Executive Summary

This report contains a detailed description and preliminary analysis of the structure of Parkridge Center – Phase VI. The scope of this report is limited to basic gravity analysis of columns, beams, and girders and a simplified lateral check of the lateral resisting elements within the building. It was found in this report that the assumed loading and distributions of loads throughout the building were within allowable tolerances; with checked members ending up at or within one member size of the original design by the projects structural engineer.

The only area within this report that falls outside these tolerances is in the calculation of the building seismic base shear. The calculated result is approximately 42% larger than that used by the original designers. The primary reasoning for this difference is in my own inexperience with the seismic design criteria. Although this loading fell outside the tolerances it is a conservative error which would result in larger member sizes than those in the original design.

Introduction

A seven story steel building with a composite floor system and sloping columns on its south face, Parkridge Center – Phase VI is to be the “crown jewel” of the office complex it is located in. The building utilizes an open plan on the 6 stories above the lobby. Its mechanical system, a variable air volume system, is set up to service each half of the building separately through two individual duct loops.

The main structural challenges with this building are its sloping columns and the garden located on the west side of the roof. The sloping columns will require special attention with regards to the moment created at the base of the building and transferred to the foundation. The challenges with garden will be primarily focused on loading from the individual elements for example the weight of a mature tree can vary depending on the conditions it is grown under.

Structural System Description

Foundation and Slab on Grade

The foundation is comprised of shallow footings ranging in size from 5’-0” x 5’-0” x 12” to 20’-0” x 20’-0” x 42”. The specified concrete strength of the footings is 4000 PSI. The allowable bearing pressure on the soil was found to be 3000 PSF by the geotechnical engineer. The slab on grade shall be 4” thick and reinforced with 6x6-10/10 Welded Wire Mesh (WWM) on a 6 mil vapor barrier. Also beneath the slab on grade there shall be 4” of drainage fill. There are no floors below grade in this building.

Typical Floor

Each floor contains the same three by ten bay core. The south most exterior bay on each floor varies based on the slope of the columns on the south face creating larger floor area on higher level floors. Floors 2 thru 5 contain extra floor area on the north side of the building above the arcade. The North-South (N-S) spans of the core three bays are 37’-2” for the exterior bays and 35’-0” for the interior bay. The East-West (E-W) spans of the core bays are 25’-8” for the first bay and then 25’-0” for the remaining bays. Intermediate beams are spaced at the third points of each bay and span in the N-S direction. Typical beam sizes for the core bays are W21’s for the interior girders, W18’s for the exterior girders, and W16’s for the intermediate beams. Each beam is cambered to 1-1/4" this was done most likely to minimize ponding of concrete at center span.

Floor System

Each floor above grade uses a composite deck made up of 3 ¼" Lightweight concrete on 2"-20 gage steel deck. The total floor thickness is 5 1/4". The slab itself is to be reinforced with 6x6-10/10 WWM.

Columns

Each column extends 3 floors and is spliced above the slab. The columns along the south face of the building, column line A.1, are sloped outward from the ground to the roof. Typical sizes for the sloped columns begin at a W12x65 at the roof to the 7th floor, W12x96 from the 7th floor to the 4th floor, and W12x152 from the 4th floor to the foundation.

Lateral System

Five braced frames make up the lateral system for the building. There are two frames in the N-S direction and three frames in the E-W direction. The diagonal members of the frames are HSS 10x10x1/2 for the N-S frames and HSS 8x8x1/2 for the E-W frames. Frames two and three are connected by two intermediate frames at the roof. The diagonal members of the two intermediate frames are HSS 8x8x1/4. Frame three is an eccentric braced frame while all the other frames are concentrically braced.

Codes

The codes used in the design of Parkridge Center – Phase VI are as follows:

- International Building Code 2000 (IBC 2000)
    - 1607.0 – Live Loads
    - 1609.0 – Wind Loads
    - 1614.0-1620.0 – Seismic Loading
- American Concrete Institute (ACI)
    - 301 – Structural Concrete for Buildings
    - 315 – Manual of Standard Practice for Detailing Reinforced Concrete Structures
    - 318 – ACI Building Code Requirements for Reinforced Concrete

For this review I have chosen to check the design using IBC 2003, ASCE7-05, and the latest version of ACI were applicable. Using the most recent codes may yield different results than obtained by the original designers; however the differences will be limited to changes in the applicable sections of each code.

Gravity Loads

Live Loads – IBC Table 1607.1	
Roof Garden	100 PSF
Roof	18 PSF
Offices	50 PSF
Corridors	80 PSF
Stair and Exits	100 PSF
Lobbies and First Floor Corridors	100 PSF

To be consistent with the original design a value of 100 PSF will be used as the live load on a typical floor.

Snow Load Chapter 7 ASCE7-05	
Pg	30 PSF
C_e	0.9
C_t	1.0
I	1.0
$P_{f,\ min}$	20 PSF
$P_{f,\ Calculated}$	18.9 PSF
P_f	20 PSF

The roof live load will be taken to be equal to the calculated snow load of 20 PSF.

Dead Loads		
Typical Floor		
Composite Floor System	41 PSF	Estimated Using United Steel Deck Catalog
Misc. (Self wt., finishes, etc.)	10 PSF	Per discussion with Structural Engineer
Ponding of Concrete	10 PSF	Per discussion with Structural Engineer
Roof		
Deck	2 PSF	Estimated Using United Steel Deck Catalog
Insulation	3 PSF	
Roofing	20 PSF	
Curtain Wall		
Glass Curtain Wall	.215 KLF	Per Discussion with Structural Engineer
Pre-cast Assembly	.55 KLF	Per Discussion with Structural Engineer
Roof Garden		
	160 PSF	Per Discussion with Structural Engineer

The dead loads listed above combined with the 100 PSF live load on the floor system should allow for flexible tenant uses in the space provided on each floor.

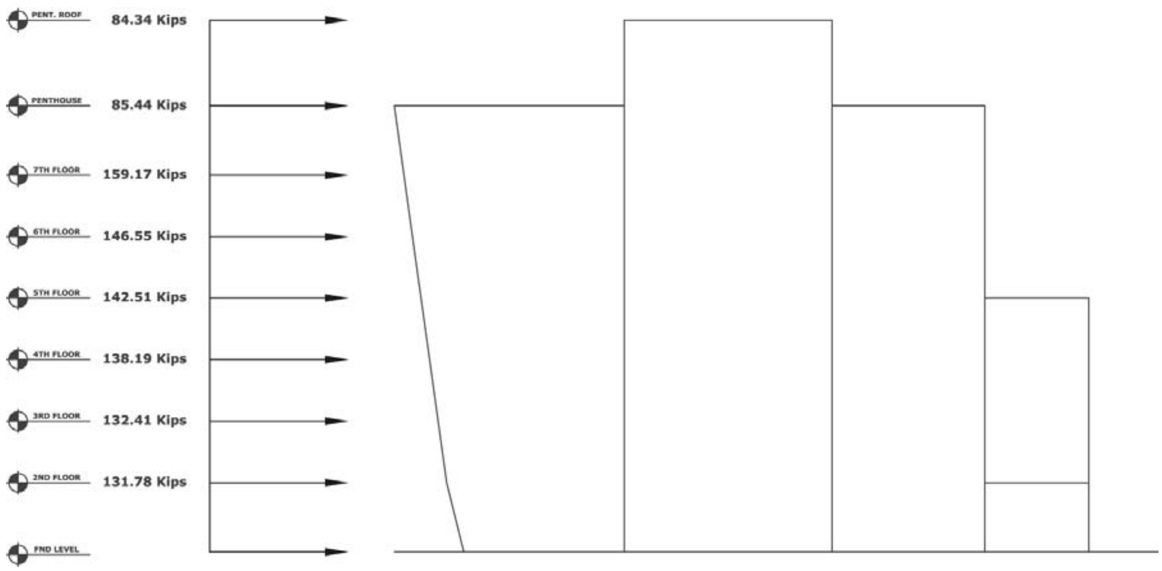
Wind

The Following wind loads were calculated using chapter 6 of ASCE7-05.
For wind pressure distributions and wind shear distributions see diagrams 1-4 in the appendix.

Total Worst Case Wind Load	
Each Direction	
z	P
0-15	21.172
20	21.810
25	22.319
30	22.829
40	23.594
50	24.231
60	24.741
70	25.251
80	25.761
90	26.143
100	26.398
115.17	26.881

Wind Shear Force at each floor (Kips)									
E-W									
Width	Floor	Height 1	Load 1	Height 2	Load 2	Height 3	Load 3	Shear	Factored
127.92	2	7.50	21.17	5.00	21.81	1.67	22.32	39.02	62.43
127.92	3	3.33	22.32	5.00	22.83	5.00	23.59	39.21	62.73
127.92	4	5.00	23.59	8.33	24.23			40.92	65.47
127.92	5	1.67	24.23	10.00	24.74	1.67	25.25	42.20	67.52
127.92	6	8.33	25.25	5.00	25.76			43.39	69.43
127.92	7	5.00	25.76	9.17	26.14			47.13	75.41
127.92	Roof	0.83	26.14	6.67	26.40			25.30	40.48
45	Penthouse Roof	9.25	26.88					11.19	17.90
N-S									
Width	Floor	Height 1	Load 1	Height 2	Load 2	Height 3	Load 3	Shear	Factored
270.00	2	7.50	21.17	5.00	21.81	1.67	22.32	82.36	131.78
270.00	3	3.33	22.32	5.00	22.83	5.00	23.59	82.76	132.41
270.00	4	5.00	23.59	8.33	24.23			86.37	138.19
270.00	5	1.67	24.23	10.00	24.74	1.67	25.25	89.07	142.51
270.00	6	8.33	25.25	5.00	25.76			91.59	146.55
270.00	7	5.00	25.76	9.17	26.14			99.48	159.17
270.00	Roof	0.83	26.14	6.67	26.40			53.40	85.44
212	Penthouse Roof	9.25	26.88					52.71	84.34

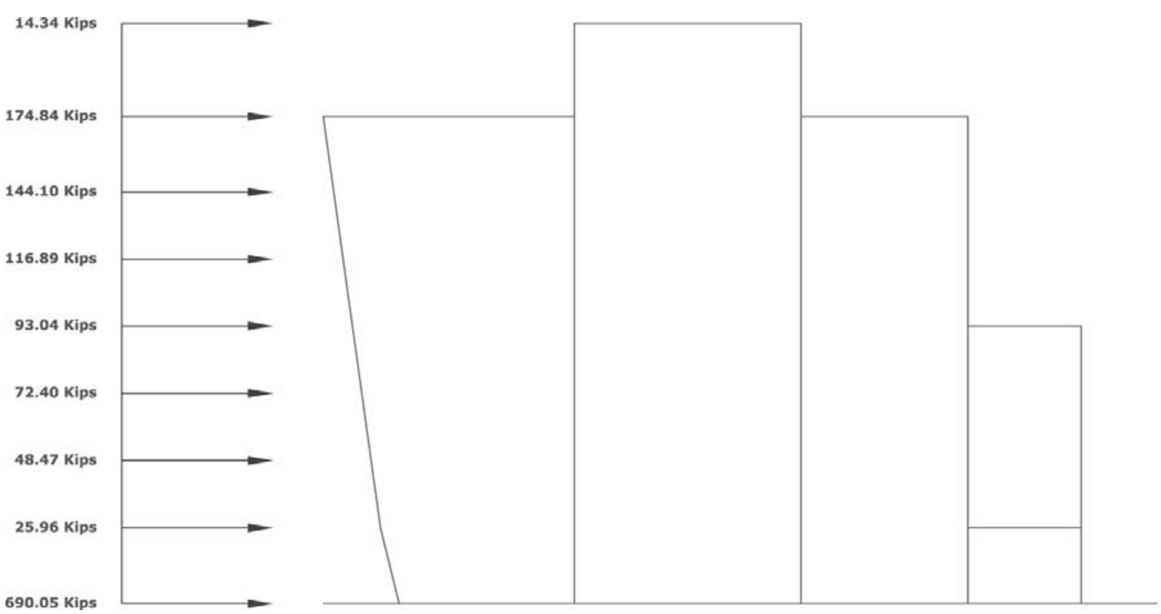
For more detailed wind calculations please see tables 1 and 2 in the appendix.



The preceding image shows the distribution of the factored wind shear on each level. The factored wind shear is equal to 1.6 * the wind shear, this was then compared to the factored seismic shear which has a factor of 1 in the load cases discussed in this report.

Seismic

The following seismic loads were calculated using Chapter 12 of ASCE7-05. The calculated seismic base shear varies from that used by the original designers by approximately +42%. The seismic base shear value in this report is overly conservative. My own inexperience with the seismic code and interpretation of the loading has contributed to this higher base shear value.



Seismic Force Distribution

Floor	w_x	h_x	k	$w_x h_x^k$	$\Sigma\, w_i h_i^k$	C_{vx}
Base	--	--	--	--	--	--
2	2346.108	15.00	1.00	35191.62	935330.09	0.038
3	2318.926	28.33	1.00	65702.91	935330.09	0.070
4	2355.326	41.67	1.00	98138.6	935330.09	0.105
5	2292.914	55.00	1.00	126110.3	935330.09	0.135
6	2318.581	68.33	1.00	158436.3	935330.09	0.169
7	2391.766	81.67	1.00	195327.5	935330.09	0.209
Roof	2451.599	96.67	1.00	236987.9	935330.09	0.253
Penthouse Roof	168.75	115.17	1.00	19434.94	935330.09	0.021
						1.000

Floor	F_x
Base	690.05
2	25.96
3	48.47
4	72.40
5	93.04
6	116.89
7	144.10
Roof	174.84
Penthouse Roof	14.34
	690.05

The calculations of building weight and base shear can be found in Table 3 and 4 in the appendix.

Other Loading

Loading that is not specifically addressed in this report is the application of mechanical systems on the roof these loads were estimated and used in the calculation of the buildings overall weight for a seismic base shear calculation. The column, beam, and girder checks that will be discussed later in this report were done in a location that is not directly affected by the mechanical equipment loads.

Also, the sky garden loading specifically from the pre-engineered trellis and mature trees, these items will be addressed in later reports.

The loading effects from the sloping columns were also omitted from this report and will be addressed in later reports.

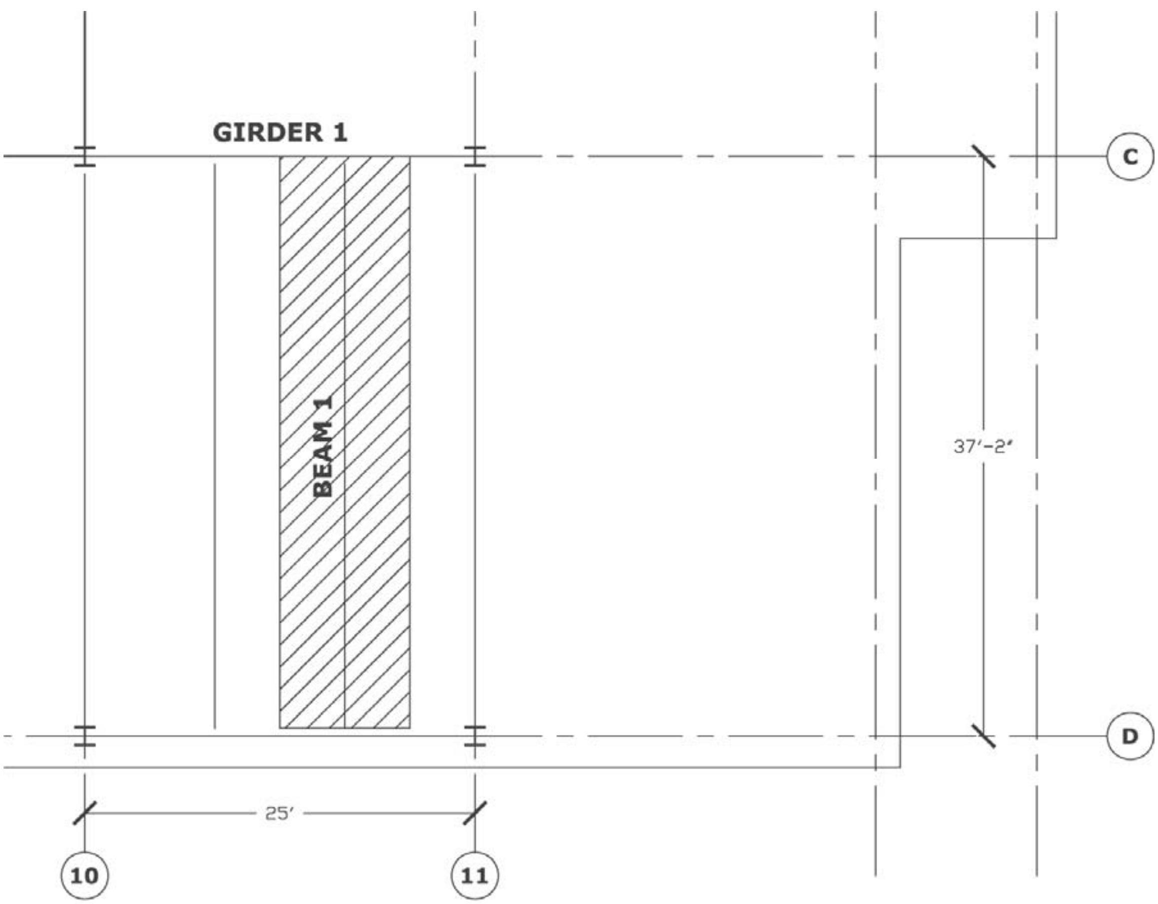
Individual Member Spot Checks

For the purposes of this report one (1) intermediate beam, girder, column, and lateral braced frame have been chosen to spot check the estimated loading against that of the original designers. All discrepancies in member size and magnitude of loads will be discussed in this section.

It was determined that the original design was done using the LRFD method. All loading on the members shall be factored and all load combinations will be checked to determine the governing combination.

Intermediate Beam "Beam 1"

The method used to check the loading on the intermediate beam was tributary area. The end conditions of the beam were assumed to be pin-pin as there is no moment connection indicated at this location on the drawings. The composite beam tables (Table 3-19, pg 3-156 to 3-189) and shear stud capacity table (Table 3-21, pg. 3-207) in the 13th edition of the AISC Manual of Steel construction were used to obtain the member sizes and amount of shear studs required. The following image will illustrate the location and tributary area of "Beam 1"



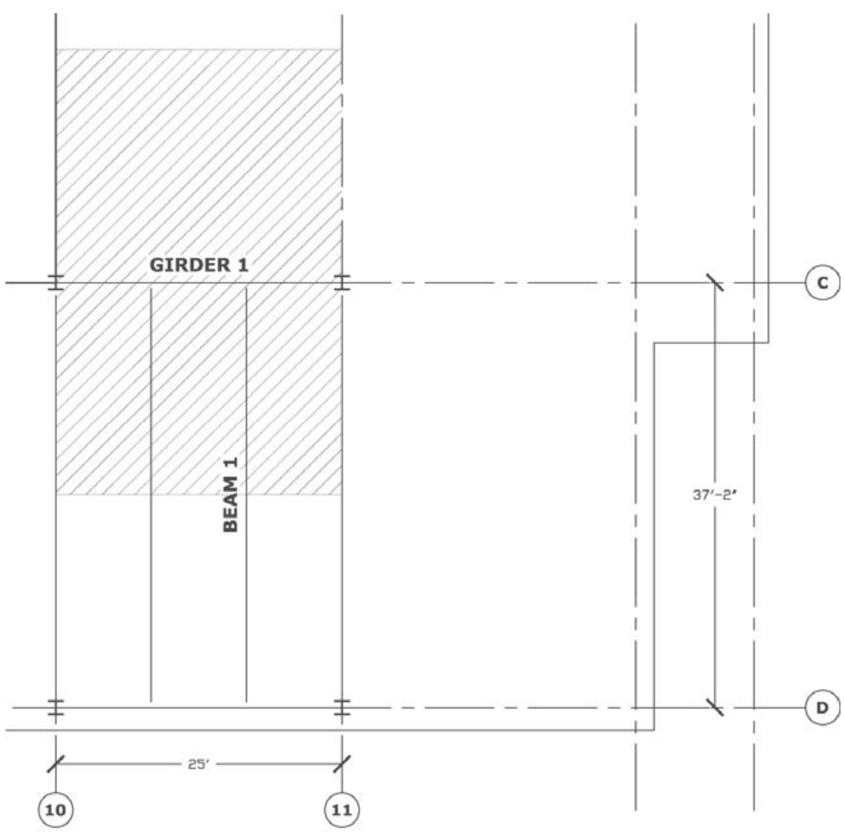
For this case I determined that the live load was reducible and ended up with a design live load of 85.27 PSF which is within the allowable limits of 0.5 * the original live load.

To simplify the process an excel spreadsheet was made to calculate the maximum moment on the beam from its applied loading as well as maximum shear. Then the user only need estimate a value for "a" and do a preliminary hand calculation of Y_2 to come up with a starting member size and plastic neutral axis (PNA) location to check. The spreadsheet then interpolates between values of ΦM_n if necessary and calculates the required number of shear studs based on ΣQ_n. This spreadsheet can be viewed in the appendix and is labeled as table 5 and 6.

From the excel calculations a W16x31 with 16 shear studs was selected. The original member is a W16x31 with 26 shear studs and is cambered at 1-1/4". The variation in amount of shear studs could be related to the amount of assumed studs per rib as well as serviceability issues. The original beam is cambered at 1-1/4" to help prevent ponding of concrete at the mid span of the beam and also to keep the beam with in serviceability requirements. I have not addressed serviceability requirements in this report but will address them in later reports.

"Girder 1"

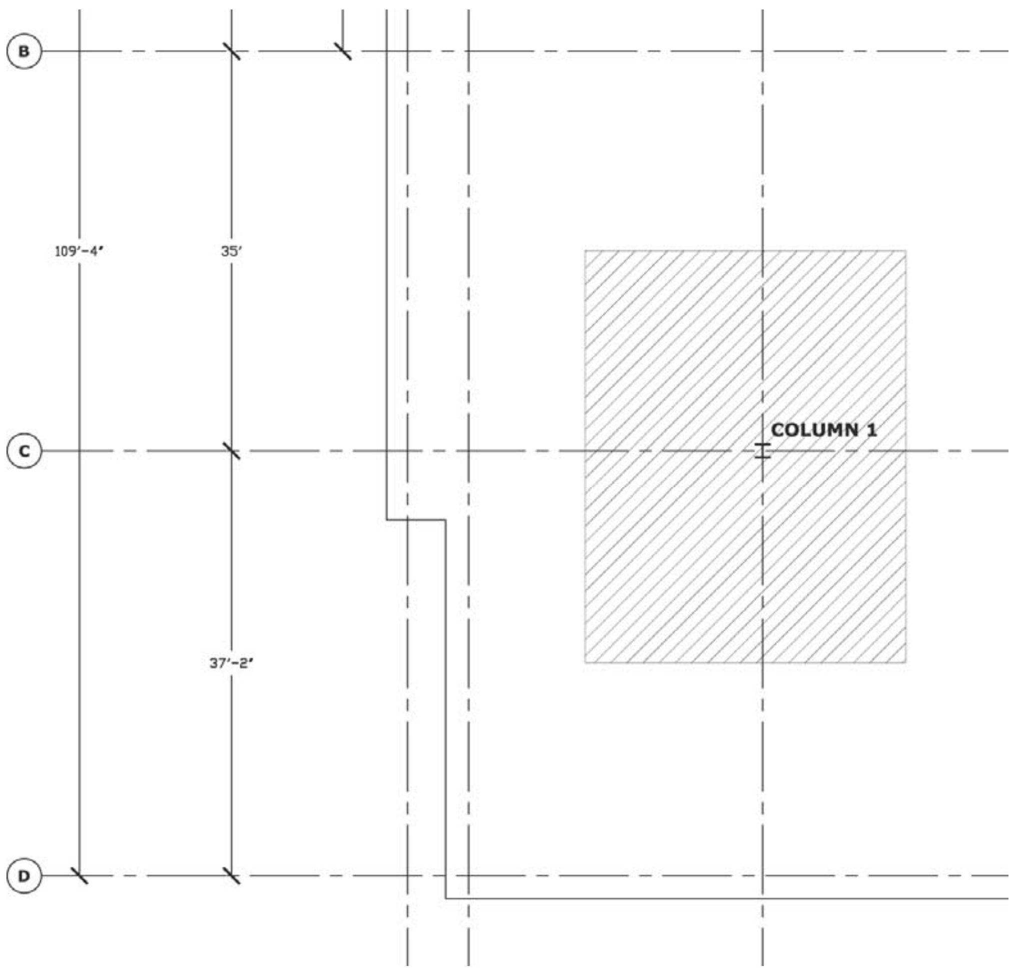
The method used to check "Beam 1" also apply to "Girder 1", also the same excel spreadsheet was used to speed up the process. The following image shows the location and tributary area of "Girder 1".



The calculated member for “Girder 1” is a W21x44 with 31 shear studs. The original design calls for a W21x44 with 20 shear studs spaced at (9,2,9) along the beams length. The variation in amount of shear studs here is most likely based on the original designers assumptions as to the amount of composite action achieved at “Girder 1”. The distribution of shear studs is based on the fact that there are two intermediate beams framing into this girder effectively placing zero (0) shear in the section between the two intermediate beams eliminating the need for shear to be transferred from the slab to the steel member. This also is a reason why my calculation contains more shear studs; a more detailed analysis would yield the same results as the original designer.

“Column 1”

“Column 1” was chosen in a location as to be minimally effected by the mechanical systems on the roof as I felt my estimated loads in some of those locations would lead to large discrepancies in sizes. It was also chosen because it is a gravity column and is much quicker to check. The following image shows the location and tributary area of “Column 1” which is located along column line 3.



The tributary area method was also used in determining the loading on “column 1”; Live load reductions were not considered in the design of the column. Second order effects were also not considered in the design of the column.

The hand calculation of "Column 1" can be found in the appendix and is labeled hand calc. - 1.

The calculated member at the base of the building for "Column 1" is a W14x145. The original design calls for a W14x132. My calculation ended in an increased member by one size. The calculated Axial load was 1612 Kips with a K=0.7 for a fixed-pin condition and L=15 ft. KL_{eff}=10.5 ft. interpolating I found that a W14x132 has a ΦP_n= 1605 Kips. My design is within a reasonable amount of error and conservative when compared to the original design. The main source of error here is over estimation of loads.

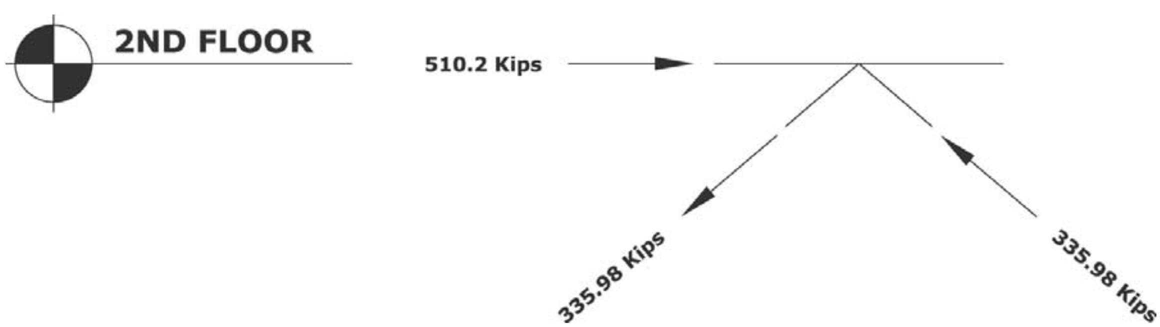
Lateral Braced Frame "Frame 1"

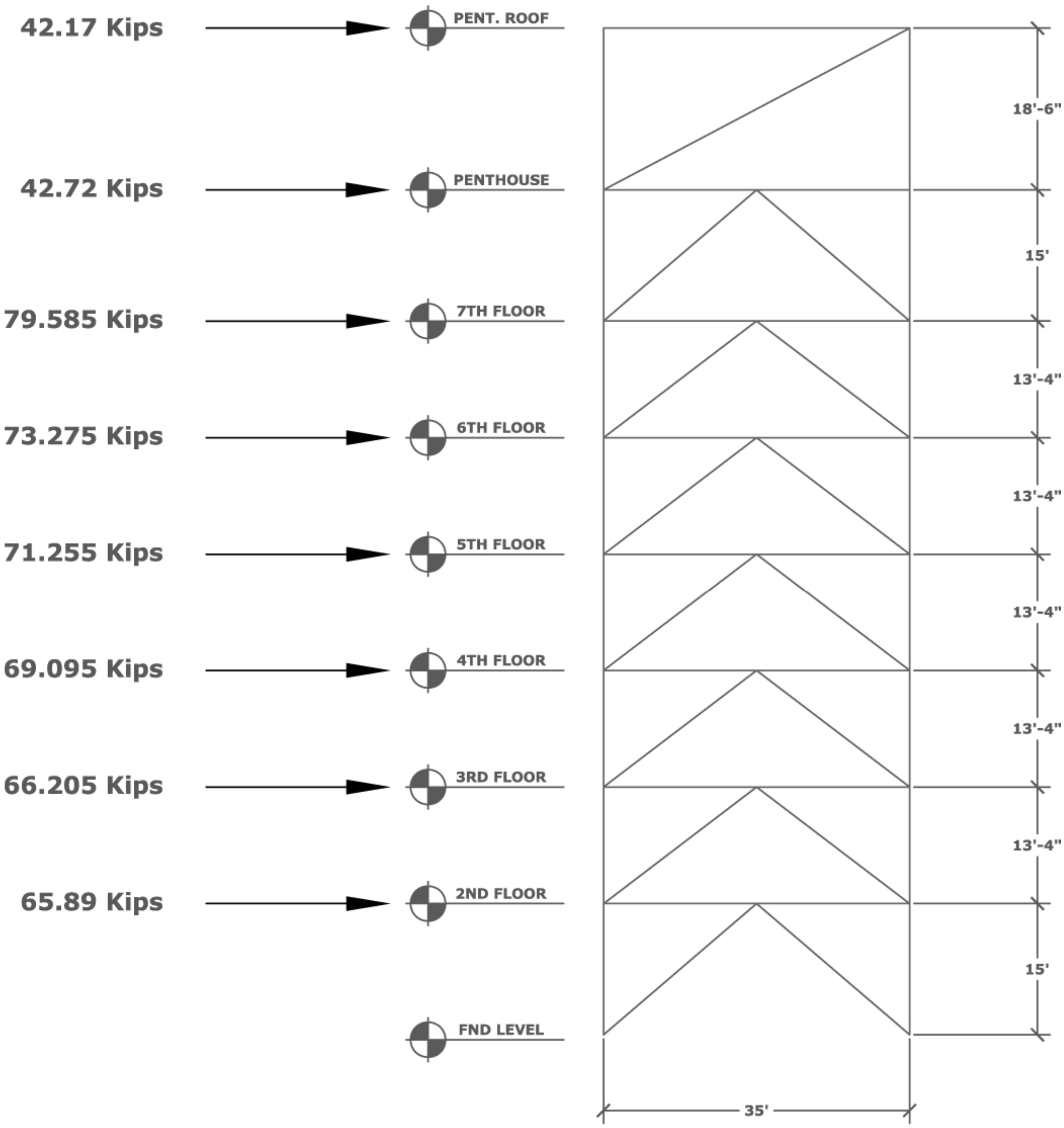
"Frame 1" is located in the eastern quadrant of the building and spans in the east-west direction. The distributions of the lateral forces to "Frame 1" were assumed to be half the applied factored shear force. This was made assuming both Frame 1 and Frame 4 are made up of the same members.

"Frame 1" was chosen as its design is controlled by wind, because of the discrepancies in my calculated seismic base shear I felt this would provide a better check of member sizes.

The general layout and applied forces can be seen in the image on the following page.

Taking the load down to the base of the frame I developed the following free body diagram at the intersection of the diagonals between the foundation level (FND) and the 2nd floor.





FRAME #1

To simplify the design I have sized the worst case and assigned that size to all other lateral members, this is consistent with what the original designers called for.

My spot check yielded that an HSS 10x10x5/16 section is sufficient for the loading. The original design calls for HSS 10x10x1/2 sections. The original design was probable bumped up to a HSS 10x10x1/2 for serviceability issues. My design is within acceptable limits of the original.

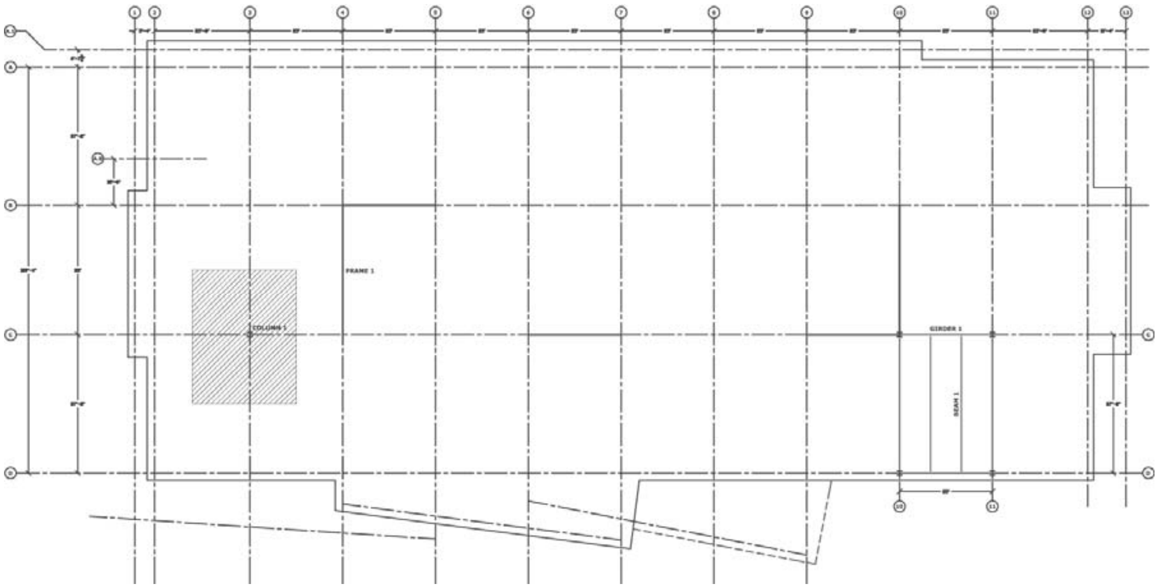
The hand calculation for “Frame 1” can be found in the appendix and is labeled hand calc. – 2.

Impact from foundation on Superstructure

With the shallow footings used in this building special consideration will have to be taken when transfer moment and shear down into the foundation. The sloping columns will also create a higher overturning moment.

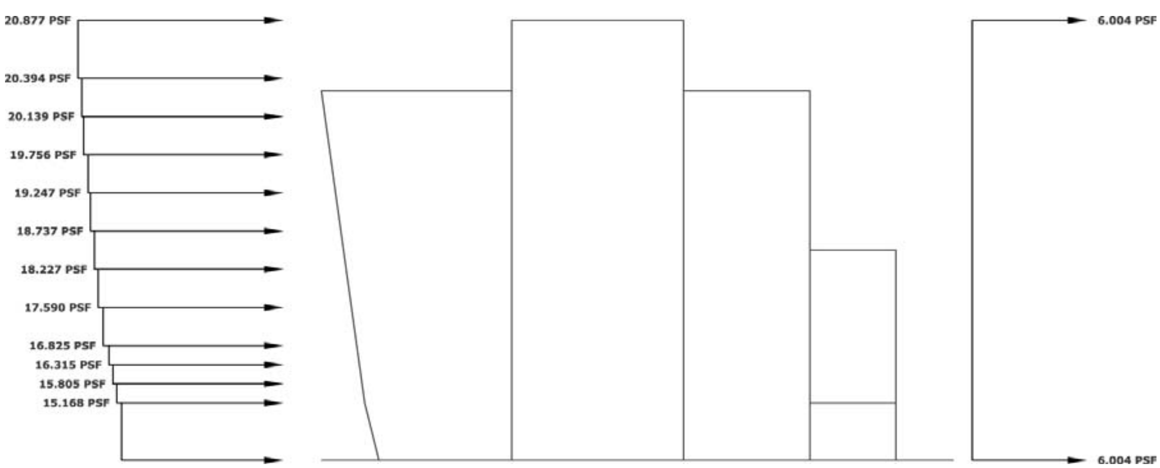
Appendix

Typical Building Footprint

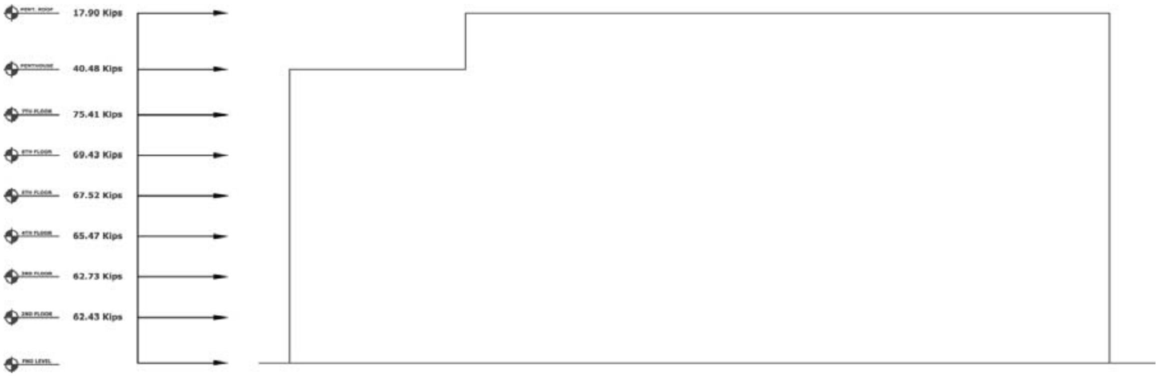


Wind Pressures E-W Diagram 1

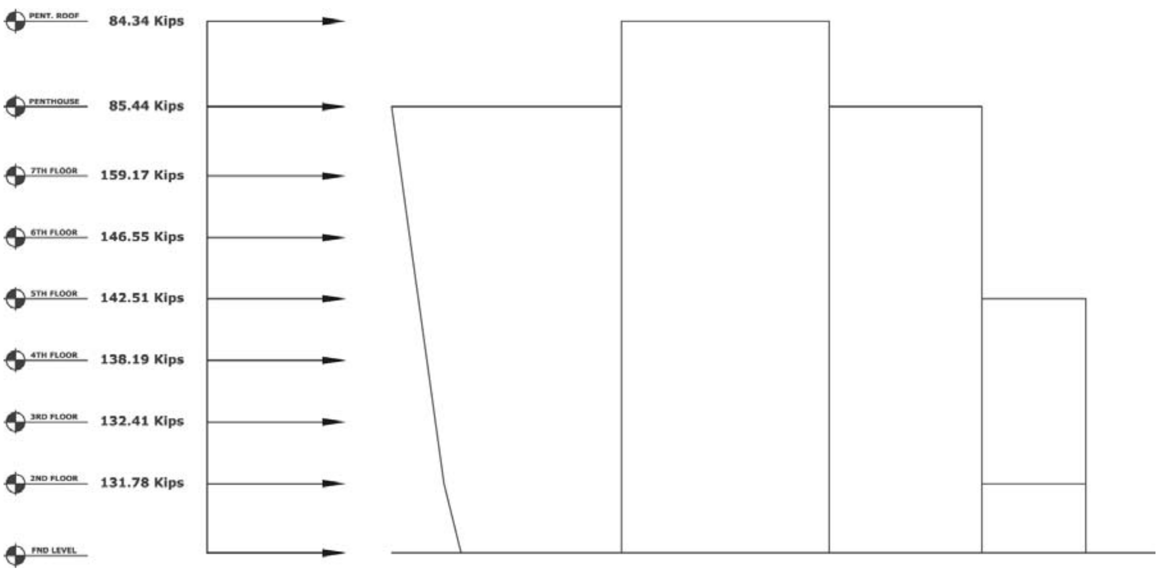
Wind Pressure N-S Diagram 2



Factored Wind Shear E-W Diagram 3



Factored Wind Shear N-S Diagram 4



Wind Load Calculation Spreadsheet – Table 1 and 2

Wind Loading for Parkridge Center - Phase VI			
ASCE7-05 Chapter 6			
Method 2			
V	90	mph	
Kd	0.85	MWFRS & Components and Cladding	
I	0.87	Importance Factor (Occupancy Category II - Non-Hurricane Prone)	
Kz, Kh	(see Table)		
Kzt	1		
G, E-W	0.859		
G, N-S	0.841		
	Enclosed	Enclosure Classification	
GCpi	0.18	+/-	
Wall Pressure			
Cp, E-W	0.8	windward	
	-0.5	leeward	
	-0.7	side wall	
Cp, N-S	0.8	windward	
	-0.3	leeward	
	-0.7	side wall	
Roof Pressure			
Cp, E-W	-0.9	0 to h/2	
	-0.9	h/2 to h	
	-0.5	h to 2h	
	-0.3	>2h	
Cp, N-S	-0.9	0 to h/2	
	-0.9	h/2 to h	
	-0.5	h to 2h	
	-0.3	>2h	
qz, qh	(see Table)		

Windward		Leeward		Sidewall		Roof	
E-W		E-W		E-W		E-W	
z	P	z	P	z	P	0 to h/2	-14.275
0-15	15.168	0-15	-6.004	0-15	-10.140	h/2 to h	-14.275
20	15.805	20	-6.004	20	-10.140	h to 2h	-6.004
25	16.315	25	-6.004	25	-10.140	>2h	-1.869
30	16.825	30	-6.004	30	-10.140		
40	17.590	40	-6.004	40	-10.140		
50	18.227	50	-6.004	50	-10.140		
60	18.737	60	-6.004	60	-10.140		
70	19.247	70	-6.004	70	-10.140		
80	19.756	80	-6.004	80	-10.140		
90	20.139	90	-6.004	90	-10.140		
100	20.394	100	-6.004	100	-10.140		
115.17	20.877	115.17	-6.004	115.17	-10.140		
N-S		N-S		N-S		N-S	
z	P	z	P	z	P	0 to h/2	-13.897
0-15	14.948	0-15	-1.742	0-15	-9.846	h/2 to h	-13.897
20	15.572	20	-1.742	20	-9.846	h to 2h	-5.794
25	16.072	25	-1.742	25	-9.846	>2h	-1.742
30	16.571	30	-1.742	30	-9.846		
40	17.320	40	-1.742	40	-9.846		
50	17.945	50	-1.742	50	-9.846		
60	18.444	60	-1.742	60	-9.846		
70	18.944	70	-1.742	70	-9.846		
80	19.443	80	-1.742	80	-9.846		
90	19.818	90	-1.742	90	-9.846		
100	20.067	100	-1.742	100	-9.846		
115.17	20.541	115.17	-1.742	115.17	-9.846		

Intermediate Tables for Wind Load Calculations

Kz, Kh

z, ft.	C
0-15	0.85
20	0.90
25	0.94
30	0.98
40	1.04
50	1.09
60	1.13
70	1.17
80	1.21
90	1.24
100	1.26
115.17	1.30
120	1.31
140	1.36
160	1.39
180	1.43
200	1.46
250	1.53
300	1.59
350	1.64
400	1.69
450	1.73
500	1.77

G

B, E-W	121.43	ft
B, N-S	267	ft
h	115.17	ft
l	500	ft
z,bar	69.102	ft
z,min	15	ft
e,bar	0.2	
c	0.2	
Iz	0.176821	
gq	3.4	
gv	3.4	
Lz	579.65	
Q, E-W	0.858	
Q, N-S	0.821	
G, E-W	0.859	
G, N-S	0.841	

L/B

L/B, E-W	0.45
L/B, N-S	2.20

z, ft.	q	
0-15	15.771	
20	16.699	
25	17.441	
30	18.183	
40	19.297	
50	20.224	
60	20.967	
70	21.709	
80	22.451	
90	23.008	
100	23.379	
115.17	24.082	<----------- qh

Seismic Building Weight and Base Shear – Table 3 & 4

Seismic Loading								
ASCE7-05								
Calculation of Building Weight								
Floor	Area		DL		Weight			
1	--	SF	--	KSF	--	kips		
2	32079.313	SF	0.061	KSF	1956.838	kips		
3	31705.554	SF	0.061	KSF	1934.039	kips		
4	32242.852	SF	0.061	KSF	1966.814	kips		
5	31415.809	SF	0.061	KSF	1916.364	kips		
6	31807.059	SF	0.061	KSF	1940.231	kips		
7	32198.309	SF	0.061	KSF	1964.097	kips		
Roof	19485.282	SF	0.025	KSF	487.132	kips		
Garden	6371.666	SF	0.16	KSF	1019.467	kips		
Mechanical	6750.000	SF	0.14	KSF	945.000	kips		
Penthouse roof	6750.000	SF	0.025	KSF	168.750	kips		
				Total:	14298.731	kips		
Precast Panels								
Wall	Perimeter		Height		DL		Weight	
1	765.813	LF	15.00	Ft	0.036	KSF	413.54	kips
2	810.978	LF	13.33	Ft	0.036	KSF	389.27	kips
3	801.849	LF	13.33	Ft	0.036	KSF	384.89	kips
4	809.401	LF	13.33	Ft	0.036	KSF	388.51	kips
5	784.479	LF	13.33	Ft	0.036	KSF	376.55	kips
6	788.229	LF	13.33	Ft	0.036	KSF	378.35	kips
7	791.979	LF	15.00	Ft	0.036	KSF	427.67	kips
						Total:	2758.78	kips
Total Building Weight:	17057.508	Kips						

Calculation of Base Shear		
S_s	0.200	
S_1	0.080	
S_{ms}	0.320	
S_{m1}	0.192	
S_{ds}	0.213	
S_{d1}	0.128	
R	3	
Ω_0	3	
C_d	3	
I	1	
Ct	0.03	
x	0.75	
h	115.17	ft
Ta	1.05	
C_s	0.040	
C_sW	690.05	kips

Composite Beam and Girder Spreadsheet – Tables 5 & 6

Composite Beam Check			
LRFD			
BEAM 1 (10-11, C-D)			
Live Load	100.00	PSF	
A_T	309.72	SF	
KLL	2.00		
Live Load after Reduciton	85.27	PSF	
Dead Load	61.00	PSF	
Factored Total Load	209.63	PSF	Max of Load combinations outlined in ASCE7-05
Spacing 1	8.33	ft	
Spacing 2	8.33	ft	
Span	37.17	ft	
w	1746.91	PLF	
M_{max}	301.64	ft*K	
V_{max}	32.46	K	
B_{eff}	100.00	in	
F'_c	3.00	ksi	
F_y	50.00	ksi	
Depth of Concrete	3.25	in	
Total Depth of Slab	5.25	in	
Steel Shape	W16x31		
PNA Location	7		
ΣQ_n	114.00	K	Input value from Table 3-19 in AISC Manual of Steel Construction 13th Edition
a	0.45	in	
y2	5.03	in	
$y2_{,high}$	5.00	in	
$y2_{,low}$	4.50	in	
$M_{n,\ high}$	325.00	ft*K	
$M_{n,\ low}$	319.00	ft*K	
$M_{n,\ Actual}$	325.32	ft*K	**Ok**
Deck Direction	Perpindicular		
w_r	--	in	Average Width of concrete rib
h_r	--	in	Nominal rib height
w_r/h_r	--		
Qn	14.60	K	Strength of a Single 3/4" Stud, 2 per rib in the weak position (Table 3-21 AISC Manual of Steel Construction)
# Studs	16.00		
Use W16x31 with 16 Shear Studs			

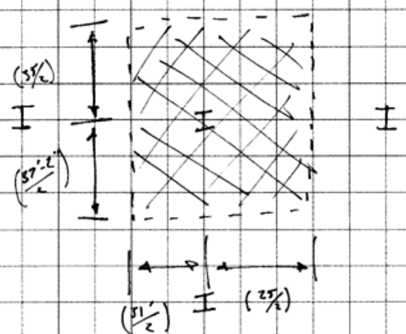
Composite Beam Check			
LRFD			
GIRDER 1 (10-11, C)			
Live Load	100.00	PSF	
A_T	902.08		
KLL	2.00		
Live Load after Reduciton	60.31	PSF	
Dead Load	61.00	PSF	
Factored Total Load	169.70	PSF	Max of Load combinations outlined in ASCE7-05
Spacing 1	35.00	ft	
Spacing 2	37.17	ft	
Span	25.00	ft	
w	6123.46	PLF	
M_{max}	478.40	ft*K	
V_{max}	76.54	K	
B_{eff}	75.00	in	
F'_c	3.00	ksi	
F_y	50.00	ksi	
Depth of Concrete	3.25	in	
Total Depth of Slab	5.25	in	
Steel Shape	W21x44		
PNA Location	7		
ΣQ_n	259.00	K	Input value from Table 3-19 in AISC Manual of Steel Construction 13th Edition
a	1.35	in	
y2	4.57	in	
$y2_{,high}$	5.00	in	
$y2_{,low}$	4.50	in	
$M_{n,\ high}$	584.00	ft*K	
$M_{n,\ low}$	574.00	ft*K	
$M_{n,\ Actual}$	575.46	ft*K	Ok
Deck Direction	Parallel		
w_r	6.00	in	Average Width of concrete rib
h_r	2.00	in	Nominal rib height
w_r/h_r	3.00		
Qn	17.10	K	Strength of a Single 3/4" Stud (Table 3-21 AISC Manual of Steel Construction
# Studs	31.00		
Use W21x44 with 31 Shear Studs			

Column 1 – Hand Calc 1

STRUCTURAL DESIGN GROUP, LIMITED
CONSULTING STRUCTURAL ENGINEERS
Gaithersburg, Maryland • Miami, Florida

JOB THESIS - COLUMN CHECK
SHEET NO. ______ OF ______
CALCULATED BY ______ DATE ______
SCALE ______

COLUMN C-3



PH ROOF: 1.2(25 PSF) + 1.6(20 PSF) = 62 PSF
(17.5)(28)(62 PSF) = 30380 lbs

ROOF LEVEL:
MECHANICAL: 1.2(96) + 1.6(100) = 275.2 PSF (UN. RED.)
ROOF: 1.2(25) + 1.6(20) = 62 PSF

MECHANICAL: (17.5)(28)(275.2) = 134848 lbs
ROOF: (18.583)(28)(62) = 32260.1 lbs

TYP. FLR: 1.2(61) + 1.6(100) = 233.2 PSF

(36.047)(28)(233.2) = 235604 lbs/FLOOR

6 FLOOR · 235604 lbs/FLOOR = 1.41365×10^6 lbs

TOTAL AXIAL AT BASE COLUMN: 1.61113×10^6 lbs
OR
1611.13 kips

FIX - PIN CONDITION K = 0.7
l = 15'
KL = 15 (0.7) = 10.5

FROM TABLE 4-1 IN AISC MANUAL OF STEEL CONSTRUCTION
TRY W14 × 132

INTERPOLATE: $\frac{11-10.5}{11-10} = \frac{1590-x}{1590-1620}$; $\frac{0.5}{1} = \frac{1590-x}{30}$
15 = 1590 − x
1605 = x < 1611.13 k N.G.

TRY W14 × 145

KL = 11 ⇒ ϕP_n = 1770 k > 1611.13 k OK

USE W14 × 145

120 NE 27th Street • Suite 500 • Miami, FL 33137 • Tel: 305.571.9400 • Fax: 305.572.9161
12 South Summit Avenue • Suite 110 • Gaithersburg, MD 20877 • Tel: 301.987.9234 • Fax: 301.987.9237

Frame 1 – Hand Calc 2

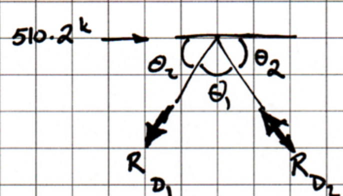
STRUCTURAL DESIGN GROUP, LIMITED
CONSULTING STRUCTURAL ENGINEERS
Gaithersburg, Maryland • Miami, Florida

JOB
SHEET NO. OF
CALCULATED BY DATE
SCALE

BASED ON FRAMING LOWER WILL TAKE SHEARS FROM UPPER LEVELS PLUS ITS SHEAR:

$$V_{①} = \left(\frac{84.94}{2}\right) + \left(\frac{85.44}{2}\right) + \left(\frac{159.17}{2}\right) + \left(\frac{146.55}{2}\right) + \left(\frac{142.41}{2}\right) + \left(\frac{138.19}{2}\right) + \left(\frac{132.41}{2}\right) + \left(\frac{131.78}{2}\right)$$

$$= 510.17^k \approx 510.2^k$$



$\theta_1 = 49.4°(2) = 98.8$
$\theta_2 = 40.6°$

$$\overset{+}{\rightarrow} \Sigma F_x = 510.2^k - R_{D_1}\cos(\theta_2) - R_{D_2}\cos(\theta_2) = 0$$

$$+\uparrow \Sigma F_y = -R_{D_1}\sin(\theta_2) + R_{D_2}\sin(\theta_2) = 0$$

$$R_{D_1} = R_{D_2}$$

$$\Sigma F_x = 510.2 - 2R_{D_1}\cos(\theta_2) = 0$$

$$-2R_{D_1}\cos(\theta_2) = -510.2$$

$$R_{D_1}\cos(\theta_2) = \frac{510.2}{2}$$

$$R_{D_2} = R_{D_1} = \frac{510.2}{2\cdot\cos(\theta_2)} = \frac{510.2}{2\cos(40.6)} = \boxed{335.98^k}$$

$F_y = 46^{ksi}(0.90) = 41.4$ ← CONTROLS
$F_u = 58^{ksi}(0.75) = 43.5$

$$\sigma = \frac{P}{A} \Rightarrow A_e = \frac{P}{\sigma} = \frac{335.98^k}{41.4\ ksi} = 8.16\ in^2$$

USE HSS 10×10×5/16 $A_c = 8.32\ in^2$ OK

~~120 NE 27th Street • Suite 300 • Miami, FL 33137 • Tel: 305.571.9400 • Fax: 305.572.9161~~
12 South Summit Avenue • Suite 110 • Gaithersburg, MD 20877 • Tel: 301.987.9234 • Fax: 301.987.9237