FDA OC/ORA Office Building

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Structural Option

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October 5th, 2009

Table of Contents

Executive Summary

The first technical assignment outlines structural existing conditions and structural concepts for the FDA OC/ORA Office Building, also known as Building 31 and Building 32. The five story office building is designed and laid out for the Office of Commissioner and Office of Regulatory Affairs. This office building is also designed with mixed spaces of food service areas and assembly areas.

The FDA OC/ ORA Office is a government building and was designed in accordance with resistance to progressive collapse. The perimeter beams on the floor system are designed as the primary progressive collapse. Building 31 has a one way slab system for the primary floor system with interior beams spanning between the columns. Building 32 has a two way flat slab system with no interior beams. Drop panels are located at the majority of the interior columns to provide punching shear resistance. The large atrium space in Building 31 uses post tension transfer girders to span the large area and to support the columns above.

ASCE 7-05 Structural Standard was used to perform the wind and seismic analysis on the office building. For simplification and ease of analysis only Wing B was considered for the lateral analysis. The simplification was only valid because expansion joints were provided between each wing of the office building. Wind pressure was not obtained from the structural drawings and no comparison could be made. The same base wind speed was used along with other basic design variables which allowed the assumption to be close to the design values. Seismic Base Shear was only provided for Wing A, and the lateral analysis was done Wing B. Therefore, no comparison of lateral forces could be made. However, the seismic design category of the office building was higher then I obtained from my analysis.

Typical floor system checks were performed on the second floor of Wing B. A typical column and typical two way flat slab were checked against the design. My spot checks did appear to be different then as designed; this is partially due my simplifying assumptions and the need for more researching information.

After compiling this report, a better understanding of the FDA OC/ORA Structural system and concept used in the office building. This report also includes comparisons of design loads to current code loads, and material specifications.

Introduction

Starting the fifth phase of the consolidation efforts by the FDA, the OC/ ORA Office building plans to move the Office of Commissioner (OC), Office of Regulatory Affairs (ORA) Office building to the White Oak Campus. On the site of the former US Navy facility at the Federal Research Center- Naval Ordnance Laboratory, the OC/ ORA Office Building sits on the southern end of the site, and forms its shape around the existing buildings.

Forming an S shaped building, the 500,000 S.F. office building was laid out and designed to mirror the existing buildings on the site and to form a unique face of the campus from the main drive off of New Hampshire Ave. Broken up into two buildings with four wings, Building 31 is comprised of Wing A, and Building 32 is comprised of wings B through D (Figure 1)

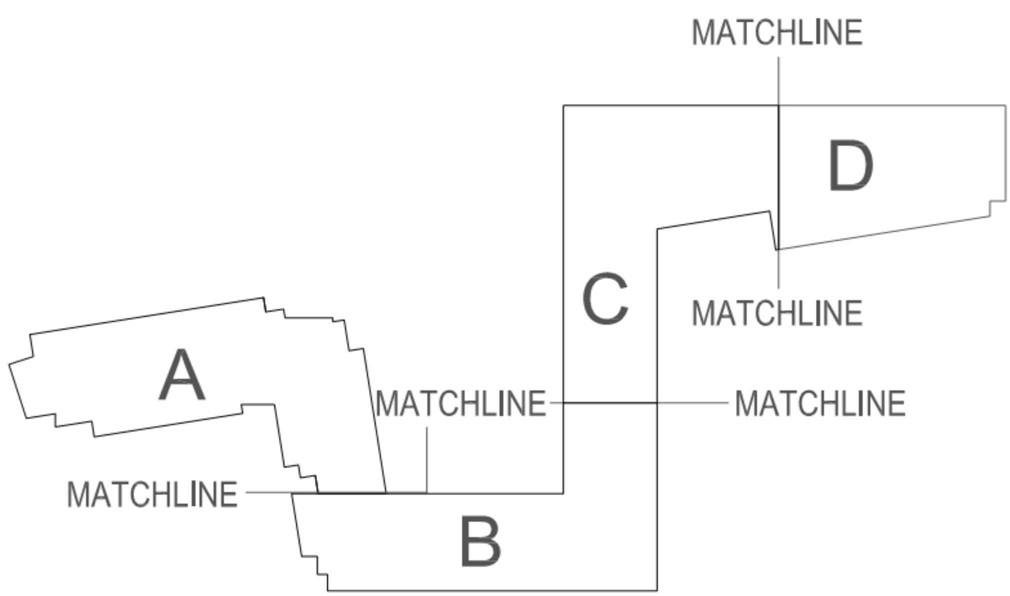


Figure 1: Key Plan

Structural System

Foundation:

The foundation of the building is separated into two categories. Spread footings that bear on undisturbed soil or spread footings that sit on a number of Geopiers. Schnabel Engineering conducted soil test to determine the bearing capacities of the soils. Where 95% compaction could not be met the use of Geopiers or vibropiers was recommended.

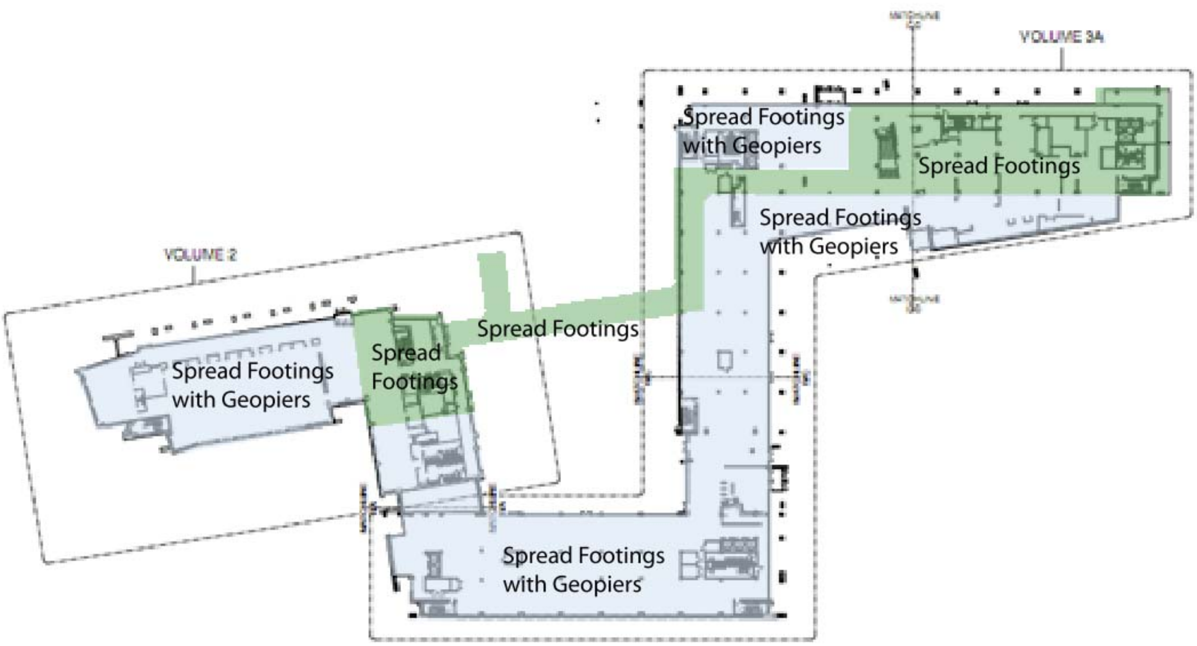


Figure 2: Foundation Key

For non-basement areas of Building 31 (Wing A), the western and central wings (Wings B and C) of Building 32, and the non-basement areas of Wing D, deep existing fill is expected within the majority of the buildings footprint. Geopiers are to be used in these areas to provide adequate bearing capacity (Figure 2). Geopiers use the concept of over consolidation to increase the soils bearing capacity. The 30 inch diameter Geopeirs should reach a depth of at least 10 feet. A detail of the typical spread footing with Geopiers is shown in Figure 3.

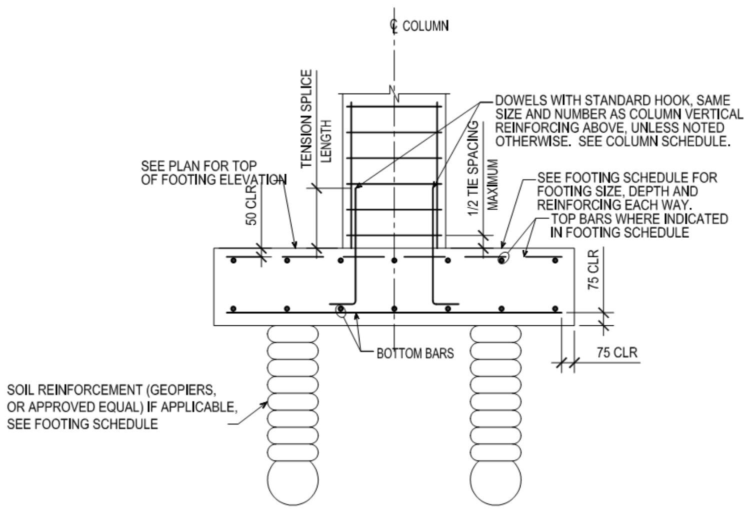


Figure 3: Typical Geopier Foundation Detail

For the basement level of Building 31 (Wing A), the basement level of Wing D of Building 32, and the underground tunnels, the foundations reach a sufficient depth where the bearing capacities on the spread footings are adequate (Figure 2).

Normal weight concrete was designed to be used with all the spread footings of the foundations. With a unit weight of 2350 kg/m^3 (147 pcf), the concrete has a 28 day strength of 28 MPa (4061 psi) concrete. A water to cement ratio of .48 is specified along with only 1% maximum chloride content.

Schnabel Engineering recommended the use minimum safe bearing capacities at the different locations of the foundation system. Where spread footings bear on undisturbed soil a bearing capacity of 192 kPa (4010 psf) was estimated. Beneath the spread footings of Wing A, where Geopiers were used, the estimated bearing capacity is 192 kPa (4010 psf). In the sections of Building 32 where Geopiers were used a bearing capacity of 287 kPa (5994 psf) was estimated.

Floor System:

Building 31:

Building 31 utilizes a one way slab floor system for the majority of the buildings layout. The typical one way slab construction is a 205 mm thick slab with 150 drop panels, unless noted differently on the drawings. On the first three floors of Wing A there is a large open assembly space, and prevents any typical bay spacing. However, on the fourth floor the typical bay spacing is 6660mm x 8150mm to 6000mm x 6000mm.

Resistance to progressive collapse was designed into the exterior reinforced beams of building 31 (Figure 4). Typical progressive collapse beam sizes ranging from 600mm x 1075mm to 460mm x 900mm. The interior beams on Building 31 are typical reinforce concrete beams with typical sizes of 460mm x 900mm to 460mm x 600mm.

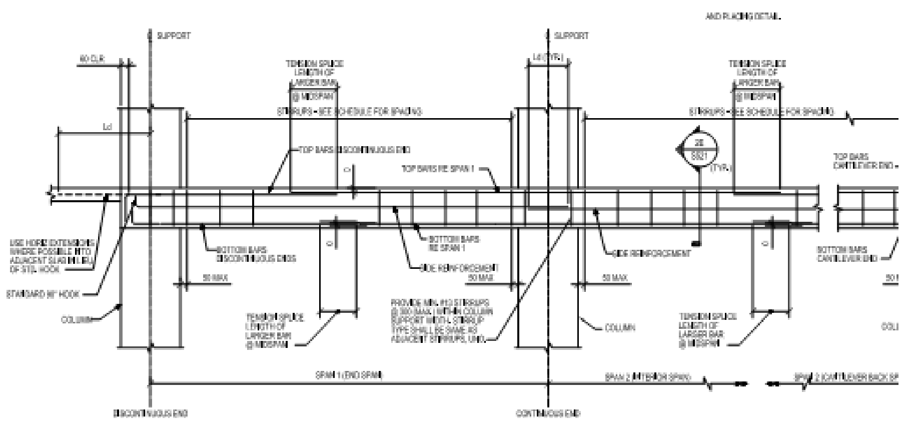


Figure 4: Progressive Collapse Beam

A large assembly pace on the first floor of Wing A that is open up through the third floor. On the fourth floor framing level, post tension transfer girders were designed to support the column loads above the fourth floor and transfer the load to the foundation (Figure 5). The post tension transfer girders are 900mm x 1800mm and have a post tension strand force of 4540 kN.

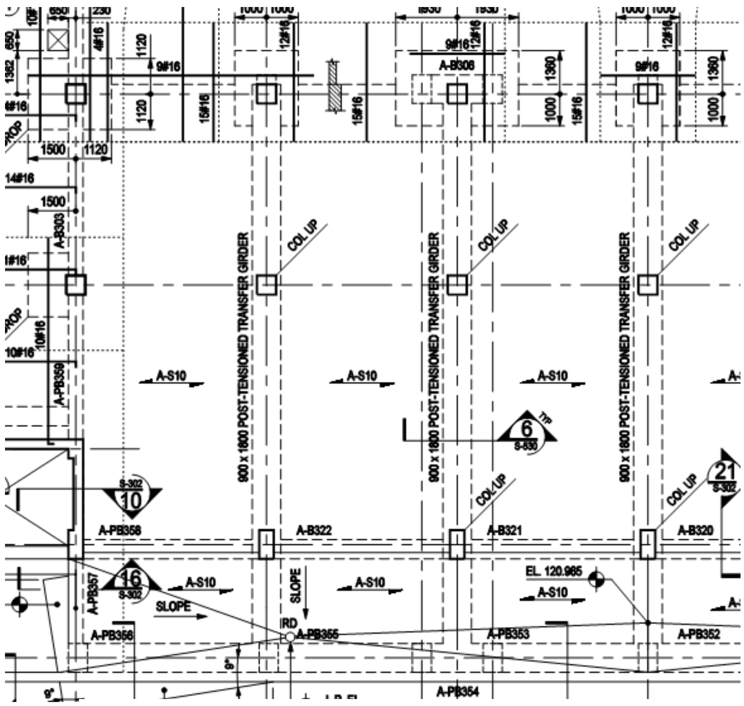


Figure 5: Framing Flan for Post Tension Transfer Girders

An atrium is provided between Wing A and Wing B that is primarily a steel superstructure with lightweight concrete on metal deck (Figure 6). The walkways over the atrium connecting the two wings are cast in place lightweight concrete on steel metal deck. The rib height on the metal deck is 50 mm with an additional 83 mm of concrete above. Supporting the walkway is W360 x 32.9 steel beams that frame into W360 x 32.9 girders with a shear connection. On the Wing A side of the atrium the girders site on an L152x152x9.5 that is attached to the concrete beam in Wing A. On the Wing B side on the atrium, an expansion joint is place, so the girders rest on a sliding connection that is connected to a beam in Wing B (Figure 7 and 8).

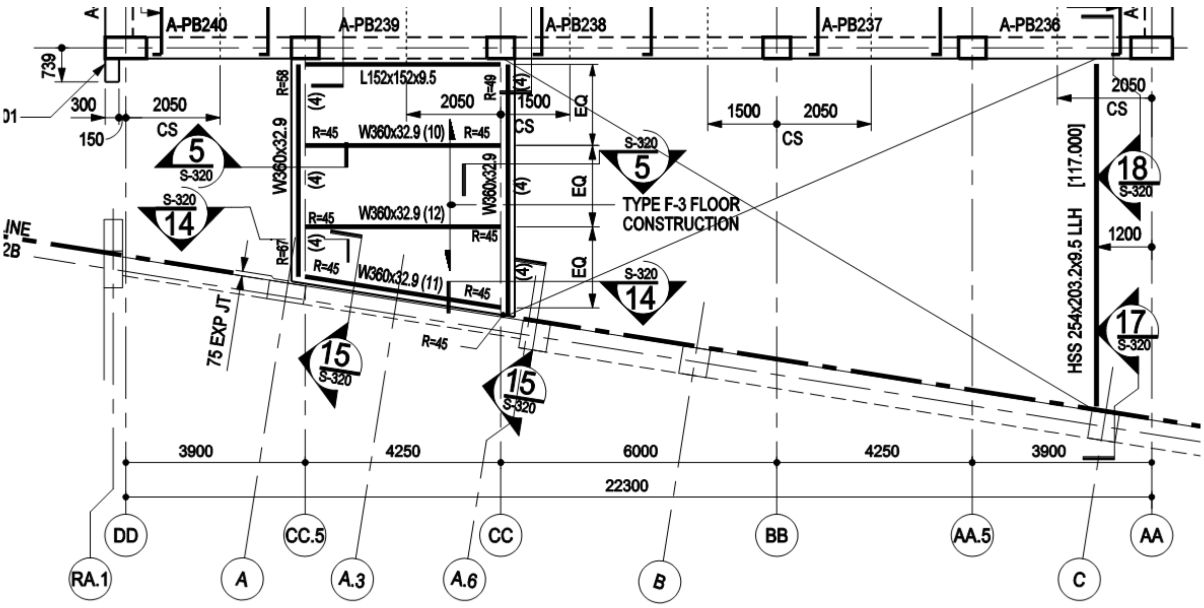


Figure 6: Wing A Atrium

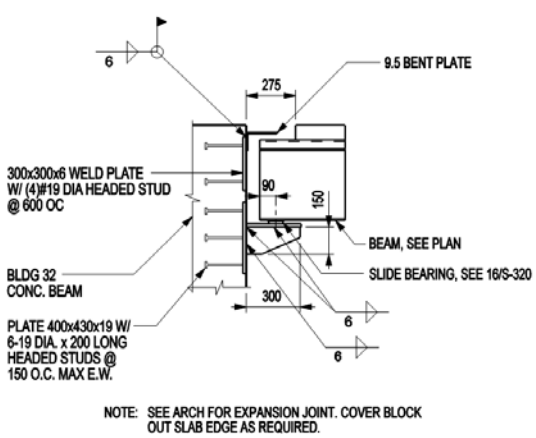


Figure 7: Expansion Joint Detail

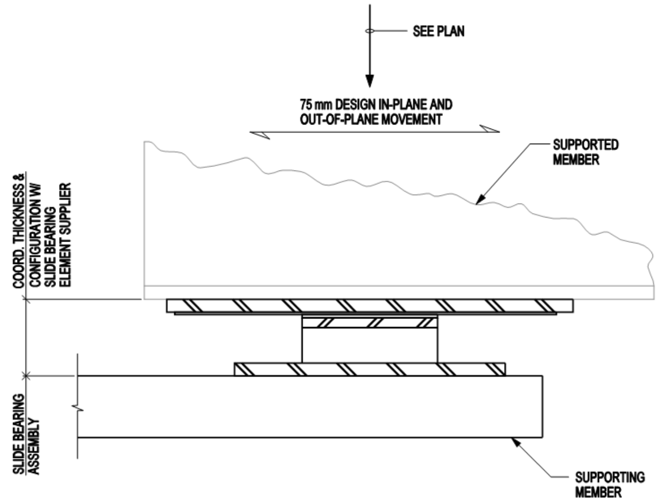


Figure 8: Sliding Connection Detail

Building 32:

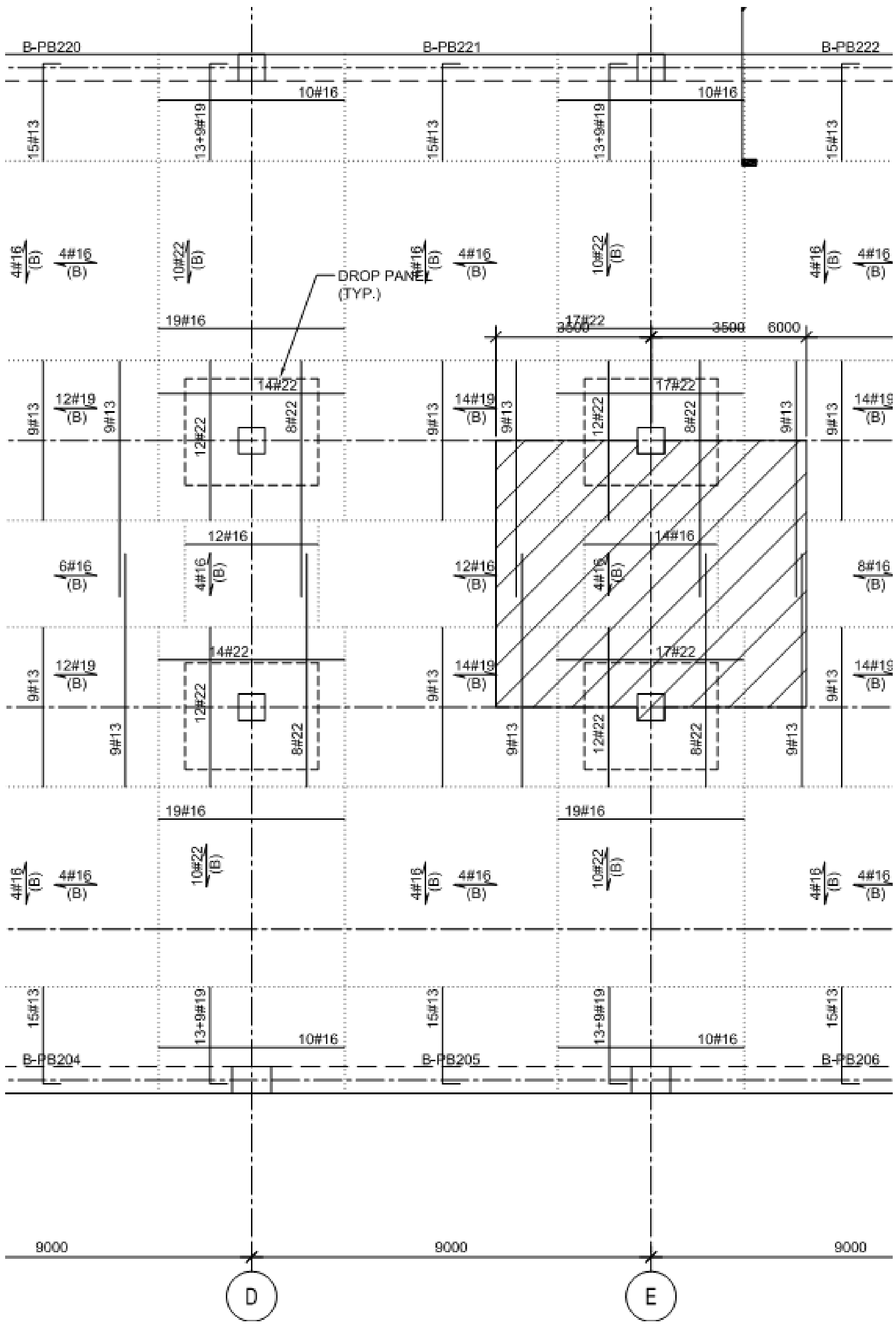


Figure 9: Typical Two Way Floor System

Building 32 utilizes a two way flat slab system for the majority of the buildings floor system. A 150 mm thick slab on grade is provided for the ground level and the basement levels of the building. The two way flat slab is typically 240 mm thick with a 180 drop panel, unless noted differently on the structural drawings. The typical bay spacing for Building 32 is 9000mm x 8400mm (Figure 9).

Resistance to progressive collapse was designed into the exterior reinforced concrete beams of building 32. Typical progressive collapse beam sizes ranging from 600mm x 1040mm to 400mm x 1040mm (Figure 10).

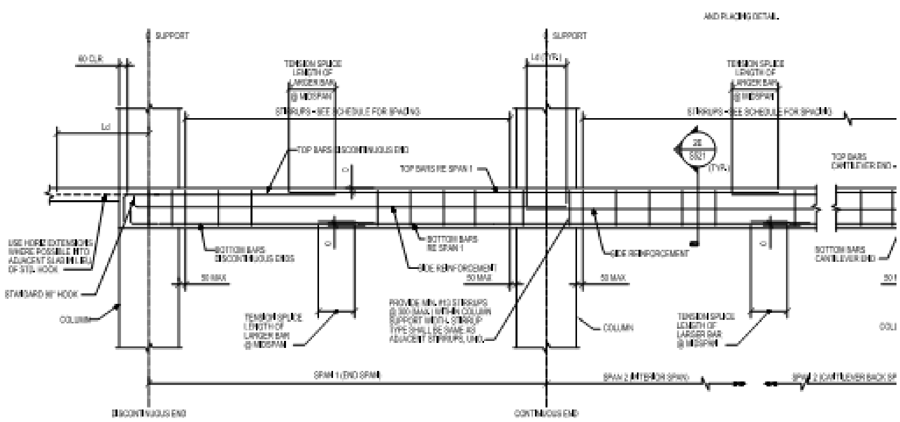


Figure 10: Progressive Collapse Beam

Atriums are provided between the connections of Wings B and C (Figure 11), and wings C and D (Figure 12). The floor system for the atriums is a cast in place lightweight concrete on metal deck. The rib height on the metal deck is 50 mm with an additional 64 mm of concrete above. Supporting the walkway is W150 x 30 steel beams that frame into W610 x 217 girders with a shear connection. Expansion joints at the Intersections of the wings are provided and sliding connections are required at the atrium walkways.

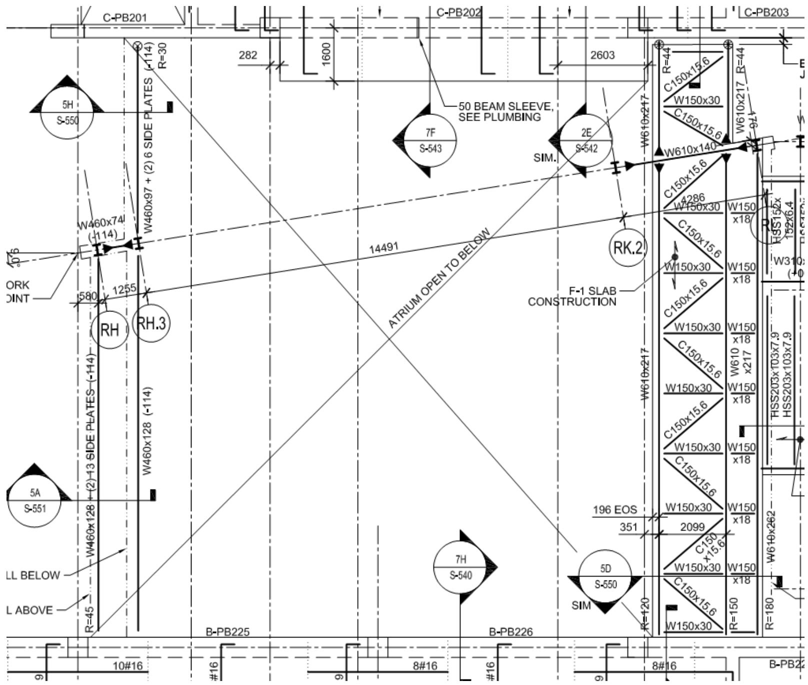


Figure 11: Wing B Atrium

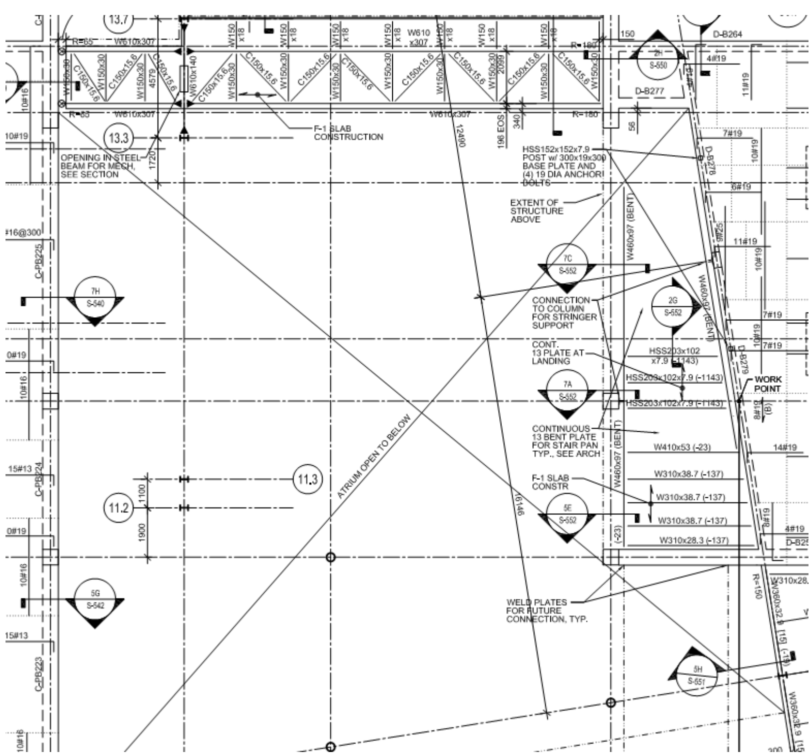


Figure 12: Wing C Atrium

Columns

Typical reinforced concrete columns were designed for the FDA OC/ ORA Office Building. Designed as the primary gravity system, the typical sizes of the columns are 600mm x 600mm, 900mm x 600mm, and 600 mm diameter. Various types of columns are provided ranging from square columns, rectangular columns and circular columns (Figure 13). The concrete for the columns is a normal weight concrete with 28 day strength of 28 MPa (4061 psi). The slab and the beams are monolithic with the columns forming a continuous system.

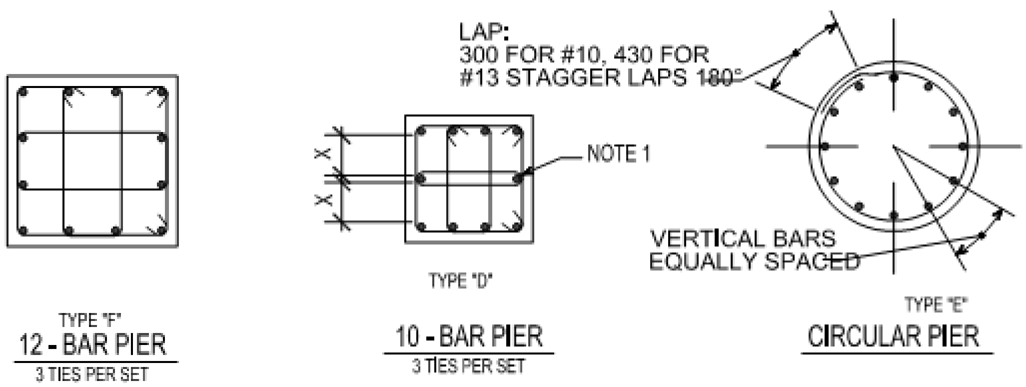


Figure 13: Typical Column Details

Lateral System

Ordinary reinforced concrete shear walls were design for the primary lateral resisting system. The typical shear wall has #16 at 300mm (#5 at 11.82 inches) for both vertically and horizontal reinforcement with 13 #16 (13 #5) for the end zone reinforcement and #13 ties at 300mm (#5 ties at 11.81 inches) for the vertical reinforcement (Figure 14 and 15).

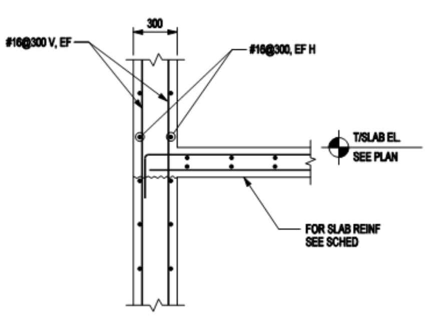


Figure 14: Shear Wall Detail

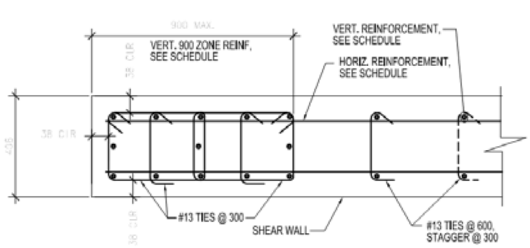


Figure 15: Shear Wall End Zone

Shear walls are provided around each elevator core and the stair shaft of Wing A, Figures 16 through 19 shows the location of the shears walls in each wing. At the intersection of each wing, in the atriums, slide bearing connections are provided at the expansion joints. These connections allow each wing's lateral systems to act independently of the other wing.

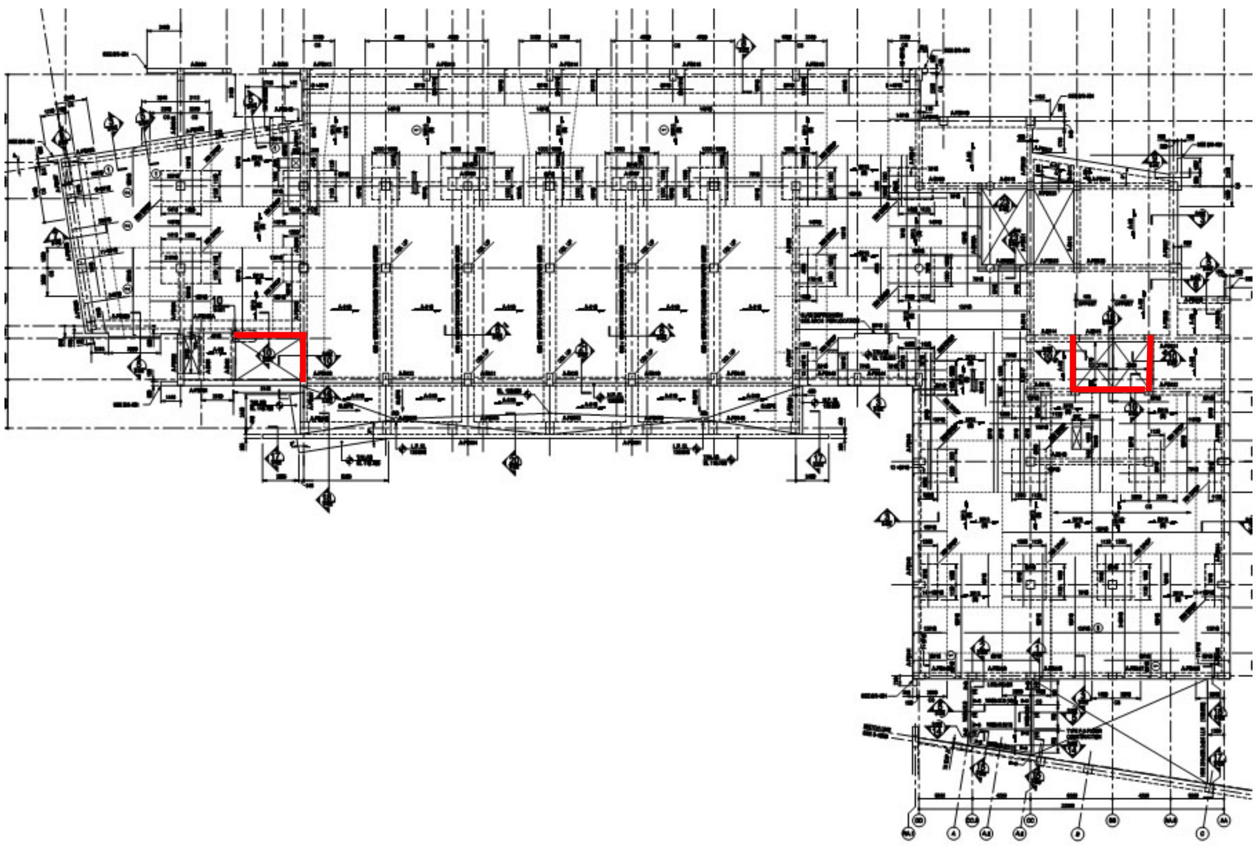


Figure 16: Shears Walls of Wing A

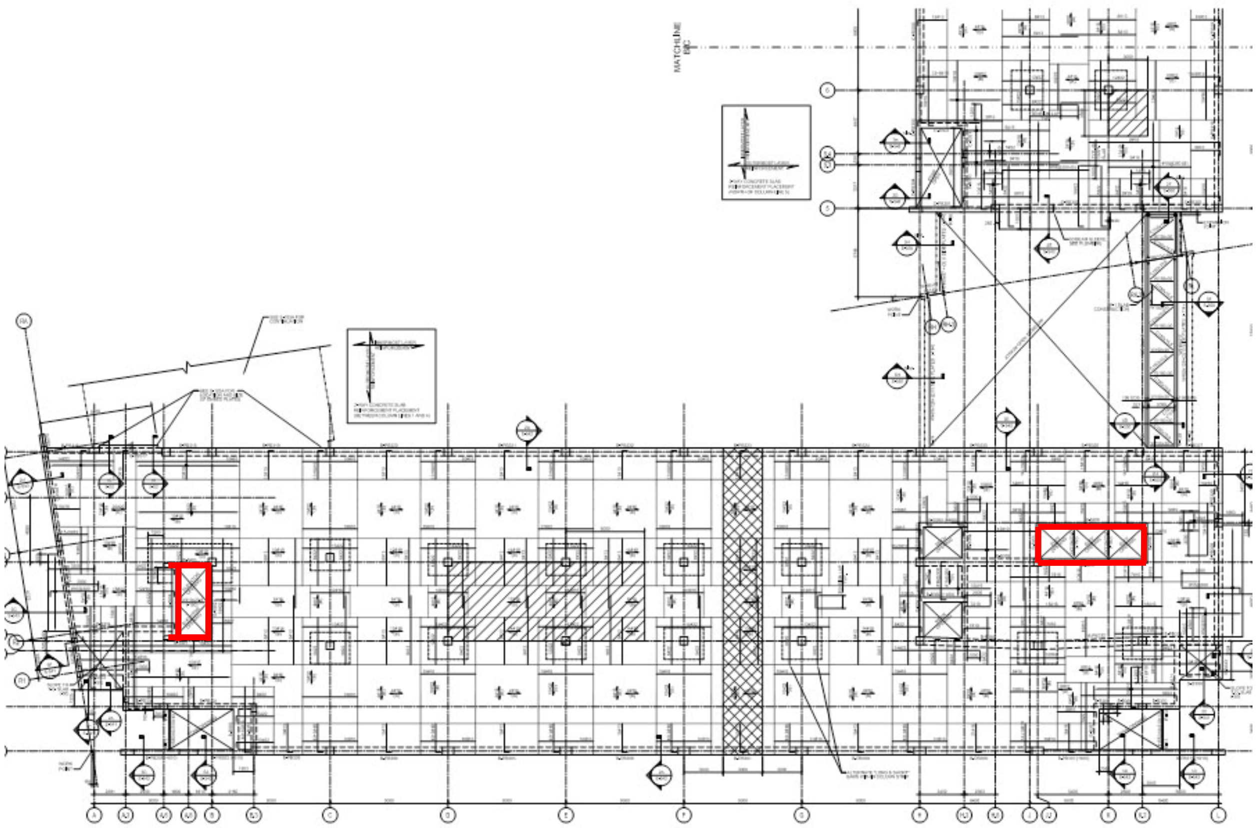


Figure 17: Shear Walls of Wing B

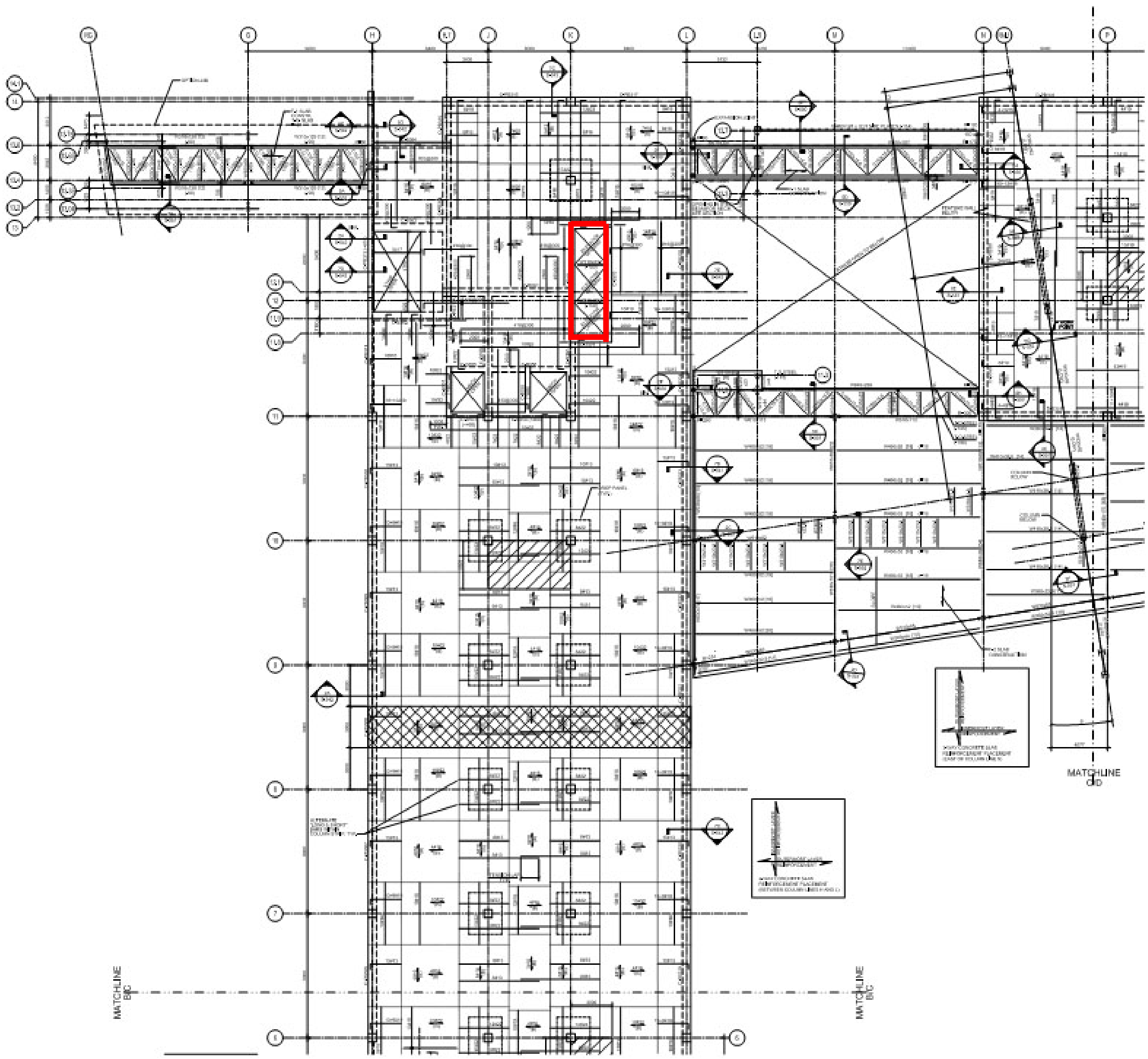


Figure 18: Shear Walls of Wing C

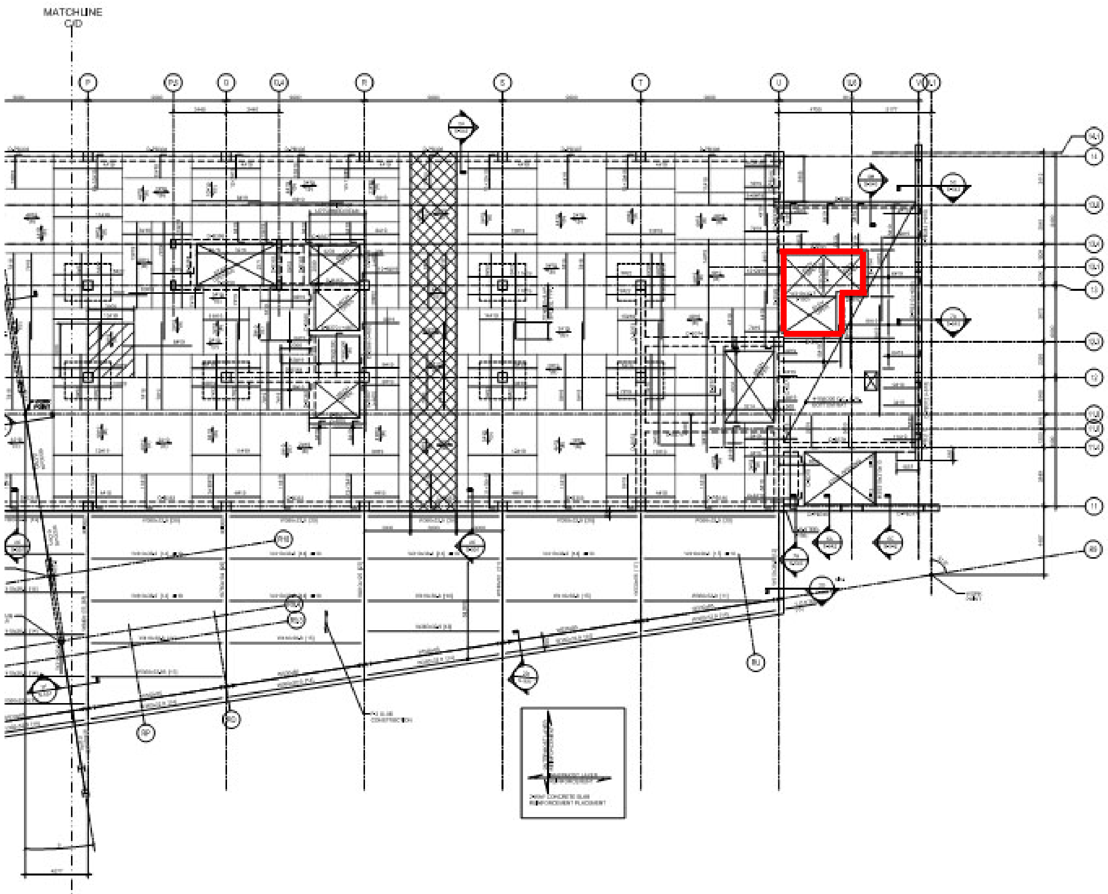


Figure 19: Shear Walls of Wing D

Load Paths

Gravity Load Resisting System:

Reinforced Concrete columns make up the primary gravity load resisting system. The live load and superimposed dead load that sits on the floor system is transferred to the reinforced concrete beams. Reinforced concrete columns pick up the loads from the beams and the load is transferred to the buildings foundations. In Wing A reinforced concrete columns bear on a post tension transfer girder. There the load is transferred from the columns into the transfer girder. Surrounding columns that the transfer girders bear on transfer the load from the girders into the columns. Columns then transfer the load into the foundation of the building.

Resistance to progressive collapse has been designed for the office building. Design considerations that are involved with this design are removing an exterior column, and the floor system above and the adjacent columns are designed to carry the additional load.

Lateral Load Resisting System:

Reinforced concrete shear walls are the primary lateral load resisting system. Lateral force is transmitted against the curtain wall of the building. Rigid floor system picks up each base shear at each level and transmits the lateral force to the shear walls located around each elevator core. Shear walls are design to resist the moment from the lateral load. The axial forces are transmitted through the shear walls onto large spread footings.

Each wing acts independently with respect to the others wings. This is primarily due to the large expansion joints provided between each wing, along with the slide bearing connections design at the atriums connections.

Materials

Structural Steel

W & WT Shapes	ASTM A992M
Channels	ASTM A36M
Angles	ASTM A36M
Rectangular and Round HSS	ASTM A500 Grade B
Round HSS	ASTM A500 Grad B

Steel Pipe	ASTM A36M
Steel Plates	ASTM A36M
Steel Bars	ASTM A36M

Metal Decking

52 mm Composite Floor Deck	20 Gage
52 mm Composite Floor Deck	18 Gage
38 mm Roof Deck	20 Gage
14 mm Form Deck	26 Gage

Cast-in-Place Concrete

Interior Pads and Curbs	f'c = 28 Mpa (4000 psi)
Exterior Retaining Walls	f'c = 28 Mpa (4000 psi)
Footings, Walls, Piers	f'c = 28 Mpa (4000 psi)
Slab on Grade	f'c = 28 Mpa (4000 psi)
Slabs, Beams	f'c = 28 Mpa (4000 psi)
Columns	f'c = 28 Mpa (4000 psi)
Lean Concrete	f'c = 17 Mpa (3000 psi)
Slab on Metal Deck	f'c = 28 Mpa (light weight concrete)

Reinforcement

Deformed Bars	ASTM A615M Grade 400
Deformed Bars (Wieldable)	ASTM A706M
Welded Wire Fabric	ASTM A185M

Codes and References

Design Codes:

National Model Code:

GSA Facilities Standards for the Public Building Service

International Building Code 2003

Structural Standards:

GSA Facilities Standards for the Public Building Service

ASCE 7-02, Minimum Design Loads for Buildings and other Structures

Design Codes:

AISC-ASD, Specifications for Structural Steel Buildings – Allowable Stress Design

ACE 318-02, Building code Requirements for Structural Concrete

Thesis Codes

National Model Code:

GSA Facilities Standards for the Public Building Service - 2005

2006 International Building Code

Structural Standards

GSA Facilities Standards for the Public Building Service – 2005

ASCE 7-05, Minimum Design Loads for Buildings and other Structures

Design Codes:

Steel Construction Manual 13th edition, American Institute of Steel Construction

ACI 318-05, Building Code Requirements for Structural Concrete, American Concrete Institute

Design of Buildings to Resist Progressive Collapse 2005, Unified Facilities Criteria

Gravity Loads

The primary design guide lines for the FDA OC/ORA Office Building are the GSA Facilities Standards for the Public Service-2005, and the ASCE 7-02. The GSA outlines general requirements to the required live load for office interiors and the telecom room. The GSA Facilities Standards for the Public Building Service requires the designer to implement progressive collapse design into the structural design.

For the use with Senior Thesis the latest design codes are to be used with the analysis of the buildings gravity and lateral systems. When comparing to the designed loads and the ASCE 7-05 required loads, only one major difference appeared. ASCE 7-05 requires a load of 100 psf for special purpose roofs, specifically green roofs. Comparing to the designed load of 31.33 psf, one possible reason for the significant difference is the dead load; the structural engineering added a green roof dead load.

Live Loads					
	Design		GSA 05	ASCE 7-05	
Location	kPa	psf	psf	psf	
Office	3.8	79.36	80	50	
Typical Roof	1.5	31.33		20	
Public Lobbies	4.8	100.25		100	
Mech Room	7.3	152.46		150	(Assumed)
Telecom Room	12	250.63	250	150	
Redestrian Bridge	4.8	100.25		60	
Balconies	4.8	100.25		100	
High Density Filing	12	250.63		250	(Assumed)
Green Roof	1.5	31.33		100	

Figure 20: Live Loads

Dead Loads		
	psf	
Superimposed Dead Load (MEP, Ceiling)	15	(Assumed)
Roofing System	40	(Assumed)
Mechanical Unit	150	(Assumed)
Exteior Curtain Wall	30	(Assumed)
Atrium Cutrain Wall	20	(Assumed)
Mechanical Pentouse Walls	20	(Assumed)

Figure 21: Dead Loads

SNOW LOADS (S)					ASCE 7-05 Ref.
Ground Snow Load	p_g =	25	psf		Figure 7-1
Exposure Factor	C_e =	1		Terrain Category B	Table 7-2
Thermal Factor	C_t =	1			Table 7-3
Importance Factor	I =	1		Occupance Category II	Table 7-3
	p_f =	17.5	psf	p_f = .7*Ce*Ct*I*pg	Eq. 7-1
	p_{fmin} =	20	psf	$p_{fmin} = p_g$*I	Section 7.3
	p_f =	**20**	**psf**		
Snow Drift					
Snow Density	γ =	30	pcf		Eq. 7-3
	h =	14.66	ft		
	h_d =	0.67	ft		
	h_c =	13.99	ft		
Snow Surcharge	**S_d =**	**52.5**	**psf**		Section 7.7.1

Figure 22: Snow Loads

Lateral Loads

To simplify the lateral analysis of the office building, I decided to look at only one wing for my wind and seismic calculations. This was aloud because the wings have different lateral systems that do not interact with the other wings. The structural engineering provided large expansion joints in the atriums that connect each wing, along with slide bearing connections. The slide bearing connections allow the wings to move and react independent from the lateral forces. Wing B was chosen for the wind and seismic calculations that follow.

Wind Loads

The wind loads were determined using Method 2 of the ASCE 7-05 Chapter 6. My first assumption under the wind analysis was that the 5 story reinforced concrete structure would act rigidly under lateral loads. After further calculation under the Chapter 6 commentary, the 5 story structure did not act rigidly. This is partially due to the size of the shear walls that were provided in Wing B. However, in the East to West direction the structure did meet the requirements to be rigid. The wind pressures were calculated out in the following tables.

Method 2: Approximate Fundamental Frequency					
H	70.14	ft.			
Ab	21435.00	s.f.			
N-S			W-E		
B =	297.55	ft.	B =	137.44	ft.
L =	137.44	ft.	L =	297.55	ft.
n =	4		n =	4	
A1 =	19.375	s.f.	A5	11.948	s.f.
A2 =	19.375	s.f.	A6	11.948	s.f.
A3 =	9.6875	s.f.	A7	26.647	s.f.
A4 =	9.6875	s.f.	A8	26.647	s.f.
D1 =	19.685	ft.	D5	12.139	ft.
D2 =	19.685	ft.	D6	12.139	ft.
D3 =	9.843	ft.	D7	27.07	ft.
D4	9.843	ft.	D8	27.07	ft.
Cw =	0.018		Cw =	0.042	
n_1 =	0.732		n_2 =	1.121	
n < 1, Therefore flexible sturcture			n > 1, Therefore rigid sturcture		
ASCE 7-05 C6-16					

Figure 23: Approximate Fundamental frequency

Method 2: N-S Gust Effect Factor: flexible Structures			
gq = gv =	3.400		ASCE 7-05 6.5.8.2
gr =	4.114		ASCE 7-05 Eq. 6-9
z =	42.085		
zmin =	30.000		ASCE 7-05 Table 6-2
c =	0.300		ASCE 7-05 Table 6-2
Iz =	0.288		ASCE 7-05 Eq. 6-5
∈ =	0.333		ASCE 7-05 Table 6-2
ℓ =	320.000		ASCE 7-05 Table 6-2
Lz =	347.019		ASCE 7-05 Eq. 6-7
Q =	0.778		ASCE 7-05 Eq. 6-6
V =	90.000	mph	
b =	0.450		ASCE 7-05 Table 6-2
α =	0.250		ASCE 7-05 Table 6-2
Vz =	63.123		ASCE 7-05 Eq. 6-14
N1 =	4.022		ASCE 7-05 Eq. 6-12
Rn =	0.058		ASCE 7-05 Eq. 6-11
Rh =	0.232		
	η =	3.739	
RB =	0.061		
	η =	15.863	
RL =	0.040		
	η =	24.529	
R =	0.174		ASCE 7-05 Eq. 6-10
Gf =	0.813		ASCE 7-05 Eq. 6-8

Figure 24: Gust Effect Factor N-S

Method 2: E-W Gust Effect Factors, G and G_f			
n_1 =	1.43		ASCE 7-05 C6-17
	n_1 > 1 therefore structure is rigid		
H/L =	0.51		
	If H/L ≤ 4 then G = .85		ASCE 7-05 6.5.8.1
G =	0.85		

Figure 25: Gust Effect Factor E-W

Basic Wind Information				(ASCE Ref)
Basic Wind Speed	$V =$	90	mph	ASCE 7-05 Figure 6-1
Directionality Factor	$k_d =$	0.85		ASCE 7-05 Table 6-4
Importance Factor	$I =$	1.0		ASCE 7-05 Table 6-1
Exposure Category		B		ASCE 7-05 6.5.6
Topgraphic Factor	$k_{zt} =$	1.0		ASCE 7-05 6.5.7
	$z_g =$	1200	ft	
	$\alpha =$	7		
Velocity Presure Expusure Coefficient evaluated at Height z	$K_z =$	Varies		
Velocity Presure Expusure Coefficient evaluated at Mean Roof Height	$K_h =$	0.8930		
Velocity Pressure at Height z	$q_z =$	Varies		
Velocity Pressure at Mean Roof Height	$q_h =$	15.7		
Equivalent height of Structure	$h =$	70.1		
Intensity of turbulance	$I_z =$	0.3		
Integral Length Scale of Turbulence	$L_z =$	347.0		
Background Response Factor (N/S)	$Q =$	0.778		
Background Reponse Factor (E/W)	$Q =$	0.829		
Gust Effect Factor (N/S)	$G =$	0.813		
Gust Effect Factor (E/W)	$G =$	0.850		
Internal Pressure Coefficients	$G_{cpi} =$	± 0.18		
External Pressure Coefficient (Windward)	$C_p =$	0.8		
External Pressure Coefficient (N/S Leeward)	$C_p =$	-0.3		
External Pressure Coefficient (E/W Leeward)	$C_p =$	-0.5		
External Pressure Coefficient (Sidewall)	$C_p =$	-0.7		
External Pressure Coefficient (Roof Section 1)	$C_p =$	-0.9	(From Windward Edget to 70.14 ft.)	
External Pressure Coefficient (Roof Section 2)	$C_p =$	-0.5	(From 70.14 to 140.28 ft.)	
External Pressure Coefficient (Roof Section 3)	$C_p =$	-0.3	(From 140.28 to 297.53 ft.)	
Basic Building Information				
Mean Building Height	$h =$	21379	mm	
		70.14	ft	
N-S	$L =$	137.44	ft	
	$B =$	297.55	ft	
E-W	$L =$	297.55	ft	
	$B =$	137.44	ft	

Figure 26: Basic Wind Information

Design Wid Pressures p in N-S Direction										
Location	Story Height		Level Height		K_z	q_z (psf)	External Pressure qGCp (psf)	Internal Pressure qh(Gcpi) (psf)	Net presure p (psf)	
	(mm)	(ft)	(mm)	(ft)					+(Gcpi)	-(Gcpi)
Windward	0	0	0	0						
	4700	15.420	4700	15.4199	0.5793	10.210	6.638	± 2.833	**3.804**	**9.471**
	3930	12.894	8630	28.3136	0.6891	12.146	7.896	± 2.833	**5.063**	**10.729**
	3930	12.894	12560	41.2073	0.7671	13.521	8.790	± 2.833	**5.957**	**11.623**
	3930	12.894	16490	54.1010	0.8291	14.614	9.501	± 2.833	**6.668**	**12.334**
	3930	12.894	20420	66.9948	0.8814	15.535	10.099	± 2.833	**7.266**	**12.932**
	959	3.146	21379	70.1411	0.8930	15.740	10.232	± 2.833	**7.399**	**13.066**
Leeward				All	0.8930	15.740	-3.837	± 2.833	**-6.670**	**-1.004**
Side				All	0.8930	15.740	-8.953	± 2.833	**-11.786**	**-6.120**
Roof	(From Windward Edget to 70.14 ft.)			70.1411	0.8930	15.740	-11.511	± 2.833	**-14.345**	**-8.678**
	(From 70.14 to 140.28 ft.)			70.1411	0.8930	15.740	-6.395	± 2.833	**-9.228**	**-3.562**

Figure 27: Design Wind Pressure for N-S

Design Wid Pressures p in E-W Direction										
Location	Story Height		Level Height		K_z	q_z (psf)	External Pressure qGCp (psf)	Internal Pressure qh(Gcpi) (psf)	Net presure p (psf)	
	(mm)	(ft)	(mm)	(ft)					+(Gcpi)	-(Gcpi)
Windward	0	0	0	0						
	4700	15.420	4700	15.4199	0.5793	10.210	6.943	± 2.833	**4.110**	**9.776**
	3930	12.894	8630	28.3136	0.6891	12.146	8.259	± 2.833	**5.426**	**11.092**
	3930	12.894	12560	41.2073	0.7671	13.521	9.194	± 2.833	**6.361**	**12.027**
	3930	12.894	16490	54.1010	0.8291	14.614	9.938	± 2.833	**7.105**	**12.771**
	3930	12.894	20420	66.9948	0.8814	15.535	10.564	± 2.833	**7.730**	**13.397**
	959	3.146	21379	70.1411	0.8930	15.740	10.703	± 2.833	**7.870**	**13.536**
Leeward				All	0.8930	15.740	-4.014	± 2.833	**-6.847**	**-1.180**
Side				All	0.8930	15.740	-9.365	± 2.833	**-12.198**	**-6.532**
Roof	(From Windward Edget to 70.14 ft.)			70.1411	0.8930	15.740	-12.041	± 2.833	**-14.874**	**-9.208**
	(From 70.14 to 140.28 ft.)			70.1411	0.8930	15.740	-6.689	± 2.833	**-9.522**	**-3.856**
	(From 140.28 to 297.53 ft.)			70.1411	0.8930	15.740	-4.014	± 2.833	**-6.847**	**-1.180**

Figure 28: Design Wind Pressure for E-W

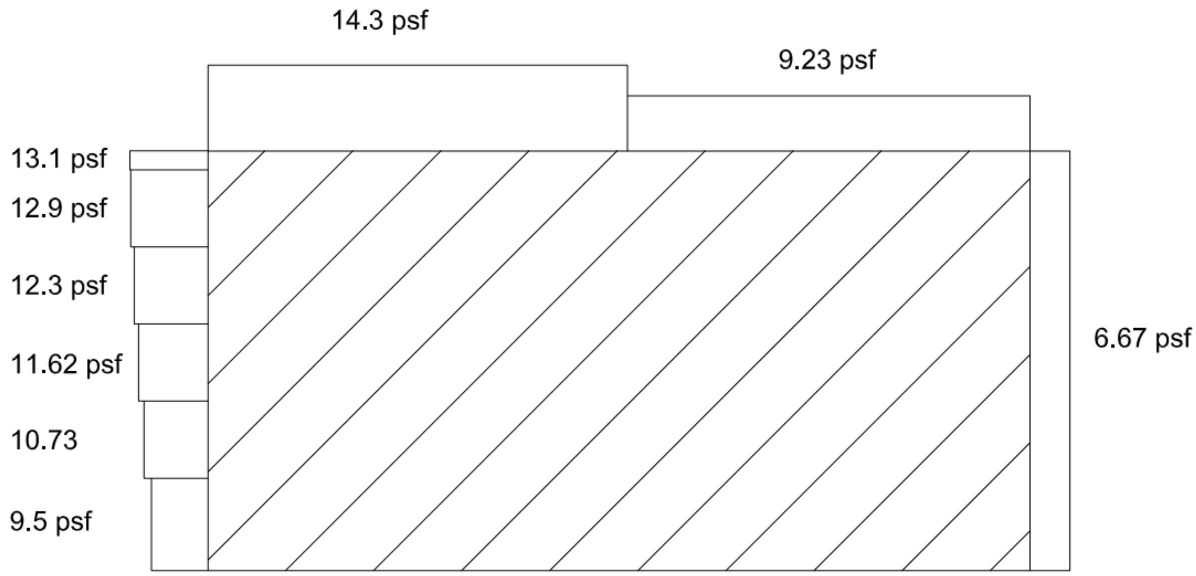


Figure 29: Wind Pressure Diagram N-S

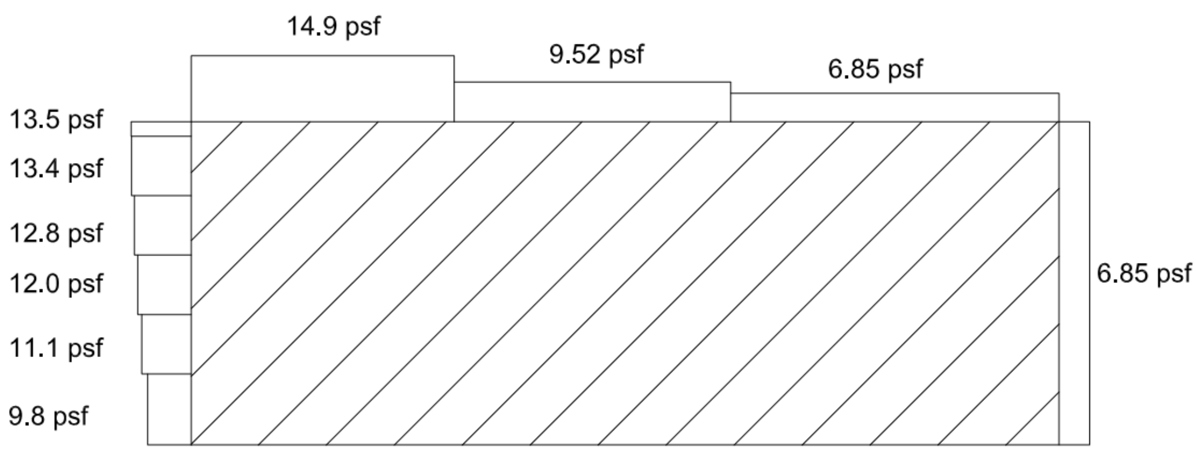


Figure 30: Wind Pressure Diagram E-W

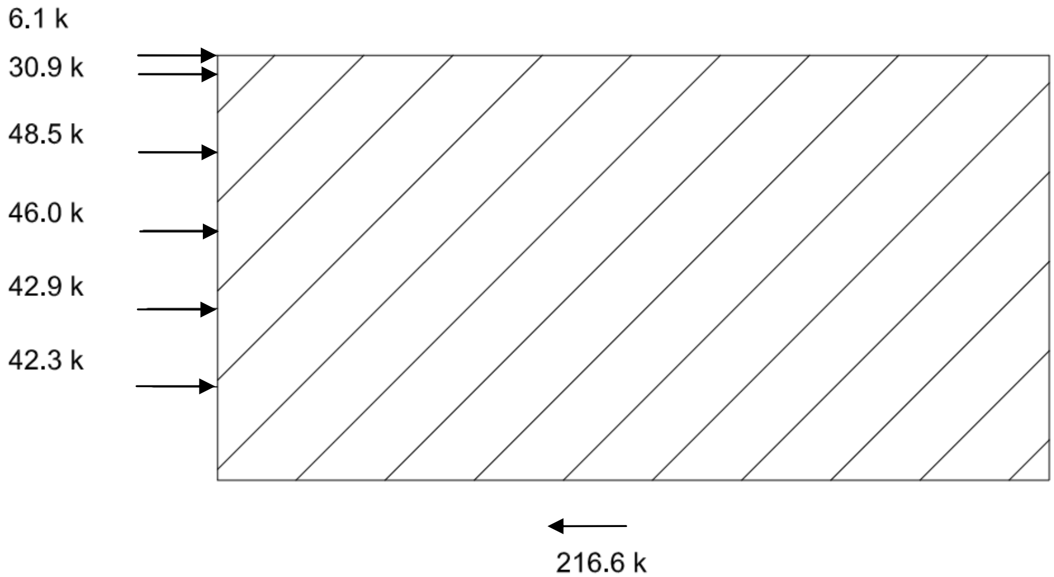


Figure 31: Wind Force Diagram N-S

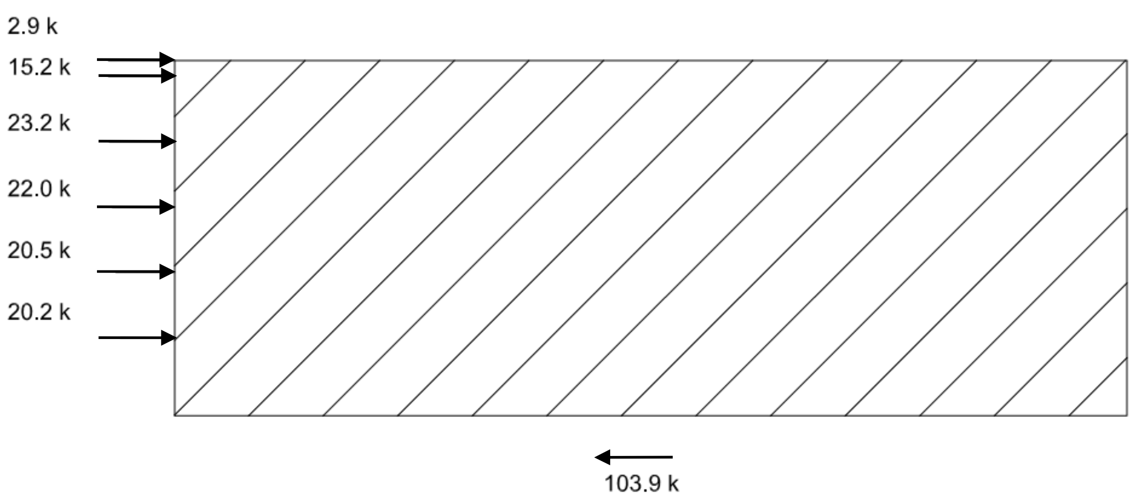


Figure 32: Wind Force Diagram E-W

In the West to East direction the wind pressures were slightly larger as seen in Figures 29 and 30. Figure 29 shows the wind pressures for the North to South direction and Figure 30 shows the wind pressures for the East to West direction. In the North to South direction the Base Shear controlled over the East to West direction, this is due to the large façade area in this direction. The wind forces are shown in Figures 31 and 32.

Seismic Loads

Seismic Loads for the FDA OC/ ORA Office Building were calculated using ASCE 7-05 Chapter 11 and 12. Initially the self weight of each floor needed to be estimated for the seismic calculations. This was done by assuming the framing systems for each floor were close enough to be approximated as the equal. The slab, beams and columns were all measured and their self weights were added up is Microsoft Excel. The exterior wall weight was assumed to be 30 psf because of the cmu backup behind the brick veneer curtain wall. The total weights were totaled in the Figure 33.

Building Weight by Floor (Kips)										
	Slab	Beams	Columns	Drop Panels	Ext Wall	SDL	Atrium Walk	Atrium Roof	roofing material	Total
Floor 2	2460	797	407	49	314.00	313	343			4683
Floor 3	2460	797	371	49	314.00	313	343			4647
Floor 4	2460	797	371	49	314.00	313	343			4647
Floor 4	2460	797	371	49	314.00	313	343			4647
Roof	2460	797	188	49	115.00	538		149	833	5129
Penthouse		9	3		38.00				49	99
									Total	**23852**

Figure 33: Building Weight

Seismic Design Variables			(ASCE 7-05 Ref.)
Soil Classification		C	
Occupancy		II	(Table 1-1)
Structural System		Building Frame System: Ordinary reinforce concrete shear walls	(Table 12.2-1)
Spectral Response Acceleration, short	S_s	0.155	(USGS)
Spectral Response Acceleration, 1 s	S_1	0.05	(USGS)
Site Coefficient	Fa	1.2	(Table 11.4-1)
Site Coefficient	Fv	1.7	(Table 11.4-2)
Soil Modified Accelerationd, short	S_{ms}	0.186	(Eq. 11.4-1)
Soil Modified Accelerationd, 1 s	S_{m1}	0.085	(Eq. 11.4-2)
Design Spectral Acceleration, short	S_{DS}	0.124	(Eq. 11.4-3)
Design Spectral Acceleration, 1 s	S_{D1}	0.057	(Eq. 11.4-4)
Approximate Period Parameter	C_t	0.002	(Table 12.8-2)
Approximate Period Parameter	x	0.750	(Table 12.8-2)
Building height (above grade)	h_n	70.14 ft	
Approximate Fundamental Period	T_a	0.485	(Eq. 12.8-7)
Fundamental Period	T_s	0.460	
80% of Fundamental Period	$.8T_s$	0.368	
Seismic Design Category	S_{DC}	A	(Table 11.6-1)
Seismic Response Coefficient	C_s	0.012	(Eq 12.8-3)
Structure Period Exponet	k	1.250	(Sec. 12.8.3)
Seismic Base Shear	V	270.3 kips	(Eq. 12.8-1)

Figure 34: Seismic Design Variables

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)
2	4683	15.82	31.55	147752	0.06	15.21	270.30	240.6306
3	4647	28.31	65.30	303457	0.12	31.24	255.09	884.4017
4	4647	41.2	104.38	485059	0.18	49.94	223.85	2057.328
5	4647	54.09	146.69	681662	0.26	70.17	173.91	3795.754
Roof	5129	66.98	191.62	982798	0.37	101.18	103.74	6776.75
PH	100	82.61	249.05	24905	0.01	2.56	2.56	211.8047
						$\Sigma F_x = V_x =$	**270**	**kips**
						$\Sigma M_x =$	**13967**	**ft-kips**

Figure 35: Seismic Loads

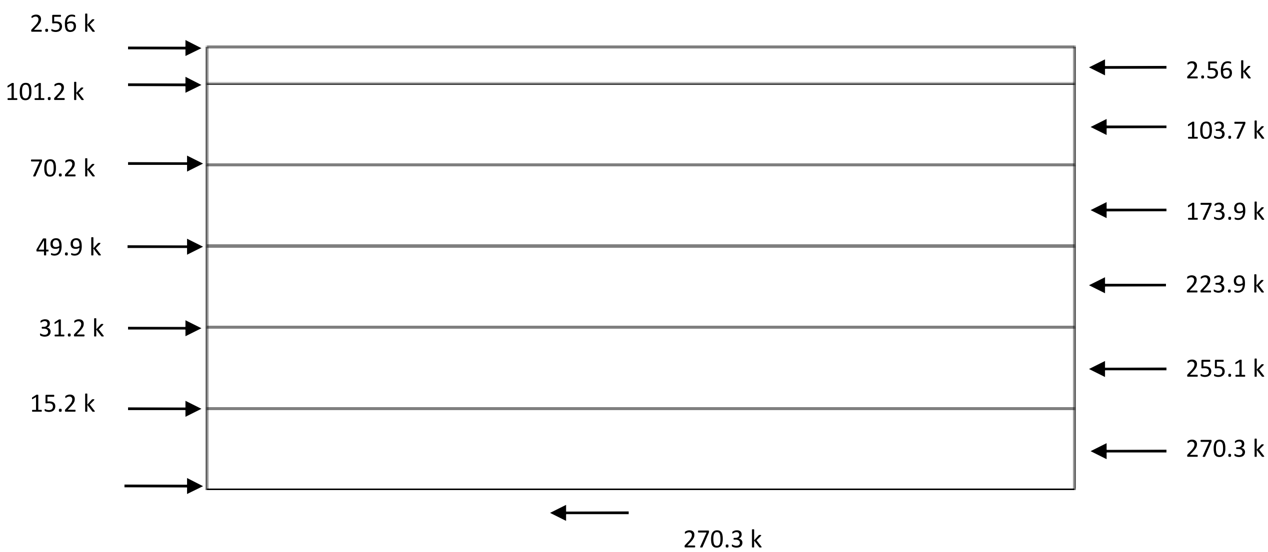


Figure 36: Seismic Load Diagram

The Seismic Design Category was calculated using Table 11.6-1 and 11.6-2 in the ASCE 7-05. A SDC of A was determined for the Wing B of the office building (Figure 34). This is different than the SDC of B that was design by the structural engineer. A possible reason for this difference is the use of the USGS Ground Motion Parameter gave a much lower mapped acceleration. The story lateral forces and story shear forces were calculated with the equivalent lateral force procedure, using excel in Figure 35. Figure 36 shows a diagram of the story forces along with the calculated base shear of 270.3 k.

Spot Checks

Gravity Column Spot Check

A column above the second floor framing system was selected to be check for compliance with newer code standards. After checking the axial strength of Column D-3, it was determined that my results were significantly lower than the designed column. The structural engineer designed the column to be 23.62" square columns with 12 #9 for vertical reinforcing. When checking the pure axial capacity of the column is was determined that the column was overdesigned. This difference is due to the assumption that the column took only axial load from gravity and no moment. Also the assumption of dead loads was over estimated with the superimposed dead load. A copy of the calculations is provided in Appendix C.

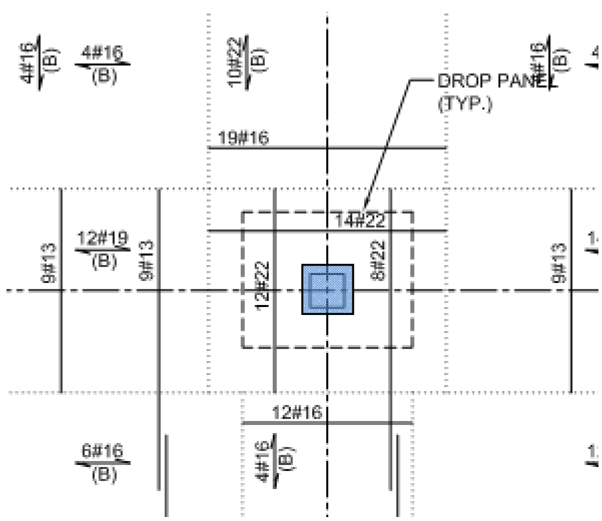
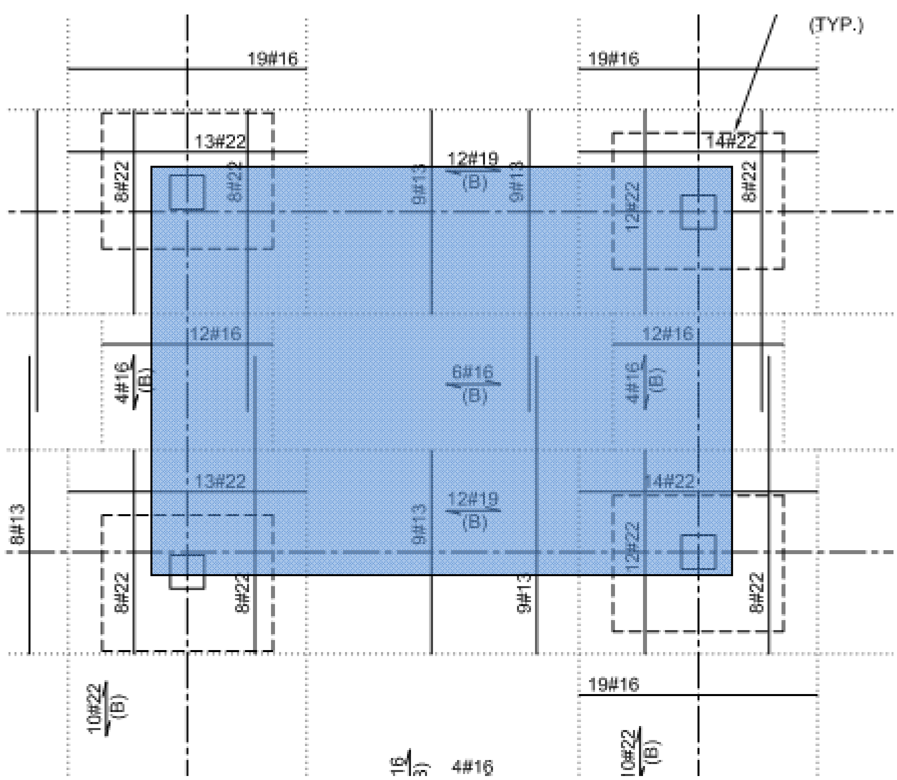


Figure 37: Column Spot Check

Two Way Flat Slab Spot Check

A two way slab spot check was done on the second floor framing system to compare to the design two way slab system, between column grid lines C and D. A copy of the calculations is provided in Appendix D. A main source of error with the two way slab spot check is the lack of knowledge on the subject. I prepared the spot check to the best of my ability and was able to breakdown the Column strip moments and the Middle Strip Moments to the perspective parts. The error arrived during the actual design of the slab, where I obtain areas of steel that I could not relate to the designed slab. Another source of area is from the complication of the drawings to understand what was designed for the two way slab system. Since Technical Assignment #2 is a consideration of floor systems, a deeper study of the two way slab system is planned for that report.



Conclusion

After performing the structural analysis as prescribed by ASCE 7-05 Structural Design Criteria, the loads that the FDA Office Building was designed to and the loads that I determined after my analysis were compared. The live loads that were prescribed by the GSA and ASCE were primarily similar to the current standards. The only exception was the green roof live load, but comparison to the green roof dead load that the structural engineering implanted in the design may have been a source of the difference.

The lateral systems were analyzed using the ASCE 7-05; Method 2 for Wind Design and Equivalent Lateral Force Procedure. For simplification and ease of analysis only Wing B was considered for the lateral analysis. The simplification was only valid because expansion joints were provided between each wing of the office building. The same base wind speed was used along with other basic design variables which allowed the assumption to be close to the design values. Seismic Base Shear was only provided for Wing A, and the lateral analysis was done for Wing B. Therefore, no comparison of lateral forces could be made. However, the seismic design category of the office building was higher then I obtained from my analysis. The difference in the SDC was due to the lower ground acceleration values obtained from the USGS.

Evaluation of the floor system spot checks revealed various oversimplifications and errors in my design considerations. A typical column was chosen to be analyzed, and my spot check showed the overdesign of the actual column. This is primarily due to the simplification of no moments in the interior columns. A two way flat slab floor system on the second floor was chosen to be checked. The design of a two way flat slab proved to have errors throughout my calculations and further research is required for a better two way slab analysis. Technical assignment 2 is primarily on floor systems, and it is planned to extend the research of the two way slab systems in that report.

Appendix A: Wind Analysis

Method 2 : Wind Analysis of Wing B By ASCE 7-05

1. Determine basic wind speed V From Fig 6-1 ASCE 7-05

    From Fig 6-1 ⇒ $V = 40$ M/s (90 mph)

2. Determine wind directionality factor K_d fro Table 6-4
    By Table 6-4
    Buildings
    MWFRS $K_d = .85$
    C-C $K_d = .85$

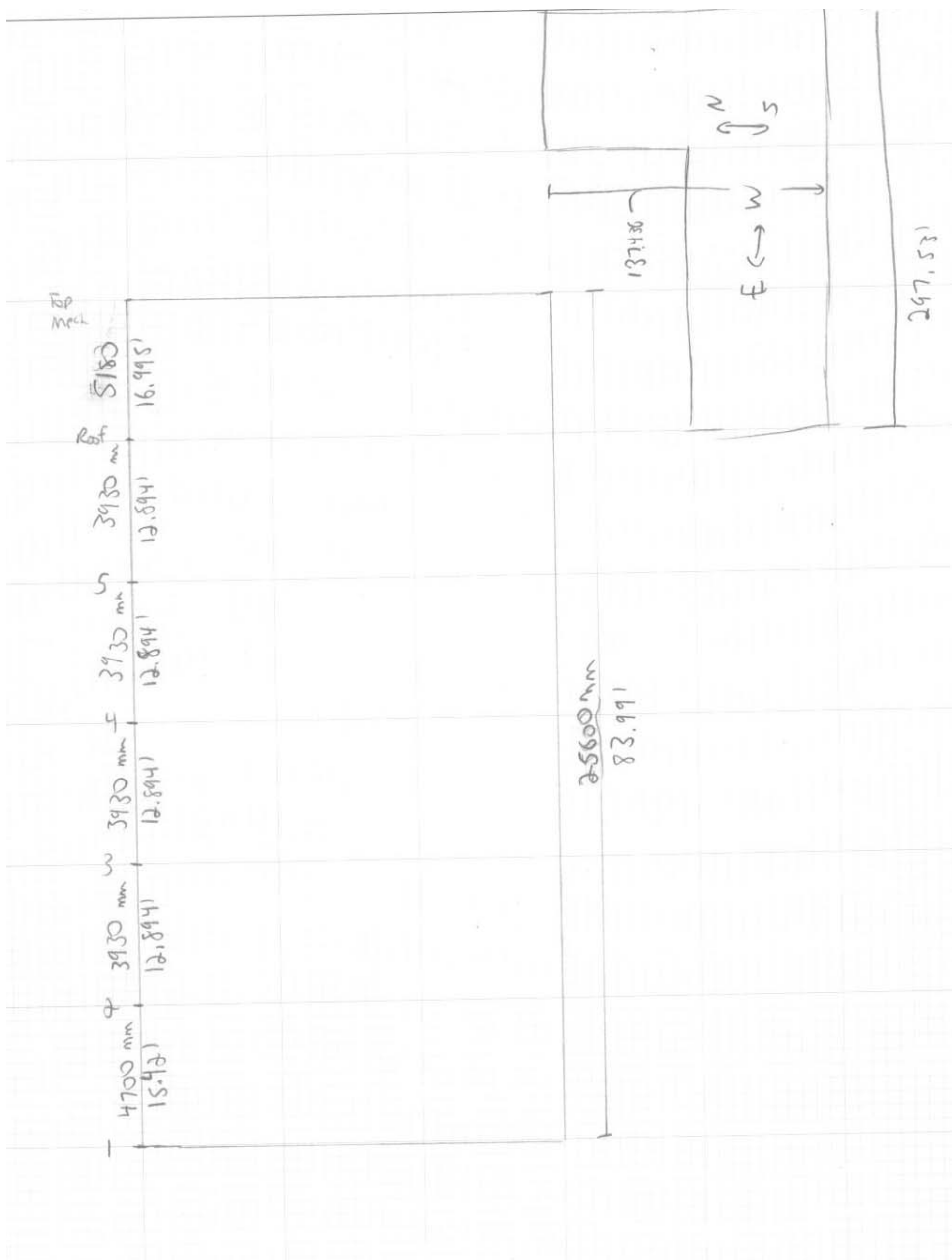
3. Determine importance Factor I from Table 6-1
    By Table 1-1 : Occupancy category of Buildings
    Occupancy category II
    ∴
    By Table 6-1 $I = 1.00$

4. Determine exposure category (6.5.6)
    By ASCE 7-5 6.5.6.2 Surface Roughness Categories
    Surface Roughness B
    By ASCE 7-5 6.5.6.3 Exposure Categories
    Exposure B applies

5. Topographic Factor : ASCE 7-5 6.5.7
    Not all 5 conditions are met $K_{zt} = 1.0$

6. Determine velocity pressure exposure coefficents K_z and K_h From Table 6-3
    From Table 6-2 w/ Exposure B $\alpha = 7$, $z_g = 1200$ ft (365.76 m)

    From Table 6-3:
    $$K_z = 2.01\left(\frac{15.4199}{1200}\right)^{2/7} = .5793$$
    $$q_z = .00256(.5793)(.85)(1)(90)^2(1) = 10.21$$
    see excel for complete table


Top Mech
5180
16.995'
Roof
3930 mm
12.894'
5
3930 mm
12.894'
4
3930 mm
12.894'
3
3930 mm
12.894'
2
4700 mm
15.42'
1
25600 mm
83.99'
13748
267.53'
N
S
E
W

		Elevation	Floor height to floor (mm)	Floor height to floor (ft)	Total Floor height
Datum -	Level 01 -	112,300	0	0	0
	Level 02 -	117,000	4700	15.42'	15.42
	Level 03 -	120,930	3930	12.894	28.314
	Level 04 -	124860	3930	12.894	41.207
	Level 05	128,790	3930	12.894	54.101
	Roof	132720	3930	12.894	66.995
	Parapet	133679	959	3.146	70.141
	Top Mech	137,900	4221	13.848	83.989

Method 2: Gust Effect factors, G and G_f

$B = 137.436'$ = 41892.6264
$L = 297.55'$ = 90693965?
$h = 83.9895'$

By C6-17 $n_1 = 100/H = 100/83.9895 = 1.1971$ rigid structure

* Structure is a 5 story reinforced concrete system rigidity is assumed and verified

By 6.5.8.1

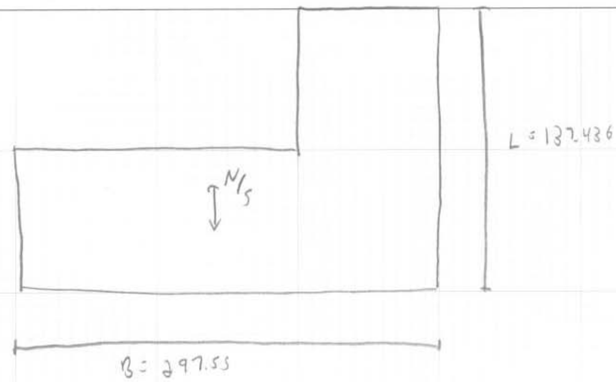
$G = .85$ is more conservative if $H/L < 4$

$\frac{83.9895}{137.486} = .61 < 4$

$G = .85$

Find n_1

For N/S



$H = 83.9855$

$n = 4$

$A_B = 27469.11\ ft^2$

$A_2 = A_1 = 300(6000)/25.4^2/12^2 = 19.375\ ft^2$ $\quad h = 83.9855'$

$A_3 = A_4 = 300(3000)/25.4^2/12^2 = 9.6875\ ft^2$

$D_1 = D_2 = 19.685'$

$D_3 = D_4 = 9.843'$ $\quad$ By excel $\underline{n_1 = .56}$

For

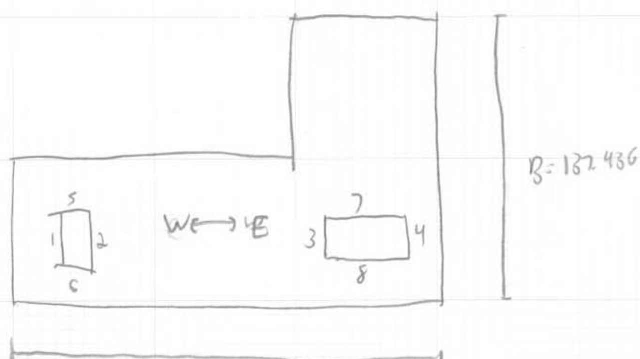
E/W

$H = 83.9855$

$n = 4$

$A_B = 27469.11\ ft^2$

$A_2 = A_1$



$A_5 = A_6 = 300(3700)/25.4^2/12^2 = 11.948\ ft^2$ $\quad L = 137.436$

$A_7 = A_8 = 300(8252)/25.4^2/12^2 = 26.647\ ft^2$ $\quad h = 83.9855'$

$D_5 = D_6 = 145.67/12 = 12.139'$ $\quad$ By excel $n_2 = .706$

$D_7 = D_8 = 324.8/12 = 27.07'$

Structure is flexible * $n_1 = .561$ (N/S) (direction dependent) $n_1 = ?$ (E/W)

1. $g_Q = g_V = 3.4$

2. $g_R = \sqrt{2\ln(3{,}600 n_1)} + \frac{.577}{\sqrt{2\ln(3{,}600 n_1)}} = 4.049$

3. $\bar{z} = .6h = .6(83.9855) = 50.3913 \geq z_{min} = 30'$ ok

4. $I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = .3\left(\frac{33}{50.3913}\right)^{1/6} = .2795$

5. $L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{50.3913}{33}\right)^{1/3} = 368.49$

*6. $Q = \sqrt{\frac{1}{1+.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{.63}}} = \sqrt{\frac{1}{1+.63\left(\frac{297.53+83.9855}{368.49}\right)}} = .778$

N/S $B = 297.53$ E/S $B = 137.436$
$h = 83.9855$
$L_{\bar{z}} = 368.49$

7. Determine basic wind speed V from Fig. 6-1 $V = 90$ mph

8. $\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = .45\left(\frac{50.3913}{33}\right)^{1/4}(90)\left(\frac{88}{60}\right) = 66.03$

$\bar{b} = .45$, $\bar{\alpha} = 1/4$

*9. $N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{.561(368.49)}{66.03} = 3.131$

10. $R_n = \frac{7.47 N_1}{(1+10.3N_1)^{5/3}} = \frac{7.47(3.131)}{(1+10.3(3.131))^{5/3}} = .068$

11. $R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta})$ for $\eta > 0$ $R_h = 1$ for $\eta = 0$

$\eta = 4.6 n_1 h/\bar{V}_z = 4.6(.561)\left(\frac{83.9855}{66.03}\right) = 3.276$

$R_h = \frac{1}{3.276} - \frac{1}{2(3.276)^2}(1-e^{-2(3.276)}) = \boxed{.2587}$

12. $R_B = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta})$ f

$\eta = 4.6 n_1 B / \bar{V}_2 = 4.6(.561)(297.53)/66.03 = 11.628$

$R_B = \frac{1}{11.628} - \frac{1}{2(11.628)}(1-e^{-2(11.628)}) = .0823$

13. $R_L = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta})$

$\eta = 15.4(n_1 L/\bar{V}_2) = 15.4(.561)\left(\frac{137.436}{66.03}\right) = 17.982$

$R_L = \frac{1}{17.982} - \frac{1}{2(17.892)^2}(1-e^{-2(17.982)}) = .054$

14. $R = \sqrt{\frac{1}{\beta} R_n R_h R_b (.53 + .47 R_L)}$ $= \frac{1}{\text{.015}}$

β ran. 1%, to 2%, by C6.5.8

$= \sqrt{\frac{1}{.015}\, \overset{R_n}{.068}\,(\overset{R_h}{.2587})(\overset{R_B}{.0823})(.53 + .47(.054))} = .232$

$=$

15. $G_f = .925\left(\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right)$

$G_f = .925\left(\frac{1 + 1.7(.2796)\sqrt{3.4^2(.778)^2 + 4.049^2(.232)^2}}{1 + 1.7(3.4)(.2796)}\right) = .8253$

Determine pressure coefficient C_p for the walls and roof from figure 6-6

For wind in the E-W

Wind Ward Wall: Cp= .8 For use with q_z

Leeward Wall (L/B) = $\frac{137.436}{297.53}$ = .46 Cp= -.5 For use w/ q_h

Side wall: Cp= -.7 For use with q_h

For wind in the N-S direction

Wind ward wall: Cp= .8 For use w/ q_z

Leeward wall (L/B) = $\frac{297.53}{137.436}$ = 2.16 Cp= -.2 For use w/ q_h

Roof. Cp= -.7 w/ q_h

For Roof ~~E-W~~ N-S

*Norm to ridge < 10° Cp are Parallel to ridge for all θ

$h/L = \frac{70.1411}{297.53} = .2357 < .25$

Cp= -.9, -.18 From windward edge to h= 70.14'

Cp= -.5, -.18 From ~~35.07~~ 70.14 ft to 2h= 140.28'

Cp= -.3, -.18 from 140.28 → 137.44 (not used)

For Roof E-W

$h/L = \frac{70.1411}{137.436} = .51 \approx .5$

Cp= -.9, -.18 from windward edge to 70.141

Cp= -.5, -.18 From 70.141 to 140.282

Cp= -.3, -.18 From 140.282 to 297.53

8. Determine design wind pressures p_z at p_h Enclos. Fig. 6-5 $GCp_i = \pm .18$

N/S $p_z = q_z G C_p - q_h (GCp_i)$

$p_z = 10.210(.818)(.8) - 15.740(\pm .18)$

6.68124 ∓ 2.8332 3.848
9.5146

Appendix B: Seismic Analysis

Seismic 1

Design Data

Location: Silver Spring, MD
Soil Classification: Site Class C
Occupancy: office occupancy where less than 300 people congregate

Material: Reinforced concrete
Structural System: shear walls,

Seismic Ground Motion Values

1. Determine the mapped accelerations S_s and S_1

From the USGS Ground Motion Parameter Calculator

$S_s = .155$

S_1 .05

2. Site Class C

3. Determine soil-modified accelerations S_{ms} and S_{m1}

By table 11.4-1 — $F_a = 1.2$

By table 11.4-2 — $F_v = 1.7$

$S_{ms} = 1.2(.155) = .186$

$S_{m1} = 1.7(.05) = .085$

4. Determine design accelerations

$S_{DS} = 2/3\,(.186) = .124$

$S_{D1} = 2/3\,(.085) = .057$

Seismic 2

Step 2: Determine the SDC

- $S_S = .124 \leq .15$ $S_1 = .057 > .04$ ∴ cannot be automatically assigned to SDC A.

- Determ the Occupancy Category : II

  is $S_1 \geq .75$ no

- check if condition in 11.6 are met

  1. Check if the approximate period T_a is less than $.8T_s$

$T_a = C_t h_n^x$ $C_t = .02$ $x = .75$

$h_n = 70.1411$

$T_a = .02(70.1411)^{.75} = .485$

$T_s = S_{D1}/S_{DS} = \frac{.057}{.124} = .4597$

$.8T_s = .368$ is less than T_a condition are not met

- Determ the SDC from Tables 11.6-1 and 11.6-2

  From Table 11.6-1, with $S_{DS} = .124$, Occupancy II

  SDC is A

  From Table 11.6-2 with $S_{D1} = .057$

  SDC is A

* Different from Building design criteria

Seismic 3

Use Equilvalent Lateral Force Procedure by Table 12.6-1

Seismic Base Shear

$V = C_s W$ Eq (12.8-1)

$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)} = \frac{.124}{5} = .0248$

$R = 5$ by 12.2-1
$I = 1.0$

Value of C_s shall not exceed

$C_{s_{N-S}} = \frac{S_{D1}}{T(R/I)}$ f. $T \leq T_L$ $T_L = 8$ by Figure 22-15

$.993 \leq 8$

$= \frac{.057}{.993(5/1)} = .0115$

$.011$

$C_{s_{N-S}} = .0115$

$V_{N-S} = .0115(23500) = 270.3$ k

$T_a = \frac{.0019}{\sqrt{C_w}} h_n$

C_w N-S $= .018$ calculated in Approximate
C_w E-W $= .042$ Fundamental Freq. for Wind

$T_{a_{N-S}} = \frac{.0019}{\sqrt{.018}} 70.14 = .993$ Eq. 12.8-9

$T_{a_{EW}} = \frac{.0019}{\sqrt{.042}} 70.14 = .65$

Seismic 4

Vertical Distribution of Seismic Forces

$F_x = C_{vx} V$ 12.8-11

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

Since .5 sec < T < 2.5 k is determined by

N-S
$k = .75 + .5T = .75 + .5(.993) = 1.25$

E-W
$k = .75 + .5(.65) = 1.075$

Appendix C: Column Spot Check

column spot check 1

Check design for gravity load in column D.3 on 2nd floor supporting roof and 3 thru 5 floor loads

23.62 x 23.62 col.

- Live load reduction is not considered for this check

Tributary area = $\left[\frac{(27.559 + 19.685)}{2}\right]$ 29.528 = 697.5 sf

Loads acting on column

Roof: RL = 20 psf; RD = 40 psf; DL_s = 118.1 psf

3x Floors: LL = 80 psf; SDL = 15 psf; DL_{slc} = $(9.448/12)(150)$ = 118.1 psf

Axial load acting on column

$w_{u_{roof}} = 1.2(118.1+40)+1.6(20) = 221.7$ psf

L4 D.N.C by inspection

$w_{u_{Floor}} = 1.2(15+118.1) + 1.6(80) = 287.72$ psf

$P_u = (221.7(697.5) + 3(287.72)(697.5))/1000 = 756.7^k$

$P_n = P_u/\phi\alpha = \frac{756.7}{.65(.8)} = 1455.2^k$

$P_o = .85 f'_c A_c + A_s f_y$

$P_o = [.85(4000)[(23.62)(23.62) - 12] + 12(69{,}000)]/1000 = 2576.1^k$

$\alpha\phi P_n = .65(.8)(2576.1) = \underline{1339.6^k}$

* This is much higher than required load, partially due to the fact of assumed loadings could vary from design and no moments were taken consideration

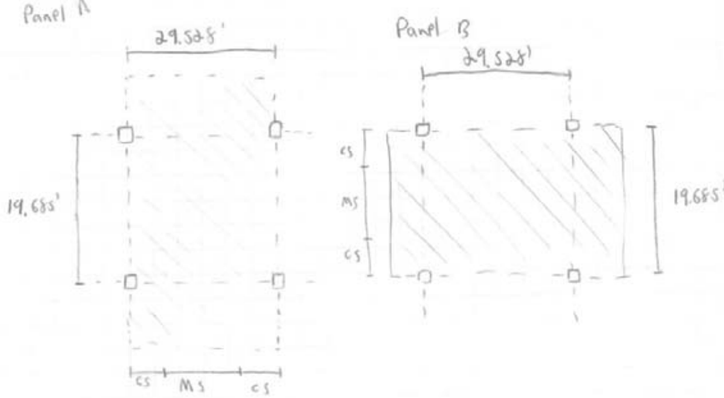
Appendix D: Two Way Slap Spot Check

Slab spot check 1

Spot check

Panel A

Panel B



l_2: 19.685'
l_n: 29.528 - 23.62/12 : 27.56'

CS: .25 l_1 or .25 l_2

.25 l_1 : 29.528(.25) : 7.382'
.25 l_2 : .25(19.685) : 4.92'

Ms: 29.528 - 2(4.92) : 19.69'

l_2: 29.529
l_n: 19.685 - 23.62/12 : 17.72'

CS: 4.92'

MS: 19.685 - 2(4.92) : 9.85'

Slab spot check 2

Spot check 1.

- 9.449" two way flat plate slab

- As designed
  - 9.449" slab
  - f'_c : 4000 psi
    - Top layer: #5 @ 15.75" o.c.
    - Bottom layer #4 @ 11.81" o.c.
  - two way slab, no int beams

DL: 118.1 psf
SDL: 15 psf
LL: 80 psf

- Direct design method - ACI 318-08 chapter 13

  - Factored load:
    $1.2(118.1+15) + 1.6(80) = 287.74$ psf $= .2877$ ksf

  - Middle strip = MS Column strip = CS

Panel A 23.62" x 23.62" collums D2, D3, C2, C3

$M_o = \frac{1}{8} w \ell_2 \ell_n^2 = \frac{1}{8}(.2877)(19.685)(29.528 - \frac{23.62}{12})^2 = 537.7$ ft-k

$\alpha_f = 0$ (since flat plate

Min thickness of slab w/o interior beams [Table 9.5.3]
For deflection control w/ drop pannels

$$\ell_n/36 = \frac{(29.528 - \frac{23.62}{12})(12)}{36} = 9.19" < 9.449" \text{ } \underline{ok}$$

* do not need check deflection

Distribute moments, m^- and m^+

For interior frames
$m^- = .65 M_o = .65(537.7) = 349.5$ ft-k
$m^+ = .35 M_o = .35(537.7) = 188.2$ ft-k

Distrib. of MS and CS M^- by ACI 318-08 sec. 13.6.4.1
$\ell_2/\ell_1 = \frac{19.585}{29.528} = .67$

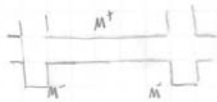
Since no int. Beam $\alpha_f = 0$
75% of M^+ to CS-A $= 262.1$ 'k
25% of M^- to MS-A $= 87.4$ 'k

Slab spot check 3

Distribute of MS and CS M^+ by ACI 318-08 Sec 13.6.4.4

Since no int beams $\alpha = 0$

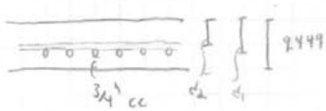
60% of M^+ to CS-A $= 112.9'^k$
40% of M^+ to MS-A $= 75.3'^k$



	M^-	M^+	M^-
Total	349.5	188.2	349.5
CS	262.1^k	112.9	262.1
MS	87.4	75.3	87.4

Design of Slab reinforcing

max spacing: $2t = 2(9.449) = 18.9''$
min steel = temperature and shrinkage reinf
$A_{s_{min}} = .0018\,bt$



$d_1 = t - cc - \frac{1}{2}\phi$
$d_2 = d_1 - \phi$

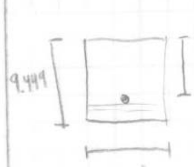
M_n	-349.5	188.2	-349.2
C_s width	118.1"	118.1	118.1"
M_u	$-285.2/.9 = -316.9$	160.3	-316.9
$M_u(12)/b$	$\frac{-316.9(12)}{59.04}$		
	35.5^{1k}	19.1^{1k}	35.5^{1k}

slab spot check 4

Design of CS reinf. of Panel A cont.

$M^-_{L,R} = 35.5^{k}$ $\quad M^+_{m} = 19.18^{\prime k}$

$A_s = \frac{M_u}{4d} = \frac{35.5}{4(7.64)} = 1.15\ in^2$



$d = 9.449 - \overset{cc}{1.5} - \frac{1}{2}(.625) = 7.64$

$A_s = 1.15\ in^2$ Try 4 #5 for reinforcing

$A_{s_{min}} = \frac{3\sqrt{4000}}{60{,}000}(12)(7.64) = .289$

* This amount of steel is much higher than as designed is is partialy due to the lack of knowledge on two way slab design. an error on the student.

$M^+_m = 19.1$

$A_s = \frac{19.1}{4(7.64)} = .625\ in^2$ Try 2 #5 for reinforcing

slab spot check 5

Design of MS reinf of Panel A

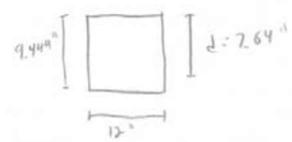
M_n^-	-87.4	75.3	-87.4
MS width	236.3	236.3	236.3
$M_n = M_u/.9$	-97.1^{1k}	83.7	-97.1^{1k}

$M_n(12)/b = \frac{-97.1(12)}{236.3}$

-4.93^{1k}	4.25^{1k}	-4.93^{1k}

$M^- = 4.93$

$A_s = \frac{M_n}{4d} = \frac{4.93}{4(7.64)} = .161\ in^2$



$M^+ = 4.25^{1k}$

$A_s = \frac{M_n}{4d} = \frac{4.25}{4(7.64)} = .139\ in^2$

slab spot check 6

Panel B 23.62" x 23.62" collums

$M_o = \frac{1}{8} w_u \ell_2 \ell_n^2 = \frac{1}{8}(.2877)(29.528)(19.685 - 23.62/12)^2 = 327.4$ 'k

$\alpha = 0$ no int beams

Min thickness of slab w/o interior beams [table 9.5.3]
For deflection control w/ drop panels.

$\ell_n / 36 = \frac{(19.685 - \frac{23.62}{12})12}{36} = 5.91$ in < 9.449" ok

* do not need to check deflection

Distribute moment m^- and m^+

For interior frames

$m^- = .65 m_o = .65(327.4) = 212.8$ 'k

$m^+ = .35(M_o) = .35(327.4) = 114.6$ 'k

Dist. of MS and CS M^- by ACI 318-08 sec 13.6.4.1

Since no int beams $\alpha = 0$

75% of M^- to CS-B = 159.6 'k

25% of M^- to MS-B = 54.7 'k

Distribute of MS and CS M^+ by ACI 318-8 sec 13.6.4.4

Since no int beam $\alpha = 0$

60% of M^+ to CS-B = 68.8 'k

40% of M^- to MS-B = 45.8 'k

M_n	-159.6"	68.8	-159.6 'k
CS width	118.08	118.08	118.08
M_n =	-159.6/.9 = 177.3 'k	76.4 'k	-177.3 'k

$\frac{M_u(12)}{b} = \frac{-177.3(12)}{118.08}$

-11.9 'k   7.76 'k   -11.9 'k

Slab spot check 7

Design of CS reinf of Panel B cont.

$d = 9.449 - 1.5 - \frac{1}{2}(.625) = 7.64$

9.449" × 12"

$M_u^- = 11.9^{1k}$ $M_u^+ = 7.76^{1k}$

$M^- = 11.9^{1k}$

$A_s = \frac{M_u}{4d} = \frac{11.9}{4(7.64)} = .39 \text{ in}^2$

Try #5 @ 9 o.c. $A_{s\,prov} = .41 \text{ in}^2$ per foot

$M^+ = 7.76^{1k}$

$A_s = \frac{M_u}{4d} = \frac{7.75}{4(7.64)} = .25 \text{ in}^2$

Try #5 @ 14" O.C. $A_{s\,pr} = .27 \text{ in}^2$ per foot

Design of MS reinf of Panel B

M_n	-54.7	45.8	-54.7
MS width	118.2	118.2	118.2
$M_u = M_n/.9$	-60.8^{1k}	50.9^{1k}	-60.8^{1k}
$M_u(12)/b$	-6.17^{1k}	5.17^{1k}	-6.17^{1k}

$M^- = 6.17^{1k}$ $A_s = \frac{M_u}{4d} = \frac{6.17}{4(7.64)} = .202$

Try #5 @ 16 o.c. $A_{sp} = .23 \text{ in}^2$ per foot

$M^+ = 5.17^{1k}$ $A_s = \frac{M_u}{4d} = \frac{5.17}{4(7.64)} = .169 \text{ in}^2$

Try #5 @ 20" o.c. $A_{sp} = .186 \text{ in}^2$ per foot