40 Bond

New York, NY

Technical Report 3

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Structural Option

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Executive Summary

This third technical assignment includes an analysis and confirmation of the original lateral system designed by DeSimone Consulting Engineers (DCE). The loads calculated in the structural concepts and existing conditions report were applied to the lateral force resisting system composed of ordinary reinforced concrete shear walls. Necessary revisions were made to the initial wind and seismic loads, which were then included in the various load combinations set forth by ASCE 7-05 for strength design. An ETABS computer model was created and its output was compared to hand calculations to verify the shear strength of the system. Torsion, overturning and the impact on foundations were all examined in this manner. Overall building and story drifts were also compared to the allowable limits set forth by code and industry.

The computer model that was created included only the shear walls and the rigid diaphragms for the building. The gravity columns were not modeled at this stage in order to simplify this first attempt to create a model of 40 Bond. Also, hand calculations were done neglecting the presence of the coupling beams because of the involved nature of such calculations. There is acknowledgement, however, of how the coupling beams would affect certain results that were computed by hand.

After making such assumptions to complete the hand calculations, comparison were done between those values computed and those output from ETABS. It was determined that the model was taking the slab's rigidity into account and shifted the center of rigidity, while the calculations treated the shear walls as the only lateral force resisting elements. For this reason, the values computed by hand were those used in subsequent calculations including that for relative stiffness, torsion, direct shear, torsional shear, drift, displacement and overturning. The results suggest that it was reasonable to look only at the shear walls in this analysis. There were no serious concerns in regards to torsion, shear or overturning which suggests that the shear walls are providing the majority of the lateral resistance with minimal assistance from the slabs and none from the columns. Also, the drifts and displacement were within the limits and the values that seem somewhat large in size are attributed to the fact that the core was not examined as a core, but rather as individual shear walls acting independently.

Introduction

40 Bond is located on a 13,600 ft^2 parcel of land located on Bond Street between Lafayette and Bowery Street in New York City. The footprint of the building is 64'-8" by 134'-4" and the building has an overall building height of 152'-0" from the cellar to the top of the penthouse structure. There is a 20'-0" setback at the seventh floor with a roof terrace that occupies this space. Typical spans range from 19'-6"×25'-0" to 23'-2 ½"×25'-0" and floor-to-ceiling heights range from 11'-10" to 14'-0". A total of 23 condominium units and 5 townhouses are contained within this building and the plans vary as the type and number of units change throughout. In addition to the building there is also a 140'-0" long, 22'-0" high cast aluminum gate located along Bond Street that was designed to withstand the lateral forces that are present at this site.

Architectural Design Concepts

40 Bond Street was designed by the Swiss firm Herzog & de Meuron with New York based Handel Architects. The idea behind this luxury residential building was to reinvent the cast iron building typology that is prevalent in this lower Manhattan neighborhood. The building consists of one below grade level that houses a fitness center, storage space and equipment rooms. The first and second floors contain five through-building, 2-level townhouses. The layout then changes to accommodate four condominium units on each level from the third to the sixth floor. Once again, at the seventh floor the plans change incorporating a 20'-0" setback and reduced number of condominium units including only two per floor from levels 7 to 9. The tenth floor is a full plan condominium with a penthouse structure that rises 20'-0" above the main roof. In the penthouse a direct relation can be made between architectural concepts and structure. A 44'-0" clear span is achieved with two hidden columns and the core shear wall as supports leaving nearly three completely glass walls.

The south face also enforced some strict tolerances in regard to structure. Operable floor-to-ceiling windows are held in place with green glass mullions (Figure 1). This entirely glass façade limits the variation in columns to less than ½". The north façade contains the same windows but the glass mullions are exchanged with pre-patina copper. These mullions then serve as a grid for the perimeter columns along the north and south faces. Small 10"×10" concrete columns are located behind these mullions and space at 6'-3" on center between the second and tenth floors. The variation in layout, fluctuating column dimensions, and necessary setbacks resulted in different transfer locations that required beams to redirect the loads.

Figure 1 – South Facade

With many buildings located in cities such as New York, there is always an awareness of retail value. The more space there is to offer the more expensive the unit may be. The flat plate concrete system allows for tall floor-to-ceiling heights that remain unobstructed because of the limited number of beams and girders dropping into the space. In order to preserve the architectural design, maximize area and create appealing spaces, the concrete structure deviates from what is typical in the design and construction of a residential building to create an aesthetically pleasing and interesting structure. As a result of these characteristics, however, this 90,000 sf building had a very high cost in comparison to its size which is attributed to such things as formwork required for transfer beams and many slender columns.

Structural System

Foundation

The geotechnical engineering study was performed by Langan Engineering & Environmental Services on September 10, 2004. In this study it was found that the water level was approximately 42.8' below the existing ground surface. The cellar extends 12'-8" below grade and therefore there was not a concern in regard to increased uplift pressures at this level. Langan noted that the bearing materials were suitable for a shallow foundation and that the recommended allowable bearing pressure would be 5 kips/ft^2. As a result, a 30" reinforced concrete mat foundation was designed with bearing walls and buttresses supported by a strip footing.

The 30" slab is 5 ksi normal weight concrete (NWC) and increases to a thickness of 48" and 84" within the core shear walls where the elevator pit is located. Reinforcement varies throughout this mat slab. Buttresses ranging in size from 14"×29 ½" to 18"×79" are located around the perimeter. Interior columns ranging in size from 12"×22" to 28"×28" have an increased strength of 8 ksi. Located at columns 3B, 3C and 3F (Figure 2), there are also foundation mat shearheads to resist punching shear due to high loads that continue from the roof down to the foundation.

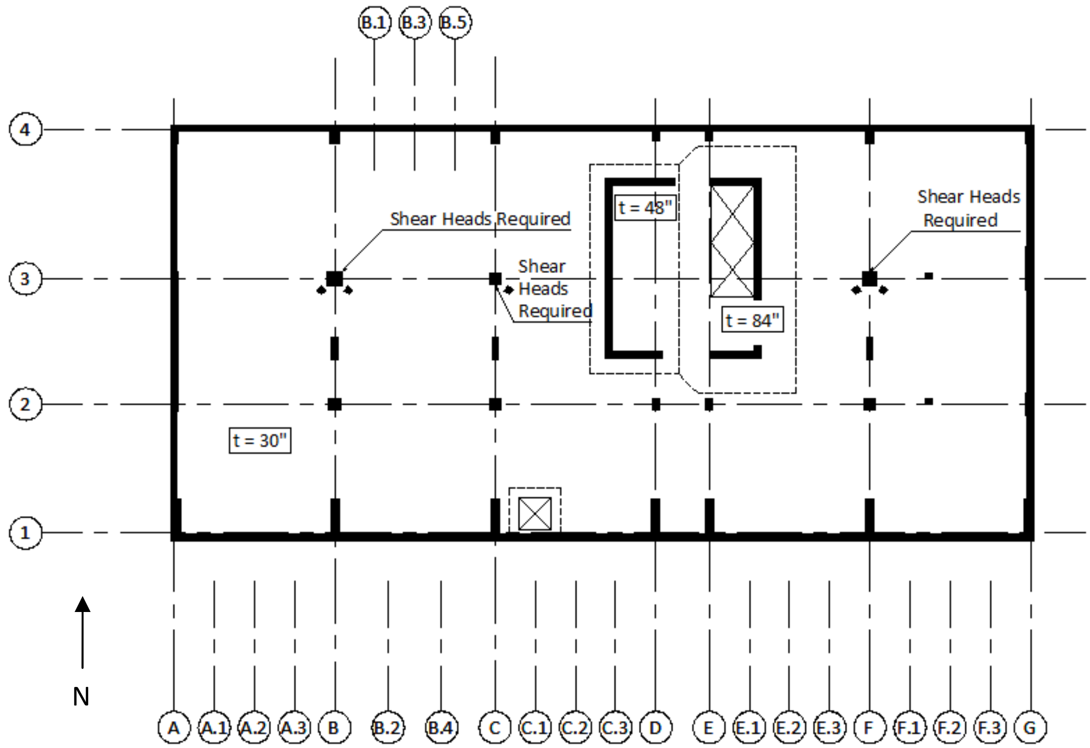


Figure 2 – Foundation Plan with Typical Column Grid and Shearhead Locations Noted

Superstructure

The ground floor is a 9" two-way flat plate (NWC) with a compressive strength (f'_c) of 5.95 ksi and typical reinforcement of #4@12 top and bottom with various sizes and spacing of bars at column locations. Located at the south face is a slab step that transitions to a 12" slab for the townhouse entrances. Typical to the floors above, there are also 3" slab depressions at the fireplaces and toilet areas and 14" slabs within the core. Perimeter columns ranging in size from 10"×24" to 16"×58" are located on the north, south and east walls while a 12" thick shear wall runs along the west face. The interior columns dimensions are then 12"×22", 22"×22" and 28"x28". All of the columns from the foundation to those supporting the fourth floor have a concrete strength of 8 ksi. There are beams located around the stair openings in the townhouses and coupling beams that tie together the core shear walls which are typical on all floors.

The second and third floors have the same two-way flat plate slab as above without the slab step. Particular to the second floor is the introduction of the 10"×10" concrete columns spaced at 6'-3" on center along the north wall that extend up the remaining height of the building. Because these closely spaced columns need to transition to fewer columns below, 14"×40" transfer beams (f'_c = 10 ksi, typical to all transfer beams) run the full length of this wall. The beams around the townhouse stair openings are also present on the second floor. The third floor then has the introduction of the 10"×10" columns spaced at 6'-3" on center along the south face. The transfer beams at this level are 60”×16” and extend the full length of this wall. These columns continue to the seventh floor where they step back 20’-0” due the setback at that level. This thin, wide transfer was implemented to limit the intrusion into the space below. Also, all the 10”×10” columns only have a 7” slab encroachment that has a 1” slab depression around each column (Figure 3).

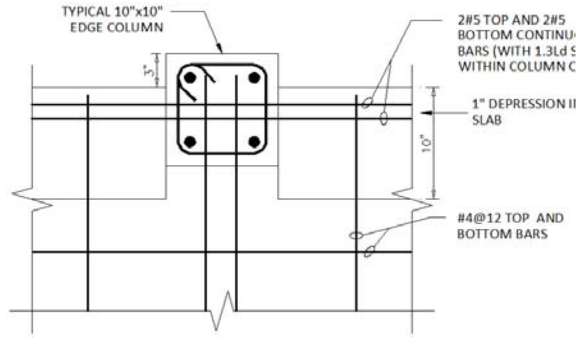


Figure 3 – Typical Perimeter Column Detail

All floors between level 4 to the penthouse level use a 9” two-way flat plate with #4@12 top and bottom plus various reinforcement at columns and a reduced compressive strength of f'_c = 5 ksi. Similar slab depressions and increased slab thickness at the core are present. The columns supporting the fifth floor and above also have a lower compressive strength of f'_c = 5 ksi. The columns along the north and south façade remain 10”×10” while those located on the east and west walls and within the interior vary between 12”×22” to 28”×28”. There is also the introduction of 22” diameter (Ø) circular columns that are used on some floors dependent on the tenant’s request in their condominium. In addition to the beams within the shear wall core, there are also spandrel beams along the east and west faces.

At the fourth floor a transfer beam is present along the east wall (Figure 4). This 12”×50” transfer was designed after construction began due to the presence of an adjacent chimney encroachment on site. Then at the seventh floor the setback takes place. It is here that loads increase due to the roof terrace provided by this setback. A 20”×24” transfer beam along line 2 is needed, due to the introduction of the 10”×10” columns along this line (Figure 5).

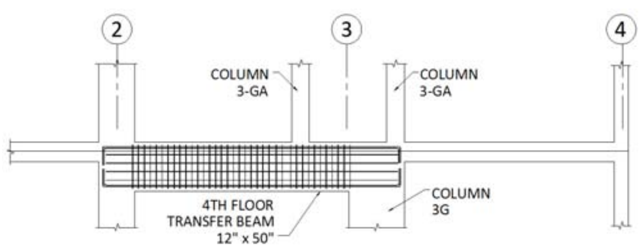


Figure 4 – Transfer Beam at Fourth Floor

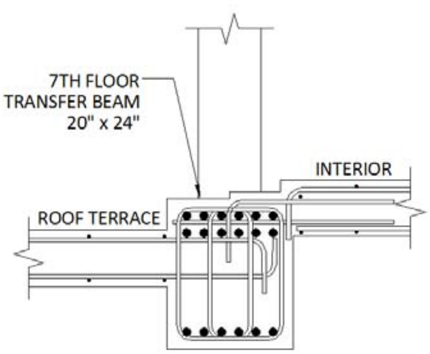


Figure 5 – Transfer Beam at Seventh Floor

The penthouse level and its roof are a great example of what can be achieved when using concrete. The dimensions of the penthouse are 23’-4”×44’-6” and it has a thickened 19” slab with #4@12 top bar reinforcement and #5@8 bottom bar reinforcement. A 44’-0” clear span is achieved with the support of the concrete shear walls to the east and two 28"×16" columns to the west. The loads from the two columns need to be transferred and a 32"×24" beam is used to direct these loads to nearby columns, one of which is only 10"×14". The roof above this long span structure is a combination of upturned beams, inclined piers, and two separate 8" slabs with #5@12 top and bottom spanning between its two supports (Figure 6). Located on the other side of the core is an enclosed elevated mechanical room. Additional loads due to the equipment and its surrounding 8" CMU walls will be applied at this level.

Figure 6 –Penthouse Roof Structure

Lateral System

The lateral system is a combination of 12" ordinary reinforced concrete shear walls (Figure 7). Elevations of these walls are located in Appendix A, which clearly defines all openings and the location of coupling beams throughout the height of the building. The typical horizontal reinforcement in these walls is #4@12 while the vertical reinforcement ranges from #4@12 to #8@6 depending on the level they are located on and which portion of the shear wall is being examined. The west shear wall is reinforced with #4@12 as the horizontal reinforcement and a range of vertical reinforcement from #4@12 to #7@12. All shear walls supporting the ground

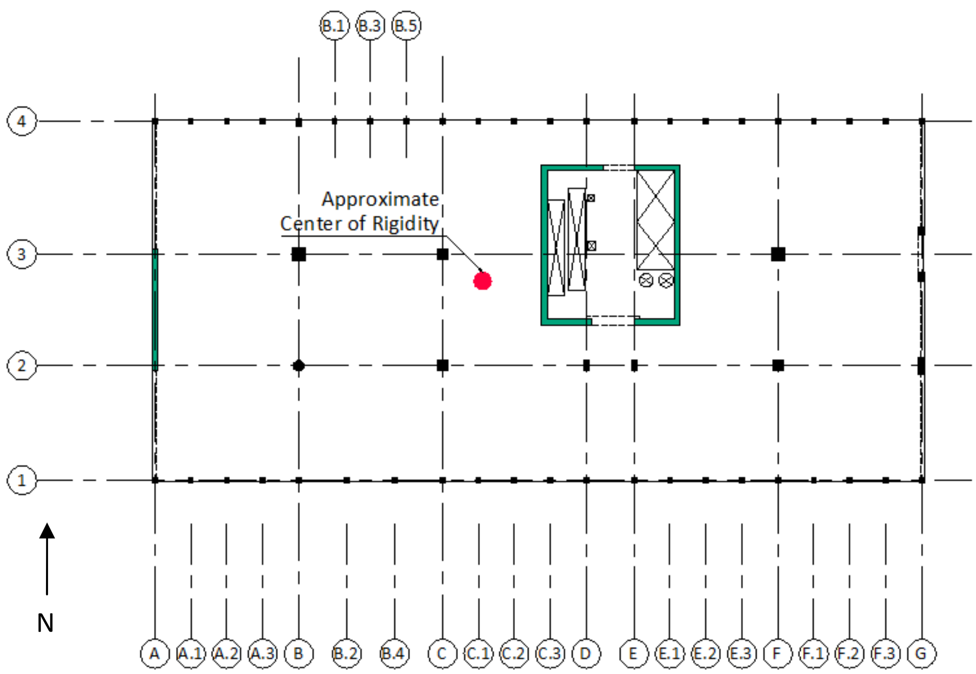


Figure 7 – Typical Plan with Lateral System Highlighted

floor to those supporting the fourth floor have concrete with a compressive strength $f'_c = 8$ ksi while those supporting the rest of the building have an $f'_c = 5$ ksi.

The presence of the west shear wall allows for the center of rigidity to move closer towards the middle of the plan. Because the core shear walls are not centralized within the building they draw the rigidity to the east. When the center of rigidity is not in line with the resultant lateral force there is eccentricity and moments due to torsion become a factor.

Loads

Gravity Loads

The determination of gravity loads by DCE was done using the New York City Building Code (NYCBC 2003), while American Society of Civil Engineers (ASCE) 7-05 was the main reference for this report. A different standard was used to comply with the requirements of AE Senior Thesis; ASCE 7-05 was the logical reference. Another note is that DCE chooses not to use live load reductions in their design. In order to keep the loading consistent, the reductions will be not be factored into the live loads determined by code. The loads that were determined from each reference as well as the design loads are noted in Table 1.

Table 1 - Gravity Loads

Description	NYCBC (2003)	ASCE 7-05	DCE Value	Design Value
DEAD (DL)				
Concrete	150 pcf	150 pcf	150 pcf	150 pcf
LIVE (LL)				
Condominiums & Townhouses	40 psf	40 psf	40 psf	40 psf
Corridor (first floor, main lobby)	100 psf	100 psf	100 psf	100 psf
Corridor (serving independent units)	40 psf	40 psf	40 psf	40 psf
*Exterior Balconies	60 psf	100 psf	60 psf	100 psf
SUPERIMPOSED (SDL)				
Finishes, MEP, Partitions	20-25 psf	20-25 psf	20 psf	25 psf
**Concrete Pavers	40 psf	40 psf	40 psf	40 psf
SNOW (S)				
***Snow	30 psf	21 psf	30 psf	30 psf

* In NYCBC, exterior balcony LL is 150% of adjacent areas. Therefore (40psf)x(1.5)=60psf.

** Superimposed load on 7th Floor and Penthouse terraces will be replaced as 40 psf over area.

*** Snow load calculations are located in appendix. Due to greater live load required on roof terraces, the roof live load on these areas will be 100 psf.

Wind Loads

Wind loads were determined using ASCE 7-05 Section 6.5 which describes Method 2-Analytical Procedure. The variables used in this analysis are located in Table 2 and these values are supported by base calculations located in Appendix B. The wind analysis done for this technical assignment varies from that done by DCE because of their use of the NYCBC. Rather than calculating the pressures at each floor, a simplified diagram found in the code was used that relates three distinct pressures at three distinct heights (Figure 8).

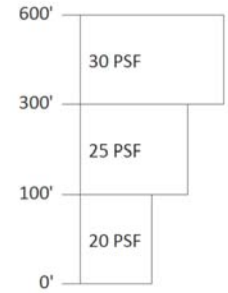


Figure 8 – Wind Load Diagram from NYCBC – RS 9-5

Table 2 - Wind Variables			(ASCE References)
Basic Wind Speed	V	110 mph	(Fig. 6-1)
Directionality Factor	k_d	0.85	(Table 6-4)
Importance Factor	I	1.00	(Table 6-1)
Exposure Category		B	(Sec. 6.5.6.3)
Topographic Factor	K_{zt}	1.00	(Sec. 6.5.7.1)
Velocity Pressure Exposure Coefficient evaluated at Height z	K_z	Varies	(Table 6-3)
Velocity Pressure at Height z	q_z	Varies	(Eq. 6-15)
Velocity Pressure at Mean Roof Height	q_h	27.909	(Eq. 6-15)
Equivalent Height of Structure	>	76.14'	(Table 6-2)
Intensity of Turbulence	$I_{>}$	0.261	(Eq. 6-5)
Integral Length Scale of Turbulence	$L_{>}$	422.8'	(Eq. 6-7)
Background Response Factor (East/West)	Q	0.85	(Eq. 6-6)
Background Response Factor (North/South)	Q	0.826	(Eq. 6-6)
Gust Effect Factor (East/West)	G	0.9097	(Eq. 6-4)
Gust Effect Factor (North/South)	G	0.828	(Eq. 6-4)
External Pressure Coefficient (Windward)	C_p	0.8	(Fig. 6-6)
External Pressure Coefficient (E/W Leeward)	C_p	-0.3	(Fig. 6-6)
External Pressure Coefficient (N/S Leeward)	Cp	-0.5	(Fig. 6-6)

Tables and calculations supporting the wind pressures in the both directions are also located in Appendix B. The winds coming in the north/south direction are those most prevalent at the site because two adjacent buildings are located on both the east and west sides of 40 Bond. The summation of windward story shear calculated by ASCE 7-05 is within 10 kips of that found by DCE, which insinuates that although there was a variation in pressures used, both methods provide reasonable answers and therefore either method can be used interchangeably. The reason behind these calculations being lower can be due to the fact that my windward pressures never exceed 25 psf and go below the lower limit of 20 psf provided by the NYCBC.

Although there are currently adjacent buildings blocking the wind on the lower levels, wind in the east/west direction must be examined in the event that these structures are absent at some point in the future and the full wind load is applied. The summation of windward story shear calculated by ASCE 7-05 is within 5 kips of that found by DCE. Similar conclusions to those stated for the north/south pressures can be applied here.

Seismic Loads

In order to calculate the seismic forces on 40 Bond, Chapters 11 and 12 were referenced from ASCE 7-05. DCE performed the seismic analysis based on the NYCBC, and there is a large difference between the base shears. After speaking with faculty in the Architectural Engineering department it was noted that such a great difference in possible when working between two separate codes/standards.

An assumption that was made in this analysis was that 40 Bond employed a rigid diaphragm which allowed for the use of the Equivalent Lateral Force Procedure found in Section 12.8 within ASCE 7-05. The variables used in this procedure are located in Table 3. The story shear, using these variables is then computed as,

$$V = C_s W$$

with W being the effective seismic weight as per Section 12.7.2.

Table 3 - Seismic Design Variables			(ASCE Reference)
Soil Classification		B	(Table 20.3-1)
Occupancy		II	(Table 1-1)
Importance Factor		1.00	(Table 11.5-1)
Structural System		Building Frame System: Ordinary Reinforced Concrete Shear Wall	(Table 12.2-1)
Spectral Response Acceleration, short	S_s	0.361	(USGS)
Spectral Response Acceleration, 1 s	S_1	0.07	(USGS)
Site Coefficient	F_a	1.00	(Table 11.4-1)
Site Coefficient	F_v	1.00	(Table 11.4-2)
MCE Spectral Response Acceleration, short	S_{MS}	0.361	(Eq. 11.4-1)
MCE Spectral Response Acceleration, 1 s	S_{M1}	0.07	(Eq. 11.4-2)
Design Spectral Acceleration, short	S_{DS}	0.241	(Eq. 11.4-3)
Design Spectral Acceleration, 1 s	S_{D1}	0.047	(Eq. 11.4-4)
Seismic Design Category	S_{DC}	B	(Table 11.6-2)
Response Modification Coefficient	R	5	(Table 12.2-1)
Approximate Period Parameter	C_t	0.02	(Table 12.8-2)
Building Height (above grade)	h_n	134.3 ft	Above Grade
Approximate Period Parameter	x	0.75	(Table 12.8-2)
Calculated Period Upper Limit Coefficient	C_u	1.70	(Table 12.8-1)
Approximate Fundamental Period	T_a	0.789 s	(Eq. 12.8-7)
Fundamental Period	T	1.34 s	(Sec. 12.8.2)
Long Period Transition Period	T_L	6.00 s	(Fig. 22-15)
Seismic Response Coefficient	C_s	0.012	(Eq. 12.8-2)
Structure Period Exponent	k	1.42	(Sec. 12.8.3)

The NYCBC makes use of different variables and equations in comparison to ASCE 7-05. In most cases it was clear that certain variables were directly related to the other and the only difference being in the coefficients used to describe them. An example of this was Site Class S_1 in the NYCBC which referred to materials with shear wave velocity greater than 2500 ft/s. This same description was used for Site Class B within ASCE 7-05. There were also instances were coefficients were not comparable, such as the response modification factor. In the NYCBC, R_w=8 for ordinary reinforced concrete shear walls within the building frame system, while R=5 in ASCE 7-05. The variables needed to calculate base shear according to the building code are located in Table 4. The calculation for base shear according to the NYCBC is,

$$V = \frac{ZIC}{R_W} W$$

with W equal to the effective building weight.

Table 4 - Seismic Design Variables			(NYCBC Reference)
Seismic Zone Factor	Z	0.15	(RS 9-6)
Importance Factor	I	1	(RS 9-6)
Site Coefficient for S_1 Soil	S	1.00	(RS 9-6)
Response Modification Coefficient	R_w	8.00	(RS 9-6)
Overall Building Height	hn	152'	Above and Below Grade
Coefficient	C	1.47	(RS 10-5c)

To adhere to the requirement of using ASCE 7-05, the story shears and overturning were calculated using this standard. To ensure, however, that the most stringent loads were accounted for, calculations were also done according to the NYCBC. These values were then used for the analysis and confirmation design of the lateral system required in this technical assignment. All supporting calculations and tables are located in Appendix B.

ETABS Model

ETABS is a computer modeling and analysis program developed by Computers & Structures, Inc. For the use in this technical assignment, the building's lateral system and diaphragms were the only components modeled (Figure 9). This simplification required the gravity loads to be applied as additional area masses to the diaphragms. The mass of each of the shear walls was incorporated into membranes that defined each portion of the wall. These walls were meshed into areas with a maximum dimension of 24”×24” that allowed those walls that were connected at the core to act together as a rigid unit. Also, for simplicity, the coupling beams were modeled as wall elements as opposed to line elements. The results from this model were compared to the values produced by hand calculations of the center of mass, center of rigidity, and story displacements. Additional information to the overall building drift and controlling load cases were also pulled from the model.

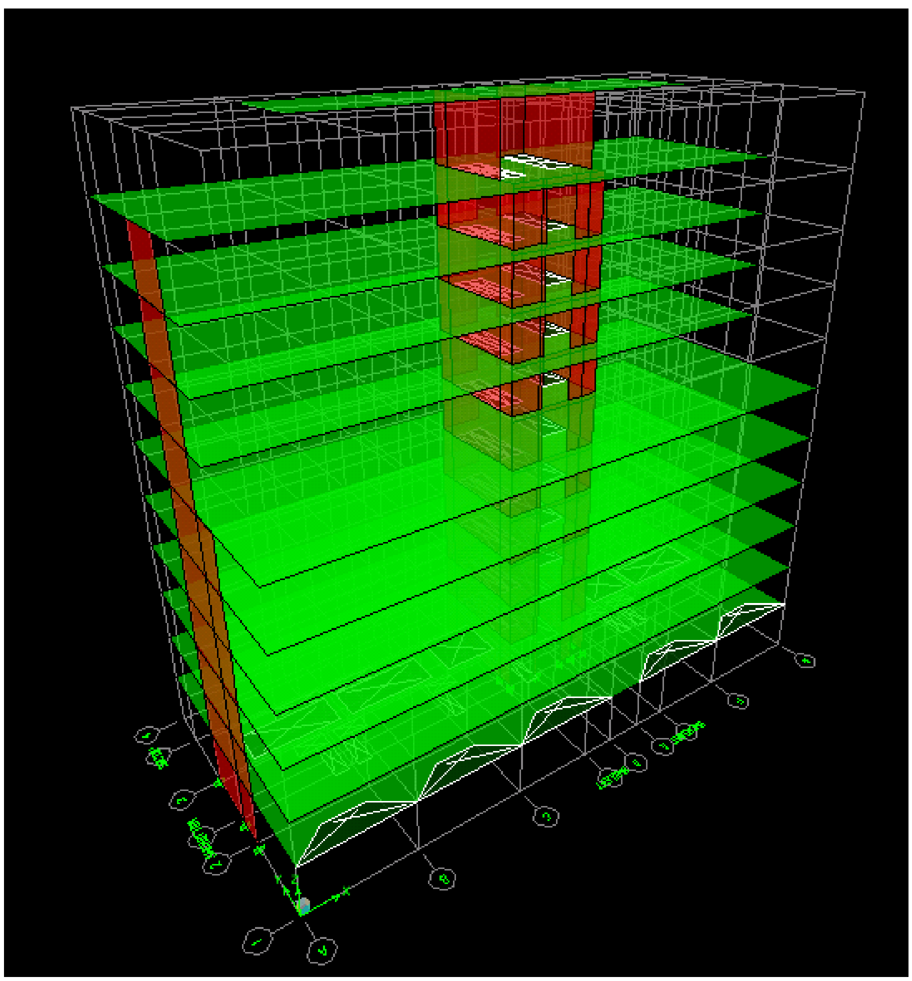


Figure 9 – ETABS Computer Model

Load Considerations

Load Combinations

The list below shows the various load cases specified by ASCE 7-05 Section 2.3 for factored loads using strength design.

$1.4(D + F)$

$1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$

$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$

$1.2D + 1.6W + L + 0.5(L_r \text{ or } S \text{ or } R)$

$1.2D + 1.0E + L + 0.2S$

$0.9D + 1.6W + 1.6H$

$0.9D + 1.0E + 1.6H$

These combinations were included in the ETABS model and after looking into drift, story shears and displacements it was determined that the controlling load case in the north/south direction was $1.2D + 1.6W + L + 0.5(L_r \text{ or } S \text{ or } R)$ and in the east/west direction $0.9D + 1.0E + 1.6H$. The wind in the north/south direction controls because of the large surface area along that face, which produces higher forces. The east and west facades are less than half the surface area seen on the north and south faces so it seems quite reasonable that seismic controls in that direction.

Load Path and Distribution

As the lateral forces come in contact with the building, the loads need a means of travelling through the structure and into the ground. The load path is assumed to be controlled by the concept of relative stiffness. Those members that are the most rigid draw the forces to them. As a result the loads are transmitted through the diaphragms, to the shear walls, and then down into the mat foundation. After completing this assignment, it is clear that the shear walls with minimal assistance from the slabs resist the lateral forces, while the columns only serve to transmit gravity loads.

40 Bond has a shear wall located along the west face of the building in addition to a shear wall core. Figure 10 shows the numbered system assigned to each wall to better reference exactly which shear walls are being discussed throughout this paper. Although all the shear walls maintain the same thickness of 12" throughout their heights, they do vary in length and are located different distances from the center of rigidity of the building. These things all affect the rigidity of the walls which in turn affects the relative stiffness of each element. Tables located in Appendix C define the rigidities of Walls 1-3 (parallel to the north/south lateral forces) and of

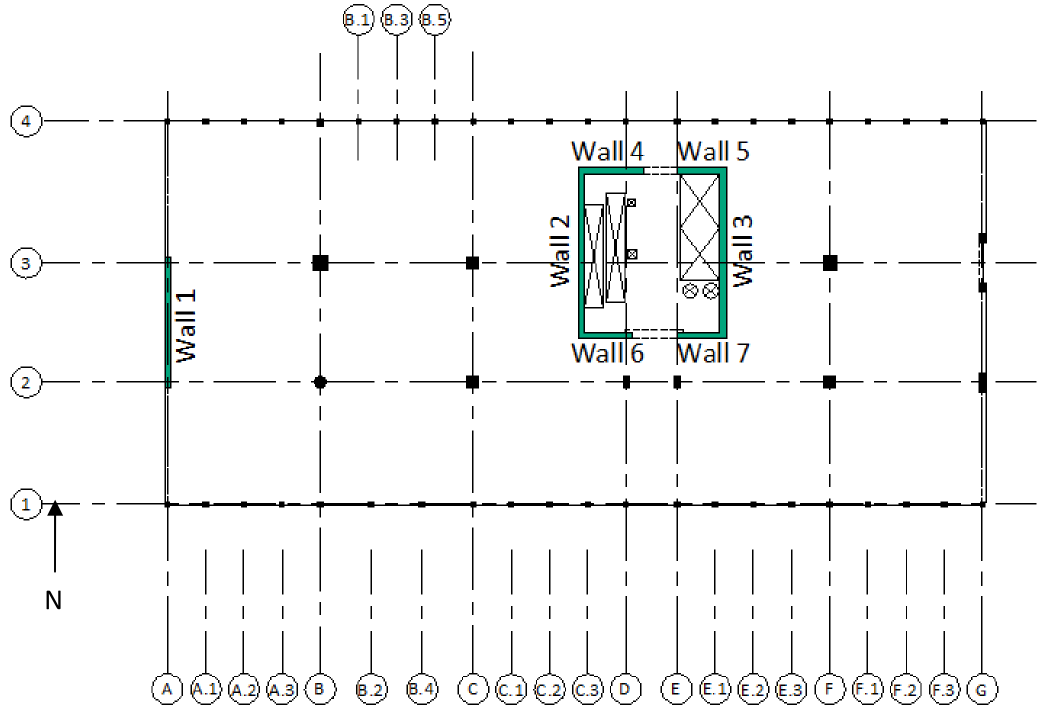


Figure 10 – Numbered Shear Walls

Walls 4-7 (parallel to the east/west lateral forces) that were calculated using the following equation:

$$R = \frac{Et}{4(\frac{h}{L})^3 + 3(\frac{h}{L})}$$

The rigidity values were then used to determine the center of rigidity on each floor which can be calculated as:

$$Center\ of\ Rigidity = \frac{\Sigma(R)(distance\ between\ element\ and\ the\ origin)}{\Sigma R}$$

The values of both the center of mass and center of rigidity are located in Table 5. The coordinates found by hand calculations and the ETABS output are put in this one table to show that the results are comparable. The center of rigidity values taken from the ETABS model suggest that the diaphragms are being considered in the determination of rigidity, as opposed to the hand calculations that are assuming that only the shear walls are to be taken into account. For the use in this technical assignment, the values produced by hand calculations will be those used whenever the center of mass and center of rigidity are needed.

Table 5								
	Center of Rigidity				Center of Mass			
	Etabs Output		Hand Calculations		Etabs Output		Hand Calculations	
	X	Y	X	Y	X	Y	X	Y
Floor 2	-	-	706.3633	642.197	800	376.250	800	376.25
Floor 3	765.475	637.376	730.124	566.123	800.000	376.250	800	376.25
Floor 4	803.596	580.913	744.207	550.703	800.000	376.250	800	376.25
Floor 5	823.685	567.171	751.278	543.547	800.000	376.250	800	376.25
Floor 6	835.359	561.839	755.113	539.356	800.000	376.250	800	376.25
Floor 7	843.340	559.478	757.485	536.020	800.000	376.250	800	376.25
Floor 8	848.189	559.509	800.786	535.059	800.000	496.250	800	496.25
Floor 9	853.058	560.267	833.890	534.052	800.000	496.250	800	496.25
Floor 10	858.271	561.266	857.899	533.013	800.000	496.250	800	496.25
Penthouse	864.205	561.852	876.731	530.580	800.000	496.250	800	496.25
Penthouse Roof	882.870	608.262	951.500	655.000	865.000	521.500	865	521.50

The rigidity of the walls is also used to determine the relative stiffness, which dictates what percentage of the lateral force is distributed it each wall. This is simply calculated as:

$$Relative\ Stiffness = \frac{R}{\Sigma R}$$

Table 6 gives the values found for all seven walls at every level. These values can then be directly applied to the loads at each floor to determine how much each wall will receive. Also, it is important to note that because the length of the walls change as they continue up the building, either due to setbacks or the addition of openings, the relative stiffness of one wall is not consistent its entire height. As the contribution of each wall changes, so does the relative stiffness of each member resisting the force in the specified direction.

Table 6 - Relative Stiffness (%)							
	North-South Force			East-West Force			
	Wall 1	Wall 2	Wall 3	Wall 4	Wall 5	Wall 6	Wall 7
Floor 1	25.93	37.04	37.04	64.90	31.14	0.09	3.87
Floor 2	23.41	38.29	38.29	50.60	21.88	20.20	7.32
Floor 3	21.92	39.04	39.04	47.71	20.00	21.67	10.62
Floor 4	21.18	39.41	39.41	46.30	19.19	22.16	12.35
Floor 5	20.77	39.61	39.61	45.46	18.74	22.33	13.47
Floor 6	20.52	39.74	39.74	44.77	18.40	22.63	14.21
Floor 7	15.94	42.03	42.03	44.58	18.29	22.50	14.64
Floor 8	12.44	43.78	43.78	44.37	18.18	22.37	15.07
Floor 9	9.90	45.05	45.05	44.15	18.08	22.25	15.52
Floor 10	7.91	46.05	46.05	43.63	17.85	22.66	15.86
Penthouse	0.00	50.00	50.00	0.00	100.00	0.00	0.00

Torsion

Torsion is present when the center of mass and the center of rigidity are not in the same location. Moments are produced by this eccentricity and torsional shear becomes an additional force to account for. Torsional shear will be discussed further when shear is reviewed.

There are two separate moments to take into consideration when looking at torsion in buildings with rigid diaphragms, like those seen in 40 Bond, according to ASCE 7-05 Section 12.8.4. First there is the inherent moment, M_t, which is due to eccentricity between the center of rigidity and the center of mass. Because of the rigidity of the slab there is also an accidental moment, M_{ta}, which needs to be accounted for in addition to the inherent moment. This moment is "caused by the assumed displacement of the center of mass each way from the actual location by a distance equal to 5% of the dimension of the structure perpendicular to the direction of the applied force." The values of the torsion, produced by forces in both directions, can be seen in Table 7.

Table 7 - Overall Building Torsion

	North/South Direction				East/West Direction			
	Factored Lateral Force (k)	M_t (ft-k)	M_{ta} (ft-k)	$M_{t,tot}$ (ft-k)	Factored Lateral Force (k)	M_t (ft-k)	M_{ta} (ft-k)	$M_{t,tot}$ (ft-k)
Floor 2	91.13	-711.09	607.38	-103.71	25.23	6710.65	79.17	6789.82
Floor 3	108.12	-629.59	720.63	91.04	64.11	12172.57	201.14	12373.71
Floor 4	99.70	-463.56	664.52	200.96	45.37	7914.97	142.35	8057.32
Floor 5	98.37	-399.41	655.66	256.24	39.52	6611.39	123.99	6735.38
Floor 6	96.60	-361.34	643.83	282.50	33.96	5539.57	106.56	5646.13
Floor 7	100.84	-357.26	672.08	314.82	42.67	6816.82	133.87	6950.69
Floor 8	92.61	6.06	617.23	623.30	33.13	1285.88	103.96	1389.84
Floor 9	89.95	254.02	599.50	853.52	25.52	964.52	80.05	1044.57
Floor 10	87.29	421.15	581.77	1002.91	18.48	679.55	58.00	737.54
Penthouse	76.66	490.19	510.94	1001.13	12.63	433.74	39.64	473.39
Penthouse Roof	84.73	610.79	564.75	1175.53	5.54	739.09	17.37	756.46
			Total	5698.25			Total	50954.85

Shear

Direct Shear

Direct shear is that which is caused by the lateral forces acting on a building that are distributed to the shear walls. To determine these values simply multiply the story shear by the relative stiffness of each member. The direct shears that will be applied to each wall can be found in Tables 8 and 9.

Table 8 - North/South Direct Shear

Load Combination 1.2D + 1.6L + L + 0.5Lr	Force (k)	Factored Force (k)	Distributed Force (k)		
			Wall 1	Wall 2	Wall 3
Floor 2	56.96	91.13	23.63	33.75	33.75
Floor 3	67.58	108.12	25.32	41.40	41.40
Floor 4	62.31	99.70	21.86	38.92	38.92
Floor 5	61.48	98.37	20.83	38.77	38.77
Floor 6	60.37	96.60	20.06	38.27	38.27
Floor 7	63.02	100.84	20.69	40.07	40.07
Floor 8	57.88	92.61	14.76	38.92	38.92
Floor 9	56.22	89.95	11.19	39.38	39.38
Floor 10	54.55	87.29	8.64	39.32	39.32
Penthouse	47.91	76.66	6.06	35.30	35.30
Penthouse Roof	52.96	84.73	0.00	42.37	42.37

Table 9 - East/ West Direct Shear

Load Combination 0.9D + 1.0E + 1.6H	Force (k)	Factored Force (k)	Distributed Forces (k)			
			Wall 4	Wall 5	Wall 6	Wall 7
Floor 2	25.23	25.23	16.38	7.86	0.02	0.98
Floor 3	64.11	64.11	32.44	14.03	12.95	4.69
Floor 4	45.37	45.37	21.64	9.08	9.83	4.82
Floor 5	39.52	39.52	18.30	7.58	8.76	4.88
Floor 6	33.96	33.96	15.44	6.36	7.59	4.57
Floor 7	42.67	42.67	19.10	7.85	9.65	6.06
Floor 8	33.13	33.13	14.77	6.06	7.45	4.85
Floor 9	25.52	25.52	11.32	4.64	5.71	3.85
Floor 10	18.48	18.48	8.16	3.34	4.11	2.87
Penthouse	12.63	12.63	5.51	2.26	2.86	2.00
Penthouse Roof	5.54	5.54	0.00	5.54	0.00	0.00

Torsional Shear

In addition to direct shear there is also a shear force present when torsion is produced by the building. To determine this value the following equation was used:

$$T = \frac{V_{tot} e d_i R_i}{J}$$

- V_{tot} = story shear
- e = distance from the center of mass to the center of rigidity
- d_i = distance from element to the center of rigidity
- R_i = relative stiffness of the element
- J = torsional moment of inertia = $= \Sigma\left(R \times {d_i}^2\right)$

As an example, the torsional shear was computed for the shear wall supporting Floor 6 and can be found in Table 10.

Table 10 - Torsional Shear in Shear Wall Supporting Floor 6							
		Factored Story Shear V_{tot} (k)	Relative Stiffness R_i	Distance from COM to COR e (inches)	Distance from Wall X to COR d_i (inches)	$(R_i)(d_i^2)$	Torsional Shear (k)
Wall 1	N/S	687.38	0.208	44.89	749.11	116555.07	25.76
Wall 2	N/S	687.38	0.396	44.89	57.39	1304.46	3.76
Wall 3	N/S	687.38	0.396	44.89	335.39	44555.08	21.99
Wall 4	E/W	250.86	0.455	163.11	115.64	6079.62	11.54
Wall 5	E/W	250.86	0.187	163.11	115.64	2506.21	4.76
Wall 6	E/W	250.86	0.223	163.11	207.36	9601.11	10.16
Wall 7	E/W	250.86	0.135	163.11	207.36	5791.62	6.13
Torsional Moment of Inertia J = Σ $(R_i)(d_i^2)$=						186393.18	

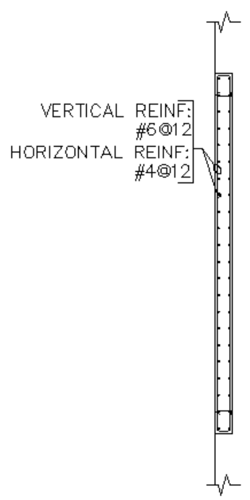
Shear Strength Check

In order to confirm the shear strength of the shear walls, a check must be done that takes into account both the torsional and direct shears being applied. According to ACI 318-08 Section 21.9.4.1 the shear strength of a reinforced concrete shear walls is defined as:

$$V_n = A_{cv}\left[\left(\alpha_c \lambda \sqrt{f'_c}\right) + \left(\rho_t f_y\right)\right]$$

The hand calculations of a strength check done at the shear walls supporting Floor 6 can be found in Appendix D. Each wall was well within the capacity determined with the above equations which can be seen in Table 11. The original shear wall details that were used to confirm the reinforcement and spacing are seen in Figure 11.

Table 11 - Shear Wall Strength Check (Supporting Floor 6)												
Floor 6	Direct Shear (k)	Torsional Shear (k)	V_u (k)	Vertical Reinf.	Spacing (in)	Length (in)	Thickness (in)	A_{cv} (in^2)	α_c	ρ_t	ϕV_n (k)	
Wall 1	81.41	25.76	107.17	(2) #6	12	256	12	3072	2	0.0061	1170.63	OK
Wall 2	273.63	3.76	277.39	(2) #5	12	323	12	3876	2	0.0043	1162.08	OK
Wall 3	273.63	21.99	295.62	(2) #4	12	323	12	3876	2	0.0028	895.61	OK
Wall 4	74.31	11.54	85.85	(2) #6	12	122	12	1464	2	0.0061	557.88	OK
Wall 5	36.05	4.76	40.80	(2) #5	8	90.5	12	1086	2	0.0065	430.81	OK
Wall 6	37.38	10.16	47.54	(2) #5	12	99	12	1188	2	0.0043	356.18	OK
Wall 7	24.21	6.13	30.34	(2) #5	8	90.5	12	1086	2	0.0065	430.81	OK



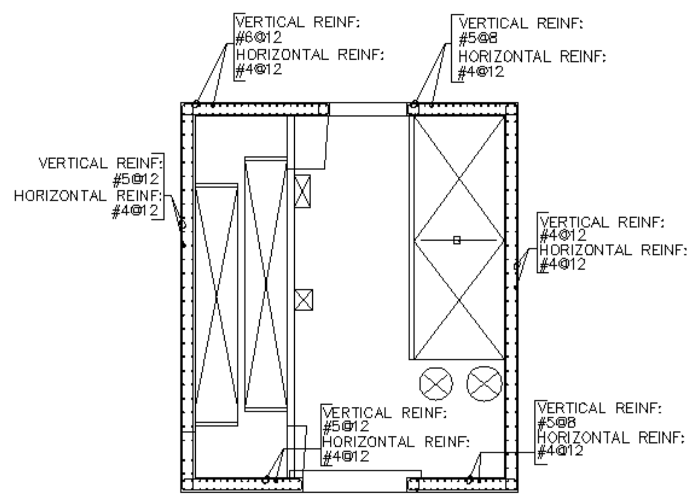


Figure 11 – Details of shear wall supporting Floor 6

Drift and Displacement

Drift is a serviceability consideration in building design that in inversely proportionate to rigidity. The overall building drift should be limited as much as possible, especially in the case of 40 Bond, because the building is attached to adjacent buildings on either side. The drift has been limited to 1/400th of the overall building height which originated from the *Structural Engineering Handbook* (1968) by Gaylord and Gaylord. In the case of 40 Bond, the drift limit is:

$$\Delta_{limit} = \left({}^{1612"}/_{400}\right) = 4.03"$$

The building drifts taken from the ETABS model describe a drift in the x-direction (due to east/west forces) = 1.1422" which is well below 4.03". Similarly, the drift in the y-direction (due to north/south forces) = 1.0474" is within the limits enforced.

Each floor can be examined independently to obtain an approximate determination of the displacements and story drifts. This was done by hand calculations using the following equation:

$$\Delta_{cantilever} = \Delta_{flexural} + \Delta_{shear} = \frac{Ph^3}{3E_cI} + \frac{1.2Ph}{E_rA}$$

The actual calculations as well as tables looking at the story drift and displacement of Walls 1-3 can be found in Appendix E. Note that the modulus of elasticity and the modulus of rigidity change values once the shear wall supporting Floor 5 is examined. The reason for this is because the concrete strength is f'_c=8000 psi for the walls supporting Floors 1-4 and then f'_c=5000 psi for the walls supporting the remaining slabs. Also, it is important to recognize that the displacements determined for Walls 2 and 3 are different than the expected values. The reason for this is because this calculation is assuming the wall is reacting to the force independently of all other walls. In actuality, however, Walls 4-7 serve as flanges for Walls 2 and 3, and will help to resist some of this movement. The above calculation was done solely as an approximation. To compute story drifts and displacements of shear walls working together by hand is beyond the scope of this technical assignment, and because of this the values therefore cannot be directly compared to the ETABS model.

Overturning

Overturning moments are due to the presence of the lateral forces and can be found by multiplying the story forces by their mid-heights. This was done with the north/south wind forces and the east west seismic forces with values shown in Table 12. These moments are transformed into axial loads as they are transmitted through the lateral elements and into the 30" mat slab foundation, which would experience the most impact from the overturning moment. To do a rough estimate of whether or not overturning would be an issue in 40 Bond, the stresses due to these lateral loads were examined and compared to the stresses due to the dead load (self weight) of the building which serves to counteract the overturning. Calculations supporting this estimate can be found in Appendix F. Because the stresses produced by the lateral forces are only a small fraction of that produced by the self weight of the structure, the overturning will have a minimal effect on the foundation. Due to the presence of the moments, however, it is expected that there will be a slight increase of force around the perimeter with a small uplift force on the windward sides and a slight downward force on the leeward sides.

Table 12 - Overturning

Floor	Height Above Ground-z (ft)	Story Height (ft)	N/S Wind Forces		E/W Seismic Forces	
			Lateral Force F_x (k)	Moment Total (ft-k)	Lateral Force F_x (k)	Moments M_x (ft-k)
PH Roof	134.30	14.75	56.96	7227.76	10.86	1378.11
PH	119.55	12.66	67.58	7649.64	31.50	3566.31
10	106.89	11.83	62.31	6290.03	25.33	2556.85
9	95.06	11.83	61.48	5478.76	25.22	2247.37
8	83.23	11.83	60.37	4665.74	25.22	1949.01
7	71.40	12.58	63.02	4100.92	37.73	2455.33
6	58.82	11.83	57.88	3059.54	36.55	1931.96
5	46.99	11.83	56.22	2306.59	36.36	1491.67
4	35.16	11.83	54.55	1592.98	36.66	1070.44
3	23.33	10.83	47.91	856.20	39.99	714.69
2	12.50	12.5	52.96	330.99	35.69	223.09
1	0	0	0.00	0.00	4.58	0.00
		Total:	641.25	43559.14	345.70	19584.83

Conclusion

Once adjusting the values found in the first technical assignment, the lateral forces were applied to 40 Bond. These loads were then factored according to ASCE 7-05 load combinations for strength design. With output taken from ETABS, it was determined that the combination of $1.2D + 1.6W + L + 0.5(L_r \ or \ S \ or \ R)$ controlled in the north/south direction, while $0.9D + 1.0E + 1.6H$ controlled in the east/west direction. A reason for wind controlling in one direction and seismic controlling in the other is most likely due to the large surface area of the north and south facades. This area, which is more than twice as large as the east and west faces, resulted in larger wind forces in that direction.

Although ETABS was used as a reference and in some comparisons to verify that the model and hand calculations were providing similar and reasonable results, the values computed by hand were those used in all subsequent calculations. There were two reasons behind this. First, it was concluded after finding the center of rigidity that the model was taking the slab into account as a member providing lateral resistance rather than acting as a null diaphragm. Secondly, because this was the first encounter using ETABS to model a structure, there was some uncertainty as to whether or not everything was input with all the proper assumptions. Therefore, to ensure consistency and to verify that only the shear walls were acting to resist lateral forces, hand calculations were done. Anything that was beyond the scope of hand calculations was taken from the ETABS model.

This report confirms that looking to the shear walls alone was a reasonable assumption. There was torsion due to the eccentricity between the center of mass and the center of rigidity that added torsional shear to the walls. Shear strength checks were done including both the direct and torsional shear and it was deduced that the thickness, length and reinforcement were designed to adequately resist the total shear. Overall building drift, as determined by ETABS output, was within the limit of H/400. The story drifts and displacements that were calculated by hand were within a reasonable range, but they neglected the effect of the core working as one unit. Because of this the values are only an approximation and are most likely smaller. Overturning is present due to the lateral loads, but a stress check concluded that the self weight of the building can do most of the work to resist this. A more complex model and additional calculations will follow when the second portion of senior thesis begins. At this stage of analysis, however, it was determined that the shear walls were satisfactorily designed to resist various combinations of loading.

Appendix A

Shear Wall Elevations

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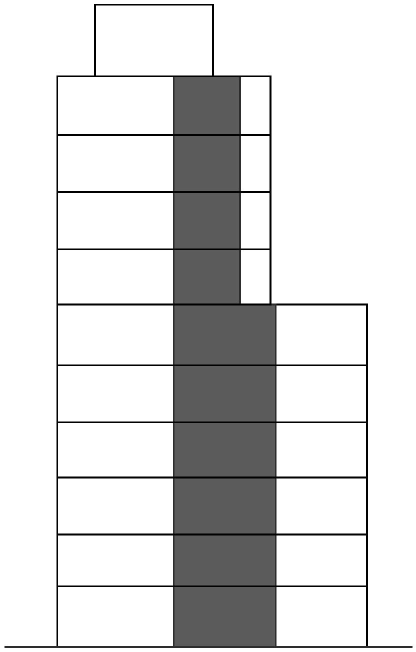

Figure 12- Elevation of Wall 1

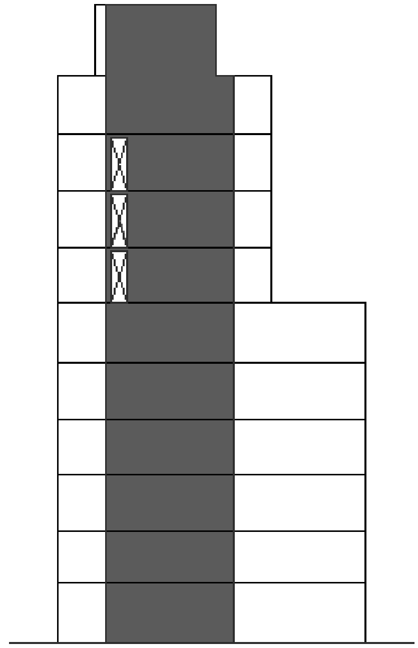

Figure 13- Elevation of Wall 2

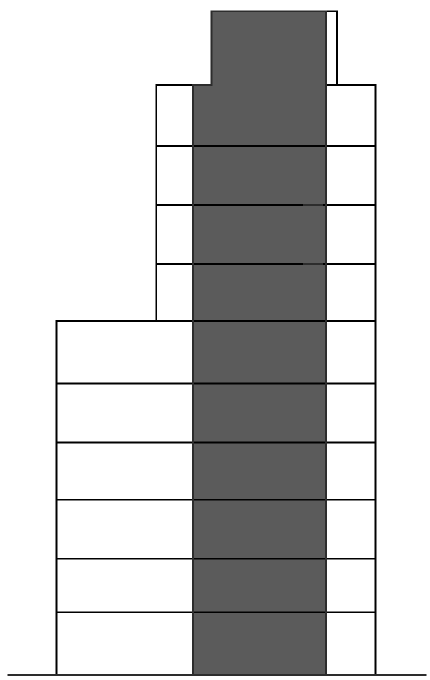

Figure 14- Elevation of Wall 3

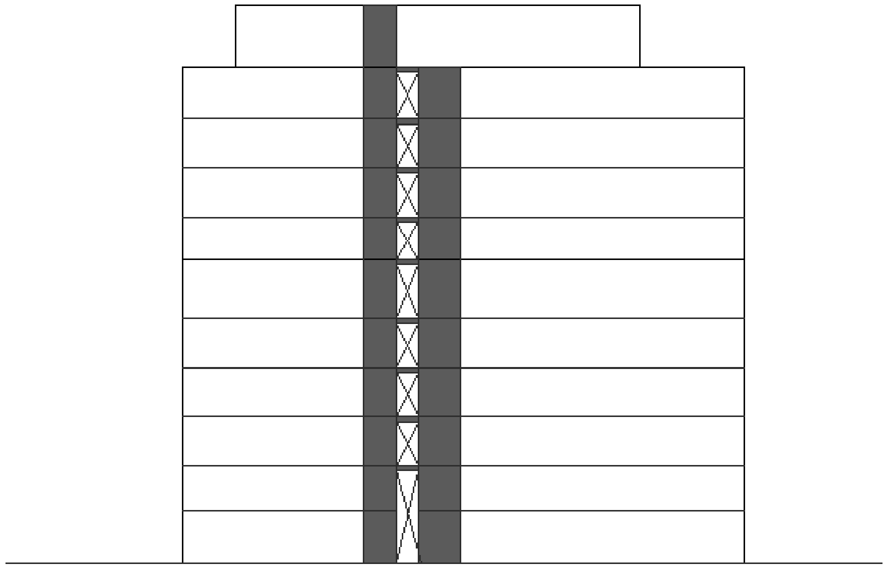

Figure 15 – Elevation of Walls 4 and 5

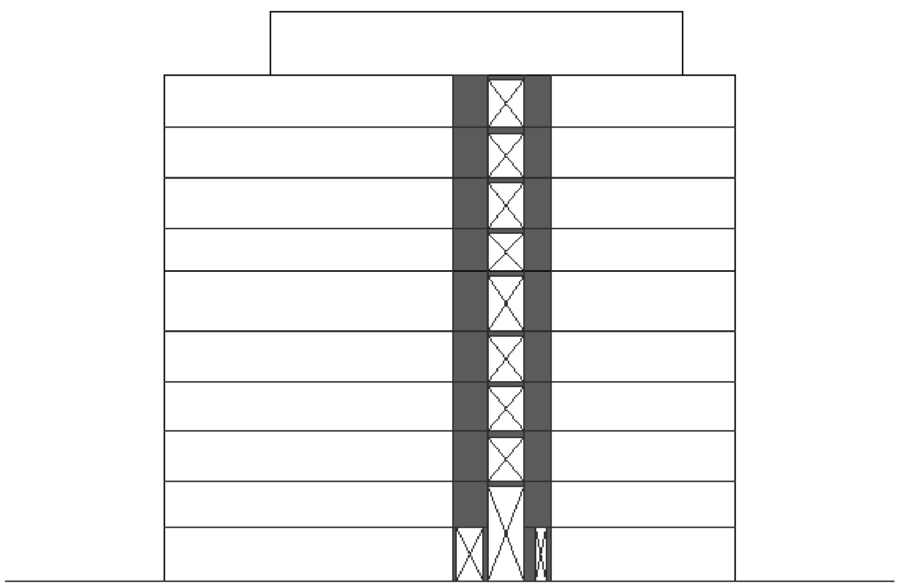

Figure 16 – Elevation of Walls 6 and 7

Appendix B

Loads

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Wind Loads

Table 13 - Wind Loads (North/South Direction) B=134'-4", L=64'-8"													
Floor	Height Above Ground-z (ft)	Story Height (ft)	K_z	q_z	Wind Pressure (psf)		Total Pressure (psf)	Force (k) of Windward Only	Force (k) of Total Pressure	Story Shear Windward (k)	Story Shear Total (k)	Moment Windward (ft-k)	Moment Total (ft-k)
					Windward	Leeward							
PH Roof	134.30	14.75	1.08	28.44	23.86	-16.57	40.43	33.61	56.96	33.61	56.96	4265.08	7227.76
PH	119.55	12.66	1.04	27.38	23.16	-16.57	39.74	39.39	67.58	73.00	124.53	4458.98	7649.64
10	106.89	11.83	1.01	26.59	22.64	-16.57	39.21	35.98	62.31	108.98	186.85	3631.46	6290.03
9	95.06	11.83	0.98	25.80	22.12	-16.57	38.69	35.14	61.48	144.12	248.33	3131.76	5478.76
8	83.23	11.83	0.94	24.75	21.42	-16.57	37.99	34.04	60.37	178.16	308.70	2630.33	4665.74
7	71.40	12.58	0.9	23.70	20.72	-16.57	37.29	35.02	63.02	213.17	371.73	2278.44	4100.92
6	58.82	11.83	0.85	22.38	19.85	-16.57	36.42	31.54	57.88	244.71	429.61	1667.31	3059.54
5	46.99	11.83	0.79	20.80	18.80	-16.57	35.38	29.88	56.22	274.59	485.83	1225.93	2306.59
4	35.16	11.83	0.73	19.22	17.76	-16.57	34.33	28.22	54.55	302.81	540.38	823.91	1592.98
3	23.33	10.83	0.65	17.11	16.36	-16.57	32.93	23.80	47.91	326.61	588.29	425.33	856.20
2	12.50	12.5	0.57	15.01	14.97	-16.57	31.54	25.13	52.96	351.74	641.25	157.05	330.99
1	0	0	0	0	0	0	0	0	0	351.74	641.25	0.00	0.00
Σ Story Shear (Windward) =		**351.74 k**	**Σ Story Shear (Total) =**		**641.25 k**	**Σ Moment (Windward) =**		**24695.58 ft-k**		**Σ Moment (Total) =**		**43559.14 ft-k**	
Σ DCE Story Shear (Windward) = 360 k						**Σ DCE Moment (Windward) = 30200 ft-k**							

Table 14 - Wind Loads (East/West Direction) B=64'-8", L=134'-4"													
Floor	Height Above Ground-z (ft)	Story Height (ft)	K_z	q_z	Wind Pressure (psf)		Total Pressure (psf)	Force of Windward Only (k)	Force of Total Pressure (k)	Story Shear Windward (k)	Story Shear Total(k)	Moment Windward (ft-k)	Moment Total (ft-k)
					Windward	Leeward							
PH Roof	134.30	14.75	1.08	28.44	25.72	-12.64	38.36	8.85	13.20	8.85	13.20	1123.09	1772.74
PH	119.55	12.66	1.04	27.38	24.95	-12.64	37.59	13.50	20.35	22.35	33.55	1528.70	2432.29
10	106.89	11.83	1.01	26.59	24.38	-12.64	37.02	12.33	18.72	34.68	52.27	1244.43	2001.07
9	95.06	11.83	0.98	25.80	23.80	-12.64	36.44	12.04	18.43	46.72	70.70	1072.67	1751.97
8	83.23	11.83	0.94	24.75	23.04	-12.64	35.68	11.65	18.04	58.37	88.74	900.31	1501.68
7	71.40	12.58	0.9	23.70	22.27	-12.64	34.91	18.11	28.40	76.49	117.13	1178.71	2027.49
6	58.82	11.83	0.85	22.38	21.31	-12.64	33.95	16.30	25.97	92.79	143.11	861.71	1527.58
5	46.99	11.83	0.79	20.80	20.16	-12.64	32.80	15.42	25.09	108.21	168.20	632.77	1179.02
4	35.16	11.83	0.73	19.22	19.01	-12.64	31.65	14.54	24.21	122.75	192.41	424.65	851.28
3	23.33	10.83	0.65	17.11	17.48	-12.64	30.12	12.24	21.09	134.99	213.50	218.73	492.06
2	12.50	12.5	0.57	15.01	15.95	-12.64	28.59	12.89	23.10	147.88	236.60	80.55	288.81
1	0	0	0	0	0	0	0	0	0	147.88	236.60	0.00	0.00
Σ Story Shear (Windward) =		**147.88 k**	**Σ Story Shear (Total) =**		**236.60 k**	**Σ Moment (Windward) =**		**9266.33 ft-k**		**Σ Moment (Total) =**		**15825.98 ft-k**	
Σ DCE Story Shear (Windward) = 150 k						**Σ DCE Moment (Windward) = 9400 ft-k**							

TECH REPORT
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WIND CALCULATIONS

METHOD 2: ANALYTICAL PROCEDURE (6.6)

K_z (Table 3)

Level	Height	K_z (via interpolation)
1	0	0
2	12.5'	0.57
3	23.33'	0.65
4	35.16'	0.73
5	46.99'	0.79
6	58.82'	0.85
7	71.40'	0.90
8	83.23'	0.94
9	95.06'	0.98
10	106.89'	1.01
PH	119.55'	1.04
PH Roof	134.3'	1.08

$q_z = 0.00256\,K_z\,K_{zt}\,K_d\,V^2 I = 0.00256\,K_z(1.0)(0.85)(110\,mph)^2\,($

varies at levels

$\bar{z} = 0.6h = \left(\frac{134.3+119.55}{2}\right)(0.6) = 76.14' \geq z_{min} = 30'$ OKAY!

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = (0.30)\left(\frac{33}{76.14}\right)^{1/6} = \boxed{0.261}$

$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = (320)\left(\frac{76.17}{33}\right)^{1/3} = \boxed{422.8'}$

$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$

N/S $B = 134.33'$; $L = 64.66'$

E/W $B = 64.66'$; $L = 134.3$

$\boxed{Q_{N/S} = 0.826}$

$\boxed{Q_{E/W} = 0.850}$

$G = 0.925\left(\frac{1+1.7\,g_Q I_{\bar{z}}\,Q}{1+1.7\,g_v I_{\bar{z}}}\right)$ w/ $g_Q, g_v = 3.4$

$\boxed{G_{N/S} = 0.828}$

$\boxed{G_{E/W} = 0.9097}$

$q_h = 0.00256\,(1.06)(1.0)(0.85)(110)^2(1.0) = \boxed{27.90'}$

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WIND CALCULATIONS (cont)

Pressure Coefficient C_p (Fig 6-6)

- NORTH | SOUTH
  - windward → $C_p = 0.8$
  - Leeward → $L/B = 0.481$
  - $C_p = -0.5$
- EAST | WEST
  - wind ward → $C_p = 0.8$
  - Leeward → $L/B = 2.007$
  - $C_p = -0.3$

Pressure $p_z = q_z GC_p - q_h(GC_{pi})$ windward
$p_h = q_h GC_p - q_h(GC_{pi})$ Leeward

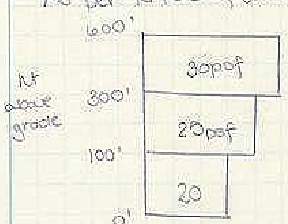
w/ $GC_{pi} = +0.18, -0.18$ for enclosed bldgs (Fig. 6-5)

- NORTH | SOUTH
  - windward $p_z = (q_z)(0.828)(0.8) - 27.909(-0.18)$
    $= (q_z)(0.6624) + 5.024$
  - Leeward $p_h = (27.909)(0.828)(-0.5) - 5.024 = \boxed{-16.574 \text{ psf}}$
- EAST | WEST
  - windward $p_z = (q_z)(0.9097)(0.8) + 5.024$
    $= (q_z)(0.72776) + 5.024$
  - Leeward $p_z = (27.909)(0.9097)(-0.3) - 5.024 = \boxed{-12.64 \text{ psf}}$

* Completed in Table for each story *

As Designed by DCE

As per NYCBC, wind loads are distributed as follows:



E|W
$V_{base} = 150^k$
$M_{base} = 9400^{\prime k}$

N|S
$V_{base} = 360^k$
$M_{base} = 30,200^{\prime k}$

(DeSimone values look only at windward pressure)

My calculated base shears and moments (E|W: 147.88^k, $9266^{\prime k}$
N|S: 351.74^k, $24695.58^{\prime k}$) for the windward direction can be directly compared to DCE values. They are slightly lower because ASCE 7 requires you have pressures vary at each level while NYCBC has pressures in three distinct increments. The pressures above 100' were under 25psf at some points and values as low as 14.97psf were calculated. Therefore lower values are expected. Base shears and moment for total pressure (windward + leeward) are shown in spreadsheet

Seismic Loads

Table 15 - Seismic Loads (ASCE 7-05)								
Level	Story Weight w_x (kips)	Height h_x (ft)	h_x^k	$w_x h_x^k$	C_{vx}	Lateral Force F_x	Story Shear V_x (kips)	Moments M_x (ft-k)
PH Roof	394.00	134.30	266.69	105075.84	0.07	10.97	0.00	1392.20
PH	1143.00	119.55	233.56	266964.03	0.19	27.87	10.97	3155.28
10	919.00	106.89	205.58	188931.24	0.13	19.73	38.84	1991.16
9	915.00	95.06	179.85	164565.43	0.11	17.18	58.57	1531.10
8	915.00	83.23	154.57	141429.55	0.10	14.77	75.75	1141.16
7	1369.00	71.40	129.78	177672.17	0.12	18.55	90.52	1207.09
6	1326.00	58.82	104.05	137975.52	0.10	14.41	109.07	761.50
5	1319.00	46.99	80.55	106250.50	0.07	11.09	123.48	455.17
4	1330.00	35.16	57.88	76974.58	0.05	8.04	134.57	234.68
3	1451.00	23.33	36.26	52612.65	0.04	5.49	142.61	98.16
2	1295.00	12.50	17.80	23054.05	0.02	2.41	148.10	15.04
1*	166.20	0.00	0.00	0.00	0.00	0.00	150.51	0.00
$\Sigma w_i h_i^k$ =	1441505.58	$\Sigma F_x = V_x$ =	150.5064 k			Σ Moments M_x =	11982.54 ft-k	
Total Building Weight (Above Grade)			12542.20 k					

* First floor story weight is only the weight of the columns whose base is at the ground floor. Weights of slab, beams and superimposed dead load on the ground floor are not considered because base shear is related to levels above grade and those components mentioned are at grade.

Table 16 - Seismic Loads (NYCBC)						
Level	Story Weight w_x (kips)	Height h_x (ft)	(ZIC/R_w)	Lateral Force F_x	Story Shear V_x (kips)	Moments M_x (ft-k)
PH Roof	394.00	134.30	0.02756	10.86	0.00	1378.11
PH	1143.00	119.55	0.02756	31.50	10.86	3566.31
10	919.00	106.89	0.02756	25.33	42.36	2556.85
9	915.00	95.06	0.02756	25.22	67.69	2247.37
8	915.00	83.23	0.02756	25.22	92.91	1949.01
7	1369.00	71.40	0.02756	37.73	118.14	2455.33
6	1326.00	58.82	0.02756	36.55	155.87	1931.96
5	1319.00	46.99	0.02756	36.36	192.42	1491.67
4	1330.00	35.16	0.02756	36.66	228.77	1070.44
3	1451.00	23.33	0.02756	39.99	265.43	714.69
2	1295.00	12.50	0.02756	35.69	305.43	223.09
1*	166.20	0.00	0.02756	4.58	341.12	0.00
$\Sigma F_x = V_x$ =	345.70 k			Σ Moments M_x =	19584.83 ft-k	
Total Building Weight (Above Grade)			12542.20 k			

* First floor story weight is only the weight of the columns whose base is at the ground floor. Weights of slab, beams and superimposed dead load on the ground floor are not considered because base shear is related to levels above grade and those components mentioned are at grade.

TECH REPORT 1
S. D'AGOSTINO

SEISMIC CALCULATIONS (Assuming rigid diaphragm for Tech 1 Analysis)

- $S_S = 0.361$; $S_1 = 0.070g$ (from USGS.gov)
  $F_a = 1.0$; $F_v = 1.0$

- $S_{MS} = F_a S_S = 1.0(0.361) = 0.361$ (Eq. 11.4-1)
  $S_{M1} = F_v S_1 = 1.0(0.070) = 0.070$ (Eq. 11.4-2)

- $S_{DS} = 2/3(S_{MS}) = 2/3(0.361) = 0.241$ (Eq. 11.4.3)
  $S_{D1} = 2/3(S_{M1}) = 2/3(0.070) = 0.047$ (Eq 11.4.4)

- Seismic Design Category (SDC) based on Short Period Response
  Table 11.6-1
  $0.167 < S_{DS} = 0.241 < 0.33 \longrightarrow B$

- Seismic Design Category (SDC) based on 1-s period Response
  Table 11.6-2
  $S_{D1} = 0.047 < 0.067 \longrightarrow A$

  * Must choose more severe design category $\rightarrow \boxed{B}$

- Response Modification coefficient $\boxed{R = 5}$
  according to Table 12.2-1 for Ordinary Reinforced concrete Shear walls (Building Frame System)

$T_a = C_t h_n^x = 0.02(134.3)^{0.75} = 0.789$
$T_L = 6s$ (Fig. 22-15)

$T = T_a = 0.789$ as per section 12.8.2 ASCE 7-05
$< T_{max} = C_u T_a = 1.7(0.789) = 1.34s$

$$C_S = \min \begin{cases} \dfrac{S_{DS}}{\left(\frac{R}{I}\right)} = \dfrac{0.241}{(5/1.0)} = 0.0482 \\[2ex] \dfrac{S_{D1}}{T\left(R/I\right)} = \dfrac{0.047}{(0.789)(5/1.0)} = \boxed{0.0120} \geq 0.01 \\[2ex] \dfrac{S_{D1} T_L}{T^2(R/I)} = \dfrac{(0.047)(6s)}{(0.789)^2(5/1.0)} = 0.0906 \end{cases}$$

$V = C_S W = (0.012)(12542\ k) = \boxed{150.5\ k}$

TECH REPORT 1
S. D'AGOSTINO

SEISMIC CALCULATIONS (CONT)

Weights shown in excel spreadsheet

$K = 0.75 + 0.5(T) = 0.75 + 0.5(0.789) = 1.14$

$w_x h_x^k$ = varies

$\sum w_i h_i^k = 1553453^{k}$

$C_{vx} = \dfrac{w_x h_x^k}{\sum w_i h_i^k}$ = varies at ht (12.8-12)

$F_x = C_{vx} V$ (Eq. 12.8-11)

In excel spreadsheet

As designed by Desimone Consulting Engineers (DCE)

Based on NYCBC

$V = \left(\dfrac{ZIC}{R_w}\right) W$

$V = \dfrac{(0.15)(1.0)(1.47)}{8} W$

$= 0.02756 W$

$> 0.012 W$ via Equivalent Lat. Force Procedure ASCE 7-05

Seismic Zone Factor: $Z = 0.15$
Important Factor: $I = 1.0$
Site Coefficient for S_1 soil: $S = 1.0$
R_w - Build Frame System - Conc. Shear Walls $= 8$
$h_n = 152'$ (* includes cellar)
Coefficient $C = 1.47$

$V = 360^k$ (provided by DCE)

$\therefore W = 13061^k > W_{calculated} = 12542.2^k$

CONCLUSION:

The variation of base shear values between DCE values and those calculated above can most likely be related to the use of different codes. The NYCBC (2003) has an entirely different equation in comparison to ASCE 7-05 with different variables. The coefficient $\left(\dfrac{ZIC}{R_w}\right)$, relatable to C_s from ASCE 7, is more than twice the value of C_s. Also, the height considered in NYCBC $h_n = 152'$ which is full building height including the cellar while the ELFP only looks at ht above grade. For NYCBC the weight of the ground floor and its supporting columns is included which explains the weight difference.

Therefore, given the use of ASCE 7-05 this base shear seems reasonable.

Appendix C

Load Distribution

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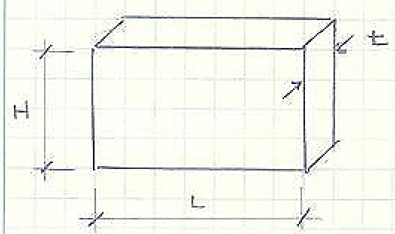
Rigidity, Relative Stiffness, and Center of Rigidity

Table 17 - Wall Rigidity Calculation (N-S Span)						
Supported Floor	Ht	Wall 1	Wall 2	Wall 3	Σ Rigidities	Center of Rigidity (x)
		Length - 256"	Length - 323"	Length - 323"		
Floor 1	150	23874	34104	34104	92082	706.4
Floor 2	280	7184	11750	11750	30685	730.1
Floor 3	422	2676	4764	4764	12205	744.2
Floor 4	564	979	1823	1823	4624	751.3
Floor 5	706	525	1001	1001	2526	755.1
Floor 6	857	302	585	585	1472	757.5
Floor 7*	999	144	379	379	902	800.8
Floor 8*	1141	74	259	259	591	833.9
Floor 9*	1283	40	184	184	409	857.9
Floor 10*	1435	23	133	133	288	876.7
Penthouse*	1612	0	74	74	147	951.5

Table 18 -Wall Rigidity Calculation (E-W Span)							
	Ht	Wall 4	Wall 5	Wall 6	Wall 7	Σ Rigidities	Center of Rigidity (y)
		Length - 122"	Length - 90.5"	Length - 99"	Length - 90.5"		
Floor 1*	150	5500	2639	8	328	8474	642.2
Floor 2	280	1107	479	442	160	2189	566.1
Floor 3	422	348	146	158	77	729	550.7
Floor 4	564	118	49	57	32	255	543.5
Floor 5	706	61	25	30	18	134	539.4
Floor 6	857	34	14	17	11	77	536.0
Floor 7	999	22	9	11	7	49	535.1
Floor 8	1141	15	6	7	5	33	534.1
Floor 9	1283	10	4	5	4	23	533.0
Floor 10	1435	7	3	4	3	17	530.6
Penthouse	1612	0	2	0	0	2	655.0

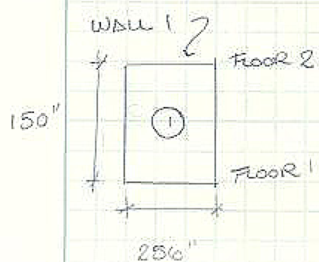
TECH 3 | 40 BOND | RIGIDITY: RELATIVE STIFFNESS + CENTER OF RIGIDITY | 1/2



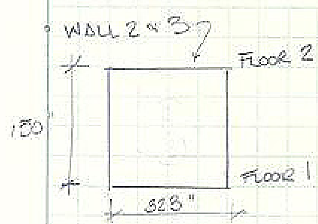
• RIGIDITY R
(from Boothby AE 431 handout)

$$= \frac{Et}{4\left(\frac{h}{L}\right)^3 + 3\left(\frac{h}{L}\right)}$$

$E = 57000\sqrt{f'c}$
$= 57000\sqrt{8000} = 5.098 \times 10^6$ (1-3)
$= 57000\sqrt{5000} = 4.030 \times 10^6$ (4-PH)
t = thickness = 12"
h = height from base to top of each level
L = length of wall element



$$R_{1\text{-}1} = \frac{(5.098 \times 10^3 \text{ ksi})(12")}{4\left(\frac{150"}{256}\right)^3 + 3\left(\frac{150}{256}\right)} = \boxed{23874}$$



$$R_{1\text{-}2} \cdot R_{1\text{-}3} = \frac{(5.098 \times 10^6 \text{ psi})(12")}{4\left(\frac{150"}{323}\right)^3 + 3\left(\frac{150}{323}\right)} = \boxed{34104}$$

$$\Sigma R = R_{1\text{-}1} + R_{1\text{-}2} + R_{1\text{-}3} = 23874 + 34104 + 34104 = 92082$$

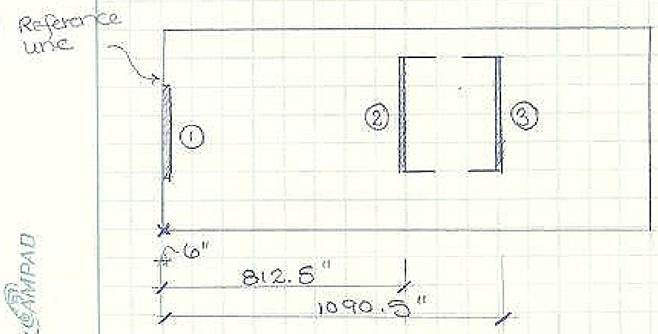
• RELATIVE STIFFNESS (%) $= \frac{R}{\Sigma R} \cdot (100)$

$$\frac{R_{1\text{-}1}}{\Sigma R} = \frac{23874}{92082} = \boxed{25.93\%} \text{ to wall 1}$$

$$\frac{R_{1\text{-}2}}{\Sigma R} = \frac{34104}{92082} = \boxed{37.04\%} \text{ to Wall 2 and Wall 3}$$

* These calculations for all walls at all levels can be found in the excel charts: WALL RIGIDITY CALCULATION (Tables 17-18) and RELATIVE STIFFNESS (Table 6)

TECH 3 | 40 BOND | RIGIDITY, RELATIVE STIFFNESS + CENTER OF RIGIDITY



- CENTER OF RIGIDITY

$$\frac{\Sigma(Rd)}{\Sigma R} = \frac{(23874)(6") + (34104)(812.5") + (34104)(10}{23874 + 34104 + 34104}$$

$= \boxed{706.4"}$ x coordinate of COR on Floor

* the calculation values for the (x,y) coordinates for all floors can be found in the excel chart:

Table 5

- CENTER OF MASS
  - center of each floor since footprint is a rectangle.
    Coordinates: $\boxed{(800, 376.25)}$

Appendix D
Shear

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TECH 3 | 40 BOND | SHEAR DISTRIBUTION | 1/1

CONTROLLING LOAD CASES

- NORTH / SOUTH : $1.2D + 1.6W + L + 0.5L_r$
- EAST / WEST : $0.9D + 1.0E$

- DIRECT SHEAR = (factored story force) × $\frac{(\text{Relative Stiffness } \%)}{100}$

- TORSIONAL SHEAR :

$$V_i = \frac{V_{tot}\, e\, d_i\, R_i}{J}$$

V_{tot} = Story Shear
e = distance from COM to COR
d_i = distance from element i to COR
R_i = Relative Stiffness of Element i
J = torsional moment of inertia = $\Sigma(R \cdot d_i^2)$

Ex. WALL 1 SUPPORTING FLOOR 7 (N/S)

- Factored Story Shear : $(1.6)(429.6) = 687.4^k$

- COR x-coordinate = 755.1
  COM x-coordinate = 800
  > calculated by hand vs. Etabs model

  $e = 800 - 755.1 = 44.89''$

- Location of Wall 1 : x-coordinate = 6''
  $d_i = 755.1 - 6 = 749.11''$

- $J = \Sigma(R \cdot d_i^2) = 186393.18 \text{ in}^2$ (from excel)

- $R_i = 0.208$

$$V_i = \frac{(687.4)(44.89)(749.11)(0.208)}{186393.18} = \boxed{25.76^k}$$

- Direct Shear : $20.69 + 14.76 + 11.19 + 8.64 + 6.06 + 20.06 = \boxed{81.41^k}$

* The calculated values for all walls can be found in excel chart DIRECT SHEAR & TORSIONAL SHEAR (Tables 8-10)

TECH 3 | 40 BOND | STRENGTH CHECK | 1/1

SHEAR STRENGTH

ACI 318.08 § 21.9.4

"Structural walls shall not exceed V_n:

$$\phi V_n = \phi A_{cv} (\alpha_c \lambda \sqrt{f'_c} + \rho_t f_y)$$

$\phi = 0.75$
A_{cv} = gross area of concrete
α_c = coefficient : 2.0 if $h_w/l_w \geq 2.0$
$\rho_t = \frac{A_v}{s \cdot h}$
s = spacing of shear reinforcement
h = thickness of wall. (always 12")

Ex. WALL 1 SUPPORTING FLOOR 6

Direct Shear = 81.41^K (from excel Table ___
Torsional Shear : 25.76^K (from excel Table ___

$- V_u = 81.41^K + 25.76^K = 107.17^K$

Vertical Reinf : (2) #6 @ 12"

$$\rho_t = \frac{(2)(0.44)}{(12'')(12'')} = 0.0061$$

A_{cv} = (256" in length)(12" thickness) = 3072 in^2

$$\phi V_n = 0.75(3072 in^2)\left[(2.0)\left(\frac{\sqrt{5000}}{1000}\right) + 0.0061\ (60\ ksi)\right]$$

$$= 1169.0^K$$

$\boxed{\phi V_n = 1169.0^K > 107.14^K}$ OKAY!

* The remaining calculated shear strengths for this wall may be found in excel Table 11

Appendix E

Drift and Displacement

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Table 19 - Wall 1 Displacement Calculations										
Supported Floor	Lateral Force (k)	E_c (ksi)	E_r (ksi)	Thickness (in)	Length (in)	Height (in)	$\Delta_{flexural}$ (in)	Δ_{shear} (in)	Story Displacement (in)	Story Drift (in)
Floor 2	14.77	5.10E+03	2.04E+03	12	256	150	0.000194	0.000424	0.00062	4.1236E-06
Floor 3	15.82	5.10E+03	2.04E+03	12	256	280	0.001354	0.000849	0.00220	7.86519E-06
Floor 4	13.66	5.10E+03	2.04E+03	12	256	422	0.004001	0.001104	0.00511	1.2099E-05
Floor 5	13.02	4.03E+03	1.61E+03	12	256	564	0.011516	0.001779	0.01330	2.35732E-05
Floor 6	12.54	4.03E+03	1.61E+03	12	256	706	0.021756	0.002145	0.02390	3.38542E-05
Floor 7	12.93	4.03E+03	1.61E+03	12	256	857	0.04013	0.002686	0.04282	4.99601E-05
Floor 8	9.23	4.03E+03	1.61E+03	12	231	999	0.061724	0.002475	0.06420	6.42637E-05
Floor 9	6.99	4.03E+03	1.61E+03	12	210	1141	0.092774	0.002357	0.09513	8.33748E-05
Floor 10	5.40	4.03E+03	1.61E+03	12	192.8	1283	0.131638	0.002229	0.13387	0.000104339
Penthouse	3.79	4.03E+03	1.61E+03	12	178	1435	0.164203	0.001895	0.16610	0.000115747
Penthouse Roof	0.00	4.03E+03	1.61E+03	12	0	1612	0	0	0.00000	0
							Total Wall Displacement (in) =		0.54723	

Table 20 - Wall 2 & 3 Displacement Calculations										
Supported Floor	Lateral Force (k)	E_c (ksi)	E_r (ksi)	t (in)	Length (in)	Height (in)	$\Delta_{flexural}$ (in)	Δ_{shear} (in)	Story Displacement (in)	Story Drift (in)
Floor 2	21.09	5.10E+03	2.04E+03	12	323	150	0.000138	0.00048	0.00062	4.94832E-05
Floor 3	25.88	5.10E+03	2.04E+03	12	323	280	0.001102	0.0011	0.00220	7.86519E-06
Floor 4	24.33	5.10E+03	2.04E+03	12	323	422	0.003547	0.001559	0.00511	1.2099E-05
Floor 5	24.23	4.03E+03	1.61E+03	12	323	564	0.010671	0.002625	0.01330	2.35732E-05
Floor 6	23.92	4.03E+03	1.61E+03	12	323	706	0.020658	0.003243	0.02390	3.38542E-05
Floor 7	25.05	4.03E+03	1.61E+03	12	323	857	0.038693	0.004122	0.04282	4.99601E-05
Floor 8	24.33	4.03E+03	1.61E+03	12	323	999	0.059532	0.004668	0.06420	6.42637E-05
Floor 9	24.61	4.03E+03	1.61E+03	12	323	1141	0.089737	0.005393	0.09513	8.33748E-05
Floor 10	24.58	4.03E+03	1.61E+03	12	323	1283	0.1274	0.006056	0.13346	0.000104019
Penthouse	22.06	4.03E+03	1.61E+03	12	323	1435	0.160017	0.00608	0.16610	0.000115747
Penthouse Roof	26.48	4.03E+03	1.61E+03	12	297	1612	0	0	0	0
							Total wall displacement (in) =		0.54682	

TECH 3 | 40 BOND | DISPLACEMENT | 1/4

STORY DISPLACEMENT

- An approximate method to determine story shear is to calculated Δ_{cont} up the building.
- Story drift $\Delta = 0.025 h_{sx}$ w/ h_{sx} = story ht below X (Table 12.12-1 ASCE 7)

$$A_{cont} = \Delta_{flexural} + \Delta_{shear}$$

$$= \frac{Ph^3}{3E_cI} + \frac{1.2Ph}{E_v A}$$

$$E_c = 57000\sqrt{8000} = 5.098 \times 10^3 \text{ ksi}$$ (Ground - Sept 4)

$$= 57000\sqrt{5000} = 4.030 \times 10^3 \text{ ksi}$$ (Sept 5 - Sept PHR)

E_v = modulus of rigidity $\approx 0.4 E_c$

$= 2.04 \times 10^3$ ksi

$= 1.61 \times 10^3$ ksi

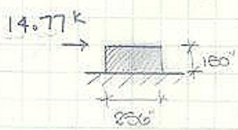
A = length (thickness) * thickness = 12"

$$I = (\text{thickness})(\text{length})^3 / 12$$

Ex. Wall 1

N/S ∘ $1.2D + 1.6W + 1.0L + 1.5L_r$

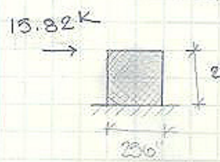
- Level 1 (Sept 2)



$$\Delta_1 = \frac{14.77^k (150")^3}{3(5.098\times10^3)\left(\frac{(12)(256)^3}{12}\right)} + \frac{1.2(14.77^k)(150")}{(2.04\times10^3)(12")(256")}$$

$$= 0.00019" + 0.00042" = 0.000618"$$

- Level 2 (Sept 3)

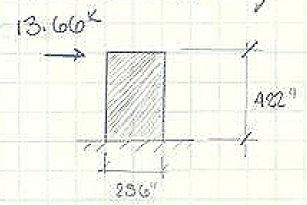


$$A_2 = \frac{(15.82^k)(280")^3}{3(5.098\times10^3)\left(\frac{12(256)^3}{12}\right)} + \frac{1.2(15.82^k)(280")}{(2.04\times10^3)(12")(256")}$$

$$= 0.00135" + 0.00085" = 0.0022"$$

TECH 3 | 40 BOND | DISPLACEMENT | 2/4

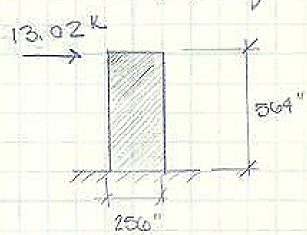
• Level 3 (Supt 4)



$$\Delta_3 = \frac{(13.66^k)(422'')^3}{3(5.098\times10^3)\left(\frac{12(256)^3}{12}\right)} + \frac{1.2(13.66^k)(422)}{(2.04\times10^3)(12'')(256'')}$$

$$= 0.0040'' + 0.0011'' = 0.0051''$$

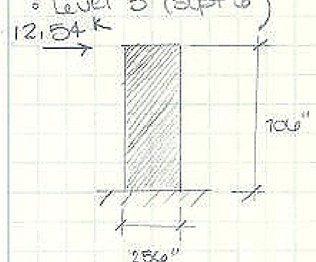
• Level 4 (Supt 5)



$$\Delta_4 = \frac{(13.02^k)(564'')^3}{3(4.030\times10^3)\left(\frac{12(256)^3}{12}\right)} + \frac{1.2(13.02^k)(564'')}{(1.61\times10^3)(12)(256'')}$$

$$= 0.0115'' + 0.0018'' = 0.0133''$$

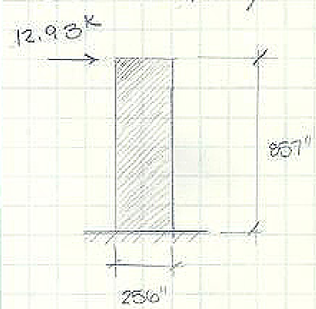
• Level 5 (Supt 6)



$$\Delta_5 = \frac{(12.54^k)(706)^3}{3(4.030\times10^3)\left(\frac{12(256)^3}{12}\right)} + \frac{1.2(12.54^k)(706'')}{(1.61\times10^3)(12'')(256'')}$$

$$= 0.0218'' + 0.00216'' = 0.0239''$$

• Level 6 (Supt 7)

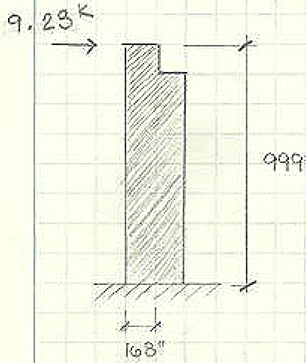


$$\Delta_6 = \frac{(12.93^k)(857)^3}{3(4.030\times10^3)\left(\frac{12(256)^3}{12}\right)} + \frac{1.2(12.93^k)(857')}{(1.61\times10^3)(12'')(256'')}$$

$$= 0.0401'' + 0.00269'' = 0.0428''$$

TECH 3 | 40 BOND | DISPLACEMENT | 3/4

Level 7 (Supt 8)



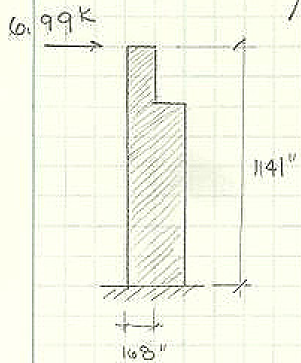
Effective Length

$$\frac{(256)(6)}{(256)(6)+(1)(168)} \times 256 = 231''$$

$$\Delta_7 = \frac{(9.23^k)(999'')^3}{3(4.030\times10^3)\left(\frac{12(231)^3}{12}\right)} + \frac{1.2(9.23^k)(999'')}{(1.61\times10^3)(12)(231)}$$

$$= 0.0617'' + 0.00248'' = 0.0642''$$

Level 8 (Supt 9)



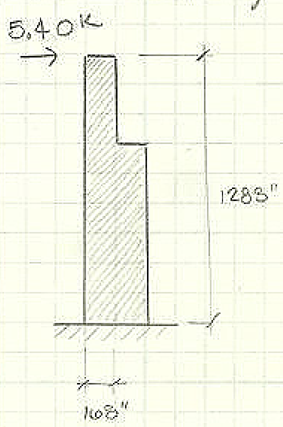
Effective length

$$\frac{(256)(6)}{(256)(6)+(2)(168)} \times 256 = 210.1''$$

$$\Delta_8 = \frac{(6.99^k)(1141)^3}{3(4.030\times10^3)\left(\frac{12(210.1)^3}{12}\right)} + \frac{1.2(6.99^k)(1141)}{(1.61\times10^3)(12)(210.1)}$$

$$= 0.0928'' + 0.00236'' = 0.0951''$$

Level 9 (Supt 10)



Effective length

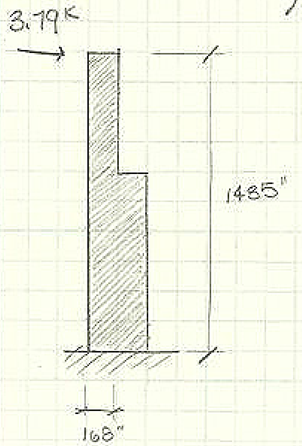
$$\frac{(256)(6)}{(256)(6)+(3)(168)} \times 256 = 192.8''$$

$$\Delta_9 = \frac{(5.40^k)(1283'')^3}{3(4.03\times10^3)\left(\frac{12(192.8)^3}{12}\right)} + \frac{1.2(5.40^k)(1283)}{(1.61\times10^3)(12)(192.8)}$$

$$= 0.1316'' + 0.0022'' = 0.1340''$$

TECH 3 | 40 BOND | DISPLACEMENT | 4/4

Level 10 (Sup+PH)



Effective Length

$$\frac{(256)(6)}{(256)(6)+(4)(168)} \times 256 = 178''$$

$$\Delta_{10} = \frac{(3.79^{k})(1435)^3}{3(4.03\times10^3)\left(\frac{12(178'')^3}{12}\right)} + \frac{(1.2)(3.79)(1435)}{(1.61\times10^3)(12)(178)}$$

$$= 0.1648'' + 0.0019'' = 0.1662''$$

OVERALL WALL DISPLACEMENT

$$\Sigma\Delta = 0.00062'' + 0.00220'' + 0.00511'' + 0.01330'' \; 0.02390'' + 0.04282'' + 0.06420'' + 0.09513'' + 0.13387'' + 0.16610''$$

$$= 0.547 \leq H/400 = (1435)/400 = 3.59''$$

okay!

* The calculated disp. for all walls 1, 2 and 3 can be found in Tables 19 - 20 *

Appendix F
Overturning

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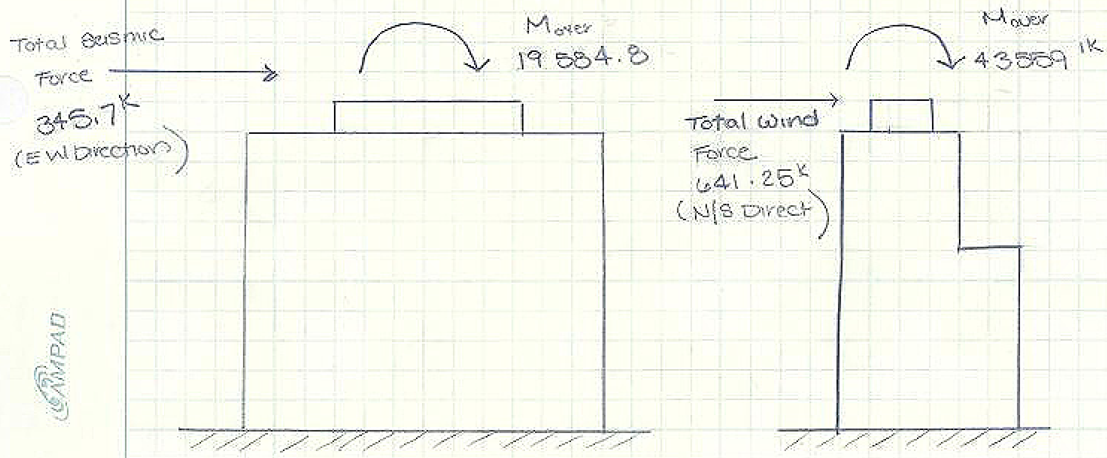
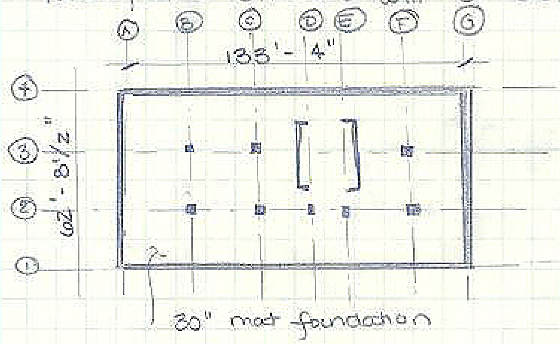
TECH 3 | 40 BOND | OVERTURNING | 1/1



LATERAL FORCES create an overturning moment, while gravity loads resist that moment.

To see if the gravity load (DEAD ONLY) exceed lateral loads, the stresses will be determined



Stress due to DL

$= \dfrac{\text{Weight of Bldg}}{SF_{mat}}$

$= \dfrac{13824^k}{(62.75')(133.3')} \cdot 1000\text{ lb}$

$= 1653$ psf

Stress due to E/W Seismic $= \dfrac{346^k(1000\text{lb})}{(62.75)(133.3)} = 41.4$ psf (2.5% of DL)

Stress due to N/S wind $= \dfrac{641.25^k(1000\text{lb})}{(62.75)(133.3)} = 76.7$ psf (4.6% of DL)

Because the stress from the lateral loads is such a smaller percentage of the resisting gravity load; overturning is not a critical issue in this design.