CENTRE COURT APARTMENTS
STATE COLLEGE, PA

Anthony Dente
Thesis Tech Three
Advisor: Ali Memari
12/03/07

Executive Summary

This report documents the analysis of the lateral resisting system of the Centre Court Apartment Building in State College, Pennsylvania. The Lateral resisting system consists of the structure consists of the 8" and 10" load bearing CMU walls. The walls contain no reinforcement on the top two floors and carry down to being fully grouted and reinforced with #5 bars. The gravity load resisting system consists of precast hollow core slabs that bear either on the exterior CMU walls or the interior Wide flange girder to column system.

The Lateral system was analyzed through ETABS 9.1.1 by converting the CMU's to an equivalent concrete shearwall system. The program analyzed the percentages of lateral loads distributed to each shearwall, the story drift at each level of the structure as well as the torsion effects on the structure. These results were then compared against code required serviceability restraints for drift and overturning moment effects on the foundation.

It was concluded that the Centre Court Apartment Building meets all drift requirements. It was also found that the foundation receives little effects from overturning moment due to lateral loads. This is due to the high mass of the CMU system itself.

Contents

Existing Structural System

Listed below are the prominent structural elements contained in Centre Court Apartments:

- 8” CMU exterior above grade and 10” CMU exterior below grade
  - Load bearing units conforming to ASTM C90
  - Net Compressive Stress = 3000 PSI
  - Above grade CMU’s contain Dur-O-Wall every other course
  - Block cells with bars are grouted a minimum 2 courses below plank bearing

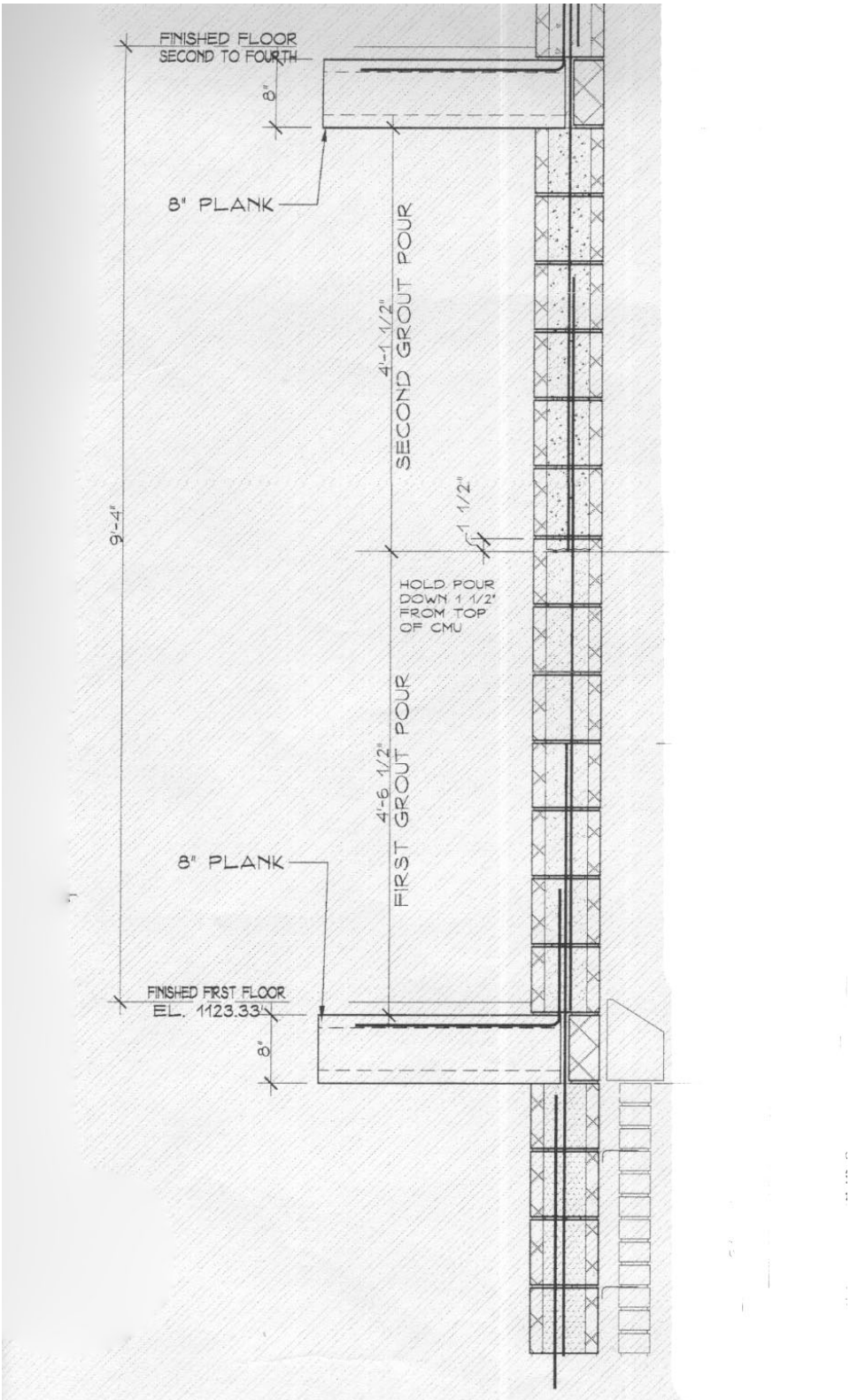
- 8” pre-cast hollow core planks
  - Conform to latest edition of ACI 318
  - Steel bearing will contain weld plates spaced 4’ O.C. max.
  - F’c=5000 PSI

- Steel beams and columns
  - Typical beam sizes: W12 X 26 and W14 X 43
  - Grade 50 or ASTM A992
  - Fabricated and erected in accordance to the latest edition of AISC specifications.

- Concrete columns, footings, and slabs
  - Mixed and placed in accordance with ACI 318 “Building Code Requirements for Concrete”
  - Footings and slabs f’c = 3000
  - Columns f’c =4000

Lateral System

The lateral system is comprised of the 8” and 10” CMU walls that wrap the main elevator and stairwell cores. The top two stories of CMU’s are unreinforced. Further down the structure the blocks are grouted where the #5 bars are present until you reach the bottom two floors which are grouted solid. See section below.

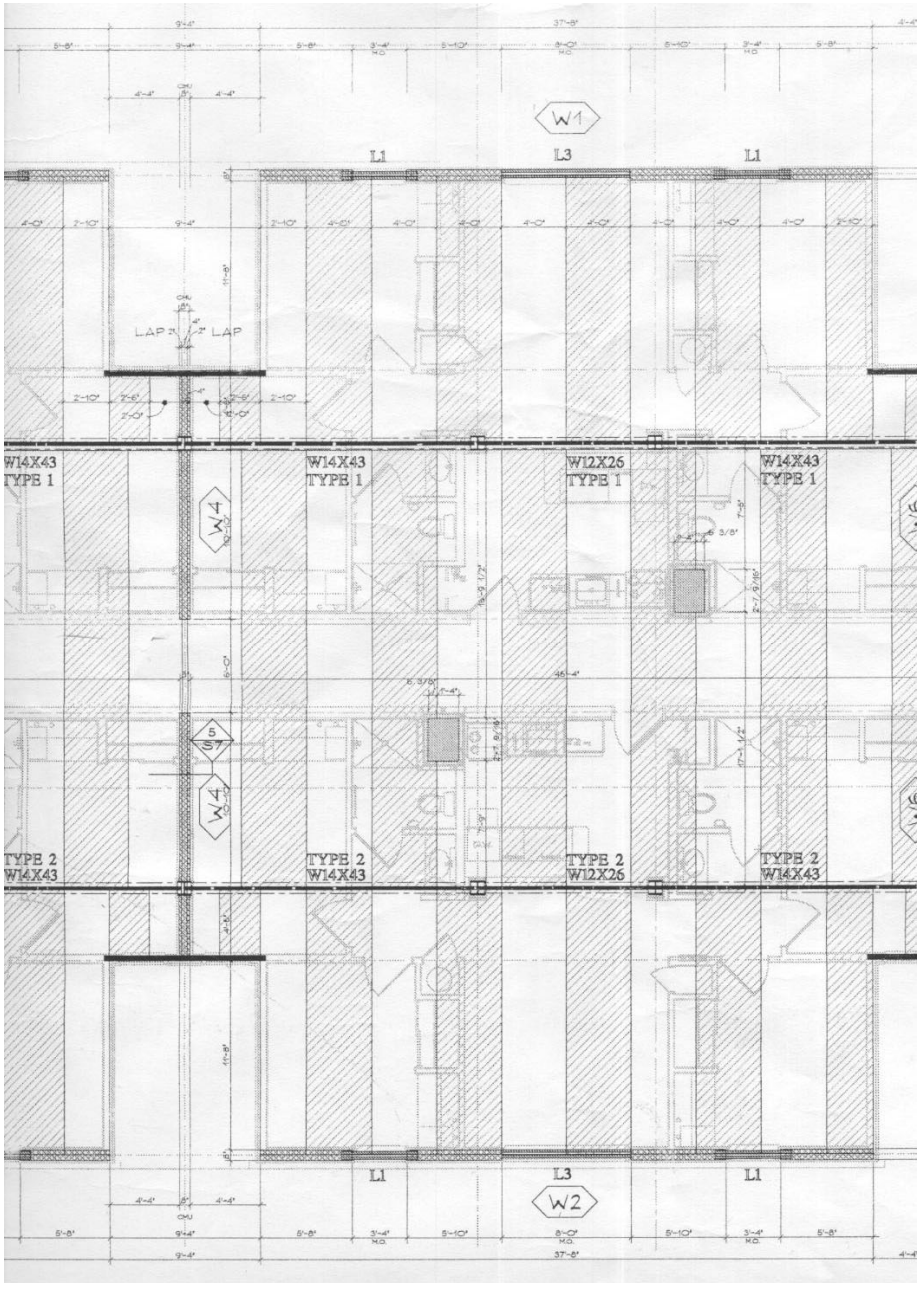


TYPICAL MASONRY WALL SECTION
FIRST TO FOURTH FLOOR LEVELS 2/S5

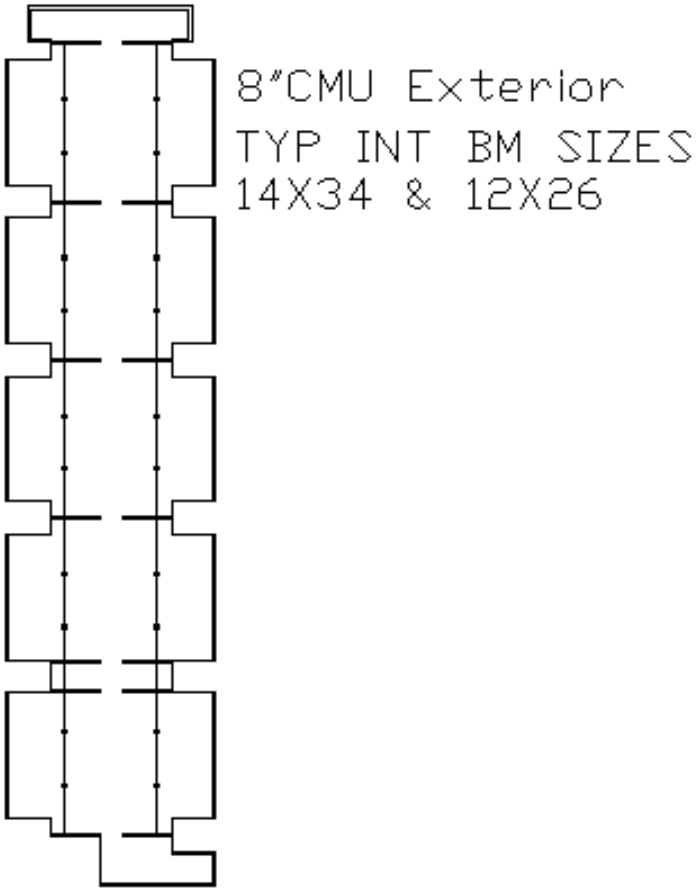
SCALE: 3/4" = 1'-0"

Gravity System

The vertical loads of the building are also mostly carried to the footing by the CMU system wrapping the exterior of the structure. The 8” precast hollow core planks distribute the floor loads to the blocks on the exterior of the building and a girder to column grid in the interior of the structure. The typical beam sizes are W12 X 26 and W14 X 43 which distribute the load to a series of W14 X 90 columns that then carry it to the concrete footings below. See typical structural bay below.



Existing Structural Plan



Codes and References

- The International Building Code 2003
- The American Concrete Institute
  - Section 530.1: Masonry
- The American Institute of Steel Construction
- CRSI 2002: Concrete Reinforcing Steel Institute
- United Steel Deck Design Manual 2002

Loads

Load Combinations

The Load Combinations of Chapter 2 in ASCE 7-05 are listed bellow. Each of these load combinations were analyzed via the ETABS modeling system in computing the lateral reactions listed below.

1. 1.4(D + F)
2. 1.2(D + F + T) + 1.6(L + H) + 0.5(Lr or S or R)
3. 1.2D + 1.6(Lr or S or R) + (L or (0.8W)
4. 1.2D + 1.6W + L +0.5(Lr or S or R)
5. 1.2D + 1.0E + L + 0.2S
6. 0.9D + 1.6W + 1.6H
7. 0.9D + 1.0E + 1.6H

Gravity Loads

Gravity Loads have been calculated in accordance with ASCE 7-05 with the Live Loads interpreted from section four. These loads where used to calculate masses and uniform loads applied to the ETABS structural model of the Centre Court Apartment Building of with the results are listed below. Assumptions were made for proper distribution of Gravity Loads.

Dead Load

Hollow Core Planks	60	psf
Concrete	150	pcf
Partitions	15	psf
MEP	10	psf
Misc.	5	psf
Brick	38	psf
8' CMU	60	psf
Windows	8	psf

Live Loads

Corridors	100	psf
Garages	40	psf
Private Rooms	40	psf
Public Rooms	100	psf
Roof	20	psf
Snow	21	psf

Wind Loads

Per the results of Technical Assignment One it was concluded that wind is the controlling lateral force of the Centre Court Apartments. This result backs up previous assumptions due to the buildings location in Centre County Pennsylvania and the size of the structure.

The lateral drift of the structure due to wind was calculated through the ETABS Structural Analysis Model created of the building. The loads applied to the model were calculated through ASCE-7 '05 Chapter 6 and was treated as a solid rectangular mass, neglecting minor indentations and curvatures of the façade. The Center Court Apartments were found to be an Exposure Category B, with an importance factor of 1.0 and with wind speed V=90mph. Refer to appendix for full listing of wind calculation parameters.

Drifts Calculated Through ETABS Analysis

	N/S	**E/W**
Roof :	0.5"	0.34"
Story 4:	0.45"	0.27"

Wind Analysis																		
					Pressures													
					N/S			E/W			Area	Forces (kip)		Shear(kip)		Moment (ft.k)		Area
Ht.	L	hx	Kz	qz	Windwa	Leeward	Side Wa	Windwai	Leewar	Side Wal	N/S	N/S	E/W	N/S	E/W	N/S	E/W	E/W
9.21	R	67.518	0.89	15.7	11.00	-6.59	-9.23	10.67	-2.64	-9.23	2,468	43.41	7.72	0	0	2,931	521	580
9.33	4	58.31	0.85	15.0	10.51	-6.59	-9.23	10.19	-2.64	-9.23	2,500	42.75	7.54	43	8	2,493	440	588
9.33	3	48.98	0.81	14.3	10.01	-6.59	-9.23	10.01	-2.64	-9.23	2,500	41.52	7.43	86	15	2,033	364	588
9.33	2	39.65	0.76	13.4	9.39	-6.59	-9.23	9.39	-2.64	-9.23	2,500	39.97	7.07	128	23	1,585	280	588
9.33	1	30.32	0.71	12.5	8.78	-6.59	-9.23	8.78	-2.64	-9.23	2,500	38.43	6.71	168	30	1,165	203	588
10.7	P2	20.99	0.63	11.1	7.79	-6.59	-9.23	7.79	-2.64	-9.23	2,953	42.46	7.89	206	36	891	166	757
10.3	P1	10.33	0.57	10.0	7.05	-6.59	-9.23	7.05	-2.64	-9.23	2,861	39.02	7.10	249	44	403	73	733
												287.56	51.46	**288**	**51**	**11,502**	**2,047**	
		Internal Pressure=			qh(Gcpi)= **+/- 2.792 lb/sf**													

Seismic Loads

The seismic design loads were calculated using Section 11 of ASCE 7-05 the equivalent lateral force design method. The original building period was used because the average period being displayed by the model's animation sequences was within close proximity to the originally calculated period of 0.199s. See appendix for the complete list of deign parameters.

Drifts Calculated Through ETABS Analysis

	N/S	E/W
Roof :	0.46"	0.44"
Story 4:	0.34"	0.38"

Level	Wx (kips)	hx (ft)	$Wxhx^k$	Cvx	Load Fx (K)	Shear Vx (K)	Moment Mx (FT.K)
Roof	1,369	67.54	173,995.29	0.22	52.40	0.00	3,539.19
5	1,675	58.33	179,845.13	0.23	54.16	52.40	3,159.34
4	1,683	49.00	147,867.18	0.19	44.53	106.56	2,182.09
3	1,683	39.66	115,944.97	0.15	34.92	151.10	1,384.87
2	1,683	30.33	85,172.53	0.11	25.65	186.02	778.00
1	1,802	21.00	59,735.32	0.08	17.99	211.67	377.79
P1	1,524	10.34	22,369.34	0.03	6.74	229.66	69.66
Totals	11,420		784,929.76		236.39	**236.39**	**11,490.95**

ETABS Analysis

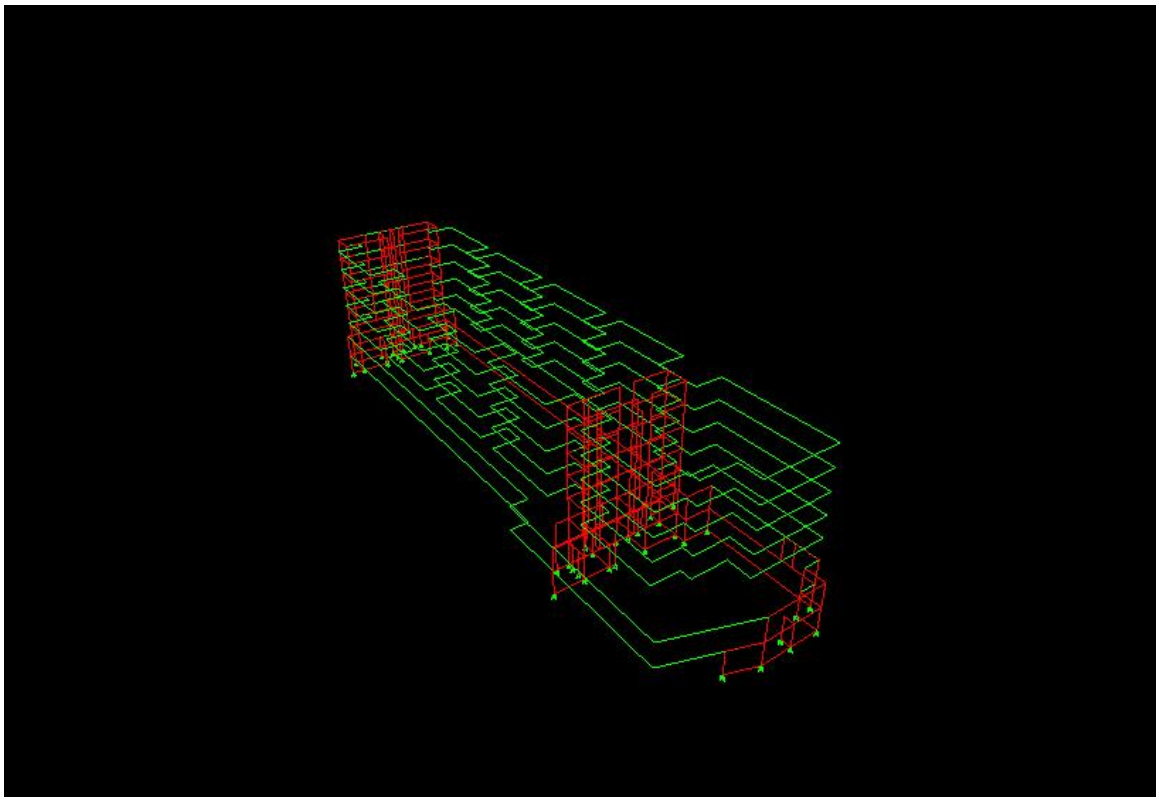
ETABS version 9.1.1 was used in the modeling on the Centre Court Apartments lateral resisting system. ETABS, like most structural analysis programs, is not designed to conveniently analyze CMU structures. Therefore the hollowcore blocks needed to be represented as solid concrete shearwalls that represent the same structural qualities. The analysis then consisted of modeling the entire shape of the building for mass purposes and the detailing of the shear resisting system. Information from the structural engineer indicated that all CMU walls that continued to the ground were designed to participate in the lateral resisting system. Therefore all walls of these characteristic were modeled and analyzed, and per the results walls were removed to until only the required lateral resisting system remained. The results indicated that only the two main stairwell and elevator cores represented the main lateral resisting systems. From east to west (left to right on the diagram in the appendix) the percentages or the loads in the north/south direction carried by the shear walls was found to be 3.6%, 34%, 31.2%, 31.3% respectively on the 5th story. On the 3rd story of apartments the percentages were calculated as 8.7%, 28%, 31.6%, and 31.5%. A spot check on the wall resisting these loads was doe and is present in the appendix.

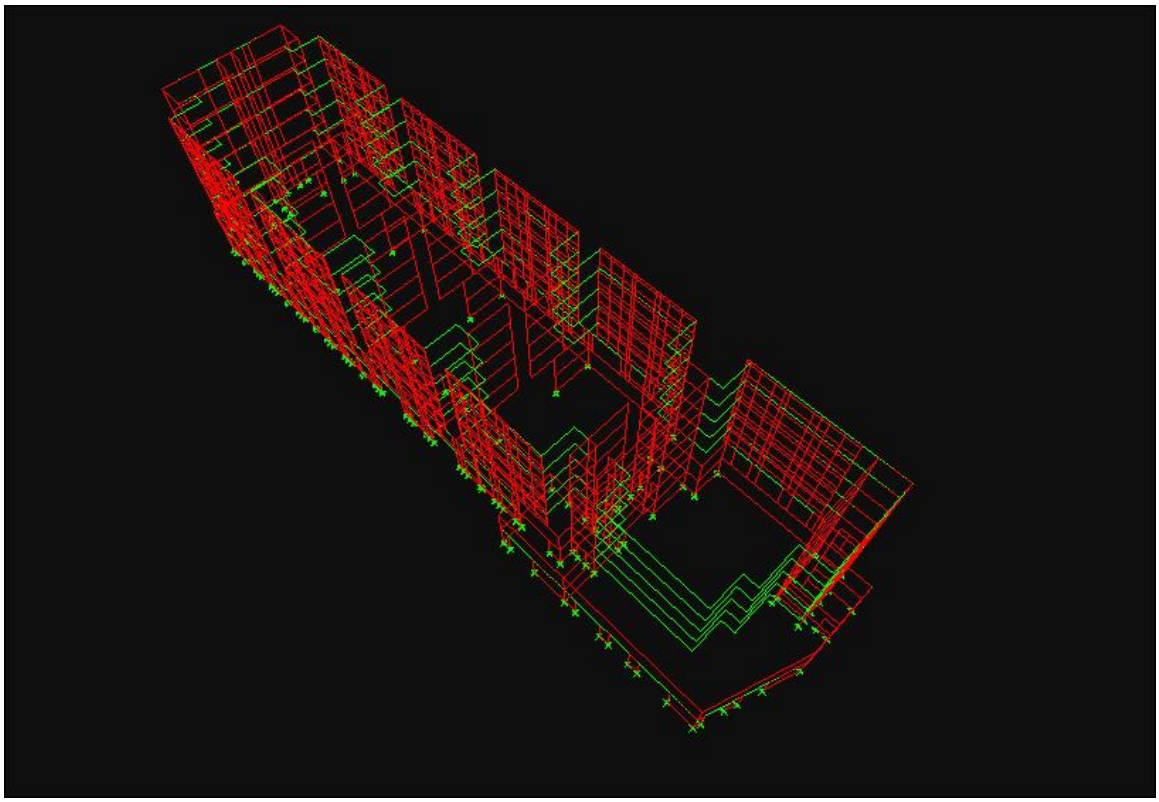
Conclusion

The analysis of the lateral system in The Centre Court Apartments resulted in the conclusion that only the main elevator shaft and the East stairwell core were required to resist the lateral loads applied to the structure. Seismic code specified serviceability limits for story drift from IBC Table 1617.3 of $\Delta_s = 0.015h_{sx}$ as well as limits for wind loads of $\Delta_w = H/400$ were satisfied. The ETABS model proved to be an excellent source to pinpoint the load percentages carried by each shearwall as they carry down the structure. Although, due to the questionably low drift values, if the analysis was to be done again and or more time were permitted a full hand calculated analysis of story drift would be recommended. This is only due to the inaccuracies that may have resulted in the process of adapting CMU shear walls to the ETABS modeling program which does not traditionally entertain such materials.

The final analysis of the overturning moment found that the resisting moment caused by the building's self weight was much higher than the largest overturning moment brought about by lateral forces. Therefore, no new tension will need to be considered for foundation design.

Appendix





ASCE7-05
Wind Analysis
6.5 Method 2- Analytical Procedure

Height	h=	67.54	Ft
Basic Wind Speed	V=	90	mph
Enclosure Classification		Enclosed	
Wind Directionality	Kd=	0.85	
Importance Factor	I=	1	
Exposure		B	
Pressure Coefficient	Kh=	0.88	
Topographic Factor	Kzt=	1	
Gust Effect Factor	G=	0.85	
Velocity Pleasure	qz=	17.626	
	GCpi=		
Internal Pressure Coef.	+/-	0.18	

N/S

Base Shear Results	288	Kip
Overturning Moment	11502	Ft.Kip

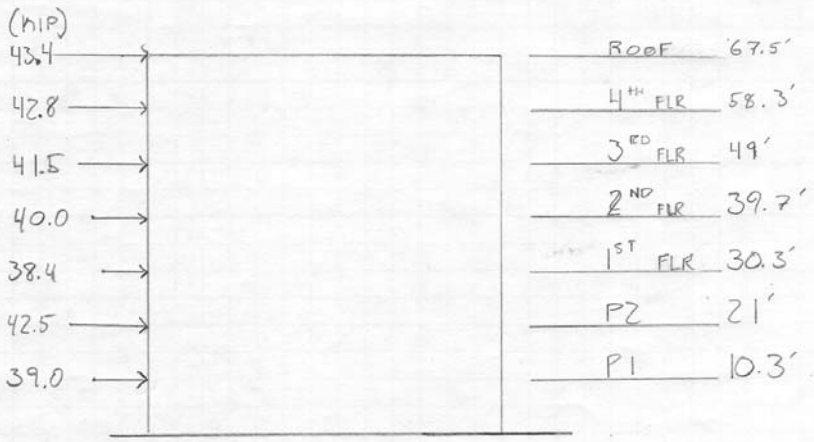
E/W

Base Shear Results	51	Kip
Overturning Moment	2047	Ft.Kip

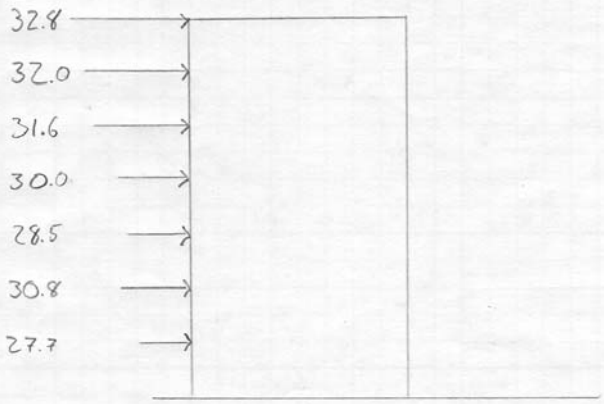
Seismic Analysis
Section 11

Site classification		B	
Seismic Design Category	SDC	A	
Seismic importance factor	I	1	
Short period spectral response	Ss	0.157	g
Acceleration based Site coefficient	Fa	1	
Maximum short period spectral response	SDS	0.105	
Spectral Response at 1 sec 1	S1	0.05	g
Velocity based site coefficient	Fv	1	
Maximum spectral response at 1 sec	SD1	0.033	g
Response modification factor	R	2	
building period	T	0.799	s
Long period transition period	TL	6	s
N/S Seismic design coefficient	CS	0.0207	

WIND FORCES

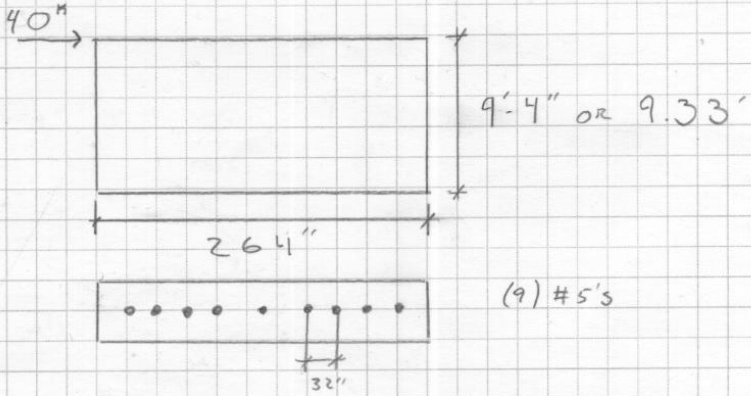


N/S



E/W

Shearwall Spot Check
Center System 3rd Story



$S_v = \frac{V}{bd}$ $b = 7.625"$ (8" CMU)
$V = 40^{k}$

ASSUME $c = 7"$
$a = 0.85(7) = 5.95$

$X = 0.5(a) = 0.5(5.95) = 2.975$

$A_s F_y = A_s F_s + 0.85 f'_m b a$

$7(.31)60 = 1(.31)60 + 0.85(3)(7.625)(5.95)$

$130.2 = 134.3 \rightarrow c = 7$ is ok

$d = 264 - 2.975 \therefore d = 261"$

$S_v = \frac{40}{7.625(264)} = 20$ PSI

$M/Vd = \frac{0.5(9.33 \times 12)40}{40(264)} = 0.21 < 1$ ✓

$F_v = 0.5\left[4 - \frac{M}{Vd}\right]\sqrt{f'_c} \leq 120 - 45\left(\frac{M}{Vd}\right)$

$0.5[4 - (.21)]\sqrt{5000} = 103.79$

$120 - 45(.21) = 110.5$

$103.79 \leq 110.5$ ✓

$20^{k} < 103.79^{k}$

$s_v < F_v$ ✓

VERTICAL REINF. CHECK

$A_v = \frac{V_s}{F_s d} = \frac{40^{k}(32'')}{60(240)} = 0.08$

$\frac{1}{3} A_v = \frac{1}{3}(0.08) = 0.026 < 0.31$ ✓

Torsion Analysis

$$K = \frac{1}{\Delta_c}$$

$$\Delta_c = \frac{Vh^3}{3E_M I} + \frac{1.2Vh}{E_V A}$$

$E_M = 2,250,000$ psi $E_V = 0.4(2,250,000) = 900,000$ psi

STIFFNESSES IN N/S DIRECTION

WALL LENGTHS 22', 15', 49'

$$I_{22'} = \frac{7.625(22\times12)^3}{12} = 11.7E^6 \text{ in}^4$$

$I_{15'} = 3.7E^6$

$I_{49'} = 129.2E^6$

$I_{10'} = 1.1E^6$

$V = 42^k$ - 5TH STORY

$$\Delta_{c22'} = \frac{(42)(9.33\times12)^3}{3(2250,000)(11.7E^6)} + \frac{1.2(42)(9.33\times12)}{(900,000)(9.33\times22)}$$

(1710)

$= 2.13\ E^{-7}$

$K_{22'} = 4,694$

$\Delta_{c15'} = 3.14\ E^{-7}$

$K_{15'} = 3184$

$\Delta_{c49'} = 9.53E^{-8}$

$K_{49'} = 10,490$

TORSION

$\Sigma Kd = 10,490(255 \times 12) + 10,490(265 \times 12) + 58'(12)(4694 + 3184) + 67'(12)(4694 + 3184)$

(49' walls; 22' & 15' walls)

$77,274,600$

$V_T = \frac{Ve\,(d \cdot R_o)}{J}$

CENTER OF MASS $x = 140'$

CENTER OF RIGIDITY

$\Sigma K's = 36736$

↳ 2 WALLS OF EACH LENGTH

$\frac{77,274,600}{36736\,(12')}$ = CENTER OF RIGIDITY $x = 175.3$

$\therefore e = 175.3 - 140 = 35.3'$

$J = \Sigma R_i d_i^2$

$J = 2.13\ e''$

d OF 22' SHEAR WALL SPOT CHECKED ABOVE

$V_T = 3^K$

OF THE 40^K LOAD 3^K WAS CAUSED BY TORSION