Dulles Town Center Building One

Dulles, Virginia

Technical Report I

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Executive Summary

In this first technical report the existing structural conditions of Dulles Town Center Building One are analyzed and discussed through a series of detailed descriptions and figures of the foundation, floor, column, and lateral systems. Current standards and designer standards will be used to help explain the design.

Building One is primarily a reinforced concrete structure. The structure uses caisson, slab on grade, concrete column, flat slab, and post-tension beam and non-post-tension one-way slab systems. Hollow structural steel (HSS) is also used as support for the curtain wall system along the east face. Lateral loads are resisted by ordinary reinforced concrete moment frames in the East-West direction and eccentrically braced frames made of HSS members in the North-South directions.

Seismic loads were calculated using the Equivalent Lateral Force Procedure, which is found in ASCE 7-05, Section 12.8. Through the use of diagrams, tables, and equation 12.8-1, the base shear was calculated. With the base shear and equation 12.8-12, the seismic shear at each floor was determined. Due to the location of Dulles Town Center Building One and the type of lateral force resisting system utilized, seismic loads did not control. This differed from the findings of the engineer. When compared to the loads used by the designer, the seismic base shear I obtained was much lower. Possible reasons will be explained later in the report.

A wind analysis was also performed using the Analytical Procedure outlined in ASCE 7-05, Section 6.5. The building's west face is slightly curved, but for calculation purposes it was conservative to assume the building to be rectangular. After the base shear was calculated, it was determined that wind loads controlled.

Finally, spot checks were conducted on a typical continuous beam, floor slab, and column. The post-tensioned continuous beam was analyzed as a non-post-tensioned beam and failed in all three spans proving that post-tensioning is needed for the long spans. The one-way slab system was checked, as well, and my calculations for the reinforcement came close to the current design. The thickness of the slab, however, was not thick enough. The column was analyzed with only considering gravity loads and ended up being considered as overdesigned. Reasons for these conclusions will be discussed later in the report.

Introduction

The Dulles Town Center Building One project consists of seven stories of office space above grade and one story below grade that includes rentable space, storage, mechanical rooms, a loading area, a trash room, building service offices, and a workout space. It is located in Dulles, Virginia; five minutes north of Dulles International Airport and 25 miles outside of Washington, D.C. The building's architectural use of precast concrete and glass curtain-wall have helped set the tone for the modernist themes conveyed along the Route 28 corridor. At night, this building is one of the most recognizable buildings along Route 28 with its linear neon focal points.

The building is approximately 202,000 square feet and reaches a height of 118 feet above grade. The building has an open floor plan and an average floor-to-floor height of 12'-6" making it ideal for office space. A typical bay is 20 feet by 40 feet, and consists of a post-tension concrete beam and non-post-tension one-way slab system.

The post-tension concrete beams allow for long spans and an open floor area, making it flexible for any tenant. The large bays, however, place large loads on the beams and in effect, post-tensioning is needed. Large bays leave little room for a lateral system. This report will begin to focus on these issues through the use of simplified and detailed analysis.

Structural System Overview

Foundation

The foundation system consists of a slab on grade with strap beams and caissons. The slab is 5″ thick and reinforced with 6x6 – W2.0xW2.0 welded wire fabric. It sits on a 6 mil. polyethylene vapor barrier over 6″ of washed, crushed stone. Strap beams ranging from 24″x 36″ to 48″x 48″ rest on a 2′-0″ thick foundation wall to help support the slab at grade changes. The cast-in-place caissons are capped with reinforced concrete and have shaft diameters that range from 30″ to 75″.

Columns

The vertical supporting elements are reinforced rectangular concrete columns with widths that range from 1′-0″ to 9′-2″. These 12″ x 110″ columns help support the stairwell and could act as small shear walls. Vertical reinforcement ranges in size from #8 to #11 rebar with #3 horizontal stirrups. The typical column is 24″ x 24″ with reinforcement consisting of (8) #8 vertical rebar, (3) #3 stirrups spaced at 3″ on center, and a hooked dowel extending 2′-6″minimum into the floor slab. These columns are also used for lateral resistance.

Floor Systems

The ground floor is flat slab construction consisting of an 8″ thick slab with a bottom bar mat of #4 rebar at 10″ on center each way. At column locations there are 5 ½″ drop panels and heavy reinforcement. The typical floor is a post-tensioned beam and non-post-tensioned one-way slab system. The 7″ thick slab is of normal weight with continuous edge drops that are 3′ wide and 5 ½″ deep along the east face to help support the precast concrete and ribbon window façade. The typical bay size is 20′ x 40′ with a typical beam length of 40′. Slab reinforcement consists of #4 top bars spaced at 6″ on center and #4 bottom bars at 12″ on center. Reinforced concrete beams are located at stairwells and elevator shafts. The second-floor is unique in that steel C and HSS members cantilever over the east entrance to support the curtain wall above. The penthouse floor system is the same as the typical system, but has a 9″ thick slab due to mechanical equipment.

Lateral System

The lateral resistance system is comprised predominantly of concrete moment frames with typical columns being 24″ x 24″. In addition, there is an eccentrically braced steel frame, or K-Brace, located on the roof within the architectural fin. This consists mostly of galvanized steel HSS members connected by fillet welds. The K-Brace is fillet welded to a 12″ x 1′-0″ x ½″ steel plate tied into the concrete roof with (4) ¾″ dia. x 12″ galvanized lightgage studs.

Typical Floor - Concrete Moment Frame

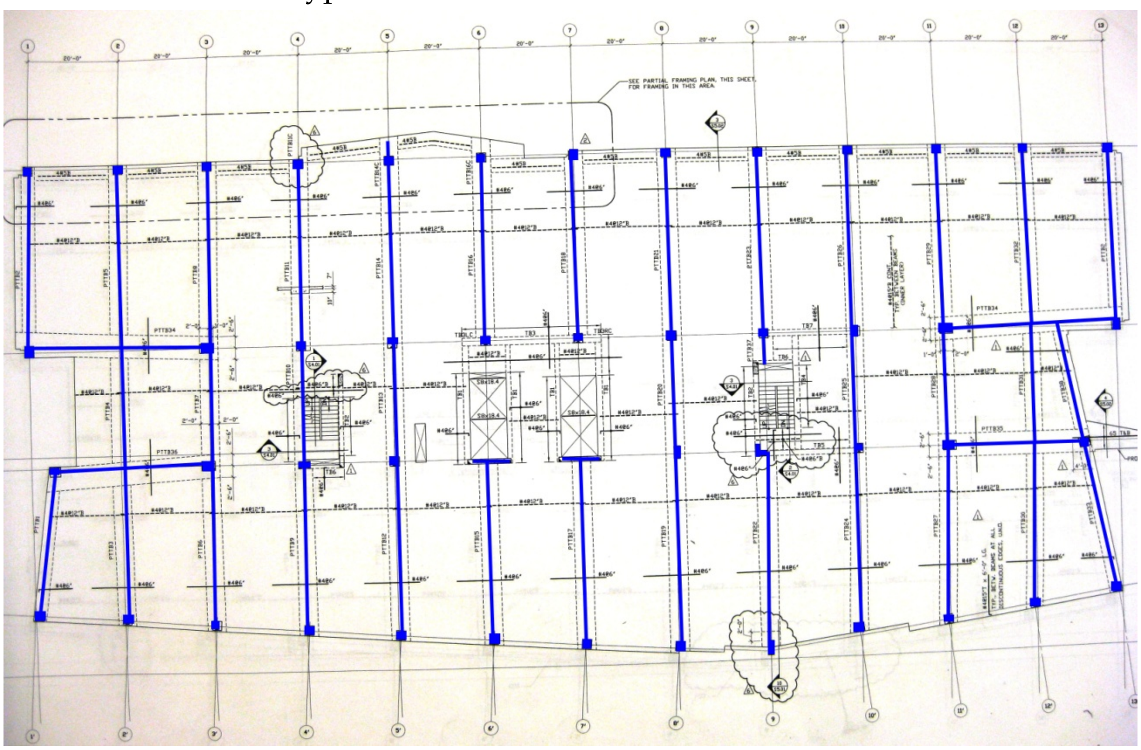


Figure 1

Roof System

The typical roof system also consists of a post-tension beam and non-post-tension one-way slab system. This typical roof system is just like the typical floor system in thickness, reinforcement, bay size, and beam length. Slab areas that support mechanical equipment, however, are 9″ thick and have #5 top bars at 8″ on center and #4 bottom bars at 6″ on center. The penthouse roof differs with its 8″ thick slab and #6 top bar- and #5 bottom bar- reinforcement at 12″ on center.

Material Strengths

Concrete

Typical Post-Tension Beam and
Non-post-tensioned one-way slab construction……………………..f'_c = 5,000 psi
Columns (Cellar – Level 3)……………………………………………………..f'_c = 5,000 psi
Columns (Level 3 – Roof)……………………………………………………....f'_c = 4,000 psi
Penthouse roof slab………………………………………………………………..f'_c = 4,000 psi
Beams……………………………………………………………………………….f'c = 4,000 psi
Slab on grade………………………………………………………………………f'_c = 3,500 psi
Walls and piers…………………………………………………………………….f'_c = 3,000 psi
Caissons…………………………………………………………………………….f'_c = 3,000 psi
Grade beams……………………………………………………………………….f'_c = 3,000 psi
All other concrete…………………………………………………………………f'_c = 3,000 psi

Reinforcement

Welded Wire Fabric………………………………………………………………F_y = 70,000 psi
Reinforcing bars…………………………………………………………………...F_y = 60,000 psi
Column and pier ties……………………………………………………………...F_y = 40,000 psi

Structural Steel

Wide flange shapes………………………………………………………….. F_y = 50,000 psi
Hollow Structural Steel (HSS)……………………………………………………F_y = 50,000 psi
Channels……………………………………………………………………………F_y = 36,000 psi
Angles………………………………………………………………………………F_y = 36,000 psi
Plates………………………………………………………………………………...F_y = 36,000 psi

Codes

- *Original Design:*

  Building Code

  BOCA, National Building Code, 1996
  Virginia Uniform Statewide Building Code

  Concrete

  American Concrete Institute (ACI), ACI 318

  Lateral Loads

  BOCA, National Building Code, 1996

  Design Loads and Standards

  BOCA, National Building Code, 1996
  American Society of Civil Engineers (ASCE), ASCE 7
  CABO ANSI A-117

- *Substitutions for Thesis Analysis:*

  Building Code

  American Society of Civil Engineers (ASCE), ASCE 7-05
  International Building Code (IBC) 2006

  Concrete

  American Concrete Institute (ACI), ACI 318-08

  Lateral Loads

  American Society of Civil Engineers (ASCE), ASCE 7-05

  Design Loads and Standards

  American Society of Civil Engineers (ASCE) ASCE 7-05
  International Building Code (IBC) 2006

Dead Loads and Live Loads

Dead Loads

The following weights were calculated using 150 pcf for reinforced concrete, an assumed 15 psf for ceiling load, an assumed 15 psf for the curtain wall system, an assumed 3 psf for metal paneling, and the designated linear weights for steel members.

Dead Loads					
	Building Component	Weight (kips)		Building Component	Weight (kips)
Ground Level			Level 7		
	Slab	2480		Slab	2188
	Drop Panels	284		Ceiling	381
	Non-PT Beams	127		Non-PT Beams	76
	Dropped Slab Edge	284		Dropped Slab Edge	487
	Columns	245.6		PT-Beams	1091
	Ceiling	372		Columns	387
	Walls	233.4		Wall	233.4
	Total	4026		Total	4843.4
Level 2			Roof		
	Slab	2223		Slab	2415
	Ceiling	381		Ceiling	375
	Non-PT Beams	70.3		Non-PT Beams	74.4
	Dropped Slab Edge	487		Dropped Slab Edge	487
	PT-Beams	1091		PT-Beams	1109
	Columns	420		Columns	287
	Wall	233.4		Wall	116.7
	Steel	17	Penthouse and Architectural Fin		
	Total	4922.7		Slab	292
Level 3 to Level 6				Ceiling	43.7
	Slab	2223		PT-Beams	87
	Ceiling	381		Columns	287
	Non-PT Beams	70.3		Steel	4.4
	Dropped Slab Edge	487		Metal Panels	31.7
	PT-Beams	1091			
	Columns	420		Total Roof	5609.9
	Wall	233.4			
	Total X 4	19622.8			
Total Building Weight = 39,031 k					

Live Loads

Below are the only live loads used for this report's analyses. The designer also used 150 lb/ft^2 for mechanical, 125 lb/ft^2 for the elevator machine room, and 100 lb/ft^2 for slab on grade.

Floor Live Loads		
Area	Design Load (psf)	ASCE 7-05 (psf)
Floors	100	100
Corridors	100	100
Roof Live Loads		
Area	Design Load (psf)	ASCE 7-05 (psf)
Roof Live Load	35	20
Snow	21	19

Lateral Loading

- *Seismic Loads: ASCE 7-05, Chapter 12*

  Seismic Forces were determined using the Equivalent Lateral Force Procedure found in Section 12.8. The base shear that I calculated was much smaller than that of the designer's. This can be explained by a few possible causes; the use of different codes, my assumptions on material weights, and my interpretation of the moment-resisting frame system being utilized. I assumed the system was made up mainly of ordinary reinforced concrete moment frames, which alters the response modification coefficient to be smaller than that of the designer's. This change in R ultimately lowered the base shear, enabling wind loads to control. Calculations can be found in the Appendix of this report. Refer to the following tables for variables used and seismic loads.

Variables Used

General Seismic Information		
Occupancy Category		II
Site Class		B
Seismic Design Category		A
Short Period Spectral Response	S_S	0.16
Spectral Response (1 sec)	S_1	0.051
Maximum Short Period Spectral Response	S_{MS}	0.16
Maximum Spectral Response (1 sec)	S_{M1}	0.051
Design Short Period Spectral Response	S_{DS}	0.107
Design Spectral Response (1 sec)	S_{D1}	0.034
Response Modification Coefficient	R	3
Seismic Response Coefficient	C_s	0.01
Effective Period	T	
Height Above Grade	h_n	108 ft
Base Shear		391k
Overturning Moment		23,527.81 ft-k

Seismic Loads

Seismic Base Shear								
Floor	Height (ft)	Tributary Height (ft)	Dead Load (kips)	$w_x h_x^k$	C_{vx}	Lateral Force (F_x)	Story Shear (V_x)	Overturning Moment (k-ft)
Ground	0	7.5	4026	0	0	390.31	390.31	23527.81
Level 2	15	13.75	4927	162084.1	0.0263	10.25	390.31	23527.81
Level 3	27.5	12.5	4906	352749.8	0.0572	22.32	380.06	18436.37
Level 4	40	12.5	4906	571985.4	0.0927	36.18	357.74	13807.03
Level 5	52.5	12.5	4906	812331	0.1317	51.39	321.56	9535.71
Level 6	65	12.5	4906	1070005	0.1734	67.69	270.17	5803.55
Level 7	77.5	12.75	4844	1325572	0.2148	83.86	202.48	2801.56
Roof	90.5	6.5	5610	1875165	0.3039	118.62	118.62	771.03
		Total	39031	6169892	1.0000	390.31		

- *Wind Loads: ASCE 7-05, Chapter 6*

  Wind loads for each level were calculated using the Analytical Procedure found in Section 6.5. Using Equation 6-19, wind loads were determined and used to find the base shear. This shear was higher than that of the seismic base shear and thus controlled. My wind calculations were similar to those of the designer which means my assumptions and analysis method were similar. Refer to the following tables for variables used and wind loads. Refer to Figure 2 and Figure 3 for wind loading. Blue denotes windward forces and red denotes leeward forces.

Variables Used for All Directions

Gust Factor Variables				
H (ft)	n_1	g_Q	g_v	g_R
118	0.549	3.4	3.4	4.057
V (mph)	b	c	β	α
90	0.45	0.3	2	7

Variables Used

East - West Wind Direction								
$\dot{z}$ (ft)	I_z	L_z	B	L	Q	V_z (ft/s)	N_1	h
67.5	0.266	412.72	240	105.5	0.837	71.04	3.4	112.5
Rn	η_h	R_h	η_B	R_B	η_L	R_L	R	G_f
0.065	4.3	0.206	9.23	0.102	13.58	0.071	0.196	0.83

Wind Loads

Wind (East - West Direction)										
Floor	Height (ft)	Tributary Height (ft)	K_z	q_z	Windward (psf)	Leeward (psf)	Total (psf)	Story Force (kips)	Story Shear (kips)	Overturning Moment (k-ft)
Ground	0.00	0.00	0.575	0.000	0.00	0.00	0.00	0.00	411.92	22114.73
Second	15.00	13.75	0.575	10.130	6.73	-7.48	14.20	46.87	411.92	17990.47
Third	27.50	12.50	0.683	12.045	8.00	-7.48	15.47	46.42	365.05	13149.40
Fourth	40.00	12.50	0.761	13.406	8.90	-7.48	16.38	49.13	318.63	8794.95
Fifth	52.50	12.50	0.822	14.489	9.62	-7.48	17.10	51.29	269.50	5746.76
Sixth	65.00	12.50	0.874	15.401	10.23	-7.48	17.70	53.11	218.21	3351.09
Seventh	77.50	12.75	0.919	16.195	10.75	-7.48	18.23	55.78	165.10	1608.62
Roof	90.50	15.25	0.960	16.928	11.24	-7.48	18.72	68.50	109.32	449.02
Mean Fin Ht.	112.50	8.75	1.022	18.014	11.96	-7.48	19.44	40.82	40.82	0.00

East-West Wind Diagram

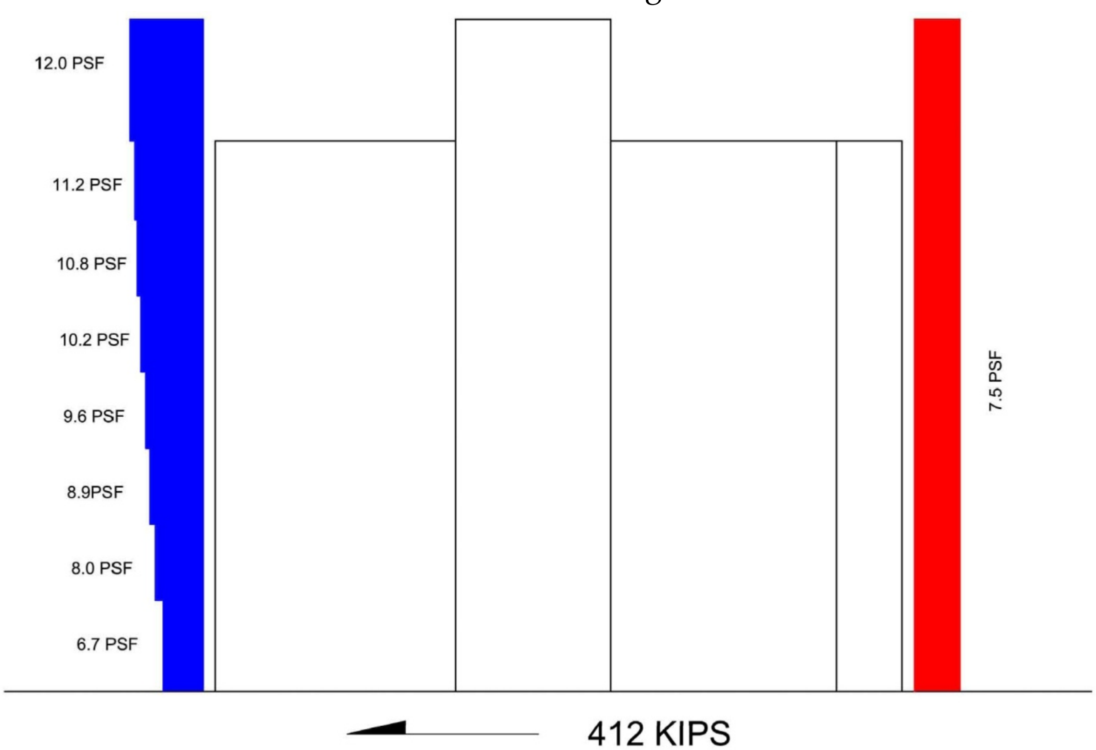


Figure 2

Variables Used

North - West Wind Direction								
$\bar{z}$ (ft)	I_z	L_z	B	L	Q	V_z (ft/s)	N_1	h
70.8	0.264	406.21	105.5	240	0.797	71.89	3.41	118
Rn	η_h	R_h	η_B	R_B	η_L	R_L	R	G_f
0.0645	4.485	0.1981	4.01	0.218	28.23	0.0344	0.276	0.87

Wind Loads

Wind (North - South Direction)										
Floor	Height (ft)	Tributary Height (ft)	K_z	q_z	Windward (psf)	Leeward (psf)	Total (psf)	Story Force (kips)	Story Shear (kips)	Overturning Moment (k-ft)
Ground	0.00	0.00	0.575	0.000	0.00	0.00	0.00	0.00	155.95	8247.02
Second	15.00	13.75	0.575	10.130	7.04	-4.76	11.80	17.12	155.95	6209.70
Third	27.50	12.50	0.683	12.045	8.37	-4.76	13.13	17.32	138.83	4582.58
Fourth	40.00	12.50	0.761	13.406	9.32	-4.76	14.08	18.57	121.51	3179.77
Fifth	52.50	12.50	0.822	14.489	10.07	-4.76	14.83	19.56	102.94	2015.27
Sixth	65.00	12.50	0.874	15.401	10.71	-4.76	15.47	20.40	83.38	1100.52
Seventh	77.50	12.75	0.919	16.195	11.26	-4.76	16.02	21.55	62.98	472.91
Roof	90.50	20.25	0.960	16.928	11.77	-4.76	16.53	35.31	41.43	84.15
Top of Fin	118.00	13.75	1.036	18.262	12.70	-4.76	17.46	6.12	6.12	0.00

North-South Wind Diagram

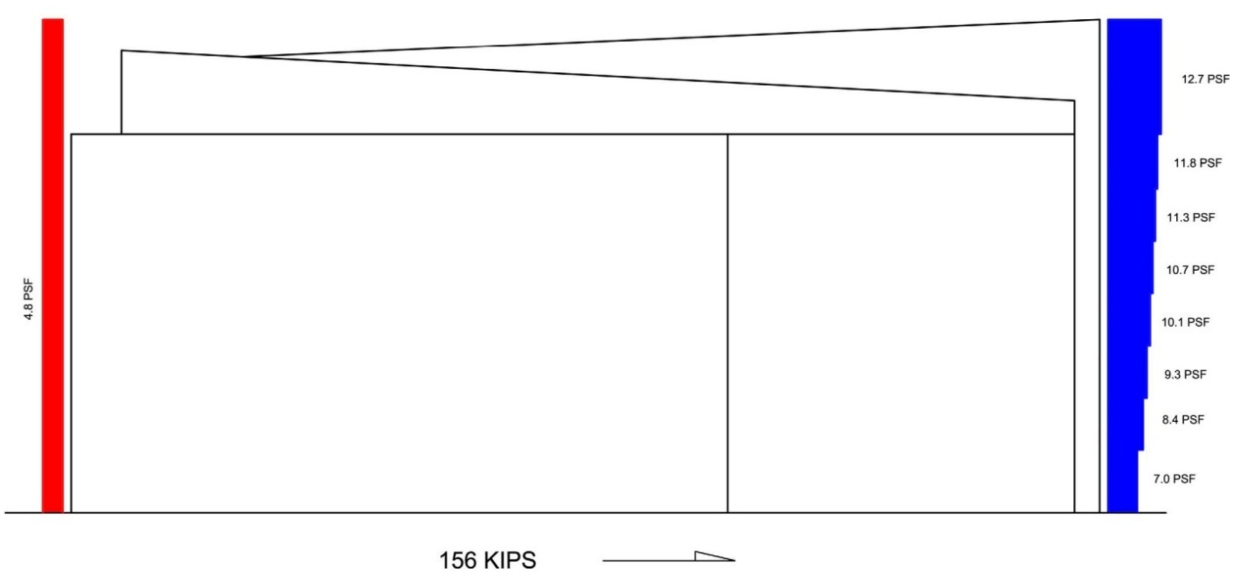


Figure 3

Preliminary Design Analysis

Gravity Load Spot Checks

For the spot checks I analyzed the three most common structural components of this building; a column, a continuous beam, and a one-way slab.

A column on the second floor was considered in this analysis. Load combinations such as 1.2D + 1.6L + .5S and 1.2D and 1.6L were used to find gravity loads on the member. Even without live load reduction, it was determined that the column was over-designed by about 600 kips. A possible reason could be that forces unaccounted for add load to this column. The use of different codes could also be a factor.

The continuous beam was analyzed as a typical reinforced continuous beam with no post-tensioning. My shear calculation seemed to be higher in all three spans than that of the original designer. This was seen in my shear reinforcement calculations requiring more stirrups than the beam schedule showed. My calculations also showed that the spans would fail if only typically reinforced, proving the necessity of post-tensioning. The outcome of the shear could possibly be because I used 1.2D + 1.6L instead of another, older, load combination. The post-tensioning of the continuous beam will be analyzed further later in the thesis process.

Using the Direct Design Method to analyze the slab, two things were concluded. The present reinforcement of #4 top bars at 6 inches on center and #4 bottom bars at 12 inches on center both support the moments calculated at their respective places in the slab. The minimum thickness for the slab supported on the beam came out to be larger than that of the actual slab. The only way to get the required thickness down would be to increase β, which you cannot do without changing the bay size, therefore I conclude this is due to code changes.

Refer to Figure 4 in Appendix for locations and calculations of slab, beam, and column.

Appendix A

Calculations

Seismic

SEISMIC | | DRG

LOCATION: DULLES, VIRGINIA

STRUCTURAL FRAME: ORDINARY REINFORCED CONCRETE MOMENT FRAMES

1. DESIGN SPECTRAL RESPONSE ACCELERATION

$S_s = 16\%\,g$ p 211 FIG. 22-1 ASCE 7-05

$S_1 = 5.1\%\,g$ p 213 FIG. 22-2 ASCE 7-05

SITE CLASS B

$S_{MS} = (F_a)(S_s) = (1.0)(.16) = .16$ F_a FROM TABLE 11.4-1 ASCE 7-05

$S_{M1} = (F_v)(S_1) = (1.0)(.051) = .051$ F_v FROM TABLE 11.4-2 p 115

$S_s < .25 \therefore$ USE 1.0 $S_1 < .1 \therefore$ USE 1.0

DESIGN VALUES FOR S_{MS} AND S_{M1}

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(.16) = .107$$

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(.051) = .034$$

2. SEISMIC DESIGN CATEGORY

OCCUPANCY CATEGORY = II

S_{DS}: SDC = A p 116 TABLE 11.6-1 ASCE 7-05

S_{D1}: SDC = A p 116 TABLE 11.6-2

ORDINARY REINFORCED CONCRETE MOMENT FRAMES

$R = 3$ $\Omega_0 = 3$ $C_d = 2.5$ TABLE 12.2-1

CONCRETE MOMENT-RESISTING FRAMES

$C_t = .016$ $x = .9$

CONSERVATIVELY USING PENTHOUSE AS TOP LEVEL

$T \cong T_a = (.016)(108')^{.9} = 1.08\ s$

$T_S = \frac{S_{D1}}{S_{DS}} = \frac{.034}{.107} = .318$

$T_L = 8$ FIGURE 22-15 ASCE 7-05

$C_u = 1.7$

UPPER LIMIT ON CALCULATED PERIOD $= C_u T_a = (1.7)(1.08) = 1.84\ s$

$T < T_L$

$C_S = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{.034}{1.08\left(\frac{3}{1}\right)} = .01 \leq \frac{S_{DS}}{\left(\frac{R}{I}\right)} = \frac{.107}{\left(\frac{3}{1}\right)} = .036$

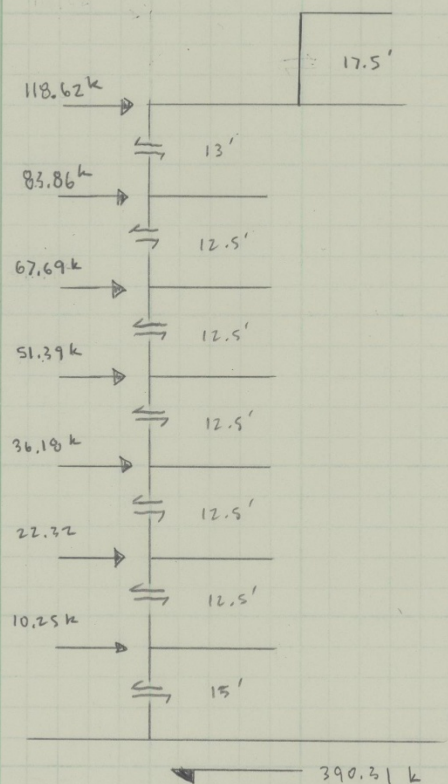
$W = 39031\ k$

BASE SHEAR

$V = C_s W = .01(39031\ k) = 390.31$

$k = .75 + .5(1.08) = 1.29$

OVERTURNING MOMENT



RF $OM_{@84'} = (118.62^k)(6.5') = 771.03$ k-ft

7 $OM_{@71.25'} = (118.62k)(19.25') + (83.86^k)(6.25') = 2807.56$

6 $OM_{@58.75'} = (118.62)(31.75) + (83.86k)(19.25') + (67.69)(6.25) = 5803.55$

5 $OM_{@46.25} = (118.62)(44.25) + (83.86)(31.75) + (67.69)(19.25) + (51.39)(6.25) = 9535.71$

4 $OM_{@33.75} = (118.62)(56.75) + (83.86)(44.25) + (67.69)(31.75) + (51.39)(19.25) + (36.18)(6.25)$
$= 13807.03$

3 $OM_{@21.25} = (118.62)(69.25) + (83.86)(56.75) + (67.69)(44.25) + (51.39)(31.75) + (36.18)(19.25)$
$+ (22.32)(6.25) = 18436.37$

2 $OM_{@7.5} = (118.62)(83) + (83.86)(70) + (67.69)(57.5) + (51.39)(45) + (36.18)(32.5)$
$+ (22.32)(20) + (10.25)(7.5)$
$= 23527.81$ k-ft

Wind

WIND DRG

EAST-WEST WIND

$n_1 = .594$ ✓

$g_Q = g_v = 3.4$ ✓

$g_R = 4.057$ ✓

$\bar{z} = .6h = .6(112.5) = 67.5' \geq z_{min} = 30' \therefore$ ok ✓

$c = .3$ ✓

$I_{\bar{z}} = .3\left(\frac{33}{67.5}\right)^{1/6} = .266$ ✓

$L_{\bar{z}} = 320\left(\frac{67.5'}{33}\right)^{1/3} = 406.21$ ✓

$$Q = \sqrt{\frac{1}{1+.63\left(\frac{240+112.5}{406.21}\right)^{.63}}} = .797 \checkmark$$

$\bar{V}_{\bar{z}} = .45\left(\frac{67.5}{33}\right)^{1/4}(90)\left(\frac{88}{60}\right) = 71.04$ ✓

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{(.594)(406.21)}{71.04} = 3.45$$

$$R_n = \frac{7.47(3.45)}{(1+10.3(3.45))^{5/3}} = \frac{25.4}{392.9} = .065$$

$$\eta_h = \frac{4.6(.59)(112.5)}{71.04} = 4.3$$

$$R_h = \frac{1}{4.3} - \frac{1}{2(4.3)^2}\left(1-e^{-2(4.3)}\right) = .233 - .027(.9998) = .206$$

$$\eta_B = \frac{4.6(.594)(240)}{71.04} = 9.23$$

$$R_B = \frac{1}{9.23} - \frac{1}{2(9.23)^2}\left(1-e^{-2(9.23)}\right) = .108 - .0059(1) = .102$$

WIND DR 6

EAST WEST WIND

$$n_L = \frac{15.4(.594)(105.5)}{71.04} = 13.58$$

$$R_L = \frac{1}{13.58} - \frac{1}{2(13.58)^2}\left(1 - e^{-2(13.58)}\right) = .0736 - .0027(1) = .071$$

$$R = \sqrt{\frac{1}{.02}(.065)(.206)(.102)(.53 + .47(.071))} = .196$$

$$G_f = .925\left(\frac{1 + 1.7(.266)\sqrt{(3.4)^2(.797)^2 + (4.057)^2(.196)^2}}{1 + 1.7(3.4)(.266)}\right)$$

$$= .925\left(\frac{2.28}{2.54}\right) = .83$$

C_p

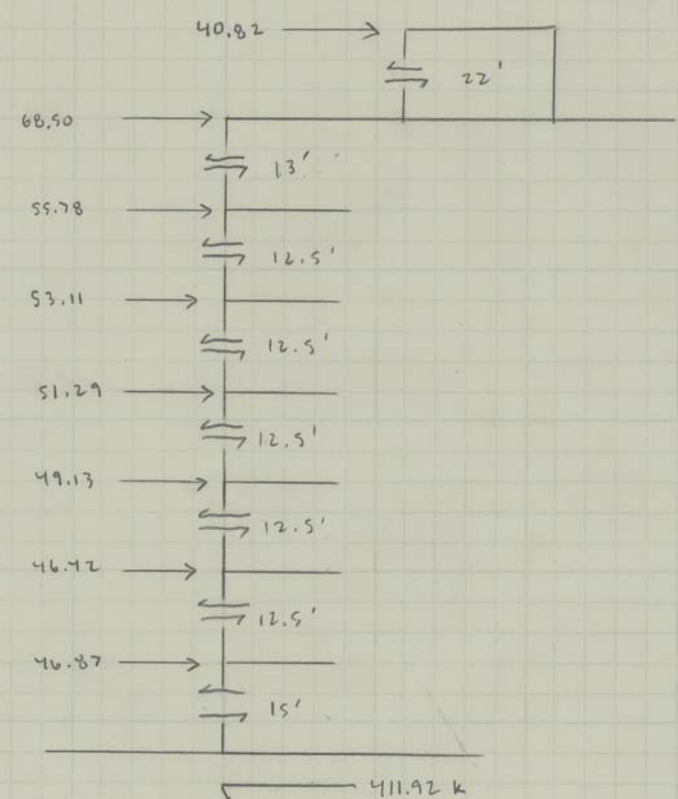
$L/B = \frac{105.5}{240} = .4396$

	WINDWARD	LEEWARD
	.8	-.5

$GC_{pi} = \pm .18$

WIND | | DRG | 3

EAST - WEST



RF $OM_{@101.5'} = 40.82\left(\frac{22'}{2}\right) = 449.02$

7 $OM_{@84'} = 40.82\left(22+\frac{13}{2}\right) + 68.5\left(\frac{13}{2}\right) = 1608.62$

6 $OM_{@71.25'} = 40.82\left(22+13+\frac{12.5}{2}\right) + 68.5\left(13+\frac{12.5}{2}\right) + 55.78\left(\frac{12.5}{2}\right) = 1683.83 + 1318.63 + 348.63$
$= 3351.09$

5 $OM_{@58.75'} = 40.82(53.75) + 68.5(31.75) + 55.78(18.75) + 53.11\left(\frac{12.5}{2}\right) = 5746.76$

4 $OM_{@46.25'} = 40.82(66.25) + 68.5(44.25) + 55.78(31.25) + 53.11(18.75) + 51.29(6.25)$
$= 8794.95$

3 $OM_{@33.75'} = 40.82(78.75) + 68.5(66.25) + 55.78(44.25) + (53.11)(31.25) + 51.29(18.75) + 49.13(6.25)$
$= 13149.4$

2 $OM_{@21.25'} = 40.82(91.25) + 68.5(78.75) + 55.78(66.25) + 53.11(44.25) + 51.29(31.25)$
$+ 49.13(18.75) + 46.42(6.5)$
$= 17990.47$

1 $OM_{@7.5'} = 40.82(105) + 68.5(83) + 55.78(70) + 53.11(57.5) + 51.29(45) + 49.13(32.5)$
$+ 46.42(20) + 46.87(7.5)$
$= 22114.73$

NORTH-SOUTH WIND

$$n_1 = \frac{43.5}{H^{.9}} = \frac{43.5}{(118')^{.9}} = .594 \checkmark$$

$$g_Q = g_v = 3.4 \checkmark$$

$$g_R = \sqrt{2\ln(3600(.59))} + \frac{.577}{\sqrt{2\ln(3600(.59))}} = 3.91 + .147 = 4.057 \checkmark$$

$$\bar{z} = .6h = .6(118') = 70.8' \geq z_{min} = 30' \therefore ok \checkmark$$

$c = .3$

EXP.	α	z_g (ft)	$\hat{a}$	$\hat{b}$	$\bar{\alpha}$	$\bar{b}$	c	ℓ (ft)	$\bar{\epsilon}$	z_{min} (ft)
B	7	1200	1/7	.84	1/4	.45	.3	320	1/3	30'

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = .3\left(\frac{33}{70.8'}\right)^{1/6} = .264 \checkmark$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{70.8''}{33}\right)^{1/3} = 412.72 \checkmark$$

$$Q = \sqrt{\frac{1}{1+.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{.63}}} = \sqrt{\frac{1}{1+.63\left(\frac{105.5+118}{412.72}\right)^{.63}}} = .837 \checkmark$$

$V = 90$ MPH

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = .45\left(\frac{70.8}{33}\right)^{1/4}(90)\left(\frac{88}{60}\right) = 71.89 \checkmark$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{.594(412.72)}{71.89} = 3.41 \checkmark$$

$$R_n = \frac{7.47(N_1)}{(1+10.3N_1)^{5/3}} = \frac{7.47(3.41)}{(1+10.3(3.41))^{5/3}} = \frac{25.47}{394.74} = .0645 \checkmark$$

$$\eta_h = \frac{4.6 n_1 h}{\bar{V}_{\bar{z}}} = \frac{4.6(.594)(118)}{71.89} = 4.485 \checkmark$$

$$R_h = \frac{1}{4.485} - \frac{1}{2(4.485)^2}\left(1-e^{-2(4.485)}\right) = .223 - .0249(1) = .1981 \checkmark$$

WIND | DRG

NORTH – SOUTH

$$\eta_B = \frac{4.6(.594)(105.5)}{71.89} = 4.01$$

$$R_B = \frac{1}{4.01} - \frac{1}{2(4.01)^2}\left(1-e^{-2(4.01)}\right) = .249 - .0311\left(.9997\right) = .218$$

$$\eta_L = \frac{15.4 n_1 L}{\bar{V}_{\bar{z}}} = \frac{15.4(.549)(240)}{71.89} = 28.23$$

$$R_L = \frac{1}{28.23} - \frac{1}{2(28.23)^2}\left(1-e^{-2(28.23)}\right) = .035 - .0006\left(1\right) = .0344$$

$\beta = 2\% = .02$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (.53 + .47 R_L)} = \sqrt{\frac{1}{.02}(.0645)(.1981)(.218)(.53 + .47(.0344))}$$

$= .276$

$$G_f = .925\left(\frac{1 + 1.7(.264)\sqrt{(3.4)^2(.837)^2 + (4.06)^2(.276)^2}}{1 + 1.7(3.4)(.264)}\right)$$

$$= .925\left(\frac{2.373}{2.53}\right) = .869$$

C_p | WINDWARD | LEEWARD

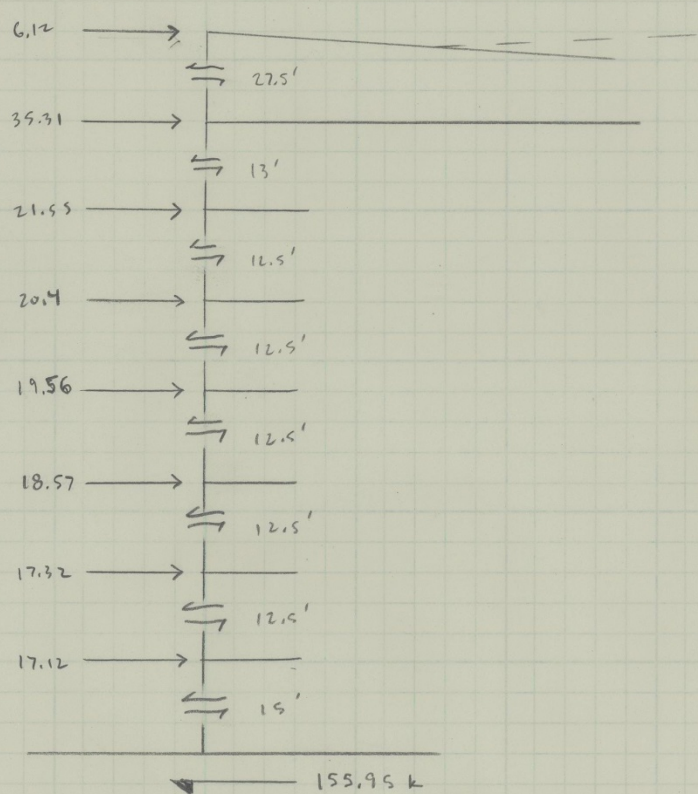
$L/B = \frac{240}{105.5} = 2.275$ | .8 | –.3

$GC_{pi} = \pm .18$

WIND | DRG

NORTH-SOUTH

104.25
84
71.25
58.75
46.25
33.75
21.25
7.5



RF $OM_{@104.25'} = 6.12(13.75) = 84.15$

7 $OM_{@84'} = 6.12(34) + 35.31(7.5) = 472.91$

6 $OM_{@71.25'} = 6.12(46.75) + 35.31(19.25) + 21.55(6.25) = 1100.52$

5 $OM_{@58.75'} = 6.12(59.25) + 35.31(31.75) + 21.55(18.75) + 20.4(6.25) = 2015.27$

4 $OM_{@46.25'} = 6.12(71.75) + 35.31(44.25) + 21.55(31.25) + 20.4(18.75) + 19.56(6.25) = 3179.77$

3 $OM_{@33.75'} = 6.12(84.25) + 35.31(56.75) + 21.55(43.75) + 20.4(31.25) + 19.56(18.75) + 18.57(6.25)$
$= 4582.58$

2 $OM_{@21.25'} = 6.12(96.75) + 35.31(69.25) + 21.55(56.25) + 20.4(43.75) + 19.56(31.25)$
$+ 18.57(18.75) + 17.32(6.25) = 6209.7$

1 $OM_{@7.5} = 6.12(110.5) + 35.31(83) + 21.55(70) + 20.4(57.5) + 19.56(45)$
$+ 18.57(32.5) + 17.32(20) + 17.12(7.5)$
$= 8247.02$

Spot Checks

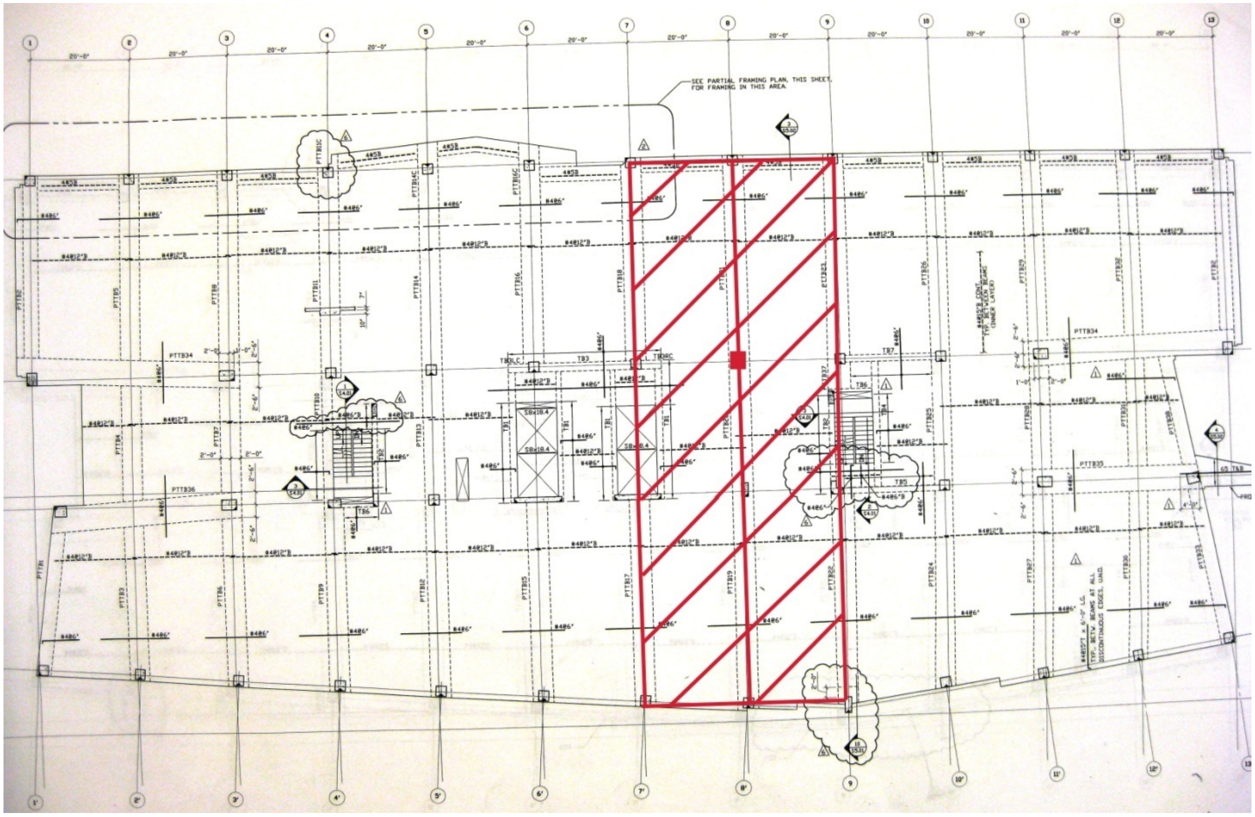


Figure 4

Above is a diagram of where in the building my spot checks were conducted.

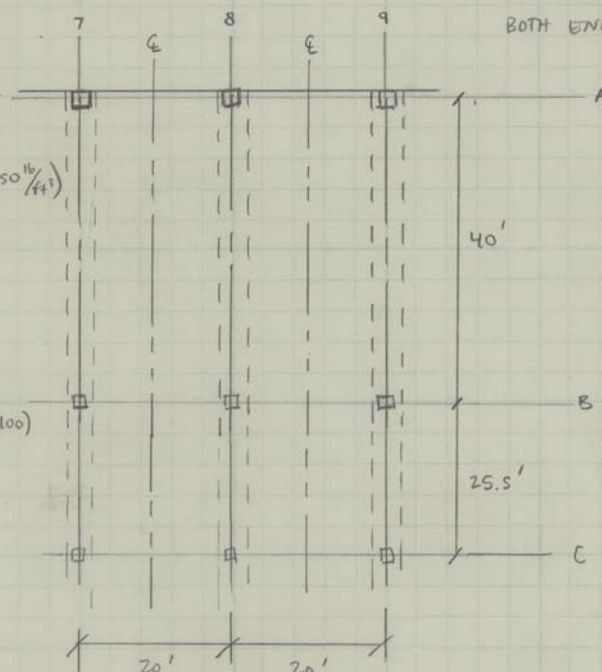
SPOT CHECKS | FLOOR SLAB | DAVID GEIGER | 1

TYPICAL BAY
20' x 40'

BOTH ENDS CONTINUOUS
$\ell/28$

LIVE
$LL = 100\ psf$

CEILING
$DL = 15\ psf + \left(\frac{7''}{12}\right)(150\ lb/ft^3)$
$= 102.5\ psf$



$\ell_1 = 40'$

$\ell_2 = 20'$

$\ell_n = 40' - \frac{24''}{2}$
$= 39'$

$F'_c = 5000\ psi$

$F_y = 60{,}000\ psi$

$w_u = 1.2w_D + 1.6w_L$
$= 1.2(102.5) + 1.6(100)$
$= 283\ psf$

INTERIOR SUPPORTS: $-M = -\frac{1}{10}(.283\ k/ft^2)(20')^2 = -11.32\ ft\text{-}k$

MIDSPAN: $+M = \frac{1}{14}(.283\ k/ft)(20')^2 = 8.09\ ft\text{-}k$

RATIO OF FLEX. STIFF.

$$\alpha = \frac{EI_B}{EI_S}$$

$$I_B = k\frac{b_w h^3}{12} = \frac{1.28\,(48'')(24'')^3}{12} = 70778.9\ in^4$$

$$\left.\begin{aligned} \frac{b_E}{b_w} &= \frac{b_w + 2h_w}{b_w} = \frac{48'' + 2(17'')}{48''} = 1.708 \\ \frac{t}{h} &= \frac{7''}{24''} = .292 \end{aligned}\right\} \text{FIND } k$$

$$k = \frac{1 + (1.708-1)(.292)\left[4 - 6(.292) + 4(.292)^2 + (1.708-1)(.292)^3\right]}{1 + (1.708-1)(.292)} = \frac{1 + .207[2.607]}{1.207}$$

$$= 1.28$$

SPOT CHECKS	FLOOR SLAB	DAVID GEIGER	2

$$I_s = \frac{l_2 t^3}{12} = \frac{(20')(12)(7'')^3}{12} = 6860 \text{ in}^4$$

$$\alpha = \frac{70779 \text{ in}^4}{6860 \text{ in}^4} = 10.32$$

MINIMUM SLAB THICKNESS

$5'' < 7''$ ∴ ok

MINIMUM THK. FOR SLAB SUPPORTED ON BEAM

$$\beta = \frac{l_n}{S_n} = \frac{39'}{20'} = 1.95$$

$\alpha_m = 10.32 > 2$ STIFF BEAM

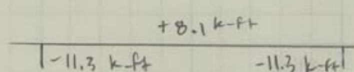
$$t_{min} = \frac{l_n(.8 + f_y/200{,}000)}{36 + 9\beta} = \frac{(39 \times 12)(.8 + 60000/200{,}000)}{36 + 9(1.95)} = 9.61''$$

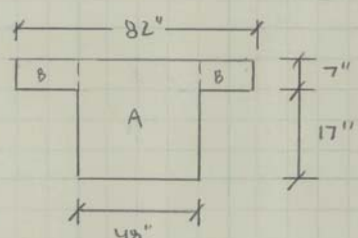
$9.61'' > t_{DESIGNED} = 7''$ ∴ NOT OKAY

POSSIBLE EXPLANATION: CODE CHANGES

$M^-_{INTER} = -11.32$ k-ft

$M^+_{MID} = 8.09$ k-ft





$$C = \Sigma\left(1 - .63\frac{x}{y}\right)\left(\frac{x^3 y}{3}\right)$$

$$C = \left(1 - .63\left(\tfrac{24}{48}\right)\right)\left(\frac{24^3(48)}{3}\right) + 2\left(1 - .63\left(\tfrac{7}{17}\right)\right)\left(\frac{7^3(17)}{3}\right)$$

$$= 151511 + 2879 = 154390 \text{ in}^4$$

SPOT CHECKS | FLOOR SLAB | DAVID GEIGER

$\ell_2/\ell_1 = \frac{20'}{40'} = .5 ; \quad \alpha(\ell_2/\ell_1) = 10.32(.5) = 5.16 > 1.0$

∴ COLUMN STRIPS SHALL BE PROPORTIONED TO RESIST 90% OF M^+ — TABLE 13.6.4.4 ACI

TOTAL M	-11.32	8.09	-11.3
BEAM	-8.7	6.2	-8.7
CS SLAB	-1.5	1.1	-1.5
MS SLAB	-1.13	.81	-1.13

-11.32 k-ft
- 90% TO CS = -10.2
  - 85% TO BM = -8.7 k-ft
  - 15% to SC = -1.5 k-ft
- 10% TO MS = -1.13 k-ft

8.09 k-ft
- 90% = 7.3
  - 85% = 6.2 k-ft
  - 15% = 1.1 k-ft
- 10% = .81 k-ft

SLAB REINF.

MAX. SPACING
$2 \times 7'' = 14''$

MIN. STEEL ≡ TEMP. & STRESS REINF.

$A_{s_{min}} = .0018\,bt = .0018(96)(7'') = 1.21 \text{ in}^2$

$d_{short} = 7'' - 3/4'' - \frac{1}{2}(.5) = 6''$
↑ CLEAR COVER

	DESCRIPTION	INT. SPAN M^-	INT. SPAN M^+
1.	M_u (k-ft)	-11.32	8.09
2.	CS SLAB b (in)	96	96
3.	EFFECTIVE d (in)	6	6
4	$M_n = M_u/.9$ (k-ft)	-12.6	9
5.	$M_n \times 12/b$	-1.58	1.13
6.	$R = \frac{M_n}{bd^2}$	43.75	31.25
7.	ρ = (FROM A-5a NDD)	.0007	.0005
8	$A_s = \rho bd$	.4032	.288

#3 d_{min} USING ρ_{MAX}:

ρ_{MAX} FOR $f'_c = 5000$ psi, $f_y = 60000$ psi FROM TABLE A-4 = .0243

$$M_u = \phi M_n = \phi \rho f_y b d^2 \left(1 - .59 \rho f_y / f'_c\right)$$

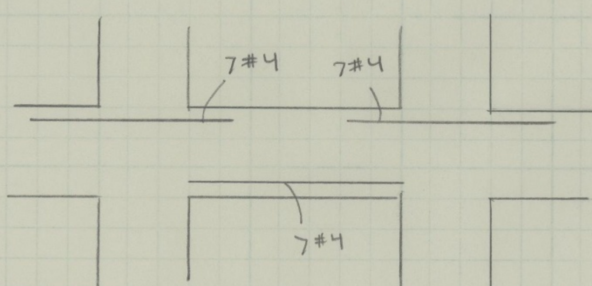
$$d^2 = \frac{M_u}{.9(.0243)(60000)(12'')(1-.59(.0243)(60/5))} = \frac{M_u}{13037}$$

$$d_{min} = \sqrt{\frac{(1.58)(12000)}{13037}} = 1.21 \text{ in}^2 \quad \therefore d_{USED} = 6'' \text{ OK}$$

9. $A_{s_{min}} = 1.21 \text{ in}^2$

10. $N = \frac{1.21 \text{ in}^2}{.2 \text{ in}^2} = 6.05 \approx 7$

11. $N_{min} = \frac{96}{2(7'')} = 6.86 \approx 7$



COLUMN STRIP

$\frac{7 \text{ BARS}}{10'} = .7 \text{ BARS/FT}$

$\therefore$ #4 @ 6" O.C. OK

MIDDLE STRIP
OK BY INSPECTION
WITH #4 @ 12" O.C.

SHEAR

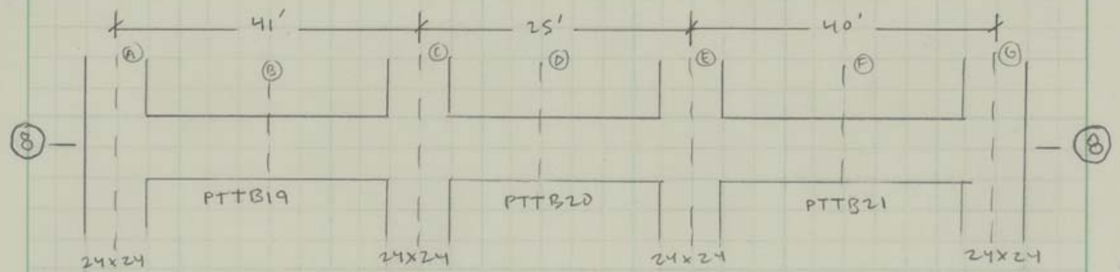
$$V_u = (.283 \text{ k/ft}^2)\underbrace{(1')}_{12'' \text{ STRIP}}\left(10' - \underbrace{\frac{24''/2}{12}}_{\text{COLUMN}} - \underbrace{\frac{6''}{12}}_{d}\right) = 2.41 \text{ k}$$

$$\phi V_c = \phi 2 \sqrt{f'_c}\, bd = \frac{.75(2)\sqrt{5000}(12'')(6'')}{1000} = 7.64 \text{ k}$$

$$\phi V_c = 7.64 \text{ k} > V_u = 2.41 \text{ k} \quad \therefore \text{OK}$$

Continuous Beam

SPOT CHECKS | CONT. BEAM | DRG | 1



l_n 39'   23'   38'

$41' - \frac{24''}{12} = 39$ $25' - \frac{24''}{12} = 23$ $40' - 2' = 38'$

LOADS

SUPERIMPOSED DL = 15 psf (CEILING, AIR DUCT, ELEC., COMM., SPRINKLER)

SLAB CONTRIBUTION = $(7''/12)(150\ ^{lb}/_{ft^3}) = 87.5$ psf

BEAM CONTRIBUTION = $\frac{48''(17'')}{144}(150\ ^{lb}/_{ft^3}) = 850\ ^{lb}/_{ft}$

$DEAD_{TOT} = (15\ ^{lb}/_{ft^2} + 87.5\ ^{lb}/_{ft^2})(20') + 850\ ^{lb}/_{ft} = 2.9\ ^{k}/_{ft}$

$LIVE_{TOT} = (80\ ^{lb}/_{ft^2} + 20\ ^{lb}/_{ft^2})(20') = 2\ ^{k}/_{ft}$

FACTORED LOAD

$w_u = 1.2D + 1.6L = 1.2(2.9\ ^{k}/_{ft}) + 1.6(2\ ^{k}/_{ft}) = 6.7\ ^{k}/_{ft}$

PTTB19

SHEAR

(1) $V_u = \frac{1.0(6.70\ ^{k}/_{ft})(39')}{2} = 130.6$ k

(B) $V_u = \frac{1.15(6.70\ ^{k}/_{ft})(39')}{2} = 150.3$ k

$d = 17'' - 1.5'' - .375 - \frac{1}{2}(.625) = 14.8''$

SPOT CHECKS | CONT. BEAM | DRG

$V_c = 2(1)\sqrt{5000}\,(48'')(14.8'') = 100.5\text{ k}$

$\phi V_n = .5(1)(.75)(101\text{ k}) = 38\text{ k}$

Ⓛ $V_s = \dfrac{131\text{ k}}{.75} - 101\text{ k} = 74\text{ k} \leq 8\sqrt{5000}\,(48)(14.8) = 402\text{ k} \quad \therefore \text{ ok}$

$74\text{k} \leq 201\text{k} \quad \therefore \quad s_{MAX} = \min \begin{cases} 14.8/2 = 7.4'' \leftarrow \text{CONTROLS} \\ 24'' \end{cases}$

$A_{v_{min}} = \max \begin{cases} \dfrac{.75\sqrt{5000}\,(48'')(7.4'')}{60000} = .314 \leftarrow \text{CONTROLS} \\ \dfrac{50(48'')(7.4'')}{60000} = .296 \end{cases}$

$A_v = .3 \text{ LEGS} \times \dfrac{.11\text{ in}^2}{\text{LEG}} = .33\text{ in}^2 > .314\text{ in}^2 \quad \therefore \text{ ok}$

$s = \dfrac{(.33\text{ in}^2)(60)(14.8'')}{74\text{ k}} = 3.96''$

Ⓡ $V_s = \dfrac{150\text{ k}}{.75} - 101\text{ k} = 99\text{ k}$

$s = \dfrac{(.33\text{ in}^2)(60)(14.8'')}{99\text{ k}} = 2.96''$

CONCLUSION:

COULD HAVE ASSUMED DEAD LOADS THAT WERE TOO HIGH OR THEY USED ASD TO CALCULATE SHEAR.

IF TOTAL LOAD IS KEPT AT DL + LL THEN SPACING GOES UP TO 11'' FROM THE LEFT WHICH IS CLOSE TO THEIR 1@2'', R@12''. FROM THE RIGHT THE SPACING JUMPS TO 6''

SPOT CHECKS | CONT. BEAM | DRG

CRITICAL MOMENTS

(L) $M_u = -\frac{w_u l_n^2}{16} = -\frac{(6.70^k/ft)(39')^2}{16} = -637$ k-ft

(M) $M_u = \frac{w_u l_n^2}{14} = \frac{(6.70)(39')^2}{14} = 728$ k-ft

(R) $M_u = -\frac{w_u l_n^2}{10} = -\frac{(6.70)\left(\frac{39'+23'}{2}\right)^2}{10} = -644$ k-ft

PER NOTES COVER = 1.5" STIRRUPS ARE #3

	TOP BARS	BOTTOM
LE	13#5	—
MID	—	9#6
RE	12#6	—

SECTION @ A

$f'_c = 5000$ psi

$f_y = 60\,000$ psi

$\beta = .8$

• CHECK $A_{s,min}$ & $A_{s,max}$: $A_s = 13(.31) = 4.03 \text{ in}^2$

$$A_{s,min} \begin{cases} \frac{3\sqrt{5000}(48)(17)}{60000} = 2.88 \text{ in}^2 \\ \frac{200(48)(17)}{60000} = 2.72 \text{ in}^2 \end{cases}$$

$A_s > A_{s,min}$ ∴ ok

$$\rho_{max} = .85(.8)\left(\frac{5000}{60000}\right)\left(\frac{.003}{.003+.004}\right) = .0243$$

$A_{s,max} = .0243(48")(14.8") = 17.25 \text{ in}^2$ $A_s < A_{s,max}$ ∴ ok

LEAVES PLENTY OF ROOM FOR PT TENDONS

M_n

ASSUME $f_s \geq f_y$

$$a = \frac{(4.03 \text{ in}^2)(60)}{.85(5)(48")} = 1.19"$$

$$c = \frac{1.19}{.8} = 1.48"$$

CHECK $\varepsilon_s > \varepsilon_y$ $\varepsilon_s = \frac{.003}{1.48"}(14.8"-1.48") = .027 > .00207$ ∴ ok

SECTION @ A
CONT'D

$\varepsilon_t \geq .005 \therefore \phi = .9$

$\phi M_n = .9(4.03 in^2)(60)(14.8'' - \frac{1.19''}{2}) = 3091.3$ k-in
$= 258$ k-ft

637 k-ft >> 258 k-ft ∴ NO GOOD

SECTION @ B (MID-PTTB19)

$d = 17'' - 1.5'' - .375 - \frac{1}{2}(.75) = 14.75''$

• CHECK $A_{s,min}$ & $A_{,max}$: $A_s = 9(.44) = 3.96 in^2$

$A_s > A_{s,min} \therefore$ OK * SEE SECTION @ A CALCULATIONS

$A_{s,max} = .0243(48'')(14.75'') = 17.2 in^2$

$A_s < A_{s,max} \therefore$ ok

M_n

ASSUME $f_s \geq f_y$

$a = \frac{(3.96 in^2)(60)}{.85(5)(48'')} = 1.16$ in

$c = \frac{1.16}{.8} = 1.45$

$\varepsilon_s = \frac{.003}{1.45}(14.75 - 1.45) = .0275 > .00207 \therefore$ ok

$\phi = .9$

$\phi M_n = .9(3.96 in^2)(60)(14.75'' - \frac{1.45''}{2}) = 2999.1$ k-in
$= 250$ k-ft

728 k-ft >> 250 k-ft ∴ NO GOOD

SECTION @ C_{RIGHT}
CONT'D

$\underline{M_n}$ SEE SECTION @ C_{LEFT}

ASSUME $f_s \geq f_y$

$a = 1.55''$

$c = 1.94''$

$\varepsilon_s = .0213 > .00207 \therefore$ OK

$\phi M_n = 328$ k-ft

585 k-ft > 328 k-ft $\therefore$ NO GOOD

SECTION @ D

$A_s = 7(.44) = 3.08 \text{ in}^2$ SEE SECTION @ A

$A_s = 3.08 \text{ in}^2 > A_{s,min} \therefore$ OK $\rho_{MAX} = .024$

$A_{s,MAX} = 17.25 \text{ in}^2$

$A_s < A_{s,MAX} \therefore$ OK

$\underline{M_n}$

ASSUME $f_s \geq f_y$

$$a = \frac{(3.08 \text{ in}^2)(60)}{.85(5)(48'')} = .91''$$

$$c = \frac{.91}{.8} = 1.14''$$

CHECK

$\varepsilon_s > \varepsilon_y$

$$\varepsilon_s = \frac{.003}{1.14}(14.8'' - 1.14'') = .036 > .00207 \therefore \text{ok}$$

$\phi = .9$

$$\phi M_n = .9(3.08 \text{ in}^2)(60)\left(14.8'' - \frac{.91''}{2}\right) = 2386 \text{ k-in} = 199 \text{ k-ft}$$

$M_u = 222$ k-ft $> \phi M_n = 199$ k-ft

$\therefore$ NO GOOD

SPOT CHECKS | CONT. BM | DRG | 6

PTTB20

SHEAR

(L) $V_u = \frac{(6.7^{k/ft})(23')}{2} = 77\ k$

(R) $V_u = 77\ k$

$V_c = 101\ k$

$\phi V_n = 38\ k$

$V_s = \frac{77k}{.75} - 101\ k = 2\ k$

$s = \frac{(.33\ in^2)(60)(14.8'')}{2\ k} = 146.5''$ ∴ USE 12''

CONCLUSION:

MATCHES 1@2'', R@12'', BUT NOT SURE HOW OFF OR HOW ON TO ORIGINAL CALCS.

CRITICAL MOMENTS

(L) $M_u = \frac{-w_u l_n^2}{11} = -\frac{(6.7^{k/ft})\left(\frac{39+23}{2}\right)^2}{11} = 585\ k\text{-}ft$

(M) $M_u = \frac{w_u l_n^2}{16} = \frac{(6.7)(23)^2}{16} = 222\ k\text{-}ft$

(R) $M_u = \frac{w_u l_n^2}{11} = \frac{(6.7)\left(\frac{23+38}{2}\right)^2}{11} = 567\ k\text{-}ft$

	TOP BARS	BOTTOM
LE	12 #6	—
MID	—	7#6
RE	12 #6	—

SECTION @ C_{RIGHT}

$A_s = 12(.44\ in^2) = 5.28\ in^2$

$A_{s,max} > A_s > A_{s,min}$ * SEE SECTION @ A

SPOT CHECKS | CONT. BM | DRG

SECTION @ C_{LEFT}

$d = 14.75''$

$A_s = 12(.44) = 5.28 \text{ in}^2$

$A_s > A_{s,min} \therefore$ OK * SEE SECTION @ A CALCULATIONS

$A_s < A_{s,max} \therefore$ * SEE SECTION @ B CALCULATIONS

$\underline{M_n}$

ASSUME $f_s \geq f_y$

$$a = \frac{(5.28 \text{ in}^2)(60)}{.85(5)(48'')} = 1.55 \text{ in}$$

$$c = \frac{1.55}{.8} = 1.94 \text{ in}$$

$$\varepsilon_s = \frac{.003}{1.94''}\left(14.75'' - \frac{1.94''}{2}\right) = .0213 > .00207 \therefore \text{OK}$$

$\phi = .9$

$$\phi M_n = .9(5.28 \text{ in}^2)(60)\left(14.75'' - \frac{1.94''}{2}\right) = 3929 \text{ k-in} = 328 \text{ k-ft}$$

$644 \text{ k-ft} >> 328 \text{ k-ft} \therefore$ NO GOOD

CONCLUSION:

CRITICAL MOMENT MUCH LARGER THAN ϕM_n

DUE TO POST-TENSIONING BEING NEGLECTED

SPOT CHECKS	CONT. BM	DRG

SECTION @ E_{LEFT}

SEE SECTION @ C_{RIGHT} FOR CALCS AND CONCLUSION

$$M_u = 567 \text{ k-ft} > \phi M_n = 328 \text{ k-ft}$$

BY INSPECTION -

SECTION @ E_{RIGHT}, SECTION @ F, AND SECTION @ G CAN BE ASSUMED TO HAVE THE SAME OUTCOMES AS SECTIONS @ C_{LEFT}, B, AND A RESPECTIVELY.

Appendix B
Photos

Rendering of Southwest Wiew

Northwest View

East View

Northeast View at Night