TECHNICAL REPORT 1

Dominick Lovallo | Dr. Hanagan Advisor | The Pennsylvania State University

Embassy Suites Hotel, Springfield Virginia

Table of Contents

Executive Summery

The purpose of Technical Report 1 is to analyze the existing conditions and structural components of The Embassy Suites Hotel in order to develop an understanding of the structural system in building. This technical report will contain a general overview of the primary structural components which include foundation, framing, and floor systems, load calculations, and summaries of lateral loadings due to wind and seismic. Information gathered from wind, seismic dead, live, and snow analyses will let one comprehend the building's structural system and how the loads are distributed to these components. In the Appendices, detailed calculations and floor plans are provided to reinforce the summaries of the structural concepts.

The Embassy Suites Hotels is a 7 story all-suite hotel located in Springfield Virginia. Situated a few miles away from downtown Washington DC, Embassy suites contains 219 guestrooms and a host of amenities like a pool and bar areas. The building will also contain many retail stores located on the lower lever. The building stands at 91feet 10 inches and is approximately 185000 square feet. The building floor system contains an 8 inch cast in place reinforced slab connect to mostly 14x30 reinforced concrete columns. The columns run between the floors at a story height of approximately nine feet. The typical story height is 9 feet except for the ground storefront level roof level, having heights of 18 feet and 10 feet respectively. The foundation system contains a mutt mat system due to soil differentials around the site. Aside from the mud matts is some areas the Embassy Suites foundation is a typical strip footing and slab on grade system. The lateral and gravity load system are integrated and consisting of reinforced concrete moment frames.

In analyzing the structural system, ASCE 7-05 reference code was used when evaluating the lateral and gravity load conditions. Lateral loads due to wind and seismic were found to have similar effects on the building. The columns were significantly overdesigned to account for these combined conditions. This can be due in part to different assumptions made in initial design process and different design load parameters. To determine the controlling lateral load condition additional analysis is needed. It is important to note that inconstancies in values and results presented in spot checks and wind and seismic analysis are due to different design loads used in calculations. All existing structural members are sound and properly designed.

Introduction: Embassy Suites Hotel

Figure 1.2: **Site Map. (Photo taken from Google Earth)**

The Embassy Suites Hotels is the newest, 7 story, luxury, hotel to become part of the Miller Global, LLC family. Along with Miller Global, the owner the collaborative construction team on this venture include, Cooper Carry, architect; SK & A Structural Engineers, PLLC , structural designers; Balfour Beatty Construction, construction manager; Jordan and Skala, MEP firm; Christopher Consultants, LTD, civil engineering firm. The site is located at the junction of I-95 and Fairfax County Parkway. The location lies in the Springfield region of Fairfax County, Virginia. The site is approximately 16 miles away from the heart of downtown Washington, D.C... Patrons will also be in close proximity to both the Fort Belvoir Main Army Post and the National Geospatial-Intelligence Agency (NGA) facility. The construction delivery method was design –bid - build. Construction began in November 2011 and will be completed July 13th 2013.

Figure 2.2: **Facade. (Photo taken from Miller Global, LLC website)**

Upon its completion, this 31.5 million, 185,000 square foot, hotel will feature many amenities. These include a large open air atrium and spacious two room suites. The hotel will serve as a model for comfort and convenience. The building's architecture boasts long flowing curved lines that give it immense visual appeal and a unique flow. The hotel's ground floor will contain a 1300 square foot pool area, a fitness center along with multiple meeting areas, a bar, a lounge and over 1400 square feet of retail space.

The ground level and upper floors store front materials will be made up of manufactured masonry (adhered concrete stone veneer). It is comprised of boral cultured stone country ledge stone along with architectural adhered precast concrete panels. It also contains 1" insulated glass windows with aluminum frames and automatic entrances. The upper levels the exterior façade will feature an exterior insulation finish system (EIFS).

This report will be describing the various structural elements and systems in place at the Embassy Suite Hotel project. To learn how the multiple structural systems work as a part of a sound, cohesive building, one must delve into explanations of the foundation design, floor, lateral, and gravity load resisting systems design.

Structural Systems

Existing Foundation

Prior to construction, subsurface exploration and geotechnical engineering analysis were conducted on the future Embassy Suites Hotel site and was completed in January11, 2011 by ECS Mid- Atlantic, LLC. The report indicates a number of test borings were performed on 3 separate occasions. The test borings were drilled at depths ranging from 2.5 'to 79' to determine the soil composition in different areas of the site. ECS Mid- Atlantics results showed fill soil material was found in ten boring locations around the site. The fill material was composed of silty sand and clay from depths of 6.5' to 8.5' below the ground surface. Further down the

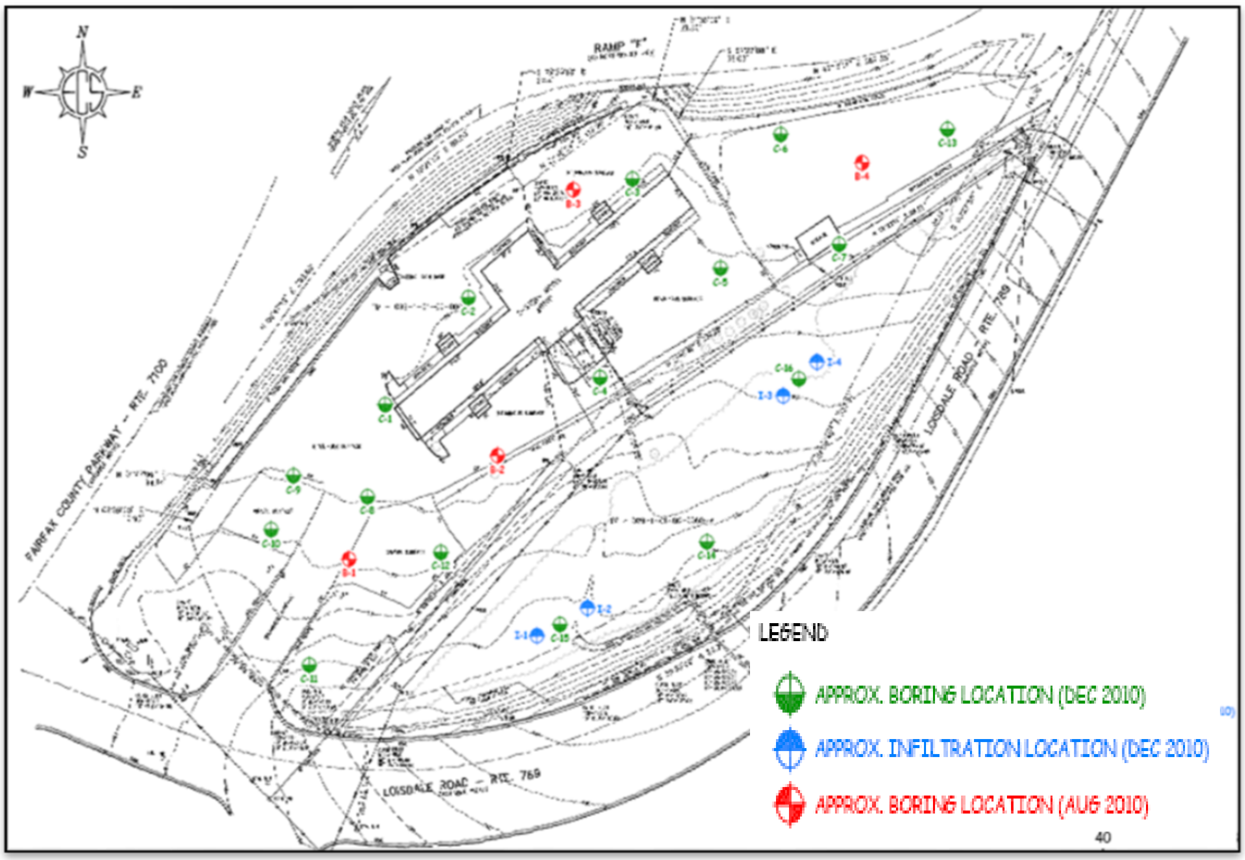
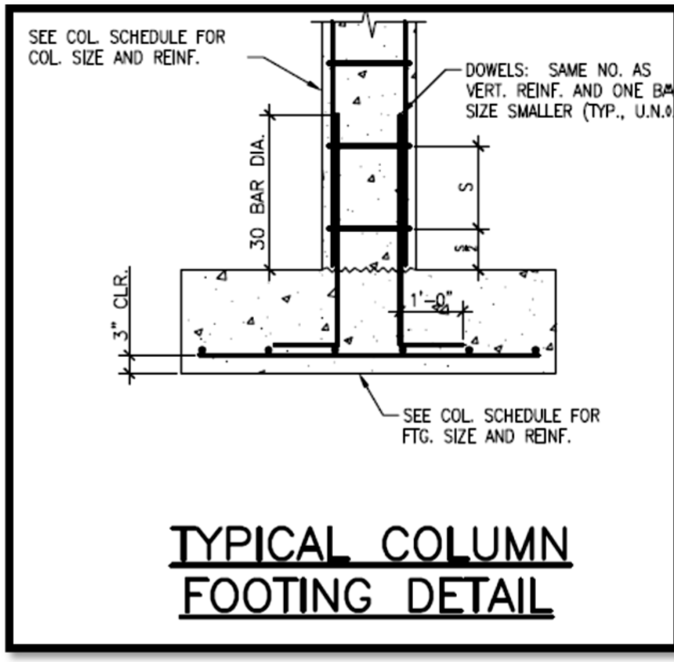


Figure 3.3: **Core Boring Locations**

borings indicated the existence of natural soils that were mainly composed of clayey sand, silt and fat clay. A weather rock material was found at 77' to 78.6'and ground water was encounter at of 18.5' to 65'.

Due to the variability in soil composition, the project team had to employ a partial mud matt system to equalize the soil capacity around the site in some areas. A mud matt system is a thin layer of lean concrete mix (in this case 2000 psi) placed over the existing soil below and allows a stable base for construction. The spread footings were designed to have an allowable bearing capacity 6000 psi. The size of footings range from 3’ by 3’ to 12’ by 8’ and extend 2’ below the slab on grade. To tie the footings together, longitudinally placed strap beams ranging from 36 width x 24 depths to 42 width x 24 depth beams were used. A strap beam is a structural element used to connect to isolated footings together. These beams help distributed the building load to the footings and eventually the ground. The beams range in size and have varied vertical and horizontal reinforcing.



The typical slab on grade is a minimum of 5 inches in depth and sits on 4 inches of washed crushed stone. The capacity of the slab is 3500 psi for the interior portions and 5000 psi for exterior slab conditions. The slab contains 6x6 – W 2.0 x W2.0 welded wire fabric and has number 4 reinforcing steel bars spaced 12 inches on center each way.

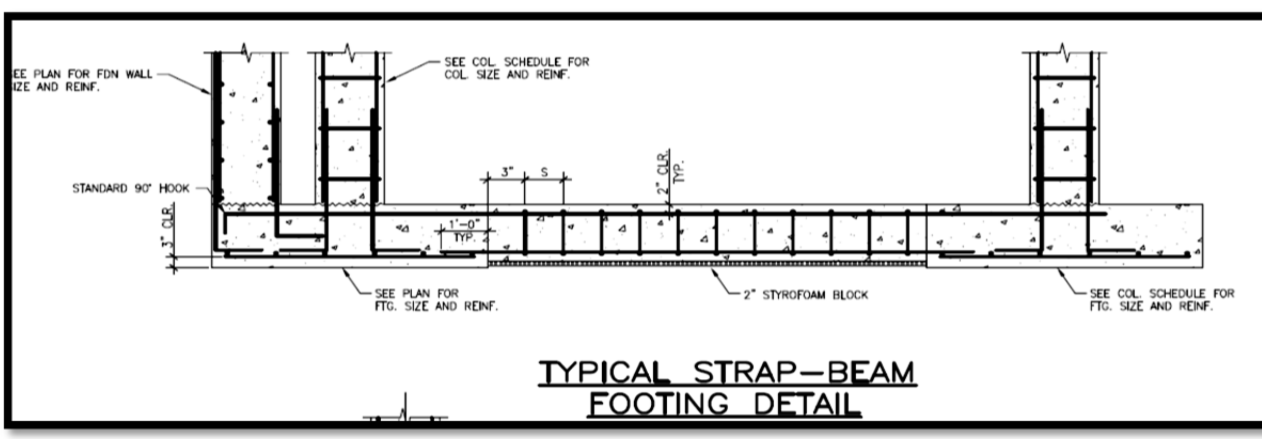
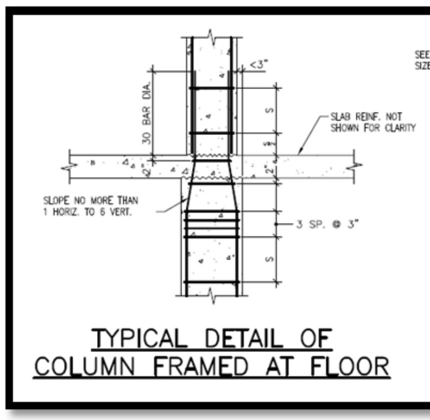


***Figure 4.4*: Strap Beam Detail**

Floor System

The Embassy Suites Hotel is made up of a typical flat slab construction. The two way slab thickness is 8 inch and the compressive strength of the normal weight concrete is 5000 psi. The slab reinforcing includes number 4 reinforcing bars spaced at 10 inches on center, either way and run the full length from column to column. The floor system also uses a thickened slab or drop panel system around the columns to protect against punching shear. Punching shear is a failure mechanism were the slab separates from the column due to concentrated shear force. Drop panels are 3.5 inches thick (total slab thickness around column on typical floor is 11.5 inches) and extend 5 feet from either side of the columns.

Framing System



In the image below, shown is a typical framing plan for floors of the Embassy Suites Hotel (Floors 3 to 7). A typical bay size is 23' by 18' for floors containing the guest suites. The columns chosen in for the framing plan were almost all 14 x 30 inch rectangular reinforced concrete columns. The majority of the columns have a minimum compressive strength of 6,000psi. There are no beams running in between the interior and exterior columns. The only reinforced beams found are located in stairwell openings and elevator shafts. Due to the increased load on the second floor, large concrete transfer girders had to be used to accommodate for the fitness and pool area. Level 2 also contains HSS columns along with a variety of wide flange shape beams. These are located in the section of the hotel where future retail stores will be housed.

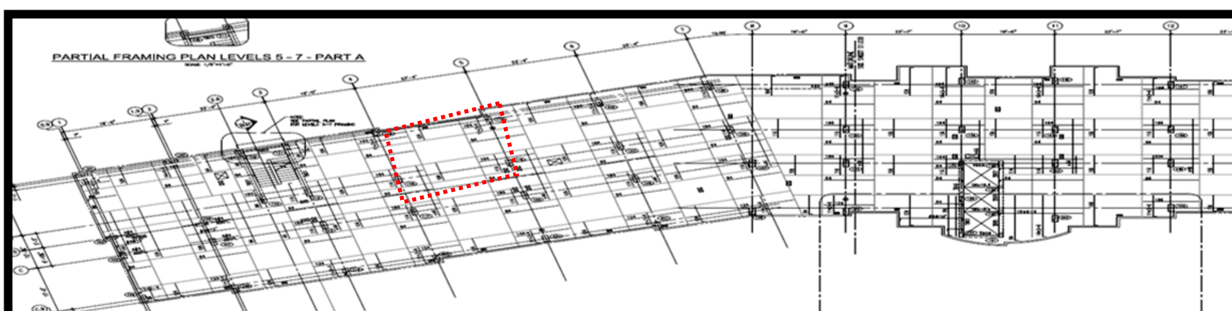
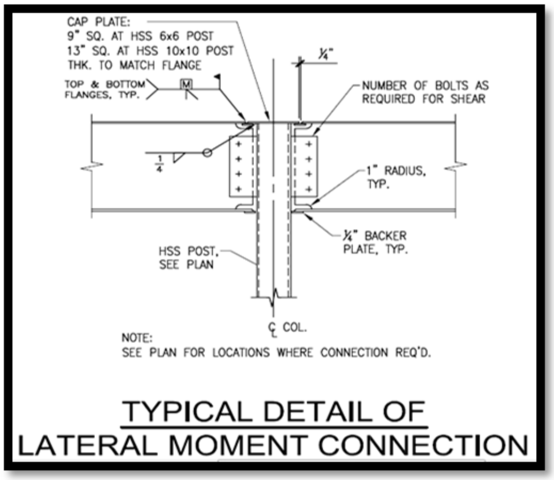


Figure 5: Typical Framing Plan Levels 3-7

Lateral System



To resist lateral forces due to wind and seismic loads the structural engineers employed reinforced concrete moment frames moment frames. A concrete moment frame load resisting system (in this case a slab and columns cast monolithically) opposes overturning moment caused by lateral loads. The concrete moment frames are the main lateral force resisting system in the building. The lower storefront levels have welded steel moment connections. The moment connections were designed to develop the full capacity of the member. The connections use high strength ¾ or 7/8 inch ASTM A325 or A490 threaded bolts. The bolts connect the ¼ x 1 inch plates to the beams were the plates are butt and penetrate welded.

Figure 9: Welded Moment Connection

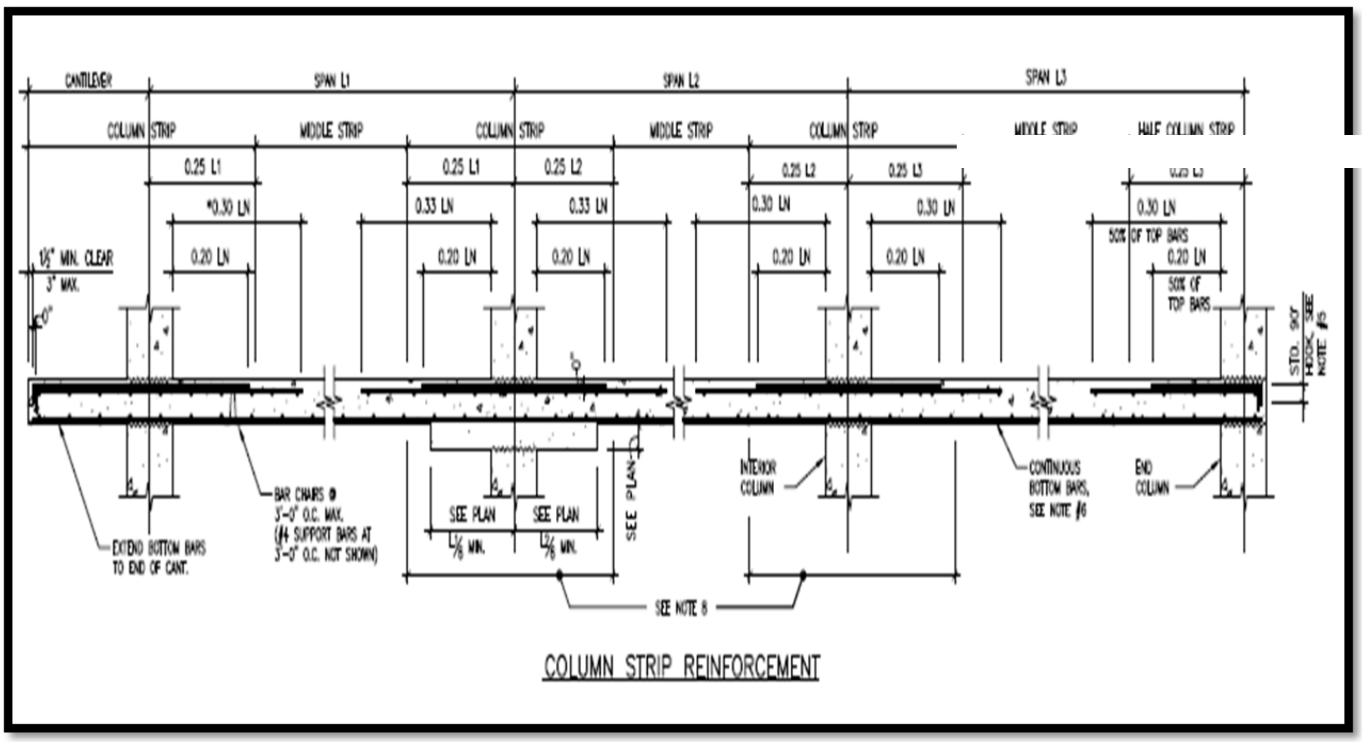


Figure 6: Main Lateral Force Resisting System

Roofing System

The high level roofing system consist of 2 inch deep 20 gauge Type N cold formed metal deck. The metal deck is topped by 3.25 inch light weight concrete slab. This slab has a compressive strength of 3,500 psi. The deck holds a minimum of a 3 span condition. The lower level roof (top of retail space) is made of 1.5 inch deep 20 gauge Type B cold formed metal deck. The roof deck systems are supported by wide flange beams, concrete reinforced beams varying in size and open web steel joists. The lower level roof system is comprised of a thermoplastic membrane fully adhered with heat welded seams and vapor retarder over a metal deck. Part of the lower level roof (top of part of the second floor) contains a green roof system that includes a pre-vegetated 50 percent extensive and a 50 percent intensive system that is placed upon a protective mat.

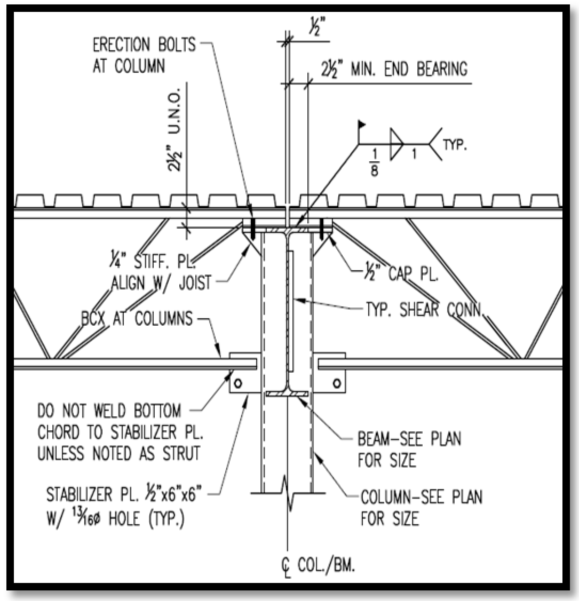


Figure 7: Lower Roof System Connection

Codes and Requirements

- 2009 Virginia Construction Code (IBC 2009) with the Virginia Statewide Building Code
- 2009 Virginia Mechanical Code (IMC 2009)
- 2008 International Electric Code
- 2009 International Pluming Code (IPC 2009)
- 2009 Virginia Fire Prevention Code (IFC 2009) with the Statewide Fire Prevention Code
- American Society of Civil Engineers (ASCE 7- 05)
- Publication #4 "Standard Recommended Practice for Concrete Formwork" (ACI 347)
- American Concrete Institute Specifications for Reinforced Cast-In-Place Concrete (ACI 318-08)
- American Concrete Institute Specifications for Structural Concrete (ACI 301)
- American Institute of Steel Construction (AISC 325 -11)
- American Iron and Steel Institute Specification for the Design of Light Gage Cold Formed Structural Steel Members (A.I.S.I)
- Steel Deck Institute Design Specifications (S.D.I)

Codes Used in Analysis

ASCE 7-05, Minimum Design Loads for Buildings

ACI 318-08, Building Code Requirements for Structural Concrete

International Building Code (IBC), 2009

Materials

Concrete		
Element	Weight	Strength (psi)
Footings	Normal	4000
Grade Beams	Normal	4000
Retaining Wall	Normal	4000
Retaining Wall Footing	Normal	4000
Interior Slab-On-Grade	Normal	3500
Exterior Slab-On-Grade	Normal	5000
Formed Slabs	Normal	5000
Formed Beams	Normal	5000
Columns	Normal	6000
Foundation Walls	Normal	4000
CMU Grout	Normal	2500
All Other	Normal	3000

Table 1: **Concrete Material Summery**

Steel		
Element	Standard	Grade
Reinforced Bars	ASTM 615	60
Welded Wire Reinforcement	ASTM 185	N/A
Pre-stressed Steel Wire	ASTM 416	N/A
Wide Flange Shapes (Beams, Girders, Columns etc.)	ASTM A992	50
Stiffener Plates	ASTM A572	50
Hollow Structural Sections	ASTM 500	B
Steel Pipe	ASTM A53	B
Angles, Channels, S-Shapes etc.	ASTM A36	36
Nuts, Bolts	ASTM A325, A490	N/A
Misc. Steel	ASTM A36	36

Table 2: Steel Material Summery

Gravity Loads

Dead and Live Loads

In this section, gravity loads (dead, live, and applicable) are presented. These loads are compared to actual building load calculations used in Embassy Suites Hotel. Assumptions for superimposed dead load are offered in Tables 3 to 5.

Live Load

Live Load		
Element	Design Live Load (psf)	Thesis Load (psf)
Guestroom Floors	40	40
Mechanical Rooms	150	150
Partitions	15	15
Elevator Machine Room	125	125
Stairs and Exit Ways	125	125
Slab on Grade	125	125
Balconies	125	125

Table 3: Live Load Values

Dead Load

Dead Load		
Element	Design Dead Load (psf)	Thesis Load (psf)
MEP	-	5
Ceiling	-	2

Table 4: Dead Load Values

Other Applicable Load		
Load Type	Load	Thesis Load (psf)
Roof Live	30	30
Concentrated Roof Load	300lb	300lb
Roof Rain Load	30	30
Snow Drift Load	20	20
Snow Load	20	20
Rain Water Load	125	125
Ponding Load	125	125
Sliding Snow Load	-	10

Table 5: Other Applicable Load Values

Lateral Loads

Wind Analysis

The wind analysis performed on the Embassy Suites Hotel was carried out in accordance with Chapter 6 of ASCE 7-05, *Wind Loads.* Due to the fact, that overall building height of the hotel exceeds 60 feet, it is necessary to use the Analytical Method of analysis. The values used in this analytical procedure can be found in Tables 6 - 8.Appendix C holds detailed wind analysis procedure. The wind directions are highlighted in the image below.

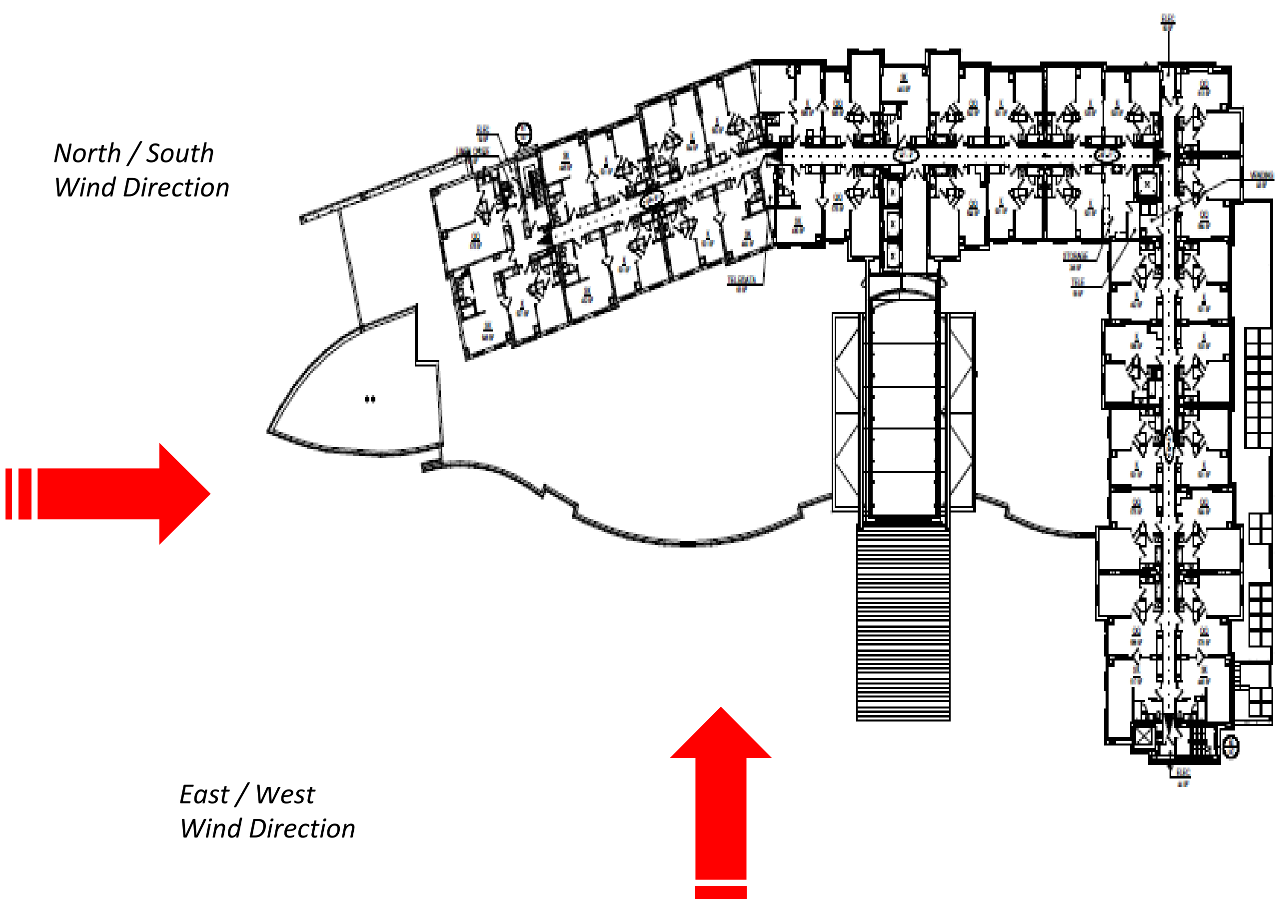


Figure: 11 North/ South and East / West Wind Direction

Wind Analysis Data			
Element	**Symbol**	**Value**	**ASCE7-05 Reference**
Basic Speed	V	90 mph	Figure 1
Directional Factor	Kd	0.85	Table 6-4
Importance Factor 1.0	I	1.0	Table 6-1
Occupancy Category		II	Table 1-1
Exposure Category B		B	Section 6.5.6.3
Enclosure Classification		Enclosed, Partially Enclosed	Section 6.5.9
Topographic Factor	Kzt	1.0	Section 6.5.7.2
Velocity Pressure Exposure Coefficient Evaluated @ Height Z	Kz	Varies	Table 6-3
Velocity Pressure @ Height Z	qz	Varies	Equation 6-15
Velocity Pressure @ Mean Roof Height	qh	.938	Equation 6-15
Gust Effect Factor	G		Section 6.5.8.1
Product of Internal Pressure Coefficient & Gust Effect Factor	GCpi	+/- 0.18, +/-.55	Figure 6-5
External Pressure Coefficient (Windward) (East /West Direction)	Cp	.8	Figure 6-6
External Pressure Coefficient (Leeward) (East /West Direction)	Cp	-.5	Figure 6-6
External Pressure Coefficient (Windward) (North /South Direction)	Cp	.8	Figure 6-6
External Pressure Coefficient (Leeward) (North /South Direction)	Cp	-.362	Figure 6-6
External Pressure Coefficient (Windward) (East /West Direction, Penthouse Roof)	Cp	-.5	Figure 6-6
External Pressure Coefficient (Leeward) (East /West Direction, Penthouse Roof)	Cp	-.18	Figure 6-6
External Pressure Coefficient (Windward) (North /South Direction, Penthouse Roof)	Cp	.51	Figure 6-8
External Pressure Coefficient (Leeward) (North /South Direction Penthouse Roof)	Cp	-.5	Figure 6-8
External Pressure Coefficient (Center Panel) (North /South Direction Penthouse Roof)	Cp	-1.14	Figure 6-8

Table 6: Wind Analysis Variables

The East/ West direction wind pressures were calculated in the analysis and presented in table (below). The wind hitting the East/ West facade had a greater impact due to it having more contact with the building. The contact length along the wall was taken as 326.4 feet. The first floor of the Embassy Suites Hotel is partially located underground, having the east face exposed (Store Front). Having the west face of the first level underground will not cause a wind load blockage and any effects of a blockage can be neglected in the analysis. Values in table may vary from actual values used in design of building. The windward and leeward pressures at all levels can be located in building elevation figure on the next page (Figure 12). Additionally, a load diagram of story shear is also provided in Figure 12 located on the following page.

East / West Direction

					Wind Pressure (psf)					
Level	Height Above Ground (ft.)	Story Height (ft.)	Kz	qz	Windward [pz]	Leeward [ph]	Total Pressure (psf)	Force of Windward Pressure	Force of Total Pressure	Sum Total Story Shear
Top Penthouse Roof	91.833		0.965	17.009	1.864	-11.623	13.487	878	6353	6.35
Roof	74.000	10.375	0.906	15.969	13.835	-10.002	23.837	46849	80721	87.07
Seventh	63.625	9.125	0.864	15.229	13.331	-10.002	23.333	39706	69494	156.57
Sixth	54.500	9.125	0.828	14.594	12.900	-10.002	22.902	38420	68210	224.78
Fifth	45.375	9.125	0.787	13.871	12.408	-10.002	22.41	36957	66745	291.52
Fourth	36.250	9.125	0.738	13.008	11.821	-10.002	21.823	35208	64997	356.52
Third	27.125	9.125	0.677	11.933	11.090	-10.002	21.092	33030	62820	419.34
Second	18.000	18	0.600	10.575	10.167	-10.002	20.169	59734	118495	537.84
First	0.000	0	0.000	0.000	0.000	0.000	0	0.000	0	0

Table 7: East / West Wind Values

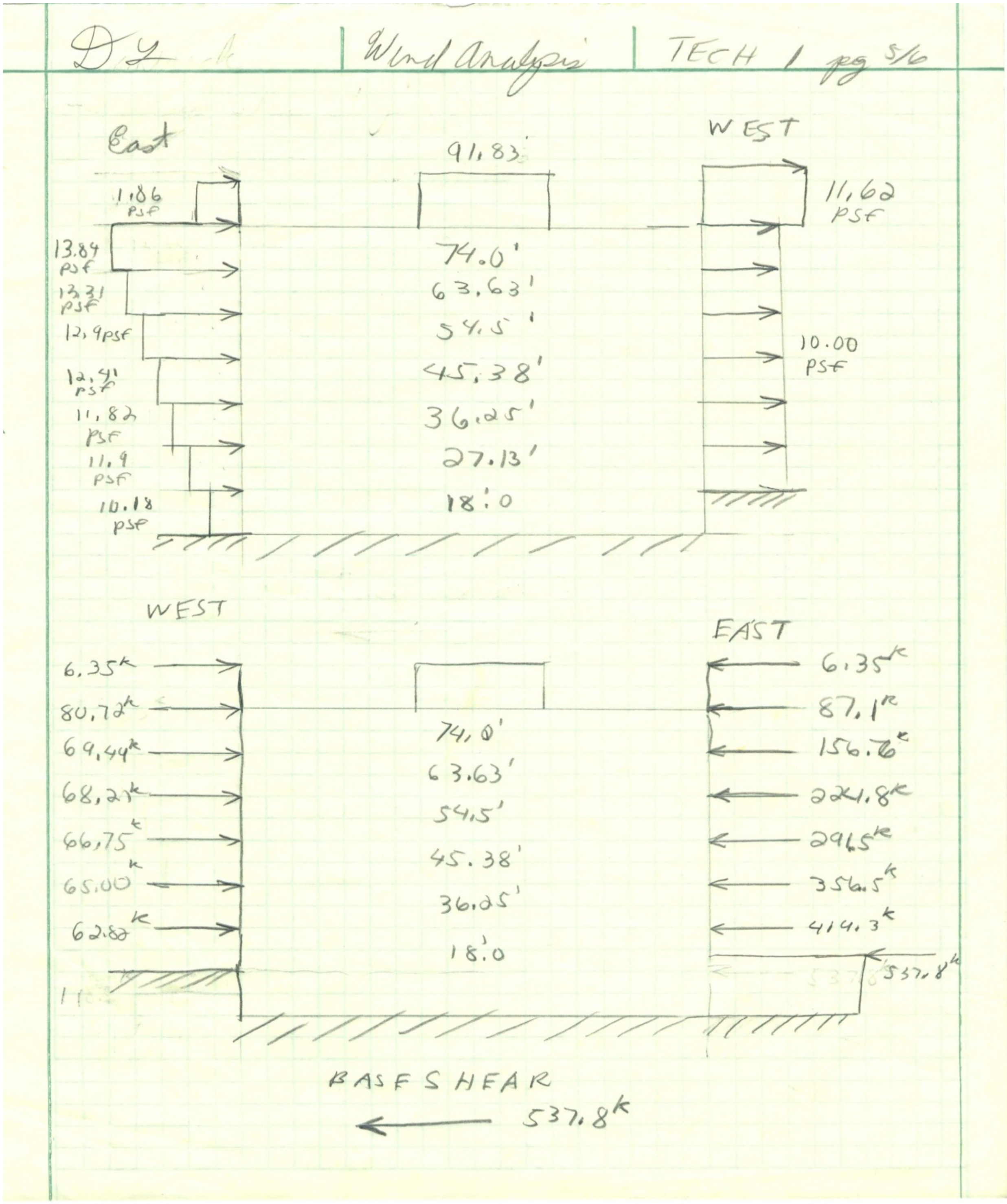


Figure: 12 East / West Wind Pressure and Story Shear

The North/ South direction wind pressures were calculated in the analysis and presented in table (below). The contact length along the wall was taken as 192.8 feet. In analyzing the wind along the North/ South facade the penthouse roof level had to be analyzed was arched roof, hence making the results for windward and leeward pressures different from the main flat roof level. Variables in table may vary from actual values used in design of building. The windward and leeward pressures at all levels can be located in building elevation figure on the next page. (Figure 13). Additionally, a load diagram of story shear is also provided in Figure 13 located on the following page.

North/ South Wind Direction										
					Wind Pressure (psf)					
Level	Height Above Ground (ft.)	Story Height (ft.)	Kz	qz	Windward [pz]	Leeward [ph]	Total Pressure (psf)	Force of Windward Pressure	Force of total Pressure	Sum Total Story Shear
Center Arched Roof	91.833	4.041	0.97	17.09	n/a	n/a	25.575	n/a	5572	5.57
Quarter Arched roof	87.792	13.792	0.953	16.797	16.232	-16.120	32.352	12100	24100	29.67
Roof	74.000	10.375	0.906	15.969	13.835	-8.063	21.898	27678	43810	73.48
Seventh	63.625	9.125	0.864	15.229	13.331	-8.063	21.394	23458	37645	111.12
Sixth	54.500	9.125	0.828	14.594	12.900	-8.063	20.963	22699	36887	148.01
Fifth	45.375	9.125	0.787	13.871	12.408	-8.063	20.471	21834	36021	184.03
Fourth	36.250	9.125	0.738	13.008	11.821	-8.063	19.884	20801	34988	219.02
Third	27.125	9.125	0.677	11.933	11.090	-8.063	19.153	19514	33702	252.72
Second	18.000	9.125	0.600	10.575	10.167	-8.063	18.23	17890	32078	284.80
First	0.000	18	0.000	0.000	0.000	-8.063	0.000	0.000	0	0.000

Table 8: North / South Wind Values

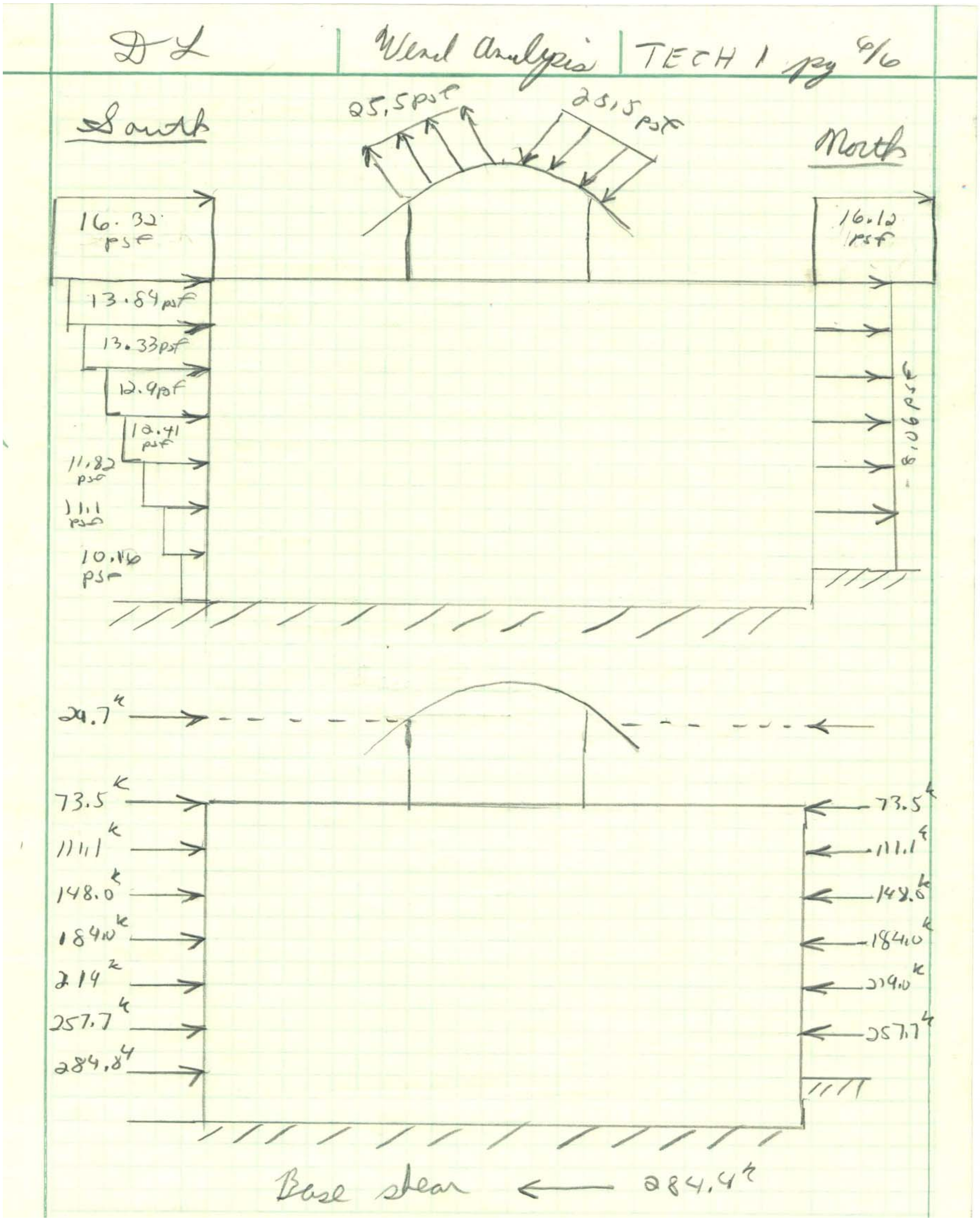


Figure: 13 North / South Wind Pressure and Story Shear

Seismic Analysis

Chapters 11 and 12 of ASCE 70-5 were used in the analysis of the seismic loads on The Embassy Suites Hotel. The hotel was designed to withstand the effects of seismic loads having the seismic design class designation B from section 1613.5.6 of the IBC 2009 and a site class designation of D from section 1613.5.2 of the IBC 2009. Seismic Design values are listed in Table 9 and 10. Seismic Design values and base shear calculation may differ from actual design values used in the design of The Embassy suites due to the use of different assumed dead loads per floor. It is important to mention the the assumed base level for calculating the building load was taken at level 2 to giving the total height above grade to be 56 feet. See Appendix D for detailed calculations of shears and gravity loads.

Seismic Analysis Data			
Element	Symbol		ASCE 70-5 References
Site Class		D	Table 20.3-1
Occupancy Category		II	Table 1-1
Importance Factor		1	Table 11.5-1
Structural System		Ordinary Reinforced Concrete Moment Frames	Table 12.2-1
Spectral Response Acceleration, short	Ss	0.155	USGS
Spectral Response Acceleration	S1	0.051	USGS
Site Coefficient	Fa	1.6	Table 11.4-1
Site Coefficient	Fv	2.4	Table 11.4-2
MCE Spectral Response Acceleration	Sms	0.248	Eq. 11.4-1
MCE Spectral Response Acceleration	Sm1	0.122	Eq. 11.4-2
Design Spectral Acceleration	Sds	0.165	Eq. 11.4-3
Design Spectral Acceleration	Sd1	0.081	Eq. 11.4-4
Seismic Design Category	Sdc	B	Table 11.6-2
Response Modification Coefficient	R	3	Table 12.212
Approximate Period Parameter	Ct	.016	Table 12.8-2
Building Height (above grade)	hn	56 feet	
Approximate Period Parameter	x	.9	Table 12.8-2
Approximate Fundamental Period	Ta	.599	Table 12.8-7
Long Period Transition Period	TL	8 s	Figure 22-15
Seismic Response Coefficient	Cs	0.055	Eq. 12.8-2
Structural Period Exponent	k	1.0	Eq. 12.8-3

Table 9: Seismic Analysis Variables

Base Shear							
Story	Floor Area	Story Ht.	Story Weight	wxhx	Cvx	Lateral Force Fx (k)	Story Shear Vx
2	23,907	0	735	0	0	0	-
3	23,946	9.125	3249.7	29609	.2258	75	781
4	23,899	9.125	3244.8	29609	.2258	176.3	781
5	23,899	9.125	3244.8	29609	.2258	176.3	603.9
6	23,899	9.125	3244.8	29609	.2258	176.3	427.6
7	23,899	9.125	3244.8	29656	.2262	176.3	251.3
Roof	23,899	10.375	3249.7	12639	.096	176.7	75

Table 10: Base Shear Values

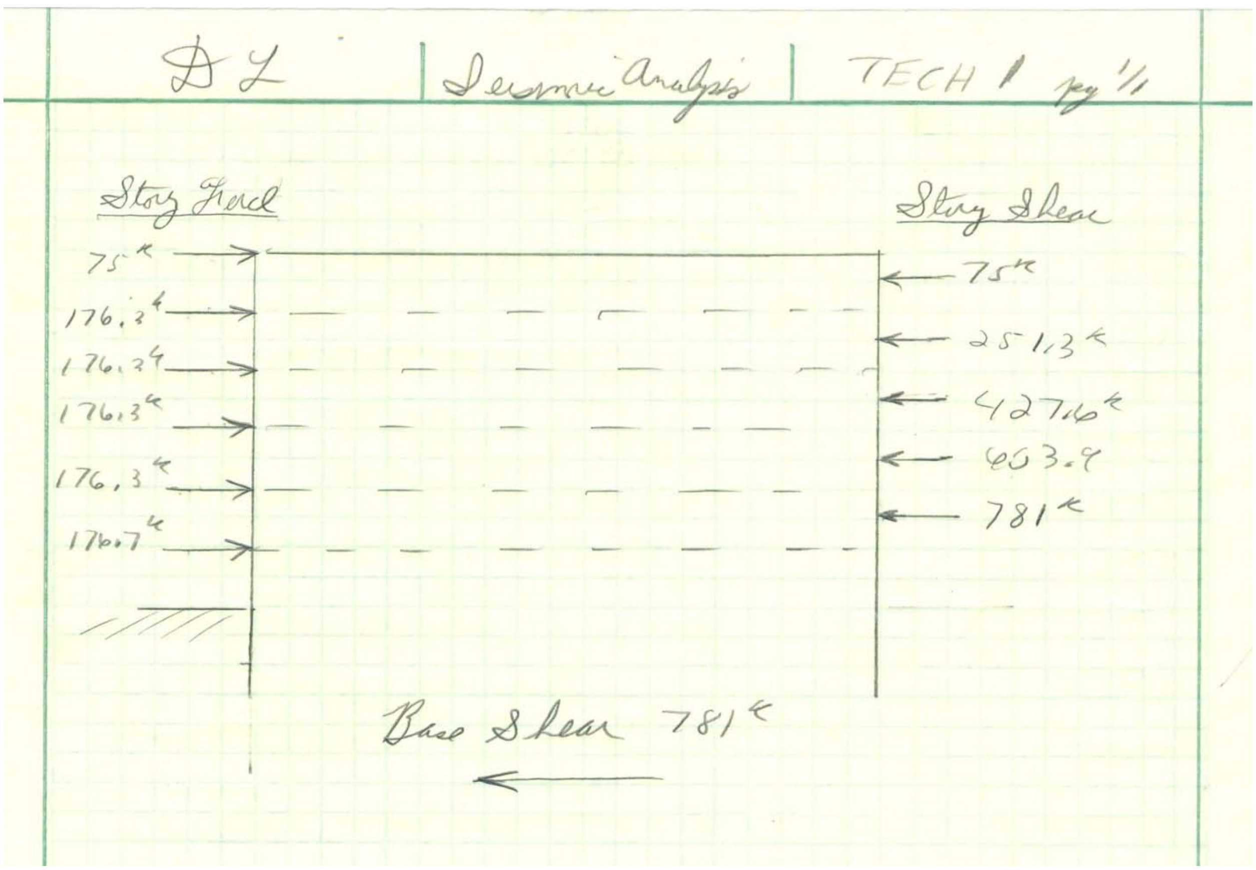


Figure 14: Seismic Story Force and Story Shear

Spot Checks

To gain a better understanding of the structural elements used in the design of the Embassy Suites Hotel a number of spot checks were performed on typical floor levels. The spot checks performed consisted of an interior and exterior column and a two way slab analysis. Gravity load calculation results may vary due to different assumptions of dead loads and the fact that lateral loads were not taken into account. Detailed spot checks are available in Appendix E. Spot Check locations are indicated in Figure 15.

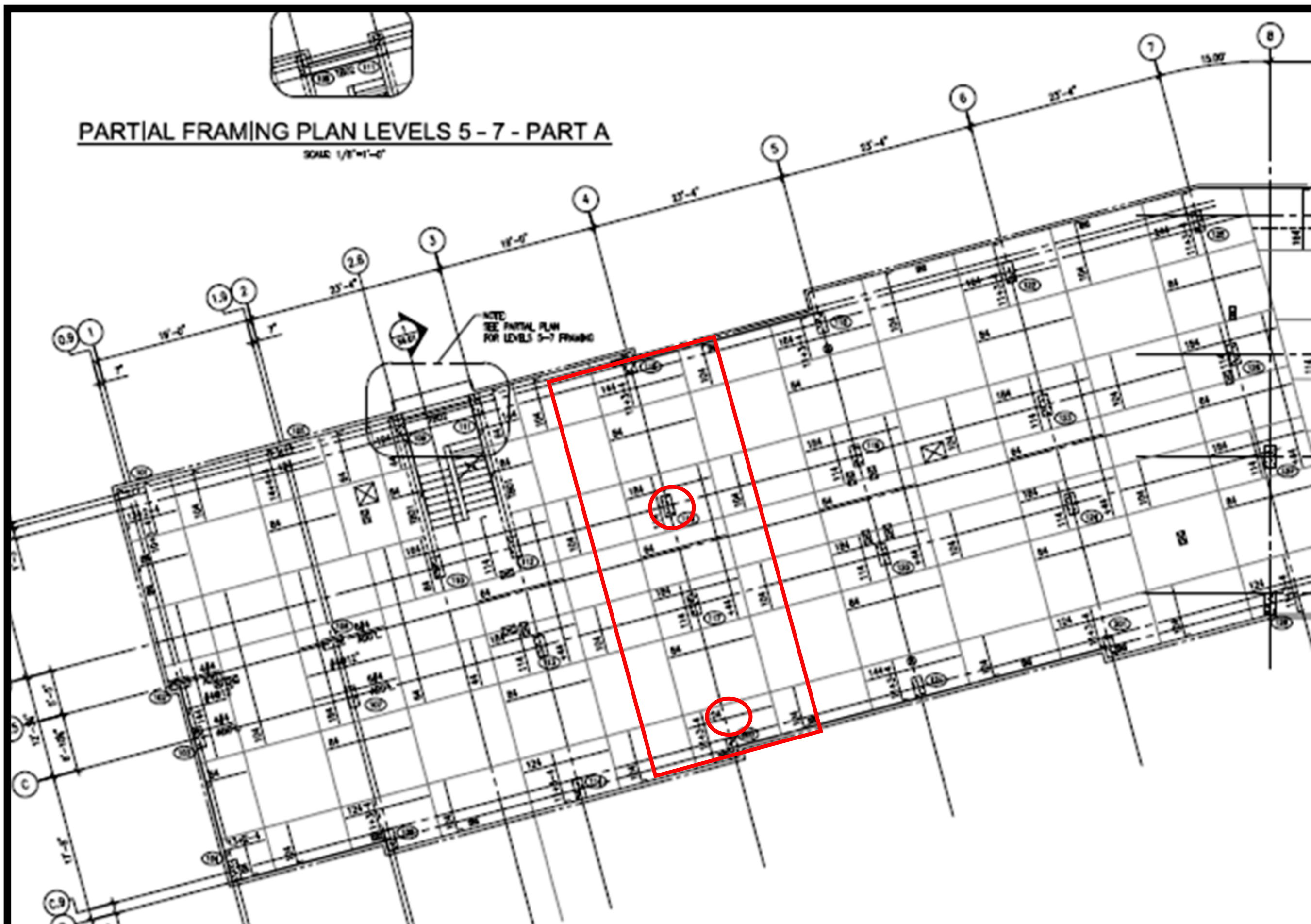


Figure 15: Spot Check Locations

A two way slab analysis was performed between column line 5. The ultimate moment was computed due to loads listed in seismic analysis. The analysis for flexure consisted of checking if the slab could resist the moments distributed to its column and middle strips by seeing if adequate reinforcing and compressive strength of the concrete was available using parameters outlined by the Direct Design Method in chapter 13 ACI 318-08. Additionally a punching shear check was performed to see if the slab had enough strength resist localized shear forces.

Additionally, spot checks for interior and exterior 14 x 30 reinforced concrete columns were computed in a typical floor. Dead and live were compute taking into account their respective tributary areas and location in building. Live load reductions were considered for both columns. Due to the tributary area being below the required 400sq area needed to reduce the live load, the exterior column takes the full percentage of the live load. Detailed calculations of spot checks are available in Appendix E.

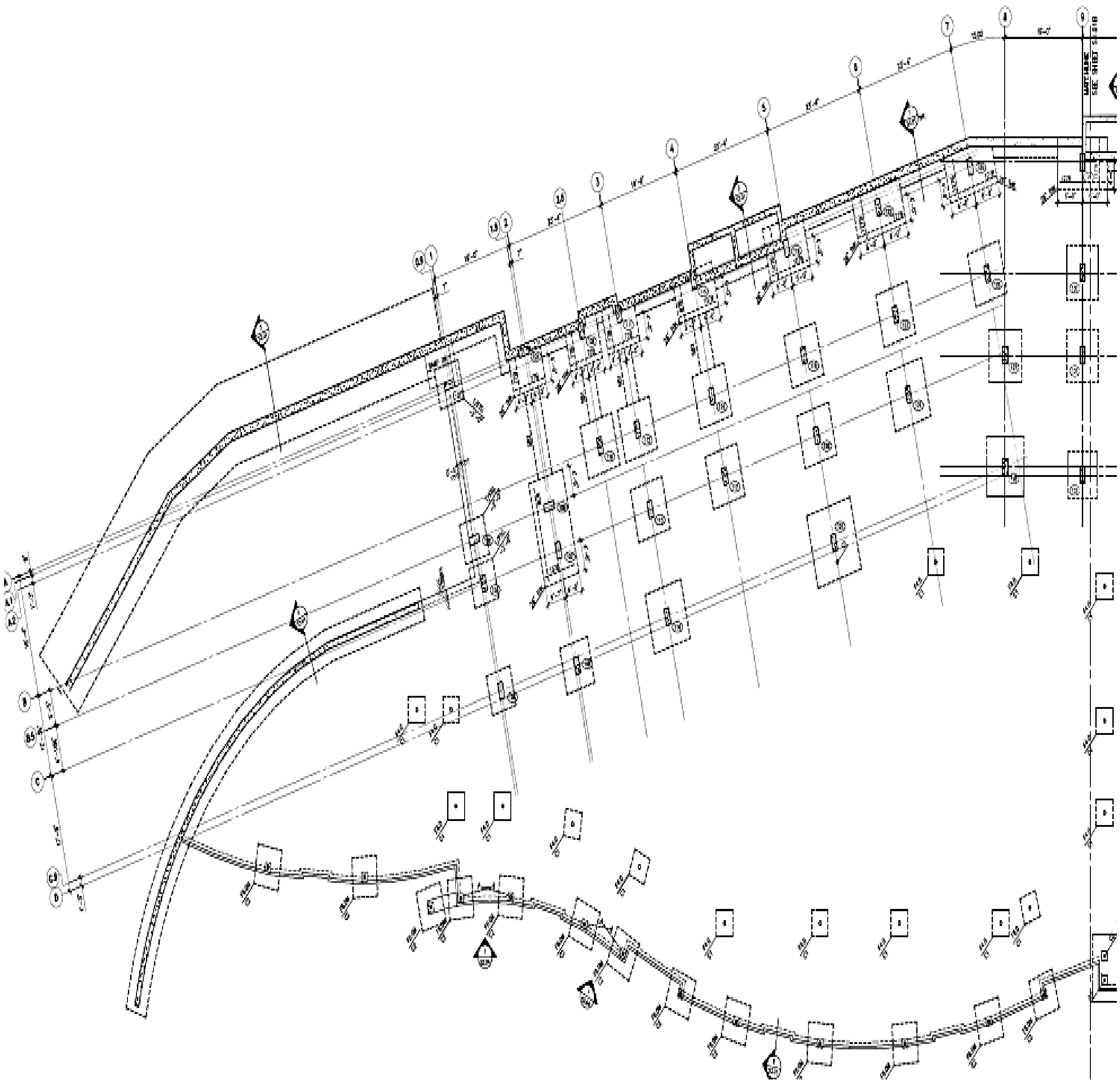
Conclusion

By analyzing the existing conditions of the Embassy Suites Hotel one gains a better understanding how multiple structural elements work together as a part of a whole structural system in a building. In developing explanations of the various systems and performing wind seismic and gravity analysis it was found that the design of the Embassy Suites Hotel was developed according to code standards and can resist the loads that will be applied.

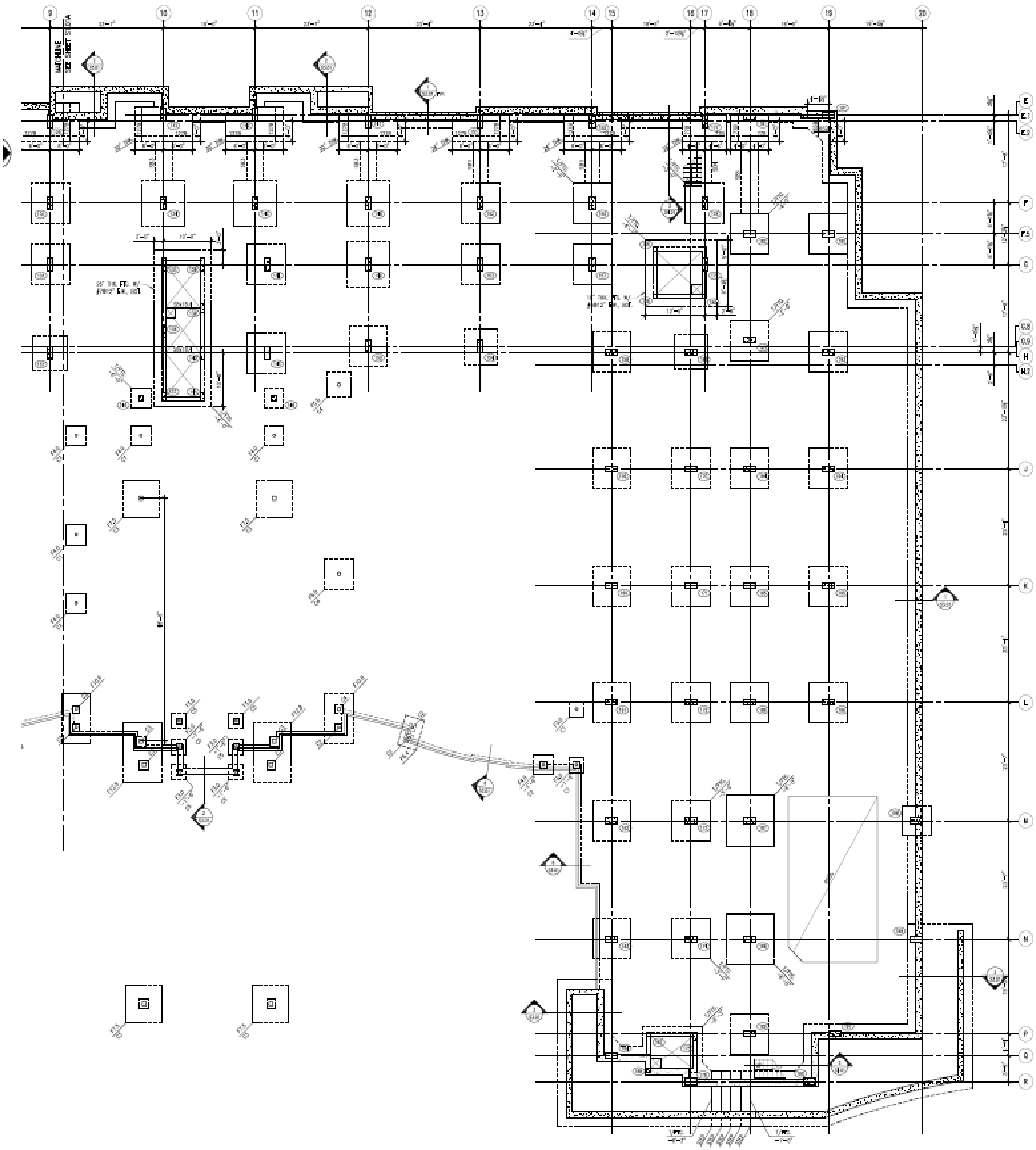
In the wind load analysis it was found that wind pressures acting on the building were greatest along the East and West faces of the Embassy Suites Hotel as opposed to the North and south façade having a much smaller face. Neglecting torsion effects it was established that the later loads due to seismic were the controlling lateral load. The difference in seismic design values are because in this analysis openings in the atriums and stair and elevator shafts were not subtracted in the calculation of the building weight. If this was performed the result would make the base shears and overturn reduce in size getter closer to the actual design values.

The main structural components of the load resisting systems include reinforced concrete moment frames, contained flat slab construction and reinforced concrete W14 x 30 columns located throughout the building. The columns that are a part of this moment frame system are designed due the combination of lateral and gravity loads that can cause different loading effects on the particular members. The slab of lateral system has found to require top reinforcing bars in the columns trip to prevent the failure mechanism known as punching shear. The difference in values obtained in the spot check as compared to the actual values can be a result of the different load assumptions made in the calculations of the capacities of members. Other reasons for inconstancies in values are due to the lack of a more intricate analysis. More detailed future analyses and more detailed calculations and research will lead to greater comprehension and understanding of the whole structural system as one cohesive unit.

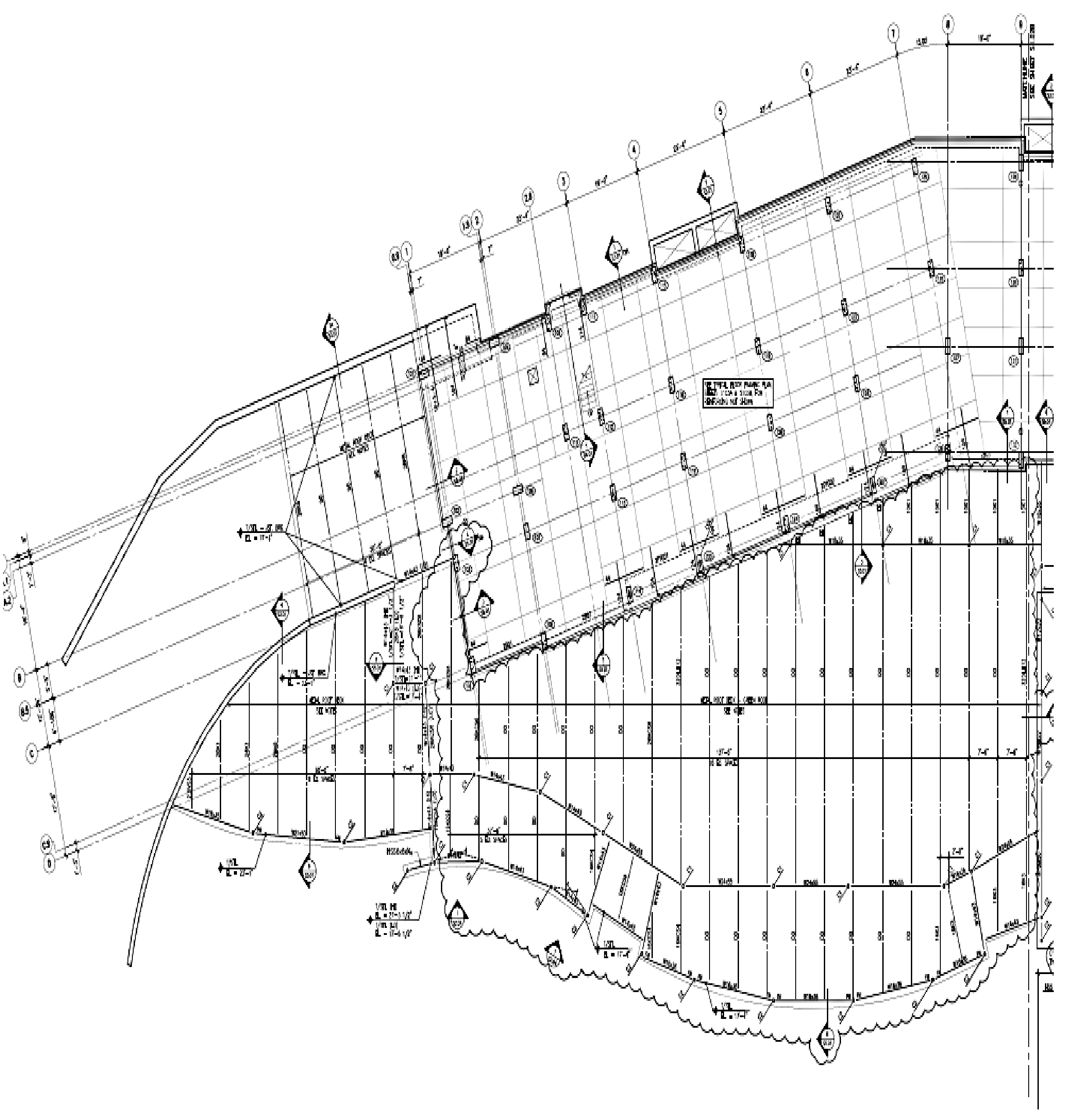
Appendix A: Plans



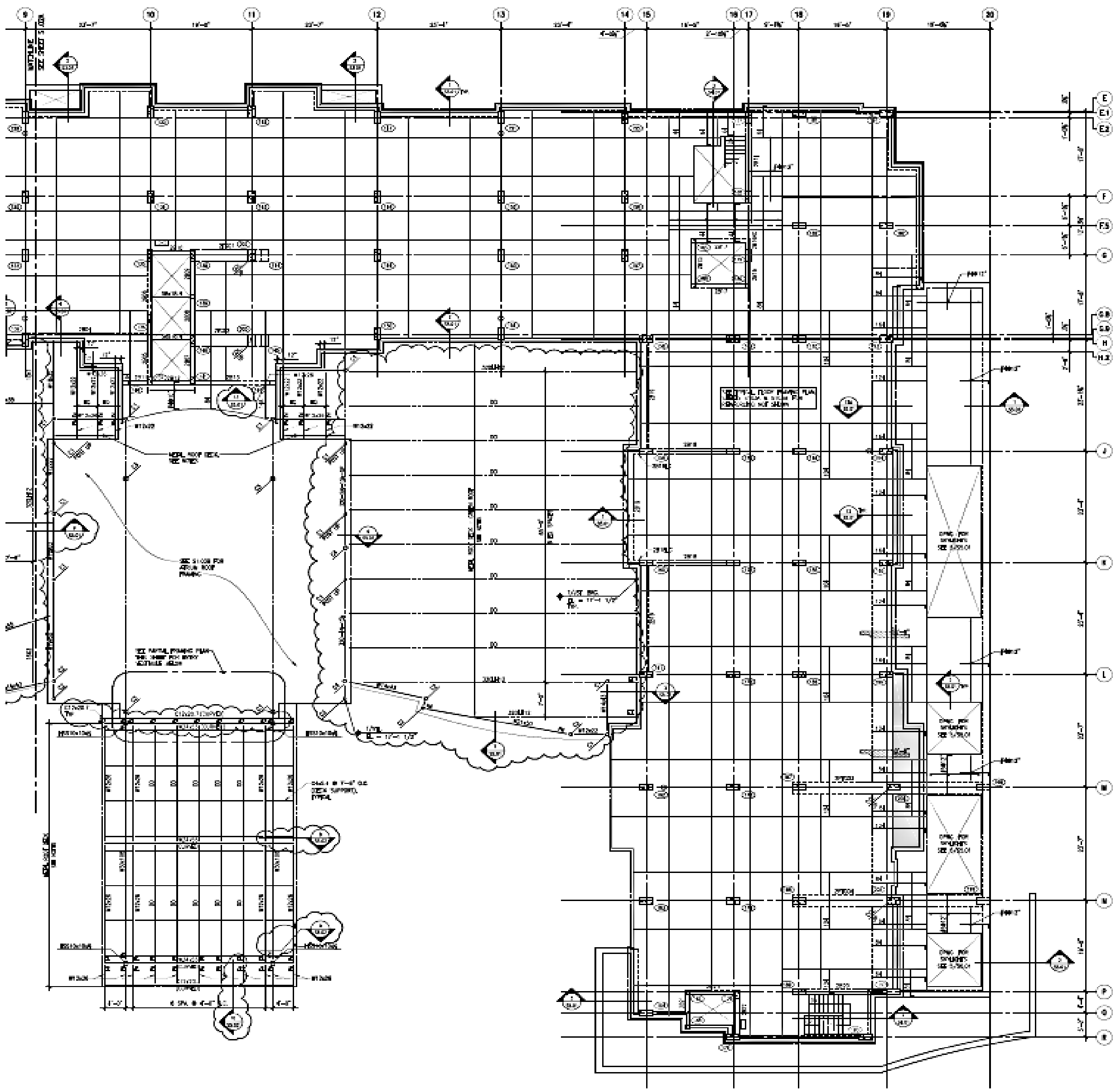
Foundation Plan A



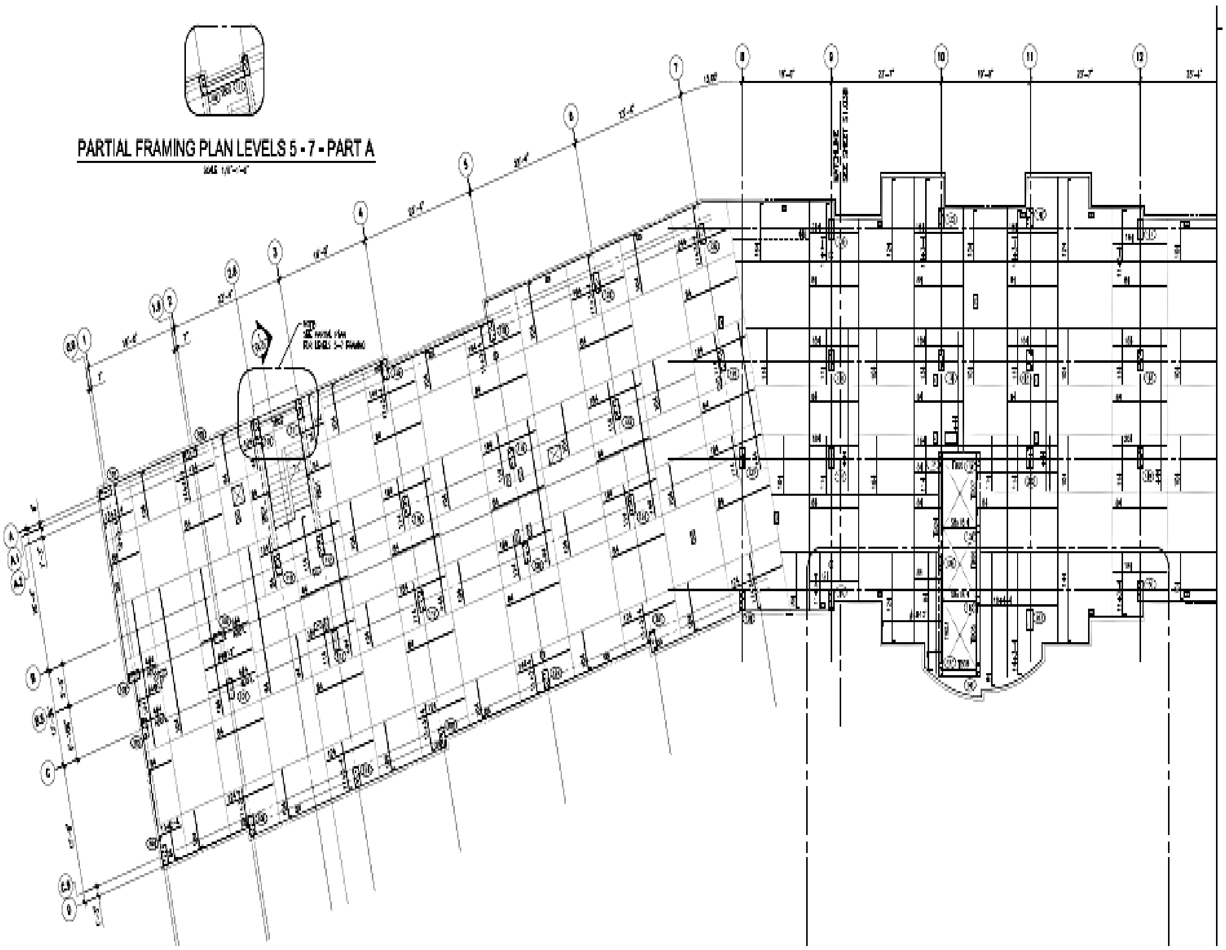
Foundation Plan B



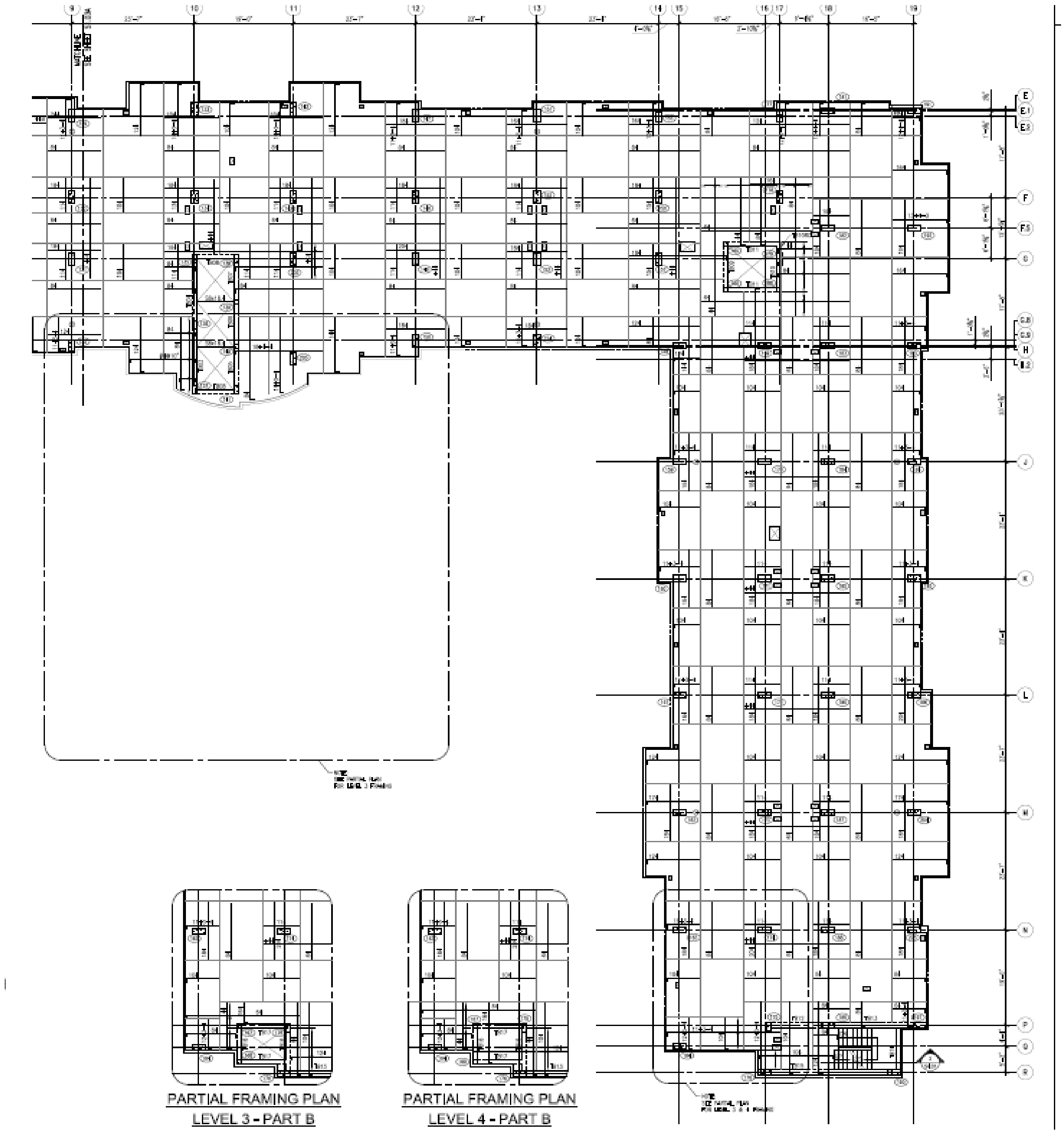
Floor Plan 2 Part A



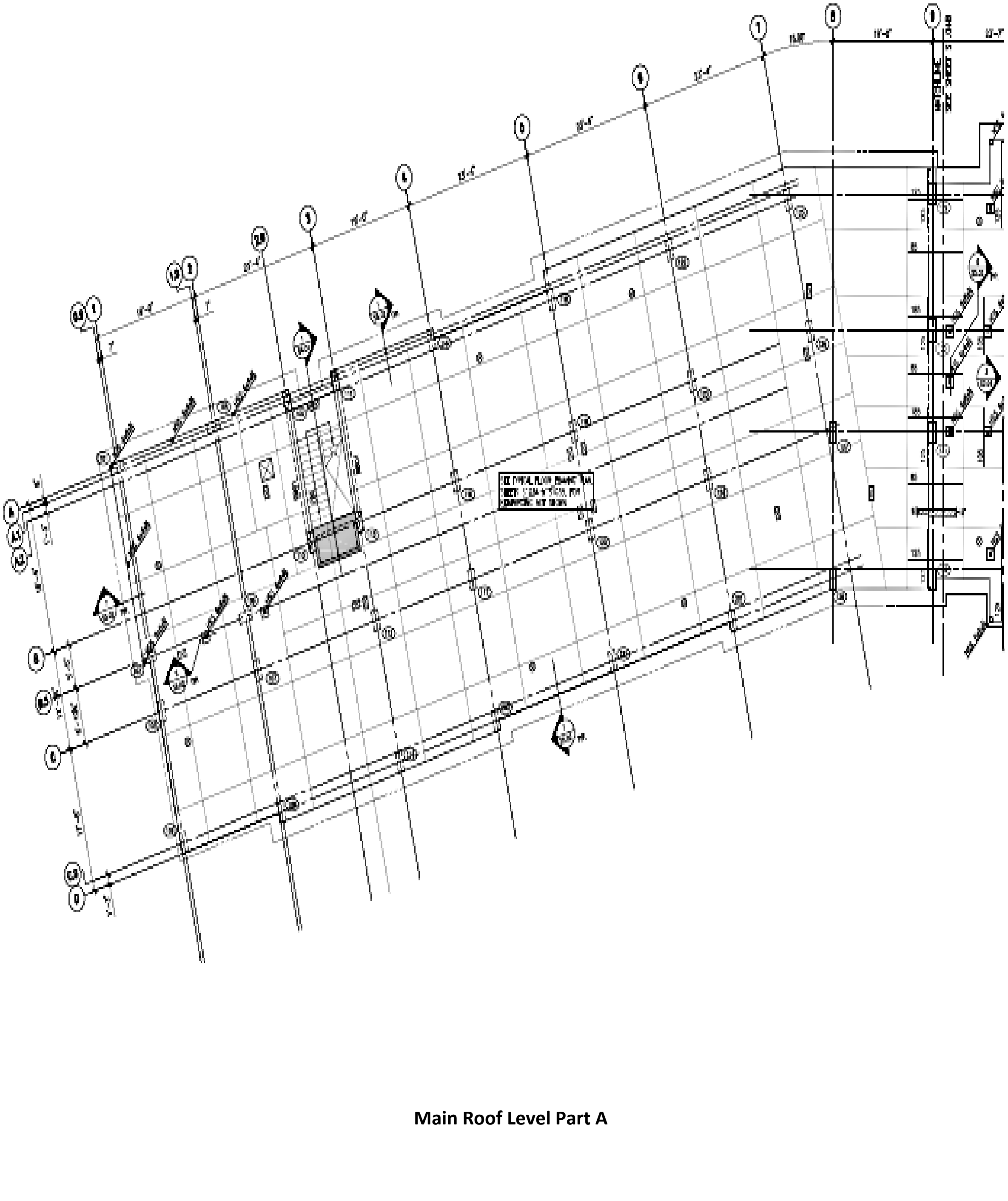
Floors Plan 2 Part B



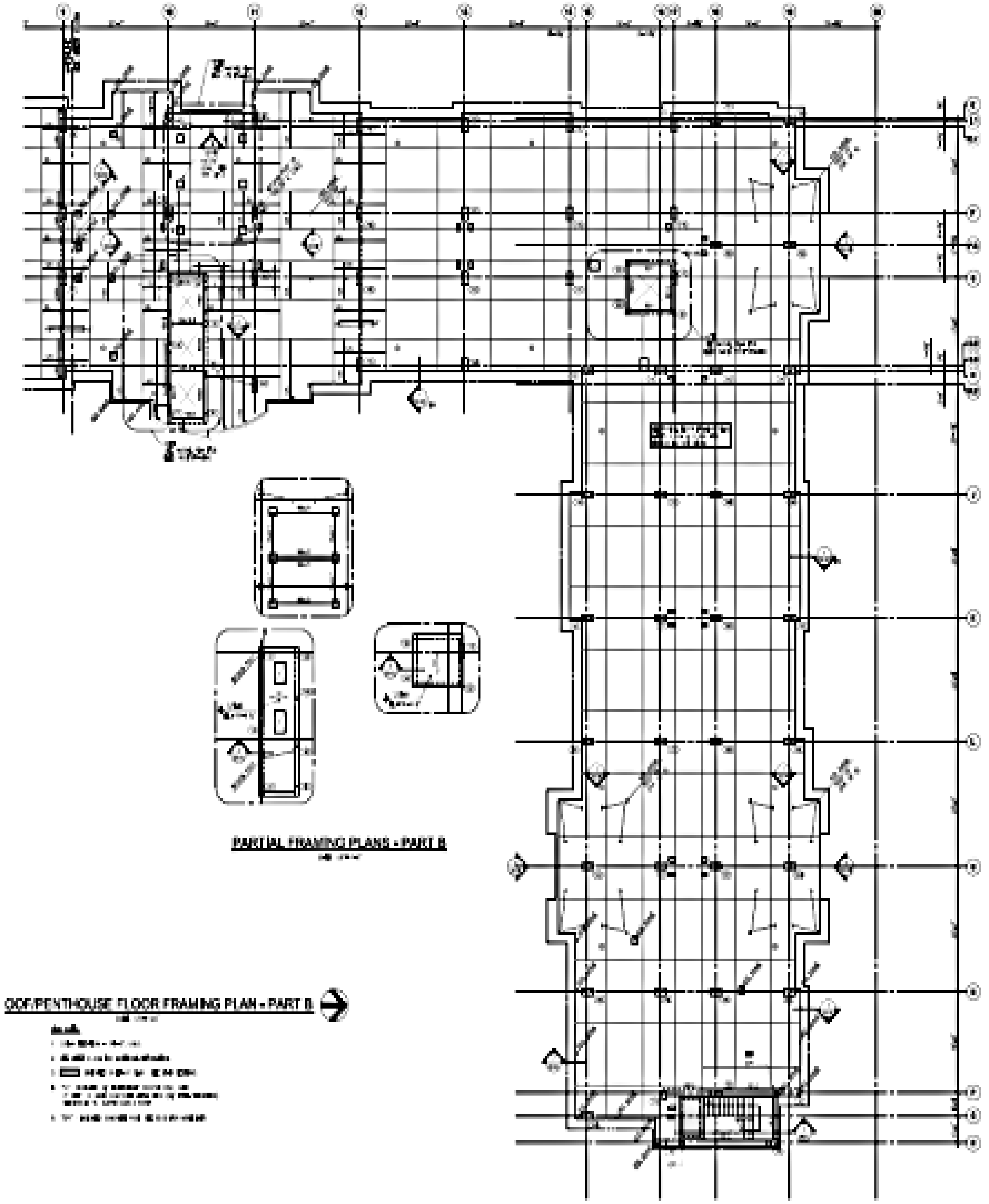
Floors Plan 3 to 7 Part A



Floor Plan 3-7 Part B



Main Roof Level Part A



Main Roof Level Part B

Appendix B: Snow and Sliding Snow Analysis

DL | Snow Analysis | TECH 1

Snow Analysis (Springfield, Va)

- Ground Snow Load → $P_g = 25$ psf
- Exposure factor → $C_e = .9$
- Thermal factor → $C_t = 1.0$
- Importance factor → $I = 1.0$

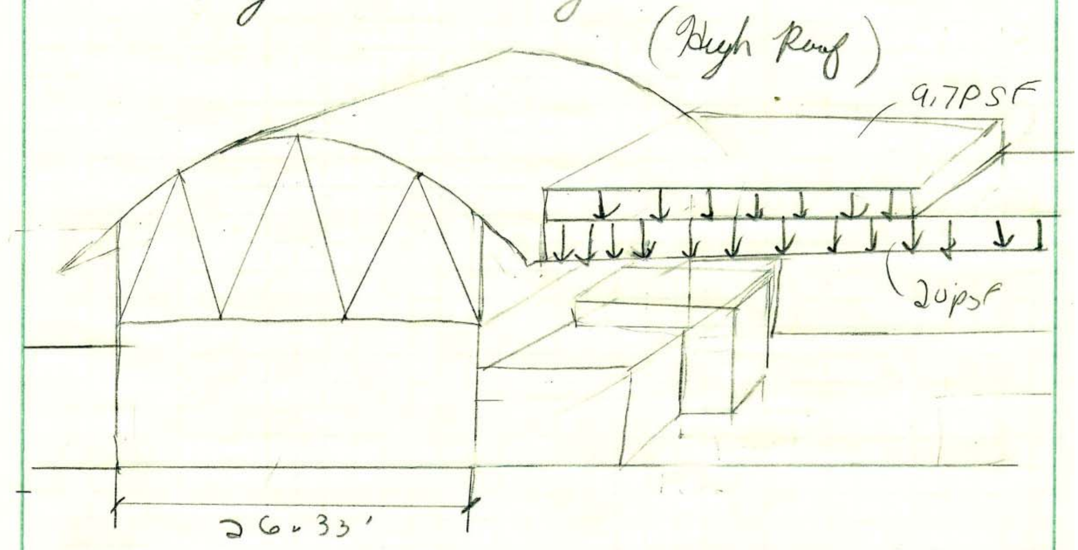
$P_f = .7(C_e)(C_t) I P_g > 20 I$

$P_f = .7(.9)(1.0)(1.0)(25) = 15.75$ psf $< 20(I)$

* Have to use 20 psf as a minimum value

DL | Sliding Snow | TECH 1 pg 1/1

Sliding Snow Analysis



- slope of roof is large enough ($8/18 = .44$) for consideration of sliding snow load

$p_s = .7 C_e C_t I p_g$

$p_s = .7(.9)(1.0)(1.0)(25) = 15.75 < 20psf \rightarrow$ use 20psf

- sliding surcharge on lower roof $\rightarrow S = .4 p_f W = .4(20)(18.21') = 145.7$ plf
- distributing over 15' width (ASCE 7-05)

  145.7 psf/15' = 9.71 psf
- an additional load can be 9.71 psf can be applied to lower flat roof within 15' of sloped high roof

Appendix C: Wind Load Analysis

DL | Wind Analysis | TECH.1 pg 1/6

- Wind Analysis (Using ASCE 7-05)
- location - Springfield, Va
- h>60' - Method 2 Analytical procedure
- Wind Variables:

$V = 90$ MPH, $K_d = .85$, $I = 1.0$, $K_{zt} = 1.0$, Exposure = B

*Kz values differ at floor heights from values presented in TBL 6-3 - Have to interpolate (CASE II)

Level	Height above Grade	Kz
1	0'-0"	0
2	18'-0"	.60
3	27'-1½"	.677
4	36'-3"	.738
5	45'-4½"	.787
6	54'-6"	.828
7	63'-7½"	.864
ROOF	74'-0"	.906
PENTHOUSE	91'-10"	.974

- $q_z = .00256\, K_z K_{zt} K_d V^2 I$ (velocity pressure)

Example calculation: $.00256(.6)(1.0)(.85)(90)^2(1.0)$
@ Level 2 $= 10.575 = q_z$

* remainder of calculations computed for q_z in TBL.
* Building considered rigid structure, $G = .85$

DL | Wind Analysis | Tech 1 pg #/6

q_h @ mean roof HT. $z = \frac{91.833 + 74}{2} = 82.917'$

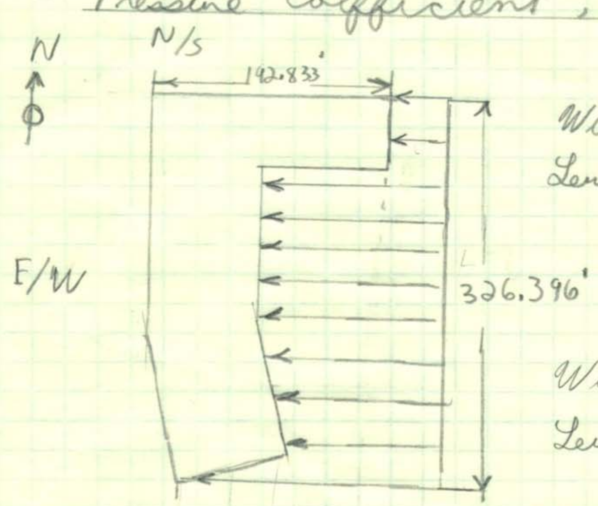
$\bar{z} = .6(82.917') = 49.745'$

$49.745' > z_{min} = 30'$ ok ✓

k_z @ $82.917 = .938$

$q_h = .00256(.938)(1.0)(.85)(90)^2(1.0) = 16.533\,psf$

- Pressure Coefficient, C_p



East / West

Winward $= .8 = C_p$

Leeward - $L/B = \frac{192.833}{326.396} = .59$

$C_p = -.5$

North / South

Winward $= .8 = C_p$

Leeward - $L/B = \frac{326.396}{192.833} = 1.69$

$C_p = -.362$

- Wind Pressure

$P_z = q_z G C_p - q_h G C_{pi}$ → windward

$P_h = q_h G C_p - q_h G C_{pi}$ → leeward

* GC_{pi} is $(+.18, -.18)$ for enclosed buildings

- example - E/W → Level 3

$P_z = (11.993)(.85)(.8) - (16.533\,psf)(-.18) = 11.090\,psf$

$P_h = (16.533)(.85)(-.5) - (16.533\,psf)(.18) = -10.002\,psf$

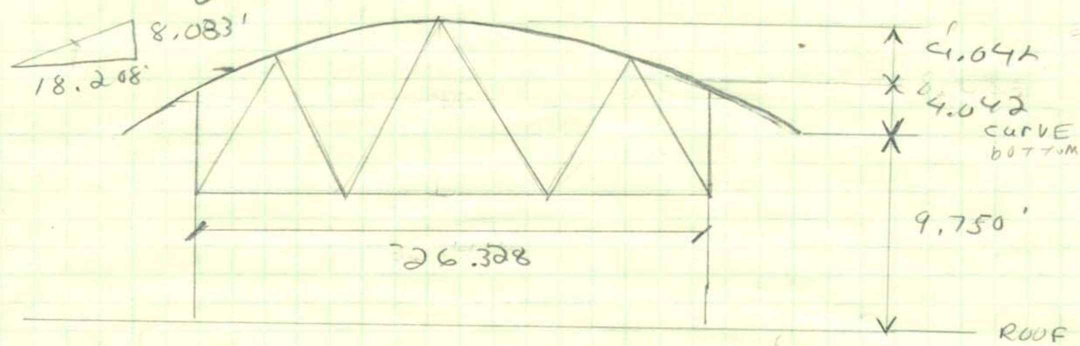
- example - N/S → Level 3

$P_z = (11.993)(.85)(.8) - (16.533)(-.18) = 11.090\,psf$

$P_h = (16.533)(.85)(-.362) - (16.533)(.18) = -8.063\,psf$

DL | Wind Analysis | TECH. 1 pg 3/6

- Penthouse level assumed to be partially enclosed use $GC_{pi} = (\pm.55$



- Roof elevated on structure (N/S direction)
  - rise to span ratio → $\frac{8.083'}{18.208'} = .44$
  - $.3 \le .44 \le .6$
  - windward quarter $C_p = 2.75(.44) - .7 = .51$
  - Center half $C_p = -.7 - .44 = -1.14$
  - Leeward quarter $C_p = -.5$

N/S $p_z = (16.799)(.85)(.51) - (16.533)(-.55) = 16.232$ (windward)

$p_h = (16.533)(.85)(-.5) - (16.533)(.55) = -16.120$ (leeward)

E/W $h/L \le .5$, h to 2h, $C_p = -.5, -.18$

$p_z = (17.004)(.85)(-.5) - (16.533)(.55) = -11.864$ psf

$p_b = (16.533)(.85)(-.18) - (16.533)(.55) = -11.623$ psf

* Center Panel N/S

$p_z = (17.004)(.85)(-1.14) - (16.533)(.55)$

$p_z = 25.575$ psf

D L | Wind Analysis | TECH 1 pg 4/6

Force of Windward wall

$F = (BASE)(STORY\ HT.)(pz)$

- example E/W → level 2 (windward wall)

$F = (326.346')(9.125')(10.575\,psf) = 30283\#/1000 = 30.3^k$

- using TOTAL pressure → level 2

$F = (326.346')(9.125)(20.164) = 60.1^k$

- example N/S → level 2 (windward wall)

$F = (192.833)(9.125)(10.575) = 18.6^k$

- using TOTAL pressure → level 2

$F = (192.833)(9.125)(18.23) = 32.1^k$

- example E/W → Penthouse (top) (windward only)

$F = (26.416)(17.833)(1.864) = .878^k$

- using total load

$F = (26.416)(17.833)(17.948) = 6.35^k$

- example N/S → Penthouse roof (windward, windward only)

$F = (53.916)(13.792)(16.232) = 12.1^k$

- using total load (@ greater)

$F = (53.916)(13.792)(32.352) = 24.1^k$

- * TOTAL LOAD E/W Center panel

$F = (53.916)(4.041)(25.575) = 5.572^k$

Appendix D: Seismic Analysis

Dead Loads(Floors 2)						
Columns						
Element	Material	Shape	Quantity	Weight (pcf)	Floor Ht. (ft.)	Load(K)
Column	Concrete	14x30	182	150	9.125	727.4
Column	Concrete	10x20	6	150	9.125	11.4
Column	Concrete	10x10	6	150	9.125	5.7

Dead Loads(Floors 3)						
Columns						
Element	Material	Shape	Quantity	Weight (pcf)	Floor Ht.(ft.)	Load(K)
Column	Concrete	14x30	184	150	9.125	735.4
Column	Concrete	10x20	6	150	9.125	11.4
Column	Concrete	10x10	6	150	9.125	5.7
Slab						
Thickness(in)		Weight			Load (K)	
8		150			2394.6	
Superimposed						
Type		Weight (psf)			Load (K)	
MEP		**5**			119.7	

Dead Loads (Floors 4-7)						
Columns						
Element	Material	Shape	Quantity	Weight (pcf)	Floor Ht.	Load(K)
Column	Concrete	14x30	184	150	9.125	735.4
Column	Concrete	10x20	6	150	9.125	11.4
Column	Concrete	10x10	6	150	9.125	5.7
Slab						
Thickness(in)		Weight		Load (K)		
8		150		2389.9		
Superimposed						
Type		Weight (psf)		Load (K)		
MEP		**5**		119.5		

Dead Loads (Roof)		
Slab		
Thickness(in)	Weight	Load (K)
3.5	150	971
Superimposed		
Type	Weight (psf)	Load (K)
Metal Deck	**4.36**	104.2
Snow Load*	**6**	143.4

*Due to ASCE 7-05 takes 20% of roof snow load

D.L. | Seismic Analysis | TECH 1 pg 1/3

Seismic Analyses (using ASCE 7-05)

- $S_1 = .051$, $S_S = .155$, $F_v = 2.4$, $F_a = 1.6$

$S_{MS} = F_a S_S = (1.6)(.155) = .248$

$S_{DS} = (2/3) S_{MS} = (2/3)(.248) = .165$

$S_{M1} = F_v S_1 = (2.4)(.051) = .1224$

$S_{D1} = 2/3(.1224) = .0816$

$S_{DS} = .165$ & OC II $\rightarrow$ SDC = "A" $\rightarrow$ use B
$S_{D1} = .0816$ & OC II $\rightarrow$ SDC = "B"

- Equivalent Lateral Force Procedure
(Seismic Base Shear)

- response mod. factor, $R = 3$ (ordinary concrete moment frames)

$T_a = C_t h_x$ $\rightarrow$ for moment frames $h_w =$, $x = .9$
$C_t = .016$

$T_a = (.016)(56')^{(.9)} = .599$

- for Springfield, Virginia, $T_L = 8$ seconds

$C_s = \frac{S_{DS}}{R/I} = \frac{.165}{3/1.0} = .055 \geq .01$ ✓

$T_a = .599 < T_L = 8s$ ✓

DL | Seismic Analysis | TECH.1 pg 2/3

- Seismic Loads

- Roof → slab $= \left(\frac{3.25}{12''}\right)(23{,}899\,ft^2)(150) = 971^k$

metal deck $= (4.36\,psf)(23{,}899) = 104.2^k$

snow load $= (20\%)(30\,psf)(23899) = 143.4^k$

- Floors 4-7 → slab $= \left(\frac{8''}{12}\right)(23{,}899\,ft^2)(150) = 2389.9^k$

MEP $= (5\,psf)(23{,}899\,ft^2) = 119.5^k$

columns (*see excel spreadsheet) $= 735.4^k$

- Floor 3 → slab $= (8/12)(23{,}946\,ft^2)(150)$ 2394.6

MEP $= (5\,psf)(23946\,ft) = 119.7^k$

columns = (*see excell spreadsheet) $= 735.4^k$

- Floor 2 → MEP $= (5\,psf)(23{,}907) = 119.5^k$

Columns $= 735.4^k$

*Do not include slab @ 2 in calculation

TOTAL LOAD → $(971^k + 104.2 + 143.4^k)_{roof}$ +

$[2389.9 + 119.5 + 735.4]$(3 FLOORS)

$+ (2394.6 + 119.7 + 735.4) + (119.5 + 735.4)$

$= 15057.6^k$

$V = C_s W = (.055)(15057.6) = 828.2^k$ = base shear

DL | Seismic Analysis | TECH 1 pg 2/3

Vertical Distribution of Seismic Forces

$F_x = C_{vx} V$, $C_{vx} = \frac{w_x h_x^k}{\Sigma w_i h^k}$ $k = 1.0$ $T \leq .5$

$C_{RL} = \frac{1218.2(10.375')^{1.0}}{(3)(FL)(3244.8)(9.125)^{1.0} + (3249.7)(9.125)^{1.0} + 1218.2(10.375)^{1.0}}$

$C_{RL} = .096$

$C_{FL_{4-7}} = \frac{(3244.8)(9.125)^{1.0}}{(3)(3244.8)(9.125)^{1.0} + (3249.7)(9.125)^{1.0} + 1218.2(10.375)^{1.0}}$

$C_{FL\,4-7} = .2258$

$C_{FL\,3} = \frac{(3249.7)(9.125)}{(3)(3244.8)(9.125)^{1.0} + (3249.7)(9.125)^{1.0} + (1218.2)(10.375)^{1.0}}$

$C_{FL_3} = .2262 \rightarrow .096 + (.2258)(3) + .2262 = 1.0$ ok ✓

$F_{RL} = .096(781^k) = 75^k$

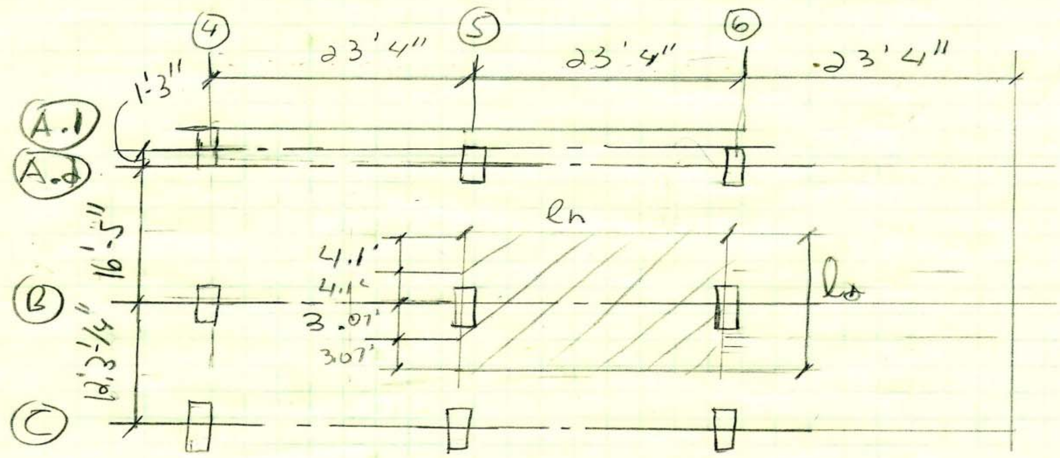
$F_{FL\,4-7} = .2258(781^k) = 176.3^k$

$F_{FL\,3} = .2262(781^k) = 176.7^k$

Appendix E: Spot Checks

DL | Slab check | TECH 1 pg 1/5

2 Way Slab Spot Check



8" - Concrete Slab

#4 reinforcing spaced 10" O.C each way

$f_y = 5000psi$, $f_y = 60{,}000psi$

$l_n = 23'.33$, $l_2 = 14.34'$

- Dead Load
  - concrete = 150pcf (8/12) = 100psf
  - MEP = 10psf
- Live Loads
  - Guestroom floors → 40psf
  - partitions → 15psf

DL | slab check | TECH 1 pg 4/5

$q_u = 1.2(110) + 1.6(55) = 220\ psf$

$M_o = \frac{q_u l_2 l_n^2}{8} = \frac{.220(14.34)(23.33)^2}{8} = 214.6\ k\text{-}ft$

- Positive & Negative Moments (interior span)

$M^- = .65M_o = .65(214.6\ k\text{-}ft) = 139.5\ k\text{-}ft$

$M^+ = .35M_o = .35(214.6\ k\text{-}ft) = 74.4\ k\text{-}ft$

- Column Strip Moments

$l_1/l_2 = \frac{23.33}{14.34} = 1.63$

$M^-_{col} = .75(M^-) = .75(139.5\ k\text{-}ft) = 104.6\ k\text{-}ft$

$M^+_{col} = .60(M^+) = .6(74.4\ k\text{-}ft) = 44.9\ k\text{-}ft$

- Middle Strip Moments

$M^-_{mid} = .25(M^-) = (.25)(139.5) = 42.25\ k\text{-}ft$

$M^+_{mid} = .4(M^+) = (.4)(74.4) = 36.4\ k\text{-}ft$

- for column strip

$b = 86.04 \quad h = 8'' \ , \ d = 8'' - .5'' - .25'' = 7.25$

(18) #4 @ 10" O.C. (bottom)

$A_s = (18)(.2) = 3.6$

$a = \frac{A_s f_y}{.85 f'_c b} \quad \frac{(3.6)(60)}{.85(5)(86.04)} = .591$

$c = a/\beta_1 = \frac{.591}{.8} = .739$

$\varepsilon_y = \frac{\varepsilon_c}{c}(d-c) \ , \ \varepsilon_y = \frac{.003}{.738}(7.25 - .739)$

$\varepsilon_y > .005$ use $\phi = .9$

DL | Slab check | TECH 1 pg 35

$\phi M_N = .9(3.6)(60)(7.25 - \frac{.738}{2}) = 111.5^{k\text{-}ft}$

$\phi M_N^- = 115^{k\text{-}ft} >, M^-_{col} = 104.6^{k\text{-}ft}$ ok ✓

- using same reinforcement (positive reinforcement)
  (18) #4 (bottom & top)
  $\phi M_N = 111.5^{k\text{-}ft} > M^+_{col} = 44.9^{k\text{-}ft}$ ok ✓

- for middle strip
  * parameters same as above
  $A_s = (9)(.2) = 1.8$
  $AS = \frac{(1.8)(60)}{(.85)(5)(86.04)} = .295$, $c = \frac{.295}{.8} = .369$
  $\varepsilon_y = \frac{.003}{.369}(7.25 - .369)$, $\varepsilon_y > .005$, use $\phi = .9$
  $\phi M_N = .9(1.8)(60)(7.25 - \frac{.295}{2}) = 57.5^{k\text{-}ft}$
  $\phi M_N = 57.5^{k\text{-}ft} > M^-_{mid} = 42.25^{k\text{-}ft}$ ok ✓

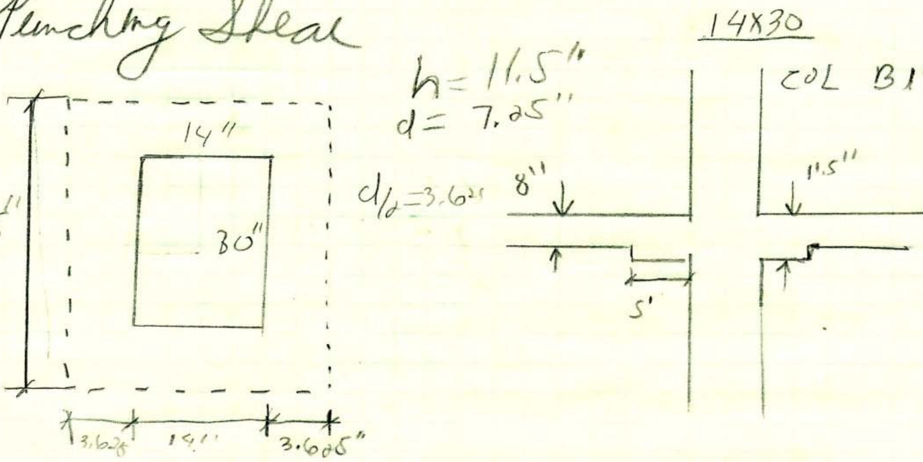
- using same reinforcement (positive reinforcement)
  $\phi M_N = 57.5^{k\text{-}ft} > M^+_{mid} = 36.4^{k\text{-}ft}$ ok ✓

- * Slab is ok for flexure

DL | Slab check | TECH 1 pg 4/5

- per ACI 318-08 Tbl. 9.5c
thickness $> \frac{l_n}{36}$, $t > \frac{279.96}{36} = 7.78''$
- actual slab thickness 8" ok ✓
- no need to check deflection

- Punching Shear



self wt $- 1.2[(\frac{11.5}{12})(150) + 10psf] + 1.6(55psf) = .261 ksf$

$V_u = .261 ksf[(23.33')(14.34') - (\frac{21.25}{12})(\frac{37.25}{12})] = 85.9^k$

$b_o = 2(21.25 + 37.25) = 117''$

$V_c = 4\sqrt{f'c}\, b_o d = 4\sqrt{5000}(117)(7.25) = 239.9^k$

DL | Slab check | TECH 1 pg 5/5

$V_c = \left(2 + \frac{4}{\left(\frac{30}{4}\right)}\right)\sqrt{5000}\,(117)(7.25) = 291.4^k$

$V_c = \left(\frac{(40)(7.25)}{117} + 2\right)\sqrt{5000}\,(117)(7.25) = 268.6^k$

$\phi V_c = .75(239.9) = 179.9 > 85.4^k$ ok ✓

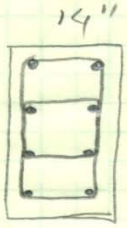
∗ slab ok for punching shear

Dominick Lovallo
Structural Option
Dr. Hanagan Advisor
Penn State University

Embassy Suites Hotel
Springfield, Virginia
Technical Report 1
September 17, 2012

Dominick Lovallo | Column Check | TECH 1 pg 1/2

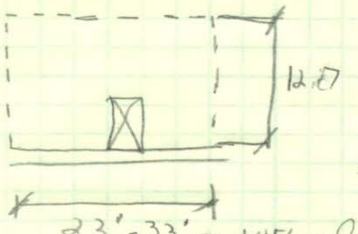
Spot Check - Column 121 EXTERIOR



#3 TIE SPACING 14" O.C.
$f_y = 60{,}000$ psi
(8) #7 BARS

Trib area → $(12.7')(23.33') = 296.3\ ft^2$

* area < 400 ft^2 cannot be reduced



Dead & Live Load

* taking col. 121 @ level 3

FL. to FL. HT.

$\left(\frac{14}{12}\right)\left(\frac{30}{12}\right)(8.45')(150) = 3.7^k$ (COL. S.W.)

slab → $(8''/12)(296.3)(150pcf) = 23.6^k$

MEP → $(5psf)(296.3) = 1.5^k$

CEILING → $(2psf)(296.3) = .59^k$

MISC → $(5psf)(296.3) = 1.5^k$

ROOF SLAB → $(3.25/12)(296.3)(150pcf) = 13^k$

METAL DECK $(4.36psf)(296.3) = 1.3^k$

Roof LIVE → $(30psf)(296.3) = 8.9^k$

Guestroom → $(40psf)(296.3) = 11.9^k$

partitions → $(15psf)(296.3) = 4.4^k$

TOTAL LOAD → $1.2[(27.2)(4\ \text{FLOORS}) + 3.7] + 1.6[(16.3)(4\ \text{FLOORS}) + 8.9] + 1.2[1.3 + 13] = 152.2^k + 118.6^k$

$= 271^k$

DL | Column Check | TECH 1 pg 2/2

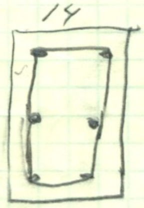
$P_{u\,max} = (.8)(.65)[.85(6)(420\,in^2 - 4.8) + 60(4.8)]$

$P_u = 1250.8^k >> 271^k$

* Columns are over designed due to lateral load

DL | Column Spot check TECH 1 pg 1/2

Column check Item Column



#3 ties spaced @ 14" o.c.

6 #7 bars

Trib. area = (23'.33)(6.14 + 9.65)
= 357 SF

- col s.w. = 3.7^k (*from xtern colcheck)
- * to be conservative LL will not be reduced
- slab → (8/12)(357)(150) = 35.7^k
- MEP → (5psf)(357) = 1.8^k
- ceiling → (2psf)(357) = .71
- MISC → (5psf)(357) = 1.8^k
- ROOF SLABS → (3.25/12)(357)(150) = 14.5^k
- METAL DECK → (4.36)(357) = 1.5^k
- Roof live → (30psf)(357) = 10.7^k
- Guest room → (40psf)(357) = 14.3^k
- Partitions → (15psf)(357) = 5.4^k

TOTAL LOAD → 1.2[(34.8(4 FLOORS) + 3.7)(+14.5+1.5)]
+ 1.6[(19.7)(4 Floors) + 10.7]
212.3 + 143.2 = 355.5^k

DG | Column Spot Check TECH 1 pg 2/2

$P_u\ max = .8(.65)[.85(4)(400 - 3.6) + 60(3.6)]$

$P_u = 848.5 > 355.5^k$ ok for design

AMPAD