NATIONAL HARBOR BUILDING M
OXON HILL, MARYLAND

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Structural Option
Technical Report 1
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EXECUTIVE SUMMARY

This is an existing conditions report investing National Harbor Building M located in Oxon Hill, Maryland. Building M is being constructed as a part of a large scale development on the banks of the Potmac River which will be know as National Harbor. It's location directly on the banks of the river along with its close proximity to other buildings going up in the development lead to many unique and interesting design cases.

This report starts with an overview of all key structural systems that comprise the building, loads used for designing the systems, and codes followed to uphold industry standards and safety. Next a more detailed analysis is conducted on both wind and seismic forces to analysis their effects on the structure. These results concluded that the building will be controlled and therefore design by wind forces in one direction while seismic forces in the other. These lateral forces were then logically distributed between the building's varying lateral resistance systems including shear walls, braced frames and moment frames. After the controlling loads were distributed analyses were conducted to determine the effectiveness of particular lateral elements. Additionally, spot checks were preformed on typical members like composite beams, composite girders and columns. While the calculation checks agreed closely with the beam and girder sizes used the moment frame columns tested out to be slightly over designed. The differential between checked moment frames/ columns and the sizes actually used was not enormous and could have easily resulted from an oversimplification of the distribution of lateral loads.

STRUCTURAL SYSTEMS OVERVIEW

Floor System:

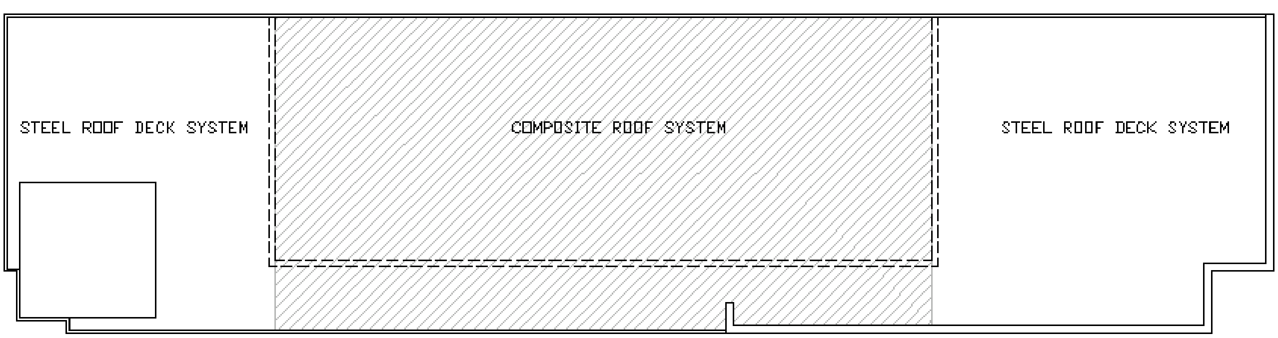
The typical floor is a 6-1/4" thick composite concrete system. It is comprised of a 3-1/4" light weight concrete slab with 3000 psi compressive strength and 3"-20 gauge A992 (50 ksi) composite steel deck. The slab is reinforced with 6x6-10/10 draped welded wire mesh (WWM) and gains it composite properties from ¾" diameter 5-1/4" long steel studs. This composite floor system is supported by A992 wide-flange beams which are typical 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. The purely 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 W14x150's. 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:

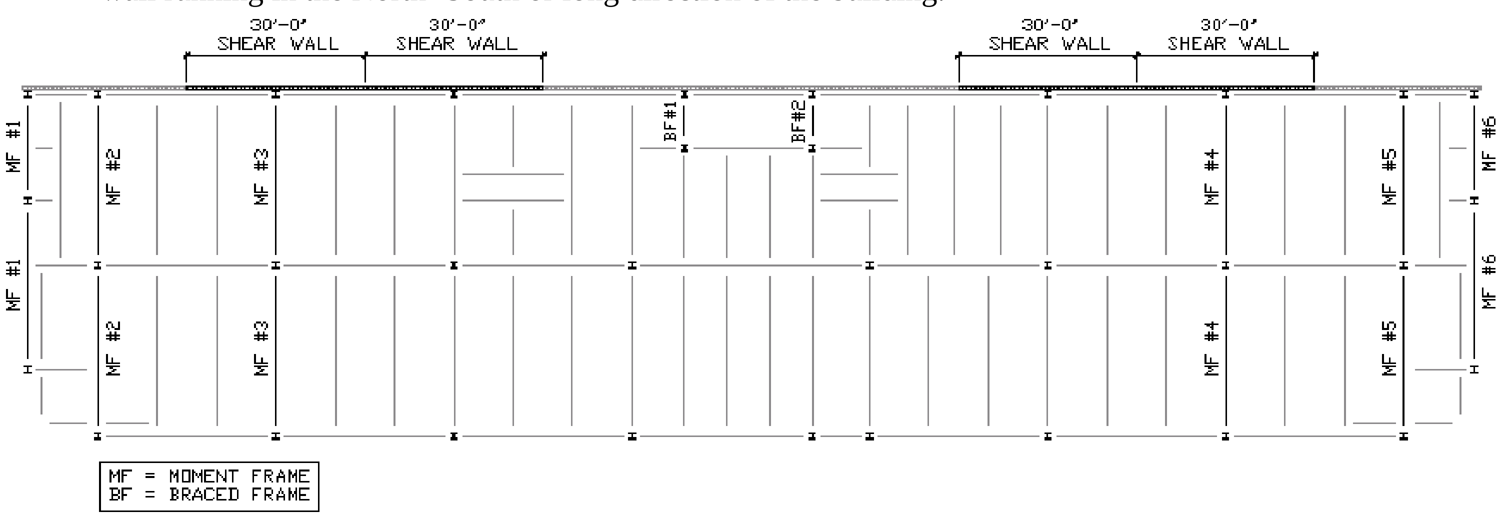
The ground floor is constructed of a 4" thick slab on grade with a compressive strength of 3000 psi and reinforced with 6x6-10/10 WWM. The columns are supported by concrete footings, compressive strength of 4000 psi, which are in turn supported by driven 14" square precast prestressed concrete piles. 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 pile group under the exterior columns. These pile group 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.

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 separats 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 an impact loads from the parking garage of 6000lbs at a height of 1'-6" above the floor levels. In addition to being a barrier sections of the CMU wall also act 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) moment frames which run in the East-West or short direction of the building. 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 braced frames running in the short 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.



TYPICAL FRAMING PLAN

CODES

Design Codes used for Original Design:

- International Building Code, 2003 Edition
- American Society of Civil Engineers (ASCE)
  - ASCE 7 – 02, Minimum Design Loads for Buildings and Other Structures
- American Institute of Steel Construction (AISC)
  - Steel Construction Manual, Thirteenth Edition (LRFD)

Code Substitutions/ Additional References used for Thesis Design:

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

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	25 psf
Composite Roof	35 psf
Non-Composite Roof	25 psf
M/E/P	25 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

SIESMIC 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. That being said 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 axis 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)	7,007K	

***Calculations found in Appendix A**

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)	1.277	12.8.2
Seismic Response Coefficient (Cs)	0.0264	12.8.1.1
Design Base Shear	185K	12.9.4

***Calculations found in Appendix A**

Longitudinal Direction:

Item	**Design Value**	**Code Reference (ASCE 07-05)**
Basic Structural System	Duel 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	237.5K	12.9.4

***Calculations found in Appendix A**

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 the building the only additional load to the total dead load which was applicable was permanent equipment loading. Also worth noting is that 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, preacast, 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	16.0K
Elevator Tower	22.1K
Roof Level	930K
5th Floor Level	1,653K
4th Floor Level	1,364K
3rd Floor Level	1,364K
2nd Floor Level	1,657K
Total	7,007K

***Calculations found in Appendix A**

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, two things became apparent: 1. The seismic base shear numbers I calculated for the longitudinal direction (237.5 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.

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, thus I simply assumed the approximate fundamental period I calculated (0.851 sec transversely and 0.752 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 0.584 sec longitudinally). Plugging the new period Tb back into my calculations I was able to obtain base shear numbers (173K transversely and 350K 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 axis. The building is located on the banks of the Potomac River with no obstructions between itself and the wind coming off the water, thus defining it as an Exposure D building. 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 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, will be standing for the life of the office building caused me to consider 3 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 affects 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.

In determining the rigidity of my building I choose to use the approximate fundamental period of my building in each direction, 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:

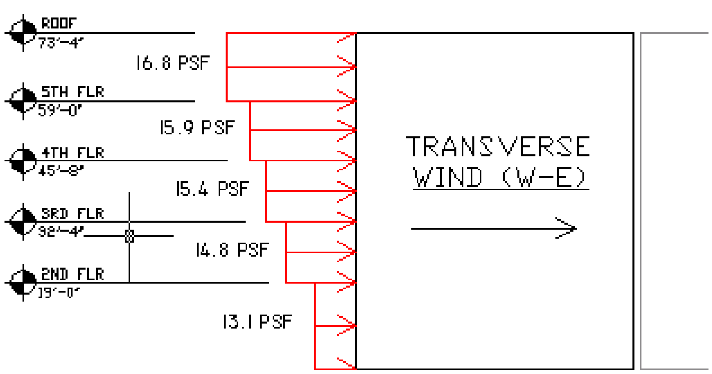
Item	**Transverse Wind**	**Longitudinal Wind**	**Code Reference (ASCE7-05)**
Build Type	Rigid	Rigid	6.2
Exposure	D	D	6.5.6
Importance Factor (I)	1.0	1.0	6.5.5
Basic Wind Speed (V)	90	90	6.5.4
Gust Factor (G)	0.861	0.884	6.5.8
Cp Windward	0.8	0.8	6.5.11
Cp Leeward	-0.5	-0.2	6.5.11
Kzt	1.0	1.0	6.5.7
Kd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

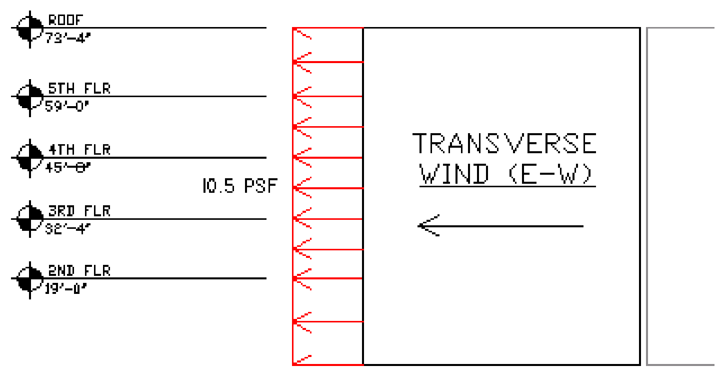
***Calculations found in Appendix B**

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 B**

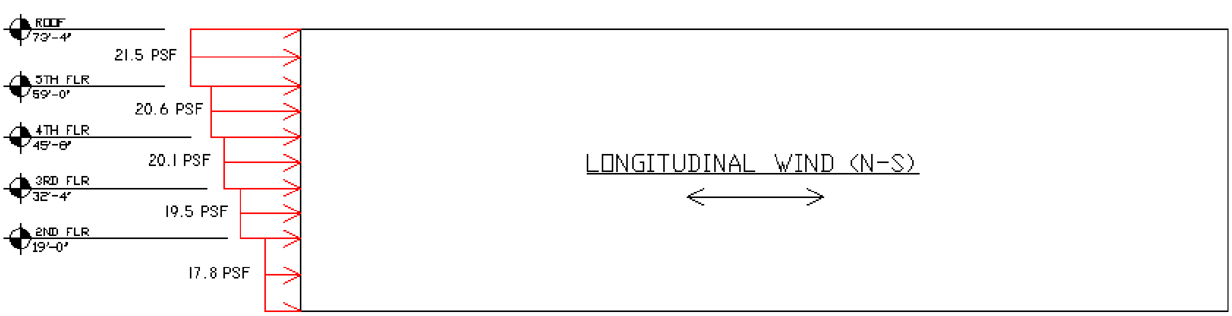




Longitudinal Wind:

			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 B**



Conclusion:

The pressure distributions indicate that in the transverse direction Case 1(windward pressure from the water side) will control over Case 2 (leeward pressure from land side). This is an expected outcome and will cause the building to be designed for the drift limit of a maximum of less than 4 inches, as it will be drifting toward the adjacent garage. At first glance it may seem odd that pressures in the longitudinal direction are greater than the pressures in the transverse direction which takes direct wind from the water. However, after looking into the numbers you can see that the longitudinal pressures are a combination of the windward and leeward forces while the transverse are only taking one set of pressures at a time. Additionally, as expected the overall base shear numbers still add up to be greater in the transverse direction due to the large disparity in surface area of each building face.

Wind Base Shear Summary:

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

***Calculations found in Appendix B**

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 269K. 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)	Transverse (E-W)	Longitudinal (N-S)
Wind	**269K**	**182K**	88K
Seismic	173K	173K	**350K**

***Numbers in Bold Control**

Once the controlling forces and load amounts were determined an assumption as to the distribution of these lateral force had to be made. In the transverse, or wind controlled, direction there are 6 moment frames two full bays wide each and 2 braced frames approximately a third of a bay wide each. I am going to assume that the size differential between the braced and moments frames led the braced frames to have a minimal effect on the overall system in that direction. Additionally, since the braced frames are centrally located around the elevator core I am going to further assume that they are in place to control drift of that specific area and not the entire building. While it is obvious that these braced frames will add some stiffness to the building for the reasons mentioned I believe it is within reason to neglect their effects for these calculations. As for the moment frames I will assume that each frame will share load equally, meaning they will see an effective tributary area (Building Length / 6) rather than each frame’s actual tributary area. I am making this assumption based on their layout in the building. Each end of the building has two moment frames as their last two frames and another splitting the distance toward the center (see lateral element layout on page 5). I feel that if regular tributary area methods were used the two centrally located moment frames would be extremely over designed and the end located moment frames would be significantly under designed. Again while I am certain this assumption is not completely correct I believe it is adequate to obtain accurate numbers for this initial analysis.

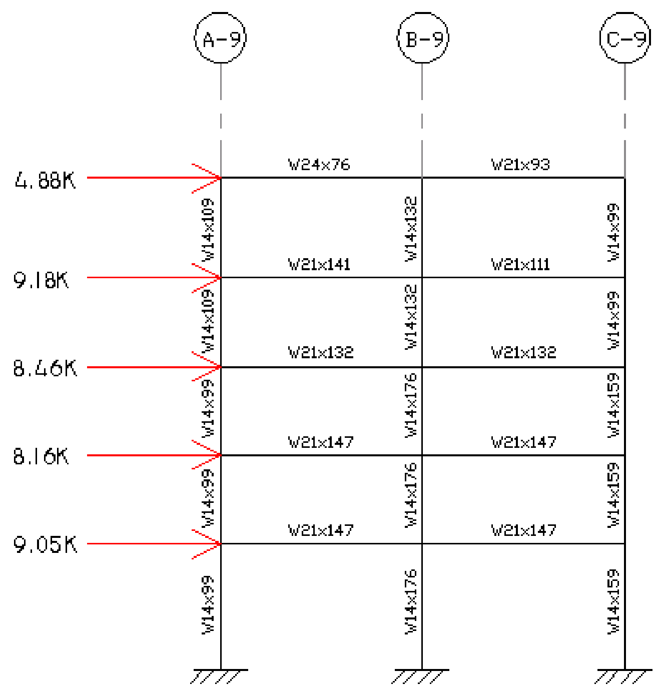
The distribution of shear in the longitudinal direction is much simpler. This resisting system contains four shear walls all falling along the same grid-line (see lateral element layout on page 5). For this layout I will simply assume that each masonry shear wall will take a quarter of the total seismic lateral load. Listed below is the lateral story force for the entire system as well as for each individual shear wall based on my distribution assumptions.

Seismic Story Force Distribution in Longitudinal Direction:

Item	Seismic Weight	Cv Factor	Story Force	Story Force per Shear Wall	Overturning Moments (Mx)
Roof Level	968K	0.318	111K	22.7K	8,273 ft K
5th Floor Level	1653K	0.320	112K	28.0K	6,608 ft K
4th Floor Level	1364K	0.202	70.7K	17.7K	3,229 ft K
3rd Floor Level	1364K	0.141	49.4K	12.4K	1,597 ft K
2nd Floor Level	1657K	0.099	34.7K	8.68K	659 ft K
Total	7,007K	1.0	350K	87.5K	20,366 ft K

***Calculations found in Appendix C**

For further analysis of the lateral resisting system I choose to concentrate on moment frame 5 loaded transverse by East – West wind load. This is a full height moment frame comprised of all W shape and joined together with all moment connections. The base connections are secured by 8 – 1" diameter anchor bolts and a HSS 6x6x1/2 shear lug with a 6" embedment. It is assumed the base of the 3 columns in this moment frame will be modeled as a fixed restraint. This frame with loadings and member sizes diagramed below was modeled in SAP2000. The model was run with only the wind load applied to compute a basic deflection number at the top of the frame. Also found below is a SAP model of the frame deflected shape and a print out of deflections for each joint. The joint numbers are labeled according to column line and the floor level on which they are located.



TRANSVERSE LOADING
ON MOMENT FRAME 5

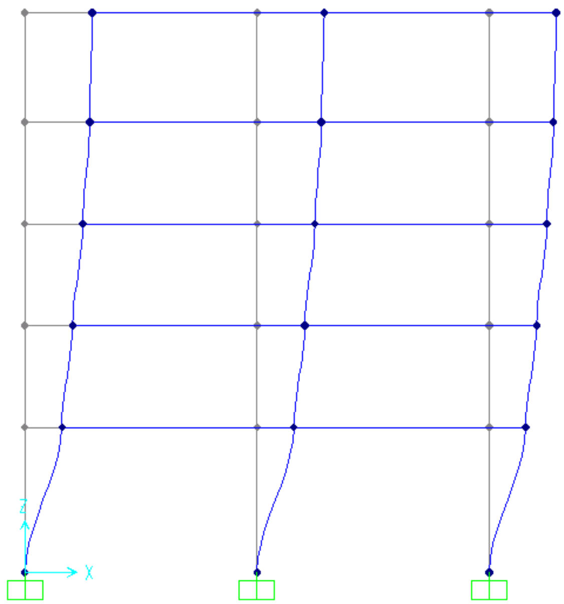


TABLE: Joint Displacements			
Joint	OutputCase	CaseType	U1
Text	Text	Text	in
A9-G	WIND	LinStatic	0
B9-G	WIND	LinStatic	0
C9-G	WIND	LinStatic	0
A9-2	WIND	LinStatic	0.291211
B9-2	WIND	LinStatic	0.289142
C9-2	WIND	LinStatic	0.288167
A9-3	WIND	LinStatic	0.376174
B9-3	WIND	LinStatic	0.373667
C9-3	WIND	LinStatic	0.372282
A9-4	WIND	LinStatic	0.457442
B9-4	WIND	LinStatic	0.455543
C9-4	WIND	LinStatic	0.454733
A9-5	WIND	LinStatic	0.508926
B9-5	WIND	LinStatic	0.50654
C9-5	WIND	LinStatic	0.505546
A9-R	WIND	LinStatic	0.530033
B9-R	WIND	LinStatic	0.528145
C9-R	WIND	LinStatic	0.527488

Conclusions:

The main calculation being investigated in this analysis of moment frame 5 is the frame's overall drift. The drift in the transverse direction strictly from a physical sense must be less than four inches. This is the width of the expansion joint separating it from the adjacent parking garage. Looking at the drift more carefully from an engineering sense you would want the drift to be under the magnitude of H/400 or 2.23 inches in this specific case. The fact that 2.97 is less than four gives you some flexibility in the fact that there is another inch or so past the maximum design drift in the case of an unforeseen loading condition (i.e. an extremely high wind storm or possibly a tornado). With that being said the modeled frame reported a total drift at the roof level of the frame of .527" a significant amount less than the allowable drift of 2.97". This would imply that possibly my distribution of the wind loads may be a little skewed, the wind pressures themselves could be a little skewed, or that the design of the members of the frame were design based on a different controlling case. After reviewing the numbers and speaking with the design engineer I came to the conclusion that the low drift may be a result of differing wind numbers. The difference stems from the defining of the building in the transverse direction as a rigid structure vs. defining it as flexible structure. In defining the rigidity of the structure I chose to use the approximate fundamental period (Ta) to derive my frequency while the design engineer used the actual frequency (Tb) which they generated from a model. (Note: for further explanation of the differing periods see the seismic conclusions on page 10) While the code does not specify which period must be used to calculate the frequency and thus the rigidity of the structure the choice as seen here can change the buildings classification. As a result the design engineer's wind loads were driven up for this loading hence increasing the member size required to control drift. When analyzed using a different classification the members appear to be somewhat oversized to control drift for the given lesser wind loads. I conclude that the design could be considered slightly conservative but further investigation would be needed to confirm that finding.

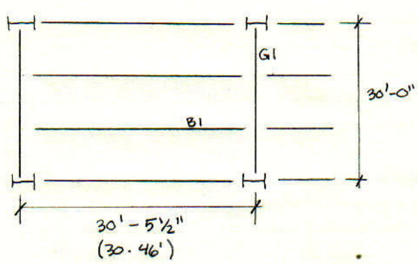
MEMBER SPOT CHECKS

Spot checks were performed on a composite beam and a composite girder in a typical bay. The checks confirmed the sizings of both were adequate and only showing difference in the number of studs required. In both cases there were more studs provided than the minimum required which my checks calculated.

Spot Check Summary:

Item	Span	Member Provided	Member – Spot Check	# Studs Provided	# Studs – Spot Check
Beam (Typ.)	30'-5 ½"	W16x26	W16x26	22	18
Girder (Typ.)	30'-0"	W18x50	W18x50	48	30

SPOT CHECK OF COMPOSITE BEAM (DECK ⊥) & COMPOSITE GIRDER (DECK ∥) IN A TYPICAL BAY.



DL = 75 psf LL = 100 psf

3¼" LWC ON 3"-20 GAUGE 50 ksi DECK

¾" DIA. 5¼" STUDS

$f'_c = 3$ ksi,

• COMPOSITE BEAM B1: T.A. = 10' x 30.46' = 304.6 SF

- LIVE LOAD REDUCTION => $K_{LL} = 2$ (INT. BEAM)

$L = 100\left(.25 + 15/\sqrt{2(304.6)}\right) = 85.8$ PSF

$W_{DL} = 75\text{psf} = 750\text{ plf} = .750\text{ klf}$
$W_{LL} = 85.8\text{psf} = 858\text{ plf} = .858\text{ klf}$
$W_U = 1.2(.750) + 1.6(.858) = 2.27\text{ klf}$

$M_U = \frac{2.27(30.46')^2}{8} = 263.2^{1k}$ => TRY W16x40, $\phi M_n = 274^{1k}$ FROM T3-2 (STEEL ONLY)

* ASSUME $a = 1$, $y_2 = 6\frac{1}{4}" - \frac{1}{2} = 5.75$ → USE $5.5 = y_2$ IN TABLE TO BE CONSERVATIVE.

$B_{eff} = 10' = 120"$
$.25(30.46') = 91.4"$ ← CONTROLS

$Q_n = 17.2$ FROM T3-21

* ASSUME COST OF 1 SHEAR STUD = COST OF 10 lbs OF STL

COMP. MEMBER	PNA LOCATION	ΣQ_n	$\phi M_n \geq 263.2^{1k}$	# STUDS	EQ. WT STUDS	EQ. WT BEAM	TOTAL WT.
W16x26	6	145	279	18	180	792	972
W14x30	6	147	284	18	180	914	1094
W14x22	3	241	266	30	300	670	970

CHECK W16x26:

$a = \frac{\Sigma Q_n}{.85 f'_c B_{eff}} = \frac{145}{.85(3)(91.4)} = .62 < 1$ => ASSUMPTION CONSERVATIVE

• CONSTRUCTION LOAD:

WT OF CONC. = $12(3\frac{1}{4} + \frac{3}{2})/144 \cdot (115\text{psf}) = 45.5\text{psf}$
CONSTRUCTION LIVE LOAD = 20 psf
$W_{U_{CONST.}} = 1.2(45.5) + 1.6(20) = 86.6\text{ psf} = .866\text{ klf}$

$$M_{u\,CONST.} = \frac{.866\,(30.46)^2}{8} = 100.4'^k \le \phi M_n\ W16\times26 = 166'^k \text{ (STL ONLY)}$$

· CONSTRUCTION DL DEFLECTION:

$$L/360 = 30.46(12)/360 = 1.02''$$

$$\Delta = \frac{5}{384}\,\frac{.455\,(30.46)^4\,(1728)}{29{,}000\,(301\,in^4)} = 1.01'' < 1.02'' \quad \underline{OK}$$

* NOTE: DWG'S CALL FOR ALL BEAM'S TO HAVE 1" CAMBER WHICH WOULD DRIVE DEFLECTION EVEN LOWER.

· LL DEFLECTION:

$$I_{LB} = 622\,in^4 \quad (T3\text{-}20)$$

$$\Delta = \frac{5}{384}\,\frac{.858\,(30.46)^4\,(1728)}{29{,}000\,(622)} = .921 \le 1.02'' \quad \underline{OK}$$

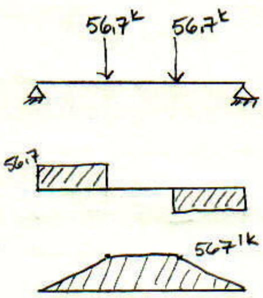
· USE COMPOSITE W16x26 W/ 18 STUDS FOR TYP. BEAM.

· COMPOSITE GIRDER G1: T.A. = 30' x 30.46' = 913.8 SF

- LL REDUCTION: $L = 100\left(.25 + 15/\sqrt{2(913.8)}\right) = 60.1\,psf$

$$W_u = 1.2(.750) + 1.6(.601) = 1.862 \qquad P_t \text{ LOADS ON BEAM} = \left[\frac{1.862\,(30.46')}{2}\right]\cdot 2$$

$$= 56.7^k \text{ @ } 10' \,\&\, 20'$$



BASED ON STL ONLY, $M_u = 567'^k$

TRY:

W24x62 ⇒ 574'k

W21x68 ⇒ 600'k

W18x76 ⇒ 611'k

$$B_{EFF} = 30(12)/4 = 90'' \leftarrow \text{CONTROLS}$$

$$30'(12) = 360''$$

* ASSUME $a = 1 \Rightarrow Y_2 = 5.75$, USE 5.5

$$Q_N = w_r/h_r = 2.1/3 \Rightarrow = 17.1 = Q_n$$

COMP. MEMBER	PNA LOCATION	ΣQ_n	$\phi M_n \geq 567$	# STUDS	EQ. WT STUDS	EQ. WT BEAM	TOTAL WT.
W18x50	6	245	572	30	300	1500	1800
W18x46	4	402	604	28	280	1380	1660 ←
W 18x40	3	430	571	52	520	1720	2240

* CHECK W18x46 $a = 402 / .85(3)(90) = 1.75 > 1$ NOT OK

* CHECK W 18x50 $a = 245 / .85(3)(90) = 1.07$

$y_2 = 6.25 - 1.07/2 = 5.715 \Rightarrow$ OK B/C ROUNDED DOWN TO 5.5 FOR Y_2 WHEN SELECTING FROM CHARTS

· CONSTRUCTION DEAD LOAD Δ

$P = .750 \text{ klf } (30.46') = 22.8^k$

$$\Delta = \frac{Pl^3}{28EI} = \frac{22.8(30)^3(144)}{28(29{,}000)(800)} = .136'' \leq \frac{30(12)}{360} = 1 \quad \text{OK}$$

· LINE LOAD Δ

$P = .601(30.46') = 18.3^k$

$$\Delta = \frac{18.3(30)^3(144)}{28(29{,}000)(1570)} = .06 \leq 1 \quad \text{OK}$$

* USE COMPOSITE W18x50 @ PNA 6 W/ 30 STUDS

COLUMN LOAD ACCUMULATION

COLUMN LOAD ACCUMULATION (B.9)

LEVEL	A_T	A_I	LL REDUCTION $(.25 + 15/\sqrt{A_I})$
ROOF	914 SF	3656	.498
5th	1828 SF	7312	.425
4th	2742 SF	10,968	.40
3rd	3656 SF	14624	.40
2nd	4570 SF	18,280	.40

UNFACTORED LOADS:

ROOF: $DL = 25psf + 25psf + 15psf + 7psf = 82psf\ (914) = 74.9^k$

$LL:\ 30psf + 30psf = 60psf\ (914) = 54.8^k$

5^{TH}: $DL = 75psf\ (914) = 68.6^k + 1.0^k SW = 69.6^k$

$LL = 100psf\ (.425)(914) = 38.8^k$

4^{TH}: $DL = 69.6^k$ (SEE ABOVE)

$LL = 100psf\ (.425)(914) = 36.6^k$

3^{RD}: $DL = 69.6^k$

$LL = 36.6^k$

2nd: $DL = 69.6^k$

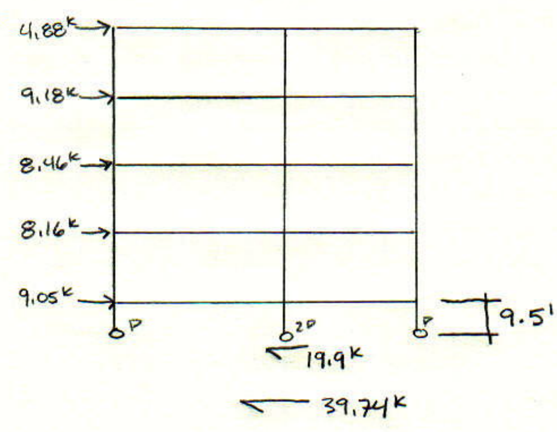
$LL = 36.6^k$

TOTAL ⇒ $DL = 353.3^k$ $LL = 148.6^k$ $RL = 54.8^k$

$1.2DL + 1.6LL + .5RL \Rightarrow 1.2(353.3) + 1.6(148.6) + 0.5(54.8)$

FACTORED GRAVITY LOAD $= 689.1^k$

MOMENT:



$39.74/4 = 9.935(2)$

$= 19.9^k$

$M = 19.9^k(9.5') = 189.1^{'k}$

COLUMN B.9 AT BASE = W14x120 , assume $KL = 19'$ (greatest unbraced length)

$\phi M_n = 750$, $M_n = 833$

$\phi P_n = 1210$, $P_n = 1344$

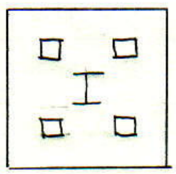
$$\frac{P_u}{P_n} = \frac{689.1^k}{1344^k} = .513 \Rightarrow \frac{P_u}{P_n} + \frac{8}{9}\left(\frac{M_u}{M_n}\right) \le 1.0$$

$$\frac{689.1^k}{1344^k} + \frac{8}{9}\left(\frac{189.1'^k}{833'^k}\right) = .715 \le 1.0$$

OK

W14 x 120 COLUMN OK FOR BASE OF B-9

COLUMN LOAD SUPPORTED BY PILES:



110 ton PILES

689.1^k PURE AXIAL

$110(2000\text{ lbs}) = 220{,}000\text{ lbs} = 220^k$

$4\text{ PILES}(220^k) = 880^k \ge 689.1^k$ OK

Conclusion:

My calculations show that the W14x120, column B9, located at the base of moment frame 5 is adequate to carry the combined loading under which it is subjected. The fact that the column is loaded around 30% below its full capacity (0.71<1.0) suggests that a smaller column may have been able to support the loading. It must be kept in mind however, that many assumptions have been made along the way to come up with these final numbers. For example, an assumption concerning the distributions of lateral wind forces in this direction was made earlier in this report (see Lateral System Analysis Section) and could ultimately effect the moment caused by story shear in this frame. If this frame would have been designed to take more lateral forces than assumed the story shear would increase thus increasing the moment at the base of this column. An increase in the moment at the base would drive the design closer to the W14x120's ultimate combined capacity. With that being said I feel safe in concluding that while the W14x120 is if anything on the conservative side making it definitely capable of supporting its loading.

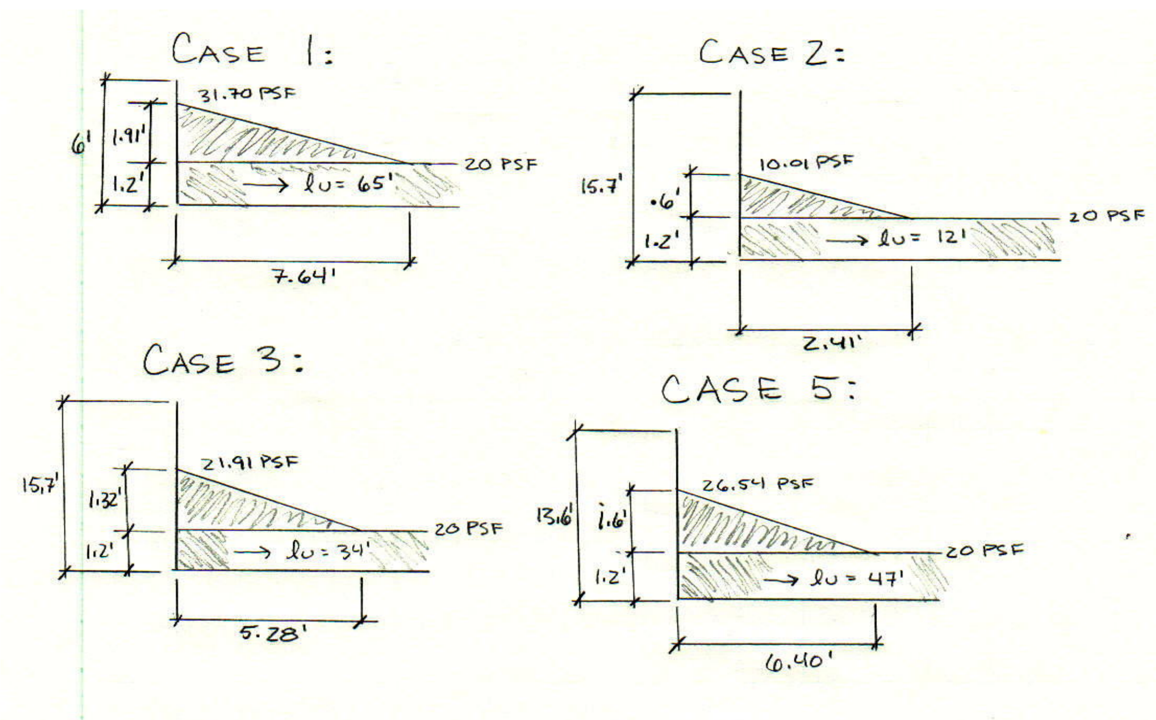
The foundation support for column B9 is a four pile footing supported by piles capable of supporting 110 tons each. The calculations show that this is an adequate foundation to support the 690K factored axial load applied by the column. It must be noted that the driven prestressed precast piles are assumed to be located in the symmetrical placement around the column as dictated by the plans. Their exact location must be verified after they are driven and their capacity must be recalculated. It is recognized that if the piles are driven even slightly out of alignment the load distribution between the piles would not be even. In the case that this situation would occur causing a pile to be loaded beyond its capacity further measures would have to be taken to correctly support the column.

SNOW DRIFT

Shown below are calculations snow drift loads as described in ASCE 07-05. The flat roof snow loads used were originally introduced in the load section (page 7). The diagrams model the snowdrift layout for case 1,2,3, and 5 with case 4 not applying because the drift height is lower than the screenwall's elevation off the roof at that particular location. In this case it is reasonable to assume the balanced snow load will proceed under the opening unobstructed and no drift will be formed.

Snow Drift Calculations:

(All numbers in feet)	Case 1 Precast Parapet (from North & West)	Case 2 Tower(from South)	Case 3 Tower (from east)	Case 4 ScreenWall (3'-6" off ground)	Case 5 Screenwall (down to ground)
Height of Structure (hc)	6	15.67	15.67	13.58	13.58
Balanced Snow Load (pg)=	20	20	20	20	20
Length of run (lu)=	65	12	34	65	47
Drift height (hd) =	2.55	0.80	1.76	2.55	2.13
Adjusted Drift Height(hd')	1.91	0.60	1.32	1.91	1.60
gamma=	16.60	16.60	16.60	16.60	16.60
Max intensity (pd)=	31.70	10.01	21.91	31.70	26.54
Balanced snow height (hb)	1.20	1.20	1.20	1.20	1.20
hc/hb >0.2	4.98	13.01	13.01	11.27	11.27
Drift width (w1)= if hd'<hc	7.64	2.41	5.28	7.64	6.40
Drift width (w2)=if hd'>hc	2.43	0.09	0.44	1.07	0.75



ADDITIONAL TOPICS

Structural elements not previously mentioned in this report which will require further investigation include but are not limited to:

- Canopies – At second floor framing level cantilever wide flanges shapes extend from the building as much as 10 feet.
- Corner Conditions – The exterior columns are geometrically recessed and are supported by cantilever members secured by moment connections.
- Roof – The effect of roof uplift and other forces on the roof created by architectural tower and structural screen walls.
- Foundation – The tolerances of the pile configurations should they be not driven exactly to plan specified locations.

- **APPENDIX A**

SEISMIC CALCULATIONS

SEISMIC CALCULATIONS:

- SOLVE FOR C_s

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

$$C_s = \text{MIN}\begin{cases} SDS/(R/I) \\ SD1/[T\,R/I] \\ SD1 \cdot T_L/[T^2 R/I] \end{cases} \geq 0.01$$

• 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 ≤ SD1 = .101 ≤ .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) $T = 1.698(.344) = .584$

(FROM MODEL)

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

$.101/.8510(3.5/1) = 0.0339 \leftarrow$ CONT. $.101/.584(3.5/1) = .050 \leftarrow$ CONT

$.101(8)/.8510^2(3.5) = 0.3188$ $.101(8)/.584^2(3.5) = .677$

$V_b = C_s W_t$ $W_t = 7{,}007^k$ (SEE CALC. ON NEXT PAGE)

$= 0.0339(7{,}007^k)$
237.5^k

(FROM MODEL)
$= .050(7{,}007^k)$
$= 350.4^k$

II. $C_t = .03$, $x = .75$ (Ecc. Braced Stl Frames)
$C_u = 1.698$

$T_a = .03(73.33)^{.75} = .752$ sec

$T = 1.698(.752) = 1.277$

$^*T_b = 1.361$ (FROM MODEL)

(FROM MODEL)

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

$.101/1.277(3/1) = .0264 \leftarrow$ CONT. $.101/1.361(3/1) = .0247 \leftarrow$ CONT.

$.101(8)/1.277^2(3/1) = .165$ $.101(8)/1.361^2(3/1) = .145$

$V_b = C_s W_t$

$= .0264(7{,}007^k)$
$= 185^k$

(FROM MODEL)
$= .0247(7{,}007^k)$
$= 173.1^k$

SEISMIC WEIGHT:

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

• TYP. FLOOR DL
  - 25 psf FLOOR + DECKING
  - 25 psf MEP
  - 15 psf STEEL STRUCTURE
  - 10 psf MISC. (FLOORING/DROP CEILING/ETC.)

• 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 = $210\,ft^2(50psf) + 2(21.6'+9.6')(6/2)(30psf) = 16{,}008$ lbs

• ARCH TOWER = $676\,ft^2(25psf) + 2(26'+26')(14.33/2)(7psf) = 22{,}116$ lbs

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

• $W_5 = 16{,}175\,ft^2(75psf) + 2(244'+61')(15/2 + 13.33/2)(35) + 243(40psf)(15/2 + 13.33/2)$

$= 1{,}653{,}231$ lbs

• $W_4 = W_3 = 16{,}175\,ft^2(75psf) + 2(244'+61')(13.33)(35psf) + 243'(13.33')(40psf)$

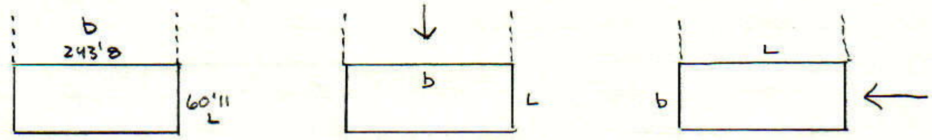
$= 1{,}364{,}042$ lbs

• $W_2 = 16{,}175\,ft^2(75psf) + 2(244+61)(13.33/2)(35psf) + 2(244'+61')$
$(19/2)(25psf) + 40psf(243')(19/2 + 13.33/2)$

$= 1{,}657{,}421$ lbs

$W_{T\,TOTAL} = 7{,}007^k$

WIND CALCULATIONS:



TRANSVERSE ① TRANSVERSE ② LONGITUDINAL ③

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

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

GUST FACTOR (G):

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

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

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

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

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

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

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

CP: WALL PRESSURE => WINDWARD = .80

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

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

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

VARIES

$K_D = .85$

	K_z	q:
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)

	WINDWARD (P)	LEEWARD (P)	WW (P)	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

· WIND BASE SHEAR

- TRANSVERSE E-W

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

- TRANSVERSE W-E

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

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

- LONGITUDINAL N-S/S-N

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

APPENDIX C
LATERAL SYSTEM CALCULATIONS

LATERAL RESISTING SYSTEM:

- WIND STORY SHEAR: (M.F. 5 - TRANSVERSE E-W WIND)
  - *ASSUME EACH MOMENT FRAME TAKES 1/6 LATERAL LOAD

- ROOF LEVEL = $16.8\,psf\,(243.67/6)(14.33/2) = 4.88^k$
- 5TH FLR $= 16.8\,psf\,(243.67/6)(14.33/2) + 15.9\,psf\,(243.67/6)(13.33/2)$
  $= 9.18^k$
- 4TH FLR $= 15.9\,psf\,(243.67/6)(13.33/2) + 15.4\,psf\,(243.67/6)(13.33/2)$
  $= 8.46^k$
- 3RD FLR $= 15.4\,psf\,(243.67/6)(13.33/2) + 14.8\,psf\,(243.67/6)(13.33/2)$
  $= 8.16^k$
- 2ND FLR $= 14.8\,psf\,(243.67/6)(13.33/2) + 13.1\,psf\,(243.67/6)(19/2)$
  $= 9.05^k$

- SEISMIC STORY SHEAR DISTRIBUTION:

- LONGITUDINAL: $K = 1.042$
- TRANSVERSE: $K = 1.320$

$T =$	.5		2.5
$T_{USED\,L}$		.584	
$K =$	1	1.042	2
$T_{USED\,T}$		1.361	
$K =$	1	1.320	2

- LONG. DISTRIBUTION

$$C_{VR} = \frac{(930^k + 16^k + 22.1^k)(74.33')^{1.042}}{(930+16+22.1)(74.33)^{1.042} + 1{,}653^k(59)^{1.042} + 1{,}364(45.67')^{1.042} + 1{,}364(32.33')^{1.042} + 1{,}657(19')^{1.042}}$$

$= .318$

$$C_{V5} = \frac{(1653^k)(59)^{1.042}}{''\quad ''} = .320$$

$$C_{V4} = \frac{(1364^k)(45.67)^{1.042}}{''\quad ''} = .202$$

$$C_{V3} = \frac{(1364^k)(32.33)^{1.042}}{''\quad ''} = .141$$

$$C_{V2} = \frac{(1657)(19)^{1.042}}{''\quad ''} = 0.099$$

		H_t (ft)	OVERTURNING M_x (1^k)
$F_R = .318\,(350^k)$	$= 111.3^k$	74'-4"	$8{,}273^{1k}$
$F_5 = .320\,(350^k)$	$= 112^k$	59'-0"	$6{,}608^{1k}$
$F_4 = .202\,(350^k)$	$= 70.7^k$	45'-8"	$3{,}229^{1k}$
$F_3 = .141\,(350^k)$	$= 49.4^k$	32'-4"	$1{,}597^{1k}$
$F_2 = .099\,(350^k)$	$= 34.7^k$	19'-0"	659^{1k}
			$20{,}366^{1k}$