CENTRE COURT APARTMENTS
STATE COLLEGE, PA

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Executive Summary

The Centre Court Apartments are located in the borough of State College, Pennsylvania and were designed by Frederick J. Fernsler, AIA with Jesse Smith, PE as the structural engineer. L. S. Fiore Construction was the general contractor on the project that completed it's 16 month construction process in August 06'. The total cost of the project was $13.6 million, which includes a large addition to an adjacent building which will not be covered in this report. The building stands at 67.5 feet and contains five levels of student housing atop two levels of parking, intermixed with lobby and commercial area on the ground floor.

This report analyzes the loads bearing on the structure as well as spot checks of key structural elements of the design in place. ASCE 7-05 was the baseline used in calculating all gravity loads, dead, live and snow, as well as the lateral wind and seismic forces acting on the system.

The results exhibited that wind was the governing lateral force system in both base shear and overturning moment, therefore its analysis was used to spot check the shear resisting masonry system that wraps the exterior of the building. This system, as well as the other systems analyzed against the obtained load in this report, was found to be sufficient or justifiably oversized.

Contents

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

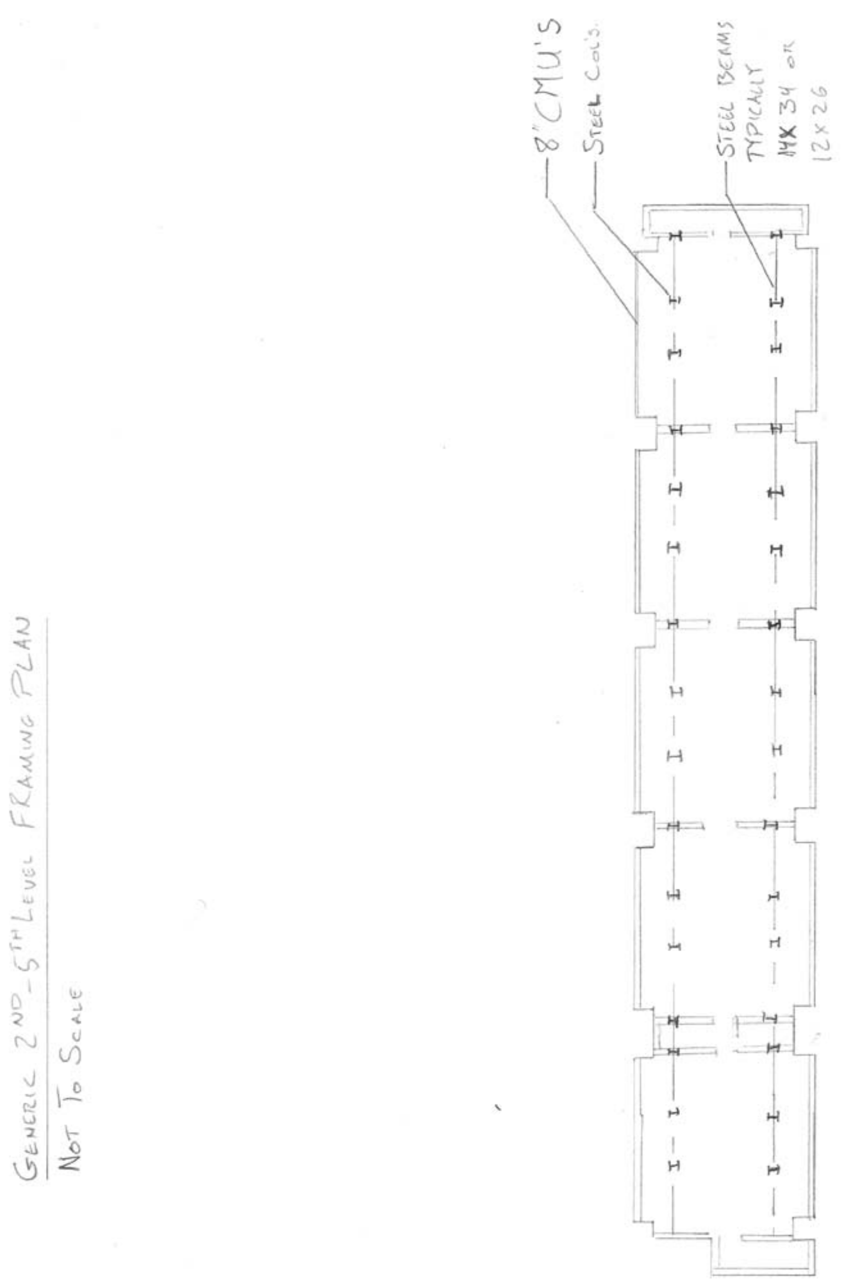
- Steel beams and columns
    - Typical beam sizes: 12 X 26 and 14 X 43
    - Grade 50 or ASTM A992
    - Fabricated and erected in accordance to the latest edition of AISC specifications.

- 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

Codes

- The International Building Code 2003
- The American Concrete Institute
    - Section 530.1: Masonry
- The American Institute of Steel Construction

Framing


8" CMU'S
STEEL COL'S.
STEEL BEAMS
TYPICALLY
14X 34 or
12x26
GENERIC 2ND - 5TH LEVEL FRAMING PLAN
NOT TO SCALE

Structural Design

The structural design is dominated by the load-bearing CMU exterior walls. This system also crosses north to south at particular portions of the interior building in order to be the primary lateral force resisting system. This structure has a number of benefits in the Centre Court Apartments. The added convenience of bearing the pre-cast hollow core slabs. Pre-cast hollow core concrete slabs make up at least 90% of all floor slabs in the building and the concrete to concrete block connection cuts down on the number of bearing plates that would be needed if the number of slab to steel connections were increased.

Another benefit of this system is the simplification of the beam to column connections throughout the building. Since no moment frames are required, all moment connections have been completely elevated from the building. There are also two non-structural benefits to the CMU design: the fire rating requirements for apartment buildings and the way it compliments the application of the aesthetic stucco applied to the exterior of the building.

The remaining interior loads are carried by a series of wide flange beams, which distribute that load to steel columns in the top five floors. The bottom two levels of parking deck then convert to concrete columns, which at the end drop the load onto the 6’ X 8’ spread footings.

Loads

Gravity Loads have been calculated in accordance with ASCE 7-05 with the Live Loads interpreted from section four. 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		
Coridors	100	psf
Garages	40	psf
Private Rooms	40	psf
Public Rooms	100	psf
Roof	20	psf
Snow	21	psf

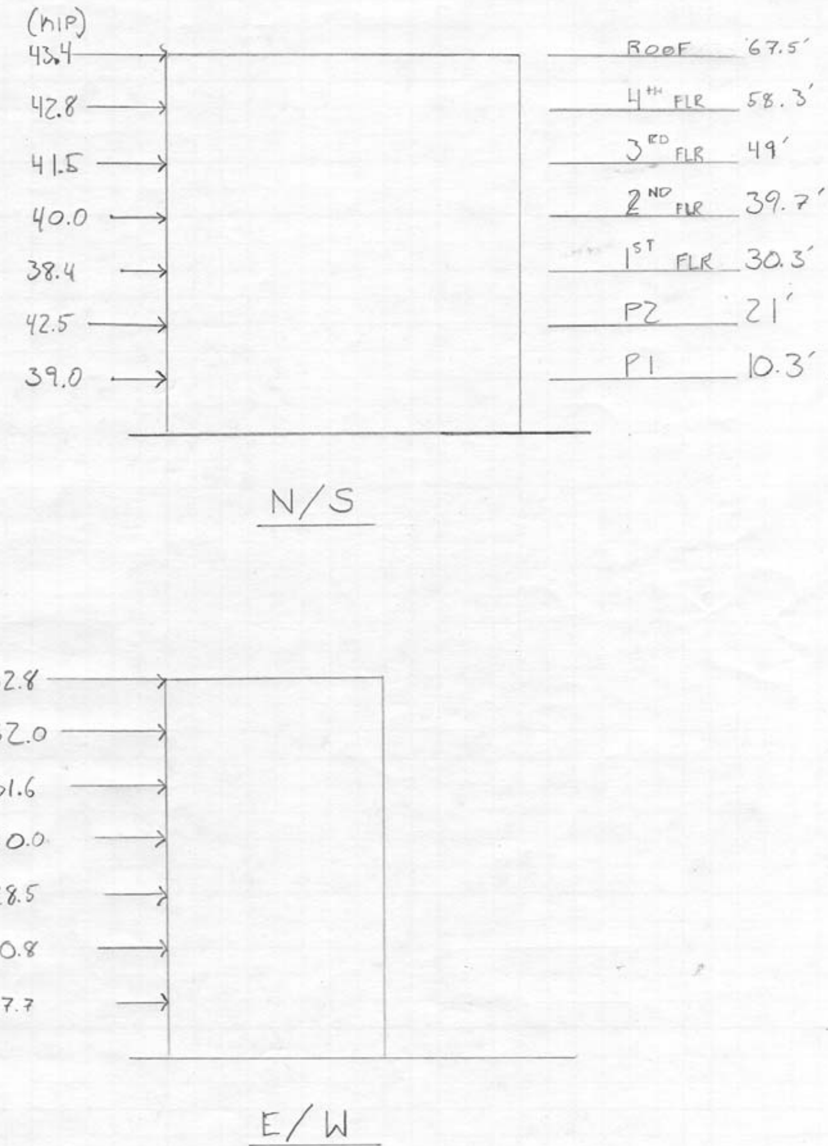
Seismic Analysis

Level	**Wx (kips)**	**hx (ft)**	**$W_x h_x^k$**	**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**

Wind Analysis

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	Windwa	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**													

WIND FORCES


(kip)
43.4
42.8
41.5
40.0
38.4
42.5
39.0
ROOF 67.5'
4TH FLR 58.3'
3RD FLR 49'
2ND FLR 39.7'
1ST FLR 30.3'
P2 21'
P1 10.3'
N/S
32.8
32.0
31.6
30.0
28.5
30.8
27.7
E/W

Spot Checks

There are three spot checks included in the appendix. The first is of an interior column on the second level of the parking deck (second level overall), the second is a masonry shear wall check of an interior shear wall on the top level of the building, and third is a check of the punching shear capacity of an interior spread footing.

No hand spot check was done for the hollow core slab system due to the complex nature of the design by the manufacturer of these products.

All spot checks complimented the original design and all over-design can be accounted for. See the appendix for more detail.

Apendix

Loads

Snow

Live Load Reduction

Wind

Seismic

Spot Checks

Column

Shearwall

Punching Shear

LIVE LOAD REDUCTION

$L > 0.4 L_o$

$L = L_o \left(0.25 + \frac{15}{\sqrt{K_{LL} A_T}} \right)$

$L = L_o \left(.25 + \frac{15}{\sqrt{1276}} \right)$

$L = L_o (0.67)$

LL ROOF REDUCTION

$R_2 = 1$

$R_1 = 1.2 - 0.001 \overset{A_t}{(319)} = 0.881$

$L_R = L_o \, 0.881$

SNOW LOADS

FLAT ROOF

$P_f = 0.7 C_e C_t I p_g$

$P_g = 30$

$I = 1.0$

$C_t = 1.0$

$C_e = 1.0$

$P_f = 0.7(30) = 21$ PSF

ASCE7-05 6.5 Method 2- Analytical Procedure
Wind Analysis

Height	h=		67.54	FT	
Current Story Height	z=		10.33	FT	
Basic Wind Speed	V=		90	mph	
Wind Directionality	Kd=		0.85		only with load
Importance Factor	I=		1		combinations of 2.3 & 2.4
Exposure		B			
Presure Coeficient					Table 6-3
	Kh=		0.88		interpolated
Topographic Factor	Kzt=		1		
Gust Effect Factor	G=		0.85		
Velocity Preasure	qz=		17.626		Kz add on other graph
	qh=		15.511		
Internal Pressure Coef.	GCpi= +/-		0.18		

Wall Pressure Coefficients		**E/W**			**N/S**	
	L		270	FT	60.67	
	B	60.67		FT	270	
	L/B		4.45		0.224703704	
Leeward	Cp=		-0.2		-0.5	use qh
Windward	Cp=		0.8		0.8	use qz
Side Wall	Cp=		-0.7		-0.7	use qh

Roof Pressure Coefficient					
	h/l=		0.25		1.113235537
	h/2-h		-0.9	>2h	-0.3
	and		-0.18		-0.18

Design Pressure	P=	qGCp-qi(GCp)	LB/FT^2

PG 1

SEISMIC EVALUATION

LAT: 40.7978
LON: 77.2541

USEING SOFTWARE FROM USGS WEBSITE

$S_s = 0.157\,g$
$S_1 = 0.050\,g$

SITE CLASS B

$F_a = 1.0 \quad F_v = 1.0$

$S_{MS} = F_a S_s = 0.157\,g$
$S_{M1} = F_v S_1 = 0.050\,g$

$S_{DS} = \frac{2}{3} S_{MS} = 0.105\,g$
$S_{D1} = \frac{2}{3} S_{M1} = 0.033\,g$

LATERAL FORCE RESISTING SYSTEM: ORDINARY REINFORCED MASONRY SHEAR WALLS

IMPORTACE FACTOR $I = 1.0$

SEIZMIC DESIGN CATAGORY = A

RESPONCE MODIFICATION COEFF. $R = 2$

Pg 2

$T_a = C_t h_n^x$

$T_a = (0.2)(67.54')^{0.75} = 0.47$

$C_u = 1.7$

$\underline{T} = 0.47(1.7) = \underline{0.799s \quad \text{or} \quad 1.25\ Hz}$

$C_t = 0.2$

$$C_{s=\min}\begin{cases} S_{DS}/(R/I) = 0.105/(2/1) = 0.0525 \\ S_{D1}/T[(R/I)] = 0.033/.799(2) = 0.0207 \\ \dfrac{S_{D1}T_L}{T^2(R/I)} = \dfrac{(0.033)(6)}{(0.799)(2)} = 0.1239 \end{cases}$$

$T_L = 6$

$C_s = 0.0207$

$V = C_s W = 0.0207(11{,}420^k) = 236.394$

$k = 1.15$

Dead Loads of Centre Court Apartments for Seismic Analysis

Floor	Area	Partitions, MEP, & Misc. Loads	Hollow Core & Slab Loads	CMU Wall Weight	Col. & Bm Weight	Total
Roof	13,900	312,750	834,000	196,122	26,566	1,369,438
5	13,900	417,000	834,000	389,506	34,911	1,675,417
4	13,900	417,000	834,000	397,330	34,911	1,683,241
3	13,900	417,000	834,000	397,330	34,911	1,683,241
2	13,900	417,000	834,000	397,330	34,911	1,683,241
1	16,200	364,500	972,000	359,000	106,185	1,801,686
P2	16,200	190,350	972,000	217,423	144,120	1,523,893
		2,535,600	6,114,000	2,354,040	416,516	11,420,156

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

Beams			**LENGTH**	**WEIGHT**
F2-Roof	W14X	43	356	15308
	W12X	26	112	2912
			TOTAL	**18220**
F1	W18X	55	17	935
	W24X	68	21	1428
	W27X	146	72	10512
	W18X	76	35	2660
	W14X	48	12	576
	W12X	26	112	2912
	W14X	43	356	15308
			TOTAL	**34331**
P2	W14X	34	35	1190
	W12X	26	12	312
	W14X	45	288	12960
	W12X	30	88	2640
			TOTAL	**17102**

Concrete beam wt. included in wall estimate.

	HT	**W10X**	**W14X**	**20X24 CONC**	**20X20 CONC**	**WT**
Columns		**49**	**90**	**500**	**4.17**	
1 to 5	9.33	31	3	0	0	16691.37
P2 & P1	10.66	13	3	22	2	127017.5244

Half of the floor above and below are used for the seismic dead loads.

Column Spot Check Loads

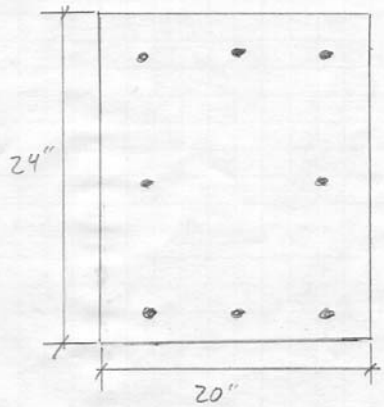
Level	Trib Area	Self Wt.	DL	L_o (psf)	Influence Area	Reduction	**LL**	**DL**	**Snow**
	ft^2	lbs	psf	psf	ft^2		kip	kip	psf
Roof	319	5000	90	20	1276	0.881	5.62	33.71	21
5	319	5000	90	40	1276	0.67	14.17	67.42	
4	319	5000	90	40	1276	0.67	22.72	101.13	
3	319	5000	90	40	1276	0.67	31.27	134.84	
2	319	5000	90	40	1276	0.67	39.82	168.55	
1	319	5000	90	40	1276	0.67	48.37	202.26	

1.2D+1.6L+.5S=
330.598848 Kip
276.468128

COLUMN SPOT CHECK

INTERIOR COLUMN ON THE 2ND LEVEL PARKING DECK

$P_u = 331^k$



(8) #8 BARS

$f_y = 60{,}000$ PSI

$f'_c = 4{,}000$ PSI

$P_o = 0.85 f'_c (bh - \Sigma A_s) + \Sigma A_s f_y$

$= 0.85(4)(20(24) - 6.32) + 6.32(60)$

$P_o = 1989.7$

$\phi P_o = 065(1989.7) = 1293^k$

$.8 \phi P_o = 1034.65^k > 331^k$

I ASSUME THE OVERSIZEING OF THE COLUMN WAS DUE MOSTLY TO A DESIRE TO HAVE UNIFORM DEMENSIONS WITH THE BELOW FLOORS.

MASONRY SHEAR WALL SPOT CHECK

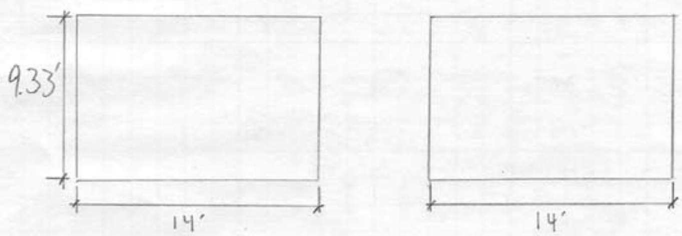
LEVEL 1 N/S CENTER WALL

% OF SHEAR LOAD BASED OFF WALL AREA ESTIMATION

TRIB. DIST. $47'(9.33') = 438.5\ FT^2/2500 \approx 18\%$

$V = .18(168) = 30.24^{KIP}$

8 IN NORMAL WT CMU
WALL HT. = 9.33'
WALL LENGTH = 28'
GRAVITY FORCES = SELF WT = NEGLECTIBLE
REINFORCEMENT = #5 @ 24" O.C.
ASSUME $f'_m = 1350$ PSI



DESIGN AID: TM 5-809-3 / NAVFAC DM-2.9 / AFM 88-3 CH. 3

$$f_v = \frac{V}{t d_v} = \frac{30240^{lbs}}{7.62 \times 28 \times 12} = 11.8 \text{ PSI}$$

$$\frac{M}{V d_v} = \frac{h}{2 d_v} = \frac{9.33' \times 12''}{2 \times 28 \times 12} = 0.167 < 1.0 \therefore$$

$$F_{vm} = \frac{1}{3}\left[4 - \frac{M}{Vd}\right](f'_m)^{1/2}$$

$$\frac{1}{3}\left[4 - 0.167\right](1350)^{.5} = 46.94 \text{ PSI}$$

SHALL NOT EXCEED $80 - 45\left[M/Vd_v\right]$
$= 72.5 > 46.91$

$f_v = 11.8 \text{ PSI} < F_{vm} = 46.91$ MULTIPLIERS NOT NEEDED ✓

PUNCHING SHEAR IN SPREAD FOOTING SPOTCHECK

LOAD FROM COLUMN = 390^k

$V_c = 4\sqrt{f'_c}\ b_o d$

$b_o = 2(20+12) + 2(24+12) = 136$

$V_c = 4\sqrt{4000}\ 136(12)/1000 = 412.8 > 390^k$