TECHNICAL ASSIGNMENT 3

LATERL SYSTEM ANALYSIS AND CONFIRMATION DESIGN

GATEWAY COMMONS
ITHACA, NEW YORK

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STRUCTURAL OPTION
December 22, 2006
AE 481W
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Executive Summary

The Gateway Commons building in Ithaca, New York is a mixed-use development building being used for retail and residential apartments. It has a basement floor below grade and six floors above grade at a height of 62 feet. CMU walls supporting precast concrete hollow core planks make up the building structure. The building façade uses a combination of brick, an Exterior Insulation Finish System (EIFS), and metal panels.

The purpose of this report is to analyze the effects of the lateral loading on the shear walls. The report includes descriptions of the foundation, walls, floor system, roof system and lateral system. An overview of the building dead loads, live loads, and code requirements are provided. An analysis of the lateral loading on the building due to wind and seismic forces is provided. It was determined that seismic would be the controlling lateral force being resisted by the shear walls. It creates a base shear of 208 kips compared to 95.1 kips due to the wind forces, and a overturning moment of 9500 ft-k compared to 3383 ft-k due to the wind forces.

A simplified ETABS model was constructed to analyze how the shear walls resist the seismic lateral loading on the building. The following load combinations provided by ASCE-07 were analyzed in the ETABS program to determine the design forces:

- 0.9D + 1.0E
- 1.2D + 1.0E + L
- 1.4D

The center of mass and center of rigidity for this building are far enough apart from each other to create torsional effects that are large enough to control the design of the buildings lateral system. This is shown in the story drift analysis by the first mode being due to torsion and having a period of 0.695s. This shows that the walls are the least stiff when trying to resist against torsion. Also, the allowable story drift of $0.01h_{sx}$ for seismic loading was compared against the values determined by ETABS. The results below show that the total drifts of the building are acceptable for loading in the X and Y direction.

Drift: X direction

Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift 0.01hsx	
6	62	0.31 <	0.62	ok
5	52	0.24 <	0.52	ok
4	42	0.17 <	0.42	ok
3	32	0.1 <	0.32	ok
2	22	0.05 <	0.22	ok
1	12	0.02 <	0.12	ok

Drift: Y direction

Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift 0.01hsx	
6	62	0.16 <	0.62	ok
5	52	0.12 <	0.52	ok
4	42	0.08 <	0.42	ok
3	32	0.05 <	0.32	ok
2	22	0.02 <	0.22	ok
1	12	0.01 <	0.12	ok

At the 4th floor design checks of one of the shear walls was conducted. It was done once with the loads on the wall that were determined by ETABS and once with loads that were obtained through hand calculations. At the end of the report appendixes include calculations that were performed to conduct the lateral system analysis.

Introduction

Gateway Commons located in Ithaca, New York is a mixed use project containing retail and residential spaces. It has a basement floor below grade and six floors above grade at a height of 62 feet. The basement has a floor to floor height of 11'-4" and the floors above grade have height of 10' except for the first floor which has a height of 12'. The total building area is 43,000 square feet. The ground floor is retail spaces and the others contain residential apartments. Construction for this project was completed in April of 2007. A typical floor plan of the building is shown in Figure 1.

The building has a basement space between grid lines A and D. The floor for this space is a 5" thick slab on grade. Between grid lines D and E there is a compacted structural fill instead of basement space. The slab on grade that lies on that compacted structural fill is the first floor's floor system between grid lines D and E. Between grid lines A and D hollow core planks are supported by concrete foundation walls that transfer the loads from above onto strip footings.

Located above the concrete foundations walls are CMU walls. Some of the walls are part of the gravity framing system and only support the gravity loads bearing on them. Other walls are part of the lateral system and are designed to resist lateral forces from wind and seismic.

The walls that are part of the lateral system are considered intermediate reinforced masonry shear walls. These walls span in both the N-S and E-W directions. These shear walls are classified as wall types MW2 and MW3. These shear walls are highlighted in green on the plan in Figure 1.

The walls that are part of the gravity framing system are considered wall type MW1. These are all of the other walls on the plan that are not highlighted in green. These walls support the precast concrete hollow core floor planks that act as the flooring system. The roof is constructed out of the same hollow core planks and is also supported by CMU walls as well as two different steel shapes that support the roof planks at their 2'-8" overhang. The building sections in Figures 3 and 4 should also help describe the structure of the Gateway Commons building.

This report will discuss the effects that the applicable load cases have on the lateral load resisting elements. The discussion will involve how loads are distributed onto the building structure, the path loads take to the foundation, centers of mass and rigidity, and story drift. Findings in this report will be based on data obtained from the ETABS computer program and hand calculations that will be used to spot check one of the shear walls.

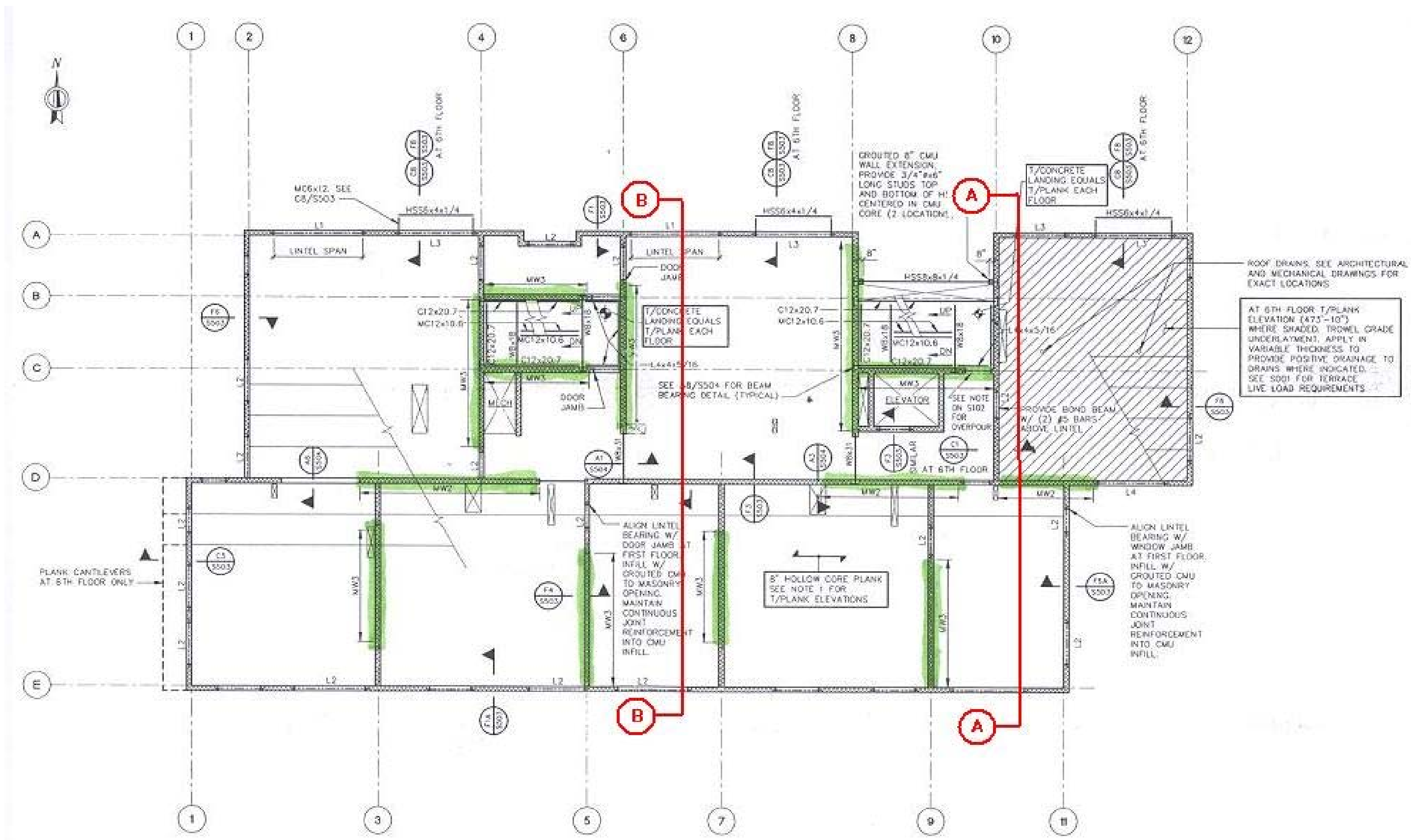


Figure 1 – Typical Framing Plan

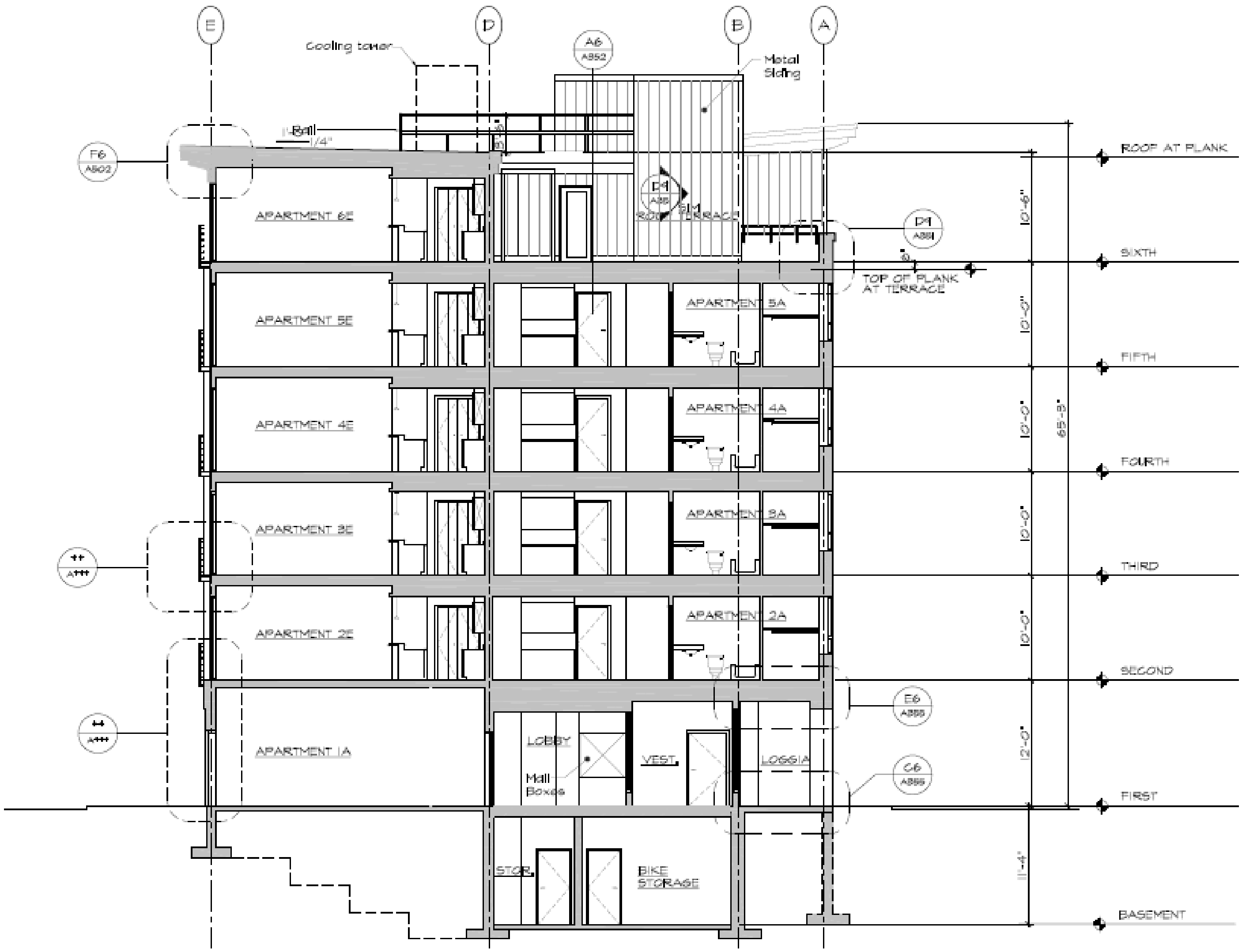


Figure 2 – Section A

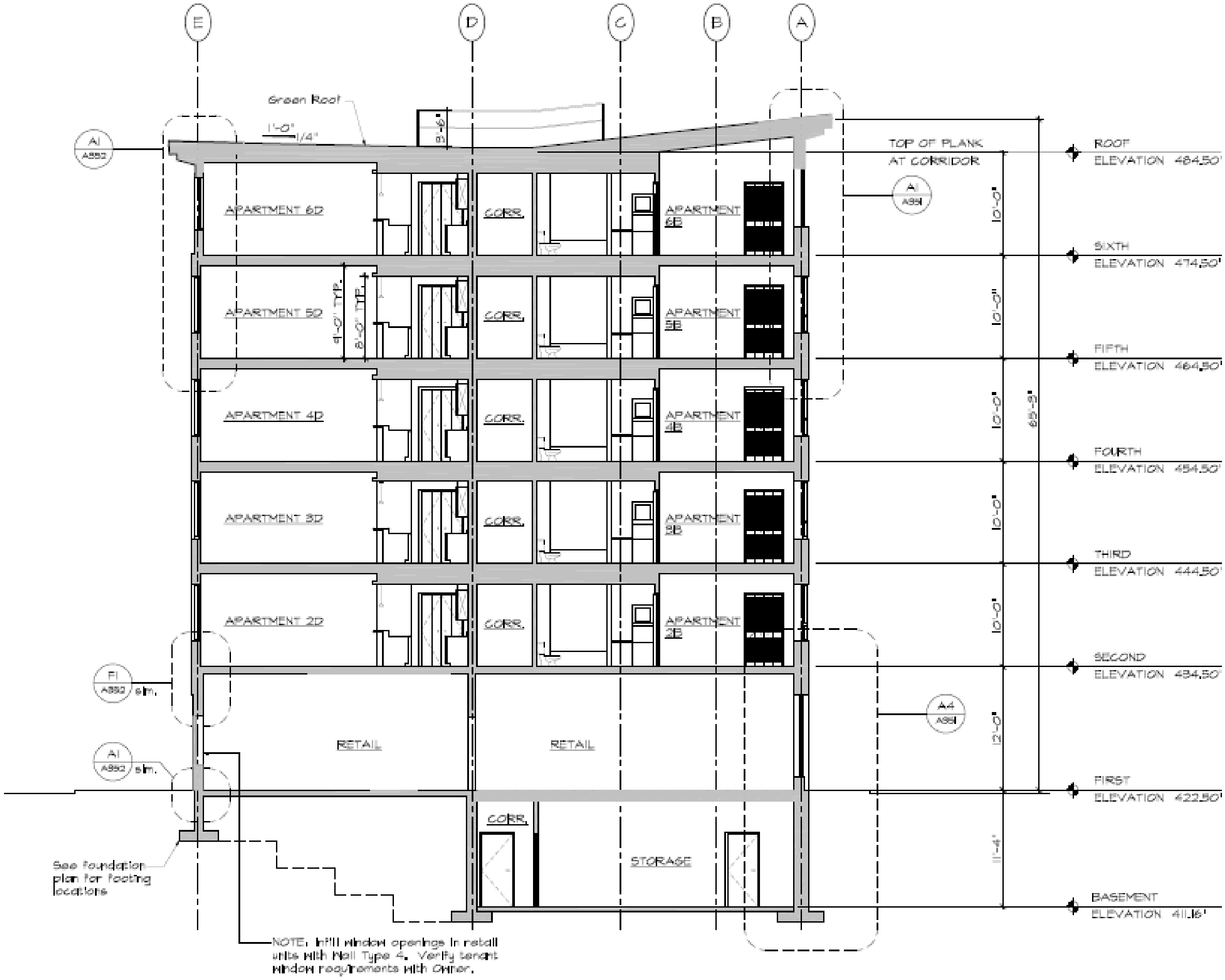


Figure3 – Section B

Structural System

Foundation

Between grid lines A and D, the basement floor slab-on-grade and loads from the concrete foundations walls are transferred onto strip footings with a 28-day strength of f'c = 3,000 psi. These strip footings sit on undisturbed indigenous soils composed of sand and gravel with an allowable bearing capacity of 5,000 psf . The slab-on-grade is 5" thick and reinforced with #4 bars at 16" on center spanning in both directions. The slab-on-grade has a concrete strength of f'c = 3,500 psi. The foundations walls will have a concrete strength of f'c = 3,000 psi or 4,000 psi depending on the type of wall. Between grid lines D and E the footings sit on a compacted structural fill that has an allowable bearing capacity of 5,000 psf. The slab on grade in this section is supported by the compacted structural fill and the foundation walls on grid lines D and E. It has the same thickness and reinforcing as the other slab on grade. The slab on grade in this section is 11'-4" higher than slab on grade between grid lines A and D.

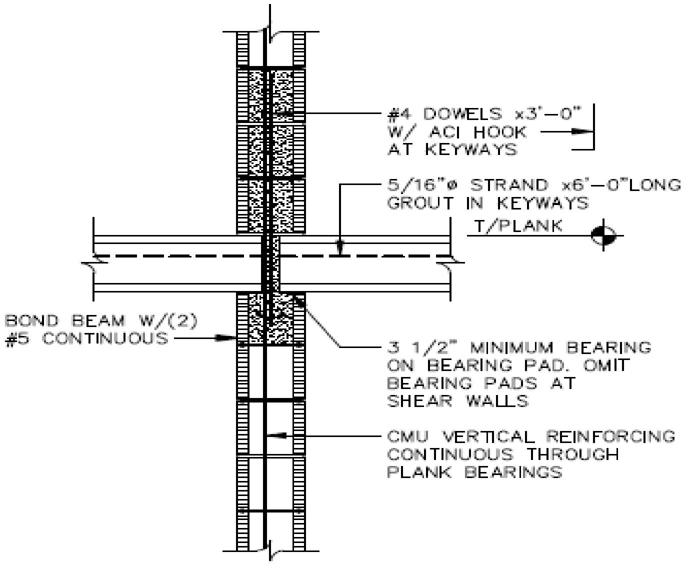
There are also five concrete piers that are supported by spot footings on the north east corner of the building. The reason for these piers is to create the loggia. At the second floor a concrete beam spans across the piers to pick up the gravity loads and distribute them onto the piers.

Masonry Walls

The walls that are not considered part of the lateral system are wall type MW1. Unlike the concrete foundations walls these walls are constructed out of 8" thick concrete masonry units (CMU). These walls act as the gravity framing system and support the precast concrete hollow core floor planks that act as the flooring system. Between the first and second floors the walls are grouted solid. Between the second and third floors the walls are grouted at 2' on center. For the rest of the floors, wall type MW1 has vertical reinforcing of #5 at 4' on center. The walls are horizontally reinforced at 16" on center. A wall schedule describing this reinforcing can be found in Figure 8. The exterior walls on the north and part of the east and west sides have a brick façade that is supported by shelf angles at each floor. The exterior walls on the south and other part of the east and west sides carry an Exterior Insulation Finish System (EIFS) façade.

Floor System

The primary flooring system for the elevated floors of the building is precast concrete hollow core planks. The planks span in the east/west direction. On the first floor the planks have a thickness of 10", but on floors two through six the plank thickness is 8". The planks on the first floor have a 2" thick concrete topping. All planks have a maximum width of 4' and are allowed to have a minimum width of 1'-6". Planks located at interior bearing partitions must be connected with a 6' long #3 bar or 5/16" diameter strand grouted into the keyway, as shown in Figure 5. Planks are often connected to exterior CMU walls with #4 dowels that are bent into the keyways, as shown in Figure 6. On the first floor, half of the floor is planks while the other half is a 5" thick slab on grade. The slab on grade described in the foundations section is the floor system for the basement.



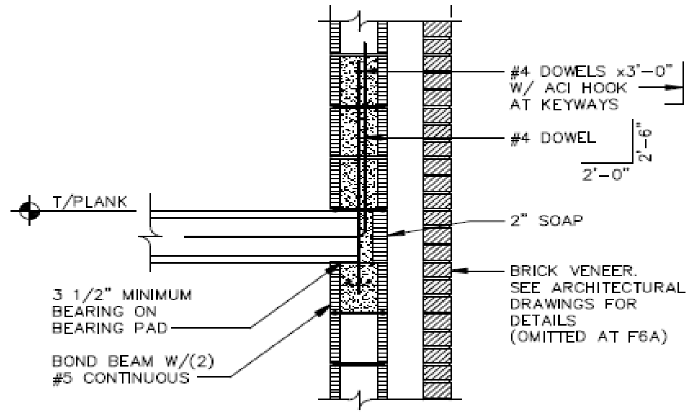


Figure 5 – Floor Planks at Interior Walls

Figure 6 – Floor Planks at Exterior Walls

Roof

The roof structure uses the same 8" thick, precast, hollow core, concrete planks as used on the floors. At gridline D the roof begins to slope up toward the building's south end at ¼"/foot. Between gridline D and C the roof begins to slope up toward the building's north end at slightly larger slope. The building section in Figure 7 shows how the roof is sloped. The roof planks have a 2'-8" roof overhang. Two different steel shapes are used to support the planks at the overhang, a WT6x43.5 and an L6x6x1/2. There is also a roof terrace on the sixth floor that uses the same planks system as used by the typical floor system. There is no roof overhang on the sixth floor roof terrace.

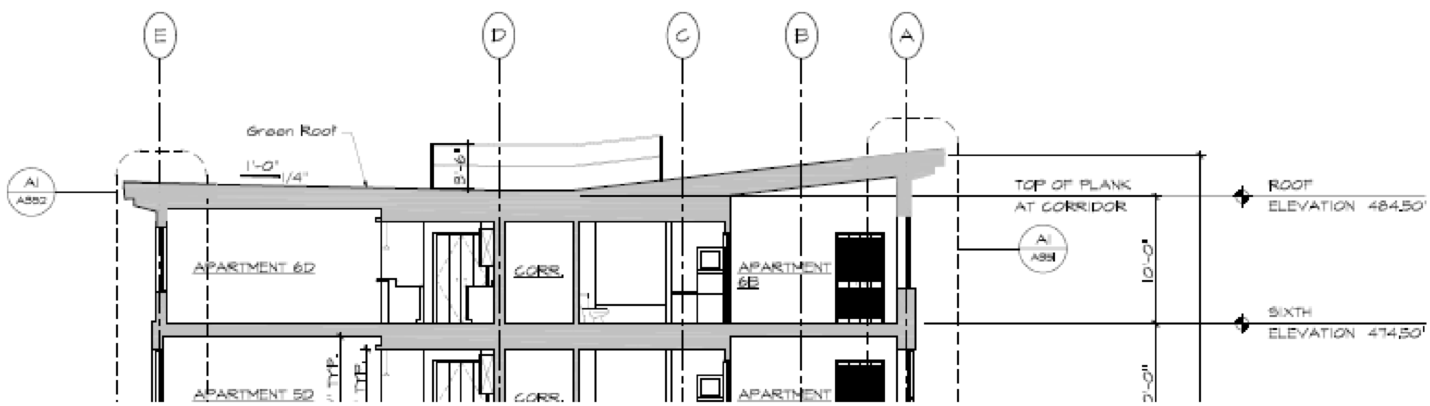


Figure 7 – Building Section for Roof

Lateral System

The structure is laterally supported by intermediate reinforced masonry shear walls in the N-S and E-W directions. Like the load bearing walls for the gravity framing system the shear walls are also 8" thick CMU walls. However, the shear walls are designed to resist the lateral loads due to seismic and wind forces. These lateral forces are distributed onto the shear walls through the rigid floor system of hollow core planks. There are two different shear wall types, MW2 and MW3. The shear walls are highlighted in green on the floor plan in Figure 1. The wall schedule in Figure 8 describes the reinforcing for both shear wall types.

WALL ~~LINTEL~~ SCHEDULE

MARK	VERTICAL REINFORCING	HORIZONTAL REINFORCING	REMARKS
MW1	#5 AT 4'-0"OC	STANDARD JOINT REINFORCING AT 16"OC	GROUT WALL SOLID 1ST-2ND FLOORS GROUT WALL AT 2'-0"OC 2ND-3RD FLOORS
MW2	#5 AT 4'-0"OC (TYPICAL) (6)#6 EACH END (1ST-2ND) (4)#6 EACH END (2ND-4TH) (2)#5 EACH END (4TH-ROOF)	STANDARD JOINT REINFORCING 1ST-2ND AND 6TH-ROOF, HEAVY DUTY JOINT REINFORCING AT 8"OC 2ND-6TH	GROUT WALL SOLID 1ST-2ND FLOORS
MW3	#5 AT 4'-0"OC (TYPICAL) (2)#5 EACH END	STANDARD JOINT REINFORCING 1ST-2ND AND 6TH-ROOF, HEAVY DUTY JOINT REINFORCING AT 8"OC 2ND-6TH	GROUT WALL SOLID 1ST-2ND FLOOR

NOTES:
1. UNLESS NOTED OTHERWISE ON PLAN, ALL WALLS ARE TYPE MW1.
2. MINIMUM REINFORCING REQUIREMENTS SHOWN ON A3/S506 APPLY TO ALL WALLS.
3. SEE F6/S506 FOR PLACEMENT OF VERTICAL BARS AT ENDS OF WALLS.

Figure 8 – Wall Schedule

Loads

This gravity load information was obtained from the general notes page of the building plans. These loads were used by the engineer to design the gravity load bearing walls. This information will also determine the total dead load on the building which in turn will be used to determine the amount of seismic loading on the building.

Live Loads

First Floor…………………………………………..100 psf
Second – Sixth Floor…………………………...40 psf
Sixth Floor Terrace……………………………..100 psf

Dead Loads

First Floor…………………………………………..100 psf
Second – Sixth Floor…………………………..70 psf
CMU Walls……………………………………….. 55 psf
Brick Façade……………………………………..40 psf
Green Roof or Roof Top Pavers………….95 psf
Other Roof Areas……………………………….75 psf
Mechanical Equipment………………………5 psf
Partition walls…………………………………..10 psf

Snow Loads

Ground Snow load (Pg)……………………...45 psf
Flat Roof Snow Load (Pf)……………………32 psf

Codes and References

The codes that were referenced to design the Gateway Commons building and the material properties of its structural components are listed below.

Applicable Codes and Standards

- 2002 Building Code of New York State (BCNYS)
- ASTM Standards
- NCMA Tek Notes
- ACI Standards
- ASCE 7-98

Cast in Place Concrete	
Member	28 Day Compressive Strength (f'c)
Columns and Beams	4,000 psi
Interior Slabs on Grade	3,500 psi
Footings, Foundations Walls, Piers, Misc.	3,000 psi
Retaining Walls, Basement Walls, Exterior Slabs	4,000 psi

Structural and Miscellaneous Steel		
Material	ASTM Standard	Fy (ksi)
Rolled Steel W Shapes	A 992	50
Rolled Steel C and MC Shapes	A 36	36
Rolled Steel Plates, Bars, and Angles	A 36	36
Hollow Structural Sections (HSS)	A 500, Grade B or C	46 or 50
Pipe	A 53, Type E or S, Grade B	35
Reinforcing Bars	A 615, Grade 60	60

Lateral Loads

Lateral loads acting on the building are the result of wind and seismic forces. Wind and seismic loads were originally calculated using the 2002 Building Code of New York State. For this report loads were calculated using methods from ASCE 7 – 05. Wind loads were calculated for the north-south and east-west directions. The following is a summary of the lateral load findings. See Appendix A for a complete set of wind and seismic calculations.

Wind

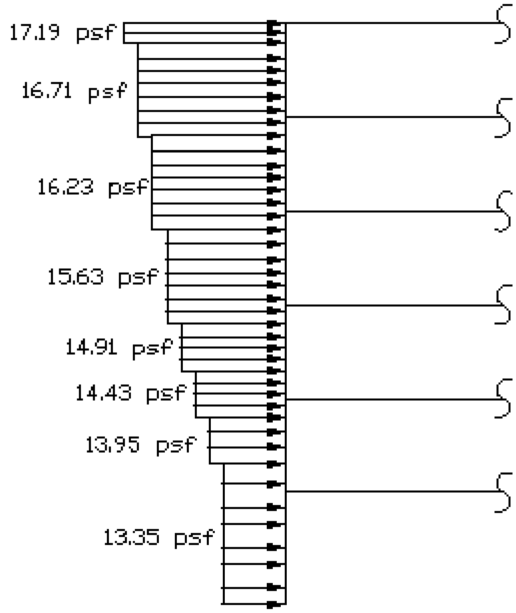
Wind loads were calculated for the north-south and east-west directions. Since the load in the north-south direction is the larger of the two a loading diagram is only provided for that direction. Some of the factors used to determine the wind loads and a chart summarizing the calculations for the wind loads acting in the north-south direction are listed below. Also listed below are diagrams of the loads at each floor due to wind forces. For detailed wind calculations see Appendix A.

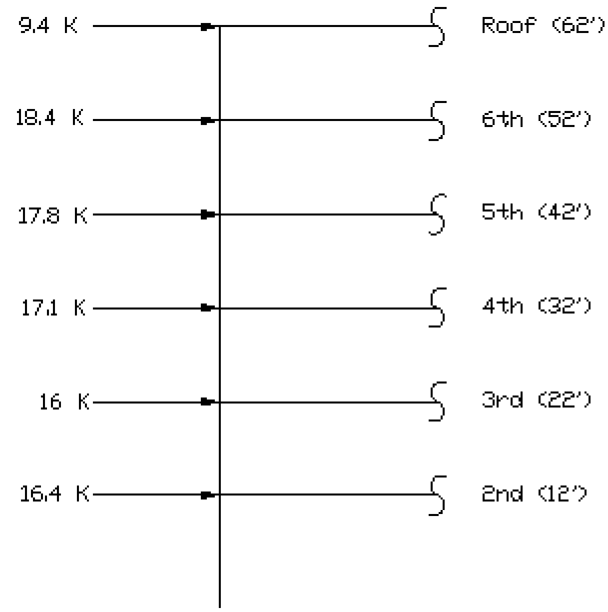
Basic Wind Speed: 90 mph
Importance Factor: 1
Exposure Category: B
$GCpi = \pm 0.18$
$G = 0.85$

Long side of building (N-S direction)

Z (ft)	Kz	qz	$P_{sidewall}$ (psf)	$P_{leeward}$ (psf)	$P_{windward}$ (psf)	P_{total} (psf)
0-15	0.57	10.04659	-5.97772224	-6.52	6.83168256	13.3516826
20	0.62	10.92787	-6.50208384	-6.52	7.43095296	13.950953
25	0.66	11.6329	-6.92157312	-6.52	7.91036928	14.4303693
30	0.7	12.33792	-7.3410624	-6.52	8.3897856	14.9097856
40	0.76	13.39546	-7.97029632	-6.52	9.10891008	15.6289101
50	0.81	14.27674	-8.49465792	-6.52	9.70818048	16.2281805
60	0.85	14.98176	-8.9141472	-6.52	10.1875968	16.7075968
70	0.89	15.68678	-9.33363648	-6.52	10.66701312	17.1870131

Base Shear: 95.1 k
Overturning Moment: 3383 ft-k



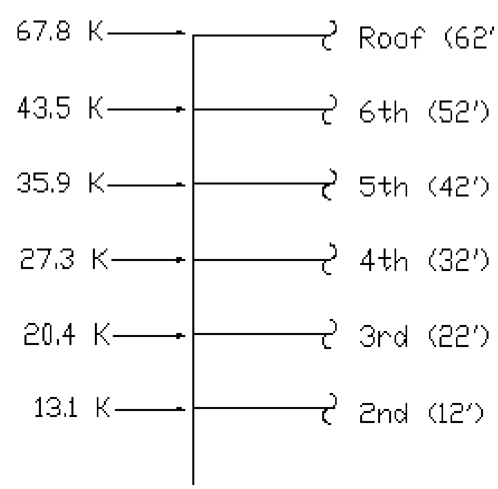


Seismic

The weight of the building is based on the framing and other dead loads on the building. Some of the other factors used to determine the seismic loads and a chart displaying a summary of seismic load calculations are shown below. The seismic forces at each story prove to be larger than those created by the wind; therefore seismic is the controlling lateral load in both directions. A diagram showing the seismic lateral loading at each story is also shown below. For detailed seismic calculations see Appendix A.

Importance Factor: 1
Occupancy Category: II
Site Class: D
Seismic Design Category: B
Response Modification Factor: 3.5

Level	Height (ft)	Wx(hx)^k	Cvx	Fx
2	12	15012	0.062912	13.08575
3	22	23408	0.098098	20.40443
4	32	31360	0.131423	27.33608
5	42	41160	0.172493	35.8786
6	52	49868	0.208987	43.46924
Roof	62	77810	0.326086	67.8259
	∑Wx(hx)^k =	238618		



Base Shear: 208 k
Overturning Moment: 9500 ft-k

Lateral Load Path

In seismic loading the ground can move horizontally and vertically. The vertical load the ground generates onto the structure during seismic loading should be able to be resisted due to the gravity loading design. The horizontal loads put onto the building will create stresses and distortions similar to those created if the base were to remain stationary and lateral loads were applied to the top of the building. The loads are transferred to the precast concrete hollow core planks that are doweled into the wall and act as a rigid diaphragm. These loads are then distributed to the CMU shear walls. These loads on the shear walls are then transferred on to the strip footings.

The seismic loads are applied at the center of the buildings mass. These loads are transferred onto the shear walls by the method of rigidity as described later on in this report. The building rotates about the center of rigidity. The center of mass, where the lateral loads are applied, is in a different location therefore creating torsional forces on the building. These torsional forces are distributed onto the shear wall as shear forces. The center of mass and center of rigidity are far enough away from each other to create torsional shear forces that control the lateral design of this building. Figure 9 shows a computer generated model of the lateral resisting system for Gateway Commons. The shear walls are red and the rigid floor system is green. The walls that are not designed to be shear walls are removed.

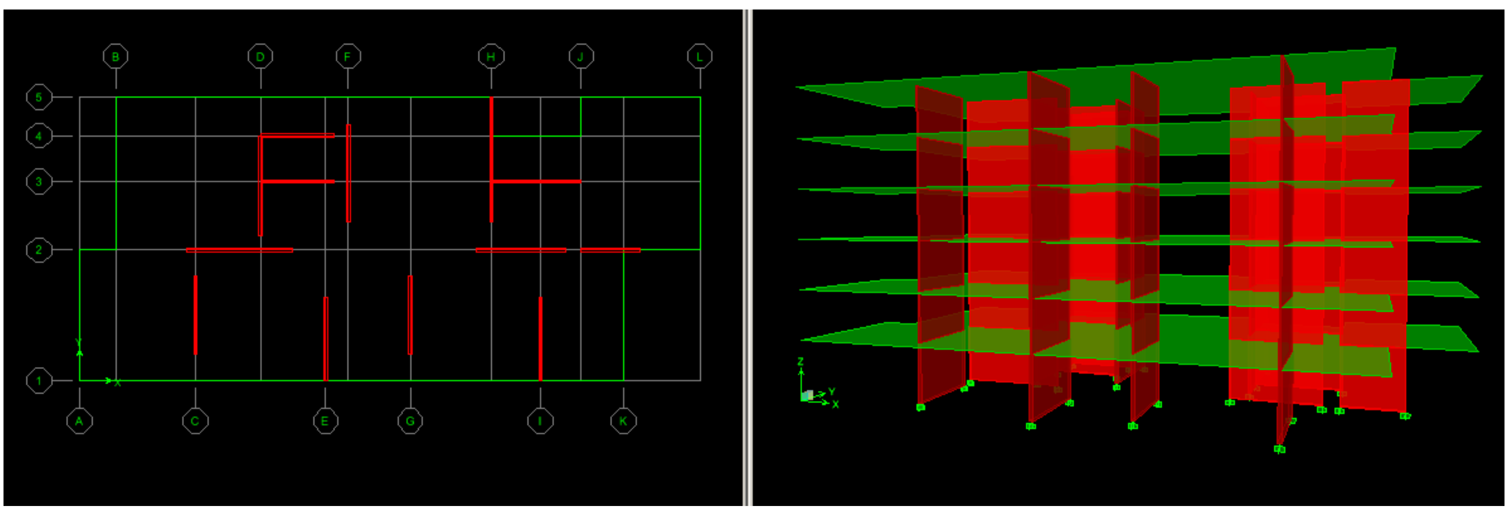


Figure 9 – ETABS Model

Lateral Analysis

ETABS Analysis

A simplified ETABS model was constructed to analyze how the shear walls resist the seismic lateral loading on the building. Shear walls were added to the model as piers and were connected with a rigid diaphragm at each floor. Due to ETABS not having masonry as a building material the walls were made thinner and designed as concrete. The following load combinations provided by ASCE-07 were analyzed in the ETABS program to determine the design forces:

- 0.9D + 1.0E
- 1.2D + 1.0E + L
- 1.4D

Center of Mass and Center of Rigidity

Figure 10 provides the ETABS calculated centers of mass and centers of rigidity of the structure. The relative stiffness of each shear wall was determined in order to distribute the lateral load at each floor onto the shear walls. With this information, the actual stiffness of each wall was determined and used to find the center of rigidity at each level. This process is known as the method of rigidity. Figure 10 shows that the center of rigidity is a considerable distance from the center of mass in both the X and Y direction.

C E N T E R S   O F   C U M U L A T I V E   M A S S   &   C E N T E R S   O F   R I G I D I T Y

STORY LEVEL	DIAPHRAGM NAME	/----------CENTER OF MASS---------- MASS	ORDINATE-X	ORDINATE-Y	//--CENTER OF ORDINATE-X	RIGIDITY--/ ORDINATE-Y
STORY6	D6	1.928E+00	736.837	347.754	703.765	499.975
STORY5	D5	1.555E+00	737.048	347.804	704.952	496.424
STORY4	D4	1.555E+00	737.048	347.804	707.366	490.294
STORY3	D3	1.555E+00	737.048	347.804	711.136	480.126
STORY2	D2	1.555E+00	737.048	347.804	716.480	463.936
STORY1	D1	1.953E+00	737.173	347.834	722.189	445.359

Figure 10 – Centers of Mass and Rigidity

Distribution of Forces

Axial loads were determined by distributing the floor loads onto the shear walls based on tributary areas. Lateral loads in the X and Y direction due to seismic forces were added at each floor. These loads are distributed on to the shear walls by the method of rigidity to determine the direct shear on each wall. There are also additional shear forces on each wall due to torsion. The seismic force at each story acts at the center of mass. The eccentricity due to the center of rigidity not being located at the center of mass and an additional 5% eccentricity cause torsion at each floor. A table is provided in Figure 11 that shows the torsion forces that ETABS determined to be at each story. These torsion forces are distributed as shear forces onto the shear walls at each floor. Torsional shear can be calculated by the following equation:

$$T = \frac{(\mathrm{V})(\mathrm{e})(\mathrm{d_i})(\mathrm{R_i})}{\mathrm{J}}$$

where V = story shear, e = eccentricity, d_i = distance from the center of rigidity to the centroid of the member, R_i = stiffness of member, J = torsional moment of inertia = $\sum(R_i)(d_i)^2$. The axial force, shear force, and moment on each shear wall on the 4th floor are shown in a table in Figure 12.

Story	Story Shear	Torsion X	Torsion Y
1	13.1	455.88	998.22
2	20.4	709.92	1554.48
3	27.3	950.04	2080.26
4	35.9	1249.32	2735.58
5	43.5	1513.8	3314.7
6	67.8	2359.44	5166.36

Figure 11 – Torsional Forces at Each Story

Pier	Axial (kips)	Shear (kips)	Moment (kip-ft)
1	16	4.82	1451.5
2	17.1	10.43	448.1
3	16	10.9	342.6
4	17.1	17.89	1750.2
5	20.44	27.25	1664.5
6	19.9	17.86	349
7	25.72	58.25	4011.4
8	21.5	36.3	1281
9	18.2	21.7	777.2
10	11.93	6.11	220
11	15	12.22	290
12	14.96	25.62	104.4
13	18.27	45.4	223.4

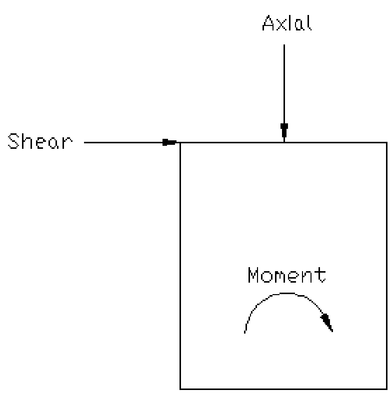


Figure 12 – Shear Wall Loading

Story Drift

The allowable story drift of $0.01h_{sx}$ for seismic loading was compared against the values determined by ETABS. The results are shown below in Figure 13. The charts show that the total drifts of the building are acceptable for loading in the X and Y direction.

Figure 14 contains the modal analysis of the shear walls. The first mode that occurred was torsion with a period of 0.695s. The second mode occurred in the X direction (east-west) at 0.452s. The third mode occurred in the Y direction (north-south) at 0.42s. This shows that the walls are the least stiff when trying to resist against torsion. The large torsional forces acting on this building are due to the center of rigidity and center of mass not being located close together.

Drift: X direction

Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift 0.01hsx	
6	62	0.31 <	0.62	ok
5	52	0.24 <	0.52	ok
4	42	0.17 <	0.42	ok
3	32	0.1 <	0.32	ok
2	22	0.05 <	0.22	ok
1	12	0.02 <	0.12	ok

Drift: Y direction

Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift 0.01hsx	
6	62	0.16 <	0.62	ok
5	52	0.12 <	0.52	ok
4	42	0.08 <	0.42	ok
3	32	0.05 <	0.32	ok
2	22	0.02 <	0.22	ok
1	12	0.01 <	0.12	ok

Figure 13 – Story Drift

M O D A L   P E R I O D S   A N D   F R E Q U E N C I E S

MODE NUMBER	PERIOD (TIME)	FREQUENCY (CYCLES/TIME)	CIRCULAR FREQ (RADIANS/TIME)
Mode 1	0.69473	1.43940	9.04403
Mode 2	0.45245	2.21018	13.88698
Mode 3	0.41968	2.38278	14.97146
Mode 4	0.18003	5.55468	34.90110
Mode 5	0.15901	6.28893	39.51450
Mode 6	0.15638	6.39453	40.17803
Mode 7	0.14208	7.03853	44.22440
Mode 8	0.13945	7.17109	45.05731
Mode 9	0.13425	7.44853	46.80050
Mode 10	0.12084	8.27522	51.99477
Mode 11	0.12055	8.29515	52.11996
Mode 12	0.12040	8.30593	52.18771

ETABS v9.1.1 File:TECH3MODEL Units:Kip-in December 23, 2007 18:23 PAGE 10

M O D A L   P A R T I C I P A T I N G   M A S S   R A T I O S

MODE NUMBER	X-TRANS %MASS <SUM>	Y-TRANS %MASS <SUM>	Z-TRANS %MASS <SUM>	RX-ROTN %MASS <SUM>	RY-ROTN %MASS <SUM>	RZ-ROTN %MASS <SUM>
Mode 1	19.11 < 19>	0.57 < 1>	0.00 < 0>	0.87 < 1>	29.34 < 29>	43.89 < 44>
Mode 2	35.71 < 55>	15.11 < 16>	0.00 < 0>	22.98 < 24>	54.33 < 84>	12.77 < 57>
Mode 3	8.25 < 63>	47.62 < 63>	0.00 < 0>	72.52 < 96>	12.60 < 96>	7.74 < 64>
Mode 4	0.01 < 63>	0.00 < 63>	0.00 < 0>	0.02 < 96>	0.01 < 96>	0.03 < 64>
Mode 5	0.02 < 63>	0.00 < 63>	0.00 < 0>	0.00 < 96>	0.00 < 96>	0.01 < 64>
Mode 6	0.22 < 63>	0.25 < 64>	0.00 < 0>	0.04 < 96>	0.04 < 96>	1.56 < 66>
Mode 7	5.17 < 68>	0.30 < 64>	0.00 < 0>	0.05 < 96>	0.83 < 97>	10.52 < 77>
Mode 8	0.01 < 69>	0.07 < 64>	0.00 < 0>	0.01 < 96>	0.00 < 97>	2.53 < 79>
Mode 9	0.24 < 69>	0.00 < 64>	0.00 < 0>	0.00 < 96>	0.03 < 97>	0.54 < 80>
Mode 10	0.46 < 69>	0.49 < 64>	0.00 < 0>	0.07 < 97>	0.06 < 97>	0.01 < 80>
Mode 11	0.00 < 69>	0.00 < 64>	0.00 < 0>	0.00 < 97>	0.00 < 97>	0.00 < 80>
Mode 12	0.01 < 69>	0.08 < 64>	0.00 < 0>	0.01 < 97>	0.00 < 97>	0.07 < 80>

Figure 14 – Modal Analysis

Spot Check: Shear Wall

The spot check was conducted on the 4th floor shear wall labeled, pier 5. The spot check showed that the wall only needed to be reinforced with vertical reinforcing for flexure. Although bond beams are not needed horizontal joint reinforcement should be added to control cracking and wall flexibility. The axial load on the wall proved to be less than the allowed axial load for the wall. Calculations from the spot check can be found in Appendix B.

Another spot check was done on the same wall. In this spot check the shear force on the wall due to direct shear and torsional shear was determined through hand calculations. Like in the other spot check only flexural reinforcement was needed for the wall. The calculations that show how the shear forces were obtained can be found in Appendix C. The calculations for this spot check can be found in Appendix D.

The overturning moment distributed onto the shear wall was able to be resisted without having to make changes that would have influenced the overall design like an increase in the block size. The soil bearing capacity will resist the load that the overturning moment distributes onto the strip footings.

Conclusion

The Gateway Commons building was analyzed for lateral forces due to wind and seismic activity. Based on methods from ASCE 7-05 it was determined that seismic is the controlling lateral force acting on the building in both directions. CMU shear walls in each direction act as the lateral system of the building. The 13 shear walls on each floor are located in such a manner that they have created a center of rigidity a considerable distance away from the center of mass. This eccentricity creates lots of torsional shear that controls the design of the lateral system.

When looking at the shear load for each shear wall the walls further away from the center of rigidity had larger shear loading due to torsion being the governing force. This can also be seen when looking at the mode analysis. Torsion created the first mode meaning that the walls are the least stiff when trying to resist against torsion. This is shown by mode one having the largest period.

Masonry is a stiff and durable building material and it shows these characteristics when the CMU shear walls were analyzed for story drift. The actual story drift that was determined by ETABS was less than the allowable story drift calculated for every story in both directions. These drift results make sense but might be inaccurate due to the complications of adjusting the masonry shear walls to work as concrete walls on the ETABS program.

The hand calculated spot checks verified that the vertically reinforced 8” CMU shear walls were adequate to resist the shear loading due to seismic forces. The calculations for the spot checks can be found in Appendix B, C, and D. Calculations for wind and seismic loads can be found in Appendix A.

Appendix A:
Lateral Loads Calculations

Wind

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ASCE 7-05

$V = 90\,mph$ Figure 6-1

$K_d = 0.85$ Table 6-4

$I = 1$ Table 6-1

$K_{zt} = 1$ Figure 6-4

$G = 0.85$ $T_a < 1$ ∴ Rigid 6.5.8.1

$GC_{p_i} = \pm 0.18$ Fig. 6.5

$K_h = 63'$ ∴ 0.87 Table 6.3

Exposure B, Case 2

enclosed Building

Surface	C_p
Windward	0.8
Sidewall	-0.7
Leeward (N-S)	-0.5
Leeward (E-W)	-0.32

ASCE 7-05 Figure 6-6

(N-S) $L/B = \frac{58}{111} = 0.52$

(E-W) $L/B = \frac{111}{58} = 1.9$

ASCE 7-05

$q_z = 0.00256\, K_z K_{zt} K_d V^2 I$ (lb/ft^2) 6.5.10, equ. (6-15)

$q_z = 0.00256(K_z)(1)(0.85)(90)^2(1)$

$q_z = 17.6256\,(K_z)$ $[lb/ft^2]$

$q_h = 0.00256(0.87)(1)(0.85)(90)^2(1)$

$= 15.33$

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6.5.12.2.1 eqn. (6-17)

Side $q_z(0.85)(-0.7) \pm 15.33(0.18) = -q_z(0.595)$

Lee(N-S) $q_h(0.85)(-0.5) \pm 15.33(0.18) = (-15.33)(0.425) = -6.52$

Lee(E-W) $q_h(0.85)(-0.32) \pm 15.33(0.18) = (-15.33)(0.272) = -4.17$

Wind $q_z(0.85)(0.8) \pm 15.33(0.18) = q_z(0.68)$

$P = q_z GC_p - q_h GC_{pi}$ – side & Windward

$P = q_h GC_p - q_h GC_{pi}$ – Leeward

Wind loading for the N-S direction

Z (ft)	Kz	qz	$P_{sidewall}$ (psf)	$P_{leeward}$ (psf)	$P_{windward}$ (psf)	P_{total} (psf)
0-15	0.57	10.04659	-5.97772224	-6.52	6.83168256	13.3516826
20	0.62	10.92787	-6.50208384	-6.52	7.43095296	13.950953
25	0.66	11.6329	-6.92157312	-6.52	7.91036928	14.4303693
30	0.7	12.33792	-7.3410624	-6.52	8.3897856	14.9097856
40	0.76	13.39546	-7.97029632	-6.52	9.10891008	15.6289101
50	0.81	14.27674	-8.49465792	-6.52	9.70818048	16.2281805
60	0.85	14.98176	-8.9141472	-6.52	10.1875968	16.7075968
70	0.89	15.68678	-9.33363648	-6.52	10.66701312	17.1870131

Wind loading for the N-S direction

Z (ft)	Kz	qz	Psidewall (psf)	Pleeward (psf)	Pwindward (psf)	Ptotal (psf)
0-15	0.57	10.04659	-5.97772224	-4.17	6.83168256	11.0016826
20	0.62	10.92787	-6.50208384	-4.17	7.43095296	11.600953
25	0.66	11.6329	-6.92157312	-4.17	7.91036928	12.0803693
30	0.7	12.33792	-7.3410624	-4.17	8.3897856	12.5597856
40	0.76	13.39546	-7.97029632	-4.17	9.10891008	13.2789101
50	0.81	14.27674	-8.49465792	-4.17	9.70818048	13.8781805
60	0.85	14.98176	-8.9141472	-4.17	10.1875968	14.3575968
70	0.89	15.68678	-9.33363648	-4.17	10.66701312	14.8370131

Seismic

Weights per floor:

2nd Floor = 1251 k
3rd Floor = 1064 k
4th Floor = 980 k
5th Floor = 980 k
6th Floor = 959 k
Roof = 1255 k

$S_{MS} = F_a \cdot S_s = 1.6(0.159) = 0.25$

$S_{M1} = F_v \cdot S_1 = 2.4(0.055) = 0.13$

$S_1 = 0.055,\ F_v = 2.4$

$S_s = 0.159,\ F_a = 1.6$

Site Class D

$S_{DS} = 2/3\ S_{MS} = 2/3\ (0.25) = 0.167$

$S_{D1} = 2/3\ S_{M1} = 2/3\ (0.13) = 0.087$

$R = 3.5$

$I = 1$

Occupancy Catagory II

SDC = B

$T_a = C_t h_n^x$

$T_a = 0.02(65.25')^{0.75}$

$T_a = 0.459$

$C_u T_a = 1.7(0.459) = 0.78$

$$C_s \geq \begin{cases} S_{DS}/(R/I) & 0.167/3.5 = 0.048 \\ S_{D1}/[T(R/I)] & 0.087/0.78(3.5) = 0.032 \\ \dfrac{S_{D1}(T_L)}{T^2(R/I)} & \dfrac{0.087(6)}{(0.78)^2(8)} = 0.107 \end{cases}$$

$\boxed{C_s = 0.032}$

$V = 0.032(6489)\,k$

$\boxed{V = 208\ k}$

Appendix B:
Spot Check 1

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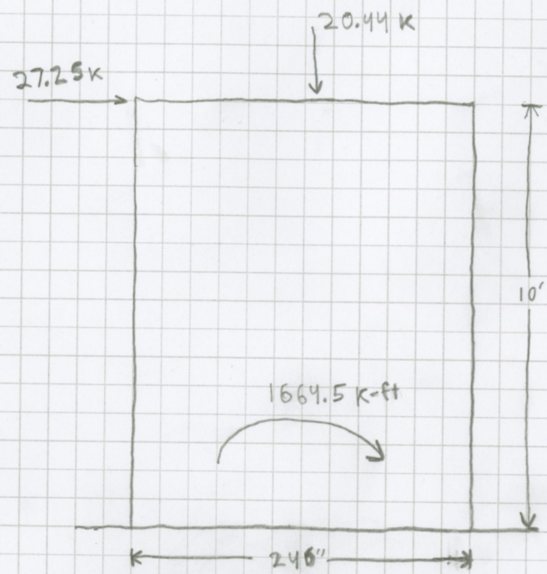

$f'_m = 2000$ psi $\quad M = 1664.5$ k-ft $\quad d = 240"$

$F_y = 50{,}000 \quad V = 27.3$ k $\quad P = 20.4$ k

$$M/Vd = \frac{1664.5}{27.3(20)} = 3.0$$

$$f = \frac{V}{bd} = \frac{27.25}{(7.625)(240)} = 14.9 \text{ psi}$$

$$F_v = 1.5\sqrt{2000} = 67.1 > 14.9 \checkmark \text{OK}$$

don't need horizontal Reinf

Reinforcing - Flexural

$$A_s = M/F_s jd = \frac{1664.5(12)}{24(0.9)(240)} = 3.8 \text{ in}^2$$

use (9) #6 $\quad A_s = 3.96$ in^2

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Axial

$h/r = 120/2.196 = 54.6$

$h = 10'(12) = 120$
$r = 0.288t = 0.288(7.625) = 2.196$

$P_a = (0.25 f'_m A_n + 0.65 A_{st} F_s)\left[1 - \left(\frac{h}{140r}\right)^2\right]$

$A_{st} = 3.96 \text{ in}^2$

$A_n = 7.625(246) - 3.96 = 1872 \text{ in}^2$

$P_a = \left[(0.25)(2)(1872) + 0.65(3.96)(24)\right]\left[1 - \left(\frac{120}{140(2.2)}\right)^2\right]$

$P_a = 649 \text{ k}$

wall self-weight

$649 > 20.44 + 16.8\text{k}$

✓ OK

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Appendix C:
Shear Wall Calculations

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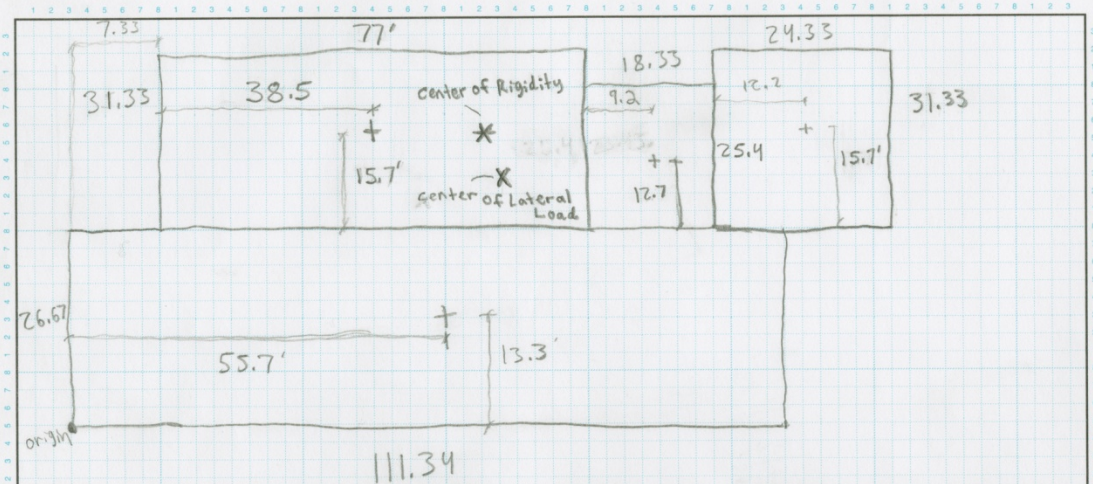

$$\bar{X} = \frac{2412.4(45.8) + 465.6(93.5) + 762.3(114.9) + 2969.4(55.7)}{2412.4 + 465.6 + 762.3 + 2969.4}$$

$$= \frac{407005.4}{6609.7} = \boxed{61.6'}$$

$$\bar{y} \quad \frac{2412.4(42.4) + 465.6(39.4) + 762.3(42.4) + 2969.4(13.3)}{6609.7}$$

$$= \boxed{29.1'}$$

Centroid of Mass

X (61.6', 29.1')

$$COM_x = \frac{\Sigma(X_{centroid})A}{\Sigma A}$$

$$Com_y = \frac{\Sigma(Y_{centroid})A}{\Sigma A}$$

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Center of Rigidity

$\Sigma K = 16{,}490.3$

1) $y = 5.33 + 8 = 13.33$
2) $y = 8.55$
3) $y = 13.33$
4) $y = 8.55$
5) $y = 26.67 + 3 + 10.2 = 39.87$
6) $y = 26.67 + 5.67 + 9.95 = 42.29$
7) $y = 26.67 + 5.67 + 12.9 = 45.24$

distance from origin to center of wall

$$\boxed{COR = \frac{\Sigma(y)(K)}{\Sigma K}}$$

stiffness of wall

$$\text{Center of rigidity}_y = \frac{2\left[13.33(2340.8) + 8.55(2362.4)\right] + 39.87(2349.6) + 42.29(2359) + 45.24(2375.3)}{16{,}490.3}$$

$$\boxed{CR_y = 24.48'}$$

1) $y = 22 + 10.75 = 32.75$
2) $y = 99.4 - 9.1 = 90.3$
3) $y = 102.4 + 6.1 = 108.5$
4) $y = 7.33 + 29.67 + 7.5 = 44.5$
5) $y = 44.5$
6) $y = 7.33 + 29.67 + 18 + 29.33 + 9.15 = 93.5$

$\Sigma K = 10{,}500.7$

$$CR_x = \frac{32.75(1741.3) + 90.3(1747) + 108.5(1754.4) + 44.5(1759.6) + 44.5(1759.6) + 93.5(1738.8)}{10{,}500.7}$$

$$\boxed{CR_x = 69'}$$

$$\boxed{CR = (69', 24.5')}$$

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11.2
45.33

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Torsional Shears

$e = 7.4'$

$V = 27.3$ K

$$V_T = \frac{V(e)(d_i)(R_i)}{J}$$

d_i	Stiffness (R)	$J = Rd_i^2$	V_T (Kips)
1) 46.7	1) 2340.8	1) 5,105,027	1) 4.3259
2) 24.55	2) 2362.4	2) 1,423,824	2) 8.2289
3) 11.25	3) 2340.8	3) 296,258	3) 17.9573
4) 29.94	4) 2362.4	4) 2,117,664	4) 6.7475
5) 35.5	5) 2349.6	5) 2,961,083	5) 5.6907
6) 22.64	6) 2359	6) 1,209,152	6) 8.9231
7) 25.74	7) 2375.3	7) 1,573,749	7) 7.8485

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Stiffness of each wall:

1. 7,575,758
2. 8,196,721
3. 7,575,758
4. 8,196,721
5. 10,101,010
6. 9,803,922
7. 12,987,013

$\Sigma = 64{,}436{,}903$

% of Load distributed on to each wall $= \frac{K_{wall}}{K_{Total}}$

1. $\frac{7{,}575{,}758}{64{,}436{,}903} = 0.118$
2. $\frac{8{,}196{,}721}{64{,}436{,}903} = 0.127$
3. 0.118
4. 0.127
5. $\frac{10{,}101{,}010}{64{,}436{,}903} = 0.157$
6. $\frac{9{,}803{,}922}{64{,}436{,}903} = 0.152$
7. $\frac{12{,}987{,}013}{64{,}436{,}903} = 0.202$

Load on 4th floor = 27.3 kips

% of load on each wall (V)

1) 0.118(27.3) = 3.2214 k
2) 0.127(27.3) = 3.4671 k
3) 0.118(27.3) = 3.2214 k
4) 0.127(27.3) = 3.4671 k
5) 0.157(27.3) = 4.2861 k
6) 0.152(27.3) = 4.1496 k
7) 0.202(27.3) = 5.5146 k

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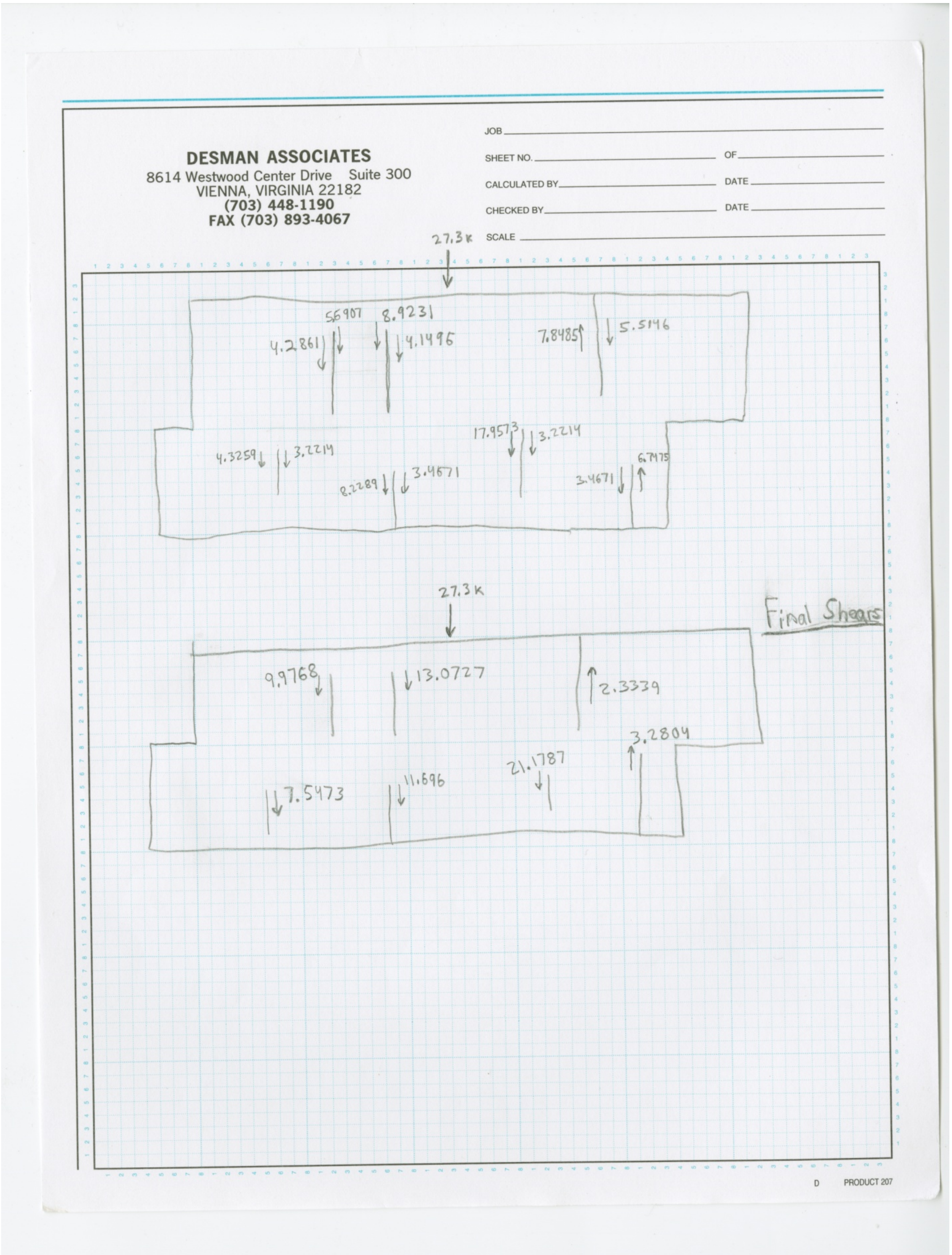
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27.3k
5.6907
8.9231
4.2861
4.1496
7.8485
5.5146
4.3259
3.2214
17.9573
3.2214
8.2289
3.4671
3.4671
6.7475
27.3k
Final Shears
9.9768
13.0727
2.3339
3.2804
21.1787
7.5473
11.696
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Appendix D:
Spot Check 2

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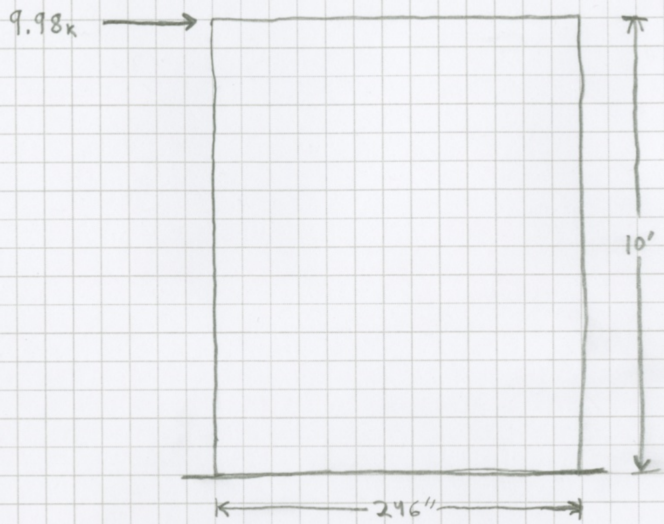

$f_y = 60$ ksi

$f'_m = 2000$ psi

$V = 9.98$ k

$M = \frac{1}{2} H V = 0.5(10)(9.98) = 50$ k-ft

$M/Vd = \frac{50(12)}{9.98(240)} = 0.25 < 1$

use $F_v = \frac{1}{2}(4 - M/Vd)\sqrt{f'_m} = 0.5(3.75)\sqrt{2000} = 83.9$

$f_v = \frac{9.98}{7.625(240)} = 5.3$ psi < 83.9 ✓OK

Reinforcing – Flexural

$A_s = M/f_s jd = \frac{50(12)}{24(0.9)(240)} = 0.12$ in^2

Spacing $\Rightarrow A_v = \frac{V_s}{F_s d} = \frac{9.98(4' \cdot 12)}{24(240)} = 0.083 \Rightarrow \frac{1}{3}A_v = \frac{1}{3}(0.083) = 0.03 < 0.12$ ✓OK

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