AMERICAN ART MUSEUM | NORTHEAST UNITED STATES

TECHNICAL REPORT 3

LATERAL SYSTEM ANALYSIS & CONFIRMATION DESIGN
ADVISOR: SUSTERSIC
NOVEMBER 12, 2012

EXECUTIVE SUMMARY

Technical Report 3 analyzes the lateral loads and resistance system of Renzo Piano's American Art Museum (AAM) through the re-creation and verification of the lateral loads and sizing of lateral elements. This is accomplished by using lateral force analyses contained in ASCE 7-05 for wind and seismic. After a careful and detailed analysis, it was determined that though the selected members are designed with adequate strength, it is extremely difficult to reconcile the differences between competing and opposing methods of lateral force analysis.

Wind loads were found using chapter 6 in ASCE 7-05. When presented with a building with geometry as complicated as AAM's (see Figure 1), chapter 6 requires that Wind Tunnel Testing be performed in lieu of the simplified procedure contained within. Technical Report 3 substitutes the appropriate wind tunnel analysis with ASCE 7's Analytical Procedure.

Similarly, a Modal Response Spectrum Analysis Procedure was performed on AAM by the design professionals, but the scope of Technical Report 3 limits the load analysis to the Equivalent Lateral Force Procedure contained in chapters 11 and 12 of ASCE 7.

Due to the intricacy of AAM, a computer model was constructed in ETABS for the purpose of completing a lateral analysis. The lateral system's vertical discontinuities and daunting size would have rendered a comprehensive ETABS model too difficult and tedious to use in assisting any deeper understanding of the building. Technical Report 3, therefore, analyzes the top portion of AAM, using level 6 as its base. This truncation drastically affects the torsional properties of the building, and is discussed further in the Lateral Analysis portion of this report.

Figure 1: Rendering of the Building (SW Corner)

Note: cover image, renderings, and CDs are used with the permission of RPBW

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INTRODUCTION

The American Art Museum (AAM) will serve as a replacement to the owner's current facility in New York City. Figure 2 shows AAM's new location in a vibrant district where aging warehouses, distribution centers, and food processing plants are being renovated and replaced by art galleries, shops, and offices. AAM will stand in place of several such warehouses, and will provide a magnificent new southern boundary to the city's recently renovated elevated park, which terminates on the eastern edge of the site.

Figure 2: Arial map showing urban location along river (www.maps.google.com)

Renzo Piano's approach to AAM's design and architecture blends a contemporary architectural style with the historical development of the city. The large cooling towers and outdoor terraces that step back towards the river on the west trace their roots back to the industrial revolution and its local impact. These outdoor terraces will also provide views of the southern skyline and space for outdoor exhibits and tall sculptures while being protected from any wind by the higher portions of the building's west side. Alternately, the large cantilevers, insets, large open spaces, exposed structural steel, and modular stainless plate cladding show no attempt to camouflage AAM with the more historical surrounding buildings.

AAM's façade is comprised of the aforementioned steel plate, pre-cast concrete, and glazing using a standard module of 3'-4" (about 1m) (shown in Figure 3). While most of the façade components are broken at each story, the long steel plates stretch 60' on the southern wall from levels 2 to 6 and from 6 to 9.

This new facility is a multi-use building with gallery and administration space, two café/restaurants, art preservation and restoration spaces, a library, and a 170-seat theater. Public space including the theater, classrooms, restaurants, and galleries are located on the south half of the building on the ground level and levels 5 through 8. Mechanical, storage, conservation, offices, and administration are dispersed on the north side at each level. The 220,000 square-foot AAM will stand 148ft tall and cost approximately $266 million. Construction began in May 2011 and is expected to be complete in December 2014.
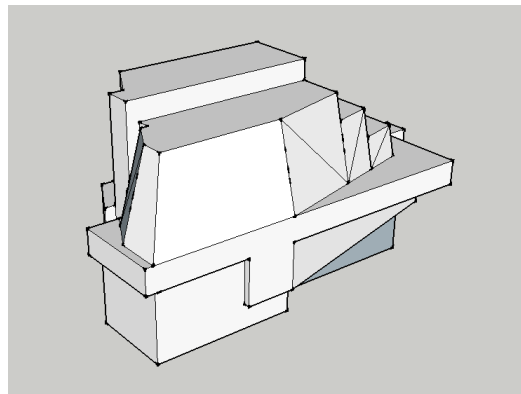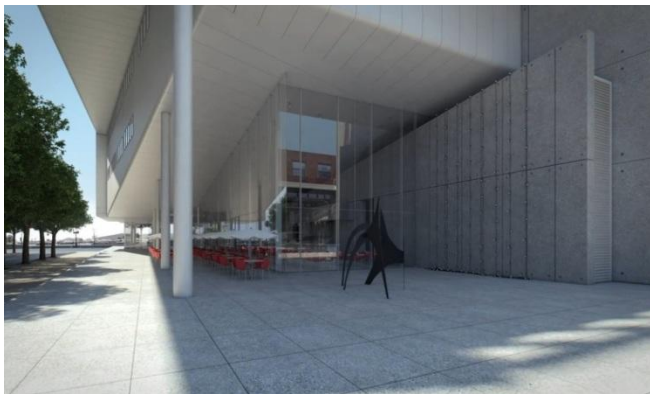

Figure 3 (left): Rendering shows façade at SE corner entrance
Figure 4 (right): Sketchup model shows building's complex geometry from the SW corner

STRUCTURAL SYSTEMS

OVERVIEW

AAM sits on drilled concrete caissons encased in steel with diameters of either 9.875" or 13.375" with pile caps. From the foundation level at 32' below grade, 10 levels rise on steel columns and trusses. Each floor will be supported by a steel-composite system. The lateral system consists primarily of braced frames spanning several stories. At some levels however, the floor system uses HSS diagonal bracing between beams and girders to create a rigid diaphragm that also transfers the lateral loads between staggered bracing. Moment frames are used for localized stability purposes. While masonry is used in AAM it is used for fire rating purposes only.

The building classifies as Occupancy Category III. This is consistent with descriptions of "buildings where more than 300 people congregate in one area" and "buildings with a capacity greater than 500 for adult education facilities."

FOUNDATIONS

URS Corporation produced the geotechnical report in February 2011 to summarize the findings of several tests and studies performed between 2008 and 2010. They summarize that while much of the site is within the boundaries of original shoreline, a portion of the western side is situated on fill-in from construction. They explain further that the portion that was formerly river has a lower bedrock elevation and higher groundwater. Due to the presence of organic soils and deep bedrock, URS suggested designing a deep foundation system and provided lateral response tests of 13.375" diameter caissons socketed into bedrock.

The engineers acted on the above suggestions and others. The caissons are specified with a 13.375" diameter of varying concrete fill and reinforcement to provide different strengths to remain consistent with URS Corp's lateral response tests. Low-capacity caissons (9.875" diameter) are individually embedded in the pressure slab, while typical and high-capacity caissons are placed in pile caps consisting of one or two caissons. The high-capacity caissons are always found in pairs and are located beneath areas of high live load or where cantilevers are supported. For a complete layout and caisson schedule, see FO-100 in Appendix A.

A pressure slab and the perimeter secant-pile walls operate in tandem to hold back hydrostatic loads created by the soil and groundwater below grade. The walls vary between 24" and 36" and are set on 6'-6" wall footers and caissons. These are isolated from the pressure slab. The cellar level floor slab consists of a 5" architectural slab-on-grade by a 19" layer of grave on top of a 24" pressure slab (Figure 5).
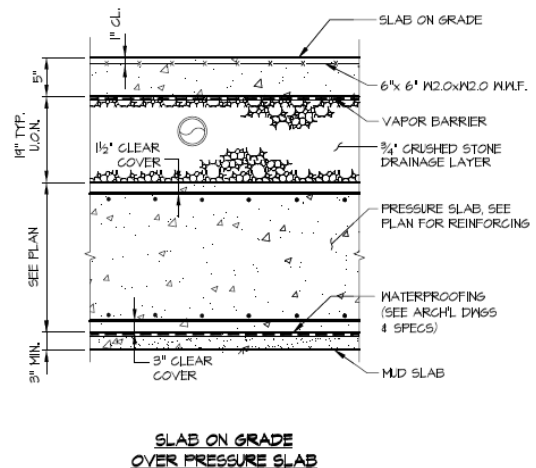

Figure 5: Pressure slab detail (S-201)

GRAVITY SYSTEM

FLOOR SYSTEM

A surprisingly regular floor layout contrasts the obscure geometry of the building (Figure 6). The engineers managed to create a grid with spacings of roughly 20' (E-W) and 30' (N-S), where the 20' sections are divided by beams which support the floor decking running E-W. Beams that do not align with the typical perpendicular grid indicate a change of building geometry below or above. Each beam is designed for composite bending with the floor slab.
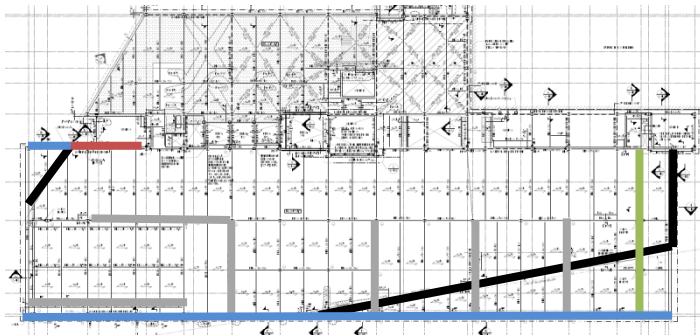

Figure 6: Level 5 framing plan showing regular layout against building footprint (S-105)

▬▬	Gravity Trusses (above)
▬▬	Gravity Trusses (below)
▬▬	Plate Girder (d=46")
▬▬	Lateral Braced Frames (part of gravity)
▬▬	Outline of Building Below

Four slab/decking thicknesses are called for depending on deck span and loading, all on 3"-18 gauge composite metal deck. The most common callout is 6.25" (total thickness) lightweight concrete. This provides a 2-hour fire rating. 7.5" normal weight is used on level 1 for outdoor assembly spaces and the loading dock, and 9" normal weight is used for the theater floor. The roof above the level 9 mechanical space calls out 5.5" composite.

While the layout can be considered relatively consistent, the beam sizes and spans selected suggest a much more complicated floor system. Though a typical bay spans 20'-30', the gallery floors (levels 6-8) span over 70'. The shorter spans require filler beams as small as W14x26, but the longer spans supporting the upper gallery levels require beams as large as W40x297s for web openings. In several places welded plate girders are specified at depths from 32.5" to 72." The plate girders are used as transfer large loads and moments as propped cantilevers, especially from gravity trusses and lateral braced frames shown in Figure 7.

FRAMING SYSTEM

Cantilevers on the south side of AAM are supported by 1 or 2-story trusses, typically running in the N-S direction. One large gravity truss runs along the southernmost column line between levels 5 and 6 to support the cantilever on the south-eastern corner of the building.

While the vast majority of columns are W12x or W14x shapes, some of the architecturally exposed steel vertical members are HSS shapes, pipes, or solid bars. Furthermore, the gravity load path goes up vertically and horizontally nearly as much as it flows directly down a column to the foundation. Figure 8 shows how large portions of the southern half of AAM's levels 3 and 4 are hung from trusses and beams on the level 5 framing system.
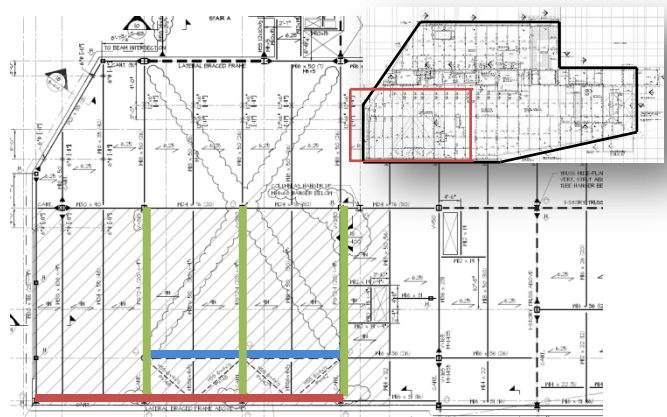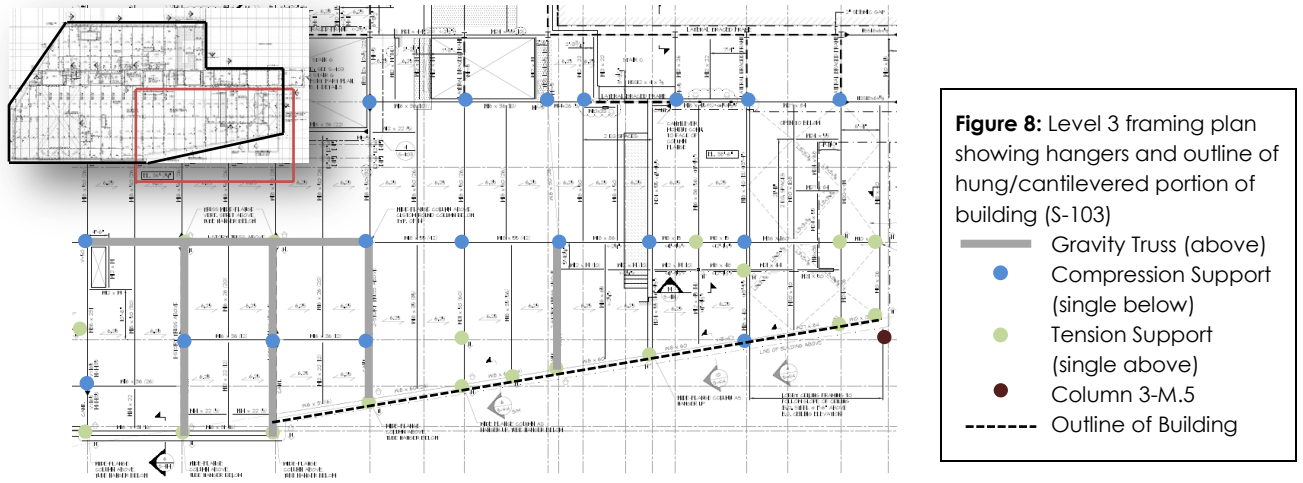

Figure 7: Level 3 framing plan showing transfer girders and lateral braced frames (S-103)

▬▬	Lateral Braced Frame (above)
▬▬	Lateral Braced Frame (below)
▬▬	Plate Girder (d=46")

Renzo Piano's designs often expose structural steel, providing an extra constraint on the design team. One example is column 3-M.5 which supports level 5 from the outdoor plaza below. The foundation column below grade specifies a W14x311, a typical shape for a column, but the architecturally exposed structural steel is called out as 22" diameter solid bar. A unique analysis would be required for a solid bar acting as a column, as AISC XIII does not have provisions for such a selection in its tables or specifications.

Figure 8: Level 3 framing plan showing hangers and outline of hung/cantilevered portion of building (S-103)

⎯⎯⎯ Gravity Truss (above)
● Compression Support (single below)
● Tension Support (single above)
● Column 3-M.5
------- Outline of Building

LATERAL SYSTEM

AAM's lateral system is as complicated as its gravity systems. A combination of moment and concentric lateral braced frames stagger up the building, transferring lateral loads via diagonal bracing within the floor diaphragms on level 3 for the southern portion and 5 for the northern portion as shown in Figure 9. Most of the braced frames terminate at ground level, but three extend all the way down to the lowest level. Those braces that terminate at upper floors transfer uplift through columns that extend underneath them. Bracing members are comprised mostly of W10x, 12x, or 14x shapes in X-braces or diagonals. There are, however, HSS shapes are used with K-braces. An enlarged floor framing plan showing the braced frames at level 5 is provided in Figure 10 below.
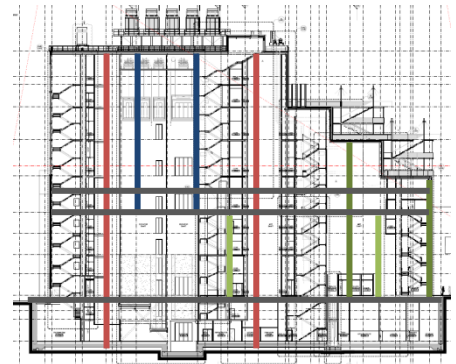

Figure 9: Section cut showing N-S braced frames at staggered heights (A-212)

Figure 10: Level 5 Framing Plan Showing Lateral System (S-105)

⎯⎯⎯ Lateral Braced Frame
⎯⎯⎯ Gravity Truss that Contributes to Lateral System
▨ Floor System with Diagonal Bracing

DESIGN CODES & STANDARDS

The design codes listed for compliance of structural design can be inferred from drawing S-200.01 and Specification Section 014100.2.B:

- International Code Council, 2007 edition with local amendments including:
  - Building Code
  - Fire Code
- ASCE 7-05: Minimum Design Loads for Buildings and other Structures
- ACI 318 -08: Building Code Requirements for Structural Concrete (LRFD)
- AISC XIII: Specifications for Structural Steel Buildings (LRFD)
- AWS D1.1: American Welding Society Code for Welding in Building Construction

Other codes not applicable to the structural systems of the building can be found in the specifications.

MATERIALS SPECIFICATIONS

The different materials specifications are summarized in Figure 11 below. Additional information can be found on drawing S-200.01 in Appendix A.

Materials Specifications						
Concrete & Reinforcement			Structural Steel			
Wt	Use	f'c (psi)	Shape	ASTM	Gr.	Fy (ksi)
LW	Floor Slabs (typ)	4000	Wide Flange	A992	-	50
NW	Foundations (walls, slab, pile caps, grade beams)	5000	Hollow Structural	A500	B	46
			Structural Pipe	A501/A53	-/B	30
NW	Composite Column Alternate	8000	Channels	A36	-	36
NW	Other	5000	Angles	A36	-	36
			Plates	A36	-	36
Gr.	Use	ASTM	Connection Bolts	A325-SC	-	80
70	Reinforcement	A185	(3/4") Anchor Bolts	F1554	36	36
70	Welded Wire Fabric	A185				
Figure 11: Summary of Structural Materials Specifications in AAM						

GRAVITY LOADS

LIVE LOADS

Typically, one would expect to see Live Loads calculated from ASCE 7 minimums (ASCE 7 Table 4-1). The structural narrative explains that much of AAM does not fit with any ASCE 7 descriptions of use types, so the engineers have provided their own design loads summarized in Figure 12. Additionally the engineers created a live load plan on S-200.01 in Appendix A which shows areas of equal live load on each floor.

The engineers, in a desire for maximum flexibility of the gallery spaces, elected to conservatively design the AAM-specific spaces for live loads, while being consistent with ASCE 7 minimums for more common areas.

LL Schedule Designation		ASCE 7 Designation	
Use	LL	LL	Description
Gallery - Typical	100	100	Assembly Area
Gallery - Level 5	200	100	Assembly Area
Testing Platform	200	150	Stage Floors
Offices	50	50	Offices
Private Assembly/ Museum Use	60	n/a	n/a
Auditorium - Movable Seating	100	100	Theater - Moveable Seats
Compact Storage	300	250	Heavy Storage
Art Handling & Storage	150	125	Light Storage
Largo and Loading Dock	AASHTO HS-20	250	Vehicular Driveways
Stairs and Corridors	100	100	Stairs and Exit Ways
Lobby and Dining	100	100	Lobby Assembly
Mech Spaces Levels 2, 9	150	n/a	n/a
Mech Spaces Cellar	200	n/a	n/a
Roof - Typical	22 + S	20	Roof - Flat

Figure 12: Comparison of design live loads and ASCE 7 minimum live loads

DEAD LOADS

Because the live loads (above) are so high, the design engineers were very precise in their dead load calculations. Similar to the live loads, the diversity of different use types and load requirements have led to a congruent variety of dead load arrangements in structural steel weight, concrete density, MEP requirements, partitions, pavers, roofing, and other finishes. A total of 37 different dead load requirements, arranged by use and location, are listed in the Dead Load Schedule on drawing S-200.01. These range from 76 PSF to 214 PSF. Since Technical Report 3 analyzes the upper floors of AAM, using Level 6 as its base, the total calculated weight is 5,849k (2,925 tons) with a total area of 53,100 square feet. This data is consistent with the area and weight information provided by Turner Construction and the results calculated in Technical Report 1 for this portion of AAM. The complete revised dead load and area calculations can be found in Appendix B.

SNOW LOADS

ASCE 7-05 was used to calculate the snow loads for AAM in consistency with the wind and seismic loads. This code was used because it is the most recent publication of ASCE 7 per the specifications (see Design Codes & Standards above). Figure 13 details the summary of this procedure, comparing the Snow Load Parameters on drawing S-200.01 to the City Building Code/ASCE 7.

Snow Load Comparison		
Design Parameters		ASCE 7 -05
Pg	25	25
Ct	1	1
Is	1.15	1.15
Ce	1	1
Pf	20.1	20.1
20 Is	22	23

Figure 13: Snow Loads

ASCE 7-05 equation 7-1 (section 7.3) states that where the ground snow load exceeds 20 PSF, the flat roof load value must not be less than $(20)I_s$. 22 PSF, the design flat roof load, is not in accordance with ASCE 7's minimum according to equation 7-1 of 23 PSF. It is important to note that the step-back terraces where drifting is a concern are designed for 100-200 PSF of live load, and it is unlikely that the building will experience snow loads exceeding those live loads. Further information on the snow load calculations can be found in Technical Report 1.

LATERAL LOADS

OVERVIEW

A preliminary calculation of the wind and seismic loads was completed for Technical Report 1. Those values, however, contained minor flaws and inconsistencies which had to be revisited for the lateral system analysis below. Both the wind and seismic load calculations executed in Technical Reports 1 & 3 use a series of simplifying assumptions allowing the analyses contained within ASCE 7-05 (Analytical and Equivalent Lateral Force Procedures, respectively). The design engineers, however, having greater resources and experience, used Wind Tunnel Testing and Modal Response Spectrum Analysis. This inconsistency in analysis procedures and assumptions will lead the calculations contained in Technical Report 3 to be conservative. Inherent torsion was also calculated per ASCE 7-05 for each loading and applied to AAM. Although the designers determined that seismic loads controlled both base shear and overturning moment in their analyses, The N-S wind case controls base shear and seismic controls overturning in ASCE 7-05 using simplifying assumptions.
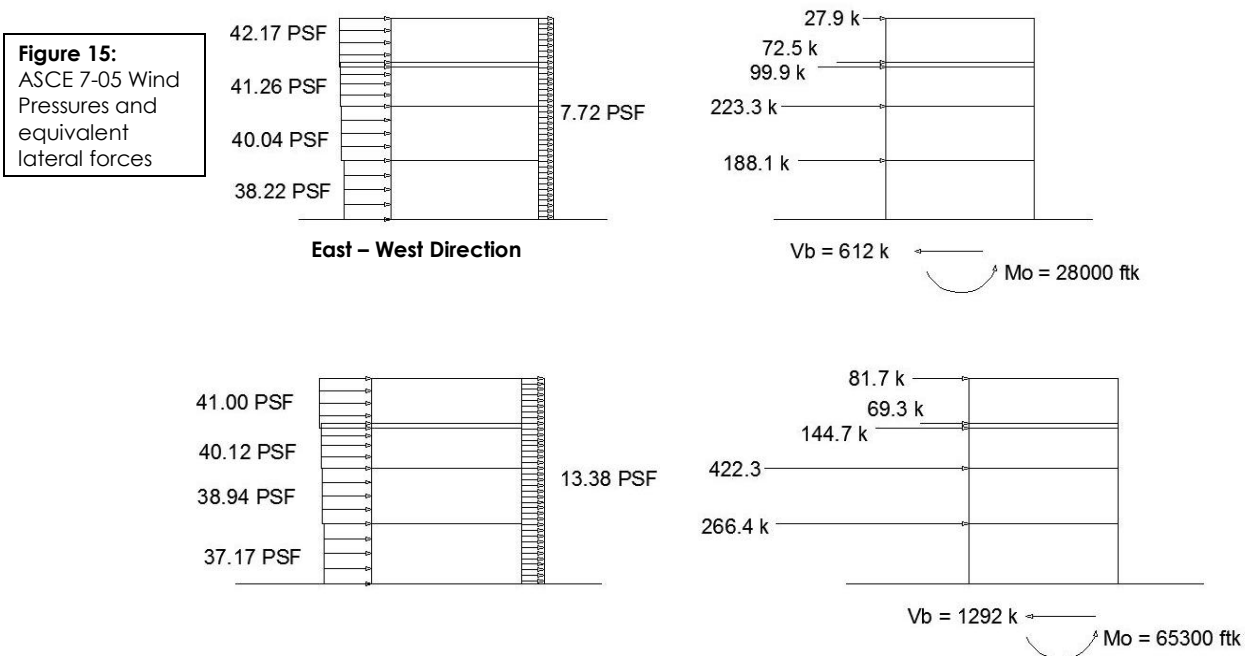
WIND LOADS

As mentioned above, the wind loads in both directions were found using Analytical Procedure (Method 2) in ASCE 7-05 chapter 6 using the same simplifying assumptions discussed in Technical Report 1. Using the factors in Figure 14 below (calculations in Technical Report 1), the wind pressures were calculated between 45 PSF and 55 PSF (Figure 15). The design professionals explained that Wind Tunnel Testing returned values of between 30 PSF and 45 PSF, making the Analytical Procedure about 12PSF conservative (a difference of about 20% - 25%).

Wind Factors		
	E - W	N - S
$G_f =$	0.89	0.85
GC pi =	0.55	-
Cp =	-0.3	-0.5
Kd =	0.85	-
Kzt =	1.0	-
I =	1.15	-

Figure 14: Wind factors for ASCE 7-05 calculations

Figures 15 below summarize the revised wind load calculations. The base shears and overturning moments were found for both the North-South (Y) and East-West (X) directions by creating equivalent lateral forces at each story level. More detailed calculations provided in Appendix C show that AAM must resist wind across a much greater surface area in the N-S direction than the E-W. This difference leads to the much greater base shear (1300k which controls) and overturning moment in the N-S direction.

Figure 15: ASCE 7-05 Wind Pressures and equivalent lateral forces

42.17 PSF
41.26 PSF
40.04 PSF
38.22 PSF
7.72 PSF

East – West Direction

27.9 k
72.5 k
99.9 k
223.3 k
188.1 k

Vb = 612 k
Mo = 28000 ftk

41.00 PSF
40.12 PSF
38.94 PSF
37.17 PSF
13.38 PSF

81.7 k
69.3 k
144.7 k
422.3
266.4 k

Vb = 1292 k
Mo = 65300 ftk

North –South Direction

SEISMIC   LOADS

The seismic loads in Technical Report 3 were calculated using the Equivalent Lateral Force Procedure found in ASCE 7-05 chapters 11 and 12. As mentioned above, this method is in contrast to the structural engineer's Modal Response Spectrum Analysis, which is considered to have a higher degree of accuracy (ELF is more conservative). The analysis contained in Technical Report 3, however, uses the assumptions provided on drawing S-200.01. Figure 16 shows which values were provided by the engineers and which were supplements needed to complete the ASCE 7-05 analysis.

These values were used alongside the revised dead load calculations to find the equivalent lateral forces, base shear, and overturning moment summarized in Figure 17 below. Further calculations can be found in Appendix C. The revised base shear was found to be 1276k for floors 6-RN, much higher than the provided base shear of 946 for the whole building, which can be explained by the different procedures. The overturning moment of 158,500 ft-k controls for both wind and seismic analysis.

Seismic Design Criteria			
S-200.01		ASCE 7-05	
Sds	0.65	Ta (s)	0.9
Sd1	0.13	Cu	1.7
I	1.25	T (s)	1.53
R	3	TL (s)	6
W (k)	5849		
Cs	0.0602		

Figure 16: Seismic Design Criteria

192 k

90 k

95 k

456 k
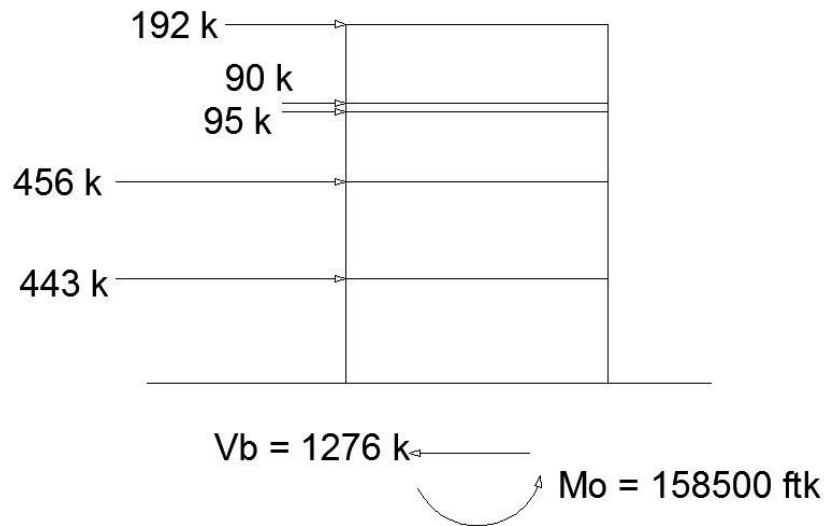
443 k

Vb = 1276 k

Mo = 158500 ftk

Figure 17: Equivalent Lateral Force Procedure Summary

LATERAL SYSTEM ANALYSIS

OVERVIEW

An in-depth analysis of AAM's lateral system was performed using the ASCE 7-05 loads calculated above. Due to the complexity and size of the AAM project, it was decided that only the top half of the building should be modeled using level 6 as the base. Hand calculations for the centers of rigidity and mass verified that the building was modeled correctly in ETABS (and would thus distribute the loads correctly), and that lateral deflections and story drifts are acceptable by code. Each lateral braced frame and moment frame was modeled twice in SAP2000 to find the stiffness in each direction assuming tension-only braces.
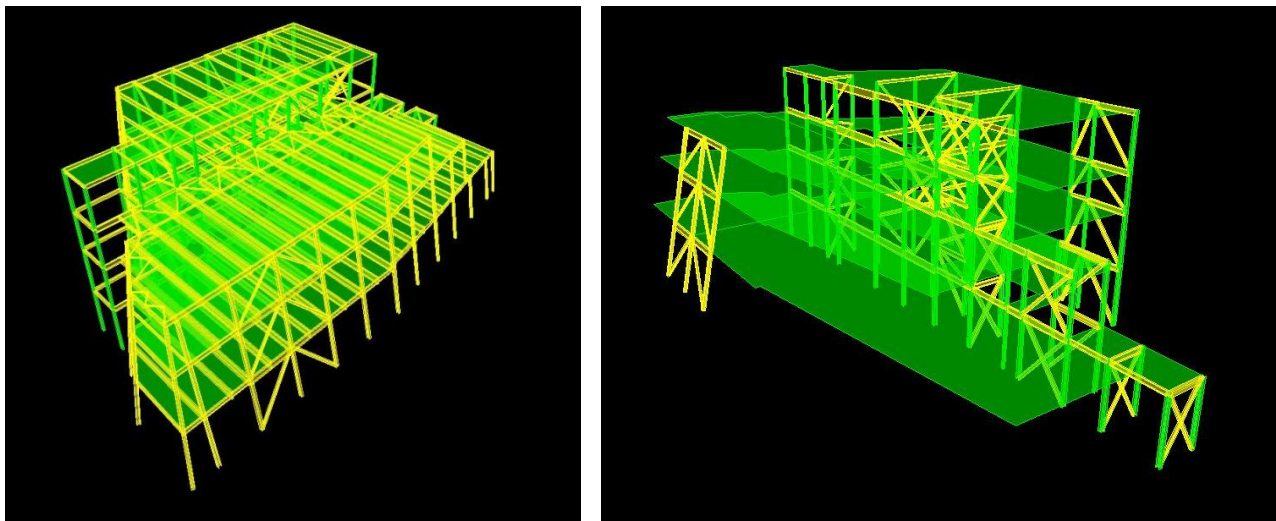
ETABS MODEL

Figure 18: ETABS model showing all members from SW corner (Left) and lateral system with diaphragms from SE corner (Right)

The steel frame shown in Figure 18 was modeled in ETABS on levels 7, 8, 9, RS, and RN according to the drawings. Columns were assumed to be moment connected vertically, and all the beams are pinned unless otherwise indicated. Each column was assumed to be on a pinned support. The drawings for each level, column schedule, and braced frame elevations can be found in Appendix A.

An accurate use of the diaphragm required some modeling gymnastics. Each diaphragm was modeled as an undefined area that covered the correct footprint of each level. Then an additional area mass was added to define the correct weight of that level. The area mass was found by removing the structural steel weight from the dead load assignments on drawing S-200.01 and converting that PSF weight to a lb-ft unit mass input at each level. Areas were then defined as separate rigid diaphragms.

Loads were applied to the center of mass for seismic and the center of the face for wind at each diaphragm and additional moments were defined about the Z-axis as required. ETABS automatically assigns insertion points such that the top of the steel aligns with the story elevation. P-delta effects were considered in the analysis.

BUILDING PROPERTIES

A series of hand calculations were performed parallel to the ETABS model to ensure and validate the accuracy of its output. Verification of the centers of mass and rigidity ensure that no obscure torsional effects will significantly alter the results of this analysis. Since no two levels have an identical layout, the stiffness and centers of mass and rigidity were analyzed on Level RN due to its simple geometry and easily calculable areas.

STIFFNESS AND CENTER OF RIGIDITY

Before calculating the centers of mass and rigidity, SAP2000 was used to find the stiffness of each lateral braced frame/moment frame in AAM, shown in drawings S-120 through S-126 in Appendix A. As mentioned above, the frames were analyzed using a 1000k horizontal force at the top level using only tension braces. Compression braces are assumed to buckle. The location and absolute stiffness for each frame is displayed in Figure 19.
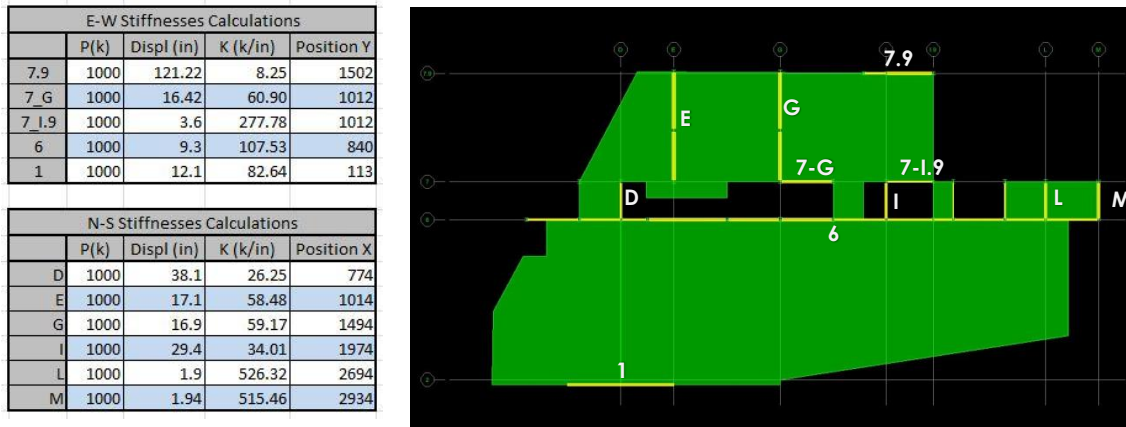

E-W Stiffnesses Calculations			
	P(k)	Displ (in)	K (k/in)	Position Y
7.9	1000	121.22	8.25	1502
7_G	1000	16.42	60.90	1012
7_I.9	1000	3.6	277.78	1012
6	1000	9.3	107.53	840
1	1000	12.1	82.64	113

N-S Stiffnesses Calculations			
	P(k)	Displ (in)	K (k/in)	Position X
D	1000	38.1	26.25	774
E	1000	17.1	58.48	1014
G	1000	16.9	59.17	1494
I	1000	29.4	34.01	1974
L	1000	1.9	526.32	2694
M	1000	1.94	515.46	2934

Figure 19: Lateral braced frame/moment frame stiffness and location on Level 7

Once the absolute stiffness of each lateral resisting frame was established, the center of rigidity for each floor was found using only the frames engaged by each level. Figure 20 summarizes the hand-calculated centers of rigidity. The stiffest frames are 6 and 7 (7-G and 7-I.9 are connected by simple beams on levels 7 and 8) in E-W resistance while frames E and G are the stiffest in N-S resistance. Frames L, M and 7.9 are considered to be outliers in this model due to their seemingly excessive or lacking stiffness. These will be discussed in the Additional Stiffness and Torsion Considerations section of Technical Report 3. The center of rigidity for Level RN is highlighted in Figure 20.

Centers of Rigidity						
Level	((K))	((KX))	X	((K))	((KY))	Y
RN	177.9115	235158.4	1322	176.6777	164345.4	930
RS	177.9115	235158.4	1322	190.1715	106428	560
9	177.9115	235158.4	1322	259.3223	177107.5	683
8	704.2272	1653053	2347	537.1001	455099	847
7	1219.691	3165424	2595	537.1001	450490.5	839

Figure 20: Hand-calculated centers of rigidity at each level

The relative stiffnesses by level were also derived from the absolute stiffness information. Figure 21 below summarizes both how much direct shear force and how much moment is resisted by a given frame. This figure is organized such that "0.0"s appear where that level does not engage a particular brace. A level may not engage a resisting frame for one of two reasons; either the frame does not extend the height of the model or there is no direct mechanical interaction between a level and a particular frame (see Figure 22).

RELATIVE DIRECT STIFFNESS (%)											
N-S	D	E	G	I	L	M	E-W	1	6	7_G	7_I.9	7.9
RN	14.8	32.9	33.3	19.1	0.0	0.0	RN	0.0	60.9	34.5	0.0	4.7
RS	14.8	32.9	33.3	19.1	0.0	0.0	RS	43.5	56.5	0.0	0.0	0.0
9	14.8	32.9	33.3	19.1	0.0	0.0	9	31.9	41.5	23.5	0.0	3.2
8	3.7	8.3	8.4	4.8	74.7	0.0	8	15.4	20.0	11.3	51.7	1.5
7	2.2	4.8	4.9	2.8	43.2	42.3	7	15.4	20.0	11.3	51.7	1.5

RELATIVE ROTATIONAL STIFFNESS (%)											
N-S	D	E	G	I	L	M	E-W	1	6	7_G	7_I.9	7.9
RN	5.1	14.8	22.1	16.8	0.0	0.0	RN	0.0	22.6	15.4	0.0	3.1
RS	5.9	17.4	25.9	19.7	0.0	0.0	RS	4.7	26.4	0.0	0.0	0.0
9	4.9	14.4	21.4	16.3	0.0	0.0	9	3.1	21.9	14.9	0.0	3.0
8	1.0	2.8	4.2	3.2	67.3	0.0	8	0.5	4.3	2.9	13.3	0.6
7	0.6	1.6	2.4	1.9	39.2	41.8	7	0.1	2.5	1.7	7.8	0.3

Figure 21: Relative stiffnesses by level
Note: direct stiffness is relative by direction and level while rotational stiffness is relative by level only
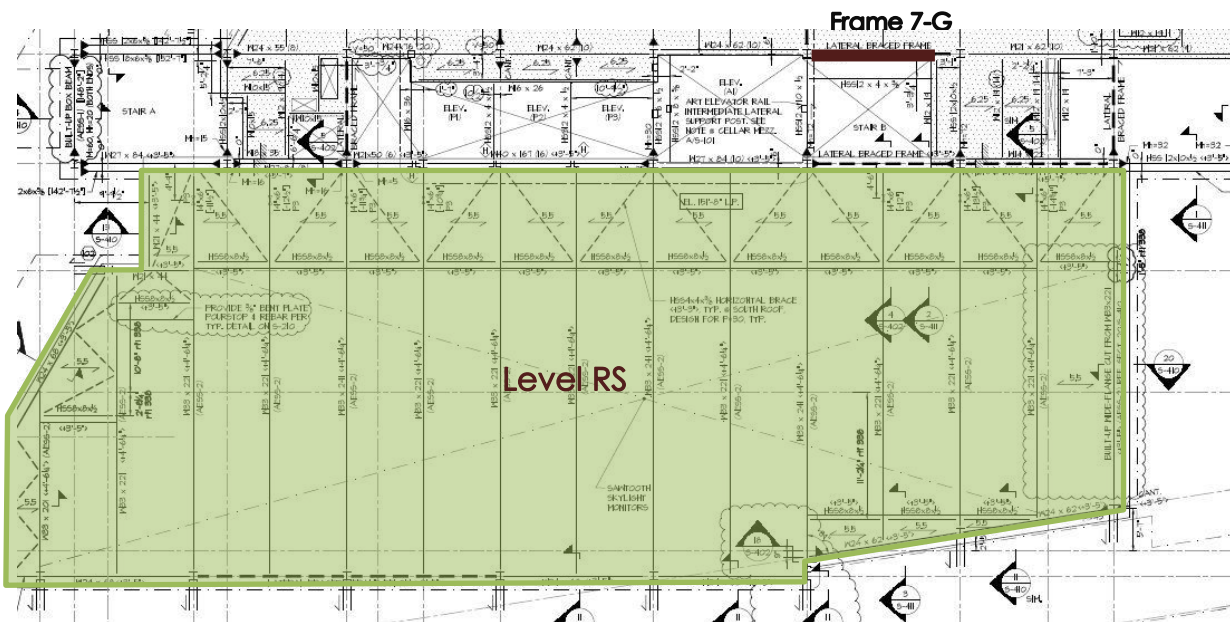

Figure 22: Level RS does not engage Lateral Braced Frame 7-G (S-109)

CENTER OF MASS

As mentioned above, the simple geometry of Level RN makes it the best level with which to verify the ETABS model. The two different floor types found on Level RN have different masses, and were analyzed accordingly. Figure 23 shows the simple shapes by which the center of mass for the whole floor was constructed using the increments. Calculations of the individual centers of masses follow in Figure 24. The center of mass was found by dividing the weight distribution (K*Xabs or K*Yabs) by the total weight (Total K) of the floor.
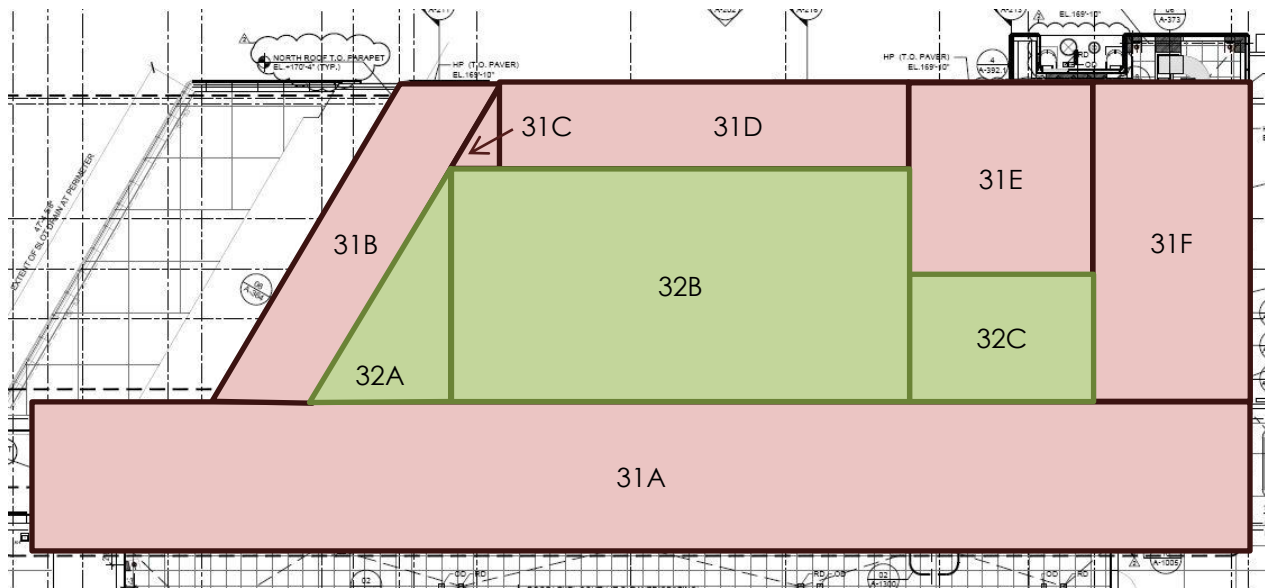

Figure 23: Diagram of area and COM calculations for Level RN (A-109R)

Center of Mass Calculation									
		Area		Weight		WX	WY	Center of Mass	
Type	Label	SQ in	SQ ft	PSF	Total K	K*Xabs	K*Yabs	X	Y
31	A	316308	2197	87	191	242223	176961	1405	1155
sqft	B	80178	557	87	48	40375	61084		
4385	C	4646.4	32	87	3	2887	4013		
	D	75802	526	87	46	62749	66405		
	E	48384	336	87	29	53495	41685		
	F	106074	737	87	64	133332	80813		
32	A	38367	266	101	27	24489	30590		
sqft	B	268002	1861	101	188	250287	225756		
2788	C	95040	660	101	67	121988	78459		
	Σ		7172		663	931824	765766		

Individual Area Centers of Mass									
		Origins		Ends		COM Relative		COM Absolute	
Type	Label	X	Y	X	Y	X	Y	Xabs	Yabs
31	A	348	840	2187	1012	920	86	1268	926
	B	613	1012	1054	1510	221	249	834	1261
	C	977	1390	1054	1510	52	40	1028	1430
	D	1054	1390	1686	1510	316	60	1370	1450
	E	1686	1342	1974	1510	144	84	1830	1426
	F	1974	1012	2187	1510	107	249	2081	1261
32	A	774	1012	977	1390	136	125	910	1137
	B	977	1012	1686	1390	355	189	1332	1201
	C	1686	1012	1974	1342	144	165	1830	1177

Figure 24: Incremental center of mass calculations for Level RN

COMPARING CALCULATED COM/COR TO ETABS COM/COR

Once both the centers of mass and rigidity could be established for Level RN, they were compared to the ETABS model shown in figure 25. The center of mass calculations were nearly identical with a difference of 9" in the X-direction (0.5%) and 6" in the Y-direction (0.9%). An observation of the center of rigidity accuracy, however, reveals a 34" difference in the X-direction (1.9%) and a 50" difference in the Y-direction (7.5%). Figure 26 below shows the locations of each point on Level RN. The ETABS model and hand calculations are returning values within 10% of each other and are therefore deemed acceptable.

ETABS vs Caluclated COM/COR Comparisson				
	Mass		Rigidity	
	X	Y	X	Y
Calculated	1405	1155	1322	930
ETABS Model	1414	1149	1356	880
Difference	9	6	34	50
Overall	1839	670	1839	670
% Difference	0.5%	0.9%	1.9%	7.5%

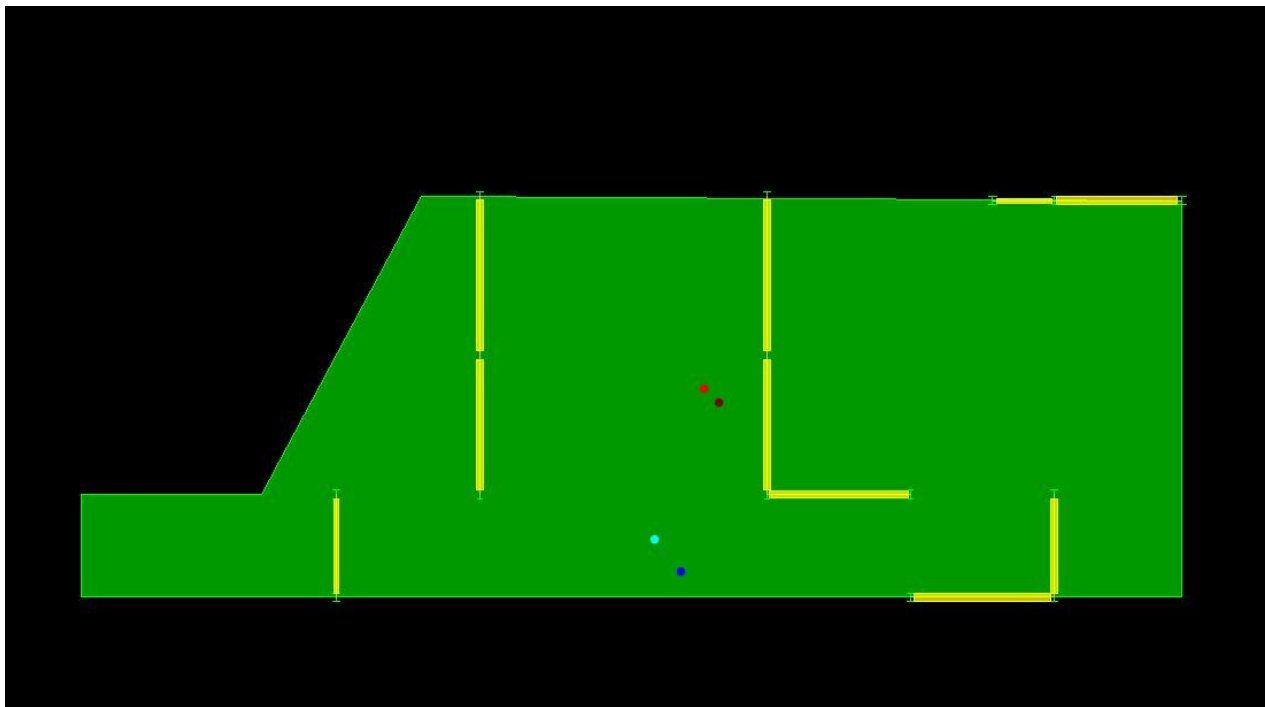**Figure 25:** Mass and rigidity comparison summary

Figure 26: Level RN showing lateral system and centers of mass and rotation comparison
- ETABS COM
- ETABS COR
- Calculated COM
- Calculated COR

Note: point placement is not to scale

ADDITIONAL STIFFNESS AND TORSION CONSIDERATIONS

STIFFNESS

As discussed in the Stiffness and Center of Rigidity portion of Technical Report 3 (above), Frames 7.9, L, and M are considered outliers due to their excessive or lacking stability. The project engineers confirmed that each brace was designed for tension-only loading, meaning frame 7.9, which inherently looks very stiff, engages only the highlighted portion in Figure 27 (right) to resist lateral forces. The indicated frame, then, is only 8'-7" wide with floor-to-floor heights of 20'. When the 1000k unit force was applied to Level RN, it deflected over 120", resulting in an 8.25k/in absolute stiffness (Figure 19).

Frames L and M are disproportionately stiff because in reality these frames extend from Level 1 to their termination height, while in the model the frames extend only from Level 6. This truncation of length also severely limits their deflections, and thus their stiffnesses. Since frames L and M are disproportionately stiff in their absolute calculations (Figure 19), the frames are responsible for more load and moment resistance than if the model extended to Level 1.

TORSION

Because frames 6 and 7 are so dominant in the upper stories (see Figure 21: Direct), the center of rigidities are drawn away from the more symmetric center of mass on Levels 9 and RN. Similarly, the dominance of frames L and M on the lower stories draws the center of rigidity right to them. The floors thus pivot about these points with very large accidental torsions shown in Figure 27.
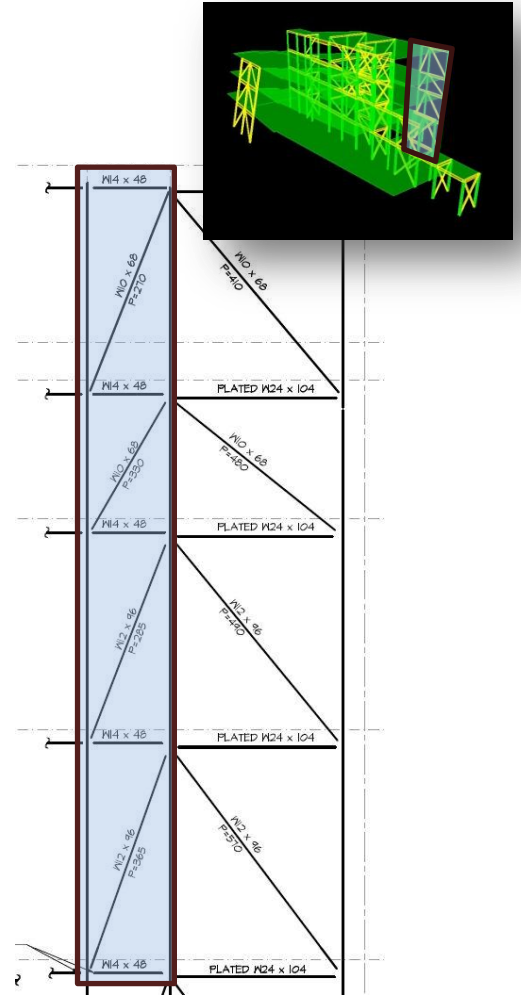

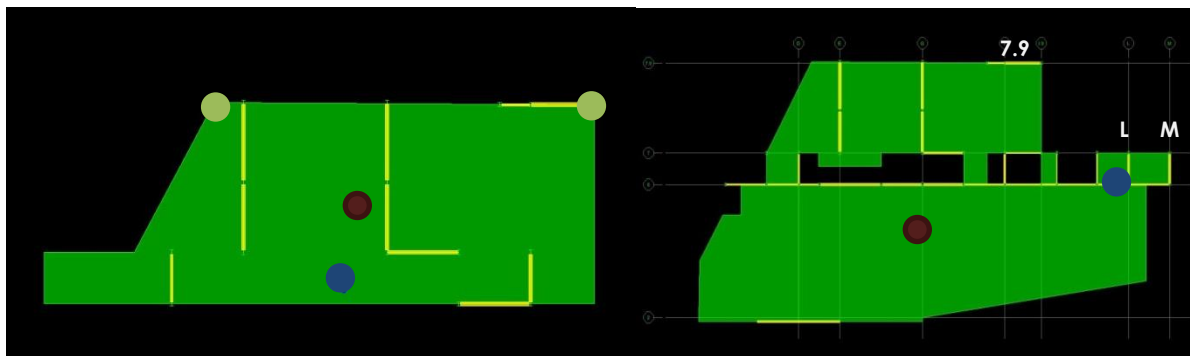**Figure 27:** Frame 7.9 from Level 6 through RN (S-126)

Figure 27: change of COM/COR from Level RN (left) to Level 7 (right)
- ETABS COM
- ETABS COR
- Points Analyzed for Torsional Amplification

This large inherent torsion greatly affects the seismic analysis. Because the West side of AAM effectively pivots about the East, an analysis of the corners marked in Figure 27 above yield amplification factors as high as 22.7. Level RS shows outlying amplification factors of 70.2 in the E-W direction and 280 in the N-S (see Figures 29 and 30 below). This makes sense for these two points because no rigid diaphragm intersects the columns at Level RS, leaving them free to deflect opposite one another. Because they deflect as such the average displacement is very close to zero; thus making the comparative maximum displacement much larger than it would otherwise be.

To analyze AAM more accurately, an alternate amplification factor was calculated according to ASCE 7-05 chapter 12 for Level RS using the points shown in Figure 28 (right). These new points were chosen because they are opposite the center of rigidity on the floors below and should thus have the largest displacements at the level. This new analysis results in more reasonable amplifications of 13.2 in the E-W direction and 1.0 in the N-S direction. Figures 29 and 30 summarize the findings. Additional information is provided in Appendix C.
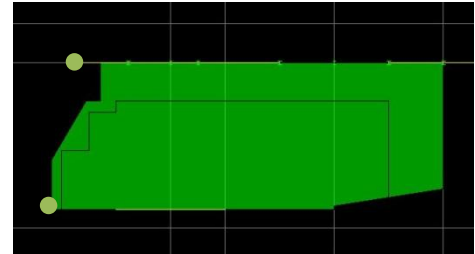

Figure 28: Amplification factor analysis for Level RS

● Points Analyzed for Torsional Amplification

	Amp Factor Maximums			RS Alternative		
	δ_{EW}	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$	δ_{EW}	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$
RN	1.057	0.185	5.71			
RS	0.920	0.092	10.05	0.705	0.1615	1.21
9	0.780	0.191	4.09			
8	0.589	0.325	1.81			
7	0.269	0.264	1.02			
	δ_{NS}	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$	δ_{NS}	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$
RN	0.978	-0.31	3.20			
RS	0.952	0.05	20.04	1.087	0.3125	1.09
9	0.899	0.22	4.06			
8	0.654	0.32	2.02			
7	0.312	0.26	1.19			

Figure 29: Alternate displacement information for Level RS

	E-W Direction							N-S Direction				
	Ht (ft)	hi	Bx	5%Bx	Ax	RS	Mz (ft-k)	By	5%By	Ay	RS	Mzy (ft-k)
RN	160	20	150	7.5	22.7		32649	56	2.8	7.1		3809
RS	142	18	144	7.2	70.2	13.2	8558	54	2.7	278.9	1.0	242
9	140	16	150	7.5	11.6		8326	56	2.8	11.4		3040
8	124	22	191	9.6	2.3		9942	114	5.7	2.8		7333
7	102	24	229	11.5	1.0		5077	118	5.9	1.0		2622

Figure 30: Amplification Factors and applied Mz for seismic analysis

EXECUTION OF THE LATERAL ANALYSIS

The analysis contained within Technical Report 3 applies the initial wind and seismic loads determined in the Lateral Loads portion of the report to the ETABS model described above. Displacements were used to determine the controlling wind load case described in ASCE 7-05 Figure 6-9 at each level. Simultaneously, torsional amplification (see Additional Stiffness and Torsional Concerns: Torsion) was added to the previously-established seismic loads (see Lateral Loads: Seismic) in both orthogonal directions. ASCE 7-05 load combinations were applied in each direction for the respective wind and seismic controls. Story shear was then used to determine the overall controlling load case, and a comparative analysis was performed accordingly.

WIND APPLICATION

Before a lateral analysis could be executed the initial wind loads were applied according to ASCE 7-05 to find the controlling wind case (see Appendix C for more information). The maximum displacements and corresponding wind load cases in both directions are shown in Figure 31 below.

				Max Displacement			
	Elev	hi	H/400	ETABS			
Level	ft	ft	in	X	Case	Y	Case
RN	160	20	2.45	0.312	3	1.003	2D
RS	142	18	1.91	0.257	3	0.757	2D
9	140	16	1.85	0.244	3	0.724	2D
8	124	22	1.37	0.185	3	0.540	2D
7	102	23.67	0.71	0.097	3	0.291	2D
	78.3						

Figure 31: Wind case maximum displacement summary

Wind Case 3 controls the E-W direction on each story, and Wind Case 2D, which corresponds to an eccentricity causing a negative moment about the Z-axis, controls the N-S direction. Wind Case 2D has greater overall deflections than Wind Case 3 and is therefore the controlling load case.

SEISMIC APPLICATION

Moments established in the Additional Stiffness and Torsional Considerations section of this report supplement the initial seismic loads found in the Lateral Loads section. Positive-direction loads coupled with negative-direction moments control in both directions because they engage frame 7.9 where it is least effective. Figure 32 summarizes the seismic loads applied to the AAM ETABS model. North-South seismic loading with a negative moment caused the greatest overall deflections as shown in Figure 33. It is therefore the controlling seismic case.

			Seismic Loads						E-W Direction					N-S Direction			
	Ht (ft)	hi	W (k)	wh^k	Cvx	fi	Vi	Bx	5%Bx	Ax	RS	Mz (ft-k)	By	5%By	Ay	RS	Mzy (ft-k)
RN	160	20	841	21539881	0.2278	192	192	150	7.5	22.7		-32649	56	2.8	7.1		-3809
RS	142	18	649	13092727	0.1385	90	282	144	7.2	70.2	13.2	-8558	54	2.7	278.9	1.0	-242
9	140	16	678	13281336	0.1405	95	377	150	7.5	11.6		-8326	56	2.8	11.4		-3040
8	124	22	1674	25746066	0.2723	456	833	191	9.6	2.3		-9942	114	5.7	2.8		-7333
7	102	24	2007	20876383	0.2208	443	1276	229	11.5	1.0		-5077	118	5.9	1.0		-2622
k	2	Σ	5849	94536394		Vb =	1276										
T	1.53 s					Mov =	158514										

Figure 32: Seismic loading summary

Seismic Diaphragm Displacemet				
	Max Displacement			
	X M-		Y M-	
Level	Overall	Story	Overall	Story
RN	1.203	0.224	1.410	0.343
RS	0.979	0.111	1.067	0.046
9	0.868	0.226	1.021	0.248
8	0.642	0.360	0.773	0.339
7	0.282	0.282	0.434	0.434

Figure 33: Seismic displacement summary

LOAD COMBINATIONS

After the controlling load cases for both wind and seismic were selected, ASCE 7-05 chapter 2 was consulted to find the controlling load combinations. The load cases are as follows:

1. 1.4(D + F)
2. 1.2(D +F + T) + 1.6(L + H) + 0.5(Lr or S or R)
3. 1.2D + 1.6(Lr or S or R) + (L or 0.8W)
4. **1.2D + 1.6W + L + 0.5(Lr or S or R)**
5. **1.2D + 1.0E + L + 0.2S**
6. 0.9D + 1.6W + 1.6H
7. 0.9D + 1.0E + 1.6H

Load combinations 4 and 5 will create both the highest compression for the column analysis and the most story shear for the tension braces. Combinations 6 and 7, which use 0.9D, would be used to consider uplift alongside a lateral strength analysis, but that is considered outside the scope of Technical Report 3. Because the live loads are applied on roof terraces where roof live or snow loads would occur, no snow, roof, or roof live loads were included in the analysis.

LATERAL ANALYSIS RESULTS

DEFLECTION AND STORY DRIFT

Load cases 4 and 5 were run and compared based on overall deflections. The results are displayed in Figure 34. Each case was compared against its respective deflection and story drift allowances per ASCE 7-05 chapters 6 for wind and 12 for seismic. Maximum story drift for seismic design is defined in ASCE 7-05 Table 12.12-1. Story drift for buildings defined as Occupancy Category III without shear walls is limited to 0.015h as shown in Figure 35. All drift and deflections are within code and serviceability limits. Load case 4, 1.2D + 1.6W + L, decisively controlled on each floor.

Allowable overall deflections were measured against the height of the model, not the overall height of the building. For instance, the allowable overall deflections at RN are taken over a height of (160' – 78.3'), or 81.7'. Deflections were analyzed in this fashion to ensure the model accurately reflects the upper stories of the actual building and that its constraints match those of the project.

	Elev	hi	Wind Y+M- (in)						Seismic Y+M- (in)					
					h/400 (in)		OVR	S			0.015h (in)		OVR	S
Level	ft	ft	Overall	Drift	Overall	Story	ok?	ok?	Overall	Drift	Overall	Story	ok?	ok?
RN	160	20	2.141	0.525	2.450	0.600	OK	OK	1.410	0.343	2.450	3.600	OK	OK
RS	142	18	1.616	0.072	1.910	0.540	OK	OK	1.067	0.046	1.910	3.240	OK	OK
9	140	16	1.544	0.392	1.850	0.480	OK	OK	1.021	0.248	1.850	2.880	OK	OK
8	124	22	1.152	0.532	1.370	0.660	OK	OK	0.773	0.339	1.370	3.960	OK	OK
7	102	23.67	0.62	0.620	0.710	0.710	OK	OK	0.434	0.434	0.710	4.260	OK	OK
	78.3													

Seismic vs. Wind Deflection Comparisson

Figure 34: Displacement Comparison

TABLE 12.12-1 ALLOWABLE STORY DRIFT, $\Delta_a{}^{a,b}$

Structure	Occupancy Category		
	I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}{}^c$	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures ^{d}	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

$^a h_{sx}$ is the story height below Level x.
^{b} For seismic force–resisting systems comprised solely of moment frames in Seismic Design Categories D, E, and F, the allowable story drift shall comply with the requirements of Section 12.12.1.1.
^{c} There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 12.12.3 is not waived.
^{d} Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.

Figure 35: Seismic story drift limits from ASCE 7-05 Chapter 12

FOUNDATION IMPACT

Unlike the maximum allowable deflection criteria above, the overturning moment associated with load combination 4 had to be measured at the actual height. A Level 6 "base" shear with the at-height overturning moment ensures an accurate reflection of the contribution of these upper floors to the building foundations. Figure 36 shows that the wind loads greatly control over the seismic conditions. The wind base shear of 2,067 k is 60% larger than its seismic counterpart, and the wind overturning moment is nearly 70% larger than that of the seismic.

			Wind		Seismic	
	ht	Py	1.6 Py	Mh	fi	Mi
Level	ft	k	k	k-ft	k	ft-k
RN	160	81.7	130.74	20918.4	192	30673.8
RS	142	219.8	351.62	49929.5	90	12769.5
9	140	301.9	483.05	67627.5	95	13327.7
8	124	422.3	675.67	83782.6	456	56545.9
7	102	266.4	426.3	43482.6	443	45197.2
Σ			2067.4	265740.6	1276.0	158514.1

Figure 36: Base shear and overturning moment comparison

LATERAL BRACED FRAME MEMBER CHECKS

Finally, Lateral Braced Frame G (Figure 37) was spot-checked to determine the accuracy of Technical Report 3's analysis. Resultant story forces found in ETABS were checked by hand calculations, verified, and applied to the appropriate level of Frame G. The three members highlighted in Figure 37 were checked:

A. W12x96 brace between levels 6 and 7
B. W14x145 column between levels 6 and 7
C. W8x48 brace between levels 8 and 9

Hand calculations for the loads on frame G were compared to ETABS results to check accuracy. The loads and moments at each level were distributed to frame G based on the relative stiffness data found in the Building Properties section of this report. This comparison is shown in Figure 38. The applied loads were all within 5% so the ETABS loads were used.

The two braces, members A and C, were analyzed as tension-only members per the design assumptions indicated by the engineers. The column is assumed to be in compression only. Complete calculations are in Appendix E.
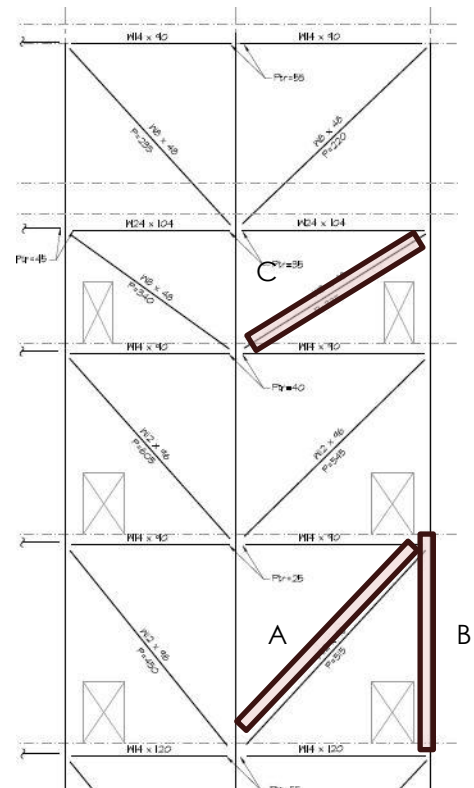

Figure 37: Lateral Braced Frame G

Loads for Lateral Braced Frame G										
	Dir K	Rot K	Wind				Seismic			
	Rel	Rel	Load	Moment	G	ETABS	Load	Moment	G	COR X
RN	33.3	22.1	130.7	2947	72.3	75	192	3809	58.9	1321.8
RS	-	-	-	-	-	-	-	-	-	1451.3
9	33.3	21.4	483.1	10887	206.6	194	95	3040	86.7	1321.8
8	8.4	4.2	675.7	19375	332.4	338	456	7333	125.4	2347.3
7	4.9	2.4	426.3	14654	368.6	371	443	2622	147.0	2595.3
					1494				1494	

Figure 38: ETABS verified with hand calculations

CONCLUSION

Technical Report 3 performed a lateral system analysis of the top floors of AAM. Loads were derived using ASCE 7-05 chapter 6 for wind loads and chapters 11 and 12 for seismic. The methods and assumptions contained in Technical Report 3 differ from those of the design engineers. For the wind analysis, the geometry of the building dictates that Wind Tunnel Testing be performed to find the design loads per ASCE 7-05. Technical Report 3, however, uses simplifying assumptions to use the Analysis Procedure. Likewise with seismic loads, the design engineers used Modal Response Spectrum Analysis while Technical Report 3 uses Equivalent Lateral Force Procedure. The methods contained in this report are conservative for the overall building.
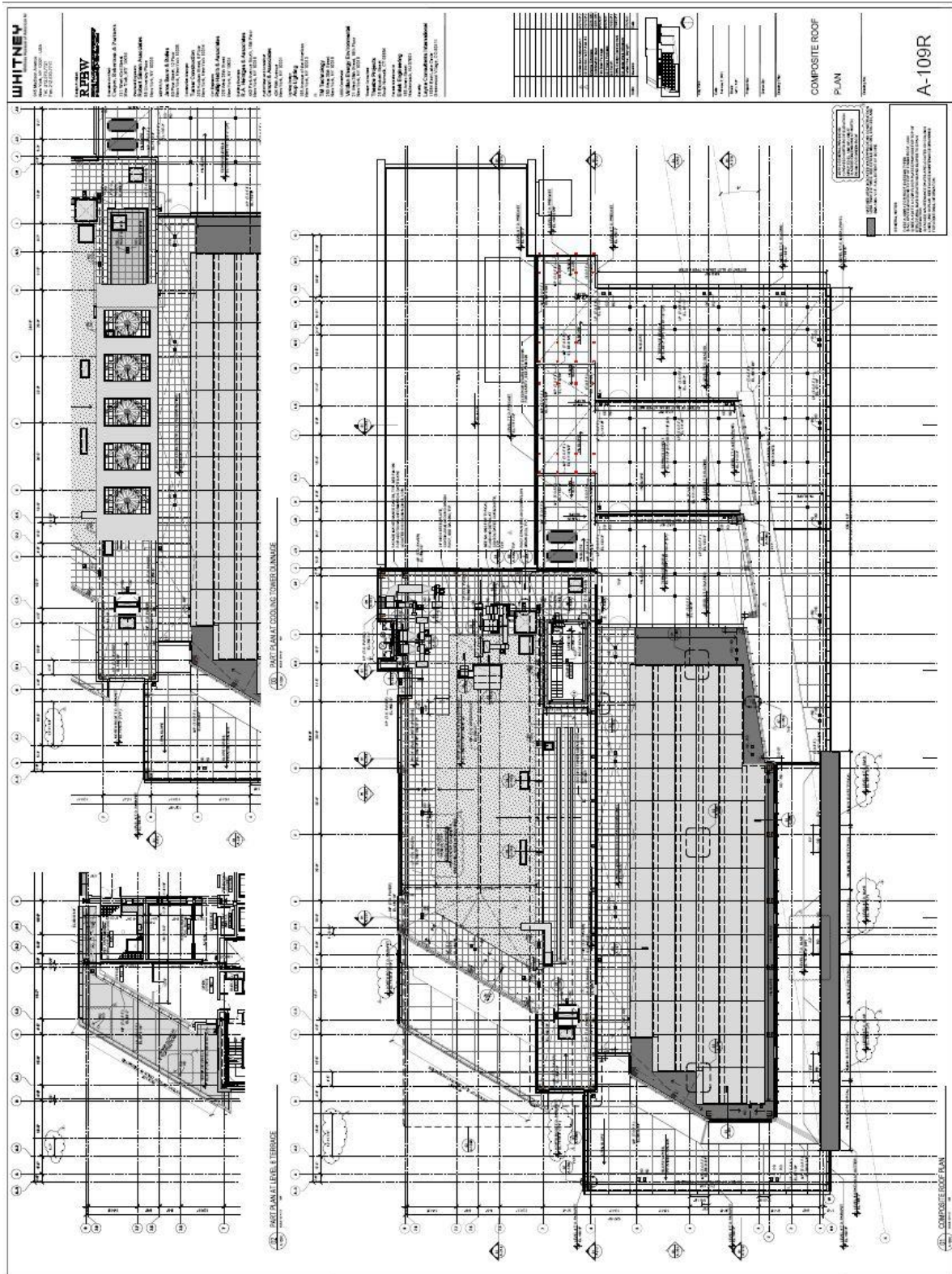
Under these assumptions, design load cases and combinations were chosen by comparing the greatest overall deflections, and verified using base shears and overturning moments. From there, loads were distributed to Frame G using relative stiffness. After the loads had been reapplied to the frame using P-Delta effect, three member forces were checked for adequacy.
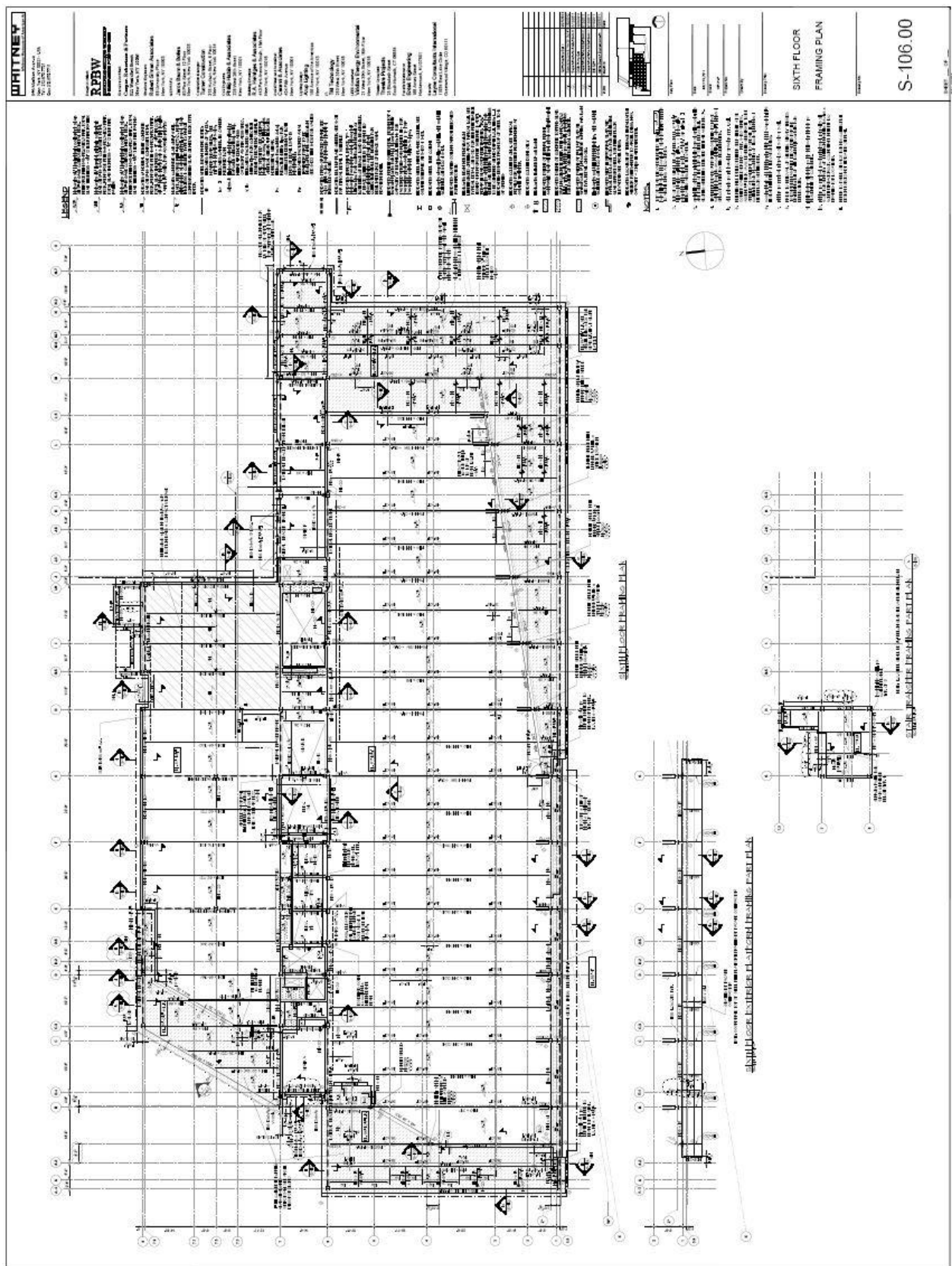
Figure 39 displays the results of the lateral spot check. Brace Member A returned expected results; a slightly conservative load, but adequate for the factored tensile capacity. Column Member B, however, returns questionable results. This can be partially explained due to the live load factor under load combination 4. 1.2D + 1.6W + L may not cause as much compression as a 1.6L. More significantly, though, both members B and C do not appear efficient enough because of the way the analysis was performed. The different load analysis procedures used by the engineers will undoubtedly result in different controlling load cases, and thus use each member more efficiently. Furthermore, AAM is so sensitive to torsion that each member could have a different controlling load combination. A resolute and highly-accurate re-creation and verification of the design loads is therefore impossible within the scope of Technical Report 3.

Member Check Summary					
Member	Description		Calc P (k)	Draw P (k)	φPn (k)
A	W12x96	Brace	560	515	1030
B	W14x145	Column	278	1375	1550
C	W8x48	Brace	240	335	517

Figure 39: Member check summary

APPENDIX A: DRAWING REFERENCES

APPENDIX B: REVISED DEAD LOAD CALCULATIONS

Total Dead Load Calculations						
Level	Type	A M	SQ ft	Wt/SFt	Wt/flr (k)	sqft
RN	31	86.1	5510	87	479	
	32	43.2	2765	101	279	8275
Total Wt	Total PSF	Building Mass (lb-ft)				
841 k	101.6775	37.89				
Level	Type	A M	SQ ft	Wt/SFt	Wt/flr (k)	sqft
RS	33	19.5	1248	141	176	
	34	19.0	1216	93	113	7328
	35	76.0	4864	59	287	
Total Wt	Total PSF	Building Mass (lb-ft)				
649 k	88.6070	33.02				
Level	Type	A M	SQ ft	Wt/SFt	Wt/flr (k)	sqft
9	16	9.0	576	89	51	
	37	78.2	5005	114	571	5581
Total Wt	Total PSF	Building Mass (lb-ft)				
678 k	121.4197	45.25				
Level	Type	A M	SQ ft	Wt/SFt	Wt/flr (k)	sqft
8	3	101.6	6502	101	657	
	6	25.4	1626	86	140	14118
	7	15.8	1011	106	107	
	8	25.2	1613	104	168	
	16	14.1	902	89	80	
	23	7.0	448	100	45	
	27	31.5	2016	167	337	
Total Wt	Total PSF	Building Mass (lb-ft)				
1674 k	118.5993	44.20				
Level	Type	A M	SQ ft	Wt/SFt	Wt/flr (k)	sqft
7	3	144.7	9261	101	935	
	6	57.3	3667	86	315	17792
	8	6.8	435	104	45	
	12	5.1	326	86	28	
	16	14.1	902	89	80	
	20	7.3	467	74	35	
	21	11.2	717	74	53	
	27	31.5	2016	167	337	
Total Wt	Total PSF	Building Mass (lb-ft)				
2007 k	112.7795	42.03				

APPENDIX C: WIND & SEISMIC LOAD CALCULATIONS

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

CASE 1

CASE 3

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

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

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

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.

Wall Pressures								
E-W				qGfCp	qiGCpi	WW	LW	Pressure
Level	ht	Kz	qz	Cp	-0.55	0.8	-0.3	PSF
RN	160	1.39	33.41	23.793	-18.37	42.17	-7.72	49.89
RS	142	1.36	32.68	23.279	-17.98	41.26	-7.72	48.98
9	140	1.36	32.68	23.279	-17.98	41.26	-7.72	48.98
8	124	1.32	31.72	22.595	-17.45	40.04	-7.72	47.76
7	102	1.26	30.28	21.568	-16.65	38.22	-7.72	45.94
6	78	1.21	29.08	20.712	-15.99	36.71	-7.72	44.43

Equivalent Point Loads					
E-W	Pressure	hi	Dist Ld	Bx	Px
Level	PSF	ft	plf	ft	k
RN	49.89	10	498.9	55.8	27.9
RS	48.98	9	1348.3	53.8	72.5
9	48.98	18	1789.1	55.8	99.9
8	47.76	19	1964.3	113.7	223.3
7	45.94	23	1589.9	118.3	188.1
Vb =	611.6 k		Mover =	27902.3 kft	

Wall Pressures								
N-S				qGfCp	qiGCpi	WW	LW	Pressure
Level	ht	Kz	qz	Cp	-0.55	0.8	-0.5	PSF
RN	160	1.39	33.41	22.631	-18.37	41.00	-13.38	54.38
RS	142	1.36	32.68	22.143	-17.98	40.12	-13.38	53.50
9	140	1.36	32.68	22.143	-17.98	40.12	-13.38	53.50
8	124	1.32	31.72	21.491	-17.45	38.94	-13.38	52.32
7	102	1.26	30.28	20.515	-16.65	37.17	-13.38	50.55
6	78	1.21	29.08	19.700	-15.99	35.69	-13.38	49.07

Equivalent Point Loads					
N-S	Pressure	ht	Dist Ld	By	Py
Level	PSF	ft	plf	ft	k
RN	54.38	10	543.8	150.3	81.7
RS	53.50	9	1527.9	143.8	219.8
9	53.50	18	2009.4	150.3	301.9
8	52.32	20	2209.0	191.2	422.3
7	50.55	23	1162.6	229.2	266.4
Vb =	1292.1 k		Mover =	65303.3 kft	

Wind Factors		
	E - W	N - S
G_f =	0.89	0.85
GC pi =	0.55	-
Cp =	-0.3	-0.5
Kd =	0.85	-
Kzt =	1.0	-
I =	1.15	-

Inherent Moments						
	Bx	ex	Mtx	By	ey +	Mt +
Level	ft	ft	k-in	ft	ft	k-in
RN	55.8	8.4	2799	150.3	22.5	22099
RS	53.8	8.1	7014	143.8	21.6	56896
9	55.8	8.4	10039	150.3	22.5	81651
8	113.7	17.1	45681	191.2	28.7	145310
7	118.3	17.8	40072	229.2	34.4	109905

	Ht (ft)	hi	W (k)	wh^k	Cvx	fi	Vi	Bx	5%Bx	Ax	RS	Mz (ft-k)	By	5%By	Ay	RS	Mzy (ft-k)
			Seismic Loads					E-W Direction					N-S Direction				
RN	160	20	841	21539881	0.2278	192	192	150	7.5	22.7		32649	56	2.8	7.1		3809
RS	142	18	649	13092727	0.1385	90	282	144	7.2	70.2	13.2	8558	54	2.7	278.9	1.0	242
9	140	16	678	13281336	0.1405	95	377	150	7.5	11.6		8326	56	2.8	11.4		3040
8	124	22	1674	25746066	0.2723	456	833	191	9.6	2.3		9942	114	5.7	2.8		7333
7	102	24	2007	20876383	0.2208	443	1276	229	11.5	1.0		5077	118	5.9	1.0		2622
		Σ	5849	94536394		Vb =	1276										
						Mov =	158514										

k = 2

T = 1.53 s

$$\delta_{avg} = \frac{\delta_A + \delta_B}{2}; \qquad A_x = \left[\frac{\delta_{max}}{1.2\delta_{avg}}\right]^2$$

FIGURE 12.8-1 TORSIONAL AMPLIFICATION FACTOR, A_x

	Amplification Factor Analysis						RS Alternative					
E-W	$(\delta_{xe})_1$	Δ_{x1}	$(\delta_{xe})_2$	Δ_{x2}	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$	$(\delta_{xe})_1$	Δ_{x1}	$(\delta_{xe})_2$	Δ_{x2}	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$
RN	1.057	0.236	1.054	0.134	0.1850	1.28 *						
RS	0.821	0.041	0.920	0.142	0.0915	1.55 *	0.556	0.128	0.705	0.195	0.1615	1.21
9	0.780	0.191	0.778	0.19	0.1905	1.00						
8	0.589	0.32	0.588	0.33	0.3250	1.02	0.428		0.510			
7	0.269	0.269	0.258	0.258	0.2635	1.02						
	$(\delta_{ye})_1$	Δ_{y1}	$(\delta_{ye})_2$	Δ_{y2}	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$	$(\delta_{ye})_1$	Δ_{y1}	$(\delta_{ye})_2$	Δ_{y2}	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$
RN	0.183	0.053	-0.238	-0.102	-0.0245	4.16 *						
RS	0.130	0.01	-0.136	-0.016	-0.0030	5.33 *	0.308	0.135	0.274	0.100	0.1175	1.15
9	0.120	0.041	-0.120	-0.061	-0.0100	6.10 *						
8	0.079	0.042	-0.059	-0.043	-0.0005	86.00 *	0.173		0.174			
7	0.037	0.037	-0.016	-0.016	0.0105	3.52 *						
N-S	$(\delta_{xe})_1$	Δ_{x1}	$(\delta_{xe})_2$	Δ_{x2}	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$	$(\delta_{xe})_1$	Δ_{x1}	$(\delta_{xe})_2$	Δ_{x2}	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$
RN	0.185	0.032	0.183	0.032	0.0320	1.00						
RS	0.153	0.007	0.151	0.006	0.0065	1.08	0.062	0.023	-0.045	-0.004	0.0095	2.42
9	0.146	0.03	0.145	0.03	0.0300	1.00						
8	0.116	0.038	0.115	0.037	0.0375	1.01	0.039		-0.041			
7	0.078	0.078	0.078	0.078	0.0780	1.00						
	$(\delta_{ye})_1$	Δ_{y1}	$(\delta_{ye})_2$	Δ_{y2}	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$	$(\delta_{ye})_1$	Δ_{y1}	$(\delta_{ye})_2$	Δ_{y2}	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$
RN	0.123	-0.829	0.978	0.218	-0.3055	2.71 *						
RS	0.952	0.053	0.760	0.042	0.0475	1.12	1.039	0.284	1.087	0.341	0.3125	1.09
9	0.899	0.245	0.718	0.198	0.2215	1.11						
8	0.654	0.342	0.520	0.306	0.3240	1.06	0.755		0.746			
7	0.312	0.312	0.214	0.214	0.2630	1.19						

Seismic Design Criteria				
S-200.01			**ASCE 7-05**	
Sds	0.65	Ta (s)		0.9
Sd1	0.13	Cu		1.7
I	1.25	T (s)		1.53
R	3	TL (s)		6
W (k)	5849			
Cs	0.0602			

Amp Factor Maximums				RS Alternative		
	δ_{EW}	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$	δ_{EW}	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$
RN	1.057	0.185	5.71			
RS	0.920	0.092	10.05	0.705	0.1615	1.21
9	0.780	0.191	4.09			
8	0.589	0.325	1.81			
7	0.269	0.264	1.02			
	δ_{NS}	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$	δ_{NS}	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$
RN	0.978	-0.31	3.20			
RS	0.952	0.05	20.04	1.087	0.3125	1.09
9	0.899	0.22	4.06			
8	0.654	0.32	2.02			
7	0.312	0.26	1.19			

APPENDIX D: CENTERS OF MASS & RIGIDITY

Stiffness and Centers of Rigidity

Engagement

	D	E	G	I	L	M		1	6	7_G	7_I.9	7.9
RN	1	1	1	1	0	0	RN	0	1	1	0	1
RS	1	1	1	1	0	0	RS	1	1	0	0	0
9	1	1	1	1	0	0	9	1	1	1	0	1
8	1	1	1	1	1	0	8	1	1	1	1	1
7	1	1	1	1	1	1	7	1	1	1	1	1

Stiffness at Location

	D	E	G	I	L	M		1	6	7_G	7_I.9	7.9
RN	26.25	58.48	59.17	34.01	0.00	0.00	RN	0.00	107.53	60.90	0.00	8.25
RS	26.25	58.48	59.17	34.01	0.00	0.00	RS	82.64	107.53	0.00	0.00	0.00
9	26.25	58.48	59.17	34.01	0.00	0.00	9	82.64	107.53	60.90	0.00	8.25
8	26.25	58.48	59.17	34.01	526.32	0.00	8	82.64	107.53	60.90	277.78	8.25
7	26.25	58.48	59.17	34.01	526.32	515.46	7	82.64	107.53	60.90	277.78	8.25
X	774	1014	1494	1974	2694	2934	Y	See Loc1	840	1012	1012	1502
K	26.25	58.48	59.17	34.01	526.32	515.46	K	82.64	107.53	60.90	277.78	8.25

Stiffness * Distance

	D	E	G	I	L	M		1	6	7_G	7_I.9	7.9
RN	20315	59298	88402	67143	0	0	RN	0	90323	61632	0	12391
RS	20315	59298	88402	67143	0	0	RS	16105	90323	0		0
9	20315	59298	88402	67143	0	0	9	12762	90323	61632	0	12391
8	20315	59298	88402	67143	1417895	0	8	9642	90323	61632	281111	12391
7	20315	59298	88402	67143	1417895	1512371	7	5034	90323	61632	281111	12391

Centers of Rigidity

Level	((K))	((KX))	X	((K))	((KY))	Y	((KD))
RN	177.9115	235158.4	1322	176.6777	164345.4	930	399503.9
RS	177.9115	235158.4	1322	190.1715	106428	560	341586.4
9	177.9115	235158.4	1322	259.3223	177107.5	683	412265.9
8	704.2272	1653053	2347	537.1001	455099	847	2108152
7	1219.691	3165424	2595	537.1001	450490.5	839	3615915

RELATIVE DIRECT STIFFNESS (%)

N-S	D	E	G	I	L	M	E-W	1	6	7_G	7_I.9	7.9
RN	14.8	32.9	33.3	19.1	0.0	0.0	RN	0.0	60.9	34.5	0.0	4.7
RS	14.8	32.9	33.3	19.1	0.0	0.0	RS	43.5	56.5	0.0	0.0	0.0
9	14.8	32.9	33.3	19.1	0.0	0.0	9	31.9	41.5	23.5	0.0	3.2
8	3.7	8.3	8.4	4.8	74.7	0.0	8	15.4	20.0	11.3	51.7	1.5
7	2.2	4.8	4.9	2.8	43.2	42.3	7	15.4	20.0	11.3	51.7	1.5

RELATIVE ROTATIONAL STIFFNESS (%)

N-S	D	E	G	I	L	M	E-W	1	6	7_G	7_I.9	7.9
RN	5.1	14.8	22.1	16.8	0.0	0.0	RN	0.0	22.6	15.4	0.0	3.1
RS	5.9	17.4	25.9	19.7	0.0	0.0	RS	4.7	26.4	0.0	0.0	0.0
9	4.9	14.4	21.4	16.3	0.0	0.0	9	3.1	21.9	14.9	0.0	3.0
8	1.0	2.8	4.2	3.2	67.3	0.0	8	0.5	4.3	2.9	13.3	0.6
7	0.6	1.6	2.4	1.9	39.2	41.8	7	0.1	2.5	1.7	7.8	0.3

APPENDIX E: LATERAL SPOT CHECK CALCULATIONS
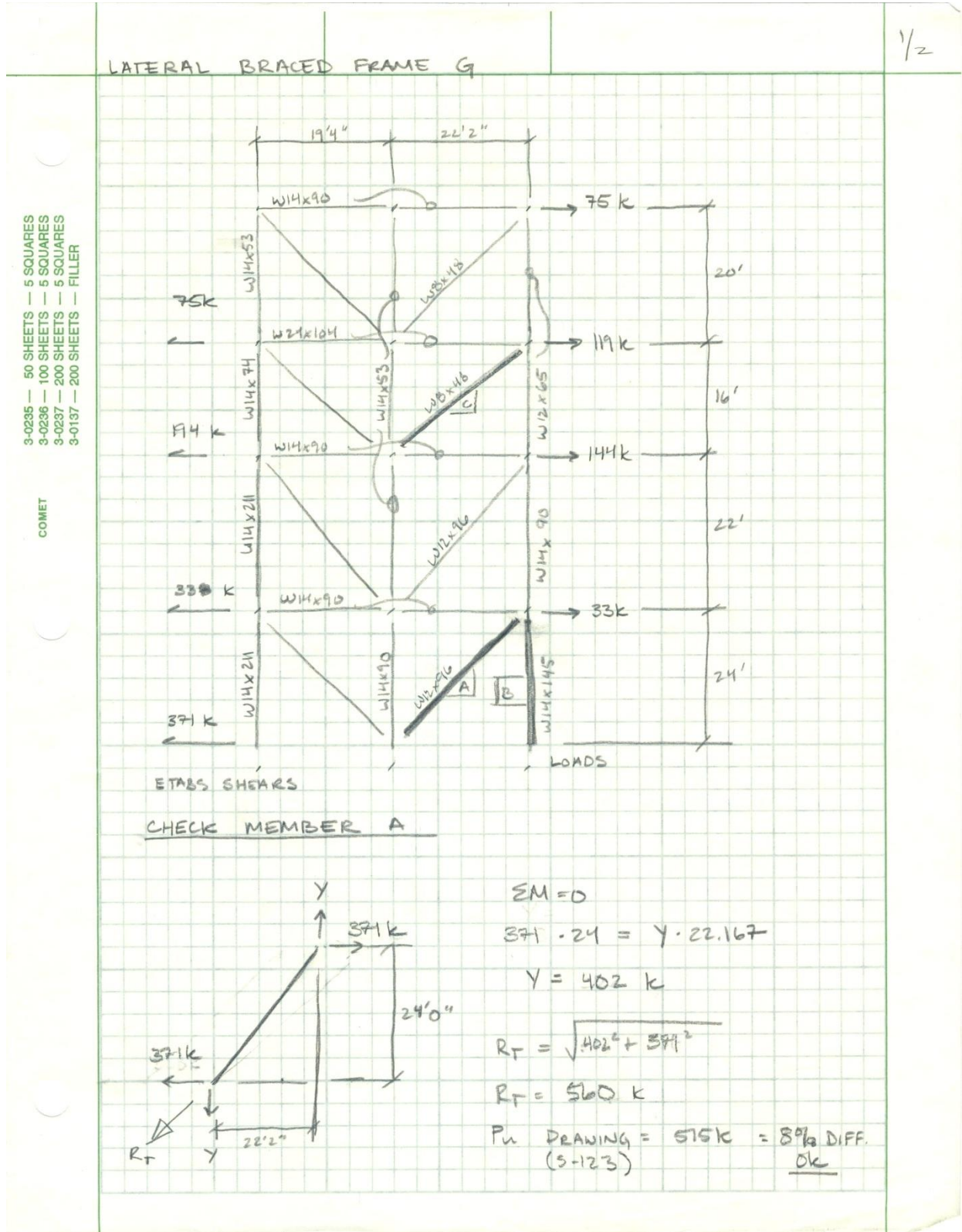
2/2

CHECK MEMBER B: COLUMN 8-G

278 k (ETABS)
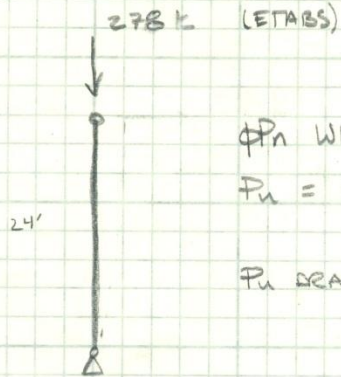
ϕP_n W14x145 = 1310 k @ 24'
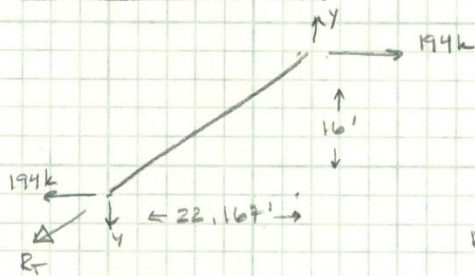
P_u = 278 $\le \phi P_n$ = 1310 ∴ OK

24'

P_u DRAWING 3 = 1375 k

POSSIBLE DIFFERENT LOAD DEFINITION CONTROLS

BY OTHER FORMS OF ANALYSIS

- RESPONSE SPECTRUM

- WIND TUNNELS

CHECK MEMBER C

194 k

16'

194 k

22.167'

R_T

$Y = \dfrac{194.16}{22.167}$

$Y = 140$ k

$R_T = \sqrt{140^2 + 194^2}$

$R_T = 240$ k

P_u DRAWING (S-123) = 335 k