TECHNICAL REPORT 3

S.T.E.P.S. Building

Lehigh University

Bethlehem, PA

Joseph S. Murray, Structural Option

Faculty Advisor: Linda Hanagan

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Lateral System Analysis

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Executive Summary

The S.T.E.P.S. Building in Bethlehem, PA sits on Lehigh University's campus. It is a mixed use facility consisting of laboratories, lecture halls, and faculty offices. The building is divided into two main wings which are bridged by a central atrium.

The structural system of the building consists of semi-rigid moment frames and full moment frames. It uses a composite floor as a rigid diaphragm to transfer lateral loads imposed on the façade to the beams and girders. The beams and girders then transfer these loads through their moment connections to a network of mainly W14 columns. The columns finally transfer the load into the soil through a combination of spread footings and mat foundations.

In order to better analyze the lateral force resisting structural system, a 3D model was produced in RAM Structural System. The model was constructed from the floor plans, specifications, and detailed drawings of the S.T.E.P.S. Building. All gravity and lateral members were appropriately sized according to the plans. RAM was used to check the center of mass and center of rigidity of each floor which were relatively close together. The RAM program checked for incidental torsional shear in the wind calculation based on this eccentricity.

To simplify the design and avoid error, only lateral loads were considered. This left the load combinations of 1.6W and 1.0E to be used in analysis. Based on the location and the seismic information on the structural drawings, wind controlled over seismic in all cases.

Story shears were checked using RAM and the results were used to check the overturning moment imposed on the building by the wind loads. This value was checked against the weight of the building multiplied by the eccentricity of the center of mass. The resisting moment was greater than the overturning moment, so uplift will not occur.

Four locations were chosen to check the wind drift limit of h/400. All of these locations passed for each story.

Two manual lateral spot checks were performed to assess the results of the computer program by hand. A braced frame and a moment frame were checked against the calculated drift from the RAM program. They were also checked for strength requirements according to a specification for the bracing on the drawings and against allowable moment in AISC 14th Edition.

Building Introduction

Lehigh University envisioned the Science, Technology, Environment, Policy, and Society (S.T.E.P.S.) Building as a way to attract new students and retain existing students in the science and engineering fields. A picture of the building is in Figure 1. The university lacked a modern laboratory building with all the amenities that have come with increases in technology over the years. In an interesting and experimental fashion, the departments have been intermixed by Health, Education & Research Association, Inc. They believe it will lead to increased communication and collaboration among faculty and researchers of various disciplines.

Figure 1: South Façade

Image Courtesy of Lehigh University

The building is oriented on the corner of East Packer Ave. and Vine St. as shown in Figure 2. The streets do not intersect at a 90 degree angle. The architects decided to use site lines to orient the building, which led to the nonlinear shape of the façade along Vine St.

Figure 2: Site Plan

Image Courtesy of BCJ Architects

Lehigh University slowly purchased the properties which were on the building site as they planned for a building to be put there. The location was ideal for expanding campus activities close to the campus core. This is shown in Lehigh's Campus Master Plan of 2000 in Figure 2.

Figure 2: Campus Master Plan

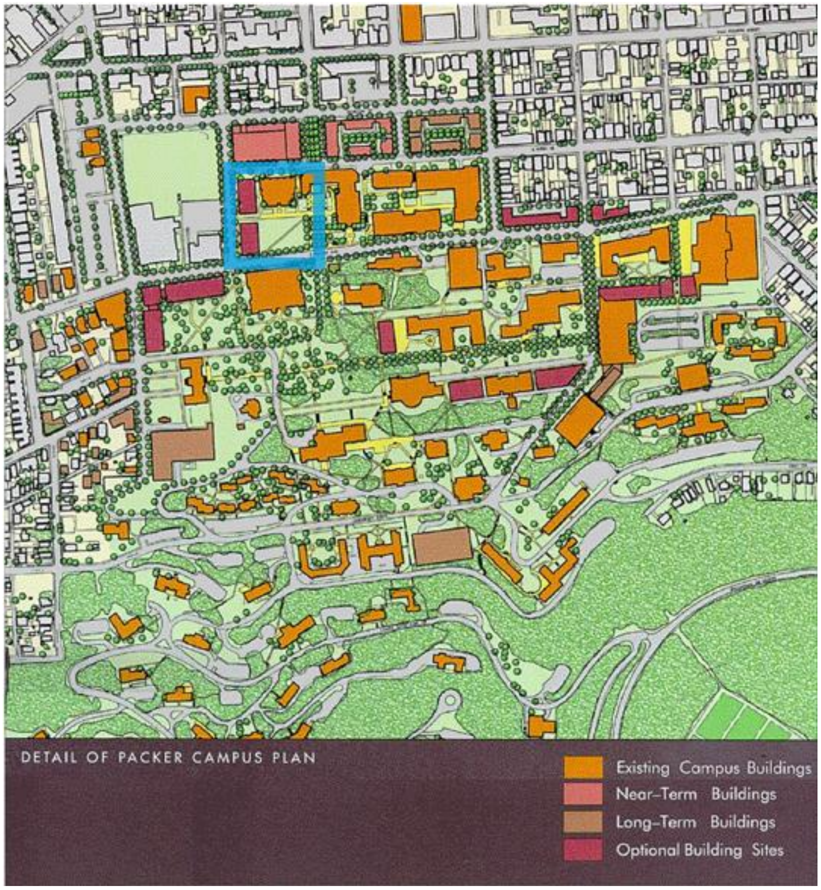


Image Courtesy of Lehigh University

The building is also connected to an existing structure through the use of a raised pathway that is enclosed. This further encourages interconnectivity between faculty, researchers, and students, because the adjoining building contains part of the College of Social Sciences. Between this adjacent building and S.T.E.P.S., there is a large open lawn. The university made a significant effort to maintain this lawn for extracurricular activities such as frisbee, croquet, and football. The S.T.E.P.S. Building is divided into three wings for the purpose of this analysis. These wings are diagramed in Figure 3.

Figure 3: Wings A, B, and C of S.T.E.P.S. Building

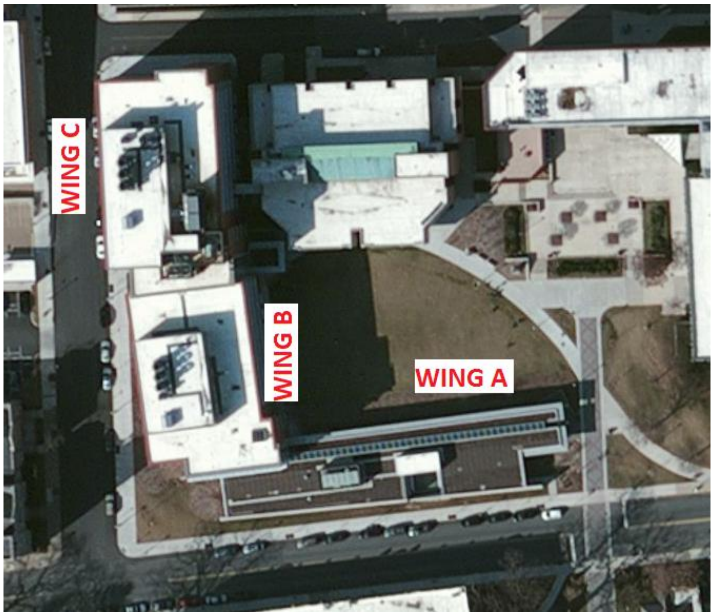


Image courtesy of Bing.com

Wing A is a one story structure with a lounge and entryway. It has raised clearstories to allow for natural daylight to illuminate the space. It also has a 12" deep green roof supported by structural wood which helped in earning LEED Certification. The building is LEED Gold certified by the United States Green Building Council (USGBC). Because of its limited building height, Wing A will not be analyzed in this report.

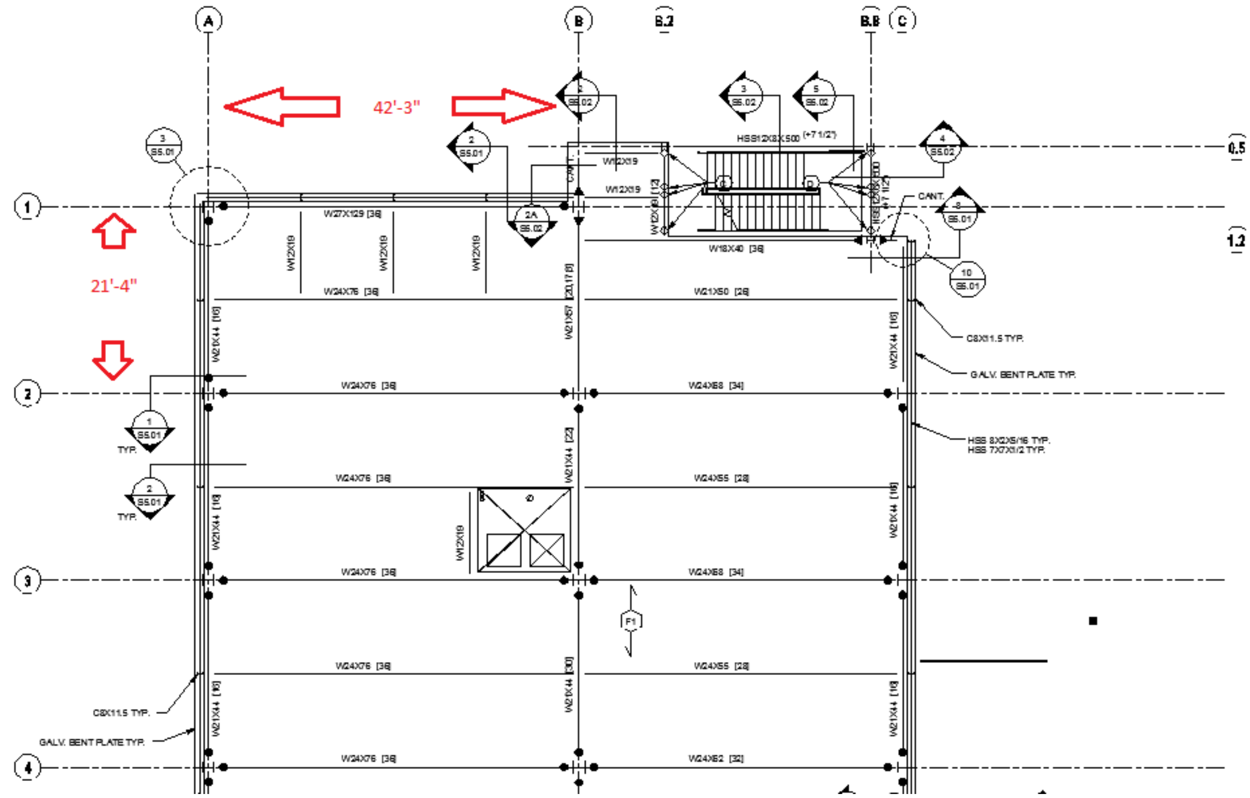
Wing B is a four story steel framed structure oriented along Packer Ave. There is a large atrium with lounge areas connecting Wing B to Wing C on each floor. Wing C is also steel framed and is 5 stories.

The gravity and lateral load resisting elements continue uninterrupted through the atrium. As a result, Wing B and Wing C will be treated as one building. The building's lateral system consists of moment connections between columns and beams throughout the building.

Sustainable features of the building include the green roof, high-efficiency glazing, sun shading, and custom mechanical systems.

Structural System

Figure 4: Typical Building Floor Plan



For a full floor plan, see Appendix A-1.

Floor System

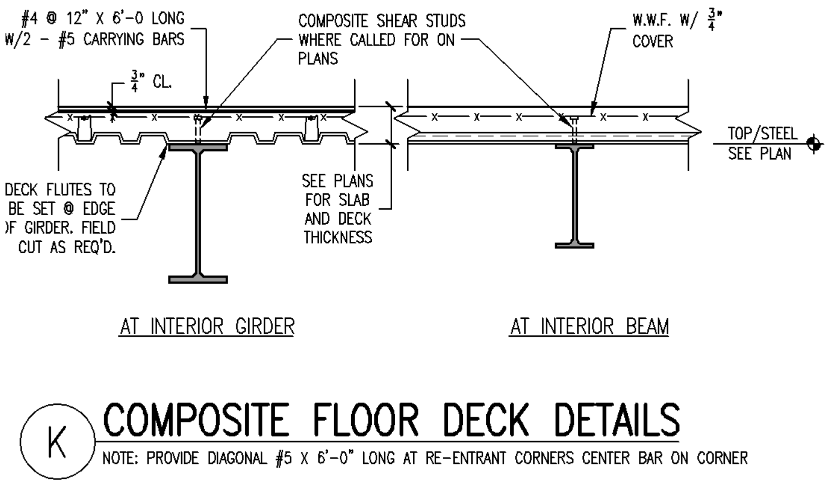
There is a composite steel deck floor system in place for all floors in Wings B & C above grade. Basement floors are slab on grade.

Along Vine St., which will be considered the longitudinal direction of the building, typical girders have a center to center span of 21'-4" with one intersecting beam at their midpoint. The transverse beams which run parallel to Packer Ave. have a span anywhere from 36'-11" to 42'8".

The decking is a 3" deep 18 gauge steel deck with 4-1/2" normal weight concrete topping and welded wire fabric. The bulk of the decking is run longitudinally throughout Wings B & C and has a span of 10'8" between beam centerlines. The exceptions to this are two bays to the very south of Wing B along Packer Ave. These bays are oriented transversely. The total thickness ends up being 7-1/2" with a 6x6" W2.9 x

W2.9 welded wire fabric embedded ¾" from the top of the slab. Figure 5 shows a typical detail of the composite floor decking.

Figure 5: Composite Floor Deck Detail



The floor system is supported by wide flange beams designed as simply supported. A combination of full moment connections, semi-rigid moment connections, and shear connections are used. Typical sizes for transverse beams are W24x55 and W24x76. The girders are W21x44. Most beams have between 28 and 36 studs to transfer shear. Figure 5 shows a typical Full Moment Connection with field welds noted. Figure 6 shows the entirety of the first floor system for Wing B. Figure 8 shows the entirety of the first floor system for Wing C.

Vertical Members

Wide flange columns are used throughout the building for gravity loads. They are arranged for strong axis bending in the transverse direction. Most spans have a column at either end with another at the midpoint.

W14 is the most common section size with weights varying from W14x90 all the way up to W14x192 on the lower floors.

Foundation

Schnabel Engineering performed a geotechnical analysis of the site in 2007. This concluded that the soil had sufficient bearing capacity to support the loads from the building.

Interior columns are supported by a mat foundation 18' wide and 3'-6" deep shown in Figure 6 and Figure 7. Exterior columns bear on square footings ranging from 11'x11' to 16'x16' with depths from 1'6" to 2'. These are tied into the foundation by base plates with concrete piers.

Figure 6: Mat Foundation Plan View

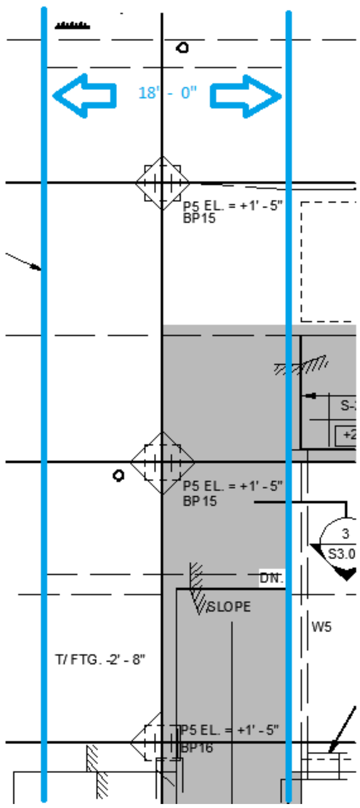


Figure 7: Mat Footing Schedule

MF1	✱ X 18'-0" X 3'-6"	#9 @ 12" O.C. E.W. BTM #9 @ 12" O.C. LONG. TOP #7 @ 12" O.C. TRANS. TOP	SEE PLAN FOR LENGTH

The reinforced foundation walls have strip footings ranging from 2’ to 6’ wide with depths between 1’ and 2’. These are monolithically cast with the piers for the exterior columns.

Roof System

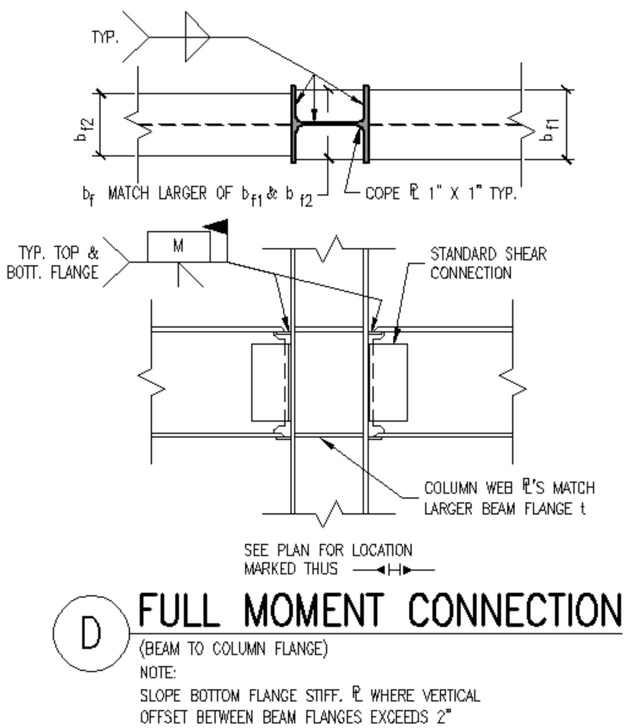
The roof decking consists of a 3” 16 gauge steel roof deck with a sloped roof for drainage. Topping ranges from ¼” to 4-1/2” to achieve a ¼”:1’ slope. Therefore, total thickness ranges from 3-1/4” to 7-1/2”. Framing is similar to floor framing with wide flanges ranging from W24x55 to W24x68.

The floor system has increased loads where the mechanical penthouses are situated. The penthouse itself is framed with square HSS tubing. Heavier W27x84 wide flange beams support this area.

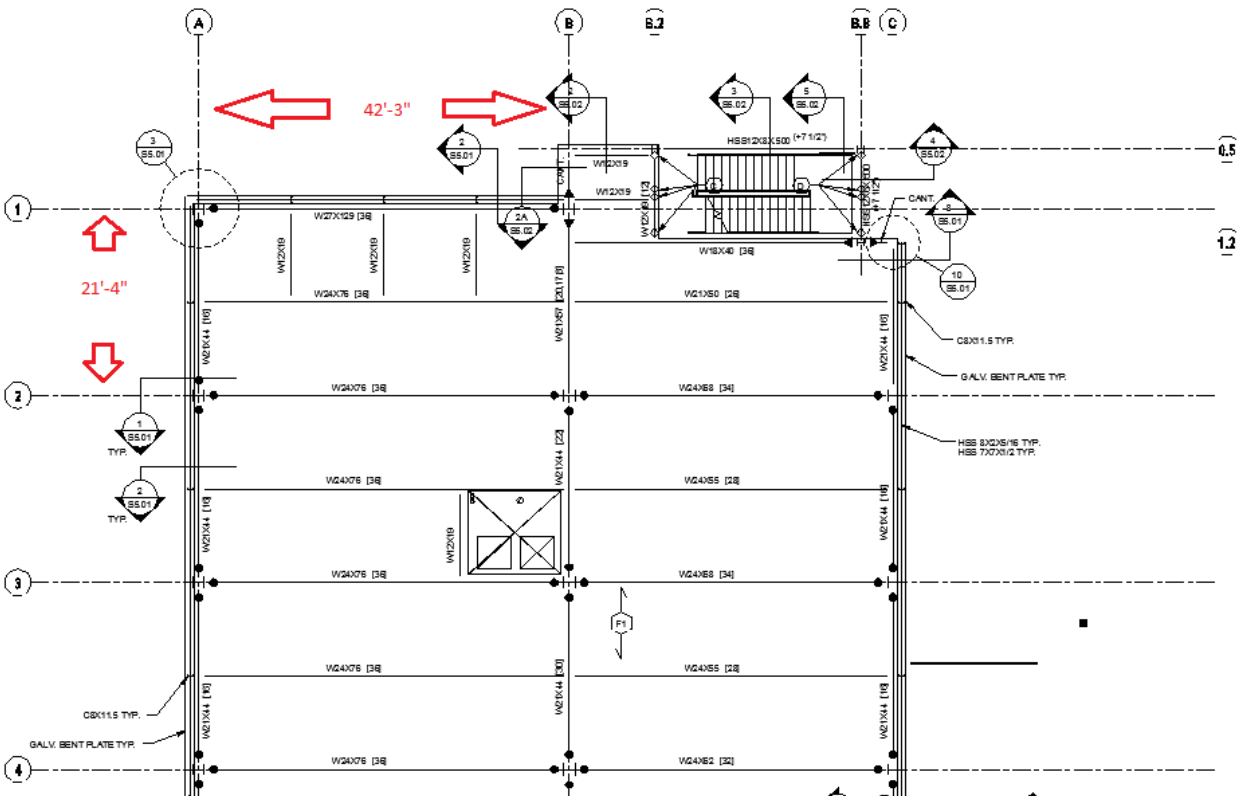
Lateral System

The building resists lateral loads by moment connections at the beam to column locations. They are continuous throughout the building and beams are designed as simply supported for gravity loads. The moment connections are designed only to take lateral loads. A typical full moment connection is shown in Figure 8. Many of these moment connections are semi-rigid connections to give the system more flexibility. An example of layout of the two types of moment connections in the floor plan is shown below in Figure 9. The triangles are full moment connections and the dots are semi-rigid.

Figure 8: Typical Full Moment Connection



<u>Figure 9:</u>



The lateral loads seen in the Penthouse are going to be the greatest based on height. At the highest Penthouse roof level, there are moment connections in the transverse direction and single angle braced frames in the longitudinal direction. The connections to the roof of the building are rigidly connected to the roof framing members. These members then transfer the load to flexible moment connections in the columns supporting the roof. These roof members are a larger W27x102 compared to adjacent members such as W24x68 or W27x84.

Design Codes

The primary design code used to construct the S.T.E.P.S. Building was the Pennsylvania Uniform Construction Code (PUCC). The PUCC is the primary code adopted by the city of Bethlehem, Pennsylvania. The PUCC is based on the International Code Council (ICC). When design was completed in 2008, the 2006 PUCC referenced the following codes:

2006 International Building Code

2006 International Electrical Code

2006 International Fire Code

2006 International Fuel Gas Code

2006 International Mechanical Code

ASCE 7-05, Minimum Design Loads for Buildings and Other Structures

AISC Steel Construction Manual, 13^{th} Edition

ACI 318-05, Building Code Requirements for Structural Concrete

ACI 530-05, Building Code Requirements for Masonry Structures

The primary codes employed in analysis were the AISC Manual and ASCE 7-05

Design Loads

Live Loads

Table 1: Live Load Values

Occupancy	Design Load on Drawings	ASCE 7-05 Load (Tables 4-1, C4-1)
Office	50 PSF	50 PSF + 20 PSF (Partitions)
Classroom	40 PSF	40 PSF
Laboratory	100 PSF	100 PSF
Storage	125 PSF	125 PSF
Corridors/Lobbies @ Ground Level	100 PSF	100 PSF
Corridors Above Ground Level	80 PSF	80 PSF

Dead Loads

Table 2: Calculated Dead Load

	Dimension	Unit Weight	Load (PSF)
3” 18 Ga. Composite Deck			2.84
4-1/2” Topping			75
Self-Weight			5
MEP Allowance			10
Ceiling Allowance			5
TOTAL			**97.84 PSF**

Roof Live Load

Table 3: Roof Live Load

Occupancy	Design Load on Drawings	ASCE 7-05 Load (Tables 4-1, C4-1)	Design Load
Roof	N/A	20 PSF	**20 SF**

Roof Dead Load

Table 4: Roof Dead Load

	Dimension	Unit Weight	Load (PSF)
3" 16 Ga. NS Roof Deck			2.46
3" Concrete Topping (Avg.)	0.290 CF/SF	150	43.5
Self-Weight			5
Roofing Allowance			10
TOTAL			**60.96 SF**

Roof Snow Load

Uniform Roof Snow Load

Table 5: Uniform Roof Snow Load

Design Factor	ASCE 7-05	Design Value
Snow Load (Pq)	Figure 7-1	30 PSF
Roof Exposure	Table 7-2	Fully Exposed
Exposure Type	Section 6.5.6.2	B
Exposure Factor (Ce)	Table 7-2	.9
Thermal Factor (Ct)	Table 7-3	1.0
Building Type	Table 1-1	III
Importance Factor (I)	Table 7-4	1.1
Flat Roof Snow Load (Pf)	Equation 7-1	20.8 PSF
Minimum Snow Load (Pf,min)	Section 7.2	22 PSF
Design Snow Load	Section 7.2	**22 PSF**

Pf = 0.7(Ce)(Ct)(I)(Pq)

Pf = 0.7(.9)(1.0)(1.1)(30) = 20.8 PSF

20.8 < Pf,min = 22 → Use 22 PSF as the Design Snow Load

5.5.2 Drift Snow Load

NOTE: For simplification of this analysis, snow drift was not considered. However, because of the raised penthouses, it will be necessary to consider snow drift later.

Penthouse Live Load

Table 6: Penthouse Live Load

Occupancy	Design Load on Drawings	ASCE 7-05 Load (Tables 4-1, C4-1)	Design Load
Mechanical Room	N/A	200 PSF	**200 PSF**

Penthouse Dead Load

Table 7: Penthouse Dead Load

	Dimension	Unit Weight	Design Load (PSF)
3" 18 Ga. Composite Deck			2.84
4-1/2" Concrete Topping			75
Self-weight			5
MEP Allowance			10
Ceiling Allowance			5
TOTAL			**97.84 PSF**

Brick Façade Load

Table 8: Brick Façade Load (Per Level)

	Height	Unit Weight (PSF)	Design Load (PLF)
Brick Veneer	10’-3”	40	410
2” Rigid Insulation	10’-3”	1.5	15.375
Cold Formed Steel Framing	10’-3”	1	10.25
Gypsum Wall Board (5/8”)	10’-3”	2.5	25.625
Window (Glass, Frame, Sash) (ASCE 7-05 Table C3-1)	5’-1”	8	40.8
TOTAL			**502.1 PLF**

Glass Curtain Wall Load

Table 9: Glass Curtain Wall Load (Per Level)

	Dimension	Unit Weight (PSF)	Design Load (PLF)
Window (Glass, Frame, Sash) (ASCE 7-05 Table C3-1)	15’-4”	8	**122.4 PLF**

Penthouse Wall Load

Table 10: Penthouse Wall Load

	Dimension	Unit Weight (PSF)	Load (PLF)
Metal Wall Panel	16’-4”	5	81.7
Steel Framing	16’-4”	2	32.7
Bracing Allowance	16’-4”	3	49
TOTAL			**163.4 PLF**

Wind Pressures

ASCE 7-05 was used for wind design. The Analytical Procedure in Chapter 6 is specifically what was instituted.

Table 11: Wind Design Factors:

Design Factor	ASCE 7-05	E/W Value	N/S Value
Design Wind Speed (V)	Figure 6-1C	90 mph	90 mph
Building Type	Table 1-1	III	III
Importance Factor (I)	Table 6-1	1.15	1.15
Exposure Type	6.5.6.2	Type B	Type B
Average Height (z)	6.5.8	84’	100’

Table 12: Design Wind Pressure by Level (Transverse Direction)

Level	Height	kz	qz	Pz (PSF) (Windward)	Ph (PSF) (Leeward)	Ptotal (PSF)
1	0’-0”	0.57	11.55	14.21	-18.18	25.47
2	15’-4”	0.58	11.76	14.46	-18.18	25.93
3	30’-8”	0.71	14.39	17.7	-18.18	31.73
4	46’-0”	0.79	16.01	19.69	-18.18	35.3
Roof/5th	60’-8”	0.85	17.22	21.18	-18.18	37.97
Roof/Penthouse	77’-0”	0.92	18.65	22.94	-18.18	41.12

Table 13: Design Wind Pressure by Level (Longitudinal Direction)

Level	Height	kz	qz	Pz (PSF) (Windward)	Ph (PSF) (Leeward)	Ptotal (PSF)
G	0’-0”	0.57	11.55	14.21	N/A	14.21
1	15’-4”	0.58	11.76	14.46	-14.67	23.33
2	30’-0”	0.70	14.4	17.70	-14.67	28.55
3	45’-4”	0.79	16.01	19.69	-14.67	31.76
4	61’-0”	0.85	17.23	21.19	-14.67	34.18
Roof/5th	77’-4”	0.92	18.65	22.94	-14.67	37.00
Roof/Penthouse	92’-0”	0.96	19.46	23.94	-14.67	38.61

Seismic Loads

Chapters 11 and 12 of ASCE 7-05 were used for seismic load design. The Equivalent Lateral Force procedure tests whether the building has the capability of handling a seismic event based on site and building properties.

Hand calculations can be found in Appendix A-2.

Seismic Design Factors

Design factors were the same for transverse and longitudinal directions since the building's lateral framing system consists of moment frames in both directions. Instead of determining the actual fundamental frequency through extensive calculation, the approximate fundamental period was determined using ASCE 7-05 Section 12.8.2.1.

Table 14: Seismic Load Design Factors

Design Factor	ASCE 7-05	Value
Short Period Spectral Response Acceleration (Ss)	USGS	0.291
One Second Spectral Response Acceleration (S1)	USGS	0.081
Site Class	Table 11.4-1	C
Short Period Site Coefficient (Fa)	Table 11.4-2	1.2
Long Period Site Coefficient (Fv)	Equation 11.4-1	1.7
Adjusted MCE Short Period Spectral Response Acceleration (Sms)	Equation 11.4-1	0.349
Adjusted MCE One Second Spectral Response Acceleration (SM1)	Equation 11.4-2	0.138
Design Short Period Spectral Response Acceleration (SMs)	Equation 11.4-3	0.233
Design One Second Spectral Response Acceleration (SM1)	Equation 11.4-4	0.0918
Maximum Height from Base (hn)	N/A	108.3'
Approximate Period Parameter (Ct)	Table 12.8-2	0.028
Approximate Period Parameter (x)	Table 12.8-2	0.8

Approximate Fundamental Period (Ta)	Equation 12.8-7	1.19 Hz
Building Type	Table 1-1	III
Importance Factor (I)	Table 11.5-1	1.25
Seismic Design Category	Table 6-2	B
Response Modification Coefficient (R)	Table 12.2-1	3.0
System Over-strength Factor (Omega)	Table 12.2-1	3.0
Deflection Amplification Factor (Cd)	Table 12.2-1	3.0
Flexible Diaphragm Condition	Section 12.3.1	Rigid
Long Period Translation Period (TL)	Figure 22-15	6
Seismic Response Coefficient (Cs)	Equation 12.8-3	0.0321

Effective Seismic Weight

Table 15: Effective Seismic Weight by Level

Level	Floor Area (SF) (96 PSF)	Roof Area (SF) (62.5 PSF)	Penthouse Floor Area (SF) (296 PSF)	Brick Façade (ft.) (510.6 PLF)	Glass Curtain Wall (ft.) (122.4 PLF)	Penthouse Wall (ft.) (246 PLF)	Effective Seismic Weight (k)
Penthouse		4497					281.06
Roof/Pent house		7894	4497			288.7	1895.5
5	10832	9375	1557	421.3		161.3	2341.47
4	21814			589.7	89.5		2406.2
3	21814			589.7	89.5		2406.2
2	21814			589.7	89.5		2406.2
1	21814			589.7	89.5		2406.2
TOTAL	98088	21766	6054	2780.1	358	450	**14143**

Design Seismic Loads

Table 16: Seismic Design Loads by Level

Level	Effective Seismic Weight (wx)	Height from Base (hx)	$(wxhx)^k$	Vertical Distribution Factor (Cvx)	Lateral Seismic Force (Fx) (k)	Seismic Design Story Shear (Vx) (k)	Overturning Moment (k-ft.)
Penthouse	281.06 k	108.3'	3298348	0.0654	29.97	29.97	3217.11
Roof/Penthouse	1895.5 k	93'	16390547	0.3250	147.57	177.54	13724.53
5	2341.47 k	76.7'	13763837	0.2729	123.92	301.46	9501.36
4	2406.2 k	61.3'	9050606	0.1794	81.48	382.94	4997.72
3	2406.2 k	46'	5091519	0.1009	45.84	428.78	2108.76
2	2406.2 k	30.7'	2263389	0.0448	20.37	449.15	625.02
1	2406.2 k	15.3'	565478	0.0112	5.09	**454**	78.05
TOTAL	**14143 k**		50423724	1.0			**34252.54**

Seismic Base Shear = 454 k

Overturning Moment = 34252.5 k-ft.

Calculations for the earthquake analysis can be made available upon request.

RAM Model

RAM Structural System was used to create a 3D model of the S.T.E.P.S. Building. Gridlines were produced from AutoCAD drawings, and line elements were used to build the framework for the lateral and gravity force resisting systems. Steel sections and member properties were added to the line elements as noted on the structural drawings. Any beams that had a moment connection were modeled as part of the lateral system along with 4 braces in the penthouse frames. The majority of the beams are W-flange members with HSS rectangular tubing used in some locations. The bracing utilized as part of the penthouse's lateral resisting system are L4x4x3/8 X-Bracing. Any columns which received a lateral beam were also modeled as part of the lateral system. The columns consist mainly of W14 sections with HSS rectangular tubing used for the elevator core and the penthouse columns. Some of the gravity beams terminated in a concrete basement wall, and an 18" thick reinforced concrete wall was modeled as shown on plan and in structural details. All exterior columns terminate in a spread footing foundation, while interior columns terminated in mat foundations.

The composite floor system that exists throughout the building was modeled as a rigid diaphragm on each floor level. Weight of steel members and the floor systems was calculated by RAM, and then the weight of the wall system was added manually to each floor based on the floor's perimeter and the weight of the wall attached. Figure 12 shows the RAM model in 3D from the west direction and Figure 13 shows it from the east direction. The red color represents lateral members, and the blue color represents gravity members. Figure 14 shows the braced frames in the penthouse in purple.

Figure 12: RAM Model (West Direction)

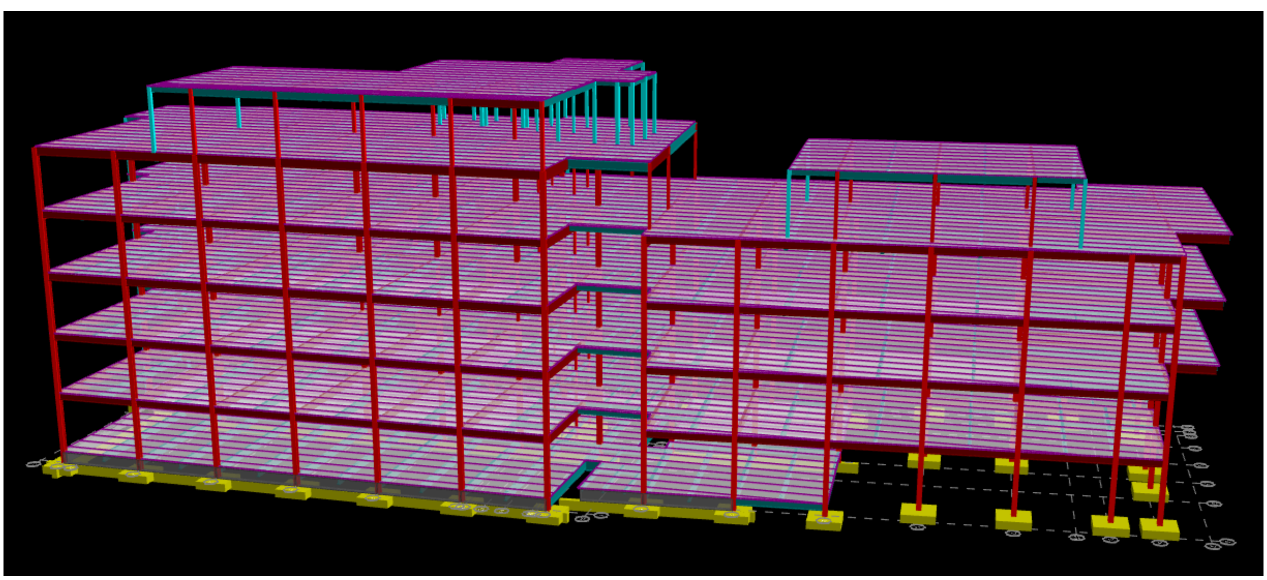


Figure 13: RAM Model (East Direction)

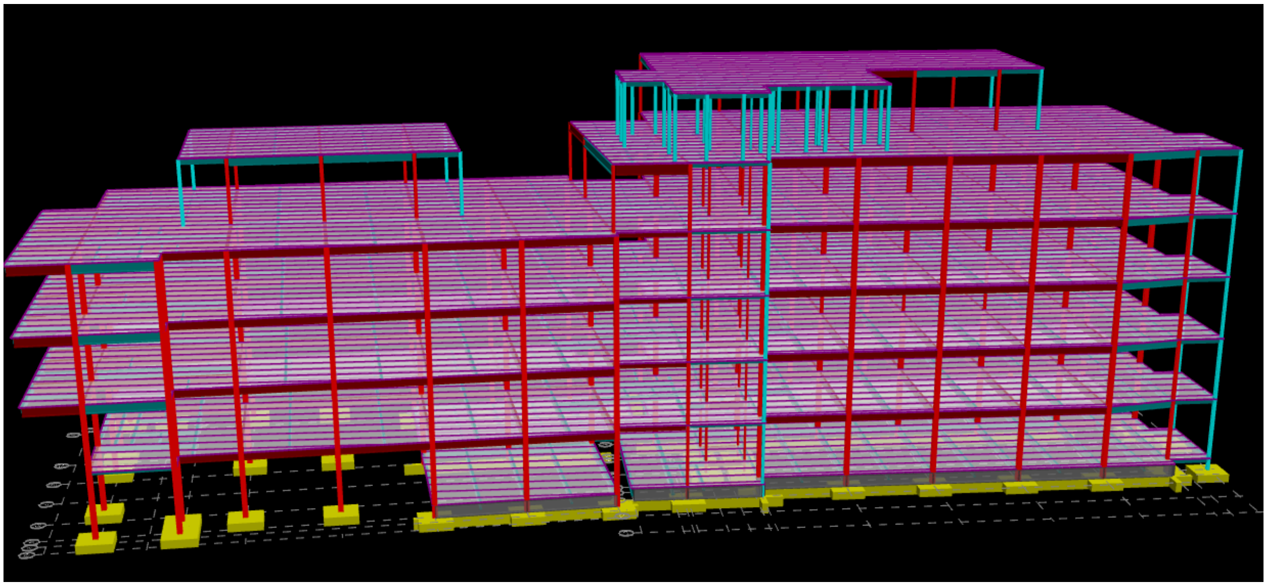
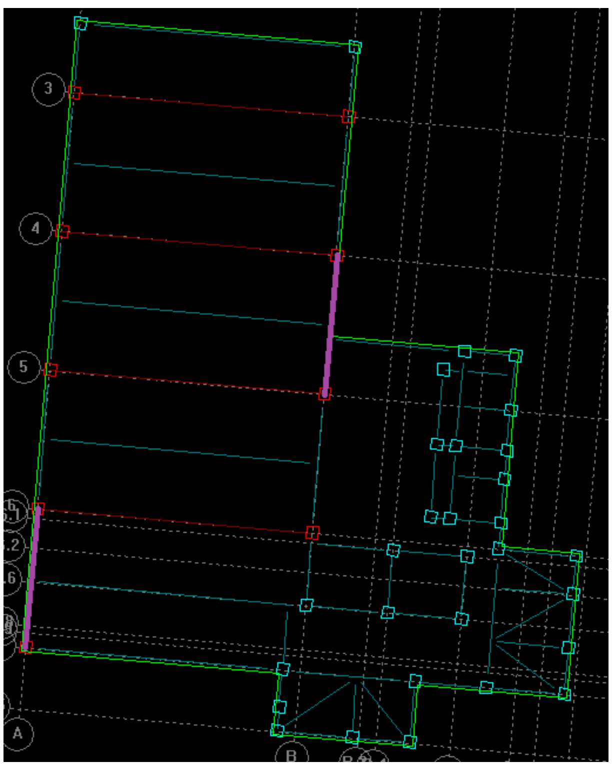


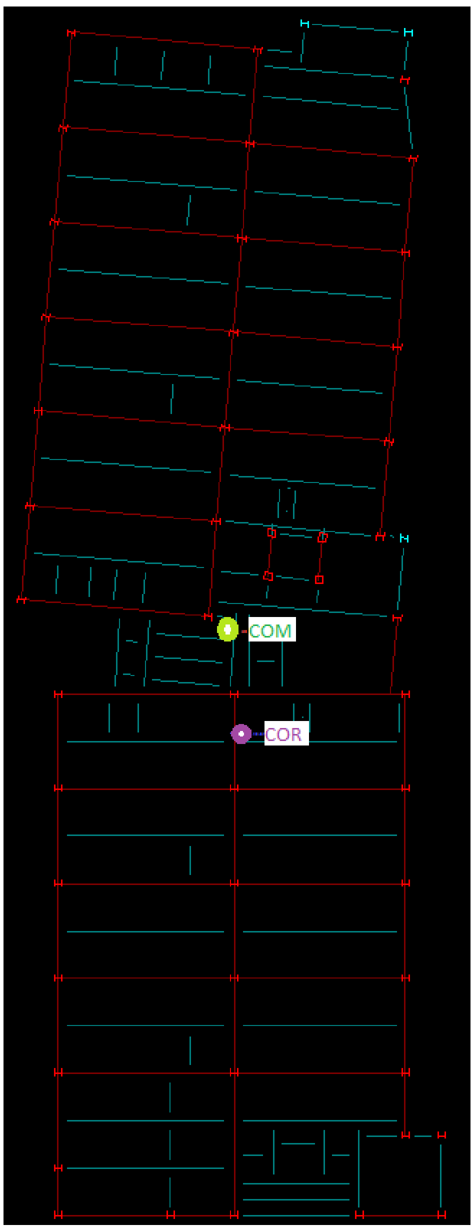
Figure 14: Penthouse Braced Frames



<u>Center of Mass and Center of Rigidity</u>

The center of mass (COM) and the center of rigidity (COR) were determined for each diaphragm by RAM. After visual inspection, the locations were confirmed, and analysis of the model proceeded. Figure 15 shows the center of mass and center of rigidity for the second floor. The COM is represented by a green circle at (38.45, 142.86), and the COR is represented by a purple circle at (40.84, 119.71).

<u>Figure 15: Center of Mass and Center of Rigidity</u>



<u>Load Combinations:</u>

The load combinations in ASCE 7-05 were considered in analysis. Figure 16 shows Table 2.3.2 from ASCE.

<u>Figure 16: LRFD Load Combinations</u>

2.3.2 Basic Combinations. Structures, components, and foundations shall be designed so that their design strength equals or exceeds the effects of the factored loads in the following combinations:

1. $1.4(D + F)$
2. $1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$
4. $1.2D + 1.6W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + L + 0.2S$
6. $0.9D + 1.6W + 1.6H$
7. $0.9D + 1.0E + 1.6H$

To simplify the analysis and limit errors, only combinations with wind or seismic were initially considered. Since the response of the lateral system is of primary concern in this analysis, gravity loads were not considered, and the load combinations were reduced to either 1.6W or 1.0E.

Wind load cases were considered from the ASCE 7-05 Main Wind Force Resisting System method (Method 2). These can be found in Figure 17.

After running the analysis for wind and viewing the story displacements from RAM, it was determined that Case 1 controlled for wind in both the x and y direction of the S.T.E.P.S. Building. All other wind cases were removed from consideration as the applied eccentricity did not produce a greater wind load.

Earthquake was checked by RAM for an Sds of 0.233 as specified by the structural drawings. Wind controlled in both directions for every story.

<u>Story Shears</u>

Story shears were checked using RAM based on the controlling wind cases. The maximum story shears in the x direction are in Figure 17, and the maximum story shears in the y direction are shown in Figure 18.

Figure 17: Maximum Story Shears in the X Direction

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
Story 7	26.82	26.82	-0.00	-0.00
Story 6	99.67	72.85	0.42	0.42
Story 5	205.47	105.80	1.26	0.84
Story 4	322.11	116.65	0.54	-0.72
Story 3	433.07	110.95	0.08	-0.46
Story 2	533.24	100.17	-0.24	-0.32
Story 1	428.95	-104.28	5.35	5.58

Figure 18: Maximum Story Shears in the Y Direction

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
Story 7	0.00	0.00	20.22	20.22
Story 6	-0.75	-0.75	66.20	45.98
Story 5	-1.49	-0.74	102.57	36.38
Story 4	-0.30	1.18	136.40	33.83
Story 3	-0.76	-0.45	167.89	31.49
Story 2	-0.11	0.65	197.08	29.18
Story 1	1.49	1.59	144.60	-52.47

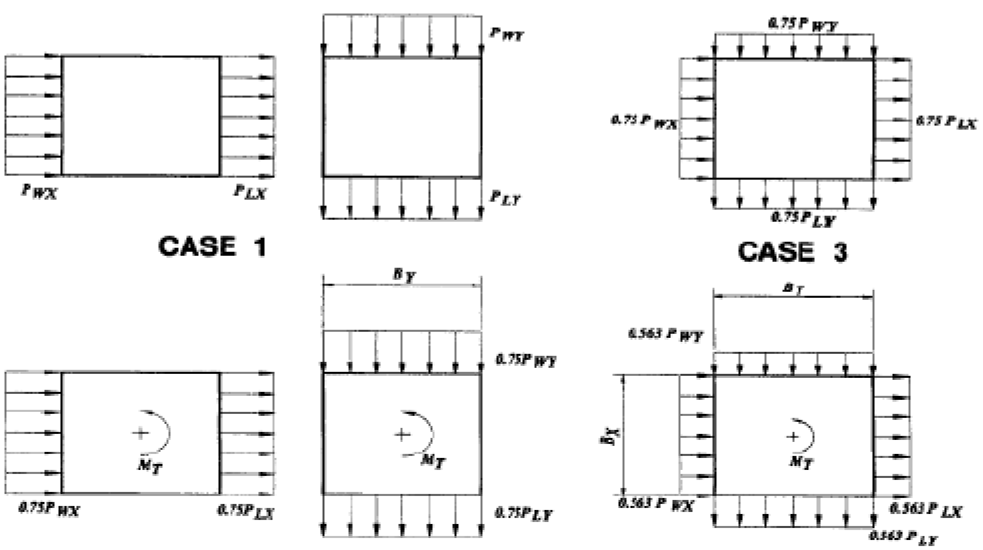
Overturning Moment

The shears for each story in the x direction were multiplied by the height of each story to produce a total overturning moment of 31,528.5 k-ft.

The resisting moment was calculated by multiplying the weight of the building by the eccentricity of the center of mass. From previous calculations, the effective building weight is 14,143 kips. The center of mass is 38.45 feet from the edge of the building. This results in a resisting moment of 543,798 kip-ft. This is enough to handle the overturning moment produced by the controlling wind case

<u>Figure 17: ASCE 7-05 Wind Load Cases</u>

Main Wind Force Resisting System – Method 2		All Heights
Figure 6-9	Design Wind Load Cases	



$M_T = 0.75\,(P_{WX}+P_{LX})B_X e_X$
$e_X = \pm\,0.15\,B_X$

$M_T = 0.75\,(P_{WY}+P_{LY})B_Y e_Y$
$e_Y = \pm\,0.15\,B_Y$

$M_T = 0.563\,(P_{WX}+P_{LX})B_X e_X + 0.563\,(P_{WY}+P_{LY})B_Y e_Y$
$e_X = \pm\,0.15\,B_X$ $\quad e_Y = \pm\,0.15\,B_Y$

CASE 2 **CASE 4**

Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes:

1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of 6.5.12.2.1 and 6.5.12.2.3 as applicable for building of all heights.
2. Diagrams show plan views of building.
3. Notation:
   - P_{WX}, P_{WY}: Windward face design pressure acting in the x, y principal axis, respectively.
   - P_{LX}, P_{LY}: Leeward face design pressure acting in the x, y principal axis, respectively.
   - $e\ (e_X, e_Y)$: Eccentricity for the x, y principal axis of the structure, respectively.
   - M_T: Torsional moment per unit height acting about a vertical axis of the building.

<u>Maximum Story Drifts</u>

RAM was used to determine the story drifts based on the controlling wind cases. Four points were chosen as control points to establish displacement and drift data show in Figure 18 as blue dots. Tables 17-20 show the results and compare to allowable drifts of h/400 as per ASCE 7-05. Some of the frames do not extend to levels 6 and 7 and are marked as “N/A”. The story drifts passed all acceptable drift limits based on the RAM output and an acceptable drift of h/400.

<u>Figure 18: Location of Control Points for Drift Analysis</u>

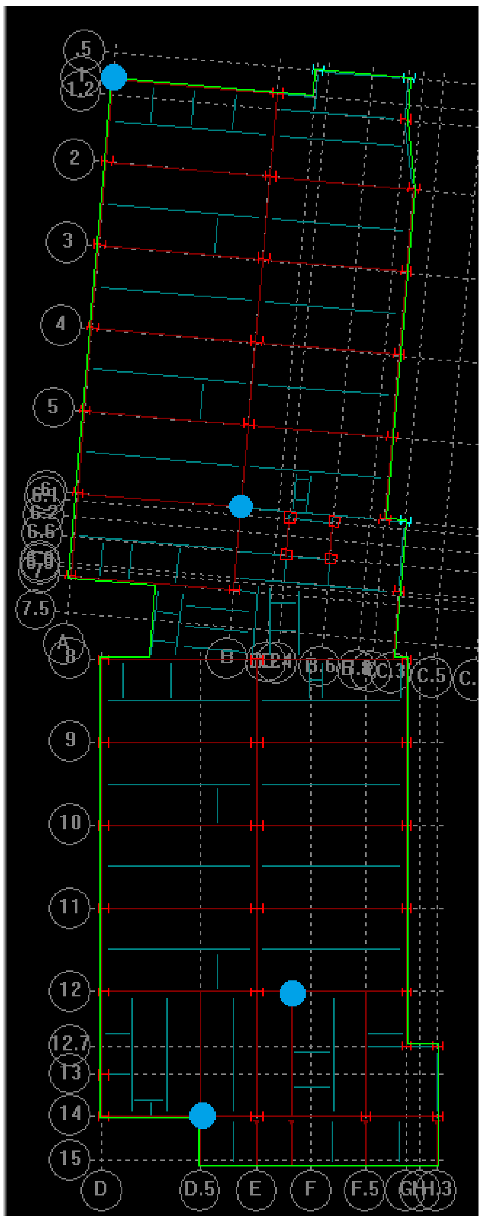


Table 17: Maximum Lateral Displacements and Story Drifts (Column A-1)

Maximum Wind Story Drift, N-S Direction				
	Story	**Story Drift (in)**	**Allowable Drift (in)**	**Adequacy**
Column A-1	7	N/A	3.24	OK
	6	-0.3066	2.78	OK
	5	0.1181	2.32	OK
	4	0.1708	1.86	OK
	3	0.1735	1.4	OK
	2	0.1755	0.94	OK
	1	0.105	0.48	OK
Maximum Wind Story Drift, E-W Direction				
	Story	**Story Drift (in)**	**Allowable Drift (in)**	**Adequacy**
Column A-1	7	N/A	3.24	OK
	6	0.1533	2.78	OK
	5	0.2713	2.32	OK
	4	0.4295	1.86	OK
	3	0.4525	1.4	OK
	2	0.4390	0.94	OK
	1	0.0270	0.48	OK

Table 18: Maximum Lateral Displacements and Story Drifts (Column B-6)

Maximum Wind Story Drift, N-S Direction				
	Story	**Story Drift (in)**	**Allowable Drift (in)**	**Adequacy**
Column B-6	7	0.3288	3.24	OK
	6	-0.2756	2.78	OK
	5	0.1178	2.32	OK
	4	0.1705	1.86	OK
	3	0.1730	1.4	OK
	2	0.1744	0.94	OK
	1	0.0103	0.48	OK
Maximum Wind Story Drift, E-W Direction				
	Story	**Story Drift (in)**	**Allowable Drift (in)**	**Adequacy**
Column B-6	7	0.3551	3.24	OK
	6	0.1755	2.78	OK
	5	0.1929	2.32	OK
	4	0.305	1.86	OK
	3	0.3245	1.4	OK
	2	0.2977	0.94	OK
	1	0.0178	0.48	OK

Table 19: Maximum Lateral Displacements and Story Drifts (D.5-14)

Maximum Wind Story Drift, N-S Direction				
	Story	**Story Drift (in)**	**Allowable Drift (in)**	**Adequacy**
Column D.5-14	7	N/A	N/A	N/A
	6	N/A	N/A	N/A
	5	0.1179	2.32	OK
	4	0.1706	1.86	OK
	3	0.1731	1.4	OK
	2	0.1851	0.94	OK
	1	N/A	N/A	N/A
Maximum Wind Story Drift, E-W Direction				
	Story	**Story Drift (in)**	**Allowable Drift (in)**	**Adequacy**
Column D.5-14	7	N/A	N/A	N/A
	6	N/A	N/A	N/A
	5	0.1364	2.32	OK
	4	0.2346	1.86	OK
	3	0.2893	1.4	OK
	2	0.2315	0.94	OK
	1	N/A	N/A	N/A

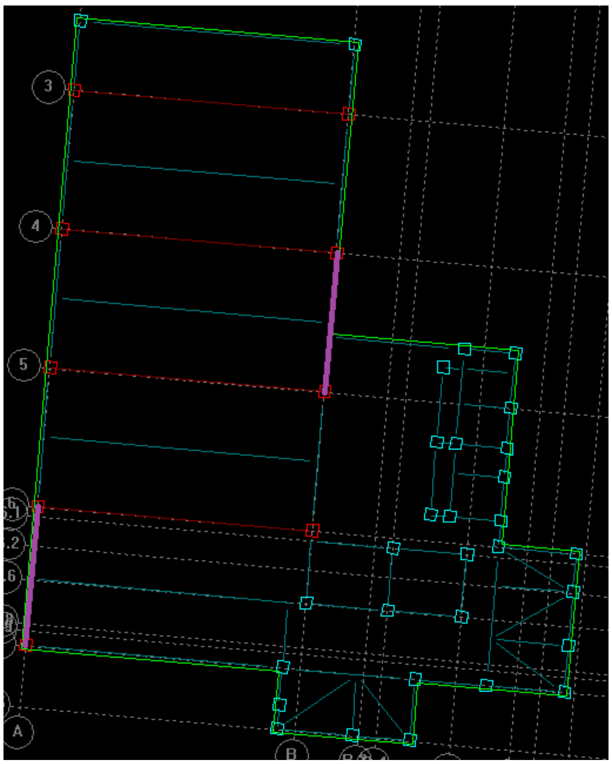
Table 20: Maximum Lateral Displacements and Story Drifts (E.5-12)

Maximum Wind Story Drift, N-S Direction				
	Story	**Story Drift (in)**	**Allowable Drift (in)**	**Adequacy**
Column E.5-12	7	N/A	N/A	N/A
	6	N/A	N/A	N/A
	5	0.1181	2.32	OK
	4	0.1708	1.86	OK
	3	0.1735	1.4	OK
	2	0.1756	0.94	OK
	1	0.0105	0.48	OK
Maximum Wind Story Drift, E-W Direction				
	Story	**Story Drift (in)**	**Allowable Drift (in)**	**Adequacy**
Column E.5-12	7	N/A	N/A	N/A
	6	N/A	N/A	N/A
	5	0.1685	2.32	OK
	4	0.2719	1.86	OK
	3	0.2975	1.4	OK
	2	0.2626	0.94	OK
	1	0.0155	0.48	OK

Lateral Spot Checks

Two spot checks were made where drift was the highest. One was made at the penthouse on story 7 in a braced frame system in the "Y direction". The braced frames are shown in purple in Figure19. The other was made on the same level in the "X direction" of a moment frame. The moment frames are shown in red on Figure 19.

Figure 19: Braced Frames on Story 7



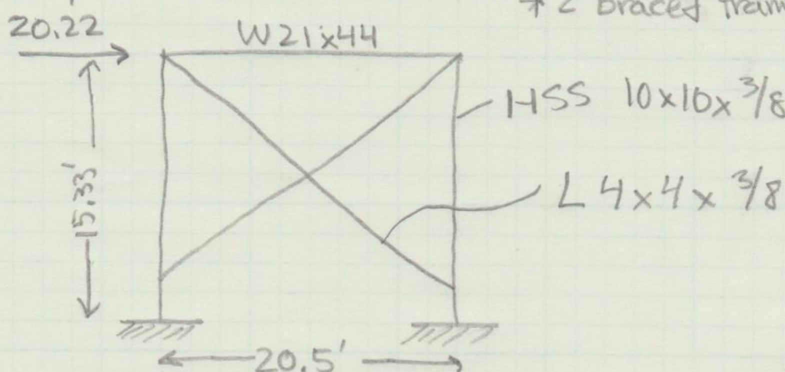
Both the braced frame and the moment frame systems on story 7 met drift requirements. They compared similarly to the drift values in the RAM output file. The braces, which were designed for a tension load of 20 kips were adequate for strength. The columns in the moment frame were adequate for flexure based on a tabulated value from AISC 14^{th} Edition. The calculated results follow.

Lateral Spot Checks

- Braced Frames at Story 7

Story Shear = 20.22 kips in Y direction

* 2 braced frames in system



* Beam does not contribute significantly in this frame

Column

$I_x = 202 \text{ in}^4$

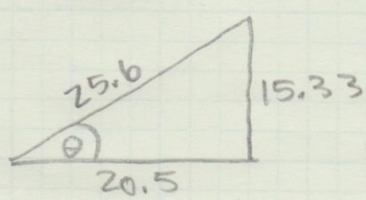
$$k_{COL} = \frac{12EI}{h^3}$$

$$= \frac{12(29000)(202)}{[(15.33)(12)]^3}$$

$k_{COL} = 11.29$ k/in

Brace

$$k_{brace} = \frac{AE}{L}\cos^2\theta$$



$\theta = 36.79°$

$A = 2.86 \text{ in}^2$

$$k_{brace} = \frac{2.86(29000)}{(25.6)(12)}\cos^2(36.79) = 173.15 \text{ k/in}$$

$$\text{Total Stiffness} = 2\left[2(11.29) + 173.15\right]$$

\# frames

$= 391.46$ k/in

$$\Delta = F/k = \frac{20.22}{391.46} = 0.0517 \text{ in}$$

This is less than the value calculated by RAM which was 0.0617 in.

However, it is still within allowable drift limit of $h/400 = 3.24''$

Plans called for braces to support 20 kips of tension.

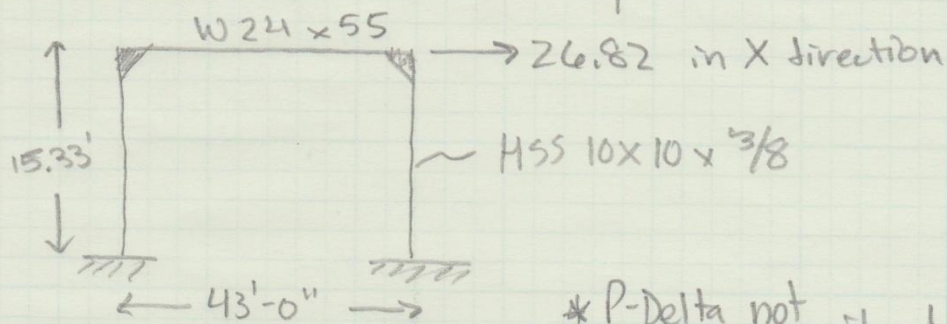
Alternatively,

20.22 k story shear / 2 braces = 10.11 k

20 k > 10.11 k applied

• Moment Frames at Story 7

4 Moment Frames in system



* P-Delta not considered

$K_{COL} = 11.29$ k/in

(4) 2 (11.29) = 90.32 k/in total stiffness

\# frames

$\Delta = F/k = 26.82/90.32$

$\Delta = 0.297''$

This is less than the value that RAM calculated which was 0.4665"

They both pass drift requirements of $h/400 = 3.24''$

Alternatively, an unbraced HSS10×10×3/8 can support $\phi M_n = 163$ k-ft

$26.82 (15.33) = M_u = 411.15$

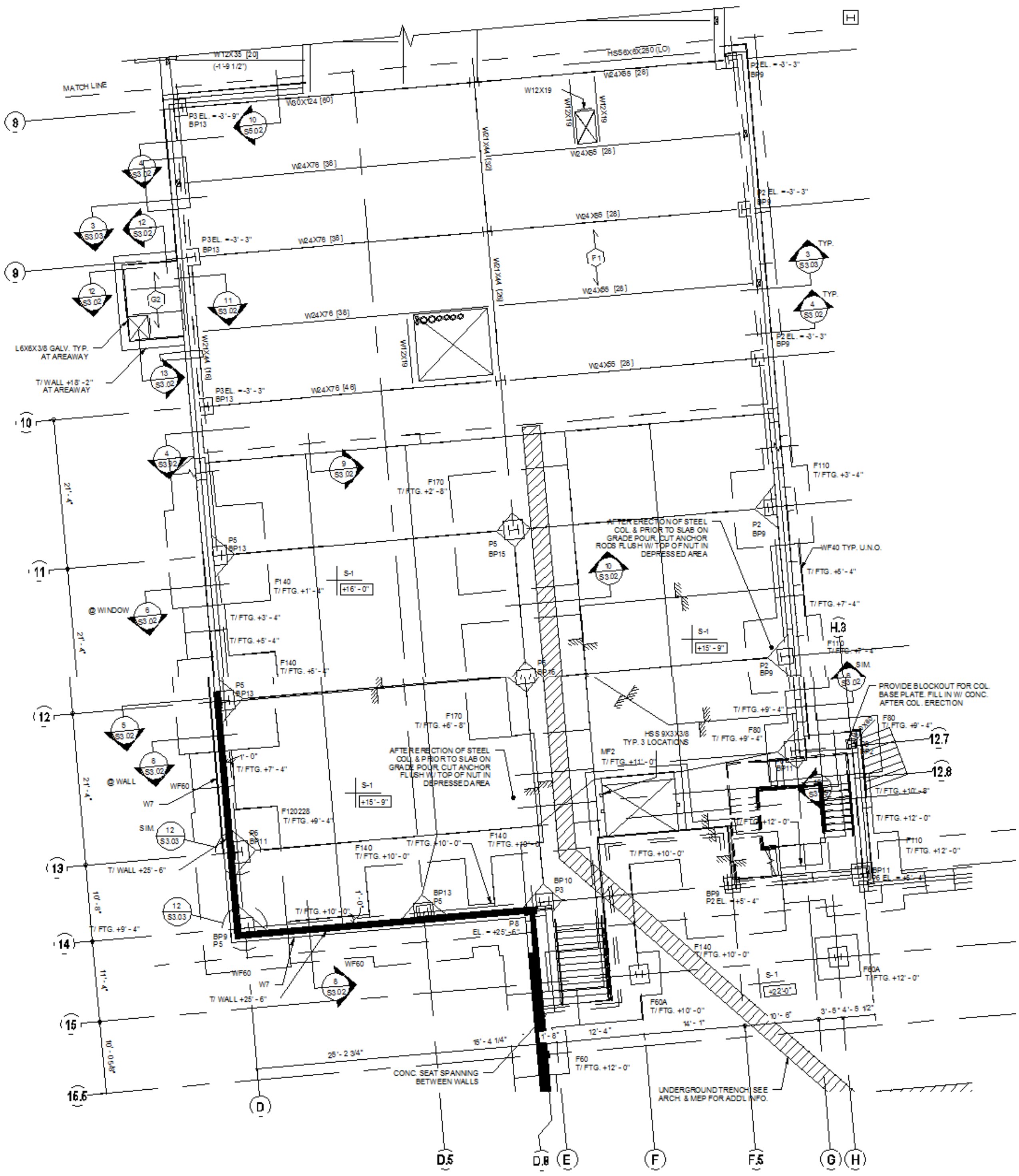
$8(\phi M_n) = 1304$ k-ft $> M_u$ OK

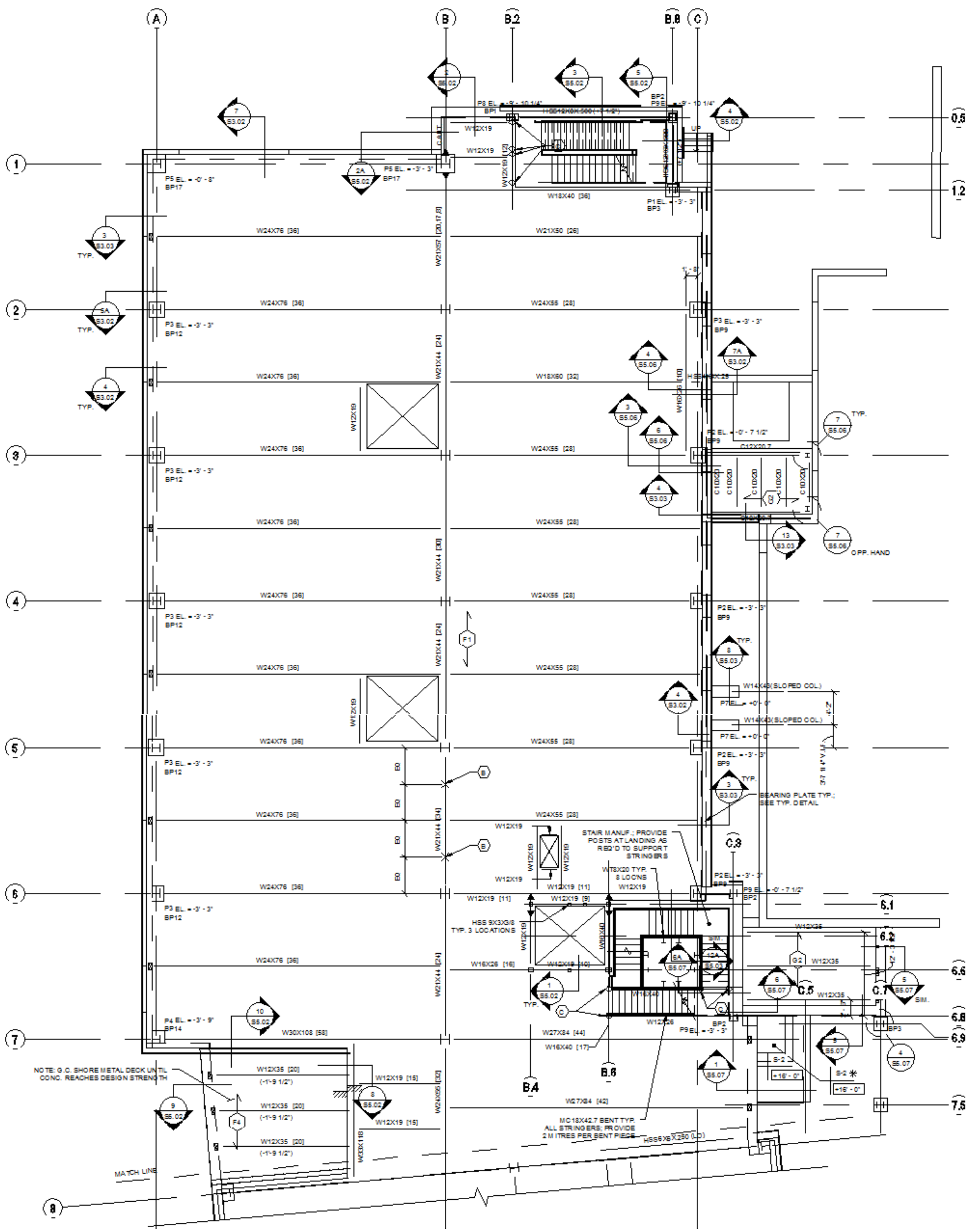
Conclusion

The RAM model produced yielded some practical results which were backed up by the lateral spot check calculations. A computer program should never be trusted blindly; especially one complicated enough to perform structural analysis. It seems that in this case though, RAM Structural System performed a relatively accurate analysis that met strength and serviceability requirements established by code. The frames in the system met drift requirements from ASCE 7-05, and the behavior of the building seemed realistic.

P-Delta effects are something which will need to be considered in future analysis, as gravity loads were neglected. They could produce much larger deflections of columns yielding in much larger drifts.

Appendix A-1




A
B
B.2
B.8
C
0.5
1
1.2
2
3
4
5
6
7
8
W24X76 [36]
W21X50 [26]
W24X55 [28]
W18X50 [32]
W18X40 [36]
W30X108 [58]
W12X35 [20]
W12X19 [15]
W27X84 [42]
W21X44 [24]
W12X19
TYP.
OPP. HAND
MATCH LINE
NOTE: G.C. SHORE METAL DECK UNTIL CONC. REACHES DESIGN STRENGTH
STAIR MANUF.: PROVIDE POSTS AT LANDING AS REQ'D TO SUPPORT STRINGERS
BEARING PLATE TYP.; SEE TYP. DETAIL
W14X43(SLOPED COL.)

Appendix A-2

ASCE 7-05: Section 6.5

V = 90 mph	(Figure 6-16)
Occupancy is 1490 > 500 for university, so TYPE III	(Table 1-1)
Importance TYPE III; V < 100 mph → therefore, I = 1.15	(Table 6-1)
Roughness Type B (Urban/Suburban)	(Section 6.5.6.2)

Figure A1: Plan View

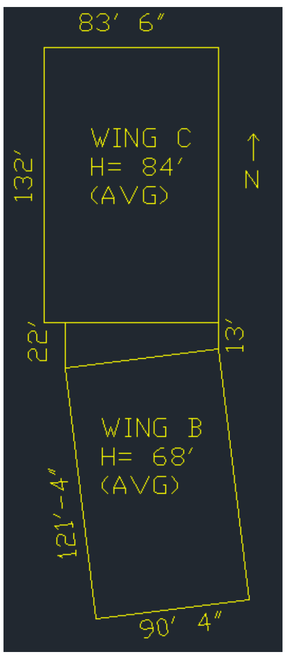
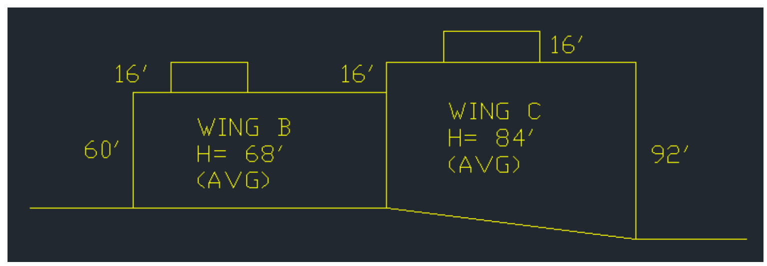


Figure A2: East Elevation



E/W: L = 86.9' B = 275.3'

$$H = \frac{(154)(84') + (121.3)(65')}{(154+121.3)} = 78'$$

N/S: L = 275.3' B = 86.9'

H = 100' to be conservative

- Building Category = III (Table 1-1)
- Exposure = B (Urban/suburban) (6.5.6.2)

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I \quad \text{(Eq. 6-15)}$$

- Determine z for top Level:

	E/W	N/S
$z =$	77'	108'-4"

- Determine K_z for Roof Level: (Table 6-3)

	E/W	N/S	
$K_z =$	.92	1.01	
$K_{zt} =$	1.0	1.0	(6.5.7)
$K_d =$	.85	.85	

$I = 1.15$ (Category III) (Table 6-1)

	E/W	N/S
$q_z =$	18.65 psf	20.47 psf

- Since $T_0 < 1 \text{ sec} \rightarrow G = .85$ (6.5.8.1)
 *Assuming $\#\text{stories}/10 = 6/10 = .6 = T_0$
- $GC_{pi} = \pm .55$ partially enclosed buildings (Fig. 6-5)

$$p = q G C_p - q_i (GC_{pi})$$

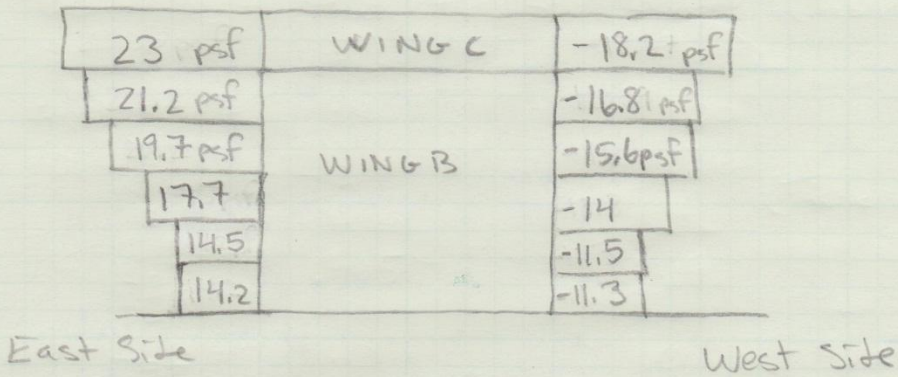
- East/West Pressures (Figure 6-6)

Windward $C_p = .8$

Leeward C_p is a function of L/B

$L/B = \frac{86.9}{275.3} = .316 \rightarrow C_p = -.5$

- Elevation:



- Sample Calc (wind ward)

$p = 18.65\,(.85)(.8) - (18.65)(\pm .55)$

$p = 22.94$ psf

- Elevation:

Leeward $C_p = -.24$