Erie on the Park
Chicago, IL

TECHNICAL ASSIGNMENT 1

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Executive Summary

It is the intent of this report to analyze the structural design procedures implemented when designing 500 W. Erie St. Chicago, Illinois.

Building Description

Erie on the Park is a 25 story condominium complex on W. Erie St. in Chicago, IL. By using steel for the main structural system the architect on this project goes against the normal practice of using concrete as the major structural system for a residential high-rise building. In doing this he allows himself greater flexibility when designing the layout of each of the tenant spaces, and provides a strong architectural statement with the steel chevrons punctuating the building's façade. The entrance to the building is through a grand lobby with a 30' high ceiling. The next four stories are part of a parking garage with many spaces for tenants to park their cars out of the elements. The sixth floor has a fitness center and is the beginning of the tenant living spaces. Floors seven through 25 are condominiums that provide a dynamic living space and spectacular views of the Chicago skyline through the floor-to-ceiling windows.

Structural Design Code

Chicago Building Code

Calculations

When designing elements of this building I used the ASCE-7 design guide which uses different loadings and force distributions than the CBC. These differences account for some of the discrepancies between the member sizes that exist and the ones that I designed. Another reason for there to be differences is that the floor slabs were designed to withstand a horizontal diaphragm load of 250 PLF per bay. When I designed the floor slabs of various bays I did not take this into account. It is for these reasons that there is a difference between the existing structural elements and those that I designed.

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Structural System

Foundation

The foundation is made-up of hardpan caissons and grade-beams. The caissons are drilled up to a depth of 85'. This depth is required to find soil with a net bearing pressure of 30 KSF. The caisson shaft diameters range from 30" to 54" and the bell diameters range from 4' to 11'. The grade beams average about 36"x60" with the larges width being 72" and the greatest depth of 100". The grade beams frame into the caisson caps which have a minimum width of 6" larger than their respective caisson and a depth of 3'. These sizes would increase to the width and depth of the largest grade beam framing in to them. These three structural elements would have a concrete bearing capacity of f'_c = 6000 psi, and use deformed rebar in accordance with ASTM A615.

Columns

There are concrete columns from the ground level to the third floor, an overall elevation of 40'. These columns are either circular with a 30" diameter or rectangular with dimensions varying from 26" to 36" on each side. The circular columns are toward the southern end of the building where they are only framing into concrete slabs. The rectangular columns are towards the northern end of the building and frame into a steel mezzanine half way between the ground and second floor. The bearing capacity of the concrete is f'_c = 8000 psi.

At the third floor the concrete columns transition to steel W-shapes that continue the remaining 250' to the roof. The columns are ASTM A992 Grade 50 rolled W14 steel shapes. The largest columns are W14x257 and are part of the lateral system. The columns that are primarily part of the gravity system are W14x132's at the third floor down to W14x61's supporting the roof. These columns were generally erected in two story lifts, which are about 21'.

Floor System

The first through third floors have a two-way, flat-slab system. The first floor is slab-on-grade and is 10" thick west of column line 4 and 12" thick east of column line 4. The second and third levels both have 12" thick slabs with 12"x24" beams running in the N-S direction along column lines 3 and 4 from column line E to H. The rebar in these slabs and beams are epoxy coated and the beams are to have a capacity of f'_c = 6000 psi. The mezzanine levels and floors 4-6 have steel girders and beams with a partially composite slab on steel deck. The beams are typically W18x35 and span 26'-4" in the E-W direction and the girders are W16x26 and span 18'-8" in the N-S direction. The deck is 4-½" of normal weight concrete on 3" 18 gage composite steel decking reinforced with 6x6xW2.1xW2.1 WWF. The seventh through 25th floors are steel joist construction where 14K6 joists, 2' O.C., span 26' between W12x108 beams that span 26'-4". A 2" slab on 0.6C26 non-composite steel deck with 6x8xW1.4xW1.4 WWF. The roof is comprised of W21x26 beams 8'-8" O.C.

spanning 26' between W12x96 girders. The girders in turn span 26'-4". On top of the beams is a 3" 22 gage, hot dipped galvanized steel deck.

Lateral System

The lateral system between the ground level and the third level is comprised of cast-in-place concrete shear walls with a bearing capacity of $f'_c = 8000$ psi. There are two 27', 18" thick shear walls running in the E-W direction. There are three running in the N-S direction with lengths of 26', 29'-4", and 52' which are also 18" thick. These walls resist the lateral loads transferred down from steel brace frames on the upper floors. The braced frames, made up of W8 and W10 shapes, distribute the shear load through large three story triangles as seen in the façade.

Codes and Code Requirements

Building Code:
Chicago Building Code – Volume 1 (CBC 2000)

Referencing American National Standard Minimum Design Loads for Buildings and Other Structures ANSI-A58.1-1982

Structural Concrete:
The American Concrete Institute (ACI 318)

Concrete Masonry:
"Building Code Requirements for Concrete Masonry Structures"
The American Concrete Institute (ACI 530)

Structural Steel:
"LRFD Specification for Structural Steel Buildings, Second Edition"
The American Institute of Steel Construction (AISC-LRFD)

Loads

Live Loads	ASCE-7 Ch 4
Ground Floor:	100 PSF
Parking	50 PSF
Residential Floors	
Units	40 PSF
Partitions	15 PSF
Corridors	40 PSF
Roof	25 PSF + Drift

Dead Loads

Metal Deck	2-3 PSF	From Deck Catalogs
Reinforced Concrete	150 PSF	
Steel Joists	8 PLF	From Joist Catalog
Steel Beams	Various	From LRFD
Superimposed Loads	ASCE-7 Commentary Ch 3	
Ground Floor	25 PSF	
Parking	8 PSF	
Residential Floors		
Units	13 PSF	
Corridors	13 PSF	
Roof	17 PSF	

Lateral Loads

Wind ASCE-7 Ch 6

The wind pressures in the table below are due to the fact that this building is a flexible structure in an exposure category C.

	Wind Ward		Leeward		Total	
	N-S	E-W	N-S	E-W	N-S	E-W
0-15	12.12	12.12	-14.06	-10.47	26.18	22.59
20	12.83	12.83	-14.06	-10.47	26.90	23.30
25	13.40	13.40	-14.06	-10.47	27.47	23.87
30	13.97	13.97	-14.06	-10.47	28.04	24.44
40	14.83	14.83	-14.06	-10.47	28.89	25.30
50	15.54	15.54	-14.06	-10.47	29.61	26.01
60	16.11	16.11	-14.06	-10.47	30.18	26.58
70	16.68	16.68	-14.06	-10.47	30.75	27.15
80	17.25	17.25	-14.06	-10.47	31.32	27.72
90	17.68	17.68	-14.06	-10.47	31.75	28.15
100	17.97	17.97	-14.06	-10.47	32.03	28.44
120	18.68	18.68	-14.06	-10.47	32.74	29.15
140	19.39	19.39	-14.06	-10.47	33.46	29.86
160	19.82	19.82	-14.06	-10.47	33.88	30.29
180	20.39	20.39	-14.06	-10.47	34.45	30.86
200	20.82	20.82	-14.06	-10.47	34.88	31.29
250	21.82	21.82	-14.06	-10.47	35.88	32.29
300	22.67	22.67	-14.06	-10.47	36.74	33.14
350	23.39	23.39	-14.06	-10.47	37.45	33.86
400	24.10	24.10	-14.06	-10.47	38.16	34.57
450	24.67	24.67	-14.06	-10.47	38.73	35.14
500	25.24	25.24	-14.06	-10.47	39.30	35.71
290	22.50	22.50	-14.06	-10.47	36.57	32.97

Table 5.1 Wind Pressure Distribution

Base Shear:

N-S Direction	735.2 kips
E-W Direction	1075.6 kip

Overturning Moment:

N-S Direction	117240 ft-kip
E-W Direction	172280 ft-kip

Seismic Loads ASCE-7 Ch 9

Since the building was a seismic design category A, I was able to use the simplified analysis procedure to determine the shear distribution found in the table below.

Floor	Height	Area	Weight	Shear Distribution
	(ft)	(ft^2)	(kip)	(kip)
Roof	291.5	3100	248.0	6.75
25	274.58	5442	462.6	12.58
24	263.58	7622	647.9	17.62
23	252.92	5534	470.4	12.79
22	242.25	5534	470.4	12.79
21	231.58	6658	565.9	15.39
20	220.92	6658	565.9	15.39
19	210.25	6658	565.9	15.39
18	199.58	9230	784.6	21.34
17	188.92	9230	784.6	21.34
16	178.25	9230	784.6	21.34
15	167.58	9230	784.6	21.34
14	156.92	9230	784.6	21.34
13	145.25	9230	784.6	21.34
12	135.58	9230	784.6	21.34
11	124.92	9230	784.6	21.34
10	114.25	9230	784.6	21.34
9	103.58	9230	784.6	21.34
8	92.92	9230	784.6	21.34
7	82.25	9230	784.6	21.34
6	71.58	9230	923.0	25.11
5	60.00	9230	923.0	25.11
4	50.00	9230	923.0	25.11
3	40.00	9230	1476.8	40.17
2	29.75	9230	1476.8	40.17
1 - Ground	0.00	9230	1615.3	0.00
		213346	19134.2	520.45

Table 5.2 Seismic Shear Distribution

Base Shear 520.5 kip
Overturning Moment 70030 ft-kip

Earth ASCE-7 Ch 5

This building does not have a basement level where there would be lateral earth pressure on a foundation wall or similar element.

Snow Loads ASCE-7 Ch 7

Other

This building has not been designed for any other loading schemes, ie explosive, blast, anti-terrorism, etc. that I am aware of.

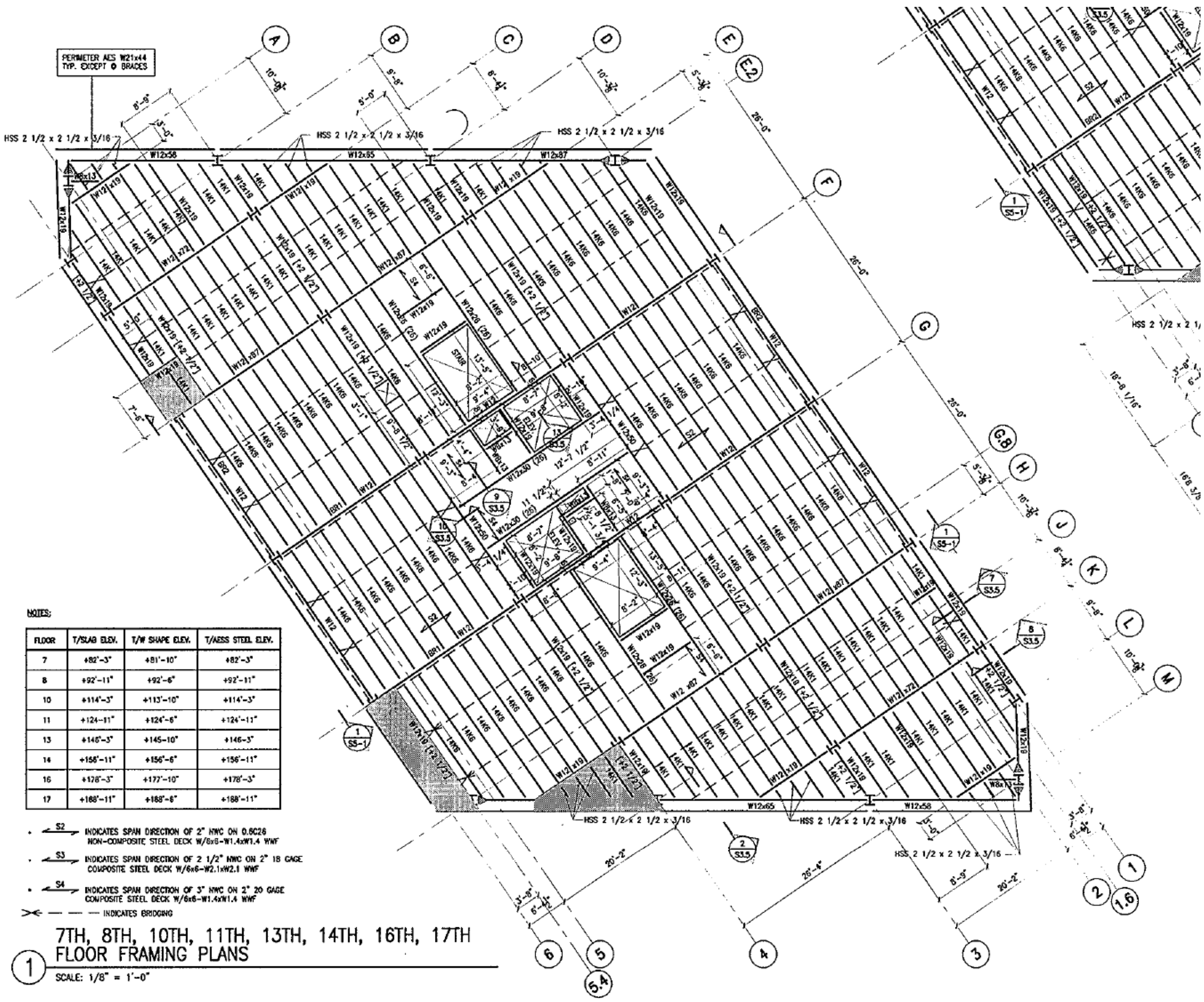
Spot-Checks

Through doing the spot checks I have noticed that most of the structural components in this building are larger than the components I designed under the given loading schemes.

I believe that there are a number of reasons specific to each structural system as to why I was designing smaller structural elements. When designing the floor slabs my designs were significantly smaller than those that are existent in the building. This is because the design engineer also designed the floor slabs to resist a diaphragm force per bay width of 250 PLF, a loading scenario I did not take into account. Using the loads of he slabs that are actually used in the building and the assumption that there was a 14" limit on the beam depth, I was able to get the same size beams and joists. When considering the loads on the columns, I neglected any residual moment that lateral forces or live load pattern loading would cause and that's the reason my column designs are smaller than the actual columns. When calculating the wind force the CBC uses a different distribution of lateral forces and that's why there is discrepancy between the size of the actual brace and the one I designed. I also did not consider torsional effects or the flexibility of lower floors.

Some of the systems in this building that I will need to check are the foundation, cladding system, and roof. The foundation will have to be designed for bearing, possible up lift due to high water tables, and tension from the lateral systems. The cladding system will have to be designed to withstand both internal and external pressures and projectiles that could become airborne in high winds. I will also have to consider roof uplift and snow drift.

A1: Floor Plan



A2: Wind Loads

Wind Load Analysis

Reference	Building Properties	
	B (ft)	79.33
	L (ft)	130
	h (ft)	290
Figure 6.4	K_{zt}	1
Table 6.4	K_d	0.85
	V (mph)	90
Table 1.1	Importance	III
Table 6.1	I_w	1.15
	Exposure	C
Table 6.2	α	9.5
Table 6.2	z_g	900
Table 6.2	z_{min}	174
Table 6.2	c	0.200
Table 6.2	$\in$	0.200
Table 6.2	l	500
Table 6.2	$\bar{b}$	0.650
Table 6.2	$\bar{\alpha}$	0.154
Table 6.2	$\underline{a}$	0.105
Table 6.2	$\underline{b}$	1.000

Reference	Period Parameters	
	Struct. Type	Other
Table 9.5.5.3	C_t	0.020
	x	0.750
	T	1.4055
	Natural f	0.7115
	Rigidity	Flex

Rigid	
$g_Q=g_v$	3.4
$\check{z}$	174
$I_{\check{z}}$	0.1516
$L_{\check{z}}$	697.2316
Q	0.8385
G	0.8553

Reference	Windward	
Figure 6.6	Cp	0.8

Flexible	
g_R	4.11
R_n	0.054
N_1	4.48
η_h	8.57
η_B	0.030
η_L	12.85
R_h	0.110
R_B	0.981
R_L	0.075
$V_{\check{z}}$	110.81
β	0.05
R	0.26
G_f	0.8794

Reference	Leeward		
	Direction	Ratio	Cp
Figure 6.6	N-S	0.610	-0.50
Figure 6.6	E-W	1.639	-0.37

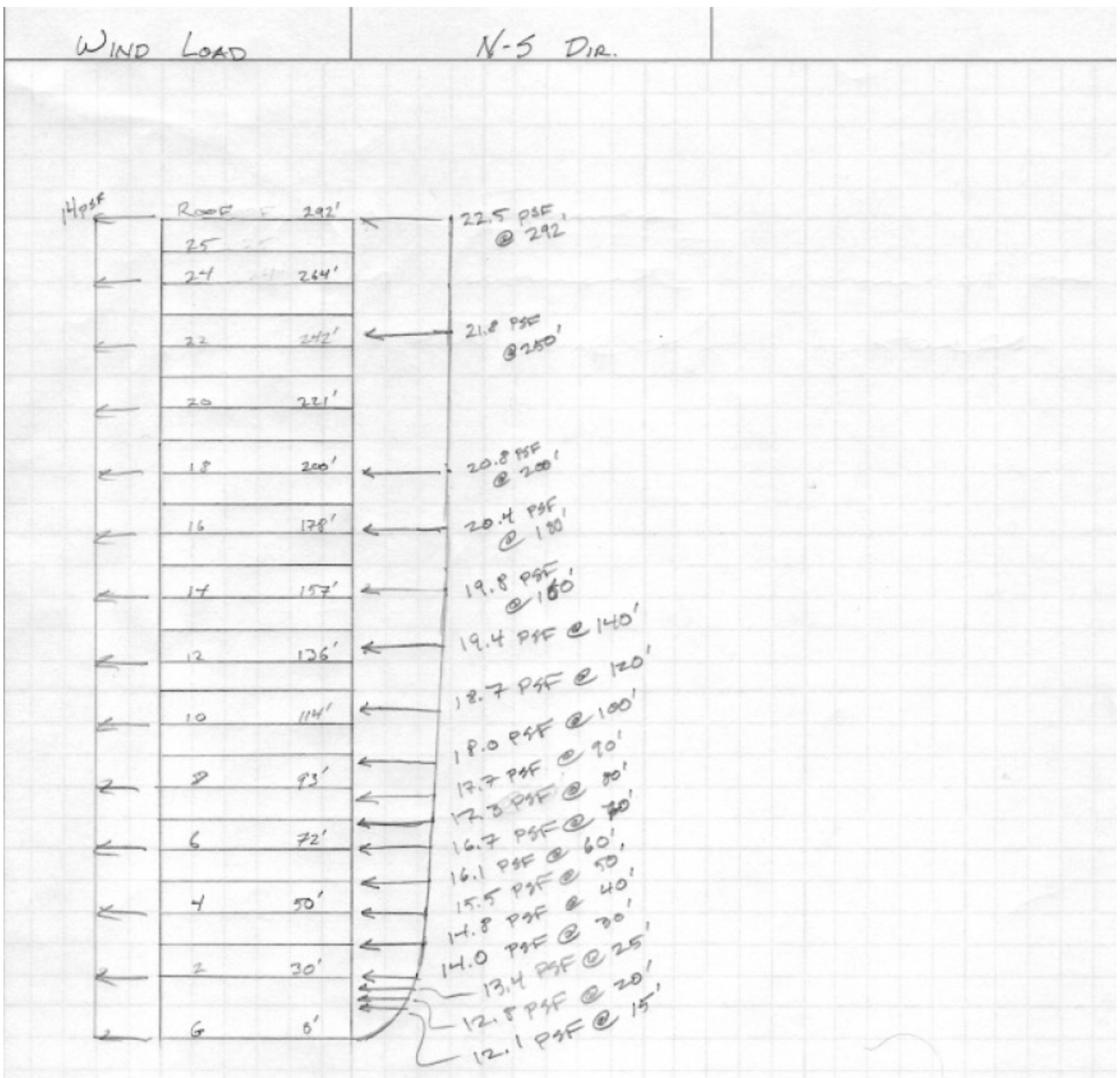
Reference	Pressure Coefficients		
	Internal		
	Enc. Type	Enclosed	
Figure 6.5	Internal (GC_{pi})	0.18	+/-

Pressures			Rigid	Flexible
Windward	N-S	P_z	0.684	0.704
	E-W	P_z	0.684	0.704
Leeward	N-S	P_h	-0.428	-0.440
	E-W	P_h	-0.318	-0.327

K_z and q_z		
Z(ft)	K_z	q_z
0-15	0.85	17.23
20	0.90	18.24
25	0.94	19.05
30	0.98	19.86
40	1.04	21.08
50	1.09	22.09
60	1.13	22.90
70	1.17	23.72
80	1.21	24.53
90	1.24	25.13
100	1.26	25.54
120	1.31	26.55
140	1.36	27.57
160	1.39	28.17
180	1.43	28.99
200	1.46	29.59
250	1.53	31.01
300	1.59	32.23
350	1.64	33.24
400	1.69	34.26
450	1.73	35.07
500	1.77	35.88
290	1.578	31.99

	Wind Ward		Leeward		Total	
	N-S	E-W	N-S	E-W	N-S	E-W
0-15	12.12	12.12	-14.06	-10.47	26.18	22.59
20	12.83	12.83	-14.06	-10.47	26.90	23.30
25	13.40	13.40	-14.06	-10.47	27.47	23.87
30	13.97	13.97	-14.06	-10.47	28.04	24.44
40	14.83	14.83	-14.06	-10.47	28.89	25.30
50	15.54	15.54	-14.06	-10.47	29.61	26.01
60	16.11	16.11	-14.06	-10.47	30.18	26.58
70	16.68	16.68	-14.06	-10.47	30.75	27.15
80	17.25	17.25	-14.06	-10.47	31.32	27.72
90	17.68	17.68	-14.06	-10.47	31.75	28.15
100	17.97	17.97	-14.06	-10.47	32.03	28.44
120	18.68	18.68	-14.06	-10.47	32.74	29.15
140	19.39	19.39	-14.06	-10.47	33.46	29.86
160	19.82	19.82	-14.06	-10.47	33.88	30.29
180	20.39	20.39	-14.06	-10.47	34.45	30.86
200	20.82	20.82	-14.06	-10.47	34.88	31.29
250	21.82	21.82	-14.06	-10.47	35.88	32.29
300	22.67	22.67	-14.06	-10.47	36.74	33.14
350	23.39	23.39	-14.06	-10.47	37.45	33.86
400	24.10	24.10	-14.06	-10.47	38.16	34.57
450	24.67	24.67	-14.06	-10.47	38.73	35.14
500	25.24	25.24	-14.06	-10.47	39.30	35.71
290	22.50	22.50	-14.06	-10.47	36.57	32.97

			Pressure per Floor											
			Wind Ward		Leeward		Total		Story Force		Cumulative Shear		Overturning Moment	
Floor	Height	Trib Height	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W
	(ft)	(ft)	(PSF)	(PSF)	(PSF)	(PSF)	(PSF)	(PSF)	(kip)	(kip)	(kip)	(kip)	(ft-kip)	(ft-kip)
1 - Ground	0.00	0	0	0	0	0	0	0	0	0	735.23	1075.62	117243	172283
2	29.75	20.00	14.40	14.40	-14.06	-10.47	28.47	24.87	45.16	64.67	735.23	1075.62	1344	1924
3	40.00	10.13	15.19	15.19	-14.06	-10.47	29.25	25.66	23.49	33.77	690.06	1010.95	940	1351
4	50.00	10.00	15.83	15.83	-14.06	-10.47	29.89	26.30	23.71	34.19	666.57	977.18	1186	1709
5	60.00	10.79	16.40	16.40	-14.06	-10.47	30.46	26.87	26.08	37.69	642.86	943.00	1565	2262
6	71.58	11.13	16.97	16.97	-14.06	-10.47	31.03	27.44	27.39	39.68	616.78	905.30	1960	2841
7	82.25	10.67	17.47	17.47	-14.06	-10.47	31.53	27.94	26.68	38.74	589.39	865.62	2195	3186
8	92.92	10.67	17.82	17.82	-14.06	-10.47	31.89	28.29	26.98	39.24	562.71	826.88	2507	3646
9	103.58	10.67	18.44	18.44	-14.06	-10.47	32.51	28.91	27.51	40.09	535.73	787.64	2849	4153
10	114.25	10.67	18.68	18.68	-14.06	-10.47	32.74	29.15	27.71	40.42	508.22	747.55	3166	4618
11	124.92	10.67	19.39	19.39	-14.06	-10.47	33.46	29.86	28.31	41.41	480.51	707.13	3536	5173
12	135.58	10.17	19.39	19.39	-14.06	-10.47	33.46	29.86	26.98	39.47	452.20	665.72	3658	5351
13	145.25	10.67	19.82	19.82	-14.06	-10.47	33.88	30.29	28.67	42.00	425.22	626.25	4165	6101
14	156.92	11.17	19.82	19.82	-14.06	-10.47	33.88	30.29	30.02	43.97	396.55	584.24	4710	6900
15	167.58	10.67	20.39	20.39	-14.06	-10.47	34.45	30.86	29.16	42.79	366.53	540.27	4886	7172
16	178.25	10.67	20.53	20.53	-14.06	-10.47	34.60	31.00	29.28	42.99	337.38	497.47	5218	7663
17	188.92	10.67	20.82	20.82	-14.06	-10.47	34.88	31.29	29.52	43.39	308.10	454.48	5576	8197
18	199.58	10.67	21.32	21.32	-14.06	-10.47	35.38	31.79	29.94	44.08	278.58	411.09	5975	8798
19	210.25	10.67	21.82	21.82	-14.06	-10.47	35.88	32.29	30.36	44.77	248.64	367.01	6384	9413
20	220.92	10.67	21.82	21.82	-14.06	-10.47	35.88	32.29	30.36	44.77	218.28	322.24	6707	9891
21	231.58	10.67	21.82	21.82	-14.06	-10.47	35.88	32.29	30.36	44.77	187.92	277.47	7031	10368
22	242.25	10.67	21.82	21.82	-14.06	-10.47	35.88	32.29	30.36	44.77	157.56	232.70	7355	10846
23	252.92	10.67	22.27	22.27	-14.06	-10.47	36.34	32.74	30.75	45.40	127.20	187.93	7777	11484
24	263.58	10.83	22.50	22.50	-14.06	-10.47	36.57	32.97	31.42	46.44	96.45	142.52	8283	12240
25	274.58	13.96	22.50	22.50	-14.06	-10.47	36.57	32.97	40.49	59.83	65.02	96.09	11118	16428
Roof	291.5	8.46	22.50	22.50	-14.06	-10.47	36.57	32.97	24.54	36.26	24.54	36.26	7152	10568



A3: Seismic Loads

Location		Chicago, IL
Number of floors:	N	25
Building Height:	h_n	291.5
Inter-story height	h_s	10
Occupancy Category		II
Seismic Category	I	I
Importance Factor		1
Site Class		B
	S_s	0.17
	S_1	0.07
	F_a	1.0
	F_v	1.0
	S_{MS}	0.17
	S_{M1}	0.07
	S_{DS}	0.113
	S_{D1}	0.047

Design Category **A**
Can use simplified analysis procedure.

Design Parameters:

Response Modifier	R		5
	Ω_0		2
	C_d		4.5
	ρ		1.0
Natural Period	T		1.405
	T_0		0.082
	T_s		0.412
Seismic Base Shear	V	$=1.2S_{DS}*W/R$	520.45

Weights:

Dead Loads:

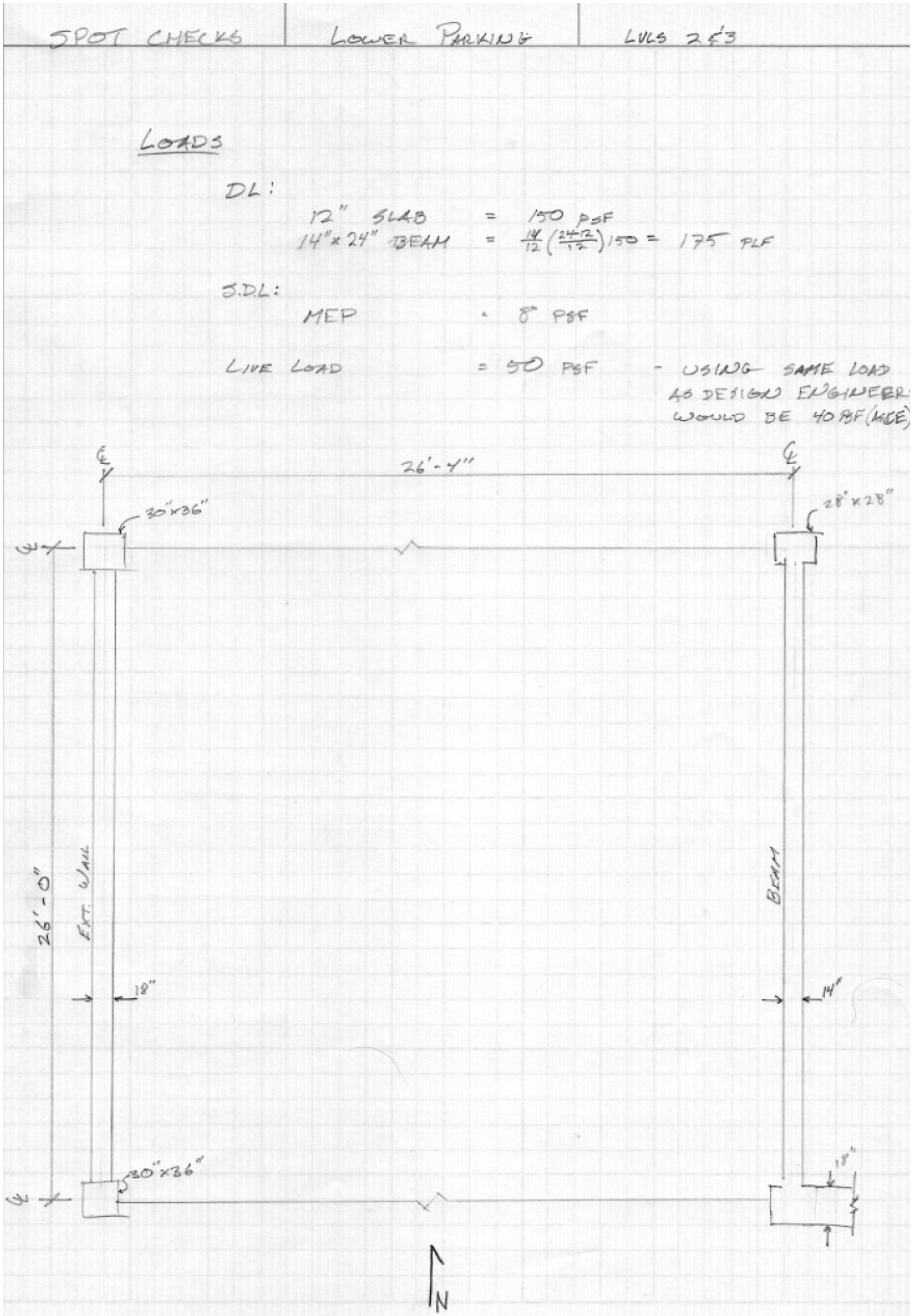
	Slab/Deck	Beams/joists	Superimposed	Total
Roof	50	10	20	80
7-25	40	15	15	70
4-6	80	10	10	100
2-3	150	N/A	10	160
Ground	150	N/A	25	175

Live Loads:

Grond Floor	100	
Parking	50	
Residential Units	40	Dwelling
	15	Partition
Corridors	40	
Balconies	40	
Roof	25 + Drift	

Floor	Height	Area	Weight	Shear Distribution	Overturning Moment	Cumulative Shear	Cumulativ e Moment
	(ft)	(ft^2)	(kip)	(kip)	(ft-kip)	(kip)	(ft-kip)
Roof	291.5	3100	248.0	6.75	1966.3	6.75	1966.34
25	274.58	5442	462.6	12.58	3454.8	19.33	5421.12
24	263.58	7622	647.9	17.62	4644.9	36.95	10066.01
23	252.92	5534	470.4	12.79	3236.0	49.74	13301.98
22	242.25	5534	470.4	12.79	3099.5	62.54	16401.47
21	231.58	6658	565.9	15.39	3564.8	77.93	19966.30
20	220.92	6658	565.9	15.39	3400.6	93.33	23366.94
19	210.25	6658	565.9	15.39	3236.4	108.72	26603.38
18	199.58	9230	784.6	21.34	4259.1	130.06	30862.44
17	188.92	9230	784.6	21.34	4031.4	151.40	34893.87
16	178.25	9230	784.6	21.34	3803.8	172.74	38697.69
15	167.58	9230	784.6	21.34	3576.2	194.08	42273.87
14	156.92	9230	784.6	21.34	3348.6	215.42	45622.44
13	145.25	9230	784.6	21.34	3099.6	236.76	48722.04
12	135.58	9230	784.6	21.34	2893.3	258.10	51615.35
11	124.92	9230	784.6	21.34	2665.7	279.44	54281.04
10	114.25	9230	784.6	21.34	2438.1	300.78	56719.11
9	103.58	9230	784.6	21.34	2210.4	322.12	58929.56
8	92.92	9230	784.6	21.34	1982.8	343.46	60912.38
7	82.25	9230	784.6	21.34	1755.2	364.80	62667.57
6	71.58	9230	923.0	25.11	1797.1	389.90	64464.71
5	60.00	9230	923.0	25.11	1506.3	415.01	65971.05
4	50.00	9230	923.0	25.11	1255.3	440.11	67226.33
3	40.00	9230	1476.8	40.17	1606.8	480.28	68833.09
2	29.75	9230	1476.8	40.17	1195.0	520.45	70028.11
1 - Ground	0.00	9230	1615.3	0.00	0.0	520.45	70028.11
		213346	19134.2	520.45	70028.1		

A4: Spot Checks



LOWER PARKING

CHECK SLAB

- TWO-WAY 12" SLAB
- NEGLECT BEAM & WALL AS BEAM!
- LOADS

$LL = 50$ PSF

$DL = 150 + 8 = 158$ PSF
(SLAB) (MEP)

$\omega_u = 1.2(158) + 1.6(50) = 270$ PSF

N-S. DIR.

Col Strip → 13'-2"
Mid Strip → 13'-2"

$l_1 = 26'$ $l_2 = 26'4"$ $l_n = 26 - 14" - 9" = 24'1"$

$M_o = \frac{\omega_u l_2 l_n^2}{8} = 575.5$ ft·k $\alpha = 0$

			ft·k	ft·k/ft	MIN A_s
Column Strip	Pos. Mom.	$0.60(0.35)(M_o)$	108.3	8.23	OK
	Neg Mom.	$0.75(0.65)(M_o)$	281.3	19.1	No Good
Mid Strip	Pos Mom.	$0.40(0.35)(M_o)$	72.2	5.48	OK
	Neg Mom.	$0.25(0.65)(M_o)$	45.1	3.43	OK

$d = -1.5" - 0.625 - 0.312" + 12" = 9.56"$

$A_{s\,min} = 0.0018(12)(12"/ft) = 0.259$ in^2

Try #5 @ 12" $A_s = 0.31$ in^2

$a = \frac{A_s f_y}{0.85 f'_c b} = 0.456"$ $c = 0.536" < 0.375d = 3.59"$

$\phi = 0.9$

$\phi M_n = 0.9 A_s f_y (d - \frac{a}{2}) = 156$ in·k $= 13.0$ ft·k

N-S DIR (CONT.)

Try #5 @ 6" $A_s = 0.62''^2$

$a = 0.912''$ $c = 1.07 < 0.375d = 3.59$

$\phi = 0.9$

$\phi M_n = 304.8$ in·k → 25.4 ft·k

OK for CS (−) MOMENT = 19.1 ft·k

C.S.

26 #5 TOP BARS
13 #5 BOT. BARS

M.S.

13 #5 T BARS
13 #5 B BARS

DESIGN ENGINEER USED:

C.S.:

28 #4 B BARS
22 #6 T BARS

M.S.:

18 #4 B BARS
18 #4 T BARS

SPOT CHECKS	UPPER PARKING	LVLS 4-6

LOADS

D.L.:

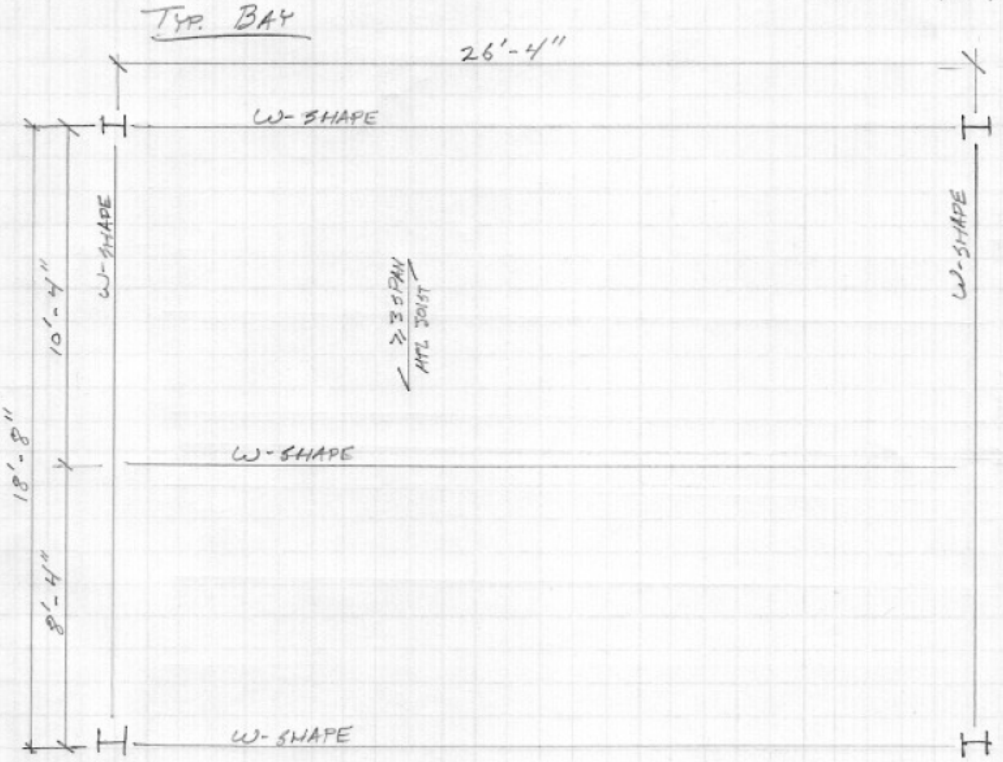
6" SLAB = 75 PSF
MTL DECK = 3 PSF

S.D.L.:

MEP = 5 PSF
WATERPROOFING = 3 PSF

LIVE LOAD: = 50 PSF — USING SAME LOADS AS DESIGN ENGINEER; WOULD BE 40 PSF (ASCE-7)

TYP. BAY



UPPER PARKING

CHECK SLAB/DECK

- ASSUME ≥ 3 SPANS ; NWC ; 7.5" SLAB DEPTH
- USE LARGEST SPAN 10'-4" → 10'-6"
- USING VULCRAFT 3" VLI COMPOSITE DECK
- SUPERIMPOSED LOADS

LL = 50 PSF

SDL = 8 PSF

58 PSF

TRY 3VLI20 w/ 5" SLAB DEPTH
w/ 6×6×W1.4×W1.4 WWF

MAX UNSHORED SPAN = 12'-4" > 10'-4" OK
ALLOWABLE LOAD = 101 PSF > 58 PSF OK

IF 7½" SLAB DEPTH REQUIRED USE:

3VLI18 w/ 6×6-W2.1×W2.1

UNSHORED SPAN = 12'-0"
ALLOW. LOAD = 227 PSF

DESIGN ENGINEER USED 18 GAGE DECK,
7½" SLAB & 6×6×W2.1×W2.1 WWF

UPPER PARKING

CHECK BEAMS

- USE 7.5" SLAB ON 3" DECK
- COMPOSITE ACTION w/ 3/4" ϕ 4" LONG STUDS
- SPAN = 26'-4"; TRIB WIDTH = $\frac{10'4"}{2} + \frac{8'4"}{2} = 9'-4"$

LL = 50 PSF → 467 PLF

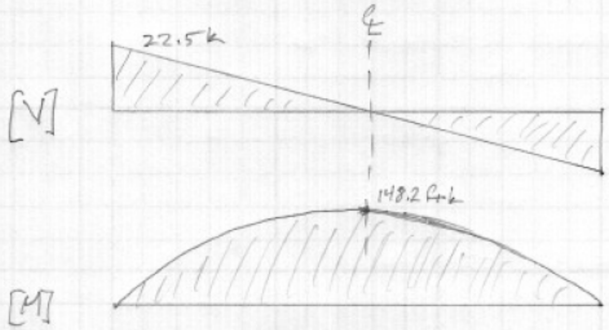
DL =

SLAB/DECK = 78 PSF → 728 PLF
S.DL = 8 PSF → 75 PLF

w_u =

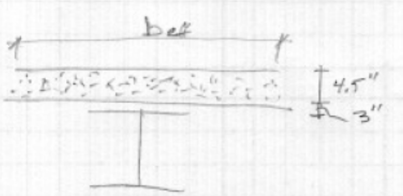
803 PLF

$w_u = 1.2(803) + 1.6(467) = 1.71$ KLF



$\frac{w_u l}{2} = 22.5^k$

$\frac{w_u l^2}{8} = 148.2$ ft·k



$b_{eff} = \begin{cases} l/4 = 6'-7" \leftarrow \text{USE} \\ \text{Trib Width} = 9'-4" \end{cases}$

Non-composite Beam
- W14×26 (TABLE 5.5)

$A_s = 7.69$ in²
$d = 13.9$ in
$f_y = 50$ ksi
$f'_c = 4$ ksi

$C = 0.85 f'_c\, b\, d = 1208^k$
$T = A_s f_y = 384.5^k$
$a = \frac{A_s f_y}{0.85 f'_c b} = 1.43"$

$\phi M_n = 0.85\left[384.5\left(\frac{13.9}{2}\right) + 384.5\left(7.5 - \frac{1.43}{2}\right)\right] = 4488$ in·k → 374 ft·k

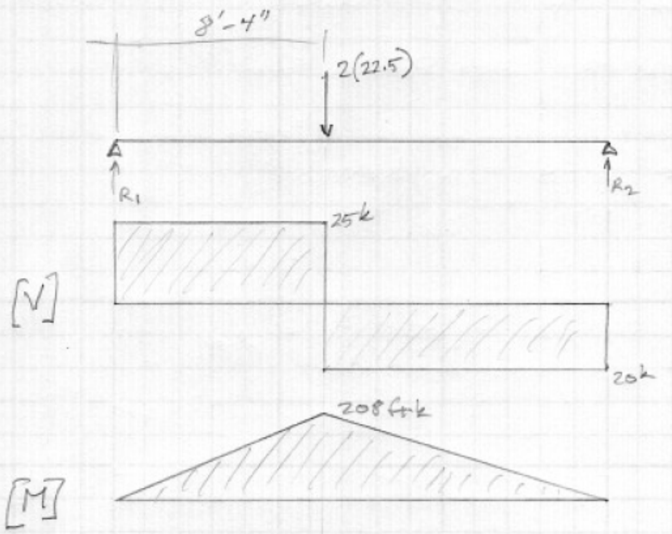
$Q_n = 26.1^k$ ∴ 30 STUDS TOTAL (15 EACH BEAM HALF)

DESIGN ENGINEER USED W18×35 w/ 12 STUDS

UPPER PARKING

CHECK GIRDERS

- SPAN = 18'-8" TRIB WIDTH = 26'-4"
- USE BEAM RXNS AS GIRDER LOADS



$R_2 = \frac{8.33(2)(22.5)}{18.66} = 20^k$

$R_1 = 25^k$

$M = 25(8'\text{-}4") = 208 \text{ ft·k}$

USE W16×36 (TABLE 5.5)

DESIGN ENGINEER USES W16×26
WITH 8 SHEAR STUDS

SPOT CHECKS | RESIDENTIAL FL. | LVLS 7-25

LOADS

D.L.:

2.5" SLAB = 31.25 PSF
MTL DECK = 2 PSF
} USE 35 PSF
JOIST = 8 PLF

SDL:

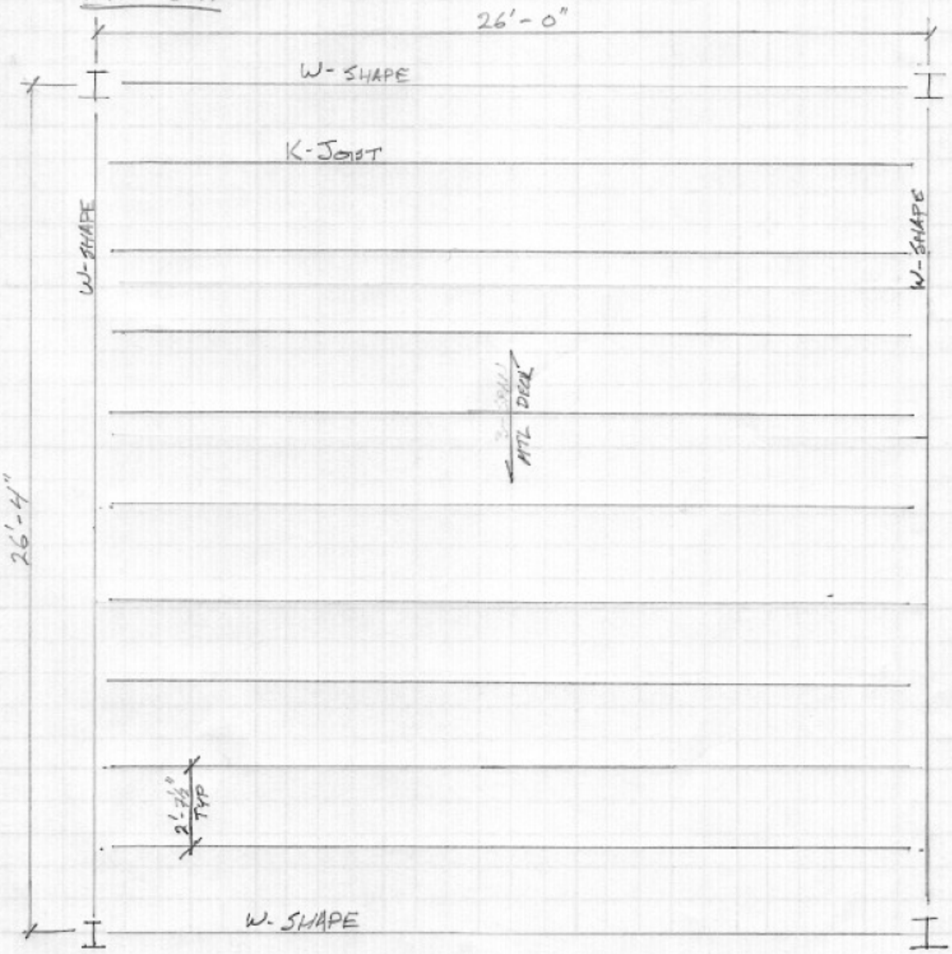
MEP = 10 PSF
FLOORING = 3 PSF

LL

IN UNITS = 40 PSF
CORRIDORS = 40 PSF
PARTITIONS = 15 PSF

- USING SAME LOADS AS DESIGN ENGINEER; WOULD OTHERWISE USE 100 PSF IN CORRIDOR AS PER ASCE-7

TYP. BAY



RESIDENTIAL FLOOR

CHECK SLAB/DECK

- USING VULCRAFT NON-COMPOSITE 0.6C METAL DECK

- ASSUME ≥3 SPANS FOR DECK

- CLEAR SPAN = 2'-7.5" → 2'-9"

- SUPERIMPOSED LOAD:

  LL: 40+15 = 55
  SDL: 10+3 = 13
  68 PSF

0.6C28 w/ 2" SLAB
6×6 - W2.1×W2.1 $A_s = 0.042^2$

TOTAL LOAD = 151 > 68 PSF

DESIGN ENGINEER USES 0.6C26 w/ 2½" SLAB & 6×6 - W1.4×W1.4 WWF

CHECK JOISTS

- USE 0.6C26 w/ 2½" SLAB/DECK 35 PSF

- CLEAR SPAN = 26' ; TRIB WIDTH = 2'-9"

- SUPERIMPOSED LOAD

LL:	45 PSF	=	124 PLF
SDL:	13 PSF	=	36 PLF
SLAB/DECK:	35 PSF	=	96 PLF
			256 PLF

- DEFLECTION LIMIT : L/240

IF NO LIMIT ON JOIST DEPTH

18K3 TOTAL LOAD = 272 > 256 PLF
LIVE LOAD = 190 > 124 PLF

IF LIMIT OF 14" FOR JOIST

14K6 TL = 308 > 256 PLF
LL = 156 > 124 PLF

JOIST (CONT.)

DESIGN ENGINEER USED 14K6 JOIST.

BEAM CHECK

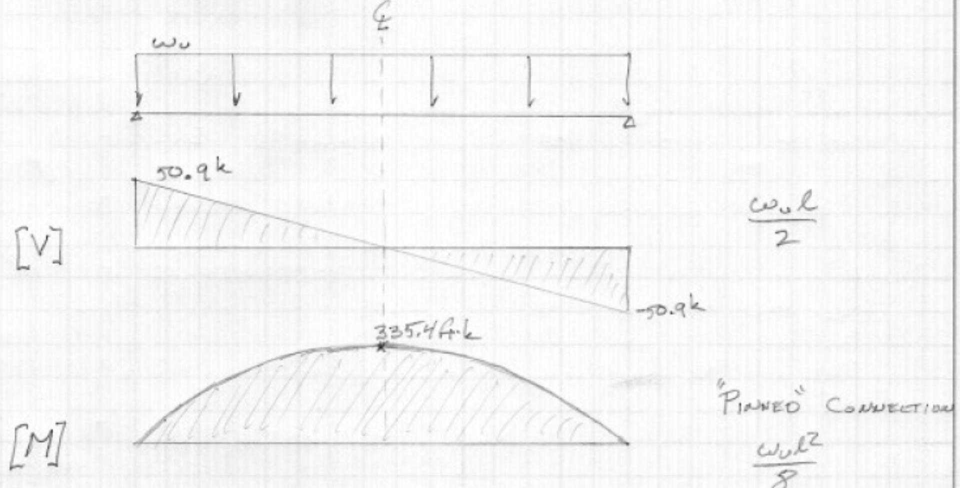
- 9 JOISTS @ EQUAL SPACING
  - CAN ASSUME DISTRIBUTED LOAD
- CLEAR SPAN = 26'-4" ; TRIB WIDTH = 26'

LL = 55 PSF → 1.43 klf

DL =

FLOOR	35 PSF →	0.91 klf
JOIST	9(8)(26)/26.33 =	71 plf
S.DL	13 PSF →	0.338 klf
		1.32 klf

$w_u = 1.2(1.32) + 1.6(1.43) = 3.87$ klf



L_b = UNBRACED LENGTH = 2'-7½"

M_u = 335.4 k·ft

Try W21×44 ϕM_n = 359 > 335 ft·k OK

BEAM CONT.

$L/360 = 0.878''$ $E = 29000$ $I_x = 847\ in^4$

$\Delta_{max} = \frac{5 w l^4}{384 E I} = 0.000093''$ OK

IF LIMIT ON BEAM DEPTH : 14" (SAME AS JOIST)

Try W12×72 $\phi M_n = 405 > 335$ ft·k   OK

$\Delta_{max} = 0.00013''$ OK

DESIGN ENGINEER USED W12×87

Column Check

Roof Live Load:

DL	80
L_o	50
N-S Span	26
E-W Span	26.33
A_T	684.58
R_1	1
R_2	1

Live Load Reduction

DL	70
L_o	40
N-S Span	26
E-W Span	26.33
A_T	684.58
K_{LL}	4

Column Below:	L_o (PSF)	Dead (PSF)	A_T (ft2)	Reduction	Live (PSF)	Live Load (kip)	Dead Load (kip)	Total Load (kip)
Roof	50	80	684.58	1.000	50	34.2	54.8	120.5
Floor 25	40	70	684.58	0.537	21.47	14.7	102.7	146.7
24	40	70	1369.16	0.453	18.11	24.8	150.6	220.4
23	40	70	2053.74	0.415	16.62	34.1	198.5	292.8
22	40	70	2738.32	0.393	16.00	43.8	246.4	365.8
21	40	70	3422.9	0.378	16.00	54.8	294.4	440.9
20	40	70	4107.48	0.367	16.00	65.7	342.3	515.9
19	40	70	4792.06	0.358	16.00	76.7	390.2	590.9
18	40	70	5476.64	0.351	16.00	87.6	438.1	666.0
17	40	70	6161.22	0.346	16.00	98.6	486.1	741.0
16	40	70	6845.8	0.341	16.00	109.5	534.0	816.0
15	40	70	7530.38	0.336	16.00	120.5	581.9	891.0
14	40	70	8214.96	0.333	16.00	131.4	629.8	966.1
13	40	70	8899.54	0.330	16.00	142.4	677.7	1041.1
12	40	70	9584.12	0.327	16.00	153.3	725.7	1116.1
11	40	70	10268.7	0.324	16.00	164.3	773.6	1191.2
10	40	70	10953.28	0.322	16.00	175.3	821.5	1266.2
9	40	70	11637.86	0.320	16.00	186.2	869.4	1341.2
8	40	70	12322.44	0.318	16.00	197.2	917.3	1416.3
7	40	70	13007.02	0.316	16.00	208.1	965.3	1491.3
6	50	100	13691.6	0.314	20.00	273.8	1033.7	1678.6
5	50	100	14376.18	0.313	20.00	287.5	1102.2	1782.6
4	50	100	15060.76	0.311	20.00	301.2	1170.6	1886.7
3	50	160	15745.34	0.310	20.00	314.9	1280.2	2040.0
2	50	160	16429.92	0.309	20.00	328.6	1389.7	2193.4

Table A4.1 Column Axial Loads

COLUMN CHECK

- CHECK FOR GRAVITY LOADS ONLY
- NEGLECT MOMENT FROM LL. PATTERN LOADING

COLUMN BELOW LEVEL 6:

AXIAL LOAD: $P_u = 1680^k$
UNBRACED LENGTH $L_b = 11.6'$

W14 × 159

DESIGN ENGINEER USED W14 × 193

COLUMN BELOW LEVEL 12:

AXIAL LOAD: $P_u = 1116^k$
$L_b = 11.6'$

W14 × 109

DESIGN ENGINEER USED W14 × 145

COLUMN BELOW LEVEL 18:

$P_u = 665.9^k$
$L_b = 11.6'$

W14 × 82

DESIGN ENGINEER USED W14 × 99

LATERAL LOADS

SEISMIC:

BASE SHEAR: 520.5^k

OVERTURNING MOMENT: 70030 ft·k

WIND:

N-S:

BASE SHEAR: 735.2^k

OVERTURNING MOMENT: 117,240 ft·k

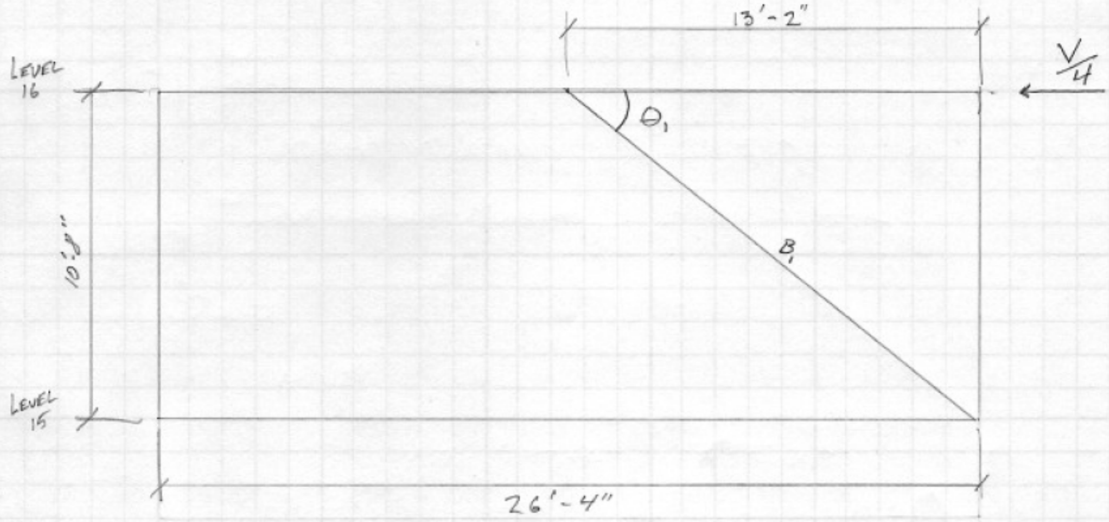
E-W:

BASE SHEAR: 1075.6^k

OVERTURNING MOMENT: 172,280 ft·k

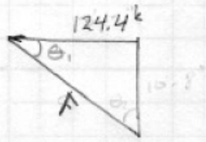
WIND CONTROLS!

LAT. BRACE



BRACE IS 1 OF 4 IDENTICAL ONES BETWEEN LEVELS 15 & 16 IN THE E-W DIRECTION

$V = 497.5^k \qquad \frac{V}{4} = 124.4^k$



$$\theta_1 = \tan^{-1}\left(\frac{10'\text{-}8''}{13'\text{-}2''}\right) = 39^\circ$$

$$L_{\theta_1} = \frac{13'2''}{\sin\theta_1} = 20'\text{-}11''$$

$$F = \frac{124.4}{\sin\theta_1} = 197.6^k = P_u$$

TENSION W8×15

COMPRESSION W10×49 ← USE

DESIGN ENGINEER USED W10×68