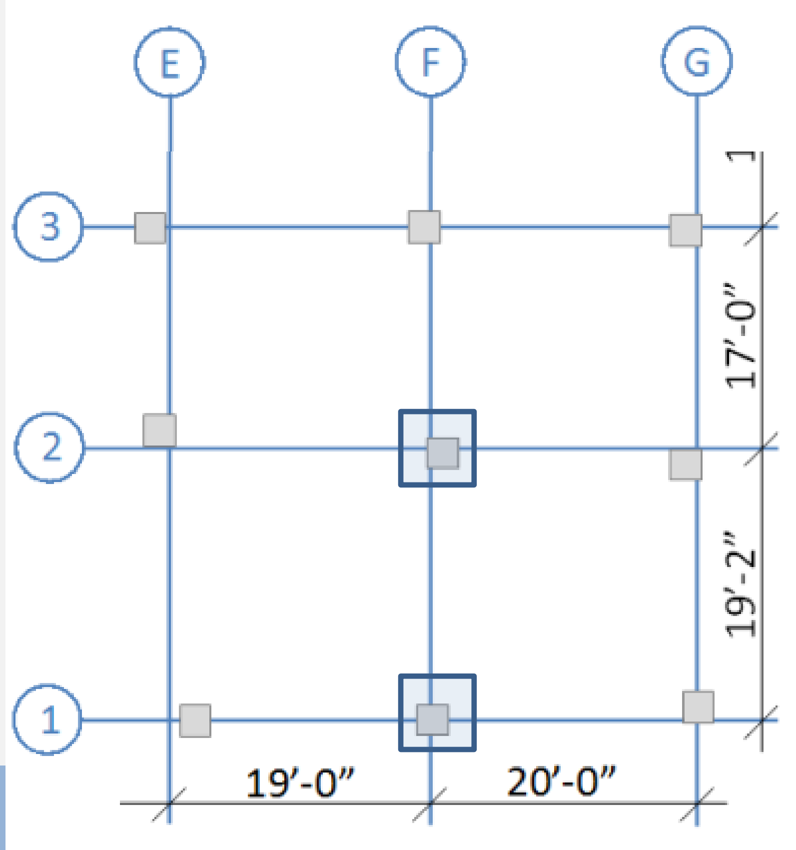
- 35 psf: SDL, MEP, partitions, ceiling
- 106 psf: 8½"slab self weight
- 18 psf: roofing

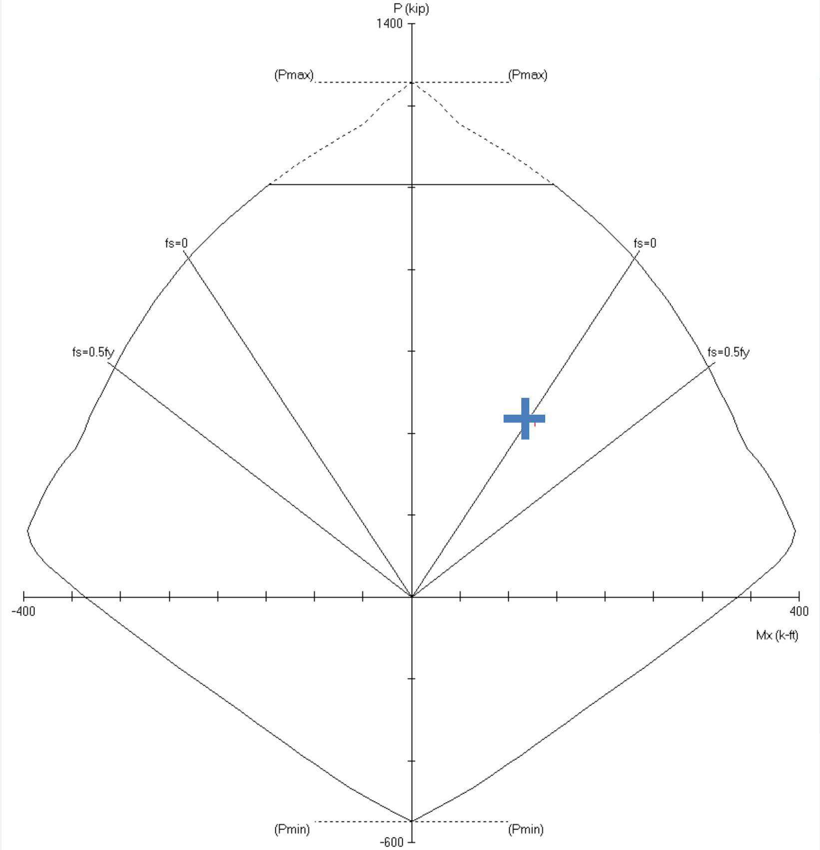
live loads

- 100 psf: 2nd floor
- 40 psf: 3rd and 4th floors
- 150 psf / 30 psf: mech. penthouse
- 20 psf: roof

column size

- 20" x 20" with (8) #10 bars spaced equally
- ground columns on grid lines F-2 and F-1 analyzed
- panel size: 19.5' x 18' (maximum)
- size controlled due to unbalanced moments by eccentricity of shear
- total compressive force applied: 437 kips
- maximum bending moment: 121 ft-kips





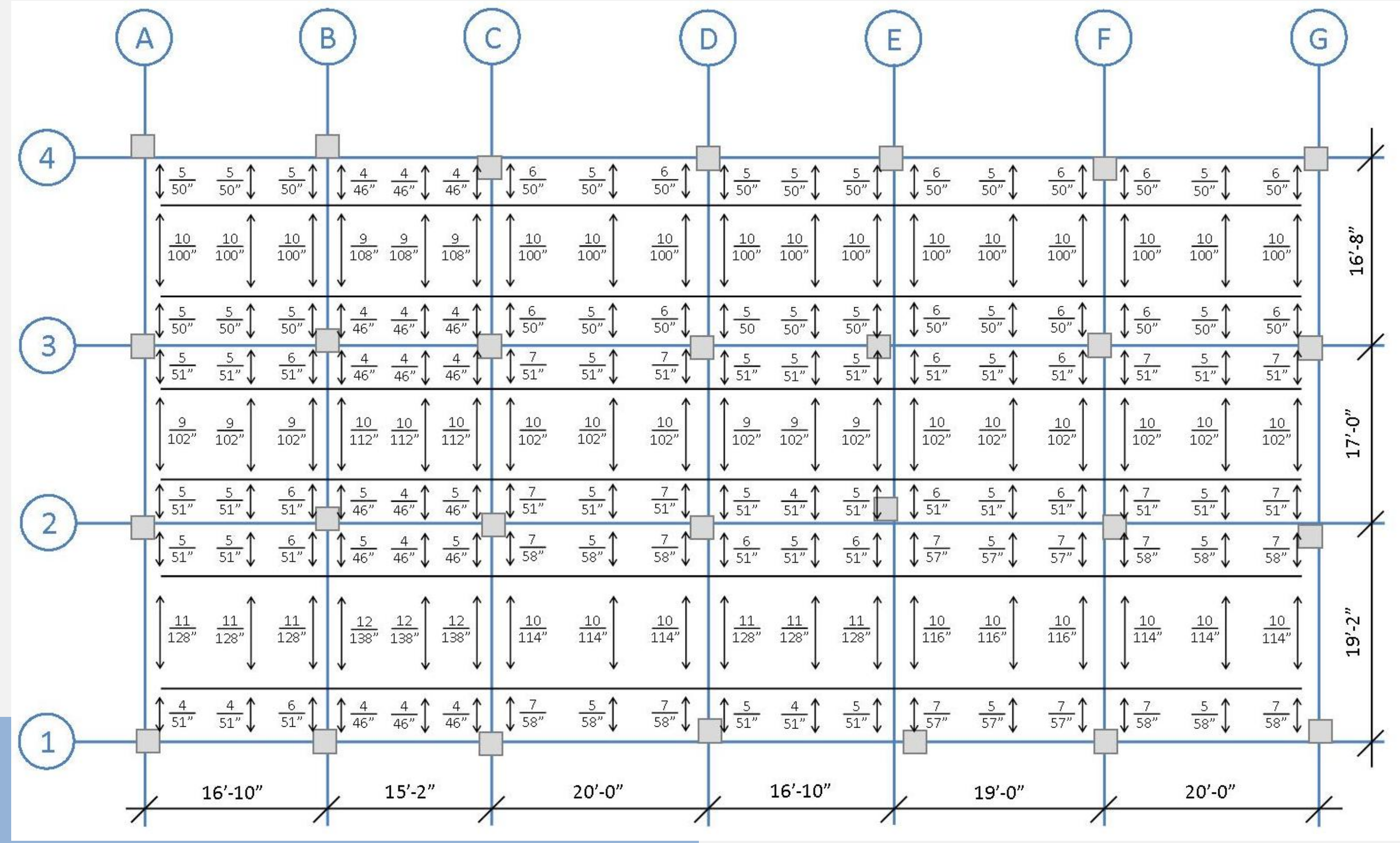
Depth – Gravity Redesign

slab thickness

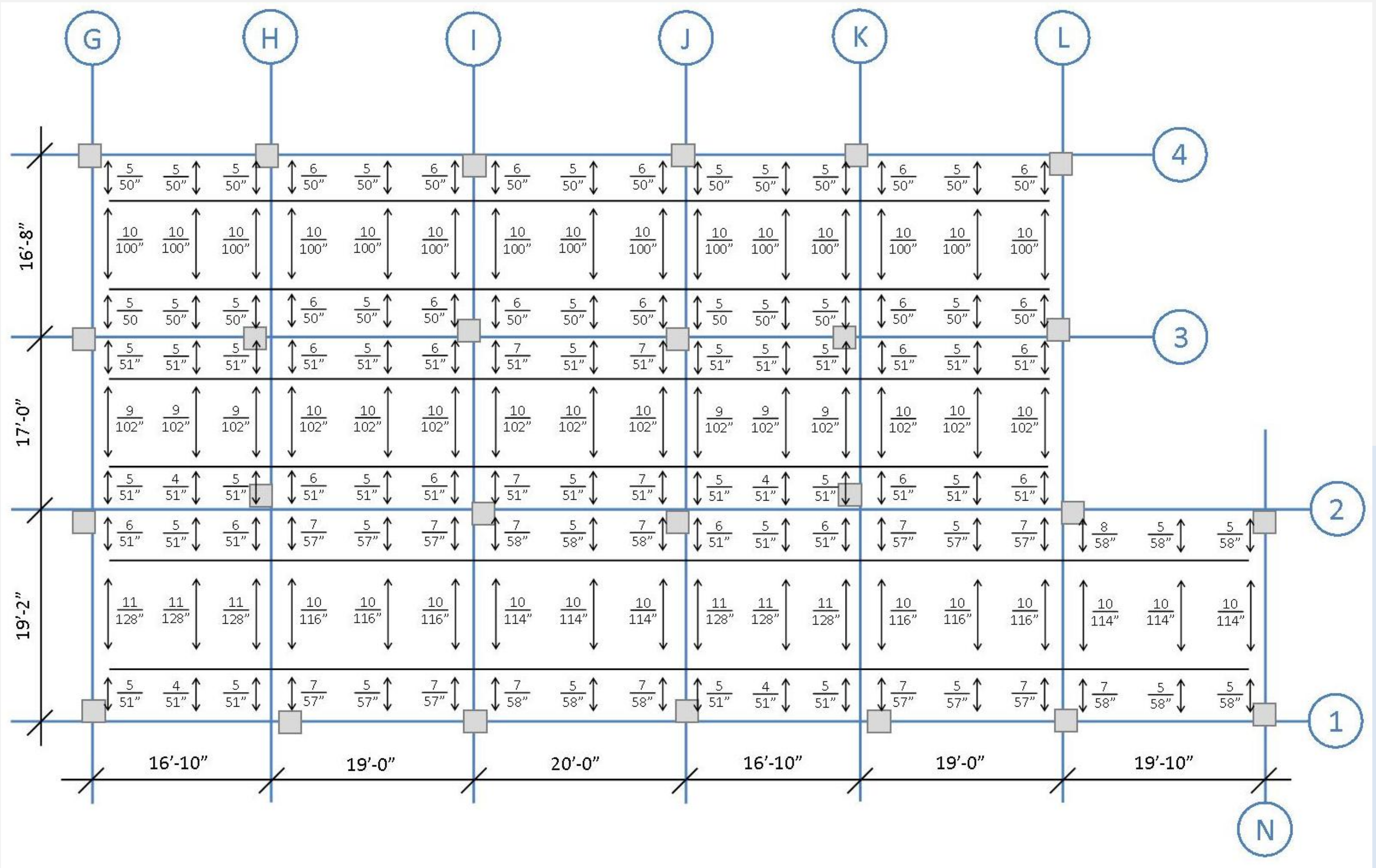
- 8½“ thick reinforced slab
- maximum clear span: 18’-4”
- 8¼” calculated using equations from Section 9.5.3 of ACI 318-08 but due to inadequate deflection checks, thickness raised and adequacy verified
- max deflection: .448”
- ACI Table 9.5b limit: .500”

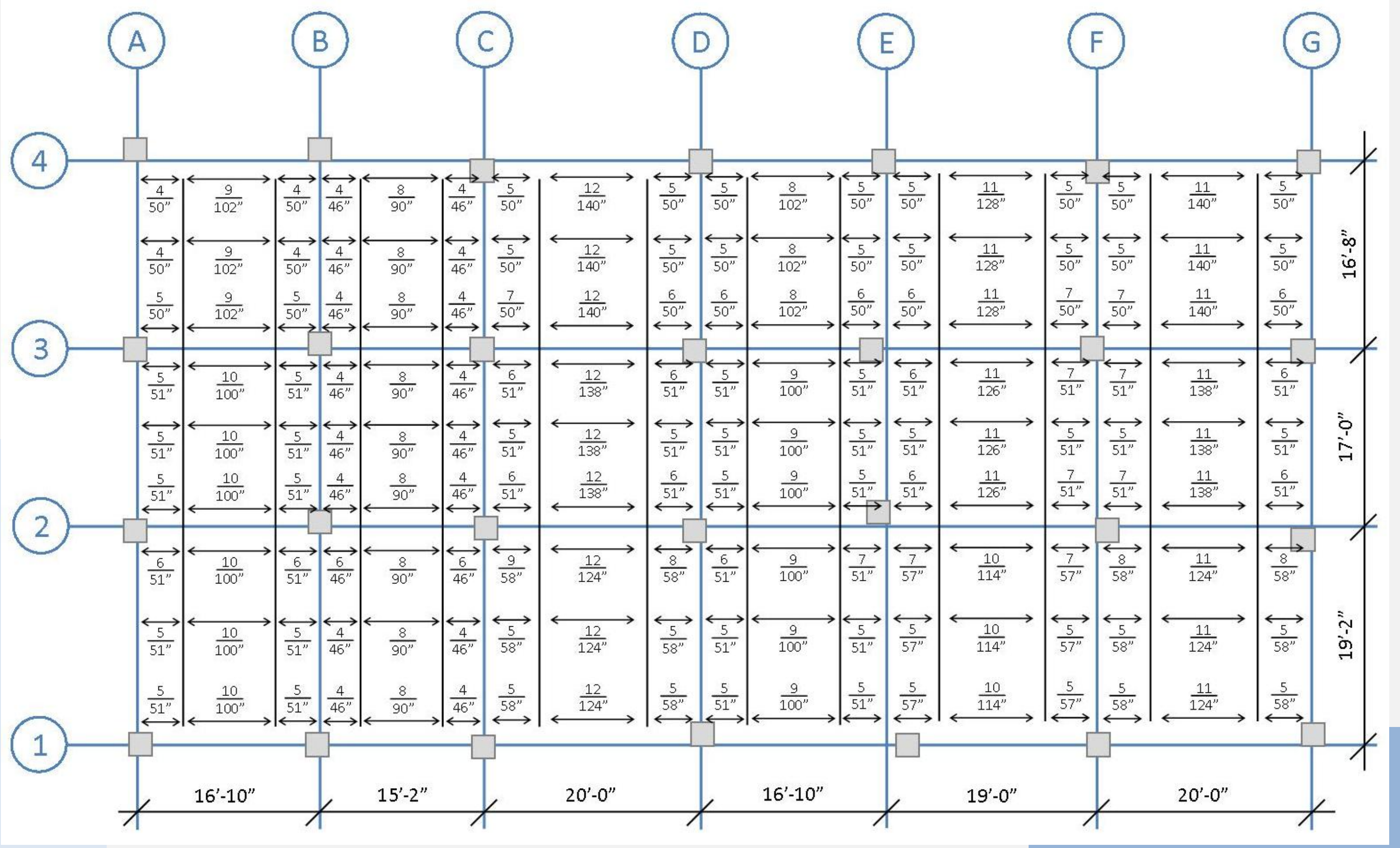
slab reinforcement design

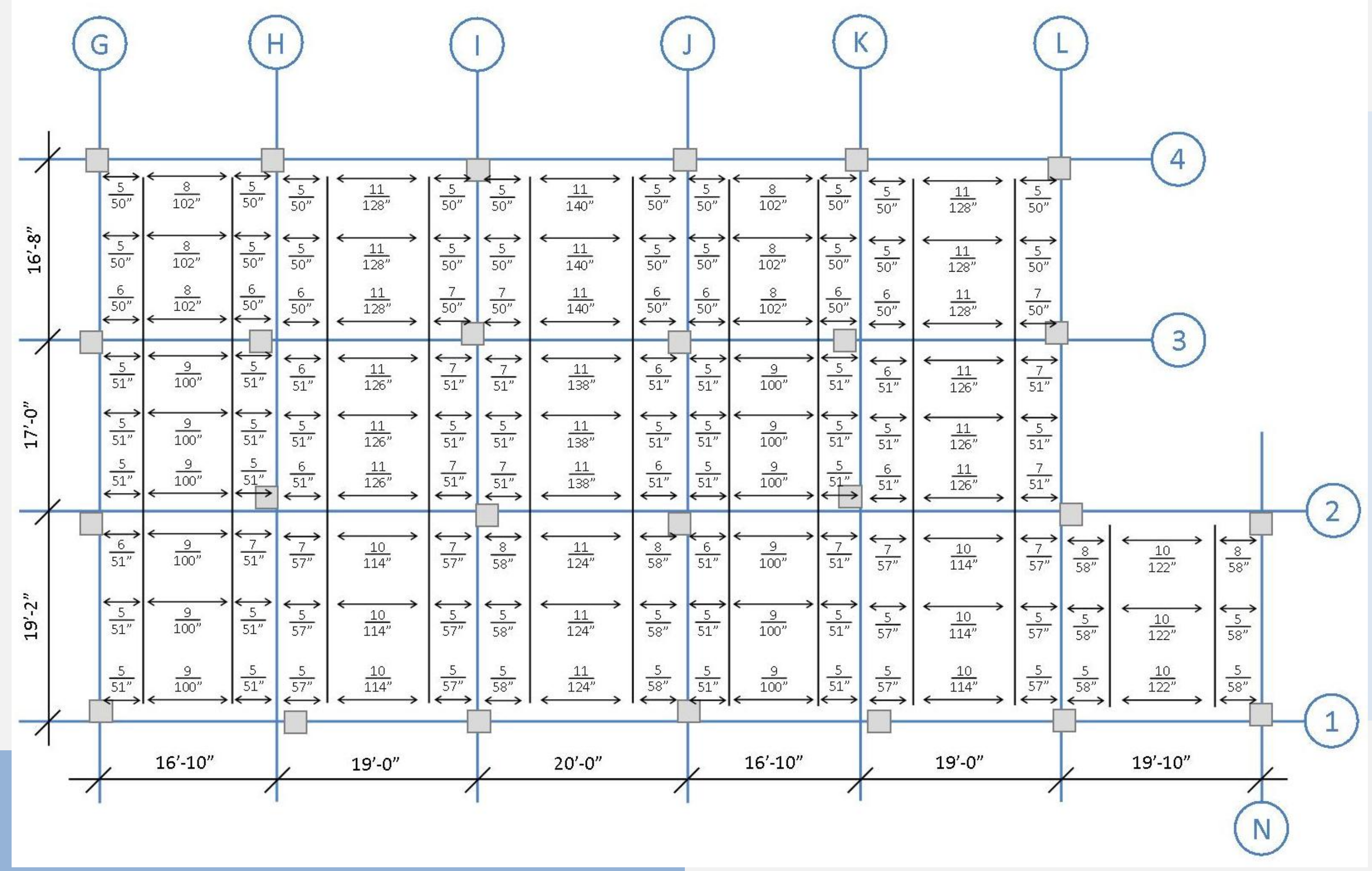
- direct design method used
- numbers refer to the amount of #5 bars that are equally spaced over the distance given
- note: bars spanning in the long direction are placed lower in slab than the bars in the short direction



Depth – Gravity Redesign





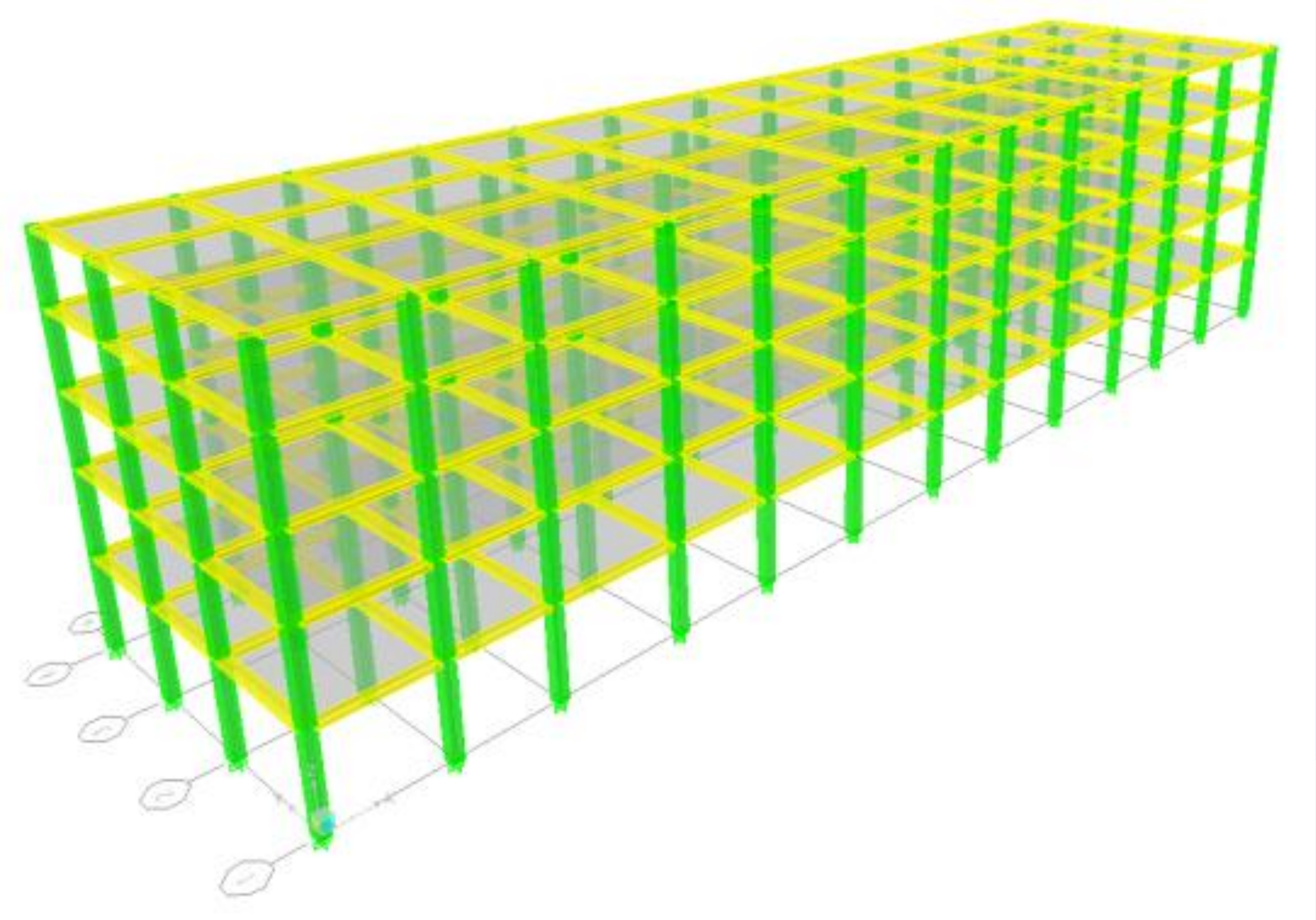


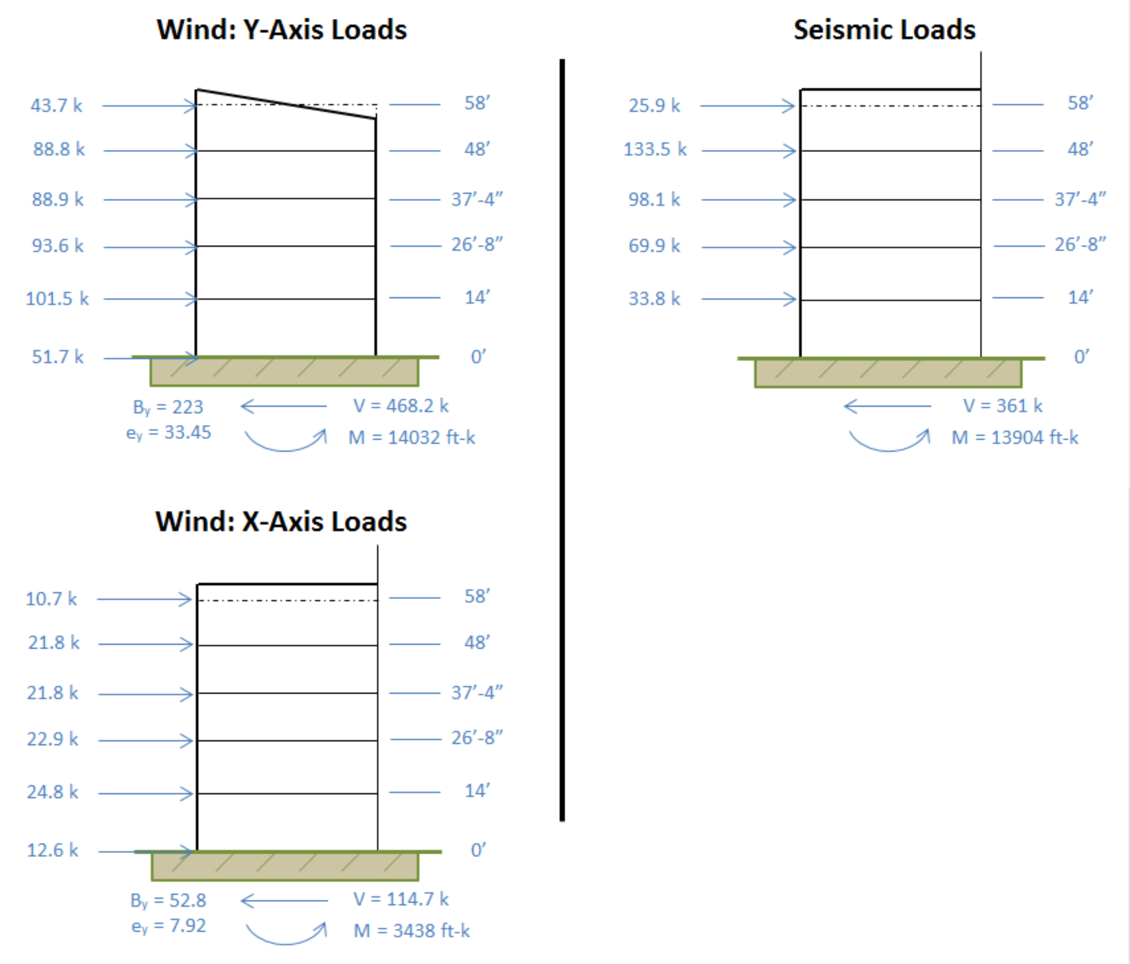
ETABS model

- columns modeled as found and slab modeled as a rigid diaphragm with the weight of each floor
- 0.7 I_g multiplied applied to columns according to Section 10.10.4.1 of ACI 318-08

assumptions

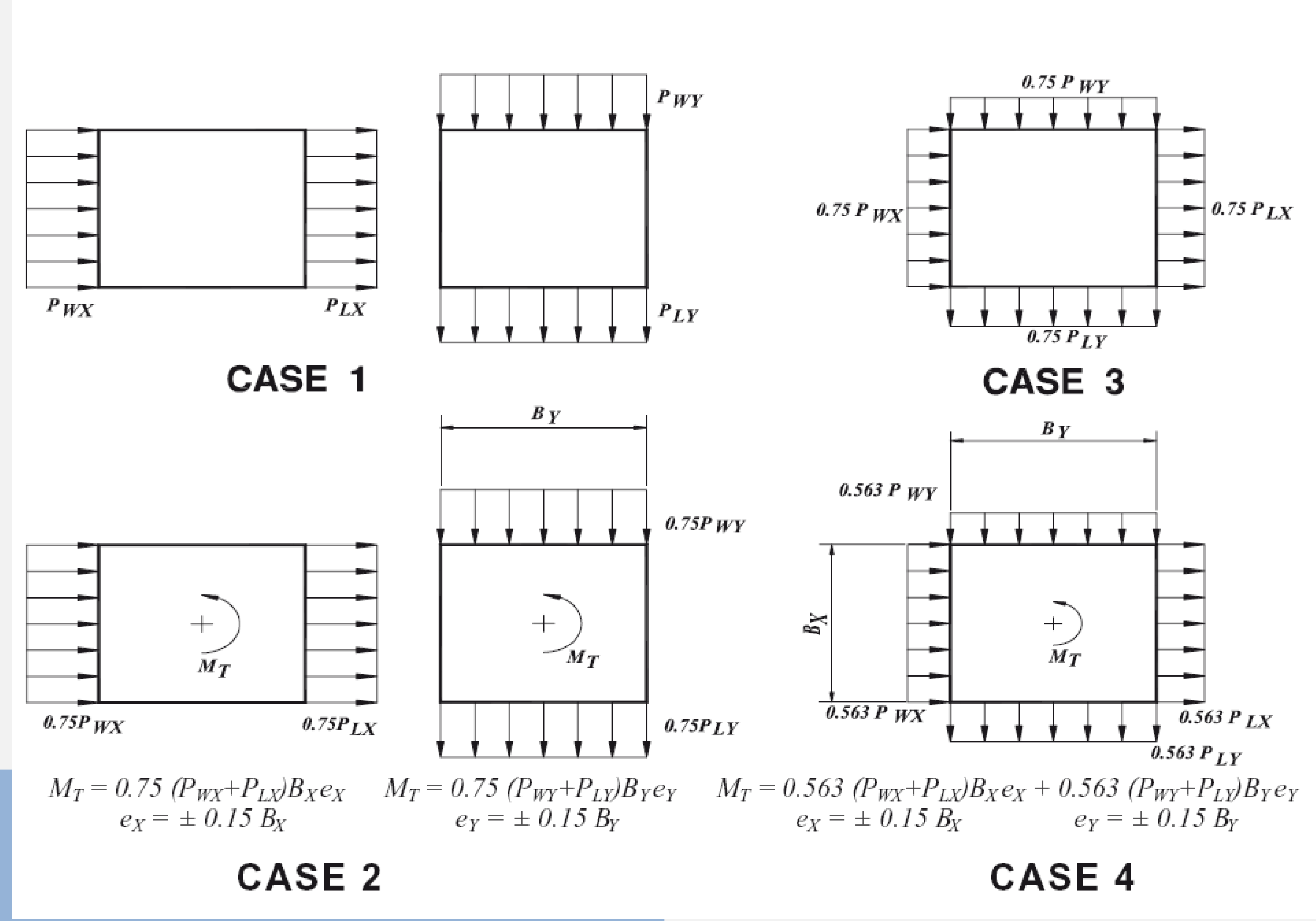
- rectangular prism with dimensions 223’ x 53’ x58’
- same ground to roof height on each side of building
- concrete beams were modeled to represent moment connections





applied loads

- eight different load cases were input into ETABS model
- ASCE Standard 7-10 used
- controlling case in x-direction: seismic
- controlling case in y-direction: wind load case 2



Depth – Lateral Redesign

lateral drift

- seismic drift check: $.015h_{sx}$
- wind story drift check: $h_{sx}/400$

Story Drifts (in)						
Level	Seismic			Wind		
	$\Delta_{X\text{-Frame}}$	$\Delta_{Y\text{-Frame}}$	$\Delta_{Allowable}$	$\Delta_{X\text{-Frame}}$	$\Delta_{Y\text{-Frame}}$	$\Delta_{Allowable}$
Roof	0.079	0.091	1.800	0.019	0.114	0.300
Pent	0.141	0.160	1.921	0.031	0.189	0.320
4th	0.201	0.223	1.921	0.046	0.267	0.320
3rd	0.272	0.300	2.279	0.066	0.378	0.380
2nd	0.178	0.193	2.520	0.046	0.261	0.420
Total Drift	**0.871**	**0.967**	**10.441**	**0.208**	**1.209**	**1.740**
	√ ok	√ ok		√ ok	√ ok	

overturning moment

- wind loads controlled creating a maximum overturning moment of 14,032 ft-kips
- resisting moment of building: 254,445 ft-kips

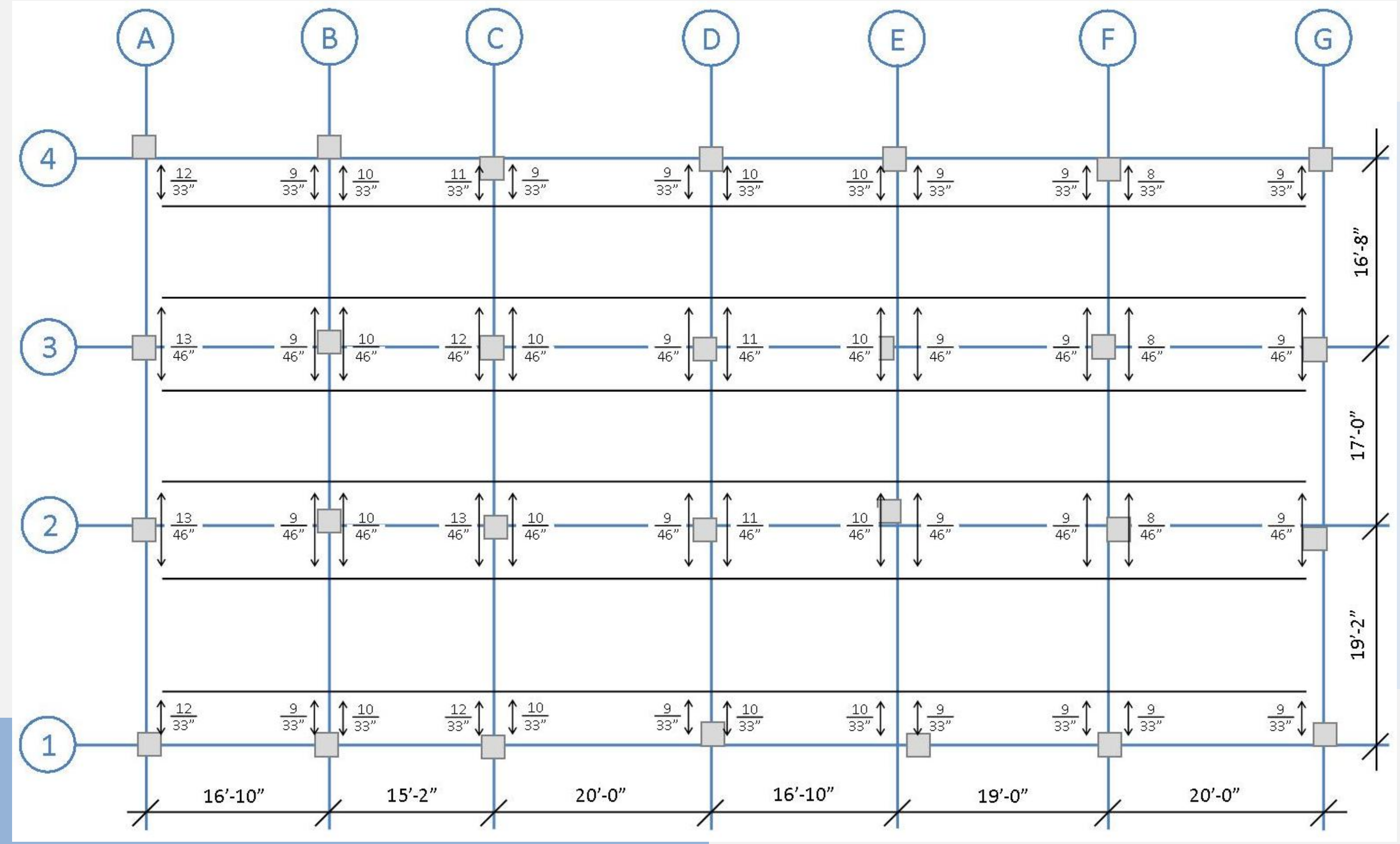
lateral load moments

- found using ETABS model

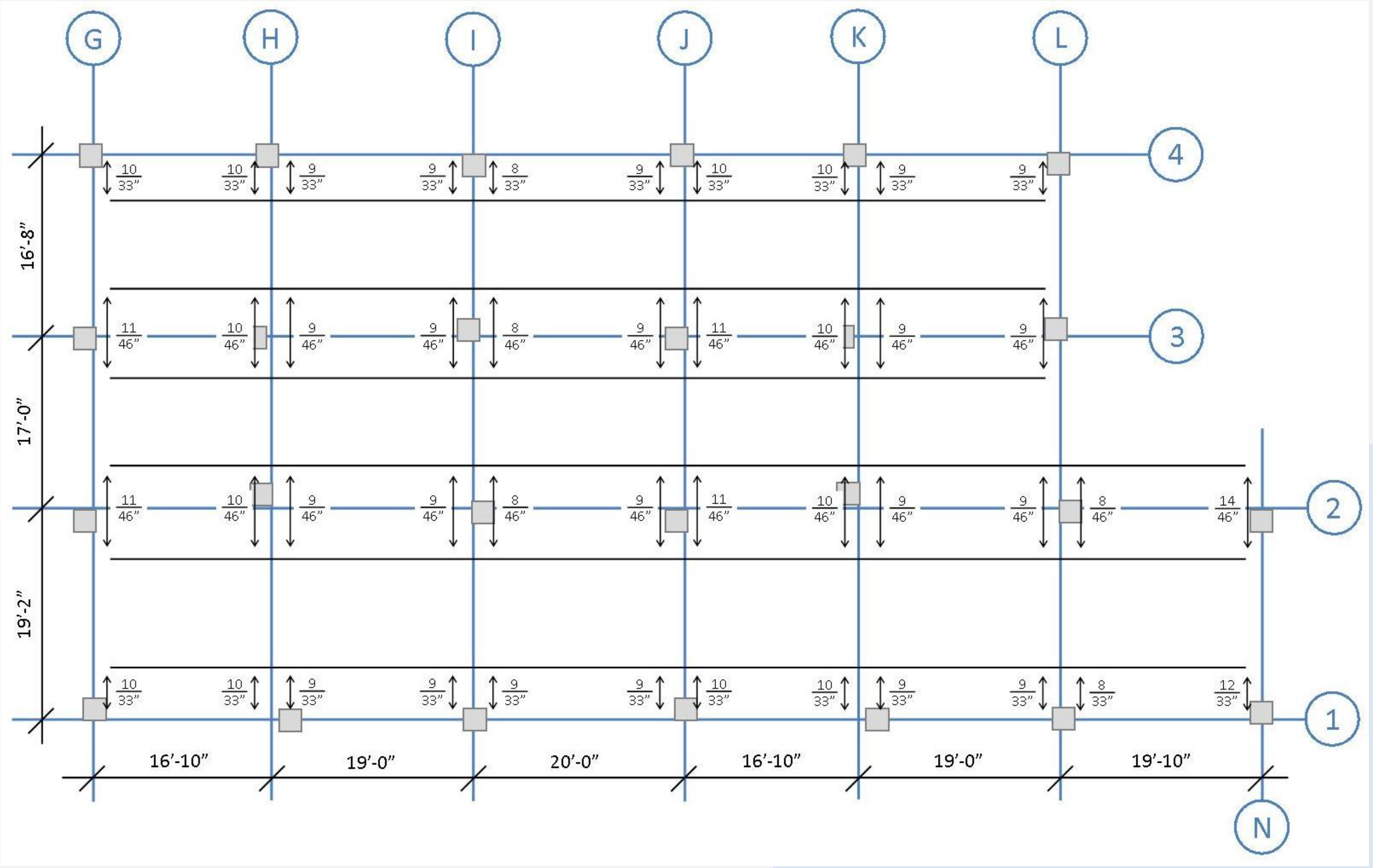
Floor	Lateral Load Moments (ft-k)	
	X-Direction	Y-Direction
2^{nd}	96	68
3^{rd}	118	64
4^{th}	106	43
Pent	48	19

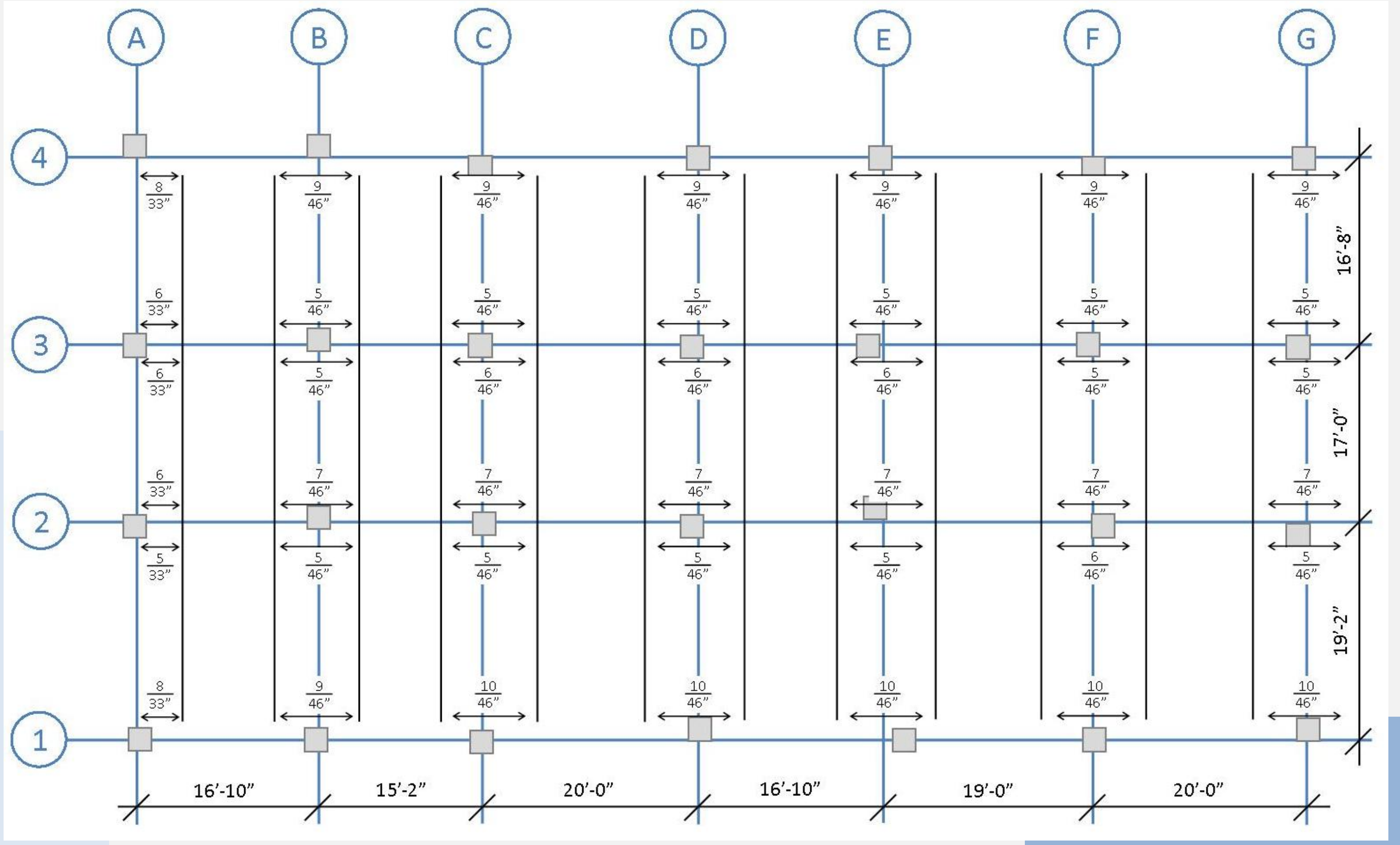
moment connection design

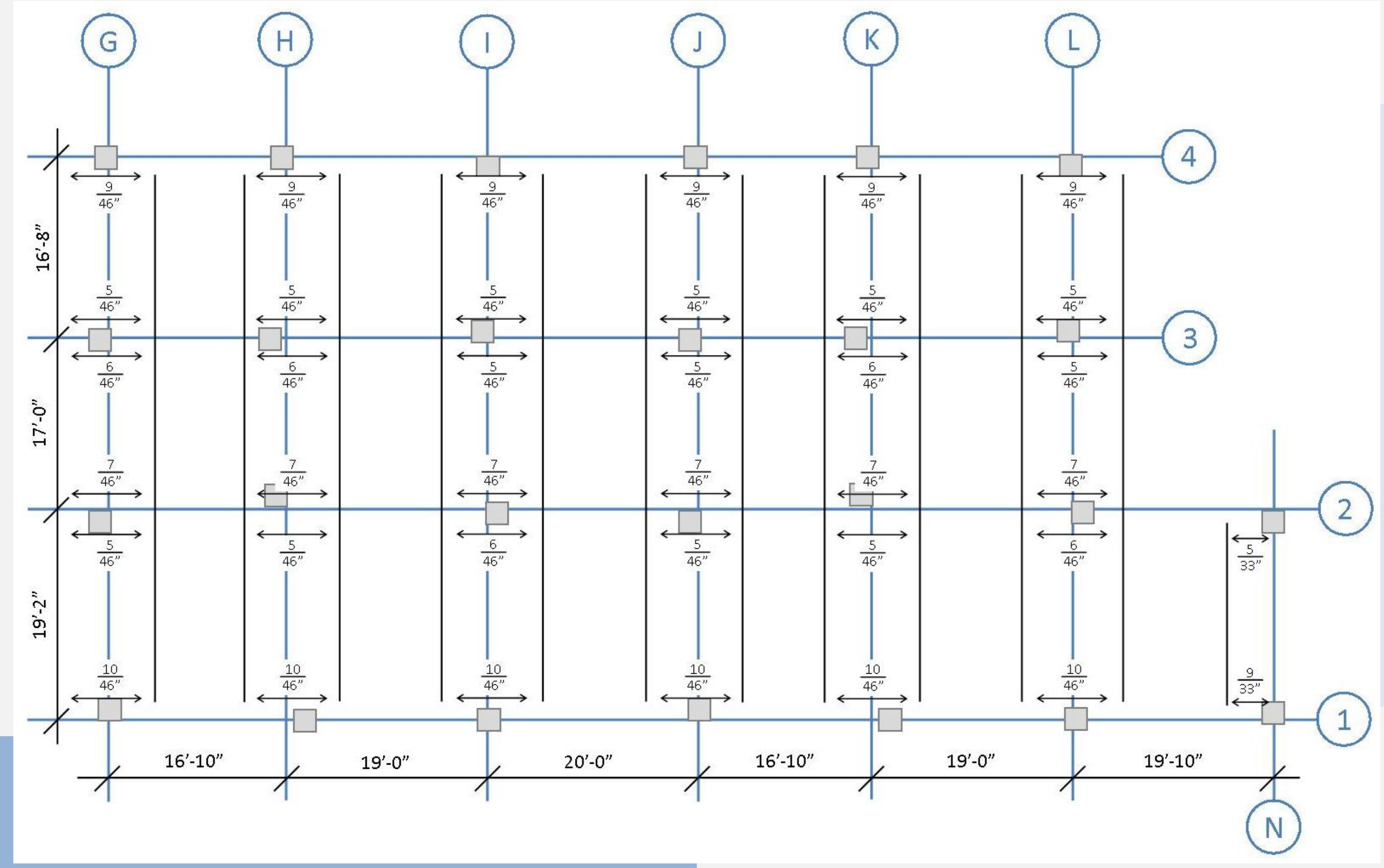
- lateral load moments added to unbalanced moment transferred by flexure due to gravity loads
- required reinforcement analyzed using total moment
- numbers refer to the amount of #5 bars that are equally spaced over the distance given



Depth – Lateral Redesign







advantages

- building more durable and lower maintenance costs
- improved fire rating and sound proofing
- stiffer structure produced
- less construction waste
- fewer firms involved

overall

- due to budget constraints, this system would not be permitted due to high upfront costs

disadvantages

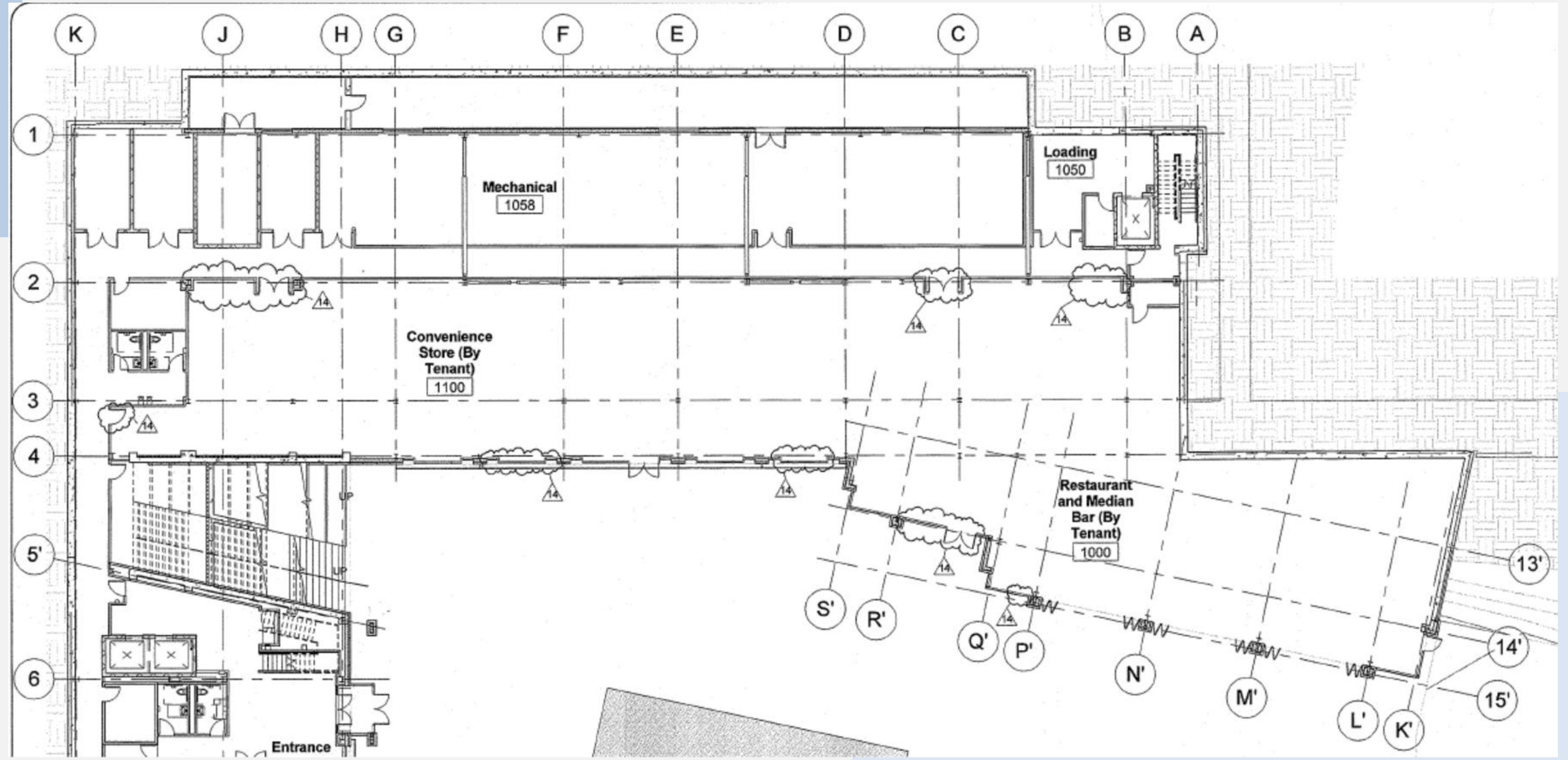
- structure cost was estimated at around triple the existing structure cost
- increased field labor

Questions

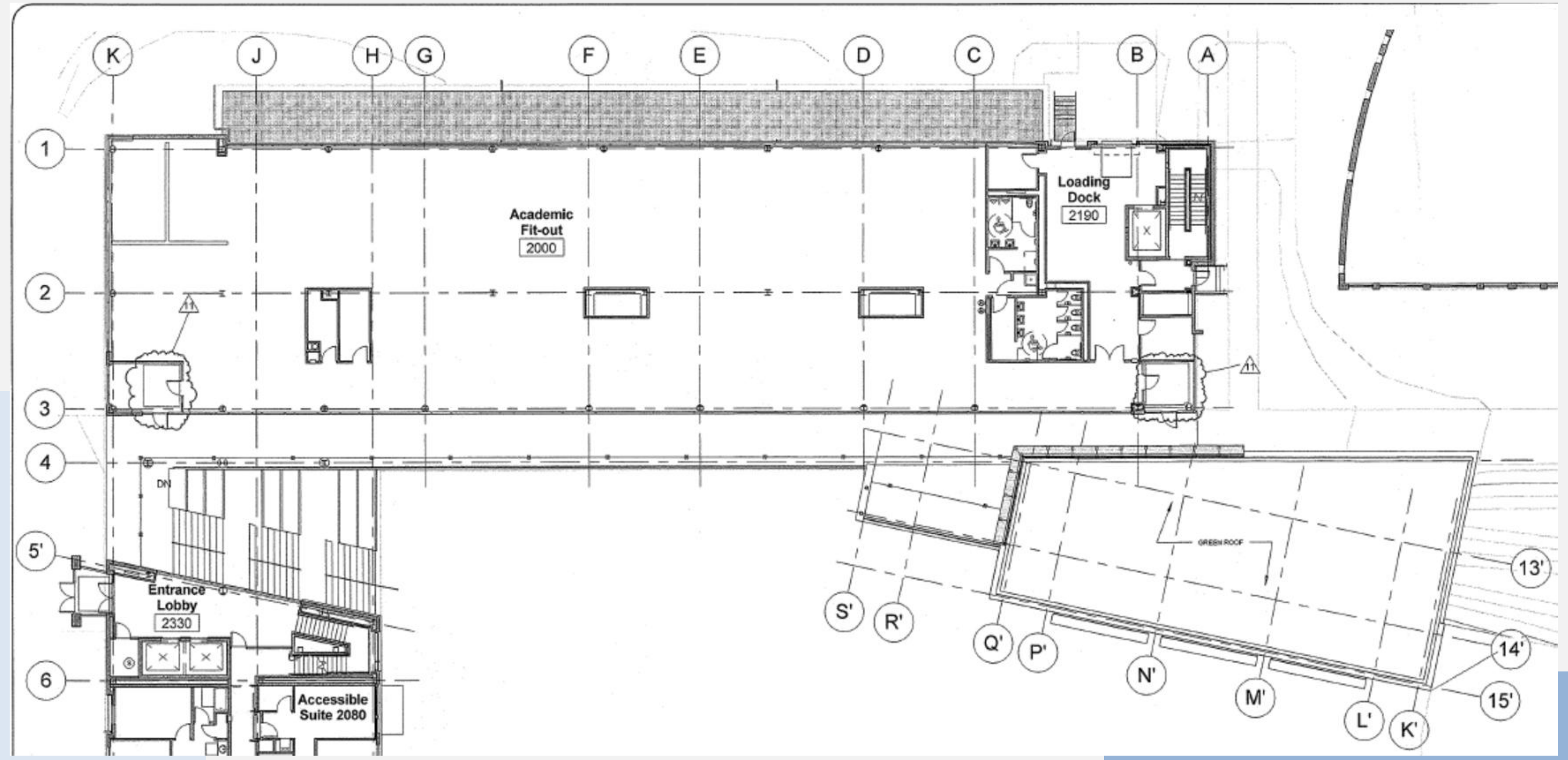
acknowledgements

- James Yarrington and Ted Weymouth (RIT)
- David Manoz (PSU)
- Dr. Linda Hangan
- Dr. Ali Memari
- Dr. Andrés Lepage
- Professor Holland
- Professor Parfitt
- Rest of the AE faculty
- Family and friends

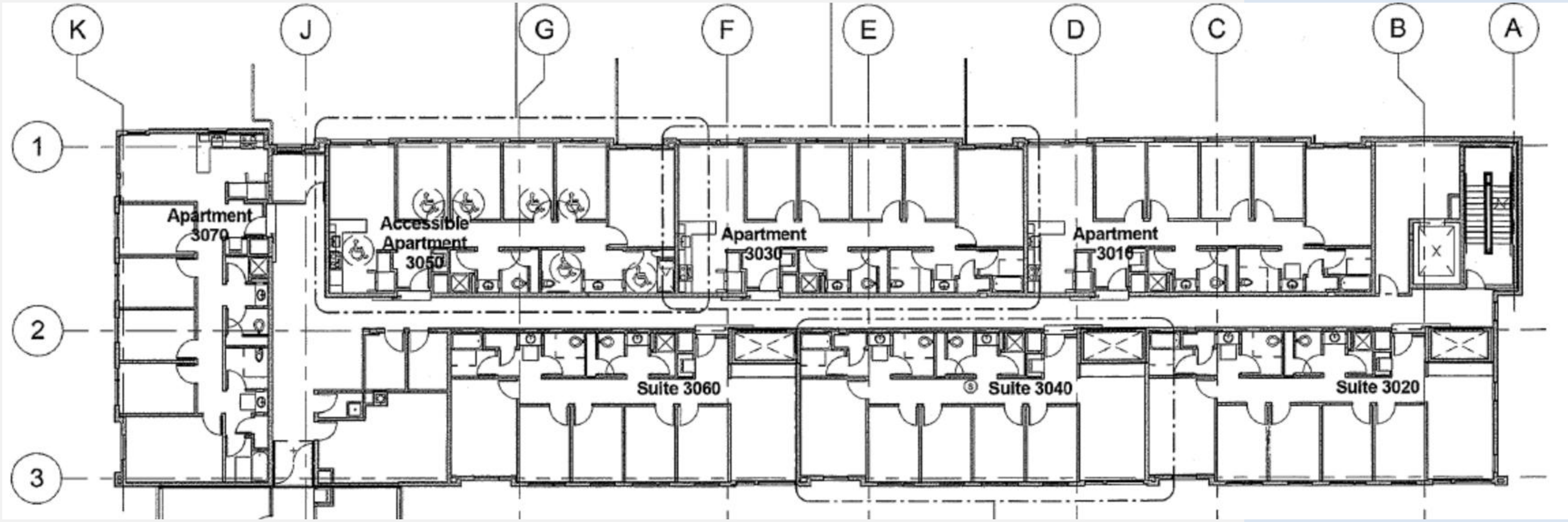
Appendix



Mechanical
1058
Loading
1050
Convenience
Store (By
Tenant)
1100
Restaurant
and Median
Bar (By
Tenant)
1000
Entrance

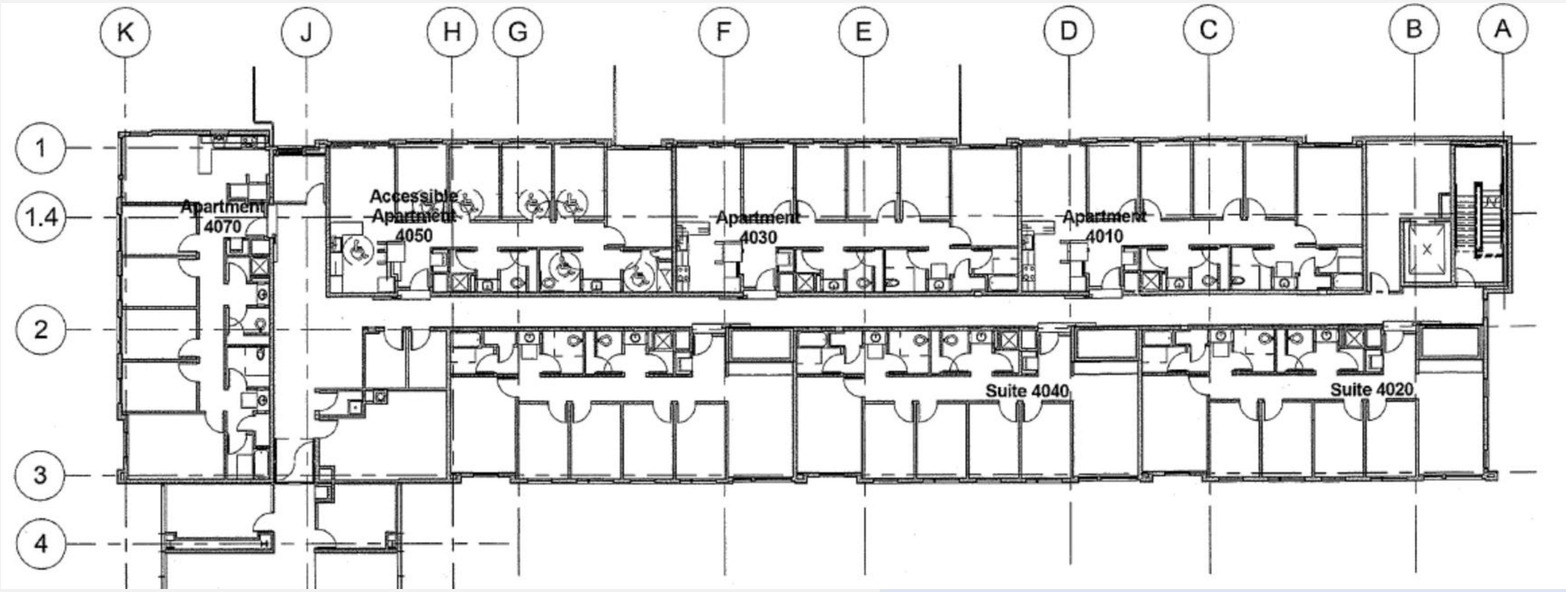


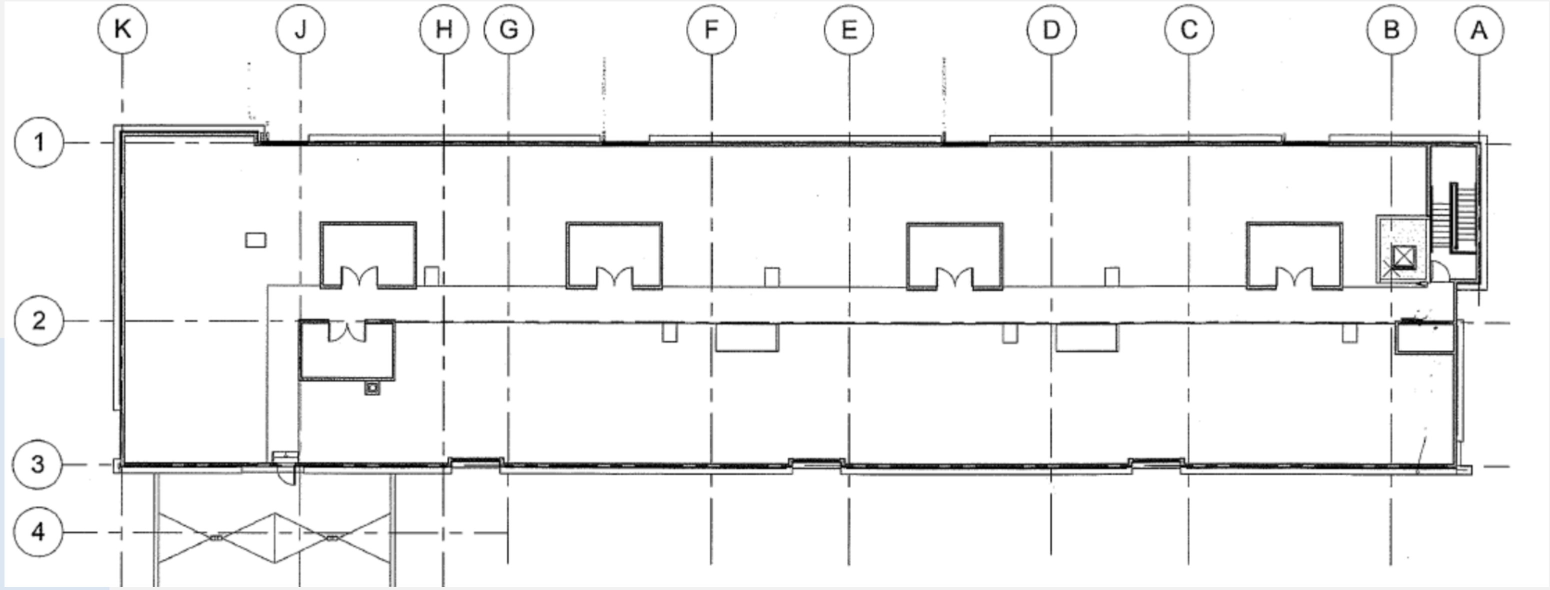
Academic
Fit-out
2000
Loading
Dock
2190
Entrance
Lobby
2330
Accessible
Suite 2080



Apartment
3070
Accessible
Apartment
3050
Apartment
3030
Apartment
3010
Suite 3060
Suite 3040
Suite 3020

Appendix


K
J
H
G
F
E
D
C
B
A
1
1.4
2
3
4
Apartment 4070
Accessible Apartment 4050
Apartment 4030
Apartment 4010
Suite 4040
Suite 4020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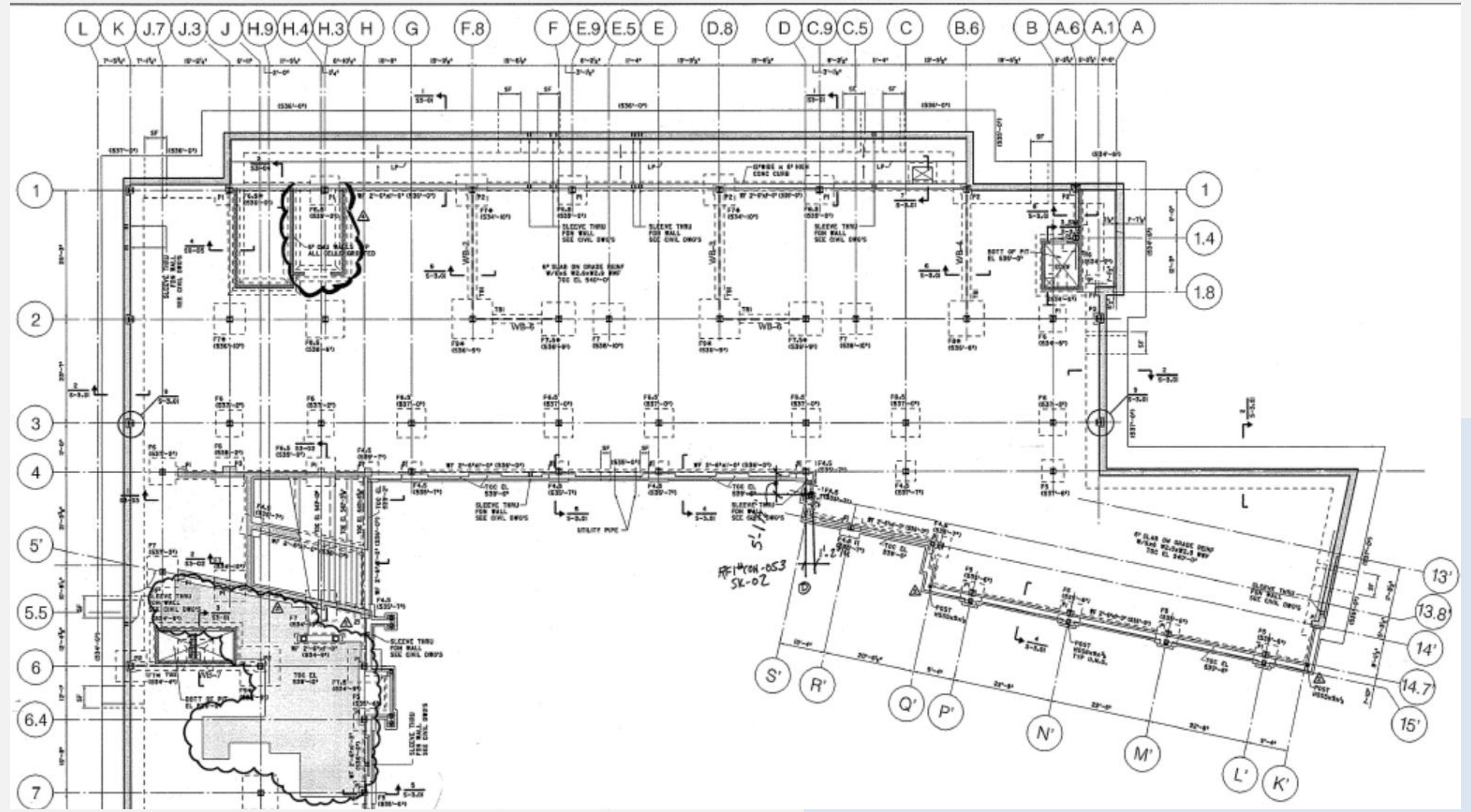

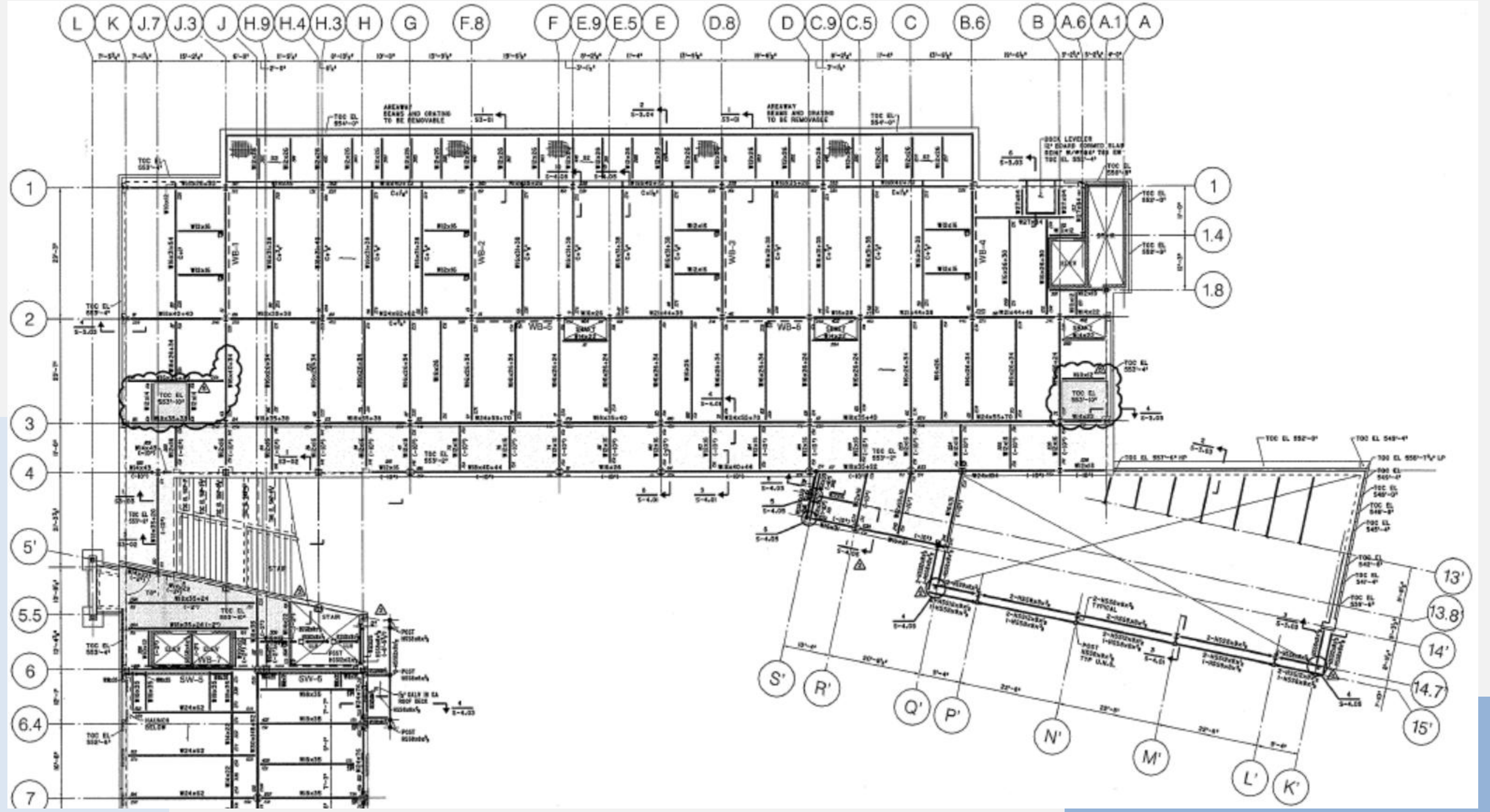

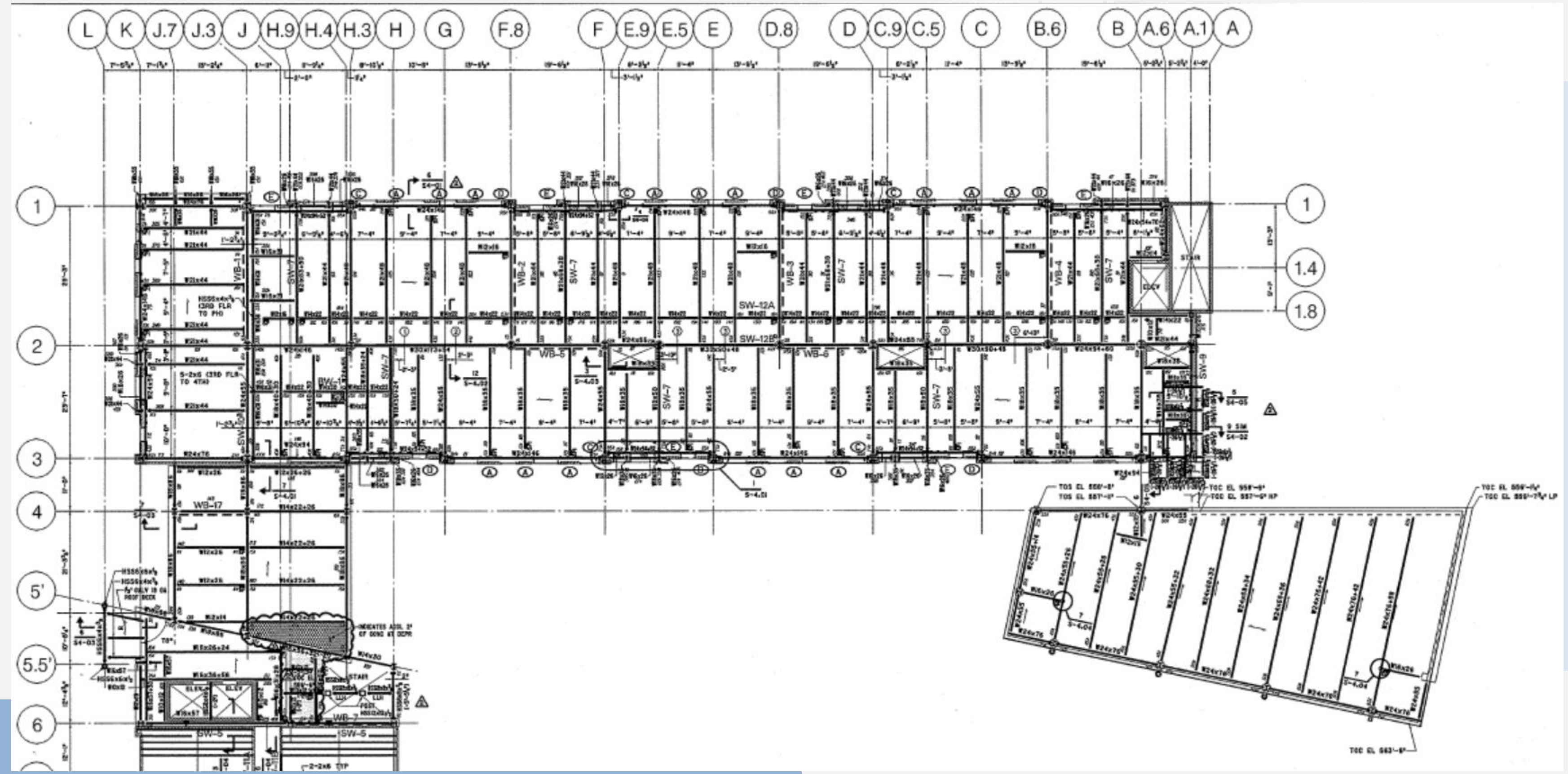

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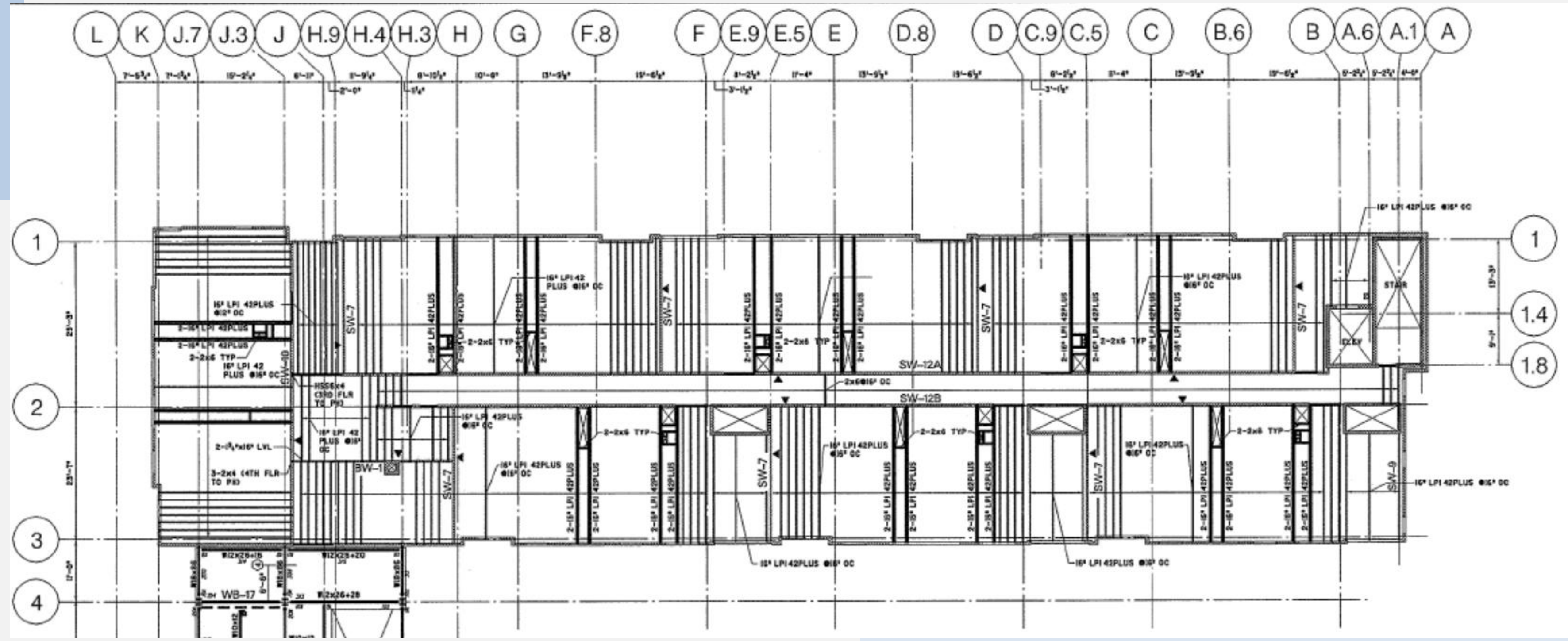
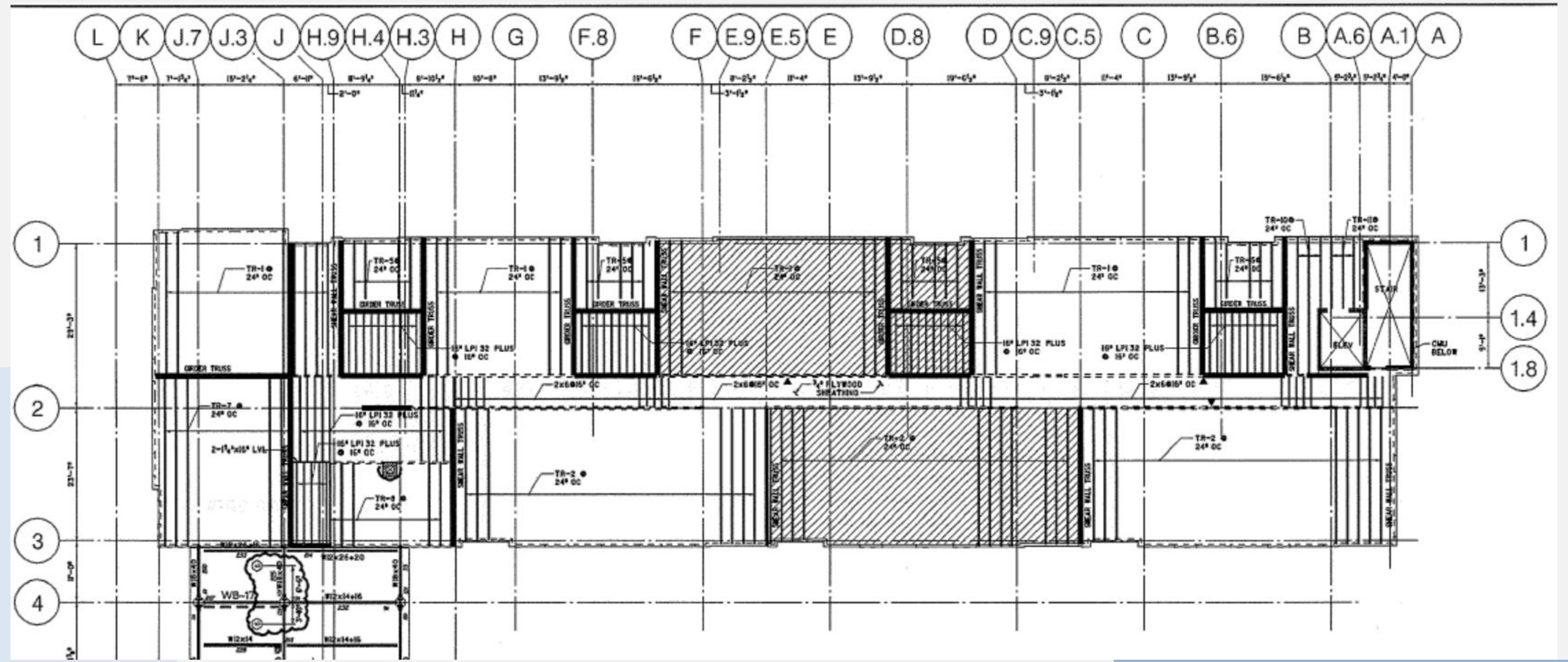




Appendix

Variable	Value	Reference
I_e=	1.25	Table 1.5-2
S_s=	0.21	USGS
S_1=	0.06	USGS
Site Class:	C	Geotech Report
F_a=	1.2	Table 11.4-1
F_v=	1.7	Table 11.4-2
S_{ms}=	0.252	
S_{m1}=	0.102	
S_{DS}=	0.168	
S_{D1}=	0.068	
Category:	B	Table 11.6-1,2
R=	3	Table 12.2-1: Ordinary RC Moment Frame
T_L=	6 sec	Fig 22-12

Equivalent Lateral Force Procedure			
C_t=	0.02		Table 12.8-2: Other Structures
x=	0.75		
h_n=	62.5	ft	
T_a=	0.445	sec	
C_u=	1.7		Table 12.8-1
T=	0.756	sec	
k=	1.128		
C_S=	0.070		
$C_{S,max}$=	0.037		
$C_{S,min}$=	0.010		
Use C_S=	0.037		

Weight of Floors

1st Floor:			2nd Floor:			3rd Floor:		
SDL=	5	psf	SDL=	5	psf	SDL=	5	psf
MEP=	10	psf	MEP=	10	psf	MEP=	10	psf
Partitions=	15	psf	Partitions=	15	psf	Partitions=	15	psf
Slab=	106.3	psf	Ceiling=	5	psf	Ceiling=	5	psf
MEP Equip=	150	psf	Slab=	106.3	psf	Slab=	106.3	psf
A_{Mech}=	4314	ft^2	A_{Total}=	12456	ft^2	A_{Total}=	12456	ft^2
A_{Other}=	12456	ft^2						
Weight:	2345	kips	Weight:	1760	kips	Weight:	1760	kips

4th Floor:			Penthouse:			Roof:		
SDL=	5	psf	SDL=	5	psf	SDL=	5	psf
MEP=	10	psf	MEP=	10	psf	Framing=	15	psf
Partitions=	15	psf	Partitions=	20	psf	Insulation=	3	psf
Ceiling=	0	psf	Ceiling=	0	psf	20% Snow=	6.16	psf
Slab=	106.3	psf	Slab=	106.3	psf			
			MEP Equip=	150	psf			
A_{Total}=	12456	ft^2	A_{Mech}=	744	ft^2	A_{Total}=	11487	ft^2
			A_{Other}=	11487	ft^2			
Weight:	1698	kips	Weight:	1735	kips	Weight:	335	kips

Seismic Forces

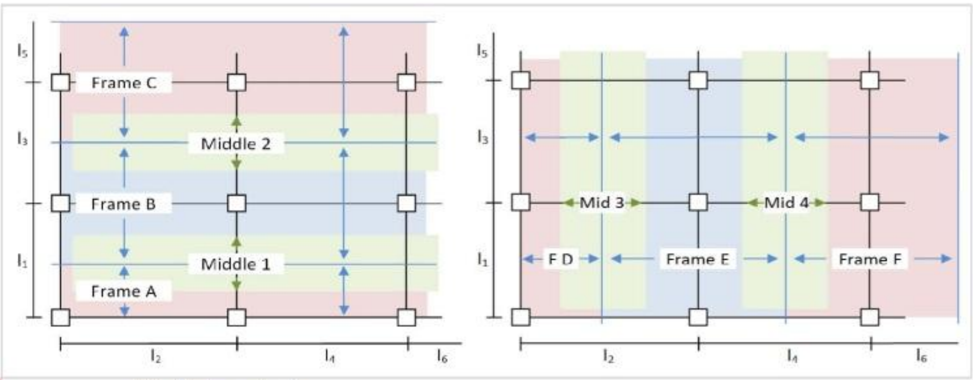
Building C

Floor	Floor Weight, w_x (k)	Story Height, h_x (ft)	$w_x h_x^k$	C_{vx}	Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)
Ground	2345	0.0	0.00	0.00	0.0	361.1	0.0
2nd	1760	14.0	89799.12	0.09	33.8	361.1	473.2
3rd	1760	26.7	185685.30	0.19	69.9	327.3	1863.3
4th	1698	37.3	260630.10	0.27	98.1	257.4	3662.1
Pent	1735	48.0	354584.69	0.37	133.5	159.3	6406.4
Roof	335	58.0	68682.22	0.07	25.9	25.9	1499.4
Sum:	9632		959381.4	1.00	361.1		
				√ ok	√ ok		
Base Shear ($V=C_sW$) =			361	Total Overturning Moment =			13904

Appendix

Flat Plate With No Edge Beams (By Direct Design Method)

$l_{max,int}$=	20	ft	l_1=	19.163	ft
$l_{max,ext}$=	20	ft	l_3=	17	ft
f'_c=	4000	psi	l_5=	16.67	ft
f_y=	60000	psi	l_2=	16.833	ft
w_{SD}=	35	psf	l_4=	19	ft
w_L=	100	psf	l_6=	20	ft
$t_{col,1dir}$=	20	in	$Width_{FA}$=	9.5815	ft
$t_{col,2dir}$=	20	in	$Width_{FB}$=	18.0815	ft
$l_{n,1}$=	17.50	ft	$Width_{FC}$=	16.835	ft
$l_{n,2}$=	15.17	ft	$Width_{FD}$=	8.4165	ft
$l_{n,3}$=	15.33	ft	$Width_{FE}$=	17.9165	ft
$l_{n,4}$=	17.33	ft	$Width_{FF}$=	19.5	ft
$l_{n,5}$=	15.00	ft	Need to change orientation so l2 > l1		
$l_{n,6}$=	18.33	ft			

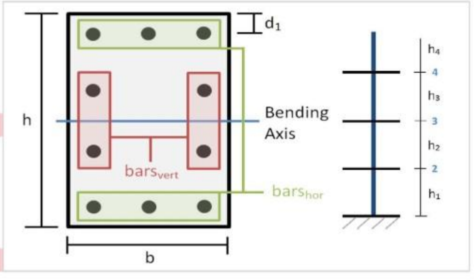


Note: Make sure $l_2 > l_1$

Column Design of Ground Floor Columns

Trial Column		
b=	20	in
h=	20	in
Use #	10	bars
d_1=	2.5	in
$bars_{vert}$=	2	
$bars_{hor}$=	6	
Floors=	5	
Note: Includes roof but not ground		
h_5=	10	ft
h_4=	10.67	ft
h_3=	10.67	ft
h_2=	12.66	ft
h_1=	14	ft

Roof Slope=	2	/ 12	
$w_{SD,roof}$=	23	psf	
$w_{L,roof}$=	20	psf	
w_{snow}=	30.8	psf	
$w_{SD,5}$=	35	psf	
$w_{L,5}$=	150	psf	Non-Reducible
$w_{SD,4}$=	30	psf	
$w_{L,4}$=	40	psf	
$w_{SD,3}$=	35	psf	
$w_{L,3}$=	40	psf	
$w_{SD,2}$=	35	psf	
$w_{L,2}$=	100	psf	Non-Reducible
$w_{SD,ground}$=	N/A	psf	
$w_{L,ground}$=	N/A	psf	



Column Strength / Strength Interaction Curve

Pure Compression		
P_o=	1915.7	kips
ϕP_o=	1245.2	kips

Pure Tension		
T_o=	-589.0	kips
ϕT_o=	-530.1	kips

Pure Bending (Solve by Hand)

Balanced-Strain Strength						
ε_y=	0.00207			β_1=	0.85	
c=	10.36	in < h	OK	A_s=	1.227	in^2
d_1=	2.50	in		f_{s1}=	60.00	ksi
d_2=	10.00	in		f_{s2}=	3.00	ksi
d_3=	17.50	in		f_{s3}=	-60.00	ksi
d_4=		in		f_{s4}=		ksi
d_5=		in		f_{s5}=		ksi
d_6=		in		f_{s6}=		ksi
d_7=		in		f_{s7}=		ksi
d_8=		in		f_{s8}=		ksi
P_b=	606.0	kips		M_b=	555.4	ft-k
ϕP_b=	393.9	kips		ϕM_b=	361.0	ft-k

Column BD				Column BE				Column BF			
$t_{col,1dir}$=	20	in		$t_{col,1dir}$=	20	in		$t_{col,1dir}$=	20	in	
$t_{col,2dir}$=	20	in		$t_{col,2dir}$=	20	in		$t_{col,2dir}$=	20	in	
A_T=	608.731779	ft^2		A_T=	1295.82878	ft^2		A_T=	1410.357	ft^2	
$A_{T,roof}$=	152.182945	ft^2		$A_{T,roof}$=	323.957195	ft^2		$A_{T,roof}$=	352.58925	ft^2	
$K_{LL}A_T$=	2434.92712	ft^2		$K_{LL}A_T$=	5183.31512	ft^2		$K_{LL}A_T$=	5641.428	ft^2	
$K_{LL}A_T$ >	$400ft^2$		OK	$K_{LL}A_T$ >	$400ft^2$		OK	$K_{LL}A_T$ >	$400ft^2$		OK
α=	0.55			α=	0.46			α=	0.45		
α_{roof}=	1.00			α_{roof}=	0.88			α_{roof}=	0.85		

Column BD			Column BE			Column BF		
$M_{ETABS,long}$=	96	ft-k	$M_{ETABS,long}$=	96	ft-k	$M_{ETABS,long}$=	96	ft-k
$M_{ETABS,short}$=	68	ft-k	$M_{ETABS,short}$=	68	ft-k	$M_{ETABS,short}$=	68	ft-k
$M_{unb,long}$=	31.7	ft-k	$M_{unb,long}$=	27.3	ft-k	$M_{unb,long}$=	25.0	ft-k
$M_{unb,short}$=	13.3	ft-k	$M_{unb,short}$=	27.5	ft-k	$M_{unb,short}$=	29.9	ft-k
P_L=	44.8	kips	P_L=	92.9	kips	P_L=	100.8	kips
P_D=	105.9	kips	P_D=	206.0	kips	P_D=	222.7	kips
$P_{S,Lr}$=	7.7	kips	$P_{S,Lr}$=	15.7	kips	$P_{S,Lr}$=	16.8	kips
$M_{u,long}$=	127.7	ft-k	$M_{u,long}$=	123.3	ft-k	$M_{u,long}$=	121.0	ft-k
$M_{u,short}$=	81.3	ft-k	$M_{u,short}$=	95.5	ft-k	$M_{u,short}$=	97.9	ft-k
P_T=	202.6	kips	P_T=	403.6	kips	P_T=	437.0	kips

Appendix

Interior Column BF (Reinforcement Needed)

$t_{col,1dir}$=	20	in	b_o=	108.50	in
$t_{col,2dir}$=	20	in	b_1=	27.13	in
$M_{u,long}$=	41.7	ft-k	b_2=	27.13	in
$M_{u,short}$=	49.9	ft-k	$V_{c,1}$=	195.6	kips
			$V_{c,2}$=	293.4	kips
			$V_{c,3}$=	226.2	kips
V_u=	116.2	kips	ϕV_c=	146.7	kips

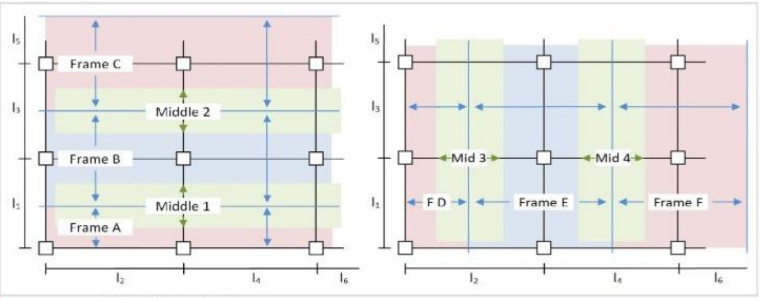
Transferred by Eccentricity of Shear

V_u=	116.2	kips				V_u=	116.2	kips			
$M_{uv,long}$=	16.7	ft-k				$M_{uv,short}$=	19.9	ft-k			
Centroid=	13.56	in				Centroid=	13.56	in			
J_c=	96434	in^4				J_c=	96434	in^4			
A_c=	773	in^2				A_c=	773	in^2			
ν_l=	122	psi				ν_l=	117	psi			
ν_r=	178	psi				ν_r=	184	psi			
ν_u=	178	psi				ν_u=	184	psi			
$\phi\nu_n$=	190	psi	>	ν_u	OK	$\phi\nu_n$=	190	psi	>	ν_u	OK

Appendix

Flat Plate With No Edge Beams (By Direct Design Method)

$l_{max,int}$=	20	ft	l_1=	19.163	ft
$l_{max,ext}$=	20	ft	l_3=	17	ft
f'_c=	4000	psi	l_5=	16.67	ft
f_y=	60000	psi	l_2=	19	ft
w_{SD}=	35	psf	l_4=	20	ft
w_L=	100	psf	l_6=	16.833	ft
$t_{col,1dir}$=	20	in	$Width_{FA}$=	9.5815	ft
$t_{col,2dir}$=	20	in	$Width_{FB}$=	18.0815	ft
$l_{n,1}$=	17.50	ft	$Width_{FC}$=	16.835	ft
$l_{n,2}$=	17.33	ft	$Width_{FD}$=	9.5	ft
$l_{n,3}$=	15.33	ft	$Width_{FE}$=	19.5	ft
$l_{n,4}$=	18.33	ft	$Width_{FF}$=	18.4165	ft
$l_{n,5}$=	15.00	ft	Need to change orientation so l2 > l1		
$l_{n,6}$=	15.17	ft			



Note: Make sure $l_2 > l_1$

Slab Thickness

t_{input}=	0.00	in			
$t_{min,int}$=	6.67	in			
$t_{min,ext}$=	8.07	in			
Use t_{slab}=	8.50	in	>	5"	OK

Wide Beam Action

$l_{max,FB}$=	18.1	ft	$l_{max,1dir}$=	19.2	ft
$l_{max,2dir}$=	20	ft	$l_{max,FE}$=	19.5	ft
d_{avg}=	7.13	in			
w_u=	329.5	psf			

Note: Dimensions from Same Bay

Long Direction

V_u=	51.1	kips			
ϕV_c=	146.8	kips	>	V_u	OK

Short Direction

V_u=	52.5	kips			
ϕV_c=	158.2	kips	>	V_u	OK

Punching Shear

	$l_{max,FB}$=	18.1	ft			
	$l_{max,FE}$=	19.5	ft			
	V_u=	114.6	kips			
	b_o=	108.5	in			
Take Minimum Value	$V_{c,1}$=	195.6	kips			
	$V_{c,2}$=	293.4	kips			
	$V_{c,3}$=	226.2	kips			
	ϕV_c=	146.7	kips	>	V_u	OK

Deflection Check

Assume:	25	% of w_L is sustained
	90	% of immediate deflection due to dead load occurs before partitions are installed
	x	Check if: Nonstructural attached elements will be damaged by excessive deflection

Interior Panel l_3 - l_4

Column Strip			Middle Strip		
$I_{g,col}$=	5552	in^4	$I_{g,mid}$=	7062	in^4
w_D=	1.724	k/ft	w_D=	0.918	k/ft
w_L=	1.221	k/ft	w_L=	0.650	k/ft
$\Delta_{D,max}$=	0.062	in	$\Delta_{D,max}$=	0.014	in
$\Delta_{L,max}$=	0.081	in	$\Delta_{L,max}$=	0.018	in
$\Delta_{long\text{-}term}$=	0.246	in	$\Delta_{long\text{-}term}$=	0.054	in

Check Live Load Deflection

Δ_L=	0.099	in	
ACI Limit=	0.667	in	OK

Check Total Load Deflection

Δ_T=	0.406	in	
ACI Limit=	0.500	in	OK

Exterior Panel l_1 - l_2

Column Strip			Middle Strip		
$I_{g,col}$=	5552	in^4	$I_{g,mid}$=	5834	in^4
w_D=	1.724	k/ft	w_D=	0.872	k/ft
w_L=	1.221	k/ft	w_L=	0.618	k/ft
$\Delta_{D,max}$=	0.050	in	$\Delta_{D,max}$=	0.025	in
$\Delta_{L,max}$=	0.066	in	$\Delta_{L,max}$=	0.033	in
$\Delta_{long\text{-}term}$=	0.201	in	$\Delta_{long\text{-}term}$=	0.100	in

Check Live Load Deflection

Δ_L=	0.099	in	
ACI Limit=	0.633	in	OK

Check Total Load Deflection

Δ_T=	0.407	in	
ACI Limit=	0.475	in	OK

Exterior Panel l_3 - l_2

Column Strip			Middle Strip		
$I_{g,col}$=	5552	in^4	$I_{g,mid}$=	6448	in^4
w_D=	1.724	k/ft	w_D=	0.872	k/ft
w_L=	1.221	k/ft	w_L=	0.618	k/ft
$\Delta_{D,max}$=	0.050	in	$\Delta_{D,max}$=	0.014	in
$\Delta_{L,max}$=	0.066	in	$\Delta_{L,max}$=	0.026	in
$\Delta_{long\text{-}term}$=	0.201	in	$\Delta_{long\text{-}term}$=	0.062	in

Check Live Load Deflection

Δ_L=	0.092	in	
ACI Limit=	0.633	in	OK

Check Total Load Deflection

Δ_T=	0.361	in	
ACI Limit=	0.475	in	OK

Exterior Panel l_1 - l_4

Column Strip			Middle Strip		
$I_{g,col}$=	5859	in^4	$I_{g,mid}$=	5884	in^4
w_D=	1.859	k/ft	w_D=	0.880	k/ft
w_L=	1.316	k/ft	w_L=	0.623	k/ft
$\Delta_{D,max}$=	0.053	in	$\Delta_{D,max}$=	0.030	in
$\Delta_{L,max}$=	0.070	in	$\Delta_{L,max}$=	0.039	in
$\Delta_{long\text{-}term}$=	0.212	in	$\Delta_{long\text{-}term}$=	0.119	in

Check Live Load Deflection

Δ_L=	0.109	in	
ACI Limit=	0.667	in	OK

Check Total Load Deflection

Δ_T=	0.448	in	
ACI Limit=	0.500	in	OK

Appendix

super-imposed dead loads

- 35 psf: SDL, MEP, partitions, ceiling
- 106 psf: 8½"slab self weight

live loads

- 100 psf: 2nd floor

stairwell corner design

- preliminary sizes and reinforcement given