40 Bond

New York, NY

Technical Report 1

40 Bond is a luxury residential building located in the NoHo neighborhood of Manhattan. The architecture and function of this building greatly impacted the design and development of its structure. A concrete flat plate system was used to maximum floor-to-ceiling heights and the lateral system is a combination of concrete shear walls. A transfer system including several sets of beams is utilized to allow for long spans, setbacks and transitions between 10”x10” columns to larger, narrower columns below. Unique to this project is the use of a grid layout that is not typical to flat plate construction. The foundation consists of a mat slab and the roof of the penthouse structure acts as a bridge over a 44’-0” clear span allowing for a seemingly support free space.

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Structural Option

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Executive Summary

The structural concepts and existing conditions report describes the structural system of 40 Bond. This 10-story luxury residential building located in the NoHo neighborhood of Manhattan is a concrete structure making use of a 30" mat foundation (Figure 1), concrete flat plate slabs, ordinary reinforced concrete shear walls and a variety of columns. The perimeter columns, only 10"x10" in dimension, are spaced at 6'-3" on center and are located along the north and south facades (Figure 2). Due to setbacks and transitions between columns, there are numerous transfer beams located throughout this building. A penthouse structure also rises 20'-0" above the main roof line. The roof of this structure, which is a combination of upturned beams and inclined piers, acts as a bridge across a 44'-0" clear span. The only two supports are the core shear wall and two 28"x16" columns (Figure 3).

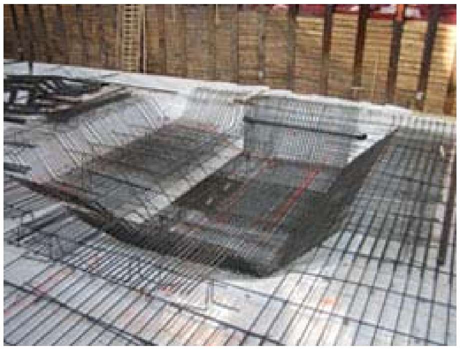


Figure 1 – Mat Foundation Reinforcing

Figure 2 – 10"x10" Columns

Figure 3 – Penthouse structure

Gravity and lateral loads were calculated using ASCE 7-05 and compared to loads determined by DeSimone Consulting Engineers (DCE) who used the New York City Building Code (NYCBC). The controlling lateral load was found to be the wind in the North/South direction with a base shear, V=641.25k for the windward and leeward pressures. The base shear in this direction due to windward pressures alone, V=351.74k, was very close to that determined by DCE, V=360 k. Similarly, the base shear in the East/West direction was within 5k of DCE calculated values. The seismic loads had a notable discrepancy and at this point in the schematic analysis it was noted that this is a possible outcome of comparing two different codes/standards.

Spot checks were conducted on a portion of the two-way flat plate slab and an interior column. These checks supported that the determination and accumulation of the gravity loads on this structure were comparable to those done by DCE. Each component was adequately designed and in any event that either appeared to be overdesigned, via the calculations within this report, it was noted that only gravity loads were taking into account. Once the analysis is done to include the lateral forces it is likely that the members designed herein will be larger and/or contain additional reinforcement bringing them to the sizes designed by DCE.

Introduction

The structural concepts and existing conditions report contains a description of the structural system of 40 Bond. The architecture is briefly examined to relate its impact on the structural design. An overview is given in regards to the framing, slabs, lateral force resisting systems, and foundations to better explain how each of the components work together. Loads are calculated based on applicable building codes and standards and then related to those originally designed by the structural engineer, DeSimone Consulting Engineers (DCE). A combination of drawings, specifications, and soils reports were used to obtain the existing conditions information. Spot checks of typical floor framing are included to verify if the required loading was calculated and considered correctly.

The building 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 has an overall building height of 152'-0" from 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 in size from 19'-6" x 25'-0" to 23'-2 ½" x 25'-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 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 cellar that houses a fitness center, storage space and equipment rooms. The first and second floors are devoted to 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. It is in the penthouse that 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.


Figure 4 – South Facade

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

4). 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 blackened copper. These mullions then serve as a grid for the perimeter columns along the north and south faces. Small 10"x10" 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 transfers, to be made in order to limit any compromise of the architectural features that are so prominent in this building.

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 there are a limited number of beams and girders dropping into the space. Also, unique to this project is the application of a column grid which is not always seen in flat plate construction. 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, however, this 90,000 sf building was not optimized. Transfer beams and many slender columns equate to a lot of formwork which is accompanied by an increased cost.

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 reported 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"x29 ½" to 18"x79" are located around the perimeter. Interior columns ranging in size from 12"x22" to 28"x28" have an increased strength of 8 ksi. Located at columns 3B, 3C and 3F (Figure 5), there are also foundation mat shear heads to resist punching shear due to high loads that continue from the roof down to the foundation.

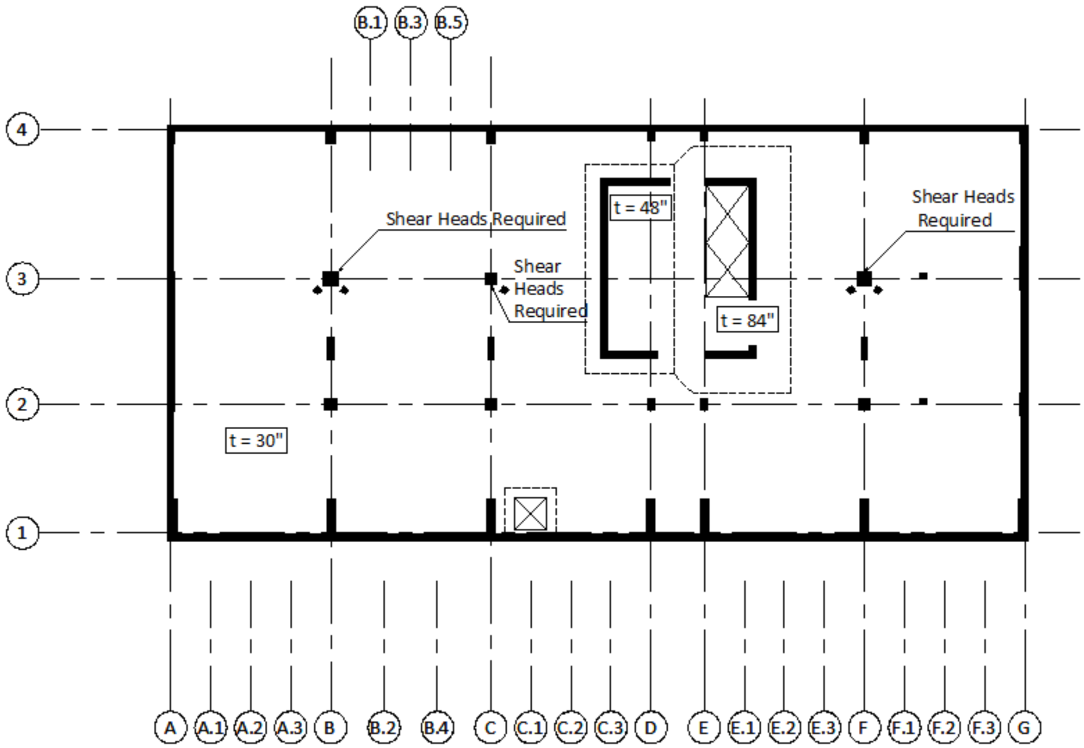


Figure 5 – Foundation Plan with Typical Column Grid and Shear Head Locations Noted

Superstructure

The ground floor is a 9" two-way flat plate slab (NWC) with a compressive strength (f'_c) of 5.95 ksi and typical reinforcement of #4@12 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"x24" to 16"x58" 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"x22", 22"x22" 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 collector 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 noted above minus the slab step. Particular to the second floor is the introduction of the 10"x10" 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"x40" 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"x10" columns spaced at 6'-3" on center along the south face. The transfer beams at this level are 60"x16" 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"x10" columns only have a 7" slab encroachment that has a 1" slab depression around each column (Figure 6).

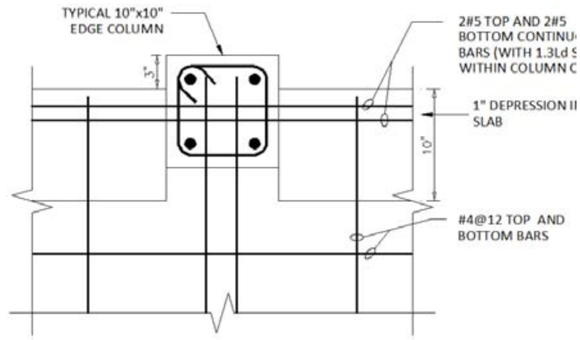


Figure 6 – Typical Perimeter Column Detail

All floors between level 4 to the penthouse level use a 9" two-way flat plate slab with #4@12 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 reduced f'$_c$ = 5 ksi. The columns along the north and south façade remain 10"x10" while those located on the east and west walls and within the interior vary between 12"x22" to 28"x28". 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 7). This 12"x50" transfer was designed after construction began due to the presence of an adjacent chimney breath 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"x24" transfer beam along line 2 is needed due to the introduction of the 10"x10" columns along this line (Figure 8).

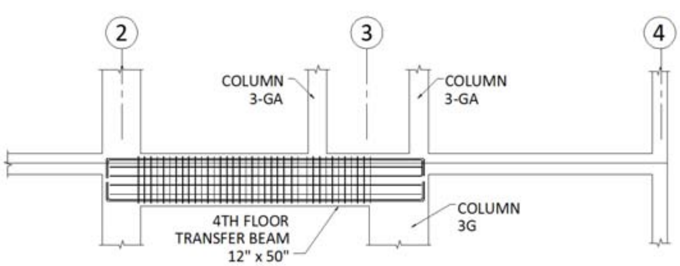


Figure 7 – Transfer Beam at Fourth Floor

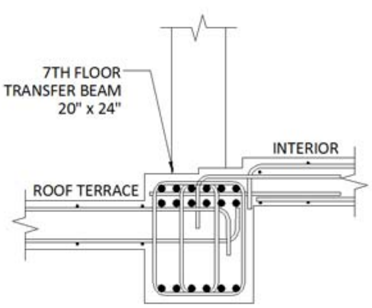


Figure 8 – Transfer Beam at Seventh Floor

The penthouse level and its roof are a perfect example of what can be achieved when using concrete. The footprint of the penthouse is 23'-4" x 44'-6" and it has a 19" slab with #4@12 top bar reinforcement and #5@8 bottom bar reinforcement. A 44'-0" clear span is achieve with the support of the concrete shear walls to the right and two 28"x16" columns to the left. These two columns need to transfer out and a 32"x24" beam is used to direct loads to nearby columns, one of which is only 10"x14". The roof above this long span structure is a combination of upturned beams, inclined piers, and two separate 8" slabs with #5@12 to act like a bridge spanning between its two supports (Figure 9). 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 9 –Penthouse Roof Structure

Lateral System

As mentioned previously, the lateral system is a combination of 12" ordinary reinforced concrete shear walls (Figure 10). Within the core shear walls there are the stair, elevator and mechanical shafts. 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 has #4@12 as the horizontal reinforcement and a range of vertical reinforcement from #4@12 to #7@12. All shear walls supporting the ground floor to those supporting the fourth floor have concrete with $f'_c = 8$ ksi while those supporting the rest of the building have an $f'_c = 5$ ksi.

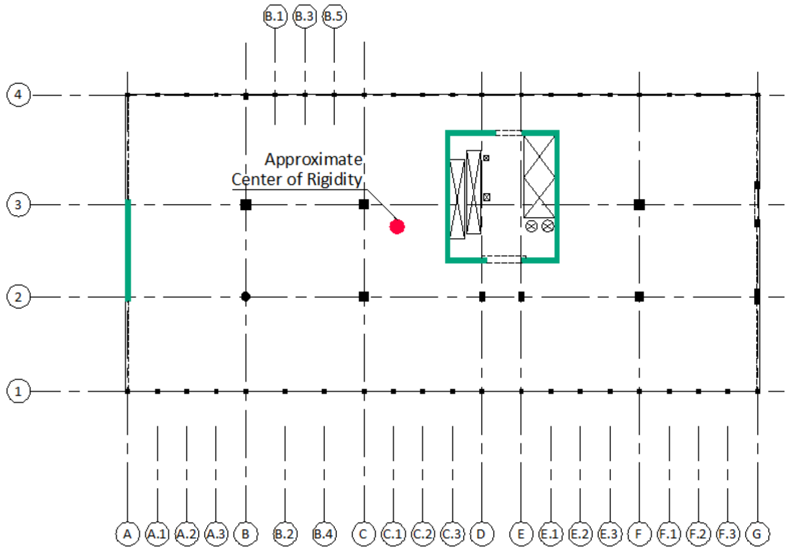


Figure 10 – Typical Plan with Lateral System Highlighted

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. These wind and seismic loads travel through the rigid diaphragm (flat plate slab) to the shear walls and then down into the foundation. This load path is governed by the concept of relative stiffness.

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 in regards to this report. The reason behind using a different standard was because numerous calculations were done, and in order to do so in accordance with the requirements of AE Senior Thesis, ASCE 7-05 was the logical reference. Another note is that DCE does not like to use live load reductions in their design. In order to keep 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 A. 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 2a and these values are supported by base calculations which can be found 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 11).

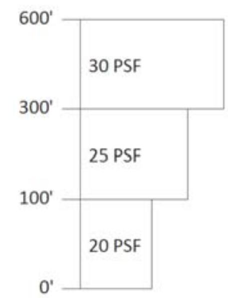


Figure 11 – Wind Load Diagram from NYCBC – RS 9-5

Table 2a - 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)

Table 2b was developed to determine the wind pressures in the north/south direction. These winds are currently those most prevalent at this 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. This is clearly seen in the diagram of the windward and leeward pressures at each level (Figure 12).

Table 2b
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**							

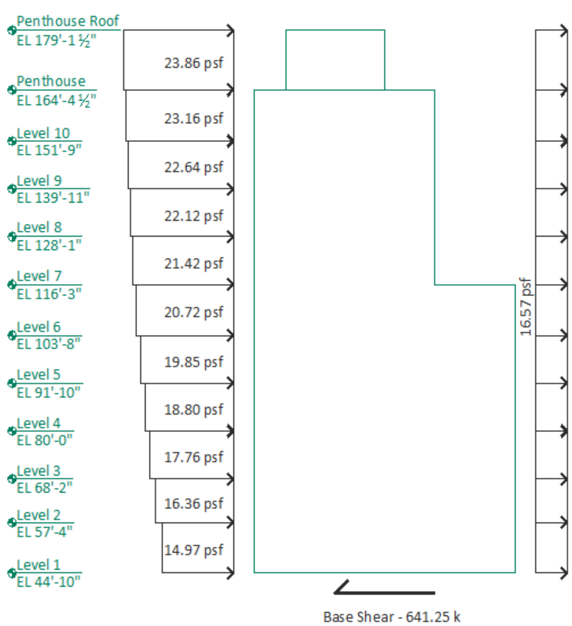


Figure 12 – North/South Wind Pressures

Table 2c was developed to determine the wind pressures in the east/west direction. Although there are currently adjacent building blocking the wind on the lower levels, wind in this 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. A diagram of the windward and leeward pressures at each level is provided to show the values and how they change as they continue up the building (Figure 13).

Table 2c
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**							

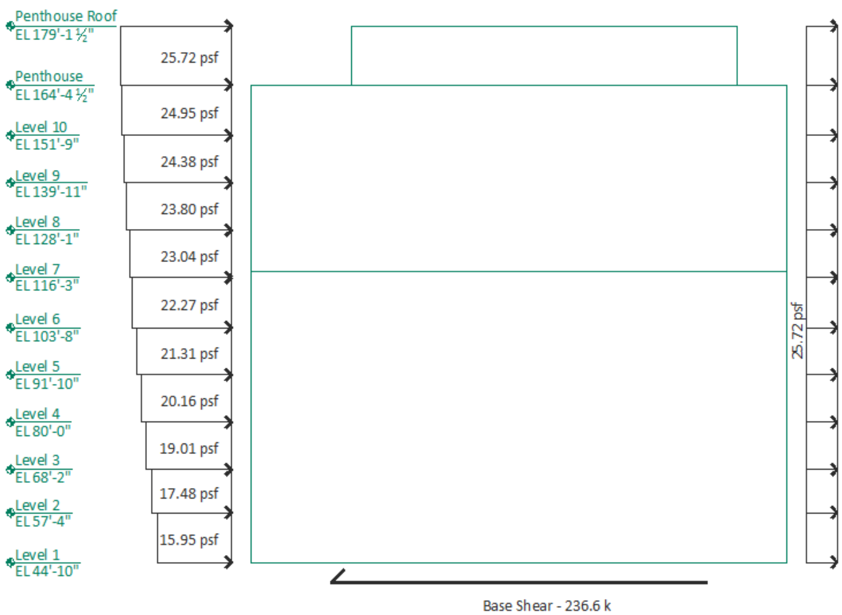


Figure 13 – East/West Wind Pressures

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 shear that the firm designed, and the base shear calculated in this report. 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. Until further analysis is done, it is assumed that this is the reason for the different base shear values.

Another 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 3a.

Table 3a - 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	
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 3b. The actual calculations to determine base shear are included in Appendix C with further description of why the two values differ.

Table 3b - 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'	
Coefficient	C	1.47	(RS 10-5c)

The base shear calculated for this report was $V=C_sW$ with W being the effective seismic weight per Section 12.7.2. A spreadsheet was set up to tally the total weight that accumulated at each floor above grade and an overall building weight was determined with the summation of each floor. An example of one floor can be found in Appendix C. The effective weight was then input into Table 3c that determined the base shear and overturning moment due to seismic loads. All supporting calculations are located in Appendix C and a diagram is provided to relate forces and shears that resulted from seismic loading (Figure 14)

Table 3c - Seismic Loads								
Level	**Story Weight w_x (kips)**	**Height h_x (ft)**	**h_x^k**	**$w_x h_x^k$**	**C_{vx}**	**Lateral Force F_x (kips)**	**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.

** DCE Values: V = 360 k (See seismic load description for reason behind varying base shear values.)

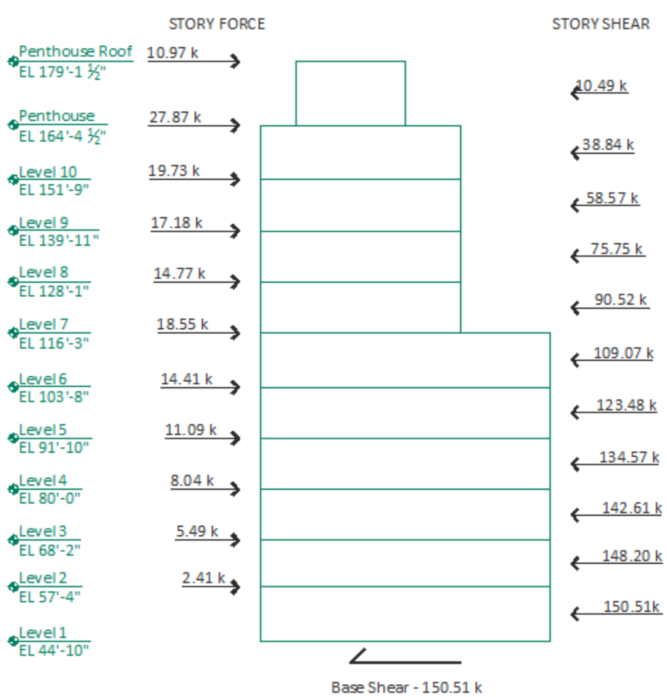


Figure 14 – Seismic Loading on 40 Bond

Spot Checks

In order to verify that the loads determined via Technical Report 1 were adequate and reasonable, spot checks of typical framing were conducted. These spot checks were imperative in being able to compare the calculations done in this report to the design of 40 Bond by DCE. Only gravity loads were applied when doing these calculations and therefore at least some variation could be attributed to the fact that lateral loads will also be present and require analysis. These typical framing elements were taken from Floor 6 and included a check of the slab and column 2C (Figure 15).

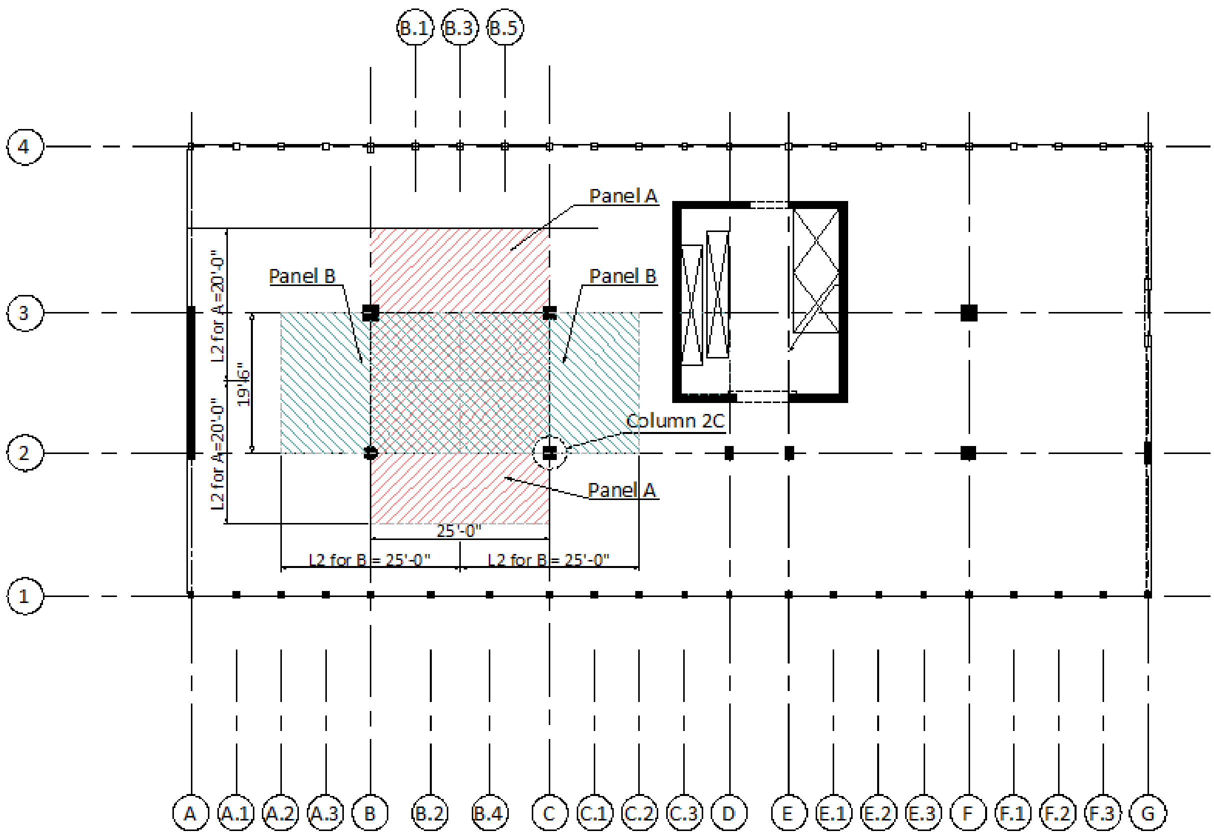


Figure 15 – Slab and Column Spot Check Locations

Using the Direct Design Method (DDM), referenced in Chapter 13 within ACI 318-08 and Chapter 13 within *Design of Concrete Structures* by Nilson, Darwin and Dolan, two panels of the flat plate slab were analyzed to validate the current design based on the determined loads. This type of analysis, in turn, can ensure that the loading was accumulated correctly. For both Panel A and B, the 9" thick slab was above the minimum thickness provided by ACI. Then the column strips and middle strips of each panel were designed for flexure. The reinforcement required was related to the reinforcement designed and in most cases the number of bars directly corresponded to that noted on the plans. There were instances where DCE had designed with additional bars in comparison to those calculated by the DDM. Reasons for this, which are supported by the fact that these areas were those located closest to the core shear walls, may include the fact that no lateral forces were applied in this preliminary design. Loads travel towards the greatest relative stiffness, which in this case is the shear wall. There will be a higher moment value in this area and therefore additional reinforcement would be required which seems like a logical variation between DCE and the calculations found in Appendix D. There was also a check of punching shear at column 2C which revealed that no special shear reinforcement was needed at the column and no note of shear heads were noted by DCE.

The spot check of column 2C was done by creating a column load take down spreadsheet seen in Tables 4a, 4b and 4c. A transfer occurs at Floor 7 and the loads that would be added due to this transfer were accounted for. The load combination of 1.2DL + 1.6LL + 0.5(S or L_r) was determined to be controlling and the loads seen in Table 4c are those that were applied to column 2C for the spot check. Using Eq. 10.2 from ACI, ϕP_n was compared to the P_u provided by the tables below and in all instances it was found that the columns designed by DCE were more than adequate. It must be noted again, however, that these loads are only those due to gravity. An axial force was applied to each column and no moments or additional forces from the lateral loads were taken into account. This leads to the idea that the large gap between the column capacity and the present loads is because of the absence of other loads that are most likely there. For this initial preliminary analysis, note of why there is a discrepancy is substantial for this portion of the report. Supporting calculation can be found in Appendix D.

Table 4a - Spot Check - Column 2C

Level Supported	Tributary Area (sf)	Dead Load (psf)	Live Load (psf)	Superimposed Dead Load (psf)	Total Dead Load (Dead + SDL)	Snow Load (psf)	Total Load (1.4DL) kips	Total Load (1.2DL + 1.6LL + 0.5S) kips	Total Load (1.2DL + 1.6S + L) kips
PH	61	112.5	100	40	152.5	0	13.02	22.7835	19.1235
10	61	112.5	40	25	137.5	0	11.74	15.6465	14.1825
9	61	112.5	40	25	137.5	0	11.74	15.6465	14.1825
8	61	112.5	40	25	137.5	0	11.74	15.6465	14.1825
7 interior	61	112.5	40	25	137.5	0	11.74	15.6465	14.1825
7 exterior	62	112.5	100	40	152.5	0	13.24	23.157	19.437
6	494	112.5	40	25	137.5	0	95.10	126.711	114.855
5	494	112.5	40	25	137.5	0	95.10	126.711	114.855
4	494	112.5	40	25	137.5	0	95.10	126.711	114.855
3	494	112.5	40	25	137.5	0	95.10	126.711	114.855
2	455	112.5	40	25	137.5	0	87.59	116.7075	105.7875
1	455	112.5	40	25	137.5	0	87.59	116.7075	105.7875

Table 4b
Transfer Loads (Transferred to Column 2C at Level 7)

Level Supported	Tributary Area (sf)	Total Load (1.4DL) kips	Total Load (1.2DL + 1.6LL + 0.5S) kips	Total Load (1.2DL + 1.6S + L) kips
PH	182.8	39.03	68.28	57.31
10	182.8	35.19	46.89	42.50
9	182.8	35.19	46.89	42.50
8	182.8	35.19	46.89	42.50
7 interior	182.8	35.19	46.89	42.50
7 exterior	187.5	40.10	70.14	58.88
	Total	**219.88**	**325.97**	**286.19**

Table 4c
Column Load Take Down

Level Supported	Total Load (1.4DL) kips	Total Load (1.2DL + 1.6LL + 0.5S) kips	Total Load (1.2DL + 1.6S + L) kips
PH	13.02	22.78	19.12
10	24.77	38.43	33.31
9	36.51	54.08	47.49
8	48.25	69.72	61.67
7	219.88	325.97	325.97
6	314.97	452.68	440.83
5	410.07	579.39	555.68
4	505.16	706.10	670.54
3	600.26	832.82	785.39
2	687.85	949.52	891.18
1	775.43	1066.23	996.97

Conclusion

After an examination of the existing structural system and calculations of various gravity and lateral loads, it was found that 40 Bond was adequately designed to withstand these forces. Through the use of ASCE 7-05 to calculate wind loads via Method 2 and seismic loads via the Equivalent Lateral Force Procedure, the controlling force was the North/South wind with a base story shear, V=651.25 k, determined by a combination of windward and leeward pressures. The base shear in this direction due to windward pressures alone, V=351.74 k, was well within range of V=360 k which was designed for by DCE. Similarly, the values for the wind force in the East/West direction were within 5 k of those calculated by DCE. More of a discrepancy was found between the seismic loads, but the use of ASCE 7-05 versus the NYCBC may be the reason for this difference.

Spot checks done on a portion of the two-way flat plate slab and an interior column also proved that the determination and accumulation of loads done within this report were comparable to those completed by DCE. These two components were found to be satisfactorily designed. In any event where they appeared to be overdesigned in comparison to the values calculated in this report, there was mention that only gravity loads were taken into account. There were no lateral forces considered which would add greater moments to the columns and slab and therefore may call for larger sizes and/or additional reinforcement. As research continues for 40 Bond, these lateral forces will be taken into account and will have an effect on the framing, shear walls and foundation.

Appendix A – Gravity Loads

1/2
Tech Report 1
S. D'Agostino

SNOW LOAD

Ground Snow Load $p_g = 25psf$ (Fig. 7-1)
DCE uses $p_g = 30psf$

Flat roof snow load $p_f = 0.7 C_e C_t I p_g$ (Eq. 7-1)
$\geq 20 (I)$

C_e = exposure factor = 1.0 (Table 7-2)
C_t = thermal factor = 1.0 (Table 7-3)
I = Importance factor = 1.0 (Table 7-4)
(for occupancy II)

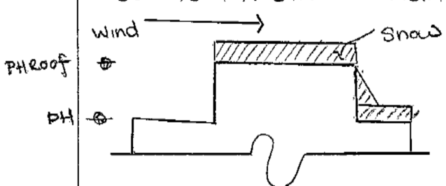
partially exposed because of parapet

$p_f = 0.7(1.0)(1.0)(1.0)(30) = 21 psf$

* Because terrace load = $\boxed{60psf \text{ (NYCBC)}}$
= $\boxed{100psf \text{ (ASCE 7)}}$
- for roof live load these values will be used rather than 21psf
- to be conservative $\boxed{p_f = 30}$ will be used on PH roof

SNOW DRIFT

Due to PH structure rising above main roof line



§ 7.7.1 in ASCE 7

$p_d = h_d \gamma$

$\gamma = 0.13 p_g + 14 = 0.13(30 psf) + 14 = 17.9 pcf$
max. intensity of drift surcharge load.

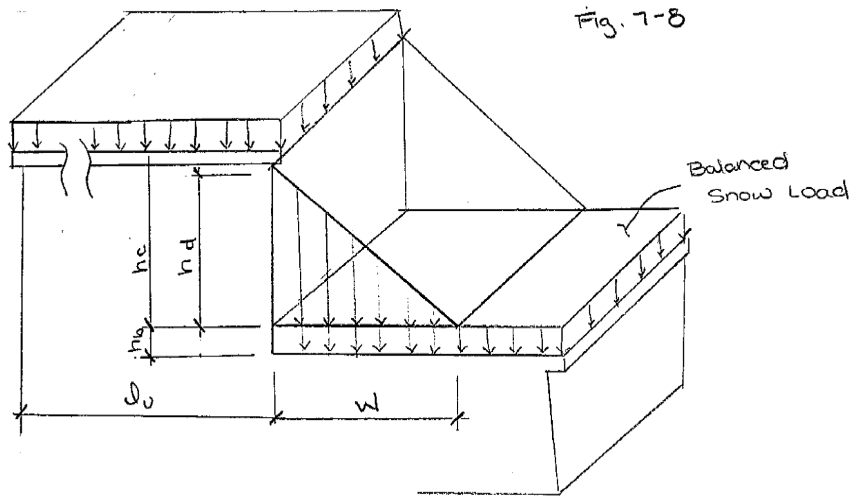
l_u = length of roof upwind of the drift = 66'-4" (E/W WIND)
= 23'-3" (N/S-WIND)

$h_d = 0.43 \sqrt[3]{l_u} \sqrt[4]{p_g + 10} - 1.5$
$= 0.43 \sqrt[3]{66.3} \sqrt[4]{30+10} - 1.5 = 2.877'$ (E/W)
$= 0.43 \sqrt[3]{23.25} \sqrt[4]{30+10} - 1.5 = 1.586'$ (N/S)

2|2

SNOW DRIFT (CONT)

Fig. 7-8



h_c = clear height from top of balanced snow load to closest upper roof.

h_b = height of balanced snow load P_s/γ

h_d = height of snow drift

W = Horizontal distance from eave to ridge
E|W: 25'-0"
N|S: 7'-7½", 11'-10"

Appendix B – Wind Loads

1/2

TECH REPORT 1
S. D'AGOSTINO

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

$q_z = 0.00256 \underbrace{K_z}_{\text{Varies at levels}} K_{zt} K_d V^2 I = 0.00256 K_z (1.0)(0.85)(110 mph)^2 (1.0)$

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

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

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

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

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

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

E/W $B = 64.66'$; $L = 134.33'$

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

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

$\boxed{G_{N/S} = 0.828}$ $\boxed{G_{E/W} = 0.9097}$

$q_h = 0.00256\,(1.06)(1.0)(0.85)(110)^2(1.0) = \boxed{27.909}$

2|2

TECH REPORT 1
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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
  Windward → $C_p = 0.8$
  Leeward → $L/B = 2.007$
  $C_p = -0.3$

Pressure $p_z = q_z G C_p - q_h (GC_{pi})$ Windward
$p_h = q_h G C_p - q_h (GC_{pi})$ Leeward

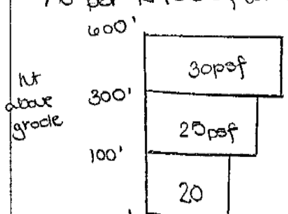
w| $GC_{pi} = +0.18, -0.18$ for enclosed bldgs (Fig. 6-5)

- NORTH|SOUTH
  Windward $p_z = (q_z)(0.828)(0.8) - 27.909(-0.18)$
  $\boxed{= (q_z)(0.6624) + 5.024}$
  Leeward $p_h = (27.909)(0.828)(-0.5) - 5.024 = \boxed{-16.574 \text{ psf}}$
- EAST|WEST
  Windward $p_z = (q_z)(0.9097)(0.8) + 5.024$
  $\boxed{= (q_z)(0.72776) + 5.024}$
  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
$V_{base} = 150^k$
$M_{base} = 9400^{'k}$

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

(DeSimone values look only at windward pressure)

My calculated base shears and moments (E|W: 147.88^k, $9266^{'k}$; N|S: 351.74^k, $24695.58^{'k}$) 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

Appendix C – Seismic Loads

Building Weight									
FLOOR 6									
Floor to Floor Height:		12.58	ft						
SLABS									
Area:		8113	sf	58	sf	310	sf		
Thickness:		9	in	15	in	14	in		
Unit Weight:		150	pcf	150	pcf	150	pcf		
Total Weight:		912.71	k	10.88	k	54.25	k		
COLUMNS									
Size	Quantity	Weight							
10 x 10	42	55.04	k						
22 x 22	4	25.37	k						
12 x 22	2	6.92	k						
12 x 18	2	5.66	k						
26 x 26	2	17.72	k						
12 x 36	1	5.66	k						
10 x 16	1	2.10	k						
	Total	118.46	k						
BEAMS									
Size	Quantity	Length		Weight					
8 x 20	2	20	ft	6.67	k				
8 x 20	2	23.25	ft	7.75	k				
8 x 20	1	19.5	ft	3.25	k				
		Total		17.67	k				
ADDITIONAL LOADS									
Superimposed (partitions, finishes, MEP)		25	psf	212.025	k				
		Total		212.025	k				
Total Weight				**1325.99**	**k**				

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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
$< T_{max} = C_u T_a = 1.7(0.789) = 1.34s$

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

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

2/2

TECH. REPORT 1
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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

$\Sigma w_i h_i^k = 1553453$ IK

$C_{vx} = \frac{w_x h_x^k}{\Sigma w_i h_i^k}$ = 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

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

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

$= 0.02756 W$

$> 0.012 W$ 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 $\left(\frac{ZIC}{R_w}\right)$, 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 D – Spot Checks

Two-Way Concrete Slab

1/8

TECH. REPORT.
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SPOT CHECK
9" SLAB (6th Floor - Refer to Figures 12+13)

As designed by DCE
- 9" slab
- f'_c = 5000 psi (NWC), f_y = 60,000 psi
- #4 @ 12 TOP & bottom w/ additional noted
- Two-way slab, no int. beams
- DL = 112.5 psf, LL = 40 psf, SDL = 20 psf

DIRECT DESIGN METHOD - ACI 318-08 : Chapter 13

Factored Loads = (1.2)(137.5 psf) + (1.6)(40 psf) = 0.229 ksf

MS = middle strip, CS = column strip

* All Panels shown in Fig 12 - Spot CHECK Locations *

PANEL A 22"x22" & 22"ϕ columns (B2 and C2)

$M_o = \frac{1}{8} w \ell_2 \ell_n^2 = \frac{1}{8}(20')(25 - \frac{22}{12})^2(0.229) = 307^{1k}$

$\alpha_A = 1.0$ (since flat plate $I_s = I_b$)

Min. thickness of slab w/o interior beams [Table 9.5c]
- interior panel w/o drop panels
$\rightarrow \frac{\ell_n}{33} = \frac{(25 - 22/12)(12)}{33} = 8.42 <$ design t = 9" OKAY!

A — M⁻ M⁺ M⁻

$M^- = 0.65\ M_o = 199.6^{1k}$
$M^+ = 0.35\ M_o = 107.45^{1k}$

$\ell_2/\ell_1 = 20/25 = 0.8$; $\alpha\ell_2/\ell_1 = 1.0(0.8) = 0.8$

ACI 318 § 13.6.4.1

ℓ_2/ℓ_1	0.5	0.8	1.0	
$\alpha\ell_2/\ell_1 = 0$	0.75	$\boxed{0.75}$	0.75	
$\alpha\ell_2/\ell_1 = 0.8$		$\boxed{0.798}$		$\rightarrow$ 79.8% of M^- to CS-A = 159.3^{1k}
$\alpha\ell_2/\ell_1 \geq 1.0$	0.90	$\boxed{0.81}$	0.75	20.2% of M^- to MS-A = 40.3^{1k}

2|8

TECH REPORT 1
S. D'AGOSTINO

BC - 9" SLAB (CONT) - PANEL A

ACI-318 § 13.6.4.4

l_2/l_1	0.5	0.8	1.0
$\alpha\, l_2/l_1 = 0$	0.60	$\boxed{0.60}$	0.60
$\alpha\, l_2/l_1 = 0.8$		$\boxed{0.768}$	
$\alpha\, l_2/l_1 \geq 1.0$	0.90	$\boxed{0.81}$	0.75

→ 76.8% of M^+ to CS A $= 82.5^{IK}$

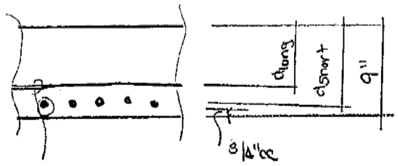
23.2% of M^+ to MSA $= 25.0^{IK}$

	M^-	M^+	M^-
Total M	-199.6	107.45	-199.6
CS	-159.3	82.5	-159.3
MS	-40.3	25.0	-40.3

PANEL A
TOTAL WIDTH = 20'
CS = 10' = 120"
MS = 10' (5 on either side) = 120"

- Design of Slab Reinf

Max spacing = 2·t = 2(9) = 18"
Min steel = temperature & shrinkage reinf.
$A_{s\,min} = 0.0018bt$ (for $f_y = 60$ ksi)



$d_{short} = t - cc - \frac{1}{2}\phi$
$d_{long} = d_{short} - \phi$

#4 → $d_{short} = 9" - 3/4" - 1/2(0.5") \approx 8"$
$d_{long} = 8" - 0.5 = 7.5"$

#5 → $d_{short} = 7.94"$
$d_{long} = 7.32"$

#6 → $d_{short} = 7.88"$
$d_{long} = 7.13"$

#4@12, #5@8, #6@8 (CS)
Mid, Left, Right

#4@12 (MS)

→
CONTINUE TO NEXT PAGE

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TECH REPORT 1
S. D'AGOSTINO

SC - 9" SLAB (CONT) - PANEL A

• DESIGN OF SLAB REINF IN. COLUMN STRIP (CS)

Item No.	Description	Left	Mid Interior Span	Right
1	Mn (IK)	-159.3	+82.5	-159.3
2	CS width, b (in)	120	120	120
3	Effective depth d (in)	7.32	7.5	7.13"
4	$M_U = M_n/\phi = M_n/0.9$ (k)	-177	91.7	-177
5	$M_U(12")/b$ ($\frac{k\text{-}ft}{ft}$, $\frac{k\text{-}in}{in}$)	-17.7	9.17	-17.7

$$\text{Check } d_{min} = \sqrt{\frac{M_U(12)(1000)}{\phi \rho b f_y (1-0.59 \rho \frac{f_y}{f'_c})}} = \sqrt{\frac{17.7(1000)}{0.9(0.0243)(60000)(1-0.59(0.0243)(\frac{60}{5}))}}$$

$= 4.04" < 7.5", 7.32", 7.13"$ OKAY

Item No.	Description	Left	Mid Interior Span	Right
6	$R_U = M_U/bd^2$	-330	163	-348
7	ρ from Table A.5a (NDD)	0.00575	0.0028	0.0060
8	$A_s = \rho bd$	$\boxed{5.05}$	$\boxed{2.52}$	$\boxed{5.13}$
9	$A_{s\,min} = 0.0018bt$	1.94	1.94	1.94
10	$N = \frac{\text{larger of step 8 + 9}}{\text{Area}}$	16.29 → $\boxed{17}$	12.6 → $\boxed{13}$	11.7 → $\boxed{12}$
	$A_L = 0.31 in^2$; $A_M = 0.20$, $A_R = 0.44$			
11	$N_{min} = \frac{\text{width of slab}}{2t}$	6.66 → 7	7	7

CALCULATED	DCE
Int: M_L^- · 17#5	Int: M_L^- · 15#5
M_M^+ · 13#4	M_M^+ · 10#4
M_R^- · 12#6	M_R^- · 15#6

The reinf. calculated for the left and middle of CS-A exceed the # of bars determined by DCE. One possible explanation is that this calculation had a SDL of 25psf while DCE had SDL=20psf.

The reinf for the right end of CSA is less than what used by DCE. A reason for this difference may have to do with the fact it is next to the core shear walls which are drawing more moment. My calculation do not incorporate any lateral loading.

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TECH REPORT
S. DAGOSTINO

SC - 9" SLAB (CONT) - PANEL A

• DESIGN OF SLAB REINF IN MIDDLE STRIP (MS)

Item No.	Description	Int Span M^-	M^+
1	Mn (IK)	-40.3	25.0
2	MS width b (in)	120	120
3	Effective depth d (in)	7.5	7.5
4	$Mu = Mn/\phi = Mn/0.9$	-44.8	27.8
5	$Mu/10 \times 12''$ $\left(\frac{k\,ft}{ft} \cdot \frac{k\,in}{in}\right)$	-4.48	2.78
	check a_{min} = 4.13" < 7.5"		
6	$Ru = Mu/bd^2$	-79.6	49.4
7	ρ from Table A.5a (WDD)	0.0013	0.0008
8	$As = \rho bd$	1.17	0.72
9	$As\,min = 0.0018bt$	$\boxed{1.94}$	$\boxed{1.94}$
10	$N = \frac{\text{larger of step 8 \& 9}}{0.20}$	9.7 → $\boxed{10}$	$\boxed{10}$
11	$N_{min} = \frac{\text{width of MS}}{2t}$	6.66 → 7	7

Calculated: 10 #4	DCE: 10 #4

My calculated value matches to exactly what DCE used for the middle strip in this PANEL A. Because they correspond it is understood that my loads and calculations are comparable to what done by DCE

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TECH REPORT 1
S. D'AGOSTINO

SC · 9" SLAB (CONT) - PANEL B

PANEL B 22"x22" columns (C2 and C3)

$M_o = \frac{1}{8} w \ell_2 \ell_n^2 = \frac{1}{8}(25)(0.229)(19.5 - \frac{22}{12})^2 = 223.4^{K}$

$\alpha_B = 1.0$ (since flat plate $I_S = I_b$)

Min thickness of slab w/o int. beams [Table 9.5c]
- Interior panel w/o drop panels

$\rightarrow \frac{\ell_n}{33} = \frac{(19.5 - 22/12)(12")}{33} = 6.427" < t_{design} = 9"$

B (M⁻ at each support, M⁺ at midspan)

$M^- = 0.65 M_o = 145.2^{K}$
$M^+ = 0.35 M_o = 78.2^{K}$

$\ell_2/\ell_1 = 25/19.5 = 1.28$, $\alpha \ell_2/\ell_1 = (1.0)(1.28) = 1.28$

ACI 318 § 13.6.4.1

ℓ_2/ℓ_1	1.0	1.28	2.0
$\alpha \ell_2/\ell_1 \geq 1.0$	75	$\boxed{66.6}$	45

↳ 66.6% of M⁻ to CSB
= 96.7 K
33.4% of M⁻ to MSB
= 48.5 K

ACI 318 § 13.6.4.4

ℓ_2/ℓ_1	1.0	1.28	2.0
$\alpha \ell_2/\ell_1 \geq 1.0$	75	$\boxed{66.6}$	45

↳ 66.6% of M⁺ to CSB
= 52.1 K
33.4 of M⁺ to MS·B
= 26.1

	M⁻	M⁺	M⁻
Total M	-145.2	78.2	-145.2
CS	-96.7	52.1	-96.7
MS	-48.5	26.1	-48.5

PANEL B
TOTAL WIDTH = 25'
CS = 12'-6" = 150"
MS = 12'-6" = 150"
(6'-3" on either side)

#6 @ 8 Left & Right (CS)
4 @ 12 middle
4 @ 12 (MS)

#4 → $d_s = 8"$
$d_L = 7.5"$

#6 → $d_s = 7.88"$
$d_L = 7.13$

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TECH. REPORT 1
S. D'AGOSTINO

8C · 9" SLAB (cont) - PANEL B

∘ DESIGN OF SLAB REINF IN COLUMN STRIP (CS)$_{int}$

Item No.	Description	M⁻	M⁺
1	Mn (lk)	-96.7	52.1
2	CS width b (in)	150	150
3	Effective depth	7.13	7.5
4	$M_U = Mn/\phi = Mn/0.9$	-107.4	57.9
5	$M_U(12")/b$ $\left(\frac{k\cdot ft}{ft}\cdot\frac{k\cdot in}{in}\right)$	-8.6	4.632

* check $d_{min} = \sqrt{\dfrac{8.6\,(1000)}{(0.9)(0.0243)(60000)(1-0.59(0.0243)(\frac{60}{5}))}} = 2.82"$

$< 7.13", 7.5" = d$

Item No.	Description	M⁻	M⁺
6	$R_U = M_U / bd^2$	-169.0	82.3
7	ρ from table A.5a (NDS)	0.0029	0.0014
8	$A_S = \rho bd$	$\boxed{3.10}$	1.575
9	$A_{Smin} = 0.0018\,bt$	2.43	$\boxed{2.43}$
10	$N = \dfrac{\text{larger Step 8 \& 9}}{\text{Area}}$ $A_6 = 0.44\,in^2$, $A_4 = 0.20\,in^2$	7.05 → 8	12.15 → $\boxed{13}$
11	$N_{min} = \dfrac{\text{Width of CS}}{2t}$	8.33 → $\boxed{9}$	9

Calculated	DCE
Int: M⁻ = 9#6	Int: 12#6 = M⁻
M⁺ = 13#4	13#4 = M⁺

The reinf calculated at the supports was 9#6 vs. DCE 12#6. A possible reason is because this span is located next to the core shear walls which, due to the relative stiffness, draw more load. Because we are not looking at lateral loading additional moments in this region are unknown. I would expect to have to increase the # of bars once lateral loads are incorporated in the analysis.

As for the middle bars, my values matches exactly with DCE designed reinf.

SC. 9" SLAB (CONT) PANEL B

• DESIGN OF SLAB REINF. IN MIDDLE STRIP (MS)

Item No.	Description	int M^-	M^+
1	M_n	-48.5	26.1
2	MS width b (in)	150	150
3	Effective depth d (in)	7.5	7.5
4	$M_u = M_n/\phi = M_n/0.9$	-53.9	29.0
5	$M_u(12")/b$ $\left(\frac{k\text{-}ft}{ft}, \frac{k\text{-}in}{in}\right)$	-4.3	2.3

$$d_{min} = \sqrt{\frac{4.3(1000)}{0.9(0.0243)(60000)(1-0.5(0.0243)(\frac{60}{5})}} = 2.0" < 7.5" = d$$

Item No.	Description	int M^-	M^+
6	$R_u = M_u/bd^2$	-76.7	41.2
7	ρ from Table A.5a (NDD)	0.00125	0.00065
8	$A_s = \rho bd$ (in²)	1.41	0.731
9	$A_{s\,min} = 0.0018bt$	$\boxed{2.43}$	$\boxed{2.43}$
10	$N = \frac{\text{larger of Step 8 \& 9}}{0.20}$	12.15 → $\boxed{13}$	$\boxed{13}$
11	$N_{min} = \frac{\text{width of MS}}{2t}$	8.33 → 9	9

Calculated = 13 #4 DCE = 13 #4

My calculated reinf is exactly the same as what designed by DCE. From this it is assumed that I applied my loads correctly and that my means of calculating MS reinf is comparable to that of DCE.

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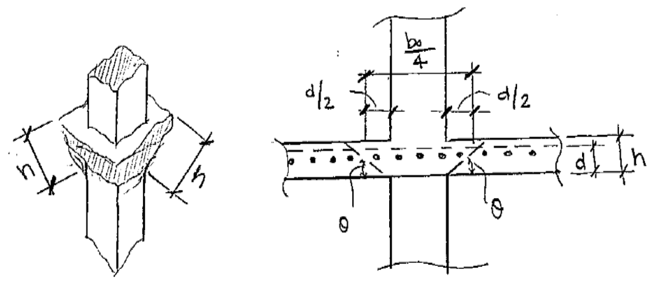
TECH REPORT 1
S. DAGOSTINO

5C · 9" SLAB (CONT) PUNCHING SHEAR

SHEAR DESIGN IN FLAT PLATE SLABS
§ 13.10 - Design of Concrete Structures



PUNCHING SHEAR

Checking:
COLUMN 2C
& 9" SLAB

- 22"x22"
- #6 @ 8" in both directions
- $f'_c = 5$ ksi

$V_c = 4\sqrt{f'_c}\, b_o d$ [Eq. 13.11a]

$d = 9" - 3/4" - \frac{1}{2}(0.75") = 7.875"$
℄ Ø#6

$\frac{b_o}{4} = 2(d/2) + 22" = 2\left(\frac{7.875}{2}\right) + 22" = 29.875" \rightarrow b_o = 119.5"$

$V_c = 4\sqrt{5000}\,(119.5")(7.875") = 266.2^k$

Must be $\leq$ $V_c = \left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f'_c}\, b_o d$

$\alpha_s = 40$ for interior columns

$V_c = \left(\frac{40(7.875)}{119.5} + 2\right)\sqrt{5000}\,(119.5)(7.875)$
$= 308.5^k$ GOOD!

$\phi V_c \geq V_u$
$\phi V_c = (0.75)(266.2^k) = 199.7^k$

$V_u = (0.229\text{ ksf})\left(494\text{ sf} - \left(\frac{22}{12}\right)^2\right) = 112.4^k$
Trib Area - Column Are

$\phi V_c = 199.7 \geq V_u = 112.4^k$ GOOD!

Because $\phi V_c \geq V_u$ the design does not require special shear reinf. and there are no punching shear related concerns at column 2C-level 6. DOE does not note that any special reinf is req'd either.

Column 2C

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Tech Report 1
S. D'Agostino

SPOT CHECK
COLUMN - 2C

* Eq. 10-2 - ACI 318-08

$\phi P_{n,max} = 0.8\phi\left[0.85 f'_c (A_g - A_{st}) + f_y A_{st}\right]$

* All loads located in spreadsheet (using LC = 1.2DL + 1.6LL + 0.5S)

SUPPORTING PENTHOUSE

As designed by DCE: 10x10, 4#7, #3@10, $f'_c = 5ksi$

$A_g = 100 in^2$
$A_{st} = 4(0.60) = 2.4 in^2$

$\phi P_n = 0.8(0.65)\left[0.85(5)(100-2.4) + (2.4)(60)\right] = 290.6^k$

$\boxed{P_u = 22.78^k < 290.6^k}$ OKAY!

SUPPORTING LEVEL 10

As designed by DCE: 10x10, 4#7, #3@10, $f'_c = 5ksi$

$A_g = 100 in^2$
$A_{st} = 4(0.60) = 2.4 in^2$

$\phi P_n = 0.8(0.65)\left[0.85(5)(100-2.4) + 2.4(60)\right] = 290.6^k$

$\boxed{P_u = 38.43^k < 290.6^k}$ OKAY!

SUPPORTING LEVEL 9

As designed by DCE: 10x10, 4#7, #3@10, $f'_c = 5ksi$

$A_g = 100 in^2$
$A_{st} = 4(0.60) = 2.4 in^2$

$\phi P_n = 0.8(0.65)\left[0.85(5)(100-2.4) + 2.4(60)\right] = 290.6^k$

$\boxed{P_u = 54.08^k < 290.6^k}$ OKAY!

SUPPORTING LEVEL 8

As designed by DCE: 10x10, 4#7, #3@10, $f'_c = 5ksi$

$A_g = 100 in^2$
$A_{st} = 4(0.60) = 2.4 in^2$

$\phi P_n = 0.8(0.65)\left[0.85(5)(100-2.4) + 4(0.60)\right] = 290.6^k$

$\boxed{P_u = 69.72^k < 290.6^k}$ OKAY!

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Tech Report 1
S. D'Agostino

Column 2C (cont)

Supporting Level 7 (* increase load due to transfer)
As designed by DCE • 22"x22" $A_g = 484 in^2$
12#6 $A_s = 5.28 in^2$
#3@10
$f'_c = 5 ksi$

$$\phi P_n = 0.8(0.65)[0.85(5)(484-5.28) + (5.28)(60)] = 1222.7^k$$

$\boxed{P_u = 325.97^k < 1222.7^k}$ OKAY!

Supporting Level 6
As design by DCE • 22"x22" $A_g = 484 in^2$
12#6 $A_s = 5.28 in^2$
#3@10
$f'_c = 5 ksi$

$$\phi P_n = 0.8(0.65)[0.85(5)(484-5.28) + 5.28(60)] = 1222.7^k$$

$\boxed{P_u = 452.68^k < 1222.7^k}$ OKAY!

Supporting Level 5
As designed by DCE • 22"x22" $A_g = 484 in^2$
12#6 $A_s = 5.28 in^2$
#3@10
$f'_c = 5 ksi$

$$\phi P_n = 0.8(0.65)[0.85(5)(484-5.28) + 5.28(60)] = 1222.7^k$$

$\boxed{P_u = 579.39^k < 1222.7^k}$ OKAY!

Supporting Level 4
As designed by DCE • 22"x22" $A_g = 484 in^2$
12#6 $A_s = 5.28 in^2$
#3@10
$f'_c = 8 ksi$

$$\phi P_n = 0.8(0.65)[0.85(8)(484-5.28) + 5.28(60)] = 1887.5^k$$

$\boxed{P_u = 706.10^k < 1887.5^k}$

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Tech Report 1
S. D'Agostino

COLUMN 2C (cont)

SUPPORTING LEVEL 3

As designed by DCE • 22"x22"
12#7
#3@12
$f'_c = 8ksi$

$A_g = 484 in^2$
$A_{st} = (0.60)(12) = 7.2 in^2$

$$\phi P_n = 0.8(0.65)[0.85(8)(484-7.2) + (7.2)(60)] = 1910.6^k$$

$\boxed{P_u = 832.82^k < 1910.6^k}$ OKAY!

SUPPORTING LEVEL 2

As designed by DCE • 22"x22"
12#7
#3@12
$f'_c = 8ksi$

$A_g = 484 in^2$
$A_{st} = (0.60)(12) = 7.2 in^2$

$$\phi P_n = 0.8(0.65)[0.85(8)(484-7.2) + (7.2)(60)] = 1910.6^k$$

$\boxed{P_u = 949.52^k < 1910.6^k}$ OKAY!

SUPPORTING LEVEL 1

As designed by DCE • 22"x22"
12#7
#3@12
$f'_c = 8ksi$

$A_g = 484 in^2$
$A_{st} = 7.2 in^2$

$$\phi P_n = 0.8(0.65)[0.85(8)(484-7.2) + (7.2)(60)] = 1910.6^k$$

$\boxed{P_u = 1066.23^k < 1910.6^k}$ OKAY!

CONCLUSIONS:

The loads calculated were actually higher than those noted by DeSimone Consulting Engineers (DCE) due to my assumptions on loading (located in this report). As seen in the previous calculations, all columns are more than sufficient to hand the loads within this structure. The reason behind the large gap between capacity and present loads may be attributed to the fact that only axial loads were considered in these preliminary calculations. Moments are likely to also be present, however, DCE designed columns are more than sufficiently designed and will pass checks.

All columns designed satisfactorily in relation to gravity analysis.