TECHNICAL REPORT 1

BRIDGESIDE POINT II

PITTSBURGH, PA

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Structural Option

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10/5/2007

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EXECUTIVE SUMMARY

In this first technical report the existing structural conditions of Bridgeside Point II are discussed through a detailed description of the floor, column, foundations, and lateral systems. Typical floor framing plans and other drawings are included for a better understanding of the systems in place. A summary of governing building codes and material strengths is provided. All wind and seismic loads were determined from ASCE 7-05. Wind loads used Method 2 – Analytical Procedure of ASCE 7-05 section 6.5 was used. Seismic design loads were determined using the equivalent lateral force procedure found in chapters 11 and 12 of ASCE 7-05. Spot checks of both gravity and lateral loads were done for a typical floor bay, a column, and lateral bracing to check the validity of the load distributions assumed. In general, assumptions and distribution of building loads are valid and slightly conservative with the exception of lateral loads, which are conflicting. Appendices to this report are included and supply the reader with back-up calculations, figures, and tables.

INTRODUCTION: BRIDGESIDE POINT II

The Bridgeside Point II project consists of five above grade stories with a combination of office and laboratory space. It is located in the Pittsburgh Technology Center, which is just east of downtown Pittsburgh, Pennsylvania. The building conveys a feeling of progression from a historic steel mill town to a fast-paced, innovation driven city through its use of clean lines, visible lateral system, and open plan. A glass curtain wall lends itself for a feeling a transparency on the upper floors, while dense, pre-cast panels wrap the ground floor.

The building is approximately 150,000 square feet and reaches a height of 75 feet above grade. The building floor template is an open plan with a design core capable of housing office and laboratory spaces as each floor is roughly 15 feet floor to floor. A typical bay is 30 feet by 32 feet, and is comprised of composite steel with a concrete slab on deck. The lateral system is a series of braced frames, two in the east – west building direction and three in the north – south building direction. The foundation system is a driven pile system. A typical pile cap hosts between three and seven piles and has a thickness of 3'-6" to 4'-6". The ground floor is a reinforced slab of grade with grade beams around the perimeter.

Flexibility is the main concept this building expresses. At the time of design, no definite tenant had been selected; therefore, this fueled the design to be extremely flexible. In order to create this flexibility two things are directly affected. The desired large bays require a heavy uniform live load, thus larger structural members; and placement of the lateral system is limited. This report begins to address these issues through the use of both simplified and detailed analysis.

STRUCTURAL SYSTEMS

Foundations

A driven pile system with pile caps containing between two and nine piles provides the foundation system for the building with a capacity of 105 to 130 tons. The pile caps vary in thickness from 3'-6" to 4'-6" and have between 9 and 12 No. 9 reinforcing bars. Depending on their location within the site, they are driven to a depth of 45 to 55 feet. These piles support the framing system as well as a 4" thick concrete slab on grade. The elevator core rests on a 3'-6" thick cap supported by 6 piles. Along the perimeter are 12" thick grade beams.

Floor System

The floor system of Bridgeside Point II is a composite system with a typical bay size of 30'-0" by 32'-0". A 3" concrete slab rests on 3" – 20 Gage composite steel decking. ¾" diameter (5 ½" long) shear studs are used to create composite action. Supporting the deck are W21 beams with the most typical being W21x44 spaced at 10'-0" center to center. W24x62 girders then transfer loads to the columns. This type of floor system is used on each floor aside from the ground floor which is the 4" slab on grade mentioned in the foundation section above. The roof has a different layout, as only part of the roof houses the penthouse. In these locations, the typical 3" slab and 3" composite deck is used; however, in all other roof locations, a 1 ½" galvanized steel deck is used in conjunction with 3" of concrete and is supported by k-series joists spaced 6'-0" center to center.

Columns

The columns used in Bridgeside Point II are fairly standard for a mid-rise building. Gravity columns range from W10's to W12's depending on their location within the building footprint. The lateral columns are bigger in size range from W14x99 to W14x145. Gravity columns have a great fluctuation in service loading as these loads range from 175^k to 600^k, where as lateral columns support 500^k to 730^k. All of the main building columns are spliced at the 3rd - 4th floor. Gravity base plates are a standard 4 bolt connection with plate thickness ranging from 1 ½" to 2". Penthouse columns are either HSS6x6x1/4 or HSS8x8x1/2.

Lateral System

Large braced frames make up the building's lateral load resisting system. In order to increase the flexibility of the building plan, the perimeter was chosen for the bracing. Four of the five bracing frames are exposed via windows. In these bays, large HSS8x8x3/8 and HSS10x10x1/2 provide the bracing at the second through fifth floors and are K-Braces, which create a two story "X" in the window. On the first floor these four frames have an eccentric brace, whereas the large fifth frame is two bays wide and is comprised of all W-shape eccentric braces. The beam and brace to column connections are all shear in nature. Typical lateral column base plates are 3" thick with significantly large pile caps in order to handle the large punching shear forces.

CODE AND DESIGN REQUIREMENTS

Codes and References

The 2006 International Building Code as amended by the City of Pittsburgh.

The Building Code Requirements for Structural Concrete (ACI 318-05), American Concrete Institute.

Specification for the Design, Fabrication and Erection of Structural Steel for Buildings – Allowable Stress Design, Ninth Edition, American Institute of Steel Construction.

Minimum Design Loads for Buildings and Other Structures (ASCE 7-05), American Society of Civil Engineers.

Deflection Criteria

Floor Deflection Criteria

L/240 Total Load and L/360 Live Load

L/600 Curtain Wall Load

L/1666 Impact Load on Elevator Support Beams

Lateral Deflection Criteria

H/500 Total Allowable Wind Drift

H/400 Story Wind Drift

H/600 Total Allowable Seismic Drift

MATERIALS

Concrete

Foundations	f'_c = 3000 psi
Walls	f'_c = 4000 psi
Slabs on grade	f'_c = 4000 psi
Interior and Exterior Slabs	f'_c = 4000 psi

Reinforcing Steel

Reinforcing Bar	ASTM A615 Grade 60
Foundations	ASTM A185

Structural Steel

Structural W-shapes	ASTM A992
Structural M, S, HP-shapes	ASTM A572 Grade 50
Channels	ASTM A572 Grade 50
Steel Tubes (HSS Shapes)	ASTM A500 Grade B
Steel Pipe (Round HSS)	ASTM A500 Grade B
Angles and Plates	ASTM A36

Metal Deck and Shear Studs

Composite Floor	3” with painted underside
Rook Deck	1 ½” Galvanized
Studs	¾” x 5 ½” headed stud

GRAVITY AND LATERAL LOADS

ASCE 7-05 was used to determine the gravity and lateral loads.

DEAD LOADS (Assumed)

Construction Dead Load

Concrete	150 PCF
Steel	490 PCF

Construction Dead Load

Partitions	20 PSF
M.E.P.	10 PSF
Finishes & Misc.	5 PSF
Windows & Framing	20 PSF
Roof	20 PSF

LIVE LOADS

Public Areas	100 PSF
Lobbies	100 PSF
First Floor Corridors	100 PSF
Corridors above First Floor	80 PSF
Office	50 PSF
Light Storage	125 PSF
Mechanical	150 PSF
Stairs	100 PSF

LATERAL LOADS

The following section reviews the Wind and Seismic load distribution per ASCE 7-05. For a detailed summary, please refer to Appendix B and C. The figure to the right (Figure 1) shows the lateral system on a typical floor.

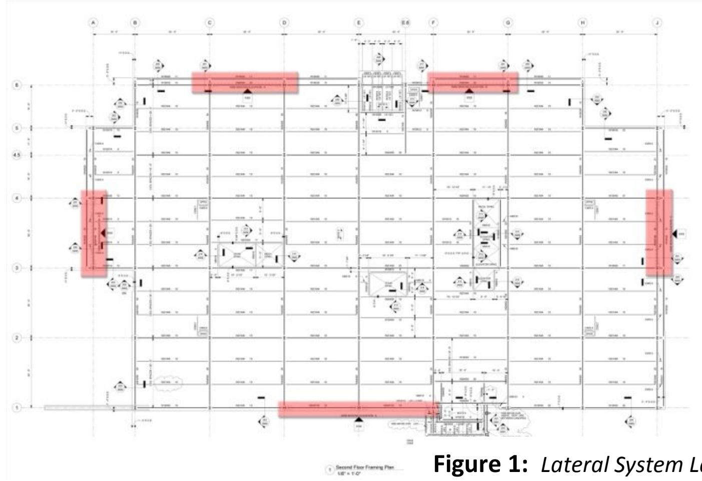

Figure 1: *Lateral System Layout*

Wind

Wind loads were analyzed using section 6.5 of the code. Appendix B contains a detailed layout of the required equations and factors required code. These factors are location and building specific; however, there are some experimental factors that are used to determine the various pressures. The building does have some irregularities in footprint. For this analysis the footprint was taken to be square. Wind effects were also neglected on the rooftop screenwall and lower level canopy. This will be considered in a later analysis as their effects on the overall variables and outcome were fairly negligible. A simplified approach was taken to determine the fundamental period (discussed in more detail in Appendix C), and it was concluded that the building was flexible in nature. Since the building is rectangular in orientation, the loads on the shorter side, in this case, winds from the East-West direction were found to control design. Due to inconsistent building floor heights, the windward pressures are not a perfect curve, but noticeable linear progression is evident (see Figures 2 & 3 below).

Floor Heights	Level	Total Height	K_z	q_z	Wind Pressures (psf)					
					N-S Windward	N-S Leeward	N-S Side Wall	E-W Windward	E-W Leeward	E-W Side Wall
16.25	Roof	75.00	0.91	16.04	10.99	-3.78	-9.62	10.81	-6.76	-9.46
14.75	5	58.75	0.85	14.98	10.27	-3.78	-9.62	10.10	-6.76	-9.46
14.50	4	44.00	0.79	13.92	9.54	-3.78	-9.62	9.38	-6.76	-9.46
14.75	3	29.50	0.70	12.34	8.46	-3.78	-9.62	8.32	-6.76	-9.46
14.75	2	14.75	0.57	10.05	6.89	-3.78	-9.62	6.77	-6.76	-9.46

Figure 2: *Distribution of Windward and Leeward Pressures*

Level	**Wind Design**					
	Load (kips)		Shear (kips)		Moment (ft-k)	
	N-S	E-W	N-S	E-W	N-S	E-W
Roof	35	70	0	0	2610	5245
5	30	61	35	70	1765	3578
4	28	57	65	131	1232	2523
3	26	54	93	188	772	1607
2	23	49	119	243	336	721
Total	142	292	142	292	6716	13,674

Figure 3: *Total Base Shear from Windward and Leeward Pressures*

The following (Figures 4 & 5) show the progression of force along both faces of the building.

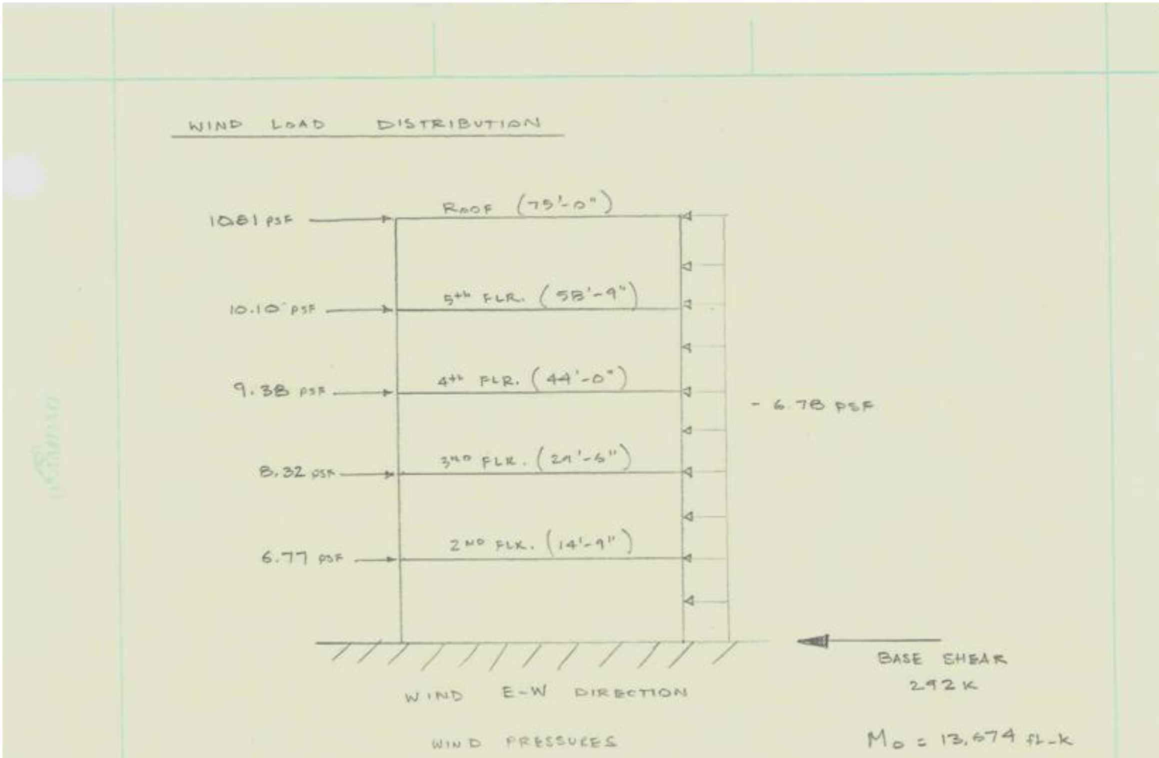

Figure 4

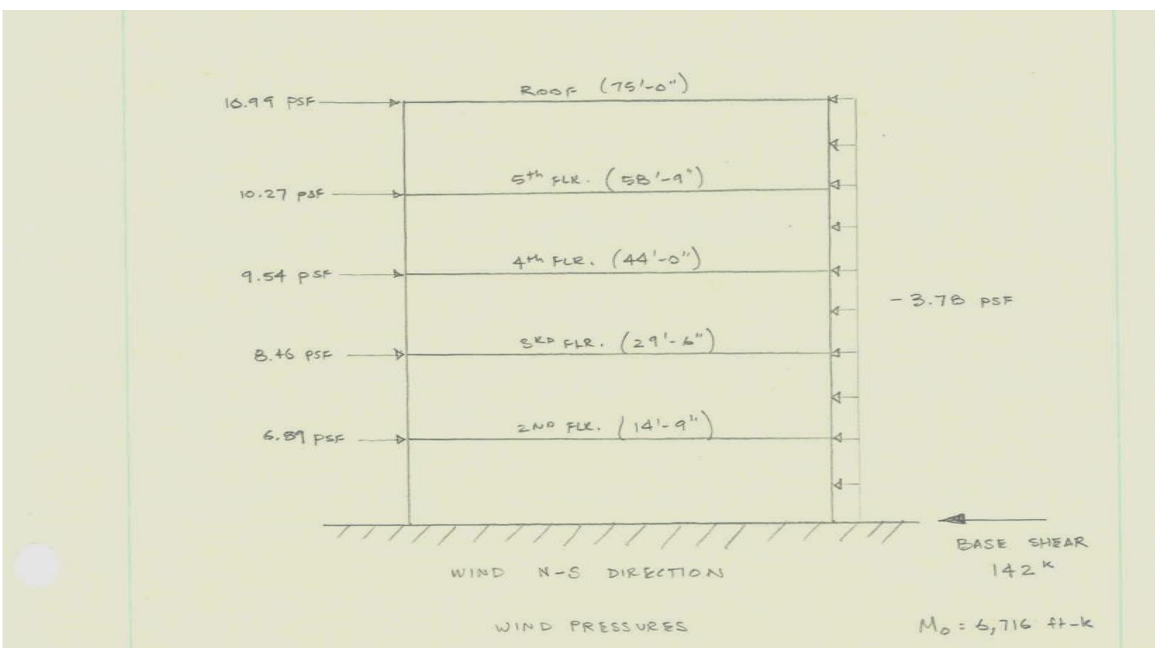

Figure 5

Seismic

Seismic loads were analyzed using chapters 11 and 12 of the code. Appendix C contains a detailed layout of the required equations and factors required code. These factors are location and building specific; however, there are some experimental factors that are used to determine the various pressures. A detailed dead load description is present in the seismic section. In this analysis, the dead loads assumed are greater than what was used in the design; therefore, the base shear and over-turning moment are approximately 25 percent larger than the design value. I realize this mistake and will make the necessary adjustments in a future technical report. Below is a load distribution table (Figure 6).

Base Shear and Overturning Moment Distribution							
Story	h_x (feet)	Floor Load (k)	$h_x^k W_x$	C_{vx}	$F_x = C_{vx}V$	V_x (k)	M_x (ft-k)
Roof	75.00	2960	1248468	0.365	118.2	0.0	8862.0
5	58.75	3310	991834	0.290	93.9	118.2	5514.9
4	44.00	3310	661701	0.193	62.6	212.0	2755.6
3	29.50	3310	378076	0.110	35.8	274.7	1055.6
2	14.75	3310	143264	0.042	13.6	310.4	200.0
1	0	2500	0	0.000	0.0	324.0	0.0
Total	75	16200	3423343	1.000	**324**	324	**18388**

Figure 6: *Base Shear and Overturning Moment Distribution*

Note: $V_{experimental}$ = 324 kips > V_{design} = 261 kips; thus seismic controls (Does **NOT** check with design)

Response to Lateral Loads

Upon completion of the lateral analysis, I have discovered two assumptions I used for analysis that are not entirely accurate.

First, my dead load assumptions are not the same as the design engineer. I know this because our C_s values are the same. I identified possible locations for error, one being an overestimation of the slab and deck weight, and others being an overestimation of the roof and penthouse loads. Further analysis will streamline these values.

Second, I assumed a perfect distribution of lateral loads among the frames. I did not consider member stiffness effects in great detail. The frames are comprised of concentric and eccentric braces with different response modifiers; therefore, this could create a "soft-story," or distribute the loads in a different manner.

From my analysis I am not entirely sure which force (wind or seismic) is governing the design. I plan on investigating this further in Technical Report 3 with a detailed model and deeper look at element stiffness. A possible thesis would be an investigation of alternate lateral systems and their response to the lateral forces in an effort to discern which force truly governs design.

CONCLUSION

In this first technical report the existing structural conditions of Bridgeside Point II are discussed through a detailed description of the floor, column, foundations, and lateral systems. Typical floor framing plans and other drawings are included for a better understanding of the systems in place. Spot checks of both gravity and lateral loads were done for a typical floor bay, a column, and lateral bracing to check the validity of the loads distribution assumed. It was found that the my design dead loads were conservative and in many cases resulted in an over-designed member. The composite steel beams were found to match the size selected by the engineer, however, the number of shear studs were considerably higher because of the assumption that T = C, which is valid but it assumes a fully composite design which is not necessarily needed. The same was true with the composite girder design, same shape as the designer by more studs because of T = C. In a future technical report, these sizes will be re-examined to determine the correct shape and stud count. The analysis of the column design was fairly accurate; however, the compounding of the larger than necessary dead loads increased the column size by two to three sizes. I also did not investigate column and beam stiffness, nor did I consider any second order effects as this will be examined in the future as well. Even though the spot check values are larger, they are conservative, which for a preliminary analysis is acceptable.

Perhaps the most conflicting information comes from the lateral analysis. Since stiffness was not considered in great detail and the mass of the building was conservative, seismic was found to control. An analysis of a frame (found in Appendix F) using wind loading almost matches the design loads for the members, which leads one to believe that wind does in fact govern. Further review of this area is necessary, and will be looked at in much greater detail in technical report three. A discussion and investigation of alternative lateral systems could lend itself to a better idea of which force is controlling design. This could ultimately be a topic for a thesis proposal.

All design values were done in accordance with the applicable codes. Detailed notes, tables, and figures are provided in the appendices for further review. Any questions and/or comments should be directed to Antonio Verne through email: adv118@psu.edu.

APPENDIX A: BUILDING LAYOUT

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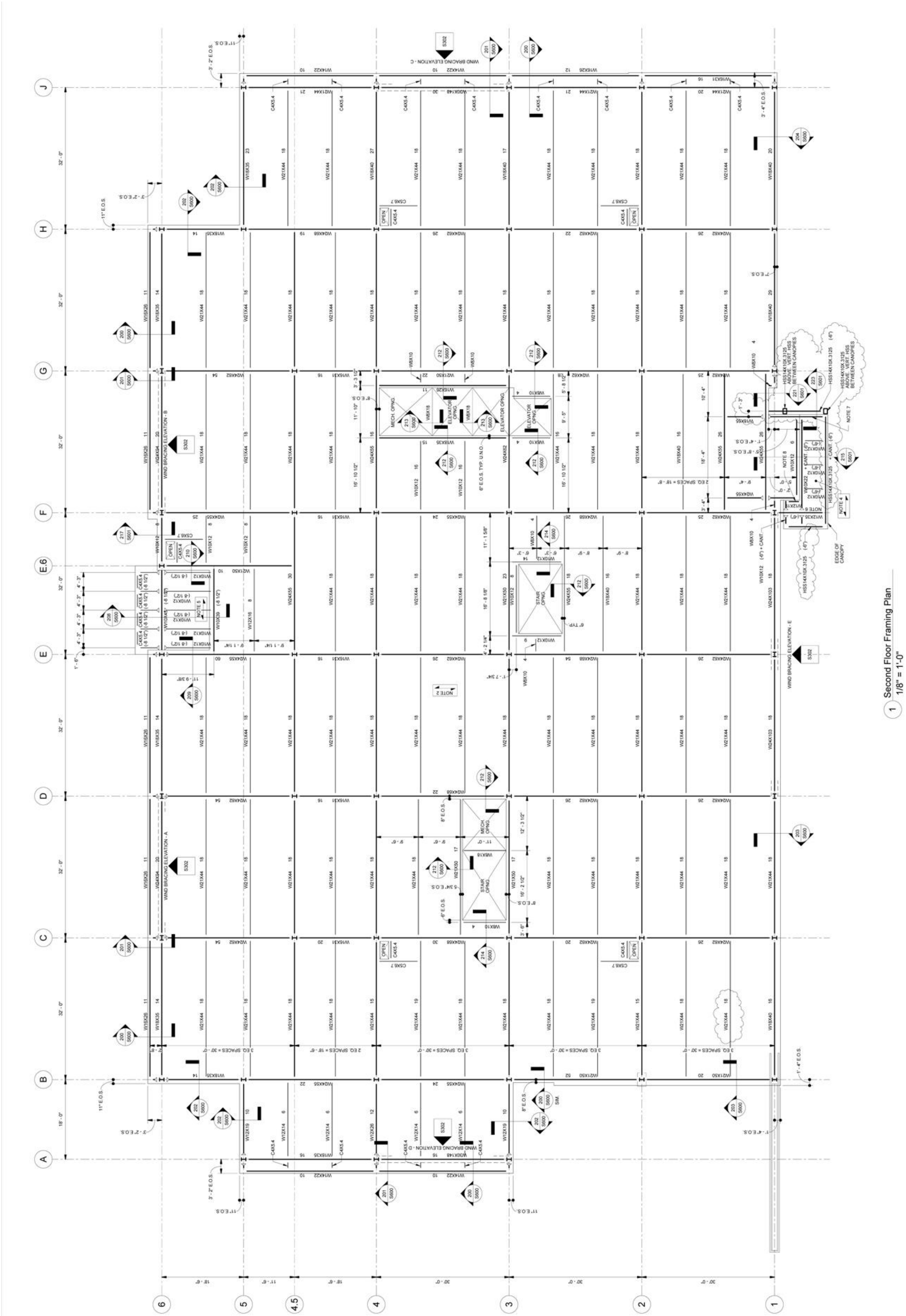
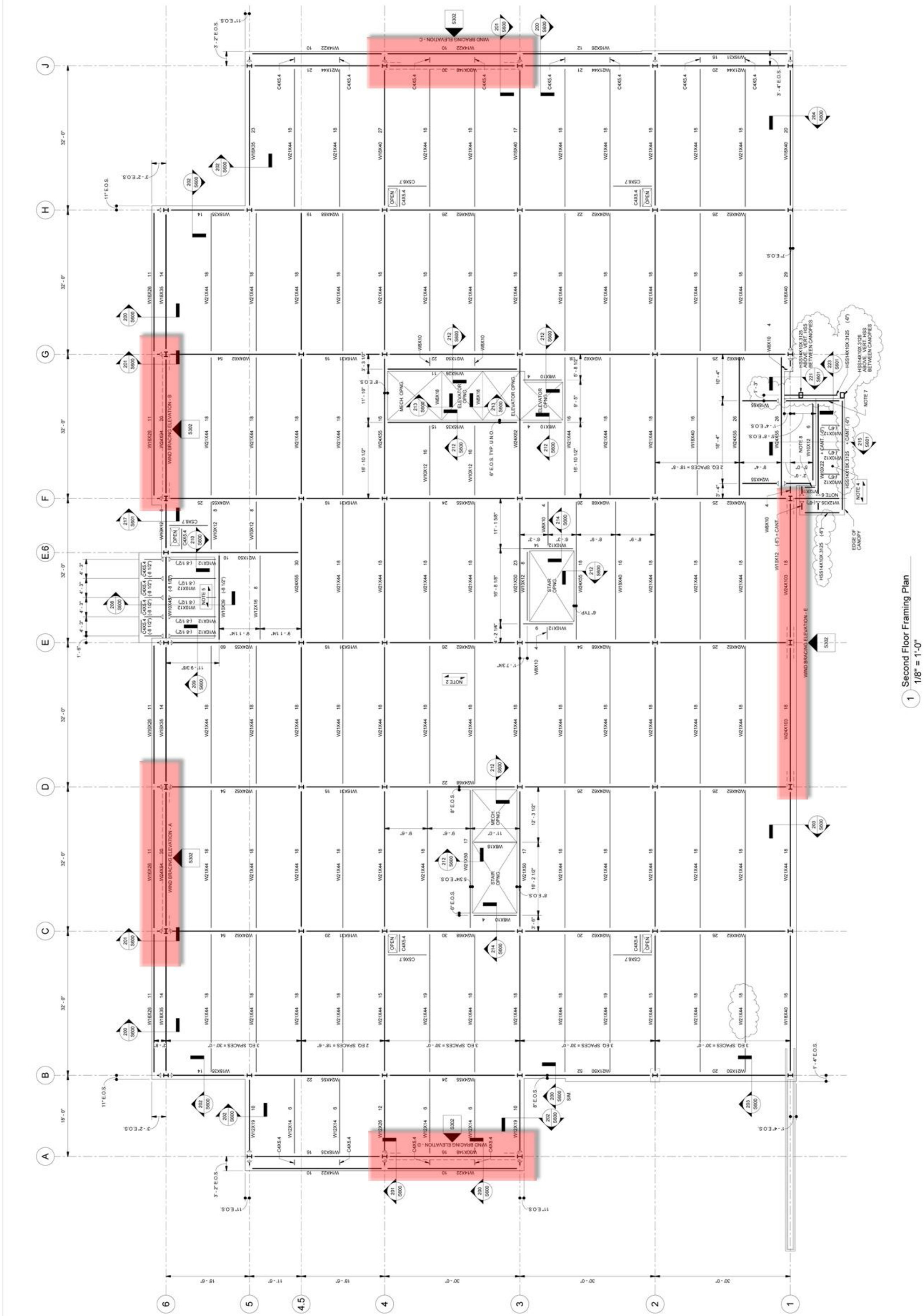
Typical Floor Layout



Framing Locations: Braced Frames



APPENDIX B: WIND ANALYSIS

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MAIN WIND-FORCE RESISTING SYSTEM (ASCE 7-05)

Bridgeside Point II -- Pittsburgh, PA

Basic Wind Speed (V) mph	90
Exposure Category	C
Importance Factor (I)	1.0
Wind Directionality Factor (K_d)	0.85
Topographic Factor (K_{zt})	1.0

Number of Floors	5
Building Height (feet)	75
N-S Building Length (feet)	245
E-W Building Length (feet)	145
L/B in N-S Direction	1.7
L/B in E-W Direction	0.6

Gust Factor		
	Wind Direction	
Variable	N-S	E-W
Stiffness	Flex.	Flex.
B	145	245
L	245	145
h	75	75
z	45	45
ℓ	500	500
ε	0.2	0.2
α	0.154	0.154
$\bar{b}$	0.65	0.65
V	90	90
V_z	90	90
η_h	3	3
η_B	7	13
η_L	32	19
L_z	532	532
n_1	0.77	0.77
N_1	5	5
R_n	0.0539	0.0539
R_h	0.2816	0.2816
R_B	0.1257	0.0766
R_L	0.0305	0.0510
β	0.50	0.50
R	0.0456	0.0359
I_z	0.19	0.19
g_r	4.13	4.13
g_q	3.4	3.4
g_v	3.4	3.4
Q	0.86	0.83
G_f	0.86	0.84

Wind Direction	C_p, windward	C_p, leeward	C_p, side wall	Gust Factor	Gcpi (+)	Gcpi (-)
N-S Direction	0.8	-0.275	-0.7	0.86	0.18	-0.18
E-W Direction	0.8	-0.5	-0.7	0.84	0.18	-0.18

MAIN WIND-FORCE RESISTING SYSTEM (ASCE 7-05)

Bridgeside Point II -- Pittsburgh, PA

Floor Heights	Level	Total Height	K_z	q_z	Wind Pressures (psf)					
					N-S Windward	N-S Leeward	N-S Side Wall	E-W Windward	E-W Leeward	E-W Side Wall
16.25	Roof	75.00	0.91	16.04	10.99	-3.78	-9.62	10.81	-6.76	-9.46
14.75	5	58.75	0.85	14.98	10.27	-3.78	-9.62	10.10	-6.76	-9.46
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14.75	3	29.50	0.70	12.34	8.46	-3.78	-9.62	8.32	-6.76	-9.46
14.75	2	14.75	0.57	10.05	6.89	-3.78	-9.62	6.77	-6.76	-9.46

Level	**Wind Design**					
	Load (kips)		Shear (kips)*		Moment (ft-k)	
	N-S	E-W	N-S	E-W	N-S	E-W
Roof	35	70	0	0	2610	5245
5	30	61	35	70	1765	3578
4	28	57	65	131	1232	2523
3	26	54	93	188	772	1607
2	23	49	119	243	336	721
Total	142	292	142	292	6716	13,674

* Note: Total Base Shear includes load from Windward and Leeward pressures

-End of Section-

APPENDIX C: SEISMIC ANALYSIS

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SEISMIC DESIGN

OCCUPANCY CATEGORY : II $\Rightarrow$ $I = 1.0$

$S_S = 0.125$ (FROM FIGURE 22-1 TO 22-14)

$S_1 = 0.049$

SITE CLASS D

$S_{MS} = S_S F_A = (0.125)(1.6) = 0.20$

$S_{M1} = S_1 F_V = (0.049)(2.4) = 0.1176$

$S_{DS} = 2/3\, S_{MS} = (2/3)(0.20) = 0.133 \longrightarrow A$

$S_{D1} = 2/3\, S_{M1} = (2/3)(0.1176) = 0.078 \longrightarrow B$ CONTROLS

SEISMIC DESIGN : B

RESPONSE MODIFICATION FACTOR : $R = 3.0$

ORDINARY COMPOSITE STEEL & CONCRETE BRACED FRAMES

$$C_S = \min \begin{cases} \dfrac{S_{DS}}{(R/I)} \\[2ex] \dfrac{S_{D1}}{T(R/I)} \\[2ex] \dfrac{S_{D1} T_L}{T^2 (R/I)} \end{cases} \geq 0.01$$

$T_a = C_t h_n^{\,x}$; FROM TABLE 12.8-2 : ECCENTRICALLY BRACED FRAMES

$h = 75$ ft $\qquad C_t = 0.03$, $x = 0.75$

$T_a = (0.03)(75)^{0.75}$; $T_a = 0.765$

$T_L = 12$ FROM P. 228 ASCE 7-05

$T = C_u T_a = (1.7)(0.765) = 1.3$

* NOTE $f = 1/T = 0.77\ H_z < 1\ H_z$ $\therefore$ FLEXIBLE STRUCTURE

$$C_s = \begin{cases} \dfrac{0.133}{(3/1)} = 0.0443 \\[2ex] \dfrac{0.078}{(1.5)(3/1)} = 0.020 \geq 0.01 \ \checkmark \text{ OK} \\[2ex] \text{MIN} \quad \dfrac{(0.078)(12)}{(1.5)^2(3/1)} = 0.185 \end{cases}$$

$C_s = 0.020$

FIND $W_{BUILDING}$:

TYP. FLOOR (2ND – 5TH) : 33,100 ft² EACH FLOOR

ROOF : 33,100 ft²

PENTHOUSE : 2100 ft²

CANOPY : 700 ft²

TYP. FLOOR LOADS

PARTITIONS	20 PSF
FIN. & MISC.	5 PSF
MEP.	10 PSF
6" SLAB / DECK	50 PSF
BEAMS / COLUMNS	15 PSF
	100 PSF

ROOF LOADS

MEP	10 PSF
ROOF MAT.	20 PSF
SLAB / DECK	40 PSF
MISC.	5 PSF
BEAMS / JOISTS	10 PSF
	85 PSF

PENTHOUSE

SLAB / DECK	35 PSF
BEAMS	10 PSF
MEP	10 PSF
MISC.	5 PSF
	60 PSF

CANOPY

ROOF	20 PSF
BEAM	10 PSF
	30 PSF

$$\begin{aligned} W_T = \ & (33,100 \text{ ft}^2)(4 \text{ FLOORS})(100 \text{ psf}) = 13,240^k \\ & (33,100 \text{ ft}^2)(85 \text{ psf}) = 2814^k \\ & (2100 \text{ ft}^2)(60 \text{ psf}) = 126^k \\ & (700 \text{ ft}^2)(30 \text{ psf}) = 21^k \end{aligned}$$

$W_T = 16,200^k$

$$V = C_S W_T$$

$$V = (0.020)(16{,}200^k)$$

$$V = 324^k \gg V_{DESIGN} = 261^k$$

* NOTE : THE ASSUMED DEAD LOADS ARE HIGHER THAN WHAT WAS USED BY THE ENGINEER.

I EXPECT THAT ANY MEMBERS DESIGN TO THE $V_{EXP} = 324^k$ WILL APPROXIMATELY 25% OVER THE ACTUAL DESIGN.

SEISMIC FORCE RESISTING SYSTEM (ASCE 7-05)

Bridgeside Point II -- Pittsburgh, PA

Occupancy Category	II
Importance Factor (I)	1.0
S_s	0.125
S_1	0.049
Site Class	D
Total Building Height (feet)	75
T_a	0.765
T_L	12
Fundamental Period (T)	1.30
Frequency (f)	0.769
Structure Behavior	FLEX.
Total Weight (k)	16,200

$S_{ms} = S_s * F_a$	0.2000
$S_{m1} = S_1 * F_v$	0.1176
$S_{DS} = 2/3*S_{ms}$	0.1333
$S_{D1} = 2/3*S_{m1}$	0.0784
Seismic Design	B
R	3.0
C_s	0.02
k	1.40
Total Shear (k)	324

Base Shear and Overturning Moment Distribution							
Story	h_x (feet)	Floor Load (k)	$h_x^k W_x$	C_{vx}	$F_x = C_{vx}V$	V_x (k)	M_x (ft-k)
Roof	75.00	2960	1248468	0.365	118.2	0.0	8862.0
5	58.75	3310	991834	0.290	93.9	118.2	5514.9
4	44.00	3310	661701	0.193	62.6	212.0	2755.6
3	29.50	3310	378076	0.110	35.8	274.7	1055.6
2	14.75	3310	143264	0.042	13.6	310.4	200.0
1	0	2500	0	0.000	0.0	324.0	0.0
Total	75	16200	3423343	1.000	**324**	324	**18388**

-End of Section-

APPENDIX D: SNOW ANALYSIS

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SNOW LOAD CALC

$P_f = 0.7\, C_e\, C_t\, I\, P_g$

LOCATION : PITTSBURGH , PA

OCCUPANCY : OFFICE / LABORATORY

$C_e = 1.0$

$C_t = 1.0$

$I = 1.0$

$P_g = 30$ PSF

$P_f = (0.7)(1.0)(1.0)(1.0)(30 \text{ PSF})$

$P_f = 21$ PSF

-End of Section-

APPENDIX E: FLOOR SYSTEM ANALYSIS

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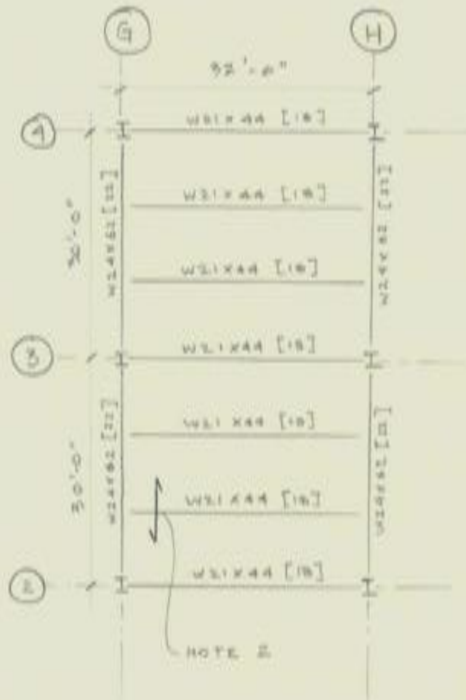
TYPICAL INTERIOR BAY : 2ND FLOOR



LOADS :

LIVE : ASSUMING 100 PSF FOR GENERAL AREA

DEAD : PARTITION = 20 PSF
FIN. & MISC. = 5 PSF
MEP = 10 PSF
SLAB & DECK = 50 PSF
SW BEAMS = 15 PSF

TOTAL DEAD = 100 PSF

NOTE 2 :

3" N.W. CONCRETE
3"-20 GAGE DECK
REINF. W/ 6X6 - W1.4XW1.4 WWF

CHECK : LRFD

1.2 D + 1.6 L

1.4 D

* ORIGINAL DESIGN WAS ASD AND ANY MINOR INCONSISTANT VALUES COULD BE FROM THIS.

SPOT CHECK BEAM

FACTORED LOAD : 1.2 D + 1.6 L

W_u = 1.2 (100 psf) + 1.6 (100 psf) = 280 psf

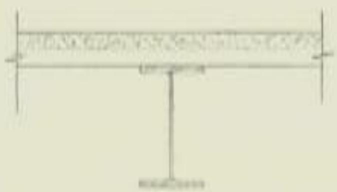
TRIB = 10'-0"

W_u = 2800 plf

$M_u = w_u l^2 / 8$ = (2800 plf)(32'-0")^2 / 8

M_u = 358 ft-k

b_{eff} = MIN | SPACING = 120" | SPAN/4 = 96" ← CONTROLS



$\Delta_{CONSTRUCTION} = \dfrac{5\,w_{CONC.}\,l^4}{384\,E\,I}$

W_{CONC} = 40 psf ⇒ 400 plf

$\Delta_{CONSTR.}$ = $l/360$ = 1.07 in

$I_{MIN} = \dfrac{5\,w_{CONC.}\,l^4}{384\,E\,\Delta_{CONSTR.}}$

$I_{MIN} = \dfrac{(5)(0.40\ kip)(32\ ft)^4(1728\ in^4)}{(384)(29000)(1.07\ in)}$ = 325 in^4

ASSUME $W_{CONC.}$ = 40 psf & W_L = 20 psf ⇒ W_u = 0.8 kip

$M_{u\,MIN}$ = (0.8 kip)(32 ft)^2 / 8 = 102.4 ft-k

NON-COMPOSITE NEEDS TO MEET REQ'D STATE DURING CONSTRUCTION PHASE.

MIN : W18x55 FOR CONSTRUCTION

NOT ACCEPTABLE FOR TOTAL LOAD...

FROM TABLE 3-19

W21X44 $\phi M_n = 558$ ft-k $\geq M_u = 358$ ft-k

$A_s = 13.0$ in^2

ASSUME: $T = C$

$d = 20.6$ in

$T = A_s f_y = (13 \text{ in}^2)(50 \text{ ksi}) = 650^k$

$\Sigma Q_n = 650^k$

$C = 0.85 f'_c b_{eff}\, a$; $\Sigma Q_n = C$ SOLVE FOR a

$a = \Sigma Q_n / 0.85 f'_c b_{eff}$

$a = 650^k / (0.85)(4 \text{ ksi})(96 \text{ in})$

$a = 2.0$ in

$Y_2 = 6.0 - a/2 = 5$ in

@ $Y_2 = 5''$, TFL $\phi M_n = 746$ ft-k

$\phi M_n = 746$ ft-k > 358 ft-k ✓ OK

CHECK DEFLECTION:

$$\Delta_L = l/360 = 1.07 \text{ in} = \frac{5 w_u l^4}{384 E I_L}$$

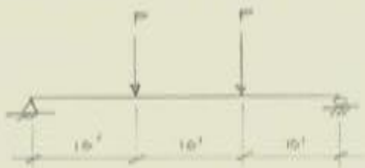
IF $Y_2 = 5 \rightarrow I_L = 2370$ in^4

$$\frac{(5)(1.6 \text{ k/ft})(32 \text{ ft})^4 (1728)}{(384)(29000)(2370 \text{ in}^4)} = 0.55 \text{ in} < 1.07 \checkmark \text{OK}$$

NUMBER OF SHEAR STUDS PER SIDE: 32 STUDS

THE DESIGN CALLS FOR 15 STUDS WHICH TELLS ME $T \neq C$ AND MY DERIVED VALUES ARE OFF. HOWEVER I STILL FEEL THE DESIGN IS OK BECAUSE $\phi M_n \geq M_u$

SPOT CHECK GIRDER



LOADS: $W_u = 2800$ plf

$$P = \frac{(2.8\ k/ft)(32\ ft)}{2}$$

$P = 45^k$ $\quad (P_L = 26^k)$

$M_{MAX} = Pa$

$M_{MAX} = (45^k)(10\ ft) = 450$ ft-k

TRY W24 x 55 $\quad \phi M_n = 503$ ft-k > 450 ft-k

$A = 16.2\ in^2$ $\quad I = 1350\ in^4$

$d = 23.6$ in

$$\Delta = \frac{P l^3}{28 E I} = \frac{(26^k)(30\ ft)^3\ (1728)}{(28)(29000)(1350\ in^4)}$$

$\Delta = 1.107$ in $> \Delta_{ALLOW}$ NOT ACCEPTABLE

TRY W24 x 62 $\quad \phi M_n = 574$ ft-k > 450 ft-k

$A = 18.2\ in^2$ $\quad I = 1550\ in^4$

$d = 23.7$ in

$$\Delta = \frac{(26^k)(30\ ft)^3\ (1728)}{(28)(29000)(1550\ in^4)}$$

$\Delta = 0.964$ in $< \Delta_{ALLOW} = 1.0$ in ✓ OK

IF $T = C$ $\quad \Sigma Q_n = 910^k \longrightarrow$ 44 STUDS PER SIDE

DESIGNER USED 22 STUDS WHICH TELLS ME $T = C$ IS NOT VALID AND PNA_6 (BFL) CONTROLS. DESIGN IS JUSTIFIED BECAUSE $\phi M_n > M_u$ AND $\Delta < \Delta_{ALLOW}$.

COLUMN SPOT CHECK : G-2

INTERIOR GRAVITY COLUMN
NO LATERAL LOAD

COLUMN IS SPLICED @ 3RD – 4TH FLOOR

LOADS : (SEE CALCS IN SEISMIC SECTION)

ROOF = 85 PSF
TYP. FLOOR = 100 PSF
PENTHOUSE = 60 PSF
$\}$ DEAD

ROOF SNOW = 21 PSF
TYP. FLOOR = 100 PSF
PENTHOUSE / ROOF = 150 PSF
$\}$ LIVE

$A_I = (64\ ft)(60\ ft) = 3840\ ft^2$
$A_T = 960\ ft^2$
TYP. FLOOR & ROOF

PENTHOUSE COLUMN : ANALYZE F-2 : LARGER TRIB

$A_I = 758\ ft^2$ (CORNER COLUMN) ; $A_T = 379\ ft^2$

$L = 12.75\ ft$

LOAD : $1.2(60\ psf) + 1.6(21\ psf) = 106\ psf$

$P_u = 40.2^k$; $KL = 12.75\ ft$

ANY W8 OR LARGER IS ACCEPTABLE

DESIGNER USED W12X58 SO NO SPLICE WAS NEEDED

ROOF COLUMN:

$A_I = 3840\ ft^2 \qquad A_T = 960\ ft^2 \qquad KL = 16.25\ ft$

NO LIVE REDUCTION

ASSUME MECHANICAL LOAD FOR LIVE LOAD

LOAD: $1.2(85\ psf) + 1.6(150\ psf) + 0.5(21\ psf)$

$= 353\ psf$

$P_u = 339^k + 40.2^k = 379^k$

W12X53 OR W10X49 $\phi P_n = 452\ k$ & $428\ k$ (RESPECTIVELY)

DESIGNER USED W12X53 ✓ OK

5th FLOOR COLUMN:

$A_I = 3840\ ft^2 \qquad A_T = 960\ ft^2 \qquad KL = 14.75\ ft$

$L = L_o\left(0.25 + 15/\sqrt{A_I}\right) = L_o\left(0.25 + 15/\sqrt{3840\ ft^2}\right)$

$L = 0.492\ L_o > 0.4 L_o$ ✓ OK

LOAD: $(1.2)(100\ psf) + (1.6)(0.492)(100\ psf) = 200\ psf$

$P_u = 192^k + 353^k = 545^k$

W12x65 $\phi P_n = 662^k > 545^k$ ✓ OK

DESIGNER USED W12X53

MY FLOOR LOADS ARE TOO LARGE, AND HAVE BEEN CONSISTANTLY TOO LARGE. STREAMLINING THE DEAD LOADS WOULD ALLOW ME TO MATCH SIZES. MY ASSUMPTIONS WERE CONSERVATIVE WHICH OK.

4TH FLOOR COLUMN :

$A_I = 3840\ ft^2$ $\quad A_T = 960\ ft^2$ $\quad KL = 14.5\ ft$

$L = L_o\left(0.25 + 15/\sqrt{3840}\right)$

$L = 0.492\ L_o$

LOAD : SAME AS 5TH = 200 PSF

$P_u = 192^k + 545^k = 737^k$

W12X72 $\quad \phi P_n = 748^k > 737^k$ ✓ OK

DESIGNER USED W12X87 ✓ OK

3RD FLOOR COLUMN :

SAME AS 4TH FLOOR ; EXCEPT $KL = 14.75\ ft$

$P_u = 192^k + 737^k = 929^k$

W12X96 $\quad \phi P_n = 997^k > 929^k$ ✓ OK

DESIGNER USED W12X87 (I AM CONSERVATIVE)

2ND FLOOR COLUMN :

$A_I = 3840\ ft^2$ $\quad A_T = 960$ $\quad KL = 14.75\ ft$

NO LIVE LOAD REDUCTION ALLOWED

LOADS : $1.2(100\ psf) + 1.6(100\ psf) = 280\ psf$

$P_u = 269^k + 929^k = 1198^k$

W12X120 $\quad \phi P_n = 1260^k > 1198^k$ ✓ OK

DESIGNER USED W12X87

AGAIN I AM OVER BY ROUGHLY 3 SIZE WHICH IS CONSERVATIVE & CONSISTANT. SINCE I AM CONSERVATIVE MY DESIGN IS OK.

-End of Section-

APPENDIX F: LATERAL SYSTEM ANALYSIS

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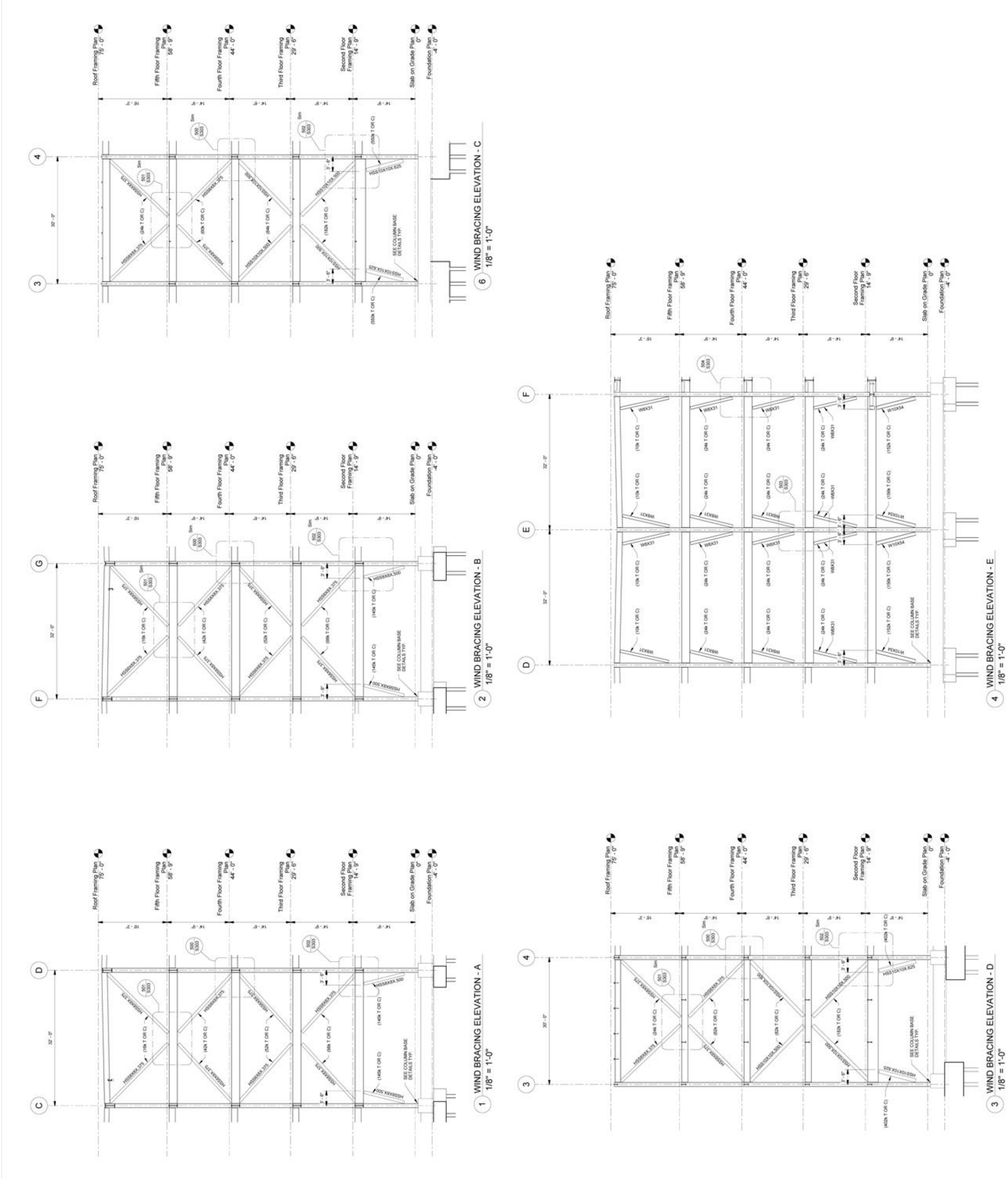
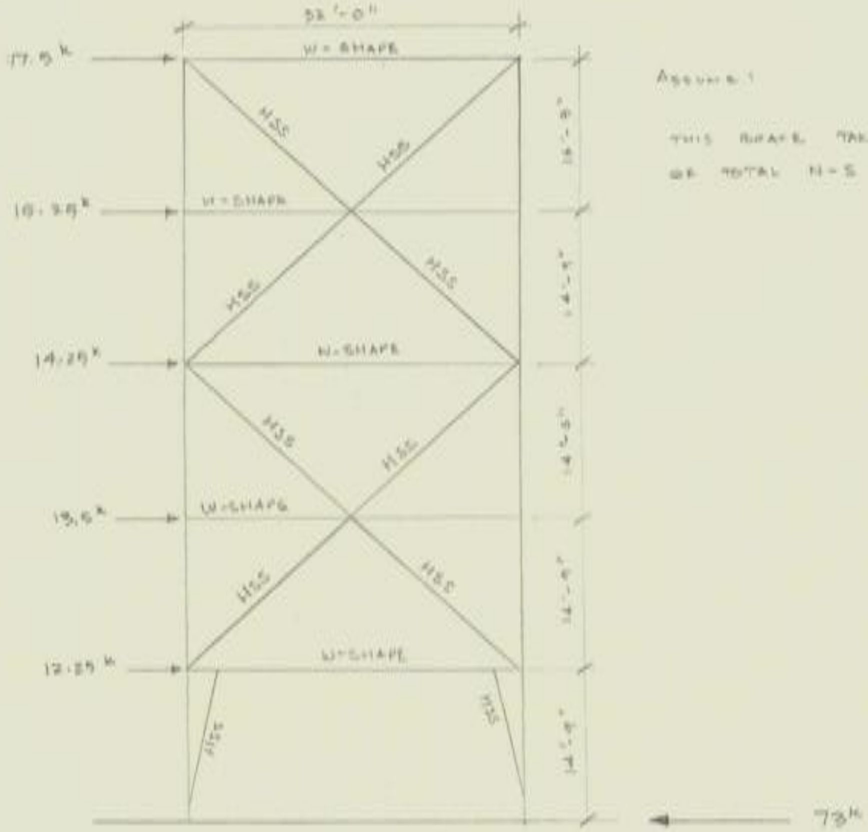
Lateral Bracing System



LATERAL SPOT CHECK : FRAME A (LINE 6 - C & D)



ASSUME:

THIS BRACE TAKES 1/4 OF TOTAL N-S WIND

ROOF

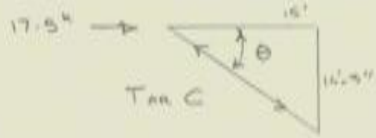
$$T \text{ or } C = \frac{17.5^k}{\cos\theta} \qquad \theta = 45.44°$$

$$T \text{ or } C = 25^k$$

DESIGNER SIZED FOR 16^k T or C

MY SIMPLE DISTRIBUTION OF LOADS COULD BE WRONG, AS THE STIFFNESS COULD VARY AMONG BRACES. THIS WILL BE INVESTIGATED FURTHER IN A FUTURE TECHNICAL REPORT.

A QUICK SPOT CHECK OF THE TWO FRAMES IN THE OTHER DIRECTION:

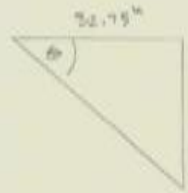


$$T \text{ or } C = \frac{17.5^k}{\cos\theta} \qquad \theta = 47.3°$$

$$T \text{ or } C = 25.8^k$$

DESIGNER SIZED FOR 24^k T or C

5TH



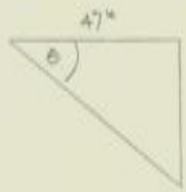
$\theta = 42.67°$

$T \text{ OR } C = 32.75^k / \cos\theta$

$T \text{ OR } C = 44.5^k$

DESIGNER SIZED FOR 42^k WHICH IS VERY CLOSE TO MY VALUE

4TH



$\theta = 42.18°$

$T \text{ OR } C = 47^k / \cos\theta$

$T \text{ OR } C = 63^k$

DESIGNER SIZED FOR 52^k ; I AM CONSERVATIVE

3RD

$\theta = 42.67°$

$T \text{ OR } C = 60.5^k / \cos\theta$

$T \text{ OR } C = 82.3^k$

DESIGNER SIZED FOR 78^k ; I AM ASSUMING MY WIND PRESSURE AT THIS LEVEL IS NOT CORRECT. FURTHER ANALYSIS SHALL BE CONDUCTED AT A LATER DATE.

2ND

$\theta = 16.26°$

$T \text{ OR } C = \left(73^k / \sin\theta \right) / 2$

$T \text{ OR } C = 130.4^k$

DESIGNER SIZED FOR 145^k ; MY ASSUMPTION OF 12'-0" COULD BE INCORRECT OR MY LOAD DISTRIBUTION MAY NOT CORRECT BECAUSE OF OVER - SIMPLIFICATION OF STIFFNESS AND IRREGULAR BUILDING LAYOUT.

-End of Section-