NATIONAL HARBOR BUILDING M

OXON HILL, MARYLAND

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Structural Option
Technical Report 3
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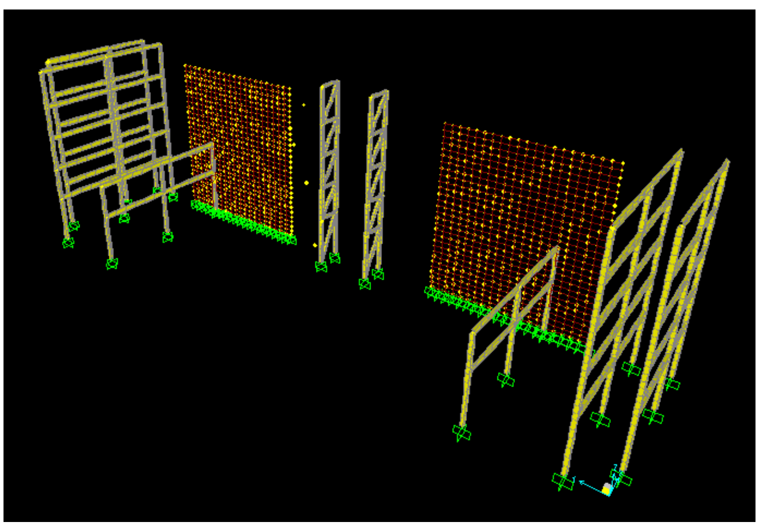
Table of Contents

EXECUTIVE SUMMARY

This report is a detailed examination of the lateral system of National Harbor Building M located in Oxon Hill, Maryland. Building M is being constructed as part of a large scale development on the banks of the Potomac River which will be known as National Harbor. It is a rectangular building in shape with rough dimensions of 243’-8” x 60’-5 ½” for approximately 14,800 square feet per floor. This five story building resists lateral forces through four masonry shear walls in the longitudinal direction, and a combination of six moment frames and two braced frames in the transverse direction.

While conducting this report, two main models of the building were prepared to aid in the analysis of the lateral systems. A RAM Structural System model was created to aid in gravity loads and mass issues, while a SAP model of the lateral system was used to analysis the systems response to lateral forces. Additionally, the results of these computer analyses were backed up by and compared to manual calculations of specific members and forces.

The controlling loads on the lateral systems were determined to be wind with a base shear of 456 kips in the transverse direction, and seismic with a base shear of 381 kips in the longitudinal direction. The results gathered through investigation suggested the transverse direction was designed according to drift criteria rather than strength criteria. The stiffness of the structure resulted in an overall maximum displacement at the roof level of 1.029”. Torsion, created by the large building width in the one direction and lateral system layout in the other, contributed some significant loads to the structure. Torsion in the transverse direction accounted for 10-15% of the total shear at the top of the building and 15-25% at the bottom. The longitudinal direction experienced much larger torsional values accounting for nearly the entire total shear, though the exactness of these numbers must be further investigated. Additionally, a more detailed investigation of distribution of the lateral loads was performed and resulted in differing factors than the previous assumptions.



STRUCTURAL SYSTEMS OVERVIEW

Floor System:

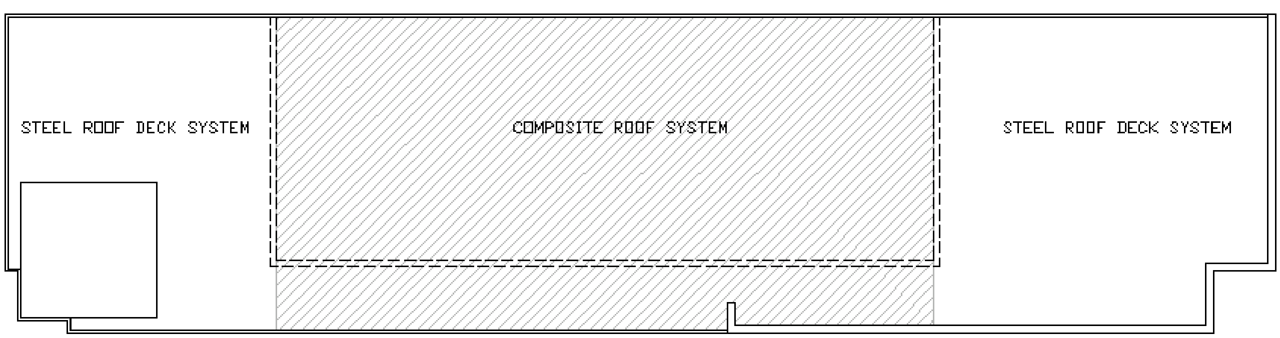
The typical floor is a 6-1/4” thick composite concrete system. It is comprised of a 3-1/4” light weight concrete slab with 3000 psi compressive strength and a 3”-20 gauge A992 (50 ksi) composite steel deck. The slab is reinforced with 6x6-10/10 draped welded wire mesh (WWM) and gains its composite properties from ¾” diameter 5-1/4” long steel studs. This composite floor system is supported by A992 wide-flange beams which are typically spaced at 10’ on center, span 30’-5-1/2” in a normal bay, and have a 1” camber. These beams range in size from W14-22 to W16x26 and are in turn supported by a grid of wide flange girders. The girders typically are spaced at 30’-5-1/2” with a 30’-0” span ranging from W18x50 to W24x84 with a 1” camber.

Column System:

The columns are ASTM 572, grade 50 or A992 steel wide flanges, and are laid out in fairly square bays (30’x30’-5-1/2” typ.) forming a mostly rectangular grid of 9 bays by 2 bays. They are the main gravity resisting members of the structure as well as a portion of the lateral resisting system. These major gravity resisting columns range from W12x65 to W14x109 at the bottom level and are spliced 4’ above the third floor level. There are lateral force resisting columns in both moment and braced frames which range from W14x99 to W14x211 at the bottom level, however, they tend to be on the order of W14x150s. These columns are also spliced at a distance 4’ above the third floor level.

Roof System:

The roof of this structure is constructed in two different systems: typical flat roof steel deck and a composite slab roof construction. The main roof is 3” 18 gauge wide rib, type N galvanized steel roof deck which is uniformly sloped. The other roof system is a 4-1/2” normal weight composite concrete slab with 3000 psi compressive strength, and reinforced by 6x6-10/10 draped WWM supported by 3” 18 gauge composite steel deck. The composite action in this slab, as in the standard floor slabs, comes from ¾” diameter 5-1/4” long equally spaced studs.



ROOF CONSTRUCTION PLAN

Foundation System:

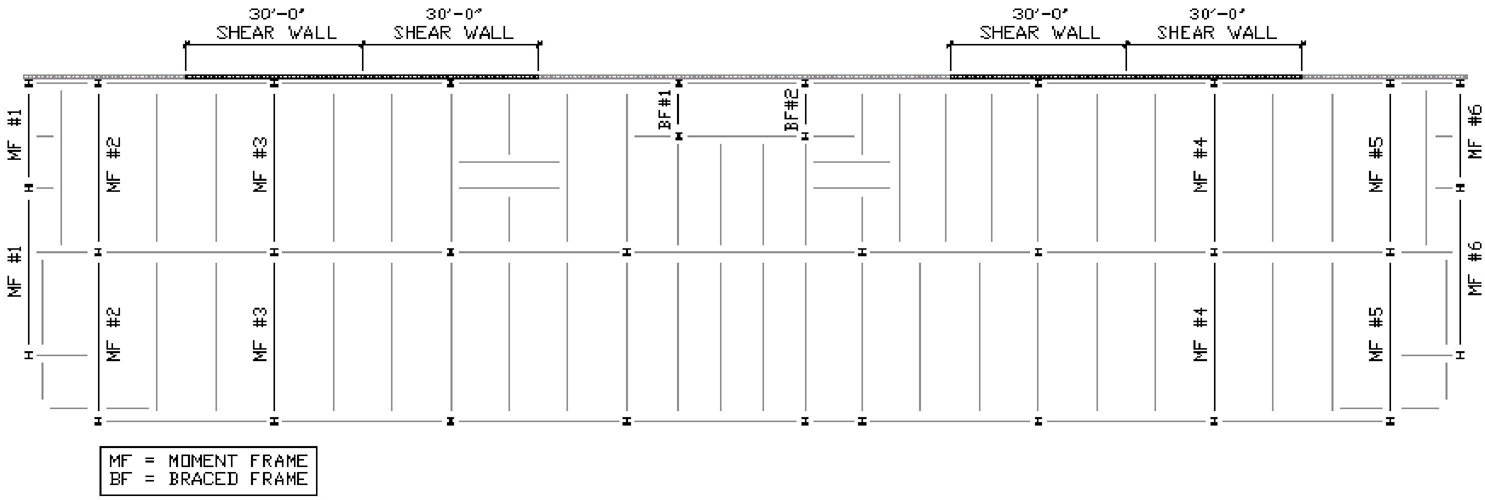
The ground floor is constructed of a 4" thick slab on grade with a compressive strength of 3000 psi and reinforced with 6x6-10/10 WWM. The columns are supported by concrete footings, compressive strength of 4000 psi, which are in turn supported by driven 14" square precast prestressed concrete piles. The piles, which have an axial capacity of 110 tons, uplift capacity of 55 tons, and a lateral capacity of 7.5 tons, are typically arranged in three pile groups under the exterior columns. These pile groups and footing combinations are connected by reinforced concrete gradebeams running around the exterior of the foundation system. The columns, which form the braced frames around the elevator core, are additionally supported by a reinforced concrete pedestal and a 43 pile mat-pile group footing. The mat supporting these piles, 18 of which are uplift piles, is approximately 21'x 48' x 64" deep.

Masonry Wall System:

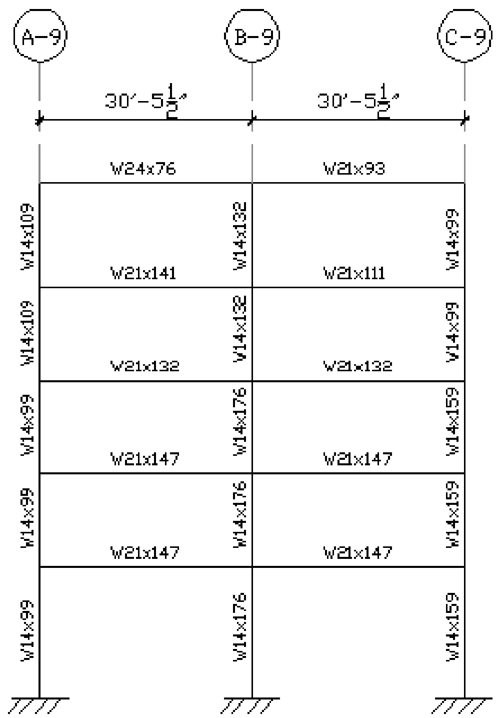
The eastern wall of the structure is backed up by a full height 8" CMU masonry wall running the length of the building, 243'-8". The wall acts as a barrier between the office building and an adjacent parking garage being concurrently constructed. It separates the two with a 4" expansion joint on the parking garage side and ties into the structure at every floor level with a standard bent plate connection every 32" on center. The wall is reinforced with one or two #6 bars at a spacing of 8"-24" on center depending on the location. It is additionally reinforced with bond beams for impact loads from the parking garage of 6000lbs at a height of 1'-6" above the floor levels. In addition to being a barrier section of the CMU wall, it also acts as (4) 30'-0" masonry shear walls to aid in the lateral force resisting system.

Lateral System:

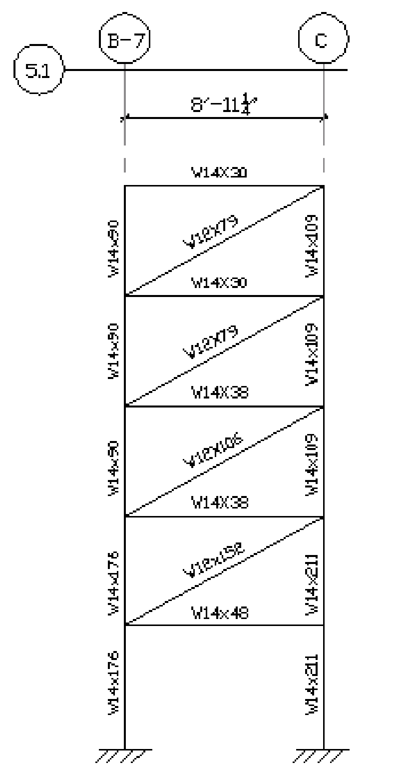
This building's lateral force resisting system is a combination of multiple system types which act together to laterally support the building. It contains (6) 2-bay moment frames which run in the east-west or transverse direction of the building. Of the (6) moment frames, only two (MF #3 and MF #4) occur at the first two levels, while the other (4) frames extend to the top of the structure. They are arranged symmetrically with (2) moment frames at each end of the grid and another at one full bay in from each end. The structure also has (2) 1-bay braced frames running in the transverse direction centrally located flanking the elevator core. These braced frames are comprised of wide flange columns, beams, and diagonal members, with the diagonal resisting members ranging from W12x79 – W12x190. The final components of the system are (4) 30'-0" reinforced masonry shear walls located in the 8" CMU wall running in the north-south or long direction of the building. The connection between the masonry wall, including the shear walls, is designed to allow the steel frames and shear walls to act independently when resisting lateral forces. Where the columns of the steel frames meet the adjacent wall the masonry is notched back to 6" from 8". The typical connection made between the concrete slab and masonry shear wall consists of a 3/8" bent plate that is vertically slotted at the shear wall face. The vertical slots allow for slabs, columns, and beams working in the transverse direction to deflect without adding out of plane stiffness to the frames. The connection, not slotted in the horizontal direction, is still able to provide lateral bracing for the masonry wall. Also, it engages the shear walls longitudinally as they resist the majority of lateral forces in that direction. Shown below is a typical framing plan calling out the lateral members, elevations of typical lateral members, and a typical connection between the frames and the masonry wall.



TYPICAL FRAMING PLAN

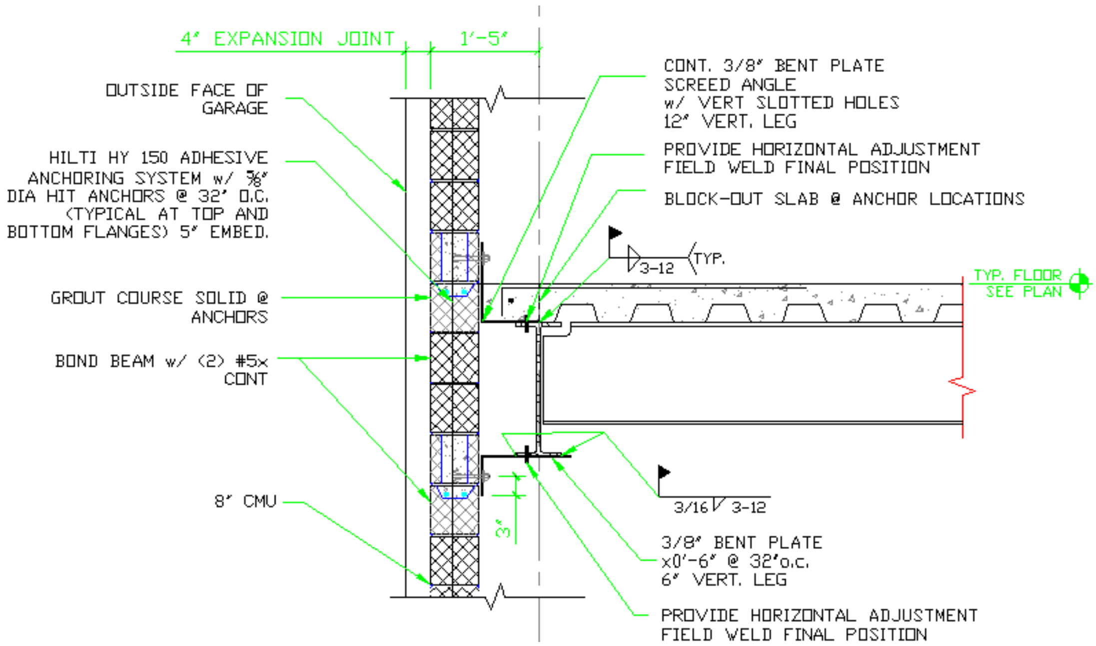


TYPICAL MOMENT FRAME



TYPICAL BRACED FRAME

*Note:
Frames not drawn to scale, all story levels are same height.



<u>Typical Slab to Shear Wall Connection</u>

LOADS

Live Loads:

Area	Design Load	ASCE 7-05 Minimum
Lobbies	100 psf	100 psf
Offices	100 psf	50 psf
1st Floor Corridors	100 psf	100 psf
Corridors above 1st Floor	100 psf	80 psf
Future Retail Tenant	100 psf	100 psf

Roof Live Loads:

Item	Design Load	Code Reference
Minimum Roof Load	30 psf + snow drift	
Ground Snow Load (Pg)	25 psf	IBC 2003 1608.2
Snow Exposure Factor (Ce)	1.0 (Exposure D, Partially exposed)	IBC 2003 1608.3.1
Thermal Factor (Ct)	1.0	IBC 1608.3.2
Snow Importance Factor (Is)	1.0	IBC 1608.4
Flat Roof Snow Load (Pf)	17.5 psf + snow drift	IBC 1608.3
Minimum (Pf) used	20 psf + snow drift	

Dead Loads:

Item	Design Load
Floor	51 psf
Composite Roof	35 psf
Non-Composite Roof	25 psf
Misc. (Flooring/Ceiling/MEP)	10 psf
Canopies	25 psf
8” CMU Wall	40 psf
Additional Loadings	As Noted in Calculations

Wall Loads:

Item/Location	Design Load (per foot along floor level)
Partition	150 plf
Glass Tower	320 plf
2nd Floor Front Glass	230 plf
3rd Floor Front Glass	150 plf
3rd Floor Architectural Precast	300 plf
3rd/4th Floor Brick	650 plf
5th Floor Front Glass	620 plf
5th Floor Brick	730 plf
5th Floor Architectural Precast	620 plf
Typical Glass Wall	280 plf
Typical Parapet	260 plf
Brick Parapet	260 plf

SEISMIC ANALYSIS

Introduction:

While seismic conditions are not generally a governing load analysis case in the coastal Maryland region, code dictates that most new structures in the United States consider its effects. The geometrical shape of the building (a long narrow rectangle) would limit the effect of wind in the longitudinal direction, opening the possibility for seismic forces to control lateral design along the path. In order to correctly analyze this building, the design professionals decided to analyze the two main axes of the building (longitudinal and transverse) separately. I concur that this is an effective approach. Since the lateral system of building differs in these two directions, it was appropriate to consider each individually. After making this distinction, I proceeded using the Equivalent Lateral Force Procedure for my analysis.

General Analysis:

Item	**Design Value**	**Code Reference (ASCE 07-05)**
Seismic Use Group	Group I	Table 1-1
Seismic Design Category	B	11.4.2
Importance Factor (I)	1.0	
Spectral Acceleration for a One Second Period (S1)	0.063g	11.4.3
Spectral Acceleration for Short Period (Ss)	0.177g	11.4.3
Design Spectral Response Acceleration Parameter for a One Second Period (Sd1)	0.101 g	11.4.4
Design Spectral Response Acceleration Parameter for a Short Period (Sds)	0.189g	11.4.4
Seismic Weight (Wt)	7,072K	

***Calculations found in Appendix**

Transverse Direction:

Item	**Design Value**	**Code Reference (ASCE 07-05)**
Basic Structural System	Steel Systems Not Specifically Detailed for Seismic Resistance	Table 12.2-1
Response Modification Factor R	3.0	12.2.3.1
Deflection Amplification Factor (Cd)	3.0	12.2.3.1
Fundamental Period (T)	1.48	12.8.2
Seismic Response Coefficient (Cs)	0.0227	12.8.1.1
Design Base Shear	160.5K	12.9.4

***Calculations found in Appendix**

Longitudinal Direction:

Item	Design Value	Code Reference (ASCE 07-05)
Basic Structural System	Dual System with Intermediate Moment Frames	Table 12.2-1
Seismic Resisting System	Intermediate Reinforced Masonry Shear Wall	Table 12.2-1
Response Modification Factor R	3.5	12.2.3.1
Deflection Amplification Factor (Cd)	3.0	12.2.3.1
Fundamental Period (T)	0.851	12.8.2
Seismic Response Coefficient (Cs)	0.0339	12.8.1.1
Design Base Shear	239.7K	12.9.4

***Calculations found in Appendix**

The seismic weight of the building is calculated by adding the buildings total dead load, 25% of the live load for storage areas, partition loads greater than 10 psf, permanent equipment loads, and 20% flat roof snow load greater than 30 psf. In this particular building, the only additional load to the total dead load that was applicable, was permanent equipment loading. Also worth noting for ease of calculation, a weighted average of the wall loads listed in the load section was calculated for each individual floor. A wall load of 7 psf was applied to the exterior of the tower, 35 psf was applied to the exterior of levels 2 -5 (combination of brick, precast, and architectural glass), and 25 psf was applied from the ground up to the 2nd level (mostly store front glass with brick and precast accents).

Seismic Weight Summary:

Item	Weight
Architectural Tower	16.3K
Elevator Tower	22.1K
Roof Level	930K
5th Floor Level	1,669K
4th Floor Level	1,380K
3rd Floor Level	1,380K
2nd Floor Level	1,674K
Total	7,072K

***Calculations found in Appendix**

Conclusion:

Upon comparing my seismic analysis with the actual seismic base shear numbers used in the design of this building by the engineers of record, three things became apparent: 1. The seismic base shear numbers I calculated for the longitudinal direction (239.7 K) were approximately 1.6 times less than the design values in the same direction (391 K). 2. Since my numbers for the factors SDS, SD1, and R matched the listed design factors on the drawings, the fundamental period used in the calculations must be where we were differing. 3. The design numbers appear to define the transverse system as an eccentrically braced steel frame system with regards to the Ct and x variables, while I choose to define them as a steel moment-resisting structural system. Since the two braced frames in the transverse direction are not eccentrically connected, and the frames' connection to the shear wall does not prevent them from deflecting, I felt comfortable defining them as such.

After looking further into the code and speaking with the design engineers of the building, I was able to determine our calculations were in fact differing in how we calculated the fundamental period of the structure. Period determination (ASCE 07-12.8.2) is allowed by code to be the minimum of an approximate fundamental period Ta (ASCE 07-12.8-7) times an optional factor Cu and the actual fundamental period Tb, where Tb is calculated in a properly substantiated computer analysis. In my calculations, because I had not compiled a full model of the building capable of the determining the fundamental period, I simply assumed the approximate fundamental period I calculated (1.48 sec transversely and 0.851 sec longitudinally) would be of close enough accuracy. In speaking with the design engineer, I discovered that they had analyzed the building for its true fundamental period (1.277 sec transversely and 0.344 sec longitudinally). Plugging the new period Tb back into my calculations, I was able to obtain base shear numbers (174.7K transversely and 381.2K longitudinally) similar to the design numbers only differing slightly. This was probably due to a result of seismic weight being off by a small percent. Looking at the new base shear numbers, it is clear that longitudinal direction will be more heavily influenced by seismic forces. My use of the approximate fundamental period would have allowed the building to be designed for 40% less seismic base shear in the longitudinal direction. Since in this direction seismic force will control over wind (see lateral analysis section for comparison vs. wind), my base shear number would have been very unconservative. Seeing these results, I would conclude that if there is even a remote chance that seismic forces could control design in a specific direction, it would be most beneficial to develop a model capable of determining the actual fundamental period of the building.

WIND ANALYSIS

Introduction:

The orientation and geometric shape of National Harbor Building M both play a role in making wind a clear controlling lateral force in at least one of its axes. The building is located on the banks of the Potomac River with no obstructions between it and the wind coming off the water. Additionally, a bend in the river at the location of Building M, making it just over a mile wide, and the building's close proximity to the edge of the water, force it to be defined as Exposure D. Building M is oriented in such a way that its largest face in terms of surface area is directly facing the water. While not an extremely tall building, at only 74 feet tall, it is fairly long in this direction, at 274 feet, creating approximately 20,000 plus square feet of surface area taking wind directly from the water. To further complicate matters, there is a parking garage being built simultaneously on the opposite side of the building (perpendicular to the main path of wind), separated by only a four inch expansion joint. Since the large surface area taking wind directly from the water will control in this direction (see lateral analysis section for comparison vs. seismic), the lateral system must be capable of resisting these forces to within a 4 inch drift.

The adjacent parking garage also played a role in the original approach I used to analyze the wind forces on Building M. The proximity of the parking garage to the building, along with an assumption that the parking garage, which serves the office building, would be standing for the life of the office building, caused me to originally consider three separate wind path cases. First, I analyzed wind coming off the water and applying forces in the transverse direction to the building. In this case I discounted the effects of leeward wind force assuming that they would be handled only by the adjacent garage. Second, I analyzed wind coming from the land side transversely into the building, in this case discounting the windward forces taken by the garage. The final case I looked at was the longitudinal direction which handled a combination of both windward and leeward forces because there were no structures adjacent to the building in that direction. After review of my first technical report and further discussion with the engineer of record on National Harbor Building M, I decided to reexamine the transverse wind case. While it is reasonable to assume the adjacent garage will be standing for the life of the office building, the fact that it is designed as a non-enclosed structure presents some problems. The openness of a parking structure will allow some wind forces to act on the masonry wall face of Building M. The use of the entire composite wind pressure will be conservative since the garage structure will absorb some of the wind load, but completely ignoring the composite effects could possibly lead to an under design of the office building. Thus the new composite numbers lead to higher base shear values for the transverse wind load.

In determining the rigidity of my building, I choose to use the approximate fundamental period (Ta) in each direction which was previously calculated in the seismic section. Taking the inverse of these numbers gave me the fundamental frequency of the building in each direction. With both frequencies being greater than a value of 1.0, I was able to assume rigidity in each direction, and used the corresponding factors and equations to compute the values below.

General Wind Data:

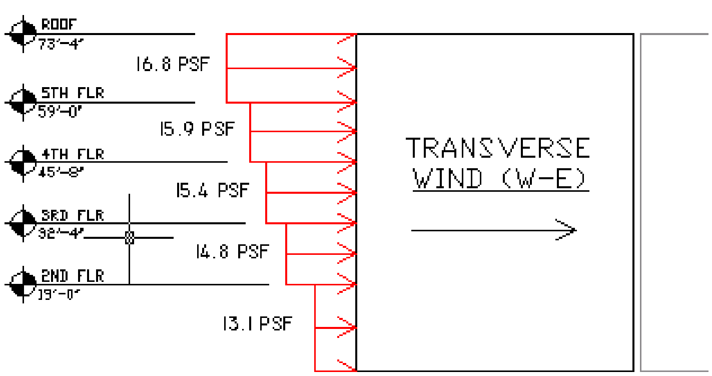
Item	Transverse Wind	Longitudinal Wind	Code Reference (ASCE7-05)
Build Type	Rigid	Rigid	6.2
Exposure	D	D	6.5.6
Importance Factor (I)	1.0	1.0	6.5.5
Basic Wind Speed (V)	90	90	6.5.4
Gust Factor (G)	0.861	0.884	6.5.8
Cp Windward	0.8	0.8	6.5.11
Cp Leeward	-0.5	-0.2	6.5.11
Kzt	1.0	1.0	6.5.7
Kd	0.85	0.85	6.5.4

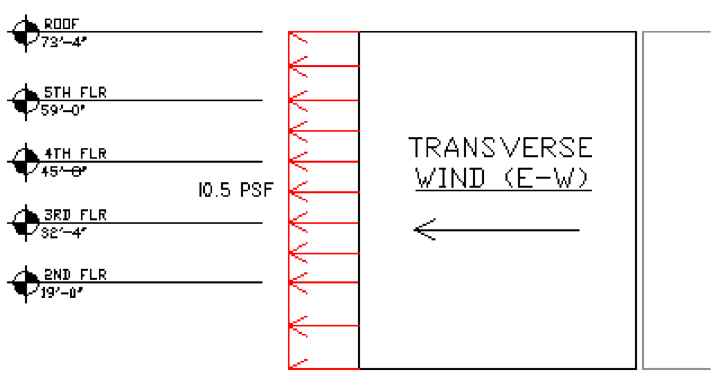
***Calculations found in Appendix**

Transverse Wind:

			Case 1: W-E		Case 2: E-W	
Elevation	**Kz**	**q**	**Windward P(psf)**	**Leeward P (psf)**	**Windward P (psf)**	**Leeward P(psf)**
0 - 19'-0"	1.08	19.04	13.1	0	0	-10.5
19'-0" – 32'-4"	1.22	21.50	14.8	0	0	-10.5
32'-4" – 45'-8"	1.27	22.38	15.4	0	0	-10.5
45'-8" – 59'-0"	1.31	23.09	15.9	0	0	-10.5
59'-0" – 74'-0"	1.38	24.32	16.8	0	0	-10.5

***Calculations found in Appendix**



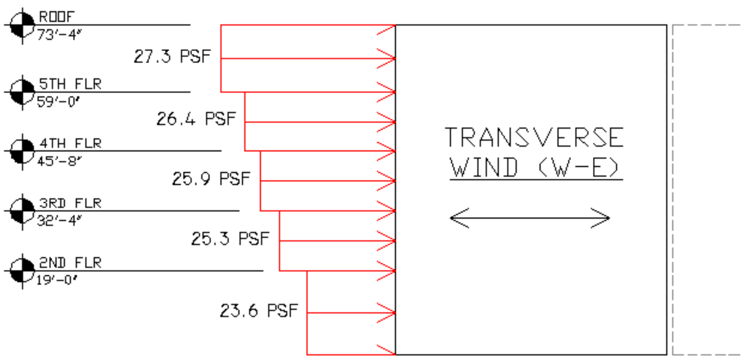


Transverse Wind (Considering Both Directions):

Case 3: E-W/W-E

Elevation	Kz	q	Windward P(psf)	Leeward P (psf)	Total P (psf)
0 - 19'-0"	1.08	19.04	13.1	-10.5	23.6
19'-0" – 32'-4"	1.22	21.50	14.8	-10.5	25.3
32'-4" – 45'-8"	1.27	22.38	15.4	-10.5	25.9
45'-8" – 59'-0"	1.31	23.09	15.9	-10.5	26.4
59'-0" – 74'-0"	1.38	24.32	16.8	-10.5	27.3

***Calculations found in Appendix**

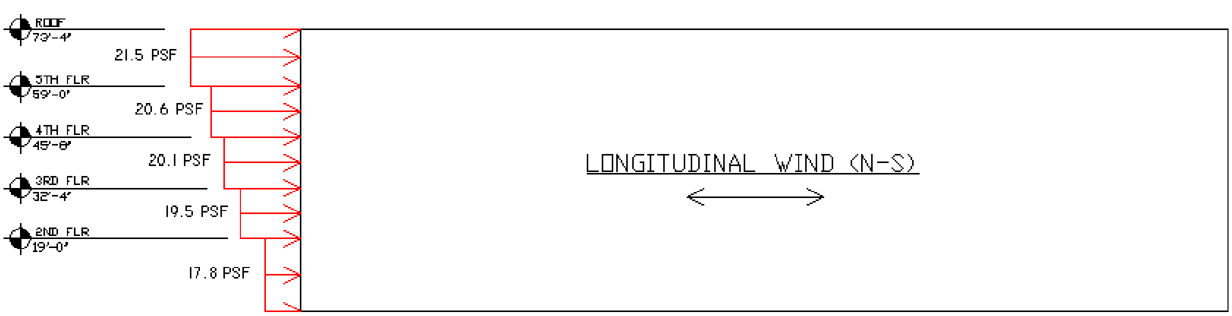


Longitudinal Wind:

Case 1: N – S/S-N

Elevation	Kz	q	Windward P(psf)	Leeward P (psf)	Total P (psf)
0 - 19'-0"	1.08	19.04	13.8	-4.3	17.8
19'-0" – 32'-4"	1.22	21.50	15.2	-4.3	19.5
32'-4" – 45'-8"	1.27	22.38	15.8	-4.3	20.1
45'-8" – 59'-0"	1.31	23.09	16.3	-4.3	20.6
59'-0" – 74'-0"	1.38	24.32	17.2	-4.3	21.5

***Calculations found in Appendix**



Conclusion:

The pressure in the transverse direction is much greater than in the longitudinal direction once both windward and leeward pressures are considered. This is an expected outcome, and will likely cause the building to be designed based on drift in that direction as it drifts toward the adjacent structure. The large differential between the three transverse cases also points out that the actual load the building will see is probably somewhere in the middle. The remainder of this report will use the combined transverse loading as it is more conservative, rather than the individual cases which may be under conservative. The relatively small longitudinal base shear backs up a previous assumption that the small surface area in that direction could lead to wind not controlling.

Wind Base Shear Summary:

Item	**Transverse (W-E)**	**Transverse (E-W)**	**Transverse (E-W/W-E)**	**Longitudinal (N-S/S-N)**
Wind Base Shear	269K	182K	456K	88K

***Calculations found in Appendix**

LATERAL SYSTEM ANALYSIS

Introduction:

As mentioned previously in this existing conditions report, the lateral support system of Building M consists of two separate systems, one along each axis of the building. The first step in beginning to analyze each of these systems is to know what lateral will control the design. After computing lateral loads in both directions of the building for both seismic and wind loads, I was able to determine which controlled for each case. As the chart below points out, the transverse axis of the building, which is laterally supported by moment and braced frames, will be controlled by wind loads with a base shear of 456K. Along the longitudinal axis, supported by four 30'-0" masonry shear walls, seismic forces will control with a total base shear value of 350K.

Controlling Base Shear Summary:

Item	Transverse (W-E)	Longitudinal (N-S)
Wind	**456K**	88K
Seismic	175K	**381K**

***Numbers in Bold Control**

Now that the controlling base shear numbers in each direction are known, they must be distributed vertically to each individual story of the building. Since each direction is controlled by a different type of lateral force, each must be distributed differently. The longitudinal seismic distribution is based on a formula which takes into account the building's period, each story's seismic weight, and each story's height from ground level. The transverse wind distribution is dependent on tributary area of each floor on the face of the building and the differing pressure on that area. It should be noted that the total story shear applied by wind does not equal the base shear number calculated because of the tributary area from the bottom half of the first level. This force is assumed to be applied at the base of the structure, and thus not affecting the lateral frame. These distributions and their accompanying overturning moments are summarized below, and more detailed calculations can be found in the appendixes. With the story forces for each lateral load determined, distribution factors were obtained to further distribute these forces to each frame or shear wall of the lateral systems. This was accomplished using a combination of both modeling analysis and hand calculations.

Seismic Story Force Distribution in Longitudinal Direction:

Item	Seismic Weight	Cv Factor	Story Force	Overturning Moments (Mx)
Roof Level	968K	0.232	88.4K	6,571 ft K
5th Floor Level	1669K	0.318	121.2K	7,151 ft K
4th Floor Level	1380K	0.203	77.4K	3,535 ft K
3rd Floor Level	1380K	0.144	54.9K	1,775 ft K
2nd Floor Level	1674K	0.102	38.9K	739 ft K
Total	7,072K	1.0	381K	19,771 ft K

***Calculations found in Appendix**

Wind Story Force Distribution in Transverse Direction:

Item	Trib. Width	Trib Height	Story Force	Overturning Moments (Mx)
Roof Level	243.67’	7.165’	47.5K	3,531 ft K
5th Floor Level	243.67’	13.85’	90.4K	5,334 ft K
4th Floor Level	243.67’	13.33’	84.9K	3,877 ft K
3rd Floor Level	243.67’	13.33’	83.1K	2,687 ft K
2nd Floor Level	243.67’	16.165’	95.6K	1,816 ft K
Total			401.5K	17,245 ft K

***Calculations found in Appendix**

Distribution:

With the story forces for each lateral load determined, distribution factors were obtained to further distribute these forces to each frame or shear wall of the lateral systems. This was accomplished using a combination of both modeling analysis and hand calculations. A model of the building including mass and gravity loading was complied in RAM Structural Systems and a simplified model of the lateral system was prepared in SAP. The centers of mass per each story level were located via the RAM model and each controlling lateral story load was applied at that point in the SAP model. The lateral system was modeled with a rigid diaphragm at each story allowing the SAP model to distribute the loads based on relative stiffness of the lateral members. After the model was run with the applied loads, the shears of all the lateral members were recorded. Summing these values and taking them as a percent as the total applied shear wall allows for calculation of a distribution factor for each individual frame or shear wall. These distribution factors can later be used to calculate loads on specific lateral frames for simplified analysis and member checks as opposed to analyzing the entire structure at once.

Distribution factors determined by relative stiffness will vary from the top of the structure to the base. In this given structure, two of the lateral frames only occur at the bottom two levels causing the distribution at the top to further differ from that at the bottom. Based on these two points, separate distribution factors were calculated for both the top and bottom of the structure. The bottom distribution factor will be applied to the lower two stories, which contain all eight transverse frames. The top distribution factor will be applied to the upper three stories, which contain the six full height frames. These distribution factors are summarized below and further detailed calculations can be found in the appendix.

TRANSVERSE DIRECTION	Distribution Factor	Story Shear(K)		Distribution Factor	Story Shear(K)		
LATERAL MEMBER	AT BOTTOM	2	3	AT TOP	4	5	R
MF #1	0.055	5.26	4.57	0.176	14.94	15.91	8.36
MF #2	0.055	5.26	4.57	0.183	15.54	16.54	8.69
MF #3	0.032	3.06	2.66	--	--	--	--
BF #1	0.358	34.22	29.75	0.159	13.50	14.37	7.55
BF #2	0.357	34.13	29.67	0.155	13.16	14.01	7.36
MF #4	0.032	3.06	2.66	--	--	--	--
MF #5	0.055	5.26	4.57	0.175	14.86	15.82	8.31
MF #6	0.055	5.26	4.57	0.152	12.90	13.74	7.22
Out of Plane SW	0.001	0.10	0.08	0	0	0	0
	TOTAL LOAD (per story):	95.6	83.1		84.9	90.4	47.9
LONGITUDINAL DIRECTION	Distribution Factor	Story Shear(K)		Distribution Factor	Story Shear(K)		
LATERAL MEMBER	AT BOTTOM	2	3	AT TOP	4	5	R
SW #1	0.244	9.49	13.40	0.25	19.35	30.3	22.28
SW #2	0.244	9.49	13.40	0.25	19.35	30.3	22.28
SW #3	0.244	9.49	13.40	0.25	19.35	30.3	22.28
SW #4	0.244	9.49	13.40	0.25	19.35	30.3	22.28
Out of Plane Frames	0.024	0.93	1.32	0	0	0	0
	TOTAL LOAD (per story):	38.9	54.9		77.4	121.2	89.12

Summary of Distribution Percentages:

TRANSVERSE	STORY 1	STORY 5
MF # 1	5.5%	17.6%
MF # 2	5.5%	18.3%
MF # 3	3.3%	0
BF # 1	35.8%	15.9%
BF # 2	35.7%	15.5%
MF # 4	3.2%	0
MF # 5	5.5%	17.5%
MF # 6	5.5%	15.2%
LONGITUDINAL		
SW # 1	24.4%	25%
SW # 2	24.4%	25%
SW # 3	24.4%	25%
SW # 4	24.4%	25%

***Calculations found in Appendix**

It can be seen that while the braced frames are not as large in terms of frame dimensions as the moment frames they defiantly absorb a large portion of the load. A difference in distribution of load can be seen from the top of the transverse direction to the bottom. While this is not uncommon in a lateral system, it is further emphasized in this one with the significant change in overall stiffness occurring at the fourth story. The longitudinal direction distributed the loads evenly, as would be expected seeing that four shear walls all run along the same axis. The drop in distribution factor from top to bottom in this direction reflects the fact that the out of plane steel columns are providing a small amount of stiffness towards the bottom of the structure.

Drift:

Drift is a critical and possibly controlling factor for National Harbor Building M in the transverse direction. In this direction there is a 4” expansion separating the building’s frame from the adjacent parking structure. The critical transverse direction is resisted by the steel moment and braced frames, and is controlled by wind loading. While the expansion joint gives four inches of clearance to the adjacent building, it is logical to assume the engineer of record would like to keep the drift somewhat less than that. Furthermore, typical engineering practice limits maximum drift of building to approximately H/400. In this case, that would limit the maximum drift of Building M to approximately 2.23” or a little over half the distance of the expansion joint. While not as critical, the longitudinal direction was also investigated to confirm that the story drift is within a reasonable amount.

Both the transverse and longitudinal story drifts for the controlling lateral forces were calculated using the SAP model. This model confirmed that both were indeed within the practical story drift guide lines. Additionally, a manual calculation was done to confirm the model’s drift calculation for the transverse or critical direction. In analyzing the story drift, the assumption that each story will drift uniformly was made. This can be seen in the model by the assignment of diaphragms to each level ensuring uniform drift. In the manual, calculation moment frame 2 was analyzed with story forces being distributed based on previously calculated distribution factors. The approximate drift calculation was then carried out using a method which took into account moments created by the distributed forces on the moments and beams, and those member respective moments of inertia. While it is a very approximate method, the drift check was done in the critical direction, just as a means to double check the model was reporting on the correct magnitude. Both the model drifts and the manual drifts are summarized below, and more detailed calculations can be found in the appendix.

Story Drift Summary:

Item	Transverse (SAP)	Transverse (approx.)	Longitudinal (SAP)
Roof Level	1.029” (+.184”)	0.956” (+.155”)	0.603” (+.080”)
5th Floor Level	0.845” (+.214”)	0.801” (+.177”)	0.532” (+.109”)
4th Floor Level	0.631” (+.223”)	0.624” (+.185”)	0.432” (+.133”)
3rd Floor Level	0.408” (+.187”)	0.439” (+.151”)	0.290 (+.117”)
2nd Floor Level	0.221”	0.288”	0.173”

***Calculations found in Appendix**

As it can be seen, both the transverse and longitudinal story drifts are below the general guideline of 2.23” for this building height. Additionally, the manual calculation of drift confirms that the model is on the right magnitude in the critical direction. With the drift numbers being safely under the 4 inch expansion joint width, it is safe to assume that the structure is stiff enough to resist impacting the adjacent structure.

Torsion:

The layout of National Harbor Building M dictates that torsion will play a significant role in the design of its lateral systems. The transverse lateral system is symmetrical about the central axis of the building; however, the width in this direction could lead to torsion problems. The longitudinal lateral system will experience a great deal of torsion. This is because all four shear walls lie along the same axis which is along the exterior of the structure.

To determine the torsion, the first step was to locate the centers of mass and centers of rigidity of each story. The story shear forces act through the center of rigidity, and each story rotates about its center of mass creating the rotational torsion force. These points were determined in a RAM model which was completely loaded and included material masses. Since the eccentricities in the transverse direction are fairly small, an "accidental" eccentricity was used in this direction. This "accidental" eccentricity is taken as 5% of the building width in the given direction, 13.68' in this case, and is used to account for frames located far from the center of mass in a long building.

Summary of RAM Output for Centers of Mass and Rigidity:

	Center of Mass		**Center of Rigidity**	
	X-Coordinate	**Y-Coordinate**	**X-Coordinate**	**Y-Coordinate**
Roof Level	29.59'	123.80'	61.60'	122.92'
5th Floor Level	27.73'	126.27'	61.60'	122.92'
4th Floor Level	28.50'	121.64'	61.60'	122.92'
3rd Floor Level	28.83'	115.15'	61.60'	122.92'
2nd Floor Level	32.86'	121.12'	61.60'	122.92'

Summary of Eccentricities:

	X-Coordinate	**Y-Coordinate**
Roof Level	32.01'	0.88'
5th Floor Level	33.87'	3.35'
4th Floor Level	33.10'	1.28'
3rd Floor Level	32.77'	7.77'
2nd Floor Level	28.74'	1.80'

After these points were obtained, the rigidity of each frame at every story level, and the torsional rigidity factors of each story were calculated. A unit load was applied to the lateral model developed in SAP, and the stiffness of the frames was determined based on their given shears and displacements. Next, a summing of the rigidity of the lateral members and the square of the distance from the center of mass was used to generate each story's torsional rigidity factor.

	RIGIDITY PER FRAME (K / INCH) AT EACH STORY				
TRANSVERSE	**5**	**4**	**3**	**2**	**1**
DELTA	1.616	1.412	1.1387	0.78	0.714
MF 1	95.0	108.7	134.8	86.0	94.0
MF 2	117.1	134.1	166.2	84.6	92.4
MF 3	0.0	0.0	0.0	45.8	50.0
BF 1	102.0	116.7	144.7	440.8	481.5
BF 2	104.0	119.0	147.5	432.6	472.5
MF 4	0.0	0.0	0.0	41.8	45.7
MF 5	111.9	128.0	158.8	74.3	81.2
MF 6	88.8	101.6	126.0	75.5	82.5
TORSIONAL RIGIDITY(J) (K / INCH) FT^2	5541062	6341612	7863665	4995525	5457296
LONGITUDINAL	**5**	**4**	**3**	**2**	**1**
DELTA	0.6	0.533	0.466	0.341	0.458
SW 1	416.6	468.9	536.4	717.0	533.8
SW 2	416.6	468.9	536.4	717.0	533.8
SW 3	416.6	468.9	536.4	717.0	533.8
SW 4	416.6	468.9	536.4	717.0	533.8
TORSIONAL RIGIDITY (J) (K / INCH) FT^2	888	1000	1143	1528	1138

Finally the torsional shear in each frame was found by using the equation:

$$\mathbf{V_{torsion} = (V_{story} * e * D * R) / J}$$

Where**:** e = eccentricity
D = distance from lateral member to center of rigidity
R = rigidity of member or its relative stiffness
J = torsional rigidity factor

The absolute value of these torsional shear values should be added to each frame, and then accounted for in design. In the transverse direction, the accidental torsion values are, for the most part, a small percent of the direct shear numbers determined. The exterior frames take greatest amount of torsional shear, as would be expected, with their distance from the center of mass. The real concern with the design of National Harbor Building M comes from the longitudinal direction, which as expected, must resist large additional shear values from torsion. These large torsional shear values stem from the only lateral resisting members in this direction, being located on the same axis at the exterior of the plan. This layout dramatically increases the eccentricity and drives down the torsional rigidity factor, thus leaving the longitudinal system vulnerable to large torsion shears. In addition to the masonry shear walls in this direction, assistance from the out of plane frames via the rigid diaphragm will be needed to resist the twisting affect produced by the torsional shear. Listed below are tables of the torsional shear calculations, direct shear, and resulting total shears.

STORY 5

UNITS	(FT)	(K)	(K / INCH)	(FT)	(K / INCH)* FT^2	(K)	(K)	(K)
TRANSVERSE	ECCENTRICITY	STORY SHEAR	RIGIDITY	DIST. TO CR	J	TORSIONAL SHEAR	DIRECT SHEAR	TOTAL SHEAR
MF 1	13.68	47.5	95.0	120.8	5541062	1.35	8.36	9.71
MF 2	13.68	47.5	117.1	108.9	5541062	1.50	8.69	10.19
MF 3	13.68	47.5	0.0	78.9	5541062	0.00	0	0.00
BF 1	13.68	47.5	102.0	10.1	5541062	0.12	7.55	7.67
BF 2	13.68	47.5	104.0	11.5	5541062	0.14	7.36	7.50
MF 4	13.68	47.5	0.0	81.89	5541062	0.00	0	0.00
MF 5	13.68	47.5	111.9	111.9	5541062	1.47	8.31	9.78
MF 6	13.68	47.5	88.8	122.92	5541062	1.28	7.22	8.50
LONGITUDINAL								
SW 1	32.01	88.4	416.6	0.73	888	969.10	22.28	991.38
SW 2	32.01	88.4	416.6	0.73	888	969.10	22.28	991.38
SW 3	32.01	88.4	416.6	0.73	888	969.10	22.28	991.38
SW 4	32.01	88.4	416.6	0.73	888	969.10	22.28	991.38

STORY 4

UNITS	(FT)	(K)	(K / INCH)	(FT)	(K / INCH)* FT^2	(K)	(K)	(K)
TRANSVERSE	ECCENTRICITY	STORY SHEAR	RIGIDITY	DIST. TO CR	J	TORSIONAL SHEAR	DIRECT SHEAR	TOTAL SHEAR
MF 1	13.68	90.4	108.7	120.8	6341612	2.56	15.91	18.47
MF 2	13.68	90.4	134.1	108.9	6341612	2.85	16.54	19.39
MF 3	13.68	90.4	0.0	78.9	6341612	0.00	0	0.00
BF 1	13.68	90.4	116.7	10.1	6341612	0.23	14.37	14.60
BF 2	13.68	90.4	119.0	11.5	6341612	0.27	14.01	14.28
MF 4	13.68	90.4	0.0	81.89	6341612	0.00	0	0.00
MF 5	13.68	90.4	128.0	111.9	6341612	2.79	15.82	18.61
MF 6	13.68	90.4	101.6	122.92	6341612	2.44	13.74	16.18
LONGITUDINAL								
SW 1	33.87	121.2	468.9	0.73	1000	1405.14	30.3	1435.44
SW 2	33.87	121.2	468.9	0.73	1000	1405.14	30.3	1435.44
SW 3	33.87	121.2	468.9	0.73	1000	1405.14	30.3	1435.44
SW 4	33.87	121.2	468.9	0.73	1000	1405.14	30.3	1435.44

STORY 3

UNITS TRANSVERSE	(FT) ECCENTRICITY	(K) STORY SHEAR	(K / INCH) RIGIDITY	(FT) DIST. TO CR	(K / INCH) *FT^2 J	(K) TORSIONAL SHEAR	(K) DIRECT SHEAR	(K) TOTAL SHEAR
MF 1	13.68	84.9	134.8	120.8	7863665	2.41	14.94	17.35
MF 2	13.68	84.9	166.2	108.9	7863665	2.67	15.54	18.21
MF 3	13.68	84.9	0.0	78.9	7863665	0.00	0	0.00
BF 1	13.68	84.9	144.7	10.1	7863665	0.22	13.5	13.72
BF 2	13.68	84.9	147.5	11.5	7863665	0.25	13.16	13.41
MF 4	13.68	84.9	0.0	81.89	7863665	0.00	0	0.00
MF 5	13.68	84.9	158.8	111.9	7863665	2.62	14.86	17.48
MF 6	13.68	84.9	126.0	122.92	7863665	2.29	12.9	15.19
LONGITUDINAL								
SW 1	33.1	77.4	536.4	0.73	1143	877.68	19.35	897.03
SW 2	33.1	77.4	536.4	0.73	1143	877.68	19.35	897.03
SW 3	33.1	77.4	536.4	0.73	1143	877.68	19.35	897.03
SW 4	33.1	77.4	536.4	0.73	1143	877.68	19.35	897.03

STORY 2

UNITS TRANSVERSE	(FT) ECCENTRICITY	(K) STORY SHEAR	(K / INCH) RIGIDITY	(FT) DIST. TO CR	(K / INCH)*FT^2 J	(K) TORSIONAL SHEAR	(K) DIRECT SHEAR	(K) TOTAL SHEAR
MF 1	13.68	83.1	86.0	120.8	4995525	2.36	4.57	6.93
MF 2	13.68	83.1	84.6	108.9	4995525	2.10	4.57	6.67
MF 3	13.68	83.1	45.8	78.9	4995525	0.82	2.66	3.48
BF 1	13.68	83.1	440.8	10.1	4995525	1.01	29.75	30.76
BF 2	13.68	83.1	432.6	11.5	4995525	1.13	29.67	30.80
MF 4	13.68	83.1	41.8	81.89	4995525	0.78	2.66	3.44
MF 5	13.68	83.1	74.3	111.9	4995525	1.89	4.57	6.46
MF 6	13.68	83.1	75.5	122.92	4995525	2.11	4.57	6.68
LONGITUDINAL								
SW 1	32.77	54.9	717	0.73	1528	616.26	13.4	629.66
SW 2	32.77	54.9	717	0.73	1528	616.26	13.4	629.66
SW 3	32.77	54.9	717	0.73	1528	616.26	13.4	629.66
SW 4	32.77	54.9	717	0.73	1528	616.26	13.4	629.66

STORY 1

UNITS	(FT)	(K)	(K / INCH)	(FT)	(K / INCH)*FT^2	(K)	(K)	(K)
TRANSVERSE	ECCENTRICITY	STORY SHEAR	RIGIDITY	DIST. TO CR	J	TORSIONAL SHEAR	DIRECT SHEAR	TOTAL SHEAR
MF 1	13.68	95.6	94.0	120.8	5457296	2.72	5.26	7.98
MF 2	13.68	95.6	92.4	108.9	5457296	2.41	5.26	7.67
MF 3	13.68	95.6	50.0	78.9	5457296	0.95	3.06	4.01
BF 1	13.68	95.6	481.5	10.1	5457296	1.17	34.22	35.39
BF 2	13.68	95.6	472.5	11.5	5457296	1.30	34.13	35.43
MF 4	13.68	95.6	45.7	81.89	5457296	0.90	3.06	3.96
MF 5	13.68	95.6	81.2	111.9	5457296	2.18	5.26	7.44
MF 6	13.68	95.6	82.5	122.92	5457296	2.43	5.26	7.69
LONGITUDINAL								
SW 1	28.74	38.9	533.8	0.73	1138	382.82	9.49	392.31
SW 2	28.74	38.9	533.8	0.73	1138	382.82	9.49	392.31
SW 3	28.74	38.9	533.8	0.73	1138	382.82	9.49	392.31
SW 4	28.74	38.9	533.8	0.73	1138	382.82	9.49	392.31

Overturning Moments on Foundation:

The overturning moments were found by multiplying the lateral story forces by the story height at which it is applied. These moments were then summed in each direction to obtain total overturning moments of 17,245 ft K in the transverse direction, and 19,771 ft K in the longitudinal direction. The tension and compression forces in the columns created by the overturning moments are resisted by the foundation. The foundation is supported by a pile group at the base of each column typically consisting of three piles per group. Additionally, each frame is secured by at least two uplift piles, one at each end of frame, capable of resisting up to 55 tons or 110 kips each. Because a greater number of axial piles, 110 ton or 220 kips axial capacity, are provided, it can be assumed a check or the uplift piles would be sufficient to determine the foundation's ability to resist the overturning moment.

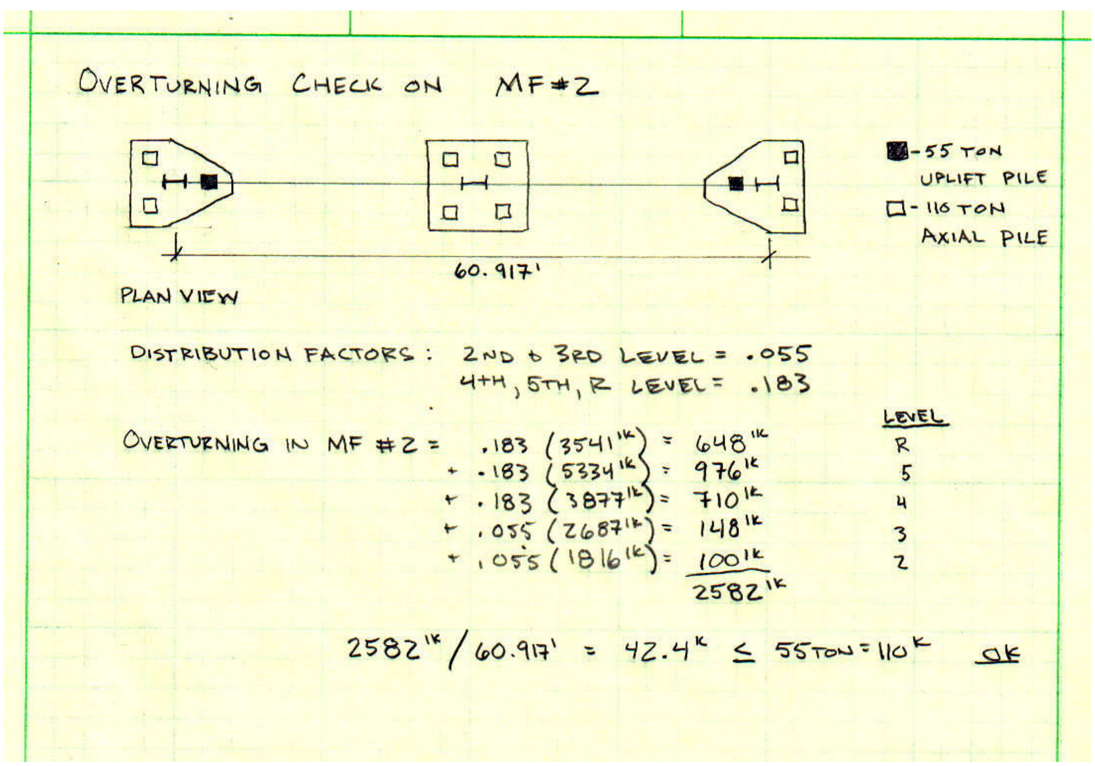
Overturning Moments in Transverse Direction:

Item	**Story Height**	**Story Force**	**Overturning Moments (Mx)**
Roof Level	74.33'	47.5K	3,531 ft K
5th Floor Level	59.00'	90.4K	5,334 ft K
4th Floor Level	45.67'	84.9K	3,877 ft K
3rd Floor Level	32.33'	83.1K	2,687 ft K
2nd Floor Level	19.00'	95.6K	1,816 ft K
Total		401.5K	17,245 ft K

Overturning Moments in Longitudinal Direction:

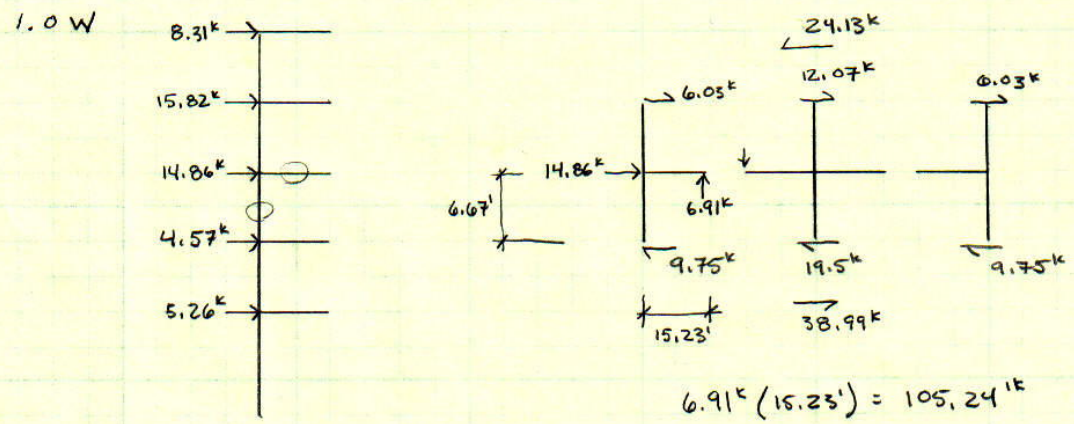
Item	Story Height	Story Force	Overturning Moments (Mx)
Roof Level	74.33’	88.4K	6,571 ft K
5th Floor Level	59.00’	121.2K	7,151 ft K
4th Floor Level	45.67’	77.4K	3,535 ft K
3rd Floor Level	32.33’	54.9K	1,775 ft K
2nd Floor Level	19.00’	38.9K	739 ft K
Total		381K	19,771 ft K

An overturning check on the foundation was performed in the transverse direction on moment frame 2. The overturning moments from the tables above were used along with previously given distribution factors. The overturning moments were generated based solely on direct shear loading from lateral forces. A more accurate check would include the addition of the torsional shear component to the overturning moment, and the gravity loads to the resisting moment. Additionally, it is reasonable to assume the load case 0.9(D) + 1.6(W) would further control increasing the overturning moment. Considering these factors, the check showed that the uplift piles in the foundation system resisted the overturning moment by a factor of safety of approximately 2.5 without the aid of the any gravity loads. The factor of safety, achieved without the aid of the dead load, makes it reasonably safe to assume the foundation would be able to handle the additional overturning moment due to torsion and the wind factor.

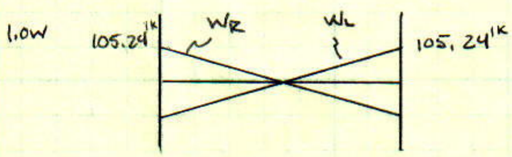


Member Strength Checks:

BEAM + COLUMN CHECK IN MF #5



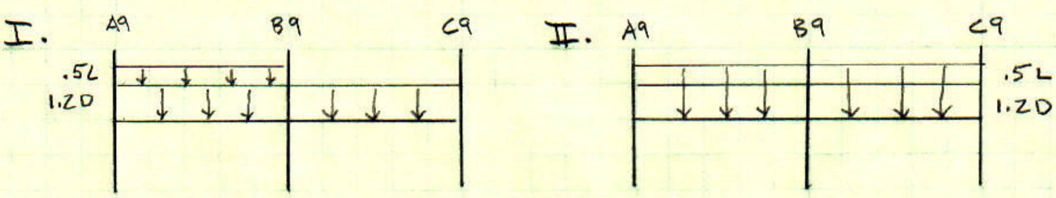
$6.91^k (15.23') = 105.24^{'k}$



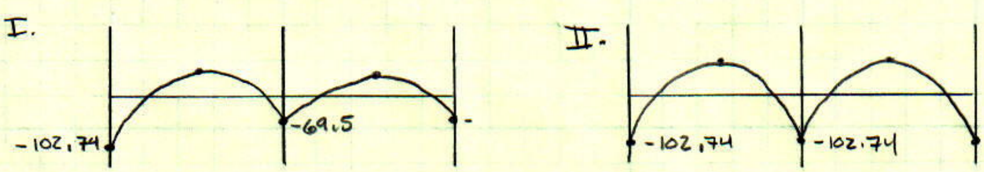
FROM TECH 1: W_{DL} = .750 klf
W_{LL} = .858 klf

1.2D = .9
.5L = .429 > 1.329

* PATTERN LOADING OF CONTROLLING LOAD CASE 1.2D + .5L + 1.6W

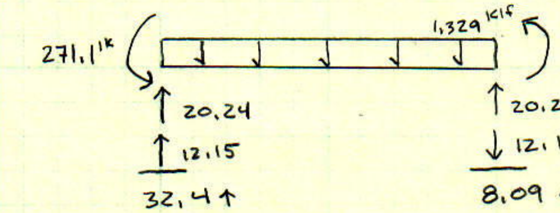


* MOMENT DIAGRAMS GENERATED IN SAP

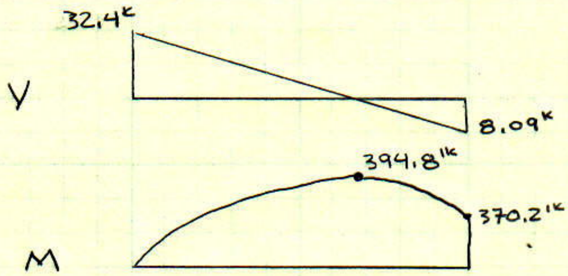


CASE I + 1.0 W_L CONTROLS

$\Rightarrow 102.74 + 105.24(1.6) = 271.124^{'k}$



$105.24(1.6) - 69.5 = 98.9^{1k}$



$M_n = 370.2^{1k} \Rightarrow W21\times48\ ,\ \phi M_n = 398^{1k}$

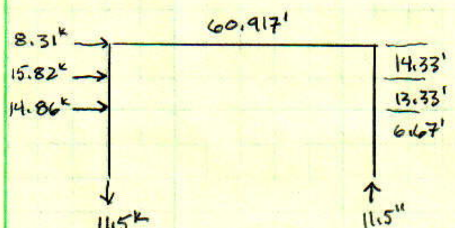
USED $W21\times132\ ,\ \phi M_n = 1250$

COLUMN CHECK @ A9 (EXT. COL.)

$H_T = 13.33'$ $\quad DL = 134$ $\quad$ * LOADS OBTAINED FROM RAM MODEL.

$LL = 49.1$

$P_U = 1.2(134) + .5(49.1) = 185.4$



$P_{U_W} = 8.31(14.33 + 13.33 + 6.67) + 15.82(13.33 + 6.67) + 14.86(6.67)$

$= 700.8^{1k}/60.917' = 11.5^{k}$

$P_{U_{TOT}} = 185.4 + 11.5 = 203.8^{k}$

$Mu_G = 102.94/2 = 51.37^{1k}$

$Mu_W = 9.75(6.67)(1.6) = 104.10^{1k}$

$> Mu_{TOT} = 155.5^{1k}$

$P_{EFF} = P_U + m M_U = 203.8 + 24/14(155.5) = 470.4^{k}$

try $W14\times16$ @ $Kl = 14 \Rightarrow \phi P_n = 572^{k}$

USED $W14\times109 \Rightarrow \phi P_n = 1240^{k}$

Conclusions:

This report outlines a number of both computer models analyses and hand calculation analyses done on the lateral systems of National Harbor Building M. The results of these analyses were used to verify the effectiveness of the existing system, find possible areas of discrepancies between the design and calculated values, and identify possibilities for further research in the ongoing thesis project. A summary of the conclusions formed during this process are listed below.

- The original assumption made in technical report one concerning the distribution of lateral forces in the transverse direction was not accurate. The original assumption was that the six moment frames would each take 1/6 of the force, while the braced frames would be neglected due to their relatively small size compared to the moment frames. The distribution analysis pointed out that the braced frames had fairly large distribution ratio particularly at the bottom levels of the structure, thus making them a vital part of the lateral system.

- The design of the lateral frames was controlled by the drift criteria of the building. This was determined through inspection of the member checks performed on lateral beams and columns and overall story drift. Checks on the beams and columns pointed out large discrepancies between the required strength and the available strength. The capacity of the existing members exceeded the required strength by an approximate factor of two. Additionally, when the same members were checked during a story drift calculation they produced, they were within 7% of the final overall drift. This suggests that the designer increased the size of the members based on their ability to resist drift rather than their strength capacity.

- Torsion is a large contributing factor to the design of the building, particularly in the longitudinal direction. The layout of the lateral system in this direction, as previously noted, leaves the building susceptible to accumulation of large torsional forces. The singular axis of resistance, located at the exterior edge of the building, creates a large eccentricity with a small torsional rigidity factor leading to these problems. The enormous torsional story shears calculated in this direction, however, still raise eyebrows. The method used to calculate this torsion does not take into account any torsional resistance from the out of plane lateral systems which is one possible reason for their inflation.   Nevertheless, the large numbers indicate an area that will probably require further investigation as the thesis process continues.   A possible solution which could be investigated may be to add lateral members, be they shear walls or frames, in the longitudinal direction to decrease the eccentricity of the system.

- As presently designed the foundation system seems capable of controlling the overturning moment through the capacity of implemented uplift piles. However, any significant change to the lateral system affecting its distribution of forces would require this condition to be re-inspected.

SEISMIC CALCULATIONS:

- SOLVE FOR C_S

  - BUILDING HT = 73'-4"
  - Ie => II , I = 1.0

$$C_S = \text{MIN} \begin{cases} S_{DS}/(R/I) \\ S_{D1}/[T\,R/I] \\ S_{D1}\cdot T_L/[T^2 R/I] \end{cases} \geq 0.01$$

- LAT/LONG = -77.008, 38.795

  => $S_S = 0.177g$, $S_1 = 0.063g$

- $F_a = 1.6$, $F_v = 2.4$

- $SMS = F_a S_s = 1.6(.177) = 0.2832g$
  $SM_1 = F_v S_1 = 2.4(.063) = 0.1512g$

- $S_{DS} = 2/3\ SMS = 2/3(.2832) = 0.1888g$
  $S_{D1} = 2/3\ SM1 = 2/3(.1512) = 0.101g$

- TWO OPTIONS FOR LATERAL SYSTEMS, ONE LONGITUDINAL ONE TRANSVERSE

  I. LONGITUDINAL - (E.4) INTERMEDIATE REINFORCED MASONRY SHEAR WALLS

  R = 3.5

  II. TRANSVERSE - (H) STEEL SYSTEM NOT SPECIFICALLY DETAILED FOR SEISMIC

  R = 3.0

- SEISMIC DESIGN CATEGORY B $.067 \leq S_{D1} = .101 \leq .133$

I. $C_t = .02$, $x = .75$ (All OTHER STRUCT SYS)

C_u = .1   .101   .15
      1.7 $\boxed{1.698}$ 1.6

$T_a = .02(73.33)^{.75} = .5012$ sec

$T = 1.698(.5012\text{sec}) = .851$

$*\ T_b = 0.344$ (FROM MODEL)

(FROM MODEL)

$C_s \geq .1888/(3.5/1) = 0.0539$ " $= .0539$ ← CONT.

$.101/.8510(3.5/1) = 0.0339$ ← CONT $.101/.344(3.5/1) = .0839$

$.101(8)/.8510^2(3.5) = 0.3188$ $.101(8)/.344^2(3.5) = 1.951$

$V_b = C_s W_t$ $W_t = 7{,}072^k$ (SEE CALC. ON NEXT PAGE)

$= 0.0339(7{,}072^k)$
$= 239.7^k$

(FROM MODEL)
$= .0539(7{,}072^k)$
$= 381.2^k$

II. $C_t = .028$, $x = .80$ (steel moment resisting frames)
$C_u = 1.698$

$T_a = .028(73.33)^{.80} = .870$ sec

$T = 1.698(.870) = 1.48$ sec

$*T_b = 1.361$ (FROM MODEL)

(FROM MODEL)

$C_s \geq .1888/(3/1) = .0629$ " $= .0629$

$.101/1.48(3/1) = .0227$ ← CONT. $.101/1.361(3/1) = .0247$ ← CONT.

$.101(8)/1.48^2(3/1) = .123$ $.101(8)/1.361^2(3/1) = .145$

$V_b = C_s W_t$

$= .0227(7{,}072^k)$
$= 160.53$

(FROM MODEL)
$= .0247(7{,}072^k)$
$= 174.7^k$

SEISMIC WEIGHT:

- TOTAL DEAD LOAD
- 25% LIVE LOAD FROM STORAGE
- PARTITION LOADS $\geq$ 10 PSF
- PERMINENT EQUIPMENT
- 20% FLAT ROOF SNOW $\geq$ 30 psf

- TYP. FLOOR DL
  - 51 psf FLOOR + DECKING
  - 15 psf STEEL STRUCTURE
  - 10 psf MISC. (FLOORING/DROPCEILING/MEP)
- WALL/PARAPET DL
  - LEVEL 2-5 WALL = 35 psf
  - LEVEL 1 WALL = 25 psf
  - TOWER WALL = 7 psf
  - ELEV EXT. WALL = 30 psf
  - CMU WALL = 40 psf
  - ROOF SCREEN WALL = 15 psf
  - TYP. PARAPT = 260 plf

- ELEV. TOWER $= 210\ ft^2(54 psf) + 2(21.6'+9.6')(6'/2)(30 psf) = 16,326$ lbs
- ARCH TOWER $= 676\ ft^2(25psf) + 2(26'+26')(14.33'/2)(7psf) = 22,116$ lbs

- $W_R = 6,928\ ft^2(35+6\ psf) + 8,200\ ft^2(25+6\ psf) + 2(26'+26')(14.33'/2)(7psf)$
  $+ 2(244'+61')(15'/2)(35psf) + 260\ plf\ (244' + 244'/2 + 61') +$
  $15psf\ (8')(48'+128')2 + 243(15/2)(40psf) = 929,800$ lbs

- $W_5 = 16,175\ ft^2(76\ psf) + 2(244'+61')(15/2 + 13.33/2)(35) + 243(40psf)(15/2 + 13.33/2)$
  $= 1,669,406$ lbs

- $W_4 = W_3 = 16,175\ ft^2(76psf) + 2(244'+61')(13.33)(35psf) + 243'(13.33')(40psf)$
  $= 1,380,217$ lbs

- $W_2 = 16,175\ ft^2(76\ psf) + 2(244+61)(13.33/2)(35psf) + 2(244'+61')$
  $(19'/2)(25psf) + 40\ psf\ (243')(19/2 + 13.33/2)$
  $= 1,673,596$ lbs

$W_{T\ TOTAL} = 7,072^k$

· SEISMIC STORY SHEAR DISTRIBUTION:

- LONGITUDINAL: $T_{USED} \leq .5 \Rightarrow k = 1.0$

$= .344$

$$\cdot C_{VR} = \frac{(930^k + 16^k + 22^k)(74.33')^{1.0}}{(930+16+22)(74.33)^{1.0} + 1669(59)^{1.0} + 1380(45.67)^{1.0} + 1380(32.33)^{1.0} + 1674(19)^{1.0}}$$

$= .232$

$$\cdot C_{V5} = \frac{(1669^k)(59)^{1.0}}{\text{'' '' ''}} = .318$$

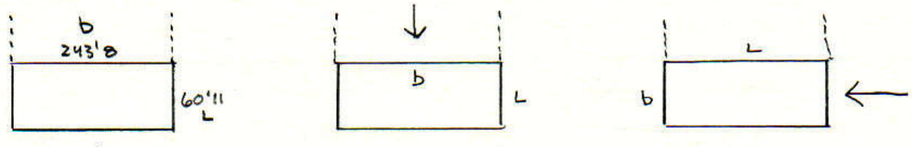
$$\cdot C_{V4} = \frac{(1380^k)(45.67)^{1.0}}{\text{'' '' ''}} = .203$$

$$\cdot C_{V3} = \frac{(1380^k)(32.33)^{1.0}}{\text{'' '' ''}} = .144$$

$$\cdot C_{V2} = \frac{(1674^k)(19)^{1.0}}{\text{'' '' ''}} = .102$$

			Ht (ft)	Overturning (1k)
F_R =	.232 (381.2k)	= 88.4k	74.33	6571'
F_5 =	.318 (381.2k)	= 121.2k	59	7151
F_4 =	.203 (381.2k)	= 77.4k	45.67	3535
F_3 =	.144 (381.2k)	= 54.9k	32.33	1775
F_2 =	.102 (381.2k)	= 38.9k	19	739
				19,771

WIND CALCULATIONS:



TRANSVERSE ①　　TRANSVERSE ②　　LONGITUDINAL ③

$$p = q\,G\,C_p - q_i\,\cancel{(GC_{pi})}^{\,0}$$

$V = 90$ MPH
$K_{zt} = 1.0$
$1/\eta_a = 1/.851 = 1.17 > 1 \Rightarrow$ RIGID (LONG.) , $1/.752 = 1.33 \Rightarrow$ RIGID (Transv.)
EXPOSURE D
IMPORTANCE $= 1.0$

GUST FACTOR (G):

$$G = 0.925\left(\frac{(1+1.7 g_Q I_z Q)}{(1+1.7 g_v I_z)}\right) \qquad I_z = c\left(\frac{33}{z}\right)^{1/6}$$

$$Q = \sqrt{\frac{1}{1+.63\left(\frac{B+h}{L_z}\right)^{.63}}} \qquad L_z = \ell\left(\frac{z}{33}\right)^{\bar{\epsilon}}$$

$z = .6h = 44'$, $\ell = 650$, $\bar{\epsilon} = 1/8.0$, $g_Q = g_v = 3.4$, $c = .15$

$$L_z = 650\left(\frac{.6(73.33')}{33}\right)^{1/8} = 673.8 \qquad I_z = .15\left(\frac{33}{44}\right)^{1/6} = .1430$$

$$Q_1 = \sqrt{\frac{1}{1+.63\left(\frac{243.7+73.33}{673.8}\right)^{.63}}} = \boxed{.848}, \quad Q_3 = \quad 243.7 \rightarrow 60.92 = \boxed{.902}$$

$$G_1 = .925\left(\frac{1+1.7(3.4)(.1430)(.848)}{1+1.7(3.4)(.1430)}\right) = \boxed{.861}$$

$$G_3 = \;.848 \rightarrow .902 \qquad = \boxed{.884}$$

C_p: WALL PRESSURE => WINDWARD = .80

$L/B = .25$ LEEWARD$_1$ = -.50

$L/B = 4$ LEEWARD$_2$ = -.20

$$q_z = 0.00256\, K_z\, K_{zt}^{\nearrow 1}\, K_D^{\nearrow .85}\, V^{2\,\nearrow 90^2}\, I^{\nearrow 1}$$

K_z ← VARIES

$K_D = .85$

	K_z	q_z
0-19'	1.08	19.04
19'-32'4"	1.22	21.50
32'4"-45'8"	1.27	22.38
45'8"-59'	1.31	23.09
59'-74'	1.38	24.32

CASE 1 (E-W) / CASE 2 (W-E)

	CASE 1 WINDWARD (P)	CASE 1 LEEWARD (P)	CASE 2 WW (P)	CASE 2 LW (P)
0-19	13.11	0	0	-10.47
19-32'4	14.81	0	0	-10.47
32'4-45'8	15.42	0	0	-10.47
45'8-59	15.91	0	0	-10.47
59'-74	16.75	0	0	-10.47

CASE 3 (N-S)

	WW (P)	LW (P)	TOTAL (P)
0-19	13.47	-4.30	17.77
19-32'4	15.20	-4.30	19.50
32'4-45'8	15.83	-4.30	20.13
45'8-59	16.33	-4.30	20.63
59-74	17.20	-4.30	21.50

* CASE 4 (E-W) NEGLECTING ADDITIONAL BUILDING

	WW (P)	LW (P)	TOTAL (P) (psf)
0-19	13.11	-10.47	23.58
19-32'4	14.81	-10.47	25.28
32'4-45'8	15.42	-10.47	25.89
45'8-59'	15.91	-10.47	26.38
59'-74'	16.75	-10.47	27.22

· WIND BASE SHEAR:

- TRANSVERSE EW

$16.8\,psf(243.67')(14.33') = 58{,}660\ lbs$
$+15.9\,psf(243.67')(13.33') = +51{,}650$
$+15.4\,psf(243.67')(13.33') = +50{,}020\ lbs$
$+14.8\,psf(243.67')(13.33') = +48{,}070\ lbs$
$+13.1\,psf(243.67')(19') = +60{,}650\ lbs$
$269{,}050 = 269^k$

- TRANSVERSE W-E

$243.67'(73.33')(10.5\,psf)$

$= 181{,}620\ lbs = 182^k$

- LONGITUDINAL N-S/S-N

$21.5\,psf(60.92')(14.33') = 18{,}770\ lbs$
$+20.6\,psf(60.92')(13.33') = 16{,}730\ lbs$
$+20.1\,psf(60.92')(13.33') = 16{,}320\ lbs$
$+19.5\,psf(60.92')(13.33') = 15{,}840\ lbs$
$+17.8\,psf(60.92')(19') = 20{,}600\ lbs$
$88{,}260\ lbs = 88^k$

- TRANSVERSE REVISED (EW)

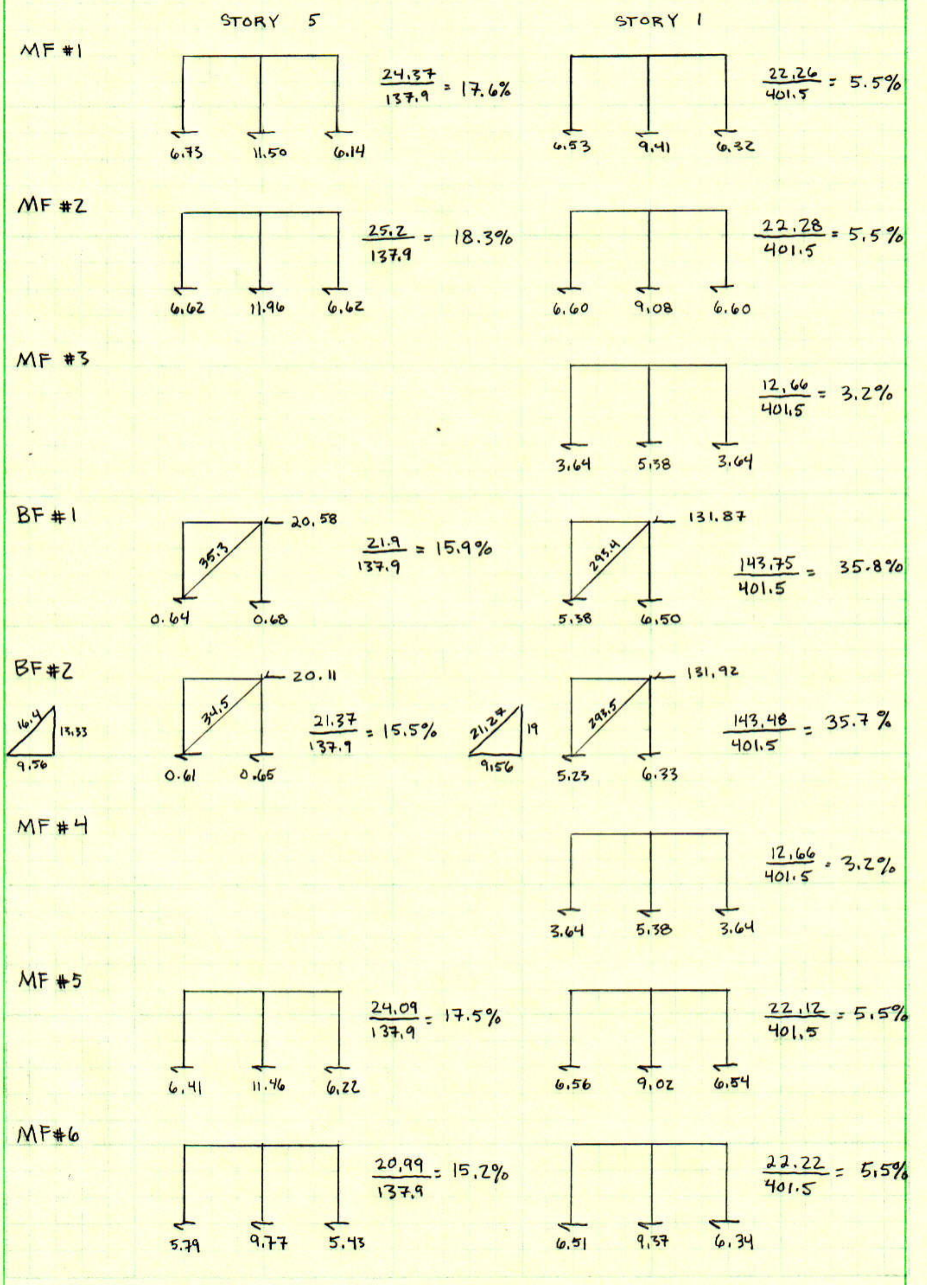
$27.22\,psf\ (243.67')(14.33') = 95{,}050\ lbs$
$+26.38\,psf\ (243.67')(13.33') = +85{,}685\ lbs$
$+25.89\,psf\ (243.67')(13.33') = +84{,}090\ lbs$
$+25.28\,psf\ (243.67')(13.33') = +82{,}110\ lbs$
$+23.58\,psf\ (243.67')(19') = +109{,}169\ lbs$
$456{,}104\ lbs = 456^k$

· CONROLLING TRANSVERSE STORY LOADING:

· ROOF: $27.22\,psf(243.67')(14.33'/2) = 47.5^k$
· 5TH FLR: $27.22\,psf(243.67')(14.33'/2) + 26.38\,psf(243.67')(13.33'/2) = 90.4^k$
· 4TH FLR: $26.38\,psf(243.67')(13.33'/2) + 25.89\,psf(243.67')(13.33'/2) = 84.9^k$
· 3RD FLR: $25.89\,psf(243.67')(13.33'/2) + 25.28\,psf(243.67')(13.37'/2) = 83.1^k$
· 2ND FLR: $25.28\,psf(243.67')(13.33'/2) + 23.58\,psf(243.67')(19'/2) = 95.6^k$

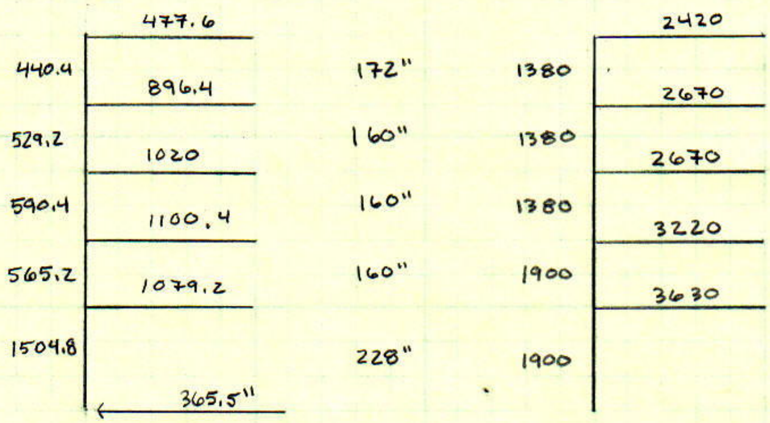
$= 401.5^k$

X-DIRECTION : DISTRIBUTION CALC.



STORY DRIFT APPROXIMATION:

MF #2



MOMENTS FROM
SAP IN 1.0W
("K)

I (in^4)

$$U_K = \frac{M_C L_C^2}{6EI_C} + \frac{L_C L_B}{12E}\left(\frac{M_{BK-1}}{I_{BK-1}} + \frac{M_B}{I_B}\right)$$

$$U_1 = \frac{1504.8(228)^2}{6(29{,}000)(1900)} + \frac{228(365.5)}{12(29000)}\left(\frac{0}{0} + \frac{1079.2}{3630}\right) = .288''$$

$$U_2 = \frac{565.2(160)^2}{6(29{,}000)(1900)} + \frac{160(365.5)}{12(29{,}000)}\left(\frac{1079.2}{3630} + \frac{1100.4}{3220}\right) = .151 + .288 = .439''$$

$$U_3 = \frac{590.4(160)^2}{6(29{,}000)(1380)} + \frac{160(365.5)}{12(29{,}000)}\left(\frac{1100.4}{3220} + \frac{1020}{2670}\right) = .185 + .439 = .624''$$

$$U_4 = \frac{529.2(160)^2}{6(29{,}000)(1380)} + \frac{160(365.5)}{12(29{,}000)}\left(\frac{1020}{2670} + \frac{896.4}{2670}\right) = .177 + .624 = .801''$$

$$U_5 = \frac{477.6(172)^2}{6(29{,}000)(1380)} + \frac{172(365.5)}{12(29{,}000)}\left(\frac{896.4}{2670} + \frac{477.6}{2420}\right) = .155 + .801 = .956''$$