CHAPTER 2: CONCRETE POST-TENSIONED REDESIGN

2.1 INTRODUCTION

The basis for the structural depth of this report was a switch from composite steel beam and concrete floor system to a concrete post-tensioned system. The post-tensioned system was designed to span the same lengths as the composite system while maintaining a relatively thin overall structural depth. In addition to the change of floor system type in National Harbor Building M, many other structural elements were redesigned or modified to maintain the completeness of the overall design. During this entire process great efforts were made to uphold the building's original function and architectural characteristics.

With the design of the new concrete based floor system, the rest of the building's structural elements, previously designed in steel, had to be designed as concrete elements. The first elements reexamined were the gravity carrying columns and lateral frames. The original design, as noted in chapter 1, supported the gravity loads with W-shape steel columns and resisted the lateral loads with a combination of steel moment and braced frames as well as masonry shear walls. To stay consistent with the concrete floor system the redesign relied on reinforced concrete columns to carry the gravity loads. A number of options were considered when determining the most effective way to handle the redesign of the lateral force resisting system. Among the options considered were concrete moment frames, blade columns or strategically placed oversized concrete columns which collect the lateral forces, a utilization of the existing masonry shear walls, reinforced concrete shear walls, as well as a combination of these systems. Ultimately, a design that relied on concrete shear walls in both directions was settled on as the most efficient way to resist the lateral forces without effecting the building's function or architectural layout.

After the basic layout of the concrete design of National Harbor Building M was completed, an analysis of the lateral loads created by wind and seismic forces was conducted. Since the general building dimensions did not change, the wind forces calculated for the original steel design remained the same. However, the switch to a concrete building and thus an increase in seismic weight of the building caused the seismic loads to increase. This change in some of the lateral forces accompanied by the obvious increase in gravity loads required that a check of the foundation system be conducted. While the general foundation system of precast prestressed concrete piles was not redesigned, an analysis of the new loads was performed and additional piles were added as needed.

Before the redesign began a few basic assumptions and material choices were made and carried through to the design of individual structural elements. It was decided that a switch from the lightweight concrete used in the composite floor system to normal weight concrete would be made. This decision was based on the fact that a majority of concrete framed buildings use normal weight concrete as their main building material. Additionally, a switch from 3000 psi compressive strength concrete to 5000 psi strength concrete was made. Conversations with people in the building profession from the greater Washington D.C. area, National Harbor is located in Oxon Hill, Maryland, assured me that using 5000 psi concrete was not outside the realm of common practices. While individual members or structural elements could have been designed more efficiently with differing concrete properties, the decision was made to maintain uniform properties throughout the project. This was done to simplify the building process by not requiring different specifications of concrete throughout construction.

The following is an explanation and presentation of the design steps taken and results determined from the post-tensioned concrete design of National Harbor Building M. Since the design process is an iterative one, the information is not necessary presented in the order the design was completed. Rather the material is presented to logically step through the compilation of analyses, calculations, and designs completed. Some of the information is very similar to material presented in chapter 1 of this report and previously submitted technical reports. However, for the completeness of this report, all material will be fully covered again.

2.2 STRUCTURAL SYSTEM OVERVIEW

2.2.1 Floor System

The typical floor is an 8" thick two-way post-tensioned concrete system. It is comprised of grade 250 0.6" dia. unbounded post-tensioned tendons laid in a slab of 5000 psi concrete. The typical tendon profile starts the tendons of the mid height of the slab dropping them to a slab depth of 1.25" at mid span and raising them to a depth of 6.75" over support lines. This tendon profile creates a typical drape of 4.125" in an exterior span and 6.75" in an interior span. The slab spans are typically 30'-5-1/2" in the transverse direction and 30'-0" in the longitudinal direction of the building. At critical locations, like the 40'-0" span at the center of the building, beams have been incorporated to help span the increased length. To simplify construction and avoid a "weave" of tendons causing poor concrete cohesiveness, the tendons are banded across the lines of support in the longitudinal direction and uniformly distributed across the transverse direction.

2.2.2 Column System

The columns are reinforced 5000 psi concrete columns, 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. These columns are the main gravity resisting members of the structure, and range in size from 20"x20" to 24”x24”. The 20”x20” columns are located around the building’s perimeter and are typically reinforced with (8) #8 bars. The 24”x24” columns are located on the building interior and are typically reinforced with (12) #8 bars. To combat punching shear issues, the columns are aided by 7.5’x7.5’ 4” thick shear caps at required locations.

2.2.3 Roof System

While not specifically designed in this report, the roof system will be comprised of an 8” post-tensioned slab similar to the typical floor system. Since the roof loading will not be as great as that of the typical floor, a reduction in the number of required tendons or their spacing is expected. It is logical to assume that the roof will require more support from tendons in the center area which is to carry the mechanical units located on the roof. Although the roof itself could be supported by a mildly reinforced slab, the fact that the post-tensioned construction would already be mobilized makes sense to stick with the post-tensioned support.

2.2.4 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 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 shear walls located at the corners of the building are supported by a nine-pile pile groups with five uplift capacity piles. The columns and shear walls which comprise 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.

2.2.5 Lateral System

The building’s lateral force resisting system is comprised of ordinary reinforced concrete shear walls in both directions. These shear walls are arranged into four separate L-shaped walls, two flanking the elevator core and two at the edges of the structure, see figure 2-1. The interior shear walls surrounding the elevator core are 16” thick and run 12’-0” in the transverse direction and 10’-0” in the longitudinal direction. The shear walls located at the structure’s corners are 2’-0” thick running 8’-0” in the transverse direction and 10’-0” in the longitudinal direction. Where boundary elements are required in the walls, the reinforcing has been kept inside the original thickness of the wall as not to protrude further into the building’s usable space. The L-shape design adds flanges to each of the four walls in both directions and allows the reinforcing of the

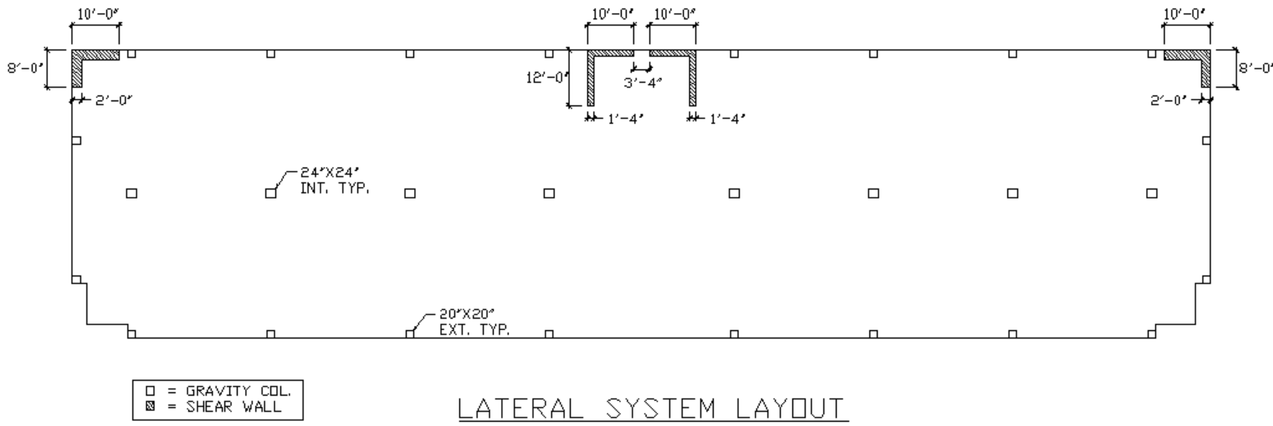

Fig. 2-1

walls to be intertwined. The effect of the flanges and the intertwining of the reinforcement adds a considerable amount of the stiffness to the lateral systems.

2.3 SEISMIC ANAYLYSIS

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 force to control lateral design along that path. Since the lateral resisting systems are the same in each direction, ordinary reinforced concrete shear walls, one calculation can be performed and used for both. For greater accuracy a dynamic model of the structure was created in ETABS to obtain the modal periods used in the calculations. This model, which will be further discussed in section 2.7.4, was analyzed considering cracked section properties and p-delta effects specified in the computer program. The resulting periods can be seen in figure 2-2, with the smallest period of 0.6678 seconds being using in the analysis to be conservative.

ETABS Building Periods:

Mode	Period T (seconds)
1	1.7340
2	1.0149
3	0.6678

Fig. 2-2

The next step in the analysis was to calculate the building's seismic weight. The seismic weight of the building is calculated by adding the building's 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 15 psf was applied from the ground up to the 2nd level (mostly store front glass with brick and precast accents). Once the seismic weight was obtained the Equivalent Lateral Force Procedure was followed to obtain the seismic base shear (See figures 2-3 and 2-4). More detailed calculations for this entire section can be found in the appendix.

Seismic Weight Summary:

Item	**Concrete Post-Tensioned**
Elevator Tower	26.6K
Roof Level	1,923K
5th Floor Level	1,977K
4th Floor Level	1,948K
3rd Floor Level	1,948K
2nd Floor Level	1,940K
Total	**9,762K**

Fig. 2-3

General Seismic Analysis:

Item	**Design Value**	**Code Reference (ASCE 07-05)**
Seismic Use Group	Group I	Table 1-1
Site Class	D	
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
(Fa)	1.6	
(Fv)	2.4	
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
Basic Structural System	Bearing Wall Systems	Table 12.2-1
Seismic Resisting System	Ordinary Reinforced Concrete Shear Walls	Table 12.2-1
Response Modification Factor R	4.0	12.2.3.1

Deflection Amplification Factor (Cd)	4.0	12.2.3.1
Approximate Period (Ta)	0.506 sec	
Calculated Fundamental Period (T)	0.859 sec	12.8.2
ETABS Period (used)	0.6678 sec	
Seismic Response Coefficient (Cs)	0.0378	12.8.1.1
Seismic Weight (W)	9,762 K	
Design Base Shear	369.0K	12.9.4

Fig 2-4

2.4 WIND ANALYSIS

As mentioned before, in redesigning National Harbor Building M as a concrete supported structure it was a top priority not to change the architectural layout of the building. This being the case, the exterior dimensions remain the same as the existing steel composite building analyzed in the technical reports. Considering that wind loads on a building are defined mainly by exterior building dimensions and factors associated with its location, it follows suit that the previous wind analysis remains valid. The following is a slightly modified wind analysis taken from previous technical reports.

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. Additionally, a bend in the river at location of Building M making it just over a mile wide and the buildings 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 an expansion joint of four to eight inches. Since the large surface area taking wind directly from the water will control in this direction (see section 2.7 for comparison vs. seismic) the lateral system must be capable of resisting these forces to within a reasonable amount.

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 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 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 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 chose to use the fundamental period of vibration obtained in an ETABS model for each direction. Taking the inverse of this number gave me the fundamental frequency of the building in each direction. With both frequencies being greater than a value of 1.0, the building could be assumed to be a rigid structure in both directions. The corresponding factors and equations used to compute the wind values are shown in figures 2-5 through 2-9.

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

Fig. 2-5

Transverse Wind (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

Fig. 2-6

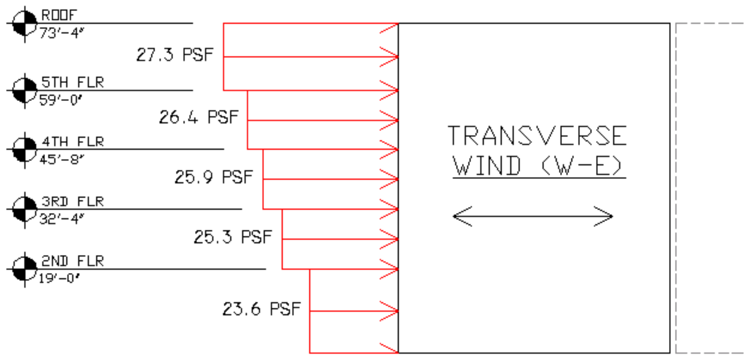

Fig. 2-7

Longitudinal Wind (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

Fig. 2-8

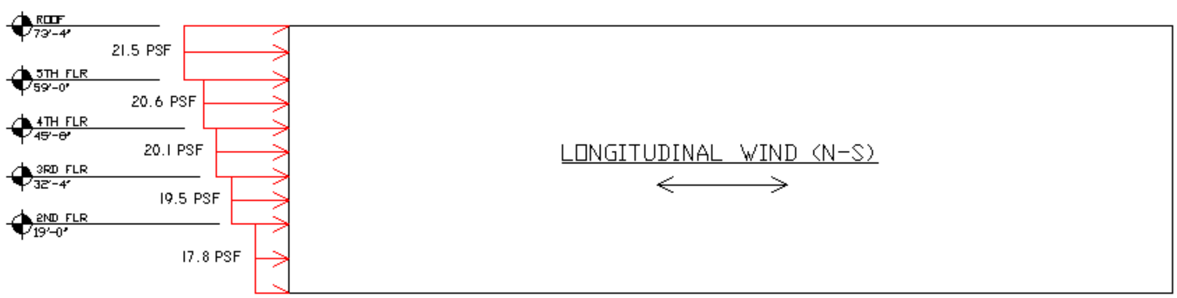

Fig. 2-9

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 that may cause the building to be designed on the drift criteria of the architectural precast panel wall perpendicular to the transverse wind. Additionally, the small surface area of the longitudinal direction limits the lateral wind forces along this path. A summary of the wind base shear service loads is shown in figure 2-10.

Wind Base Shear Summary:

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

Fig. 2-10

2.5 POST-TENSIONED FLOOR SYSTEM

A post-tensioned designed system was selected as the concrete flooring system for the redesign of National Harbor Building M. The post-tensioned system gave the design the ability to span the predetermined lengths without excessive use of beams or an extremely thick slab. This section will cover the procedure used, the assumptions made, and a summary of the final calculations and design of the floor system.

2.5.1 Design Procedure

The design of the post-tensioned floor system relied both on hand calculations and the use of a post-tensioning computer program RAM Concept 2.0. This iterative process began with a general hand calculation being preformed based on research, prior course material and information obtained during post-tension lectures. These calculations were used as a starting point for the creation of the RAM Concept model. Once analyzed the information generated by the model was used to finalize the design of the post-tensioning tendons. Finally, hand calculations were redone with the new design to verify their correctness. The other members who make up the floor system, the shear caps/drop panels and beams were designed by hand and added to the model for verification.

2.5.2 Predesign Assumptions and Decisions

At the beginning of the design process some basic assumptions and decisions were made regarding the materials and layout of the floor system. Based on some basic calculations an 8" thick, 5000 psi concrete two-way system was chosen. Originally, ½" diameter unbounded grade 250 tendons were chosen but were later replaced with 0.6" diameter tendons to reduced the number of required tendons. When designing 15% losses in strength were assumed with the tendons.

The profile layout of the tendons has them centered at the mid point of the slab, 4 inches, at exterior edges and with 1-1/4" at profile points. This leads to a drape of 4.125" in an exterior span and 5.50" drape in an interior span. The overall tendon layout bands together the majority of the tendons in the longitudinal direction with the three main bands traveling over the three column strips. This banding or grouping of tendons in one direction allows for the concrete to be poured and distributed easier throughout the slab. The transverse direction evenly distributes small groups of tendons throughout the entire slab. Figures 2-11 and 2-12 show the general layout of the tendons in each direction.

The original assumption for the load balancing was to attempt a 50/50% balance in each direction. This was made the target distribution based off of the approximately square bays, 30' x 30' 5-1/2". However, through the design process adjustments made settled the distribution to around 60% in the transverse direction and 40% in the longitudinal direction.

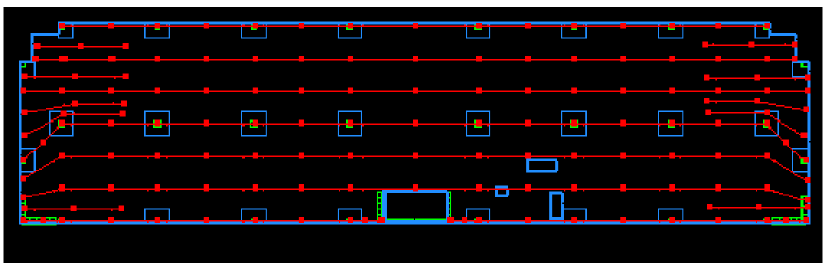

Fig. 2-11

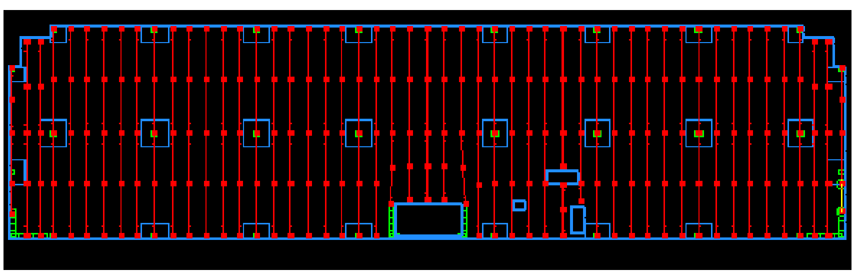

Fig. 2-12

2.5.3 Tendon Design

Once the basic calculations and layout assumptions were made the RAM Concept model was used to further design the slab. Using the auditor tool in the program, tendons were added and subtracted as needed to complete the design. Additional tendons were added in the end span of the longitudinal direction which contains the banded tendons. These additional tendons were used to help support the cantilevered corner conditions well as the offset column layout. Figure 2-13 shows the final design of the tendon layouts in the banded and distributed directions. The tendon plans show only half of the slab for clarity with the opposite half being a mirror image. Also left off the plans are tendon layouts around and locations of slab openings. Although designed for in the model, as can be seen in figures 2-11 and 2-12, they were left off the plans for clarity reasons as well. The openings which exist in the slab are relatively small and can be accounted for by slightly adjusting the tendon paths around them. Additional mild steel reinforcing would also be laid around the opening to provide extra support.

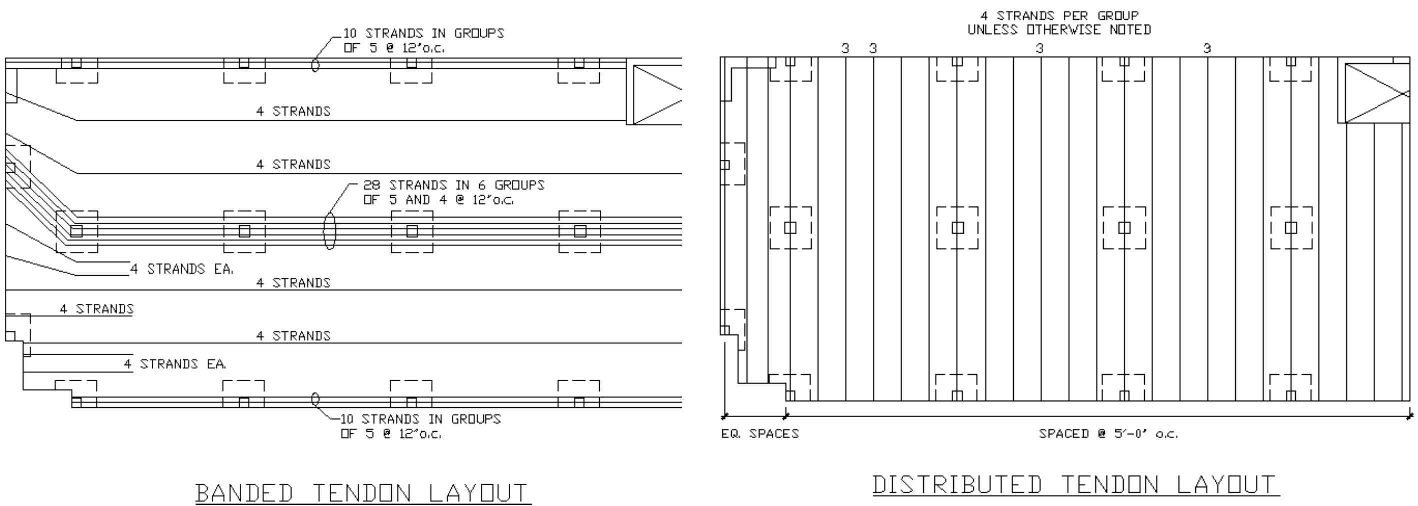

Fig. 2-13

A summary chart of the stresses present along column lines and at mid-span locations of a typical bay is presented in figure 2-14. Also a long term deflection plan generated in RAM Concept is shown in figure 2-15. This plan was used to check that the floor deflection was under the allowable limit of L/240. Using the typical 30 foot span that limit become 1.5 inches. The lightest blue present at the mid-span locations of the bays is equivalent to 1.2 inches which is under the allowable limit. The more extreme deflections present in the center 40 foot span are a result of the concrete beams, addresses in section 2.5.4, not being incorporated into the RAM Concept model.

		Service Load Stress	Unbalanced Moment Stress	Total Stress	Comp/Ten	Max Allowable Stresses	PASSED
STRESSES AT MIDSPAN AFTER LOSSES		(psi)	(psi)	(psi)		(psi)	
Transverse	fa top	-264	-279	-543	C	-2250	ok
	fa bot	-264	279	15	T	424	ok
Longitudinal	fa top	-170	-270	-440	C	-2250	ok
	fa bot	-170	270	100	T	424	ok
STRESSES AT EDGES AFTER LOSSES							
Transverse	fa top	-264	436	172	T	424	ok
	fa bot	-264	-436	-700	C	-2250	ok
Longitudinal	fa top	-170	422	252	T	424	ok
	fa bot	-170	-422	-592	C	-2250	ok

Fig. 2-14

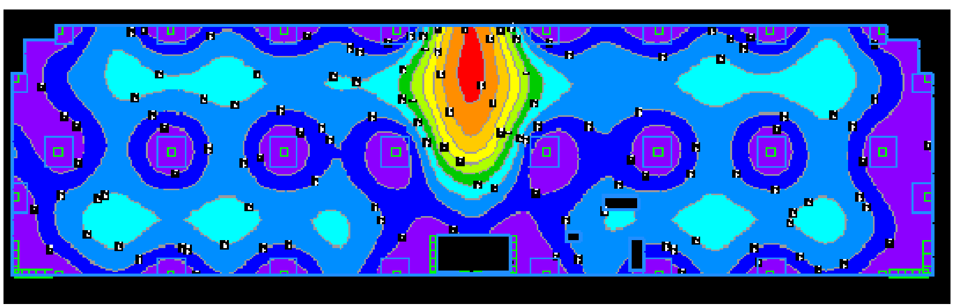

Fig. 2-15

2.5.4 Additional Design Considerations

A main issue that must be addressed in designing concrete floor systems is the punching shear at columns locations. Several methods for combating this issue in the post-tensioned slab were looked into including shear caps/drop panels, SSR or steel stud rail reinforcing, and other steel shear reinforcing. The RAM Concept program used in design of the slab offered an option for SSR reinforcing design. The advantage here was that no additional structural depth would be added to slab for shear purposes. However, not having any experience in SSR design the choice was made to investigate other options. Drop panels and shear caps were decided upon based on the ease of their design and their ability to provided shear relief without adding additional material into the slab itself. The additional structural depth this method added was not viewed as a problem seeing that the overall structural depth would still be less than the original design. Additionally, with National Harbor Building M being only five stories it was not possible to create enough additional height to add another floor. Had it been a taller building the decrease in floor-floor height may have led to achieving more stories at the same overall height. In this case the SSR system would have been further investigated to capitalize on the revenue created be additional leasable space.

Hand calculations and the RAM concept model were used to identify column locations where punching shear problems were occurring. After the investigation it turned out that the majority of the columns would require some form of reinforcement. To simplify design of the caps the worst case shear conditions at the column grid's exterior and interior were selected to be designed for. The result was a typical cap which was four inches thick and 7.5'x7.5' wide. A complete design of the floor system would have probably included a broader ranger of cap sizes designed to different shear conditions. However, for the scope of this report the extreme condition was designed for and applied at all locations where shear failure was a problem. A detailed calculation for the design of the shear cap/drop panel can be found in the appendix.

Another member considered in the completion of the post-tensioned floor system was the concrete beam spanning the 40' bay at the center of the slab. This beam was incorporated to assist the post-tensioning tendons in providing support across the longest bay of the slab. Had no beam been included the banded tendons in the longitudinal direction would have been controlled by the required capacity of the 40' span. With the addition of the beam the bay can easily be spanned and its deflections kept within the required criteria. Figure 2-16 shows the location of the beam and the shear caps/ drop panels added to the design.

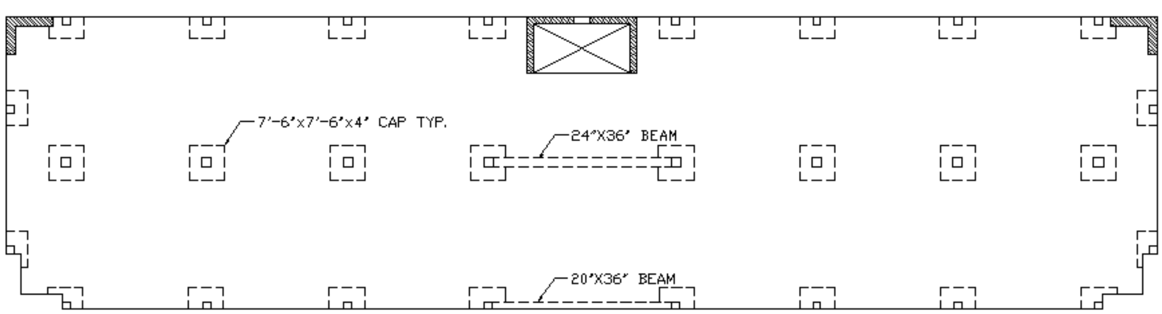

Fig. 2-16

2.6 CONCRETE COLUMN DESIGN

2.6.1 Layout

The layout of the reinforced concrete column grid very closely follows that of the existing W-shape steel column grid. The only changes in existing grid occur at the center 40' span which outlines the building's lobby/ elevator core (see figure 2-17). In this span there is an existing column splitting the 40' span along building's front façade which is removed. In the new concrete design this column is not needed because the bay is spanned by a concreted beam running in the long direction. Additionally, two exterior columns are added along the rear column line of the building. These columns are in line with the 40' span (see figure 2-17), and were added to make the post-tension floor system design less complex.

In the interest of uniformity all of the exterior columns were held to the same dimensions, 20"x20", and a typical reinforcement design was created based upon the controlling load case. Similarly, the interior columns were all designed as 24"x24" columns reinforced based on the controlling loads. In a complete design individual reinforcement requirements may have been calculated for more column locations. However, in the interest of time these two typical columns were used as model columns for the entire layout. Due to the fact that the column grid is fairly uniform this simplification should not be too conservative for the purposes of this report.

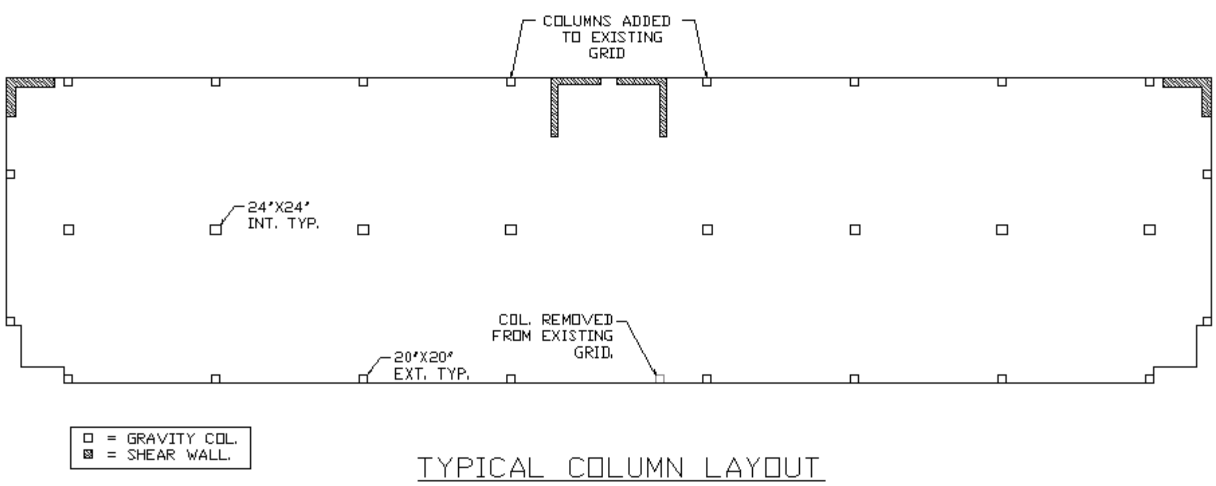

Fig. 2-17

2.6.2 Determination of Loads

Once the thickness of the post-tensioned slab was determined it was possible to calculate the gravity loads which must be carried by the reinforced concrete columns. An 8" thick slab with normal weight concrete gives it a weight of 100psf. The additional superimposed dead load of 25psf gives the floor system a 125psf total weight. The application of the live loads, reduced per ASCE code, found in the loads section allows the total axial load of the columns to be calculated. An Excel spread has been created and shows the load accumulation per floor level of dead and live loads for the typical interior and exterior columns (See figures 2-18 and 2-19). The column moments due to gravity loads were determined per the equation given by ACI 13.6.9.2. This equation takes into account full dead load on each side of the column as well as patterned live load applied to one half the greater side of the column.

Typical Column Axial Service Load Accumulations:

Column Name:	Typ. Interior		* self weight counts column above					
Column Size:	24x24		*units are in Kips and SF					
	Level	**Trib Area**	**A.I.**	**LL**	**Llred**			
	Roof	915	3660	0.03	0.030			
	2	1830	4944	0.1	0.046			
	3	2745	6228	0.1	0.044			
	4	3660	7512	0.1	0.042			
	5	4575	8796	0.1	0.041			
	base							
			DL	**Self Wt.**	**Total DL**	**LLred**	**Total LL**	**Total**
Accumulation:	Roof	915	0.125		114	0.030	27.45	142
	2	915	0.125	7.80	237	0.046	69.84	306
	3	915	0.125	7.80	359	0.044	110.11	469
	4	915	0.125	7.80	481	0.042	148.82	630
	5	915	0.125	7.80	603	0.041	186.33	789
	base			11.40	614		186.33	801

Fig. 2-18

Column Name:	Typ. Exterior	* self weight counts column above
Column Size:	20x20	*units are in Kips and SF

	Level	**Trib Area**	**A.I.**	**LL**	**Llred**
	Roof	457.5	1830	0.03	0.030
	2	915	3114	0.1	0.052
	3	1372.5	4398	0.1	0.048
	4	1830	5682	0.1	0.045
	5	2287.5	6966	0.1	0.043
	base				

			DL	**Self Wt.**	**Total DL**	**LLred**	**Total LL**	**Total**
Accumulation:	Roof	457.5	0.125		57	0.030	13.73	71
	2	457.5	0.125	5.42	120	0.052	37.46	157
	3	457.5	0.125	5.42	182	0.048	59.25	242
	4	457.5	0.125	5.42	245	0.045	79.79	325
	5	457.5	0.125	5.42	308	0.043	99.45	407
	base			7.92	316		99.45	415

Fig. 2-19

Controlling Column Service Moments:

Item	**Dead Load Moment**	**Live Load Moment**
Typical Interior	88 FT-K	77 FT-K
Typical Exterior	67 FT-K	53.3 FT-K

Fig. 2-20

2.6.3 Design of Columns

Once all axial loads and moments were determined the worst case service loading of typical interior and exterior columns were then input into PCA column. The program then used load combinations and specified design parameters to acquire the column reinforcing. The resulting typical reinforcing requirements can be found in figure 2-21.

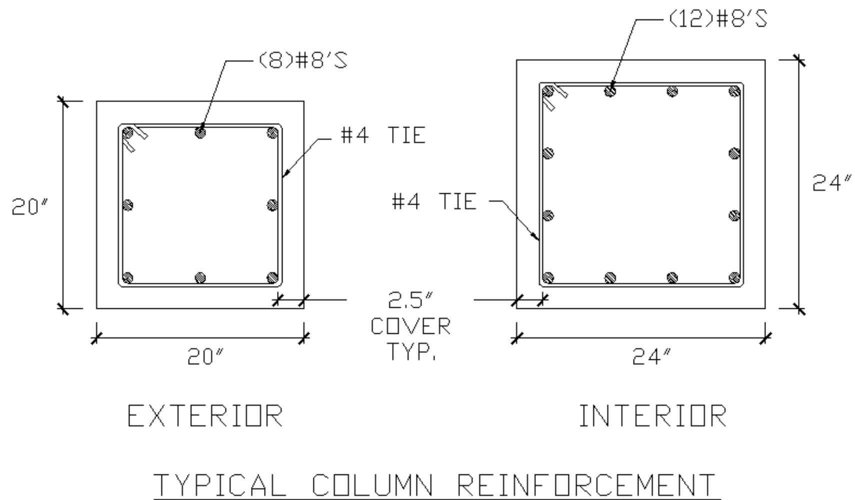

Fig. 2-21

2.7 LATERAL SYSTEM DESIGN

The redesign of the lateral system to accommodate the concrete structural system of National Harbor Building M proved to be an intricate process. Many different arrangements and combinations of lateral elements were considered including concrete moment frames, concrete shear walls, bladed concrete columns, and masonry shear walls. The process proved to be an iterative one with every change of lateral layout bringing changes in building period, weight, and stiffness thus resulting in differing seismic forces. To aid in this time intensive process an ETABS model was created and adjusted numerous times to analyze each specific layout. The model was used mainly to determine building periods, drifts, and distribution of loads among lateral members. In the end, the system which worked most efficiently within the constraints of the building's function and architectural layout was selected. This section will step through the criteria used to select this system, the controlling lateral loads and their distributions, the modeling of the selected system, and the specific design of the individual lateral members of the system.

2.7.1 Lateral System Selection

A main criterion in the selection of the lateral system, as in many other areas of this report, was the system's impact on the building's function and architectural layout. With the building's function serving as open office and retail space it was imperative to maintain large open areas without obstructions. The layout of the building being only 2 bays x 9 bays eliminated the possibility of using any shear walls along the building's interior grid.

Additionally, the front and two side facades contain large windows making it difficult to hide shear walls along those lines. The rear wall of structure acts as a barrier to the parking garage with very limited openings making it the most logical location for shear walls.

Taking all of the restricting factors into account a number of solutions were initially considered. A lateral system which employed concrete moment frames was the first system to be investigated. It was determined that using a moment frame based lateral system in conjunction with a post-tensioned floor, while doable, would not be the most efficient system to design or construct. Another system investigated utilized the masonry shear walls designed in the existing lateral system. This design would couple the masonry walls with bladed concrete columns. The bladed columns would be strategically placed reinforced concrete columns on the order of 20"x60". These columns would collect the lateral loads and provide stiffness in the transverse direction. The lateral resistance in the longitudinal direction would come from the masonry shear walls along the rear façade of the structure. These walls would be attached to the rigid diaphragm at each level by a connection that engaged the walls horizontally but was slotted vertically to allow deflections. This system, while analyzed to be an effective system, was decided against in an attempt to keep the entire structure concrete based.

The system that was ultimately decided upon consists of concrete shear walls located around the building's two rear corners and its elevator core (See figure 2-1). The walls at the building's corners are L-shaped and thicker than the walls found around the elevator core. The L-shape of the corner walls allows them to gain stiffness by taking advantage of the flanges. This along with the increased thickness of these corner walls helps the structure combat torsion. A significant amount of torsion, with it will be developed as a result of the longitudinal lateral members being located along an exterior grid line, must be resisted. Each leg of the corner walls was checked against the architectural plans to assure they do not interrupt window coursing on the side façade or stairwell openings on the rear façade. The two L-shaped shear walls flanking the elevator core were originally designed as one continuous C-wall. However, with the length of the corner walls in the longitudinal direction being restricted by the stairwell openings, the length of the elevator core caused the C-wall to dominate the stiffness. As a result the approximately 24' long side of the C-wall was taking around 90% of the lateral load in the longitudinal direction. To reduce this amount it was decided to break the C-wall in the center creating 3'-4" gap between two mirrored L-walls. Though the break reduced the overall stiffness of the building, raising the period around 0.2 seconds and slightly increasing the longitudinal drift, it did achieve the goal of evenly distributing the longitudinal loads, see section 2.7.5. With the L-walls being encased around the elevator, the entire system was able to provide ample stiffness without obstructing the building's function or layout.

2.7.2 Controlling Loads

To determine the controlling lateral loads a comparison of the base shear numbers found for seismic and wind loads in sections 2.3 and 2.4 respectively was conducted. A straight comparison of these numbers may be misleading and incorrect because of differing load factors applied to each load type. For the lateral wind loads a factor of 1.6 is applied and for seismic loads a factor of 1.0 is applied. Note that there is a slightly more specific seismic load factor that will be applied when designed individual member, but for the sake of comparison a straight 1.0 factor is used. The factored loads are shown in figure 2-22 with the controlling loads in bold.

Controlling Loads Summary:

Item	Longitudinal (N-S/S-N)	Transverse (E-W/W-E)
1.6 Wind	141K	**730K**
1.0 Seismic	**369K**	370K

Fig. 2-22

2.7.3 Vertical Load Distribution:

The vertical distribution of lateral loads is handled differently depending on the type of load that is acting on the structure. Since each axis of National Harbor Building M is controlled by a different load type, seismic in the longitudinal and wind in the transverse direction, two different distributions were performed.

For the wind distribution figure 2-7 is used and story forces are determined by taking the pressure from half of the floor above and half of the floor below multiplied by the total building width. For the seismic distribution a weighted distribution is calculated based upon the weight of each floor, the height of each floor, and an exponent determined from the building period. The story forces are summarized below and more detailed calculations of both the wind and seismic distributions can be found in the appendix. Note that the numbers used in this calculation are service loads and no factors have been applied. These are the service loads entered into the ETABS model and will be factored when they are distributed to individual members for design purposes.

Lateral Story Forces:

Story	Longitudinal (Seismic)	Transverse (Wind)
Roof	121.7K	47.5K
5th Floor	97.3K	90.4K
4th Floor	72.9K	84.9K
3rd Floor	50.0K	83.1K
2nd Floor	28.1K	95.6K

Fig. 2-23

2.7.4 Lateral ETABS Model

As mentioned earlier in this chapter an ETABS model was created and used for the dynamic analysis of the lateral system. This section will briefly identify the basic information input into the model, the tools of the program which were used and the final results taken from the analysis to be used for design of specific members. An extruded view of the lateral system ETABS model can be seen in figure 2-24.

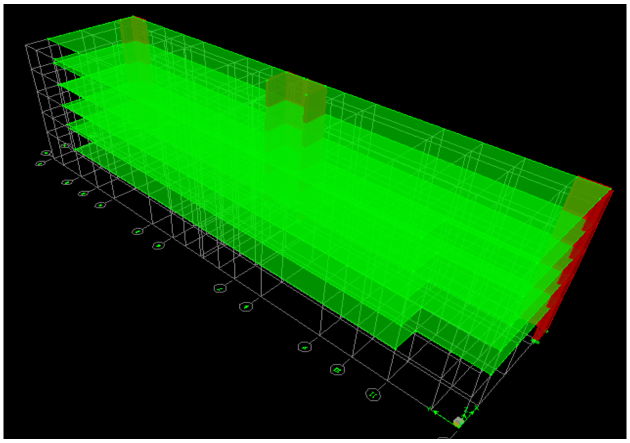

Fig. 2-24

The concrete shear walls were defined with basic concrete properties and were modified to exhibit cracked behavior. The walls were then meshed and assigned pier labels for the retrieval of forces. At each floor level a rigid diaphragm was assigned to link all of the lateral members together. At each individual diaphragm the given story mass, which was modified from the seismic weight found in figure 2-3, and lateral forces (see figure 2-23) were applied. The model was then run considering P-delta effects and three modes of response. After analysis the program identified the three modes, see fig. 2-2, which correspond to building periods. The controlling building period was then used in calculations as noted in other sections. Drift and rotation values were obtained from the deformed shapes of each load cases. The rotation values are a result of the program accounting for torsion in the structure. Finally, a print out of the assigned pier labels reported the shears and moments present at each story of the shear walls. Since the loads applied at each story level were service loads, either a load combination was added to the system or the member forces were factored during design.

2.7.5 Horizontal Load Distribution

The distribution of loads horizontally to the lateral resisting members was done based off of each member's relative stiffness. A rigid diaphragm inserted at each story level of the aforementioned ETABS model allowed the program to distribute the lateral forces automatically. Once the loads were distributed, the lateral forces acting at the base of each shear wall in each direction were compiled into a spreadsheet seen in figure 2-25. The spreadsheet was then used to check the percentage of the overall lateral load resisted by each member. As the sheets show, the loads are fairly well distributed among the members acting in each direction. It should be noted that the shear values used in this spreadsheet are not factored but are used solely for a distribution percentage check. These forces were later factored when used in the design of each respective shear wall. Also worth noting is that this is a general check of the distribution of lateral forces at the base level, the distribution may vary slightly at differing story levels.

Transverse Direction – Wind Controlled

Lateral Member	Lateral Load (K)	% of Load
SW 1A	77.3	19.15
SW 2A1	124.5	30.85
SW 2A2	124.5	30.85
SW3A	77.3	19.15

Longitudinal Direction – Seismic Controlled

Lateral Member	Lateral Load (K)	% of Load
SW 1B	88.8	22.39
SW 2B1	96	24.21
SW 2B2	96	24.21
SW3B	88.8	22.39

Fig. 2-25

2.7.6 Drift

As are the cases in other sections, the differing types of controlling loads require the building drift and displacement to be handled differently in each direction. In the transverse direction the wind load was factored by 1.6 with the displacement being compared to a general H/400 deflection ratio. In the longitudinal direction the story drifts were amplified by a 4.0 Cd factor and compared to the code dictated 0.020 Hsx drift limit. All of the drift and displacement base numbers were gathered from the ETABS model output. It can be seen that the longitudinal direction has approximately equal drifts and displacements in each direction, whereas the transverse direction does not. This can be attributed to the large eccentricity between the center of mass and center of rigidity in the longitudinal direction. The transverse direction has essentially no eccentricity with the lateral system being symmetric in that direction.

Transverse Direction - 1.6Wind **UNITS IN INCHES

STORY	DISP-X	DRIFT-X	DISP-Y	DRIFT-Y	ALLOW. DISP. (Hx/400)
R	0.849	0.216	-0.002	-0.001	2.240
5	0.633	0.198	-0.002	-0.001	1.770
4	0.436	0.183	-0.001	-0.001	1.370
3	0.253	0.150	-0.001	0.000	0.970
2	0.103	0.103	0.000	0.000	0.570

Fig. 2-26

Longitudinal Direction- 1.0Seismic

**UNITS IN INCHES

STORY	DISP-X	DRIFT-X	AMP. DRIFT-X	DISP-Y	DRIFT-Y	AMP.DRIFT-Y	ALLOW. DRIFT (0.020Hsx)
R	-1.362	-0.372	-1.488	1.077	0.293	1.172	17.600
5	-0.990	-0.331	-1.324	0.784	0.261	1.044	14.160
4	-0.659	-0.294	-1.176	0.523	0.233	0.932	10.961
3	-0.365	-0.228	-0.912	0.290	0.181	0.724	7.759
2	-0.137	-0.137	-0.548	0.109	0.109	0.436	4.560

Fig. 2-27

2.7.7 Lateral Member Design

After all load cases have been analyzed and all lateral forces have been applied and distributed, the final step is the design of the specific members of the lateral system. To begin this process the service shear and moment values for each shear wall obtained through the ETABS model were factored. The controlling LRFD load combinations for the shear walls were 0.9D + 1.6W for wind in the transverse walls and (0.9-0.2Sds)D + rhoQe for seismic in the longitudinal walls. The value of rho was taken to be 1 based on the building being classified as Seismic Design Class B.

The shear walls were next checked for their need of boundary elements. According to the ACI code, if boundary elements are required in a flanged shear wall section then the boundary element shall extend into half the flange length. The need for boundary elements was examined in both directions of the L-walls and where they were required in both directions they were designed simultaneously. Designing the shear walls in this manor helps to engage the flange of the L-walls and thus increases their stiffness. In addition to making them stiffer, engaging the flange of the walls will help the structure resist twisting induced by torsion forces. This was a main reason shear walls were placed at the corners of the building. The boundary element reinforcing was then designed to stay within the original thickness of the walls. With the shear walls lining hallways and an elevator core it was decided it would be best to prevent the boundary elements from protruding into the space. Confinement reinforcing was then added and longitudinal and transverse reinforcing was designed for the rest of the walls. The design of the two typical shear walls can be seen in figure 2-28 and more detailed calculations can be found in the appendix.

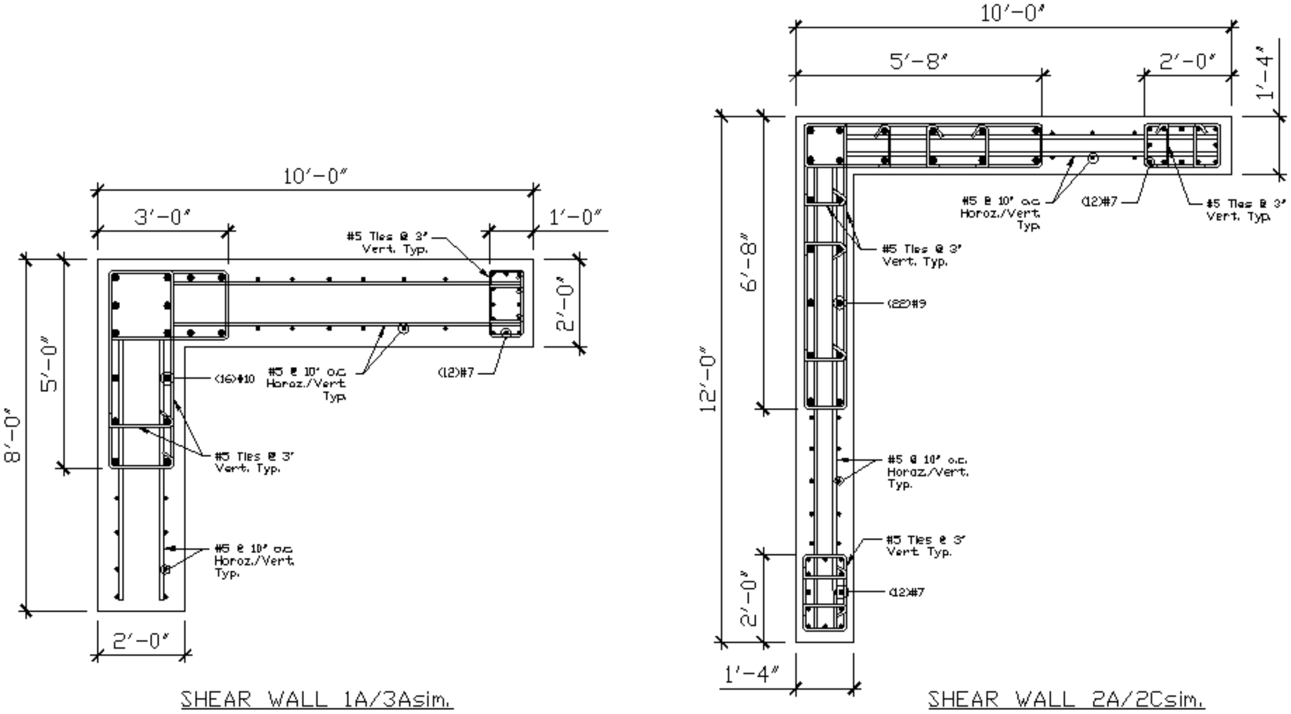

Fig. 2-28

2.8 FOUNDATION MODIFICATION

The foundation system selected for the existing design of National Harbor Building M relies on driven 14" square precast prestressed concrete piles to anchor the building into the earth. A general geotechnical investigation confirms that this is an appropriate foundation system for the given site conditions. With that in mind, a modification of the existing system, rather than a complete redesign, seemed the most logical procedure to take. To carry out the modification, changes in loads and moments ultimately transferred into the foundation system were investigated.

The most obvious modification associated with a switch to a concrete based structure from a steel based structure would be the overall gravity loads. This could potentially be a drastic change in required axial capacity of the foundation, thus possibly requiring a complete redesign. In a multi-story high rise structure, a switch from steel to concrete could defiantly warrant the investigation of another foundation system. However, the fact that Building M's weight is spread out more horizontally, with it only being 74' tall, lessens the impact the increase in weight has on the foundation.

A second area where foundation loads must be examined is the uplift and lateral forces associated with the lateral systems. The uplift forces are a resultant of the overturning moment of the structure. With the building weight increasing, the force which resists overturning moment, it is conceivable that the uplift capacity of the foundation will decrease. That being said, the relocation of lateral resisting members can affect the lateral and uplift requirements at specific foundation locations. The original lateral system relied on a design which mainly used frames and shear walls spread across the building. This system allowed for greater distribution of the lateral and uplift forces over the foundation, not applying large loads to any one pile group. The redesigned lateral system, constrained by architectural requirements and concrete design flexibility, relies on a more concentrated design. By this it is meant that all of the lateral and uplift loads are handled by the foundation at only three pile groups, two at the rear corners and one at the elevator core.

The axial loads to be handled by the foundation were calculated for the typical column locations as well as the shear wall locations. Spread sheets were created to assist in accumulating these loads and can be found in figures 2-18 and 2-19 for the typical columns and figures 2-29 and 2-30 for the shear walls. The ETABS model discussed in section 2.7 was used to distribute the lateral forces between shear walls and calculate their overturning moments. These loads were factored accordingly and used to calculate the axial, lateral, and uplift capacities required for each pile group.

Column Name:	Shear Walls 1-3		* self weight counts column above
Column Size:	10'x8'x2'		* units in K and SF

Level	Trib Area	A.I.	LL	Llred
Roof	393	1572	0.03	0.030
2	786	2856	0.1	0.053
3	1179	4140	0.1	0.048
4	1572	5424	0.1	0.045
5	1965	6708	0.1	0.043
base				

			DL	Self Wt.	Total DL	LLred	Total LL	Total
Accumulation:	Roof	393	0.125		49	0.030	11.79	61
	2	393	0.125	62.40	161	0.053	32.65	193
	3	393	0.125	62.40	272	0.048	51.63	324
	4	393	0.125	62.40	384	0.045	69.46	453
	5	393	0.125	62.40	495	0.043	86.48	582
	base			91.20	586		86.48	673

Fig. 2-29

Column Name:	Shear Wall 2		* self weight counts column above				
Column Size:	12'x10'x1.33'		* units in K and SF				
	Level	**Trib Area**	**A.I.**	**LL**	**Llred**		
	Roof	863	3452	0.03	0.030		
	2	863	4736	0.1	0.047		
	3	863	6020	0.1	0.044		
	4	863	7304	0.1	0.043		
	5	863	8588	0.1	0.041		
	base						

			DL	**Self Wt.**	**Total DL**	**LLred**	**Total LL**	**Total**
Accumulation:	Roof	863	0.125		108	0.030	25.89	134
	2	863	0.125	118.31	334	0.047	66.28	400
	3	863	0.125	118.31	560	0.044	104.53	665
	4	863	0.125	118.31	786	0.043	141.26	928
	5	863	0.125	118.31	1013	0.041	176.80	1189
	base			172.91	1186		176.80	1362

Fig. 2-30

The aforementioned precast prestressed piles have an axial capacity of 110 tons, or 220 Kips, with a lateral capacity of 7.5 tons (15K). Piles which contain uplift capacity can resist 55 tons (110K) uplift, 55 tons axial (110 tons), and 7.5 tons lateral (15K). The uplift capacity is obtained through a positive connection to the pile cap. This positive connection is achieved by drilling a threaded #14 bar through the pile cap at a minimum of 25' into the pile. It should be noted that a detail of the concrete column connection to the concrete pile cap would be required to modify the existing steel column connection. For the purposes of this report it will be assumed this connection is possible and the change will not affect the overall foundation system.

Using the accumulated loads and the given pile capacities, simple calculations were performed to check and modify where necessary the typical pile groups. It was found that the pile group's supporting the interior columns will require an additional axial pile while the exterior pile group has sufficient capacity as designed. Shear walls 1 and 3 will require 5 uplift piles and 4 axial piles which will be arranged in an L-shaped pile group to support the walls. To keep the pile group uniformly supported an additional uplift pile will be added making a total of 10 piles. Analysis of the mat-pile group supporting the elevator core and shear walls 2a-2d show it has sufficient capacity and will remain as designed. The pile cap thicknesses and reinforcing bars were adapted proportionately to the values used in the original design. Pile group layouts can be seen in figure 2-31, and more detailed calculations can be found in the appendix.

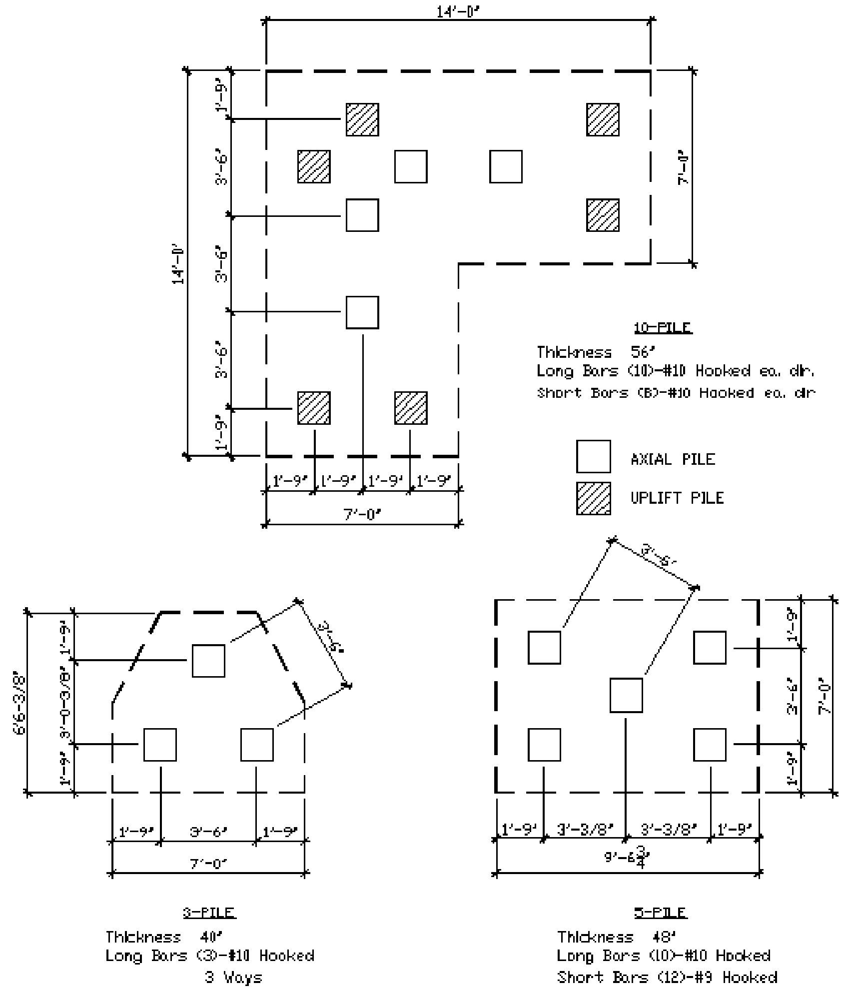
14'-0"
7'-0"
10-PILE
Thickness 56"
Long Bars (10)-#10 Hooked ea. dir.
Short Bars (8)-#10 Hooked ea. dir
AXIAL PILE
UPLIFT PILE
3-PILE
Thickness 40"
Long Bars (3)-#10 Hooked
3 Ways
5-PILE
Thickness 48"
Long Bars (10)-#10 Hooked
Short Bars (12)-#9 Hooked

Fig. 2-31