TECHNICAL REPORT 3

BRIDGESIDE POINT II

PITTSBURGH, PA

Antonio DeSantis Verne

Structural Option

Advisor: M. K. Parfitt

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TABLE OF CONTENTS

EXECUTIVE SUMMARY

In this third technical report detailed lateral analysis for Bridgeside Point II is investigated through RAM Structural System and hand calculations. Composite steel framing constitutes the gravity system, while a braced frame handles the lateral forces. This report takes an in-depth look at the seismic and wind forces present via two methods. The first method is a combination of basic computer analysis and hand calculations, which was then compared against a sophisticated computer analysis. The methods were compared to verify accuracy and expose any potential weaknesses with an analytical method.

The results verify that wind forces control the lateral design. The computed story and base shears were similar to design values, which substantiated the analytical procedure. Using those loads, strength and serviceability checks were performed to validate member size and confirm that the members were within acceptable code limits. Spot checks revealed that drift governed member design; however, it was found that the second story was less stiff than the remaining floors due the bracing configuration at that location. While the building meets code provisions for strength and drift, the results suggest that a more economical framing scheme could reduce both story drift and member size. This topic will be presented in detail in the proposal.

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 of 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 (Figure 1). 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 on 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. Also placement of the lateral system is limited. The lateral system is placed roughly at the building side's midpoints.

This report reviews and discusses the results of a detailed lateral analysis of Bridgeside Point II. Lateral analysis took advantage of RAM Structural System and SAP 2000 as well as hand calculations as noted in their respective sections. Spot checks were performed on several members to ensure the computer designed members were accurate for both strength and drift control.

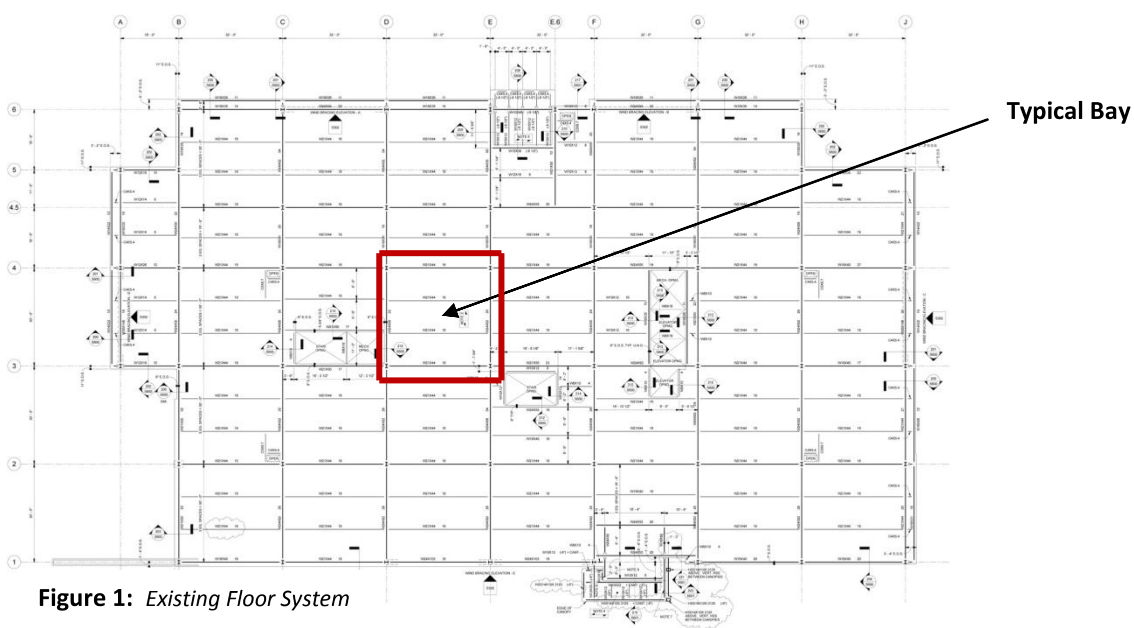


Figure 1: *Existing Floor System*

EXISTING COMPOSITE STEEL SYSTEM

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" composite steel decking. Shear studs ¾" diameter (5 ½" long) are used to create composite action. This assembly provides a 1.5 to 2 hour fire rating which meets IBC requirements. Infill beams are W21x44 spaced at 10'-0" center to center which frame into W24x62 girders. This report will not cover floors systems, for more information please reference Technical Report Two.

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 (Figure 3). 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 (Figure 2). 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.

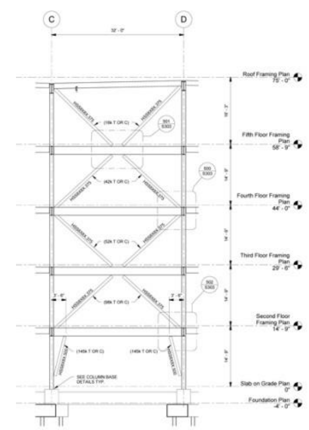


Figure 2: *Typical Lateral Frame*

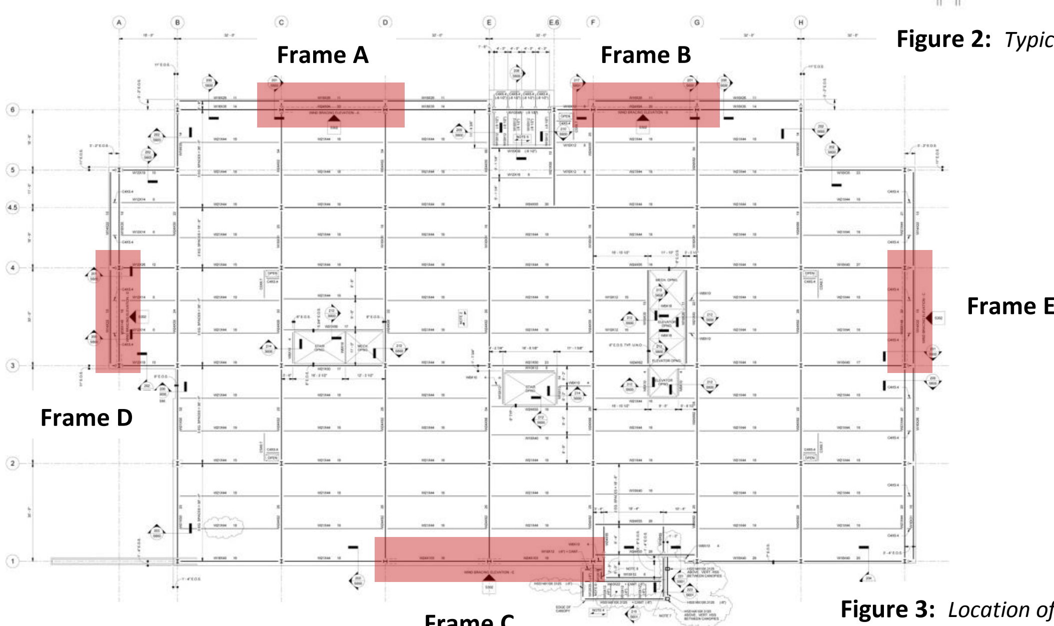


Figure 3: *Location of Lateral Bracing*

Foundations

A driven pile system with pile caps containing between two and nine piles provides the foundation system for the building with an end bearing capacity of 105 to 130 tons per pile. 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 12" thick grade beams. The main floor is a 4" concrete slab on grade. Soil conditions are from the geotechnical report provided by Professional Service Industries, Inc. dated May 2007.

CODES AND LOAD COMBINATIONS

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.

Steel Construction Manual, Thirteenth Edition, American Institute of Steel Construction.

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

This report will use Load and Resistance Factor Design for all steel design checks.

Deflection Criteria per 2006 International Building Code

Δ_{WIND} = H/400 Allowable Building Drift

$\Delta_{SEISMIC} = 0.025h_{SX}$ Allowable Story Drift

Load Cases and Combinations per 2006 International Building Code

The following are the load cases considered for this analysis per 2006 IBC, Section 1605:

1.4(Dead)
1.2(Dead) + 1.6(Live) + 0.5(Roof Live)
1.2(Dead) + 1.6(Roof Live) + (1.0 Live or 0.8 Wind)
1.2(Dead) + 1.6(Wind) + 1.0(Live) + 0.5(Roof Live)
1.2(Dead) + 1.0(Seismic) + 1.0(Live)
0.9(Dead) + 1.6(Wind)
0.9(Dead) + 1.0(Seismic)

Different load cases and combinations were applied in various directions and with varying eccentricities to the wind and seismic loads in the computer analysis. The total combinations generated for LRFD were 313. It should be noted that snow loads were not included in this analysis. A detailed listing of the load cases and combinations used are available upon request.

LATERAL RESISTING SYSTEM

Building Isometrics

As previously mentioned, Bridgeside Point II is host to five eccentrically and concentrically braced frames. They extend the full height of the building and located roughly at the midpoint of each side of the building (Figures 4 & 5).

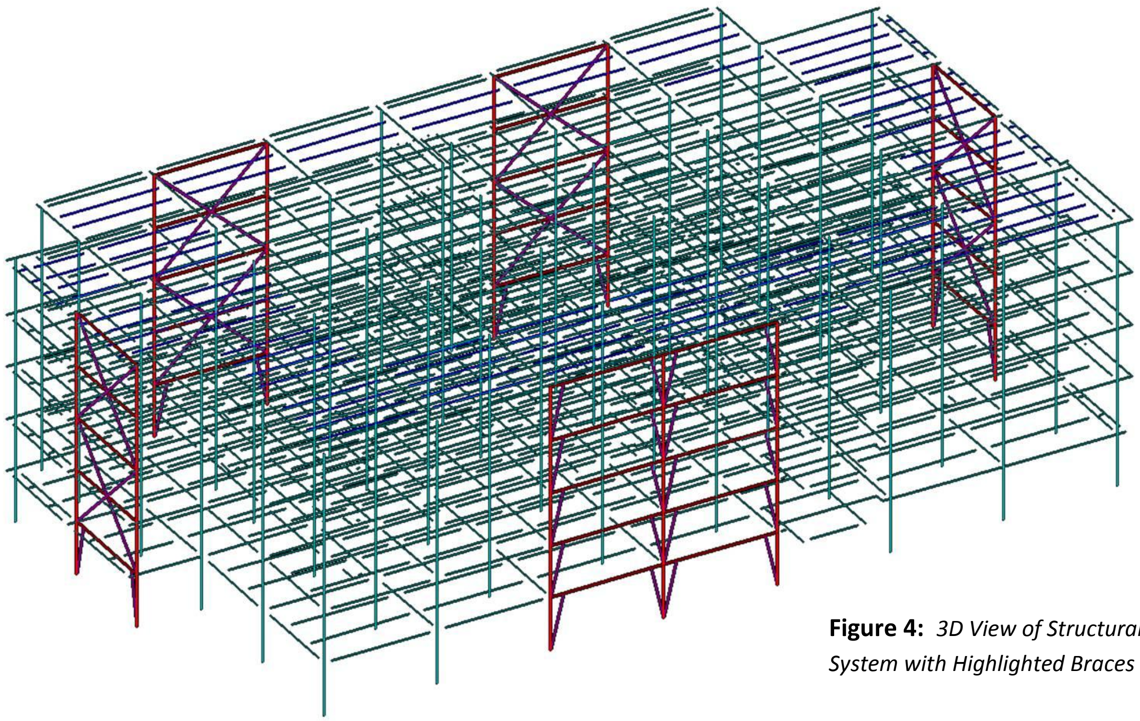


Figure 4: *3D View of Structural System with Highlighted Braces*

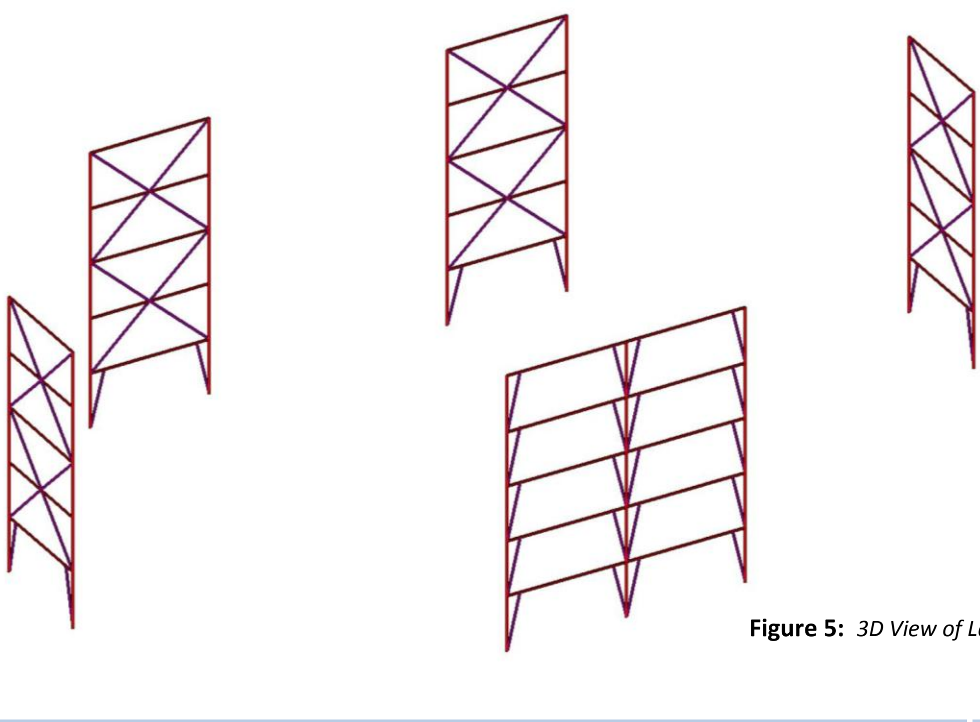


Figure 5: *3D View of Lateral Braces*

Building Loads

In order to find story shears and member strengths, design dead loads (Table 1) and live loads (Table 2) need to be introduced. In Technical Report One the design dead loads used for preliminary analysis were larger than what was used by the design engineer. It should be noted the following design dead loads are revisited and streamlined to closer emulate those used by the design engineer. All loads were referenced from the Steel Construction Manual (13^{th} Edition) and in conjunction with the 2006 International Building Code. All lateral forces were revisited and corrections were made ensure more accurate forces.

Dead Loads					
Typical Floor Loads (psf)		Roof Loads (psf)		Penthouse Loads (psf)	
Partitions	10	M.E.P.	5	M.E.P.	5
Finishes	3	Slab & Deck	50	Slab & Deck	25
M.E.P.	5	Structural Steel	10	Structural Steel	10
Slab & Deck	57	Misc.	5	Misc.	5
Structural Steel	15	--	-	--	-
Total	**90**	**Total**	**70**	**Total**	**45**

Table 1: *Dead Loads*

Live Loads	
Building Space	Load (psf)
Public Areas	100
Lobbies	100
First Floor Corridors	100
Corridors Above First Floor	80
Office	50
Light Storage	125
Mechanical	150
Stairs	100

Table 2: *Live Loads*

Wind Criteria

Wind loads were analyzed using Section 6.5 of ASCE 7-05. Below are the assumptions used to aid in the determination of the Main Wind-Force Resisting System. For a detailed layout of the corrected calculations please refer to Appendix B.

Basic Wind Speed ***V***……………………………………………… 90 mph
Exposure Category……………………………………………… C
Importance Factor……………………………………………… 1.0
Building Category……………………………………………… II
Internal Pressure Coefficient ***GCpi***……………………… +/- 0.18

Seismic Criteria

Seismic loads were analyzed using chapters 11 and 12 of ASCE 7-05. Below are the assumptions used to aid in the determination of the Seismic Force Resisting System. For a detailed layout of the corrected calculations please refer to Appendix B.

Seismic Use Group………………………………………………… II
Importance Factor………………………………………………… 1.0
Spectral Response Accelerations
S_s…………………………………………………………………… 0.125
S_1…………………………………………………………………… 0.049
Site Class…………………………………………………………… D
Site Class Factors
F_a…………………………………………………………………… 1.6
F_v…………………………………………………………………… 2.4
S_{MS}……………………………………………………………… 0.20
S_{M1}……………………………………………………………… 0.1176
S_{DS}……………………………………………………………… 0.133
S_{D1}……………………………………………………………… 0.078
Seismic Design Category……………………………………… B
Response Modification Factor……………………………… 3.0
(Ordinary Composite Steel & Concrete Braced Frame)
Seismic Period Coefficient (C_t)……………………………… 0.03
Seismic Period Coefficient (C_s)……………………………… 0.02
Period Coefficient (x)…………………………………………… 0.75

-End of Section-

LOAD DISTRIBUTION AND ANALYSIS

Distribution of lateral forces is based on frame relative stiffness. The thick composite deck and slab are treated as a rigid diaphragm, and as such allocate load to each frame with respect to their stiffness. For this analysis two computer programs were used to better understand this distribution. For simplicity, the intersection of column lines 1 and A was used y = 0 and x = 0, respectively.

SAP 2000 with Hand Calculations

A determination of each frame's relative stiffness was done using SAP 2000. For simplicity, a one kip load was applied at a specified floor. The relative stiffness of the specified floor was determined by taking the inverse of the measured deflection. This procedure was repeated at each floor and for each frame. Using the stiffness of each frame, a determination could be made as to how much story shear each frame experienced. The lateral force system is controlled by the wind forces, and as such, wind forces will be further analyzed for torsion effects. A simple estimation of the center of rigidity was conducted, and wind story shears were applied at this point inducing torsion, which introduced additional shear (Table 3).

Story	Frame Stiffness (kip/in)					Centers of Rigidity		Story Shear (kips)		Eccentricity		Torsional Rigidity
	A	B	C	D	E	X_R (ft)	Y_R (ft)	N-S	E-W	e_x (ft)	e_y (ft)	J (k*ft/in)
Roof	95	95	15	130	128	120.06	128.37	0	0	4.16	57.53	4,110,016
5th	114	114	28	165	165	121.00	123.35	55	103	3.22	52.51	5,352,735
4th	137	137	43	217	214	120.16	119.71	106	201	4.06	48.87	7,050,486
3rd	152	152	88	231	233	121.52	107.41	154	293	2.70	36.57	8,106,183
2nd	188	188	335	274	275	121.22	73.24	200	382	3.00	2.41	12,418,775
Total	137	137	102	203	203	120.79	110.42	242	464	3.43	39.58	7,407,639

Story	Direct Shear (kips)					Torsional Shear (kips)					Total Shear (kips)				
	A	B	C	D	E	A	B	C	D	E	A	B	C	D	E
Roof	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
5th	24.5	24.5	6.0	51.5	51.5	0.9	0.9	1.9	1.2	1.2	25.4	25.4	7.9	52.7	52.7
4th	45.8	45.8	14.4	101.2	99.8	1.9	1.9	3.8	3.0	3.0	47.7	47.7	18.2	104.2	102.8
3rd	59.7	59.7	34.6	145.9	147.1	3.3	3.3	6.6	2.7	2.7	63.0	63.0	41.1	148.6	149.9
2nd	52.9	52.9	94.2	190.7	191.3	0.5	0.5	1.0	3.1	3.1	53.4	53.4	95.2	193.7	194.4
Total	88.3	88.3	65.5	232.2	231.8	5.0	5.0	14.5	5.3	5.3	93.2	93.2	80.0	237.5	237.1
											266.5			**474.6**	

Table 3: *Center of Rigidity and Resulting Shears due to Wind Loads*

It should be noted that this method ignored all slab openings and any small variations in building geometry. Drift was not checked using SAP 2000 or via hand calculations.

RAM Structural System

Ram Structural System was used to produce a very detailed lateral analysis. More accurate centers of rigidity and mass were determined for each floor as small variations in building shape and slab openings were included. The values obtained in RAM are acceptable based on what was found with SAP 2000 and hand calculations (Table 4).

Story	Hand Calculated Output				RAM Structural System Output			
	Centers of Mass		Centers of Rigidity		Centers of Mass		Centers of Rigidity	
	X_R (ft)	Y_R (ft)	X_R (ft)	Y_R (ft)	X_R (ft)	Y_R (ft)	X_R (ft)	Y_R (ft)
Roof	124.22	70.84	120.06	128.37	127.97	69.48	120.29	104.05
5th	124.22	70.84	121.00	123.35	124.29	69.20	119.18	93.42
4th	124.22	70.84	120.16	119.71	124.29	69.20	119.21	84.14
3rd	124.22	70.84	121.22	73.24	124.27	69.19	118.39	75.92
2nd	124.22	70.84	120.79	110.42	124.78	68.48	117.88	67.39

Table 4: *Comparisons of Hand Calculated and RAM Output*

This computer model also determined story shears for seismic and wind forces at various orientations with respect to the building. The story shears obtained from RAM were similar to hand calculations with major differences appearing at the roof level. The RAM model has the loads in the proper positions and makes a more precise distribution of those loads; therefore, the north-south frames experience more load than what was calculated by hand. All other floors were very close in magnitude when compared. Base shears for seismic and wind forces were also comparable to hand calculations (Table 5 on next page). The hand calculations are conservative, which is acceptable for analysis. As mentioned previously, the hand calculations ignored any slab openings and variations in building geometry, but they do in fact play a role in building shears. It should be noted that the RAM Model confirmed wind lateral forces control design and are within ten percent of the hand calculations. The seismic lateral forces are higher than those specified in the hand calculations and general notes of the project; however, this analysis included all loads and member weight, which increased the total shear.

-Table on Next Page-

Story	Wind E-W Story Shears (kips)		
	Hand Calculation	RAM Calculation	Percent Difference
Roof	103.43	51.70	-100.1%
5th	97.19	100.45	3.2%
4th	92.13	95.97	4.0%
3rd	89.21	91.51	2.5%
2nd	82.47	85.16	3.2%
Total Base Shear (kips)	**464**	**425**	-9.3%
Overturning Moment (ft-k)	**21,266**	**17,955**	-18.4%

Story	Seismic E-W Story Shears (kips)		
	Hand Calculation	RAM Calculation	Percent Difference
Roof	90.24	38.77	-132.8%
5th	82.43	75.34	-9.4%
4th	54.99	71.98	23.6%
3rd	31.42	68.63	54.2%
2nd	11.90	63.87	81.4%
Total Base Shear (kips)	**271**	**319**	14.9%
Overturning Moment (ft-k)	**15,134**	**13,466**	-12.4%

Table 5: *Comparisons of Hand Calculations and RAM Output of Shears*

Serviceability Check

Limitations for seismic and wind drift were compared with the drift values determined by RAM Frame. Wind drift was compared against Δ_{WIND} = H/400 for the entire building (Table 6). Seismic drift was compared against $\Delta_{SEISMIC} = 0.025h_{SX}$ at each floor (Table 7).

Controlling Wind Drift										
Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift (in) Δ_{WIND} = H/400			Total Drift (in)	Allowable Total Drift (in) Δ_{WIND} = H/400		
Roof	74.00	0.040	<	0.458	Acceptable	2.080	<	2.220	Acceptable
5th	58.75	0.065	<	0.443	Acceptable	2.040	>	1.763	Unacceptable
4th	44.00	0.069	<	0.435	Acceptable	1.974	>	1.320	Unacceptable
3rd	29.50	0.103	<	0.443	Acceptable	1.905	>	0.885	Unacceptable
2nd	14.75	1.802	>	0.443	Unacceptable	1.802	>	0.443	Unacceptable

Table 6: *Computed Wind Drift Compared to Code Limit*

Controlling Seismic Drift										
Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift (in) $\Delta_{SEISMIC} = 0.025H_{SX}$			Total Drift (in)	Allowable Total Drift (in) $\Delta_{SEISMIC} = 0.025H_{SX}$			
Roof	74.00	0.044	<	0.381	Acceptable	1.087	<	1.850	Acceptable
5th	58.75	0.061	<	0.369	Acceptable	1.042	<	1.469	Acceptable
4th	44.00	0.057	<	0.363	Acceptable	0.982	<	1.100	Acceptable
3rd	29.50	0.058	<	0.369	Acceptable	0.924	>	0.738	Unacceptable
2nd	14.75	0.866	>	0.369	Unacceptable	0.866	>	0.369	Unacceptable

Table 7: *Computed Seismic Drift Compared to Code Limit*

As illustrated in the charts, the overall drift is acceptable; however, several individual floors exceed the allowable drift limit. The reason stems from the eccentric braces used on the first floor of each frame, which results in oversized members on the third through roof stories in order to compensate for the unacceptable level of drift. According to these calculations, the second story resembles a soft story, which means that it is more flexible then the surrounding stories and will move much more than those stories. Reduction of drift could be achieved several ways. One method would be to increase the size of beam at that level. Another possibility would be fixing the column to the foundation creating a very rigid connection. A third way would be to relocate the eccentric braces by extending them towards the center of the beam. Regardless of the solution, this issue of excessive story drift will be commented on further in the proposal later this semester and in the spring semester.

Overturning Check

The wind forces produce an overturning moment greater than 21,000 foot – kips which needs to be resisted by the foundation system. As previously described, the foundation system is a pile driven system with a depth of 55 feet. By inspection, overturning will be an issue as the combination of foundation depth and building mass will resolve uplift forces.

-End of Section-

MEMBER VERIFICATION

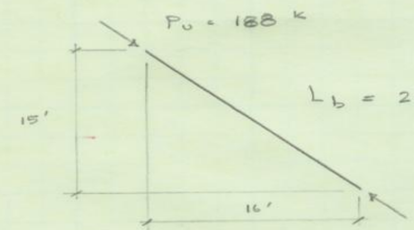
Member size design was completed by RAM Structural; however, it is necessary to verify the RAM output regarding member strength. Frame members in the east-west direction were checked using their respective controlling loads.

Bracing Members

STRENGTH CHECKS

LATERAL BRACE MEMBERS

HSS 8 x 8 x 3/8



$F_y = 46$ ksi

$A_g = 10.4 \text{ in}^2$

DESIGN CHECK (LRFD)

$$\frac{KL_x}{r} = \frac{(1.0)(21.93\text{ ft})(12\text{ in/ft})}{(3.1\text{ in})} = 84.9 < 200 \checkmark \text{ OK}$$

$$\text{LIMIT STATE}: (4.71)\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29000\text{ ksi}}{46\text{ ksi}}} = 118.26$$

$84.9 < 118.3 \therefore$ INELASTIC

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{(\pi^2)(29000\text{ ksi})}{(84.9)^2} = 39.7\text{ ksi}$$

$$F_{cr} = \left[0.658^{(F_y/F_e)}\right] F_y = \left[(0.658)^{(46\text{ ksi})/(39.7\text{ ksi})}\right](46\text{ ksi})$$

$F_{cr} = 28.3$ ksi

$$\phi P_n = 0.9 A_g F_{cr} = (0.9)(10.4\text{ in}^2)(28.3\text{ ksi})$$

$\phi P_n = 265^k > P_u = 188^k$ ✓ OK

COMPARE TO STEEL MANUAL : TABLE 4-4

$KL = 21.93$ ft

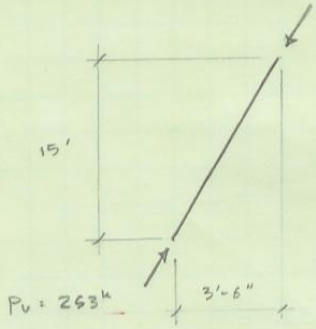
$\phi P_n = 264^k$ ✓ OK

STRESS RATIO :

$$\frac{P_u}{\phi P_n} = \frac{188^k}{265^k} = 0.712 < 1.0 \checkmark \text{ OK}$$

* FUTURE DESIGN TO BE CHECKED ONLY WITH STEEL MANUAL

HSS 8 x 8 x 1/2



$L_b = 15.4$ ft

$F_y = 46$ ksi

DESIGN CHECK :

$$\frac{KL}{r} = \frac{(1.0)(15.4\text{ ft})(12\text{ in}/\text{ft})}{3.04\text{ in}} = 60.8 < 200 \quad \checkmark \text{ ok}$$

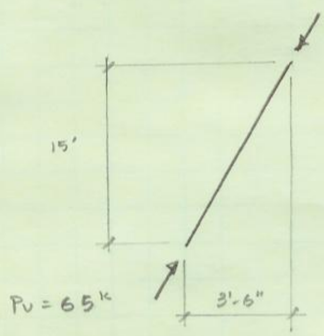
FROM STEEL MANUAL : TABLE 4-4

$\phi P_n = 434^k > P_u = 263^k$ ✓ ok

STRESS RATIO :

$$\frac{P_u}{\phi P_n} = \frac{263^k}{434^k} = 0.61 < 1.0 \quad \checkmark \text{ ok}$$

W 8 x 31



$L_b = 15.4$ ft

$F_y = 50$ ksi

$$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29000\text{ ksi}}{50\text{ ksi}}} = 113.4$$

$$\frac{KL_x}{r_x} = \frac{(1.0)(15.4\text{ ft})(12\text{ in}/\text{ft})}{3.47\text{ in}} = 53.3$$

$$\frac{KL_y}{r_y} = \frac{(1.0)(15.4\text{ ft})(12\text{ in}/\text{ft})}{2.02\text{ in}} = 91.5 < 200 \quad \checkmark \text{ ok}$$

CONTROLS

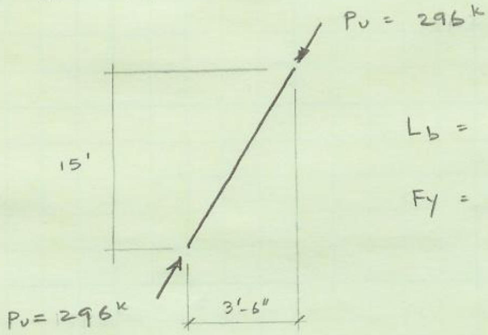
FROM STEEL MANUAL : TABLE 4-4

$\phi P_n = 223^k \; > \; P_u = 65^k$ ✓ OK

STRESS RATIO :

$$\frac{P_u}{\phi P_n} = \frac{56^k}{223^k} = 0.29 < 1.0 \quad ✓\ \text{OK}$$

W10X54



$L_b = 15.4$ ft

$F_y = 50$ ksi

$$\frac{KL_x}{r_x} = \frac{(1.0)(15.4\ \text{ft})(12\ \text{in}/\text{ft})}{(4.37\ \text{in})} = 42.3$$

$$\frac{KL_y}{r_y} = \frac{(1.0)(15.4\ \text{ft})(12\ \text{in}/\text{ft})}{(2.56\ \text{in})} = 72.2 < 200 \quad ✓\ \text{OK}$$

CONTROLS

FROM STEEL MANUAL : TABLE 4-4

$\phi P_n = 486^k \; > \; P_u = 296^k$ ✓ OK

STRESS RATIO :

$$\frac{P_u}{\phi P_n} = \frac{296^k}{486^k} = 0.61 < 1.0 \quad ✓\ \text{OK}$$

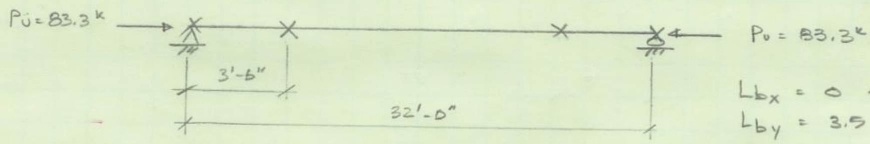
– END BRACE CHECK –

Beam Members

LATERAL BEAM MEMBER

W24x94

$M_x = 537$ ft-k $\quad M_y = 19.5$ ft-k $\quad V_u = 198^k$



$L_{bx} = 0$ ft

$L_{by} = 3.5$ ft

$\frac{KL_x}{r_x}$ = FULLY BRACED

$$\frac{KL_y}{r_y} = \frac{(1.0)(3.5\text{ ft})(12\text{ in}/\text{ft})}{1.98\text{ in}} = 21.2 < 200 \quad \checkmark \text{ OK}$$

SHEAR CHECK

$\phi V_n = \phi\, 0.6 F_y\, d\, t_w$

$\phi V_n = (1.0)(0.6)(50\text{ ksi})(24.3\text{ in})(0.515\text{ in}) = 375^k$

$\phi V_n = 375^k > V_u = 198^k \quad \checkmark$ OK

AXIAL CHECK

$L_b = 3.5$ ft

FROM TABLE 6-1 :

$p = 0.881 \times 10^{-3}$ (kips^{-1})

$pP_u = (0.881 \times 10^{-3})(83.3^k) = 0.073 < 1.0 \quad \checkmark$ OK

H1-1a GOVERNS

BENDING CHECK

$L_b = 3.5$ ft

FROM TABLE

$b_x = 0.933 \times 10^{-3}$ (k-ft)$^{-1}$ $\quad b_y = 6.32 \times 10^{-3}$ (k-ft)$^{-1}$

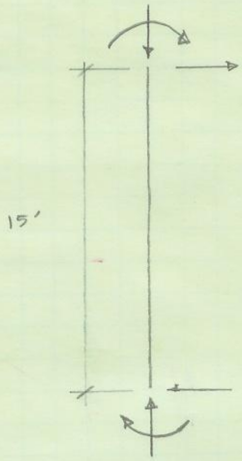
$0.073 + (0.933 \times 10^{-3})(537) + (6.32 \times 10^{-3})(19.5)$

$0.697 < 1.0 \quad \checkmark$ OK

BEAM IS ADEQUATE

Column Members

LATERAL COLUMN MEMBER



$P_u = 326^k$
$V_u = 18.3^k$
$M_x = 31.25$ ft-k
$M_y = 29.05$ ft-k

W14x132

$P_u = 326^k$
$V_u = 18.3^k$
$M_x = 177.04$ ft-k
$M_y = -55.52$ ft-k

$$\frac{KL_y}{r_y} = \frac{(1.0)(15\text{ ft})(12\text{ in/ft})}{3.76\text{ in}} = 47.9 < 200 \quad \checkmark \text{ ok}$$

SHEAR CHECK :

$$\phi V_n = \phi_v 0.6 F_y d t_w$$

$$\phi V_n = (1.0)(0.6)(50\text{ ksi})(14.7\text{ in})(0.645\text{ in}) = 284^k$$

$$\phi V_n = 284^k > V_u = 18.3^k \quad \checkmark \underline{\text{ok}}$$

COMBINED LOADING CHECK

$$B_1 = C_m / 1 - \alpha P_r / P_{e_1} \geq 1.0$$

$$C_{m_x} = 0.6 - 0.4\left(\frac{31.25}{+177.04}\right) = 0.53$$

$$C_{m_y} = 0.6 - 0.4\left(\frac{29.05}{-55.52}\right) = 0.81$$

$$P_{e_1} = \frac{\pi^2 EI}{(KL)^2}$$

$$P_{e_{1x}} = \frac{(\pi^2)(29000\text{ ksi})(1530\text{ in}^4)}{[(15\text{ ft})(12\text{ in/ft})]^2} = 13{,}516^k$$

$$P_{e_{1y}} = \frac{(\pi^2)(29000\text{ ksi})(548\text{ in}^4)}{[(180\text{ in})]^2} = 4841^k$$

$B_{1x} = 1.0$ (BY INSPECTION)

$B_{1y} = 1.0$ (BY INSPECTION)

$$B_2 = \frac{1}{1 - \frac{\alpha \Sigma P_{nt}}{\Sigma P_{e2}}} \geq 1.0$$

$\Delta H_x = 0.701$ IN

$\Delta H_y = 1.80$ IN

$$\Sigma P_{e2} = R_m \frac{\Sigma HL}{\Delta_H} \quad ; \quad R_m = 1.0$$

$$\Sigma P_{e2_x} = (1.0) \frac{(49.42^k)(180\text{IN})}{(0.701\text{ IN})} = 12690^k$$

$$\Sigma P_{e2_y} = (1.0) \frac{(85.2^k)(180\text{ IN})}{(1.8\text{ IN})} = 8520^k$$

$$B_{2x} = \frac{1}{1 - (246)/(12690)} = 1.02$$

$$B_{2y} = \frac{1}{1 - (425)/(8520)} = 1.05$$

$$P_r = (1.05)(326^k) = 342^k$$

$$M_{rx} = (1.05)(177\text{ ft-k}) = 186\text{ ft-k}$$

$$M_{ry} = (1.05)(55.52\text{ ft-k}) = 58.3\text{ ft-k}$$

$p = 0.678 \times 10^{-3}$ $\quad b_x = 1.03 \times 10^{-3}$ $\quad b_y = 2.10 \times 10^{-3}$

$$pP_r = (0.678 \times 10^{-3})(342^k) = 0.232 > 0.2 \quad \text{H1-1a}$$

$$0.232 + (1.03 \times 10^{-3})(186) + (2.10 \times 10^{-3})(58.3) = 0.563$$

$$0.563 < 1.0 \quad \checkmark \ \underline{\text{ok}}$$

These calculations verify the computer output and do confirm that most, in the lateral system, are sized for drift rather than strength. The interaction equation for most members is around the 50% percent mark, which is more than adequate for strength. Sizing members for drift is essential for this particular building as the second story drift is nearly two inches, so to limit drift on that floor as well as the others; each member has to be significantly increased to stay within acceptable drift limitations. This brings up the issue of economy of both member selection and frame layout. With a different brace configuration or layout, the member sizes could be reduced without sacrificing serviceability.

CONCLUSION

This report incorporated the use of a computer model and hand calculations to analyze the lateral system of Bridgeside Point II. The hand calculations verified the computer output, however slight inconsistencies were found, namely the conservative story shears via hand calculations. Analysis checked strength and serviceability, and the building was within acceptable code limits. However, the second story did have what could be deemed as an unacceptable amount of deflection, even though the story members have more than adequate strength. The issue lies with the brace configuration on that floor. That particular configuration lends itself less rigidity compared to the upper floors, thus producing a soft story effect.

The design strength of several members was checked and found to be well within their ultimate capacity. Knowing that the second story has drift issues and the remaining stories were sized to significantly reduce drift, the oversized members are rationalized. This then leads to the issue of economy of the lateral system, which will be discussed in detail in the 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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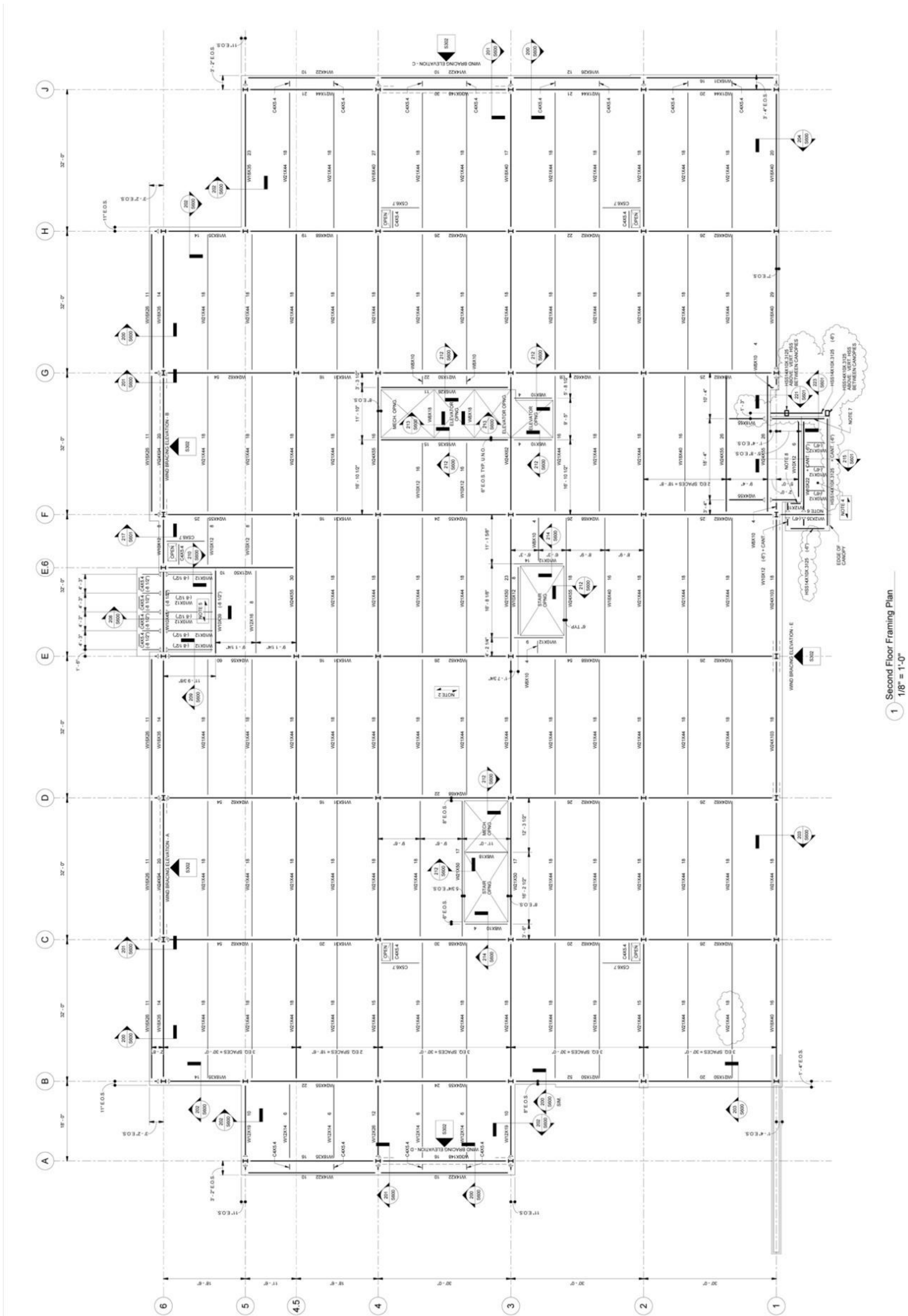
Typical Floor Layout



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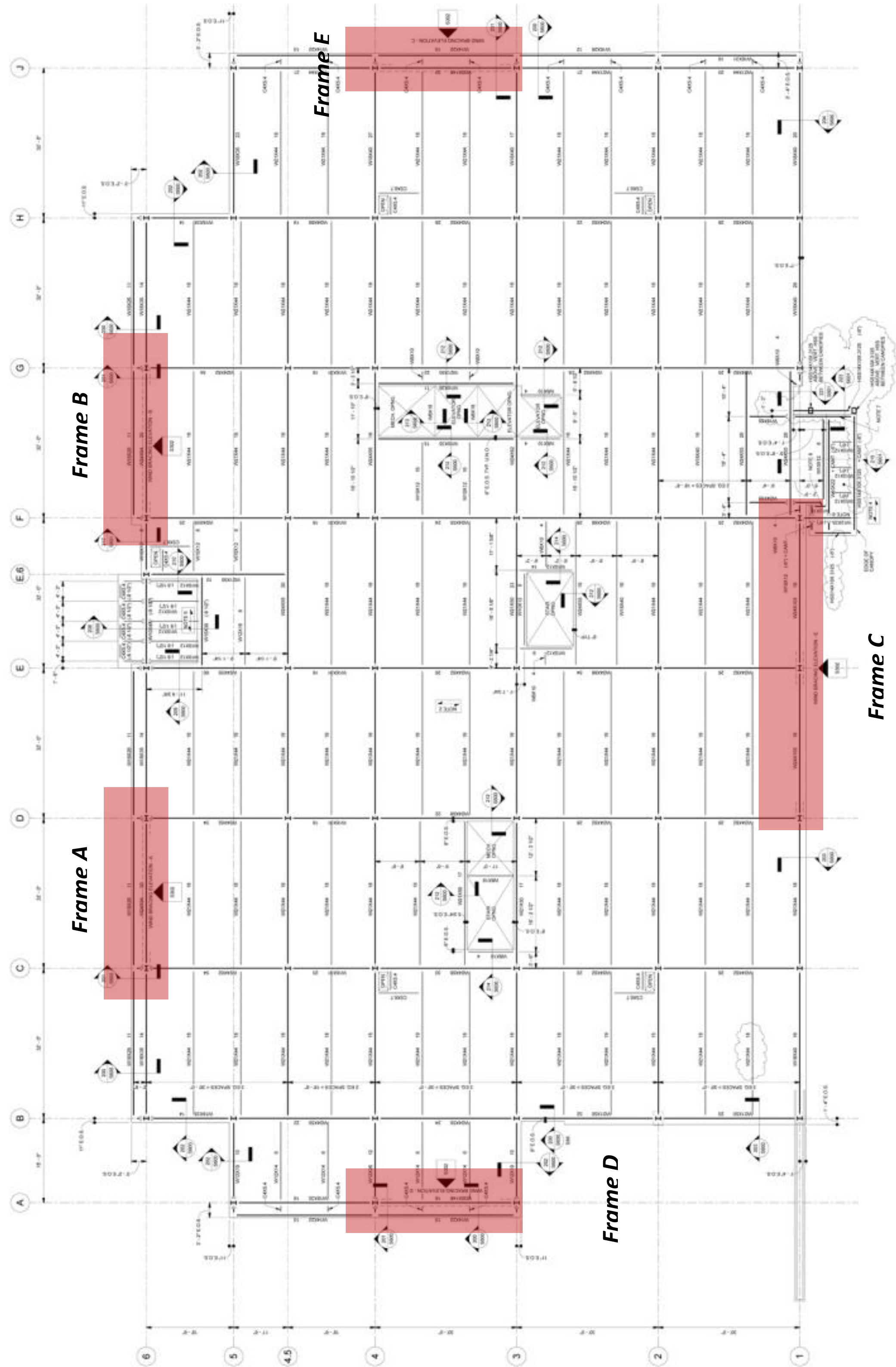
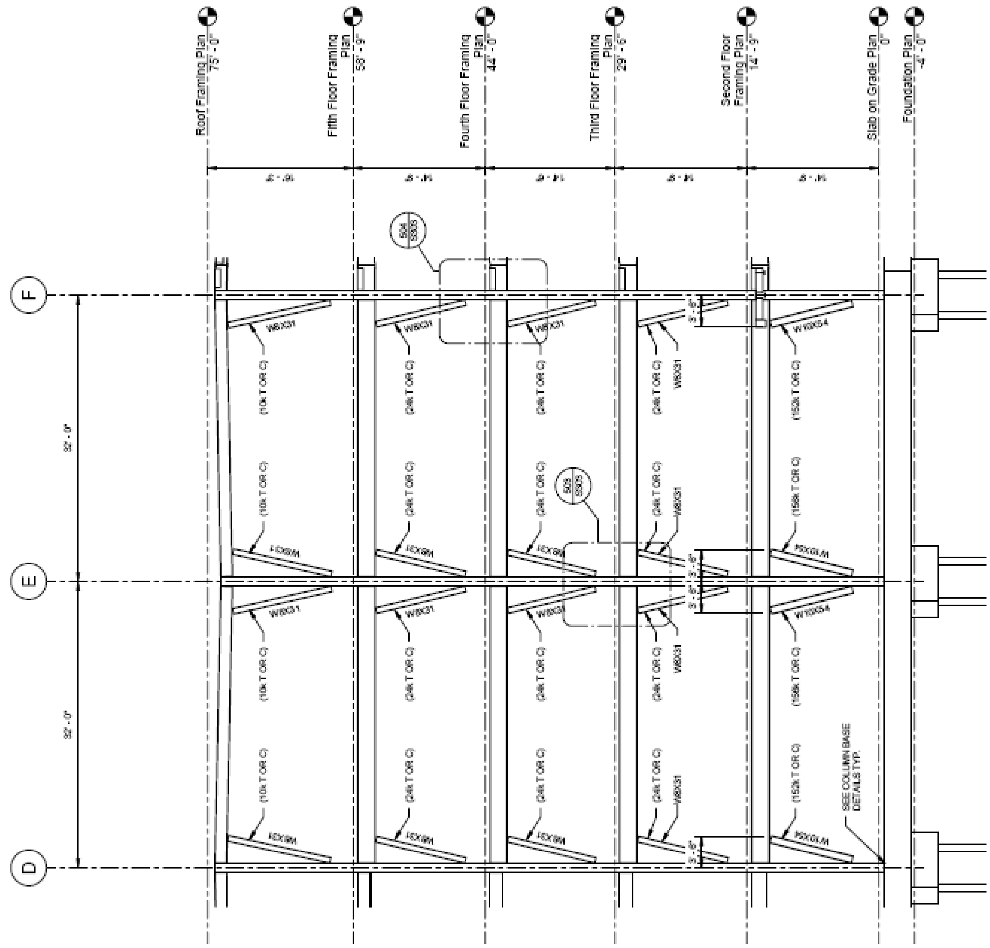
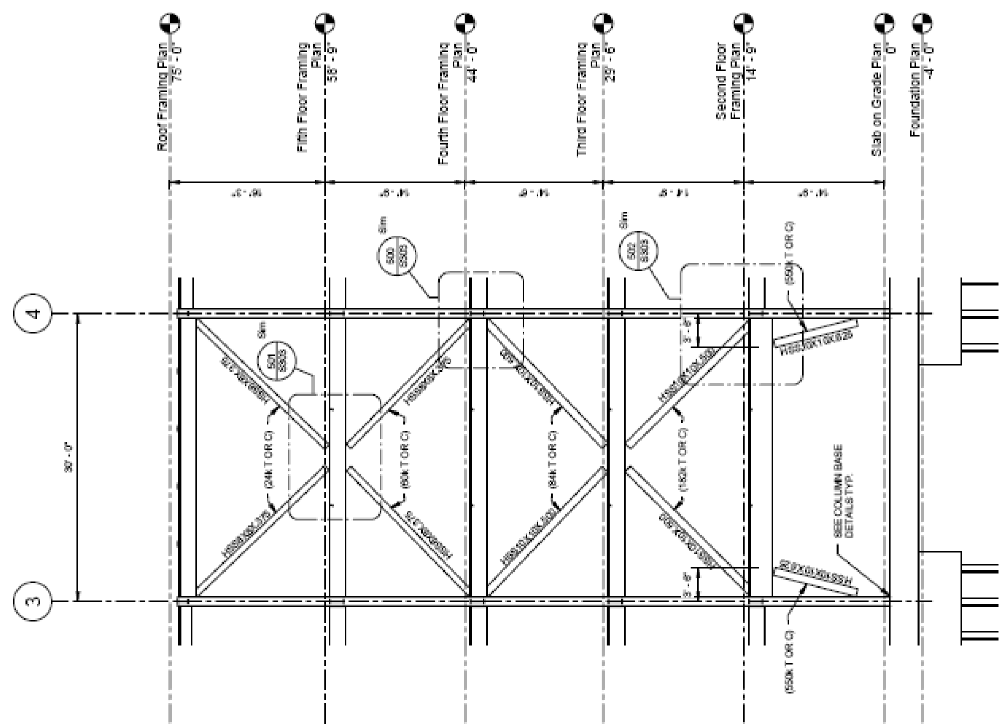
Typical Frame Layout



Frame C



Frame A, B, D, E (Sim.)



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APPENDIX B: WIND AND SEISMIC DATA

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

Bridgeside Point II -- Pittsburgh, PA

Floor Heights	Level	Total Height	K_z	q_z	q_h	Wind Pressures (psf)					
						N-S Windward	N-S Leeward	N-S Side Wall	E-W Windward	E-W Leeward	E-W Side Wall
15.25	Roof	74.00	1.188	20.94	20.94	22.23	-2.58	-12.38	20.81	-6.88	-11.14
14.75	5	58.75	1.133	19.97	20.94	21.37	-2.58	-12.38	20.02	-6.88	-11.14
14.50	4	44.00	1.066	18.79	20.94	20.33	-2.58	-12.38	19.06	-6.88	-11.14
14.75	3	29.50	0.979	17.26	20.94	18.98	-2.58	-12.38	17.81	-6.88	-11.14
14.75	2	14.75	0.849	14.96	20.94	16.96	-2.58	-12.38	15.94	-6.88	-11.14

Level	**Wind Design**					
	Load (kips)		Shear (kips)*		Moment (ft-k)	
	N-S	E-W	N-S	E-W	N-S	E-W
Roof	55	103	0	0	4059	7654
5	51	97	55	103	3009	5710
4	48	92	106	201	2119	4054
3	46	89	154	293	1360	2632
2	42	82	200	382	616	1216
Total	**242**	**464**	**242**	**464**	**11164**	**21,266**

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

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)	13,550

$S_{ms} = S_s * F_a$	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)	271

Dead Loads					
Typical Floor Loads (psf)		Roof Loads (psf)		Penthouse Loads (psf)	
Partitions	10	M.E.P.	5	M.E.P.	5
Finishes	3	Slab & Deck	50	Slab & Deck	25
M.E.P.	5	Structural Steel	10	Structural Steel	10
Slab & Deck	57	Misc.	5	Misc.	5
Structural Steel	15	--	-	--	-
Total	**90**	**Total**	**70**	**Total**	**45**

Base Shear and Overturning Moment Distribution								
Story	h_x (feet)	Area (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	31512	2206	930379	0.333	90.2	0.0	6768.5
5	58.75	31512	2836	849825	0.304	82.4	90.2	4842.9
4	44.00	31512	2836	566960	0.203	55.0	172.7	2419.8
3	29.50	31512	2836	323944	0.116	31.4	227.7	927.0
2	14.75	31512	2836	122752	0.044	11.9	259.1	175.6
1	0	31512	2500	0	0.000	0.0	271.0	0.0
Total	75	0	13550	2793859	1.000	271	**271**	**15,134**

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APPENDIX C: CENTERS OF MASS AND RIGIDITY

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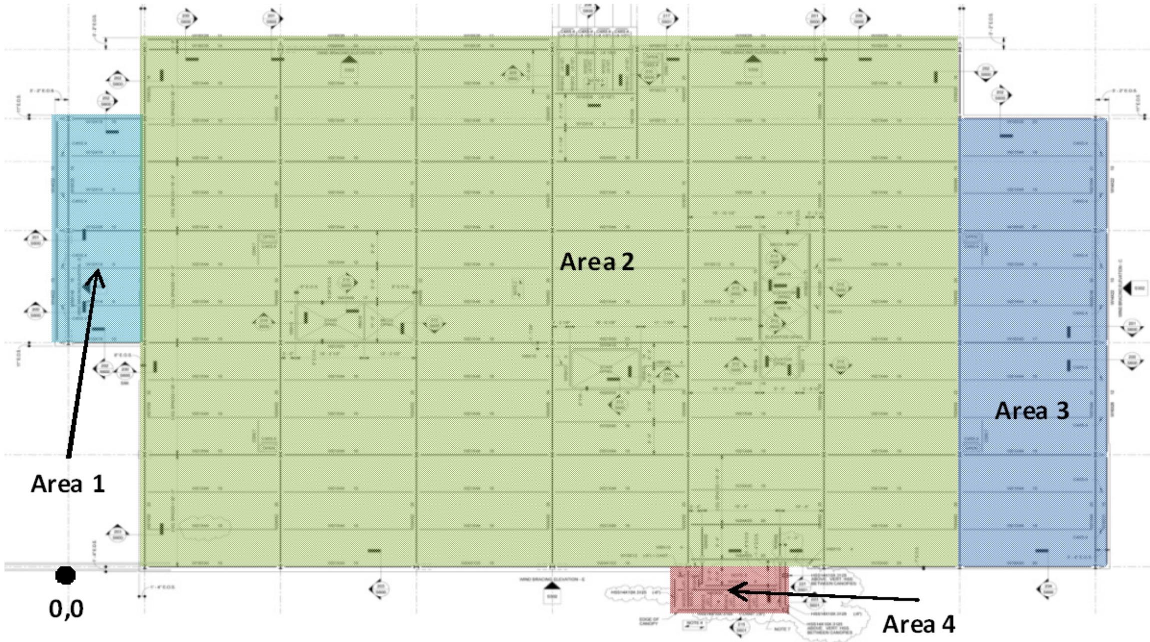
Center of Mass



				X (ft)	Y (ft)
Area 1 =	1080	(ft)	Center of Mass A1 =	9	90
Area 2 =	26,592	(ft)	Center of Mass A2 =	114	69.25
Area 3 =	3840	(ft)	Center of Mass A3 =	226	60
Area 4 =	176	(ft)	Center of Mass A4 =	154.8	-5

X_{CM} = 124.22 (ft)

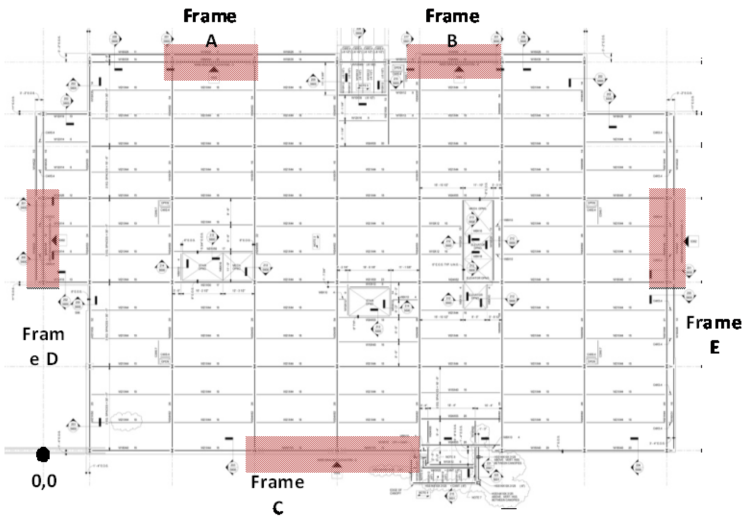
Y_{CM} = 70.84 (ft)

*Note: Floor plan is the same from floor to floor; therefore Center of Masses are constant.

Values from RAM Computer Model

Story	Diaph. #	Weight kips	Mass k-s2/ft	Inertia ft-k-s2	Xm ft	Ym ft
Roof	1	1774.0	55.093	239000	127.97	69.48
Fifth	1	4590.1	142.550	921714	124.29	69.20
Fourth	1	4601.0	142.888	924189	124.29	69.20
Third	1	4613.6	143.279	927021	124.27	69.19
Second	1	4675.6	145.206	947069	124.78	68.48

Center of Rigidity



Frame	Frame Centers x (ft)	Frame Centers y (ft)	Distance Center of Rigidity d_x (ft)	Distance Center of Rigidity d_y (ft)
A	66	138.5	55	28
B	162	138.5	41	28
C	114	0	7	110
D	0	45	121	65
E	242	45	121	65

Story	Frame Stiffness (kip/in)					Centers of Rigidity		Story Shear (kips)		Eccentricity		Torsional Rigidity
	A	B	C	D	E	X_R (ft)	Y_R (ft)	N-S	E-W	e_x (ft)	e_y (ft)	J (k*ft/in)
Roof	95	95	15	130	128	120.06	128.37	0	0	4.16	57.53	4,110,016
5th	114	114	28	165	165	121.00	123.35	55	103	3.22	52.51	5,352,735
4th	137	137	43	217	214	120.16	119.71	106	201	4.06	48.87	7,050,486
3rd	152	152	88	231	233	121.52	107.41	154	293	2.70	36.57	8,106,183
2nd	188	188	335	274	275	121.22	73.24	200	382	3.00	2.41	12,418,775
Total	137	137	102	203	203	120.79	110.42	242	464	3.43	39.58	7,407,639

-End of Section-