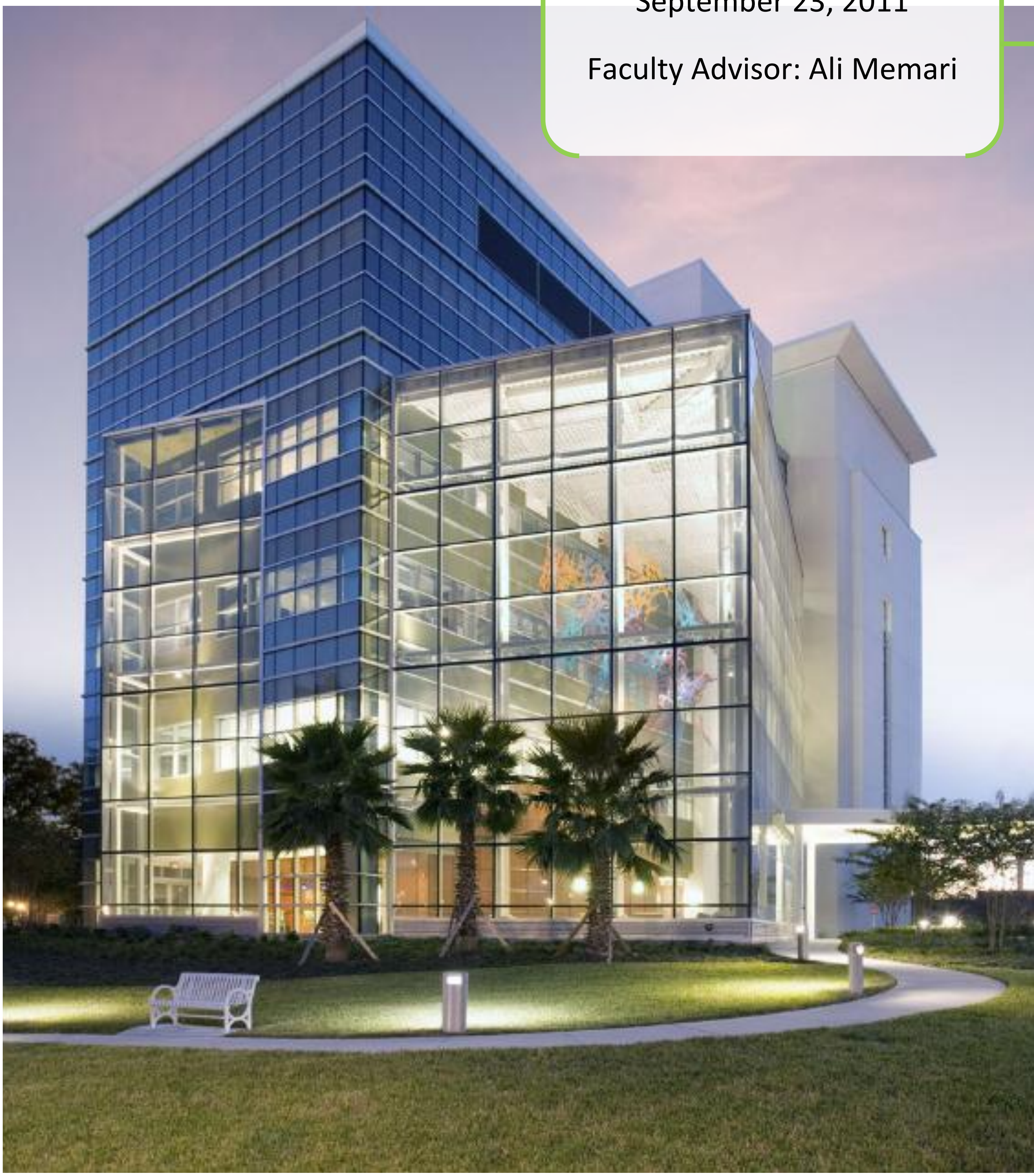
J.B. Byrd Alzheimer's Center & Research Institute

Tampa, Florida

September 23, 2011

Faculty Advisor: Ali Memari



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Executive Summary

The purpose of Structural Existing Conditions Report also known as Technical Report 1 is to gain a knowledge and understanding of the current structure of the J.B.Byrd Alzheimer's Center and Research Center. This is accomplished through descriptions and figures summarizing the foundations, floor systems, framing systems, lateral systems, atrium system and roof systems of the J.B AC&RC. Also, it lists the codes used in design, the materials used in construction, and calculation of gravity and lateral loads. These calculations were rarely compared to the ones on the structural drawings as they were not provided or are not applicable. For example, this structure uses a precast joist and beam soffit system that is provided by a manufacturer in Florida. No nominal strength was calculated by the structural engineer only the ultimate moments were provided. This framing system used makes this structure interesting and difficult to analyze.

Gravity loads were calculated or verified for the building, including the total weight of the structure using simplifying assumptions. These were investigated by spot checks of four gravity members: an interior column, a slab panel, a joist framing system and a beam. The members were chosen as typical as possible to mimic the entire building structure. They were all found to be satisfactory; hence the assumptions made were then verified to a degree.
Lateral load calculations were performed in accordance with ASCE 7-05 procedures. It was found that wind loads will control over seismic by a factor of about 3.6 in the East-West direction and 2.5 in the North-South direction. The design base shear in the North-South direction was calculated to be 682.01 k, and in the East-West direction was calculated to be 892.22k. These loads were not compared to a design base shears as they were not listed on the structural drawings. It was also found that seismic was not required for this region since wind control most of times. Further lateral analysis will be performed in Technical Report 3.

Also included in this technical report are appendices that contain all hand calculations performed on the structure, typical drawings and elevations that were useful to this technical report.

Building Introduction

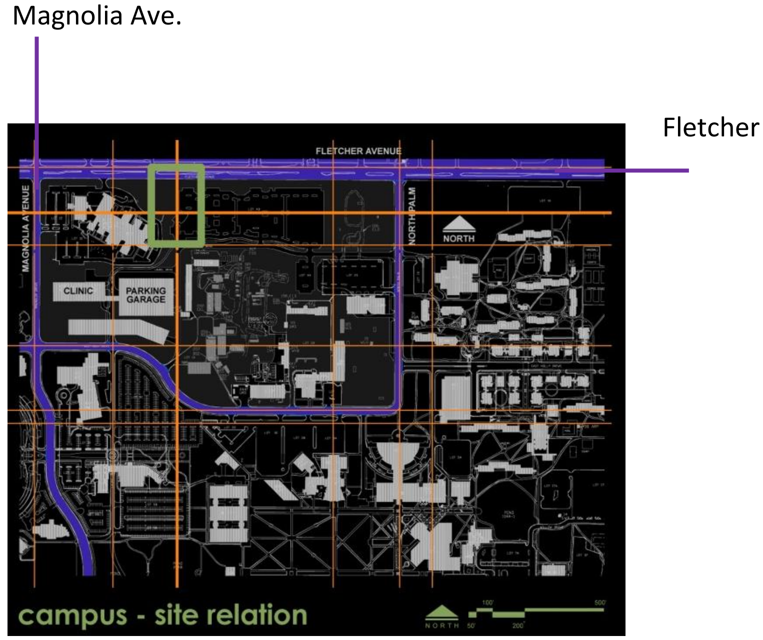


Figure 1- **Site Location on campus of USF**

The Johnnie B. Byrd, Sr. Alzheimer's Center & Research Institute or J.B Alzheimer's center is located in Tampa, Hillsborough, Florida in the University of South Florida's campus. It's located on the intersection of Fletcher Avenue and Magnolia Avenue (See Figure 1). Its occupant is the University of South Florida and it is a business occupancy used for offices and research facility. In fact, after its construction the Florida Alzheimer's center and Research facility became one of the largest freestanding facilities of its type in the world specifically devoted to this illness. It is designed to primarily function as a research unit with labs, a hub for clinic trials, and a data collection center for all Alzheimer facilities throughout the state of Florida. It is built on a 2.6 acres site and the size of the building is 108,054sqft, gross. It is 9 stories including a basement totally a height 106'10". The actual building cost was $23,602,477. It has been LEED silver accredited after construction. From start to finish the construction dates were from February 7, 2006 to July 9, 2007 hence about a year and a half.

The Owner/Client of the project is Johnnie B. Byrd Alzheimer's Center & Research Institute. They chose to have Ruyle, Masters, Hayes+Jennewein Architects PA as their representative. Since this building resides on campus of USF the agency for this project was USF Facilities Planning & Construction. The General Contractor + CM were Turner Construction Company. Everything else (i.e. Architecture, Structural Engineering, Mechanical & Electrical & Plumbing Engineering, Civil Engineering, Landscape Architecture, Security & Telecom) were handled by HDR Architecture, Inc. This project was delivered to the owner by a design-bid-build method.

The façade of the building is mainly divided into two parts. The east side consist of curtain wall glazing and Aluminum panels. The curtain wall glazing consists of: Clear Tempered, insulating laminated spandrel glass, clear insulating laminated glass, insulated fritted glass 30% silkscreen coverage pattern, insulating fritted glass 50% silkscreen coverage pattern, sunscreens and louvers. The west side consists of cement plaster with the same curtain wall like glazing and decorative grille with louver at the top. As for the roof the use of Thermoplastic Membrane roofing was chosen with ¼"/ft slope with Aluminum parapet for architectural reasons.

Basic construction materials of the building include stone column piers and a spread footing foundation system with below grade footing. The structure is composed of precast joist webs and soffit beam bottoms with concrete shear walls. Exterior walls are constructed of cement plaster and lath on steel stud back up framing. The curtain wall system has a kynar aluminum finish and integrates several glazing types. Mechanical systems include packaged air handlers, on-site chillers, and gas fired boilers.

Structural Overview

Initially, HDR Architecture Inc. structural department had designed this building as a composite system composed of steel beams, flanges, columns and a concrete slab on metal floor deck. They had their system pre-designed with specifics. However, all these ideas got tossed away when the Owner and the Contractor decided to use a more economical and efficient concrete system with precast joist webs and soffit beams. That lasts exists mainly in Florida. Hence, the use of it will be fairly new to others, which add uniqueness to this building and thesis.

The J.B. Byrd Alzheimer's Center & Research Institute rests on spread footings for columns and continuous strip footings for walls as well as a mat slab foundation system. This was advised by Nodarse & Associates, Inc. because the site lies on a potential sinkhole activity. The lower 7 floors utilize a one way concrete slab with precast joist ribs and soffit beam framing system for floor framing with cast in-place columns. Part of level 7 and level 8 (roof) still utilize the same floor framing but with larger spacing as well as concentrated reinforcing bars around roof anchors. The lateral system consists of moment frames with concrete shear walls around the main openings.

The importance factors for all calculations were based on Occupancy category II. This was chosen because the J.B A.C. & R.I. that falls under office building.

Design Codes

According to sheet S001, the original building was designed to comply with the following major codes:

- 2001 Florida Building Code with 2003 updates
- 2001 Florida Building Mechanical Code with 2003 updates
- 2001 Florida Building Plumbing Code with 2003 updates
- 2001 Florida Building Fuel Gas Code with 2003 updates
- 2001 Florida Building Accessibility Code as Ch.11 and Energy Code as Ch.13
- 2000 National Fire Protection Association.
- Building code requirements for reinforced concrete (ACI 318)
- AISC Manual of Steel Construction, Allowable Stress Design 9th ED.
- AISC Manual of Steel Construction, Load Resistance Factor Design (LRFD) 1st ED.
- American Welding Society (AWS), D1.1, D1.3, D1.4
- Minimum Design Loads for Buildings and Other Structures (ASCE 7-98)
- Masonry Construction for Buildings (ACI 530-99 AND ACI 530.1-99)

These are also the codes used to complete this technical report:

- Minimum Design Loads for Buildings and Other Structures (ASCE 7-05)
- Building code requirements for reinforced concrete (ACI 318-08)
- 2006 International Building Code (IBC 2006)

Materials Used

Various materials were used on the structure of this project. Below are the main materials derived from Sheet S-001 (see Appendix D).

Concrete		
Usage	Weight	Strength (psi)
Spread footing	Normal	3000
Mat slab foundation	Normal	3000
Precast Joist Webs and soffit beams	Normal	5000
Cast-in-place slab	Normal	4000
Columns, typical	Normal	4000
Columns, as noted	Normal	6000
Precast Masonry Lintels	Normal	5000
Housekeeping Pads	Normal	4000
General Structure Concrete	Normal	4000
Note: Normal weight concrete is at 28 day compressive strength		

Steel		
Usage	Standard	Grade
Reinforcing Steel	ASTM A615	60
Reinforcing Steel (welded)	ASTM A706	60
Welded Wire Fabric	ASTM A185	70
Prestressing Tendons	ASTM A416	270
Wide Flange, S and Tee shapes	ASTM A992	50
Angles Channels and Plates	ASTM A36	36
Tubes	ASTM A500 B	46
Pipes	ASTM A53 B	35
Bolts	ASTM A325	36
Glavanized Roof deck	ASTM A653	33
Note: Welding Electrodes used were E70XX		

Masonary		
Usage	Standard	Strength (psi)
Concrete Masonary Units	ASTM C-90	f'_m= 1500
Mortar	ASTM C270, M	f'c= 2500
Mortar	ASTM C270, S	f'c= 1800
Grout	ASTM C476	f'c= 3000
Joint Reinforcement	ASTM A82, Truss Type	

Figure 2 - **Material Used in building: Concrete, Steel, Masonary**

Foundations

Nodarse & Associates, Inc prepared a report of Preliminary Geotechnical Exploration for this project. The subsurface exploration consisted of a Ground Penetrating Radar (GPR) survey on the site and eight Standard Penetration Test (SPT) borings to depths of 50 to 75 feet below existing site grades.

The borings encountered a relatively uniform subsurface profile consisting of the following respectively with depths: clean sands, medium dense clayey sands, very soft to stiff clays, and weathered to very hard limestone formation. There are indicators in the borings that correlate with the increased risk for sinkhole occurrence. These indicators consist of very soft soils or possibly voids. They estimated that sinkhole could range at the ground level from 10 to 25 feet across. A deep foundation system was not recommended due to the possibility of damage to other adjacent structures from pile-driving vibrations. Also, a cast-in-place deep foundations such as auger cast piles or drilled shafts are not recommended because the presence of joints, fissures, soft zones, and voids within the limestone formation and overburden soils will result in excessive overages of concrete and the need for permanent steel casing. In addition, The University of South Florida expressed concerns about this method as there is the potential of water contamination.

Hence, Nodarse & Associates, Inc recommended, based on their findings the use of a vibro-flotation/stone columns to improve soil conditions so that the building can be supported on a shallow foundation system (see figure 3). The vibrating probe is intended to pre-collapse potential sinkholes to reduce the possibility of future development. After the dry bottom stone columns (42" +/-diameter) were completed, footings were designed on a maximum allowable bearing pressure of 6,000psf. The allowable soil bearing capacity is 10,000 psf after soil improvement. Minimum footing widths for columns and wall footings of 36 and 24 inches respectively were used. Footings bear at least 36 inches below finished floor elevations to provide adequate confinement of bearing soils.

The ground water on this project site appears to be below a basement depth of 10 feet below existing grade, making a basement acceptable. Retaining Walls were also designed using a maximum allowable bearing pressure of 2,000 psi.

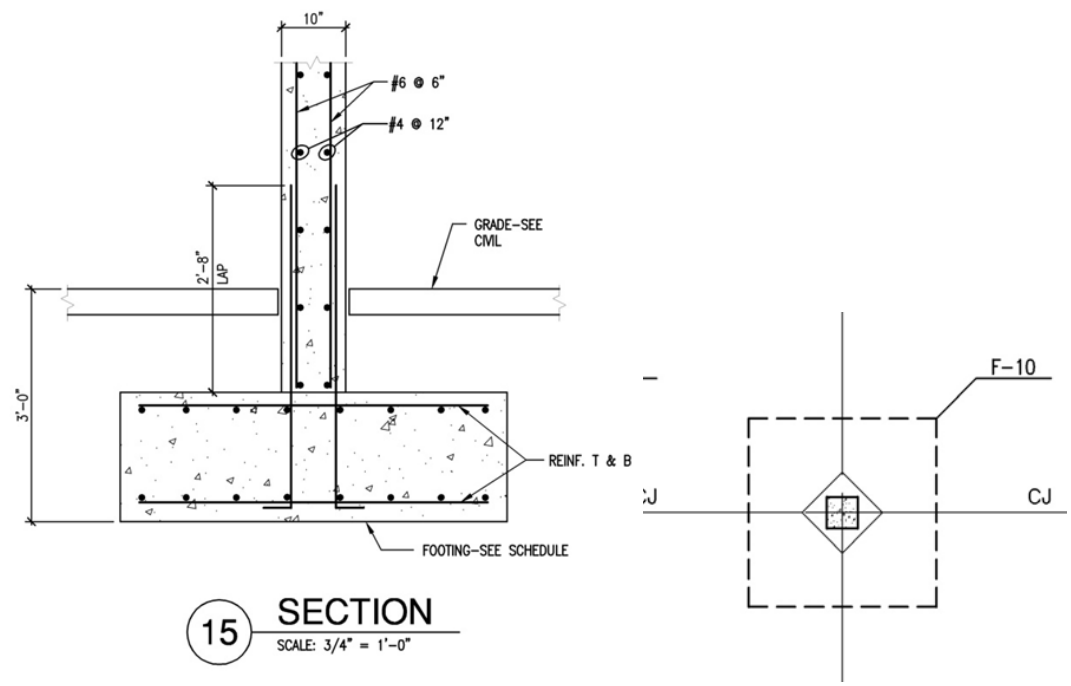


Figure 3- **Foundation section and plan showing footing-column connection and size**

Floor Systems

Even though this building is very architectural and seems like an irregular shape building with a complicated structure it can be divided into 4 simple sections. The sections also correspond to the different uses of the building. Figure 4 shows a typical floor plan with the different bay sizes highlighted with different colors.

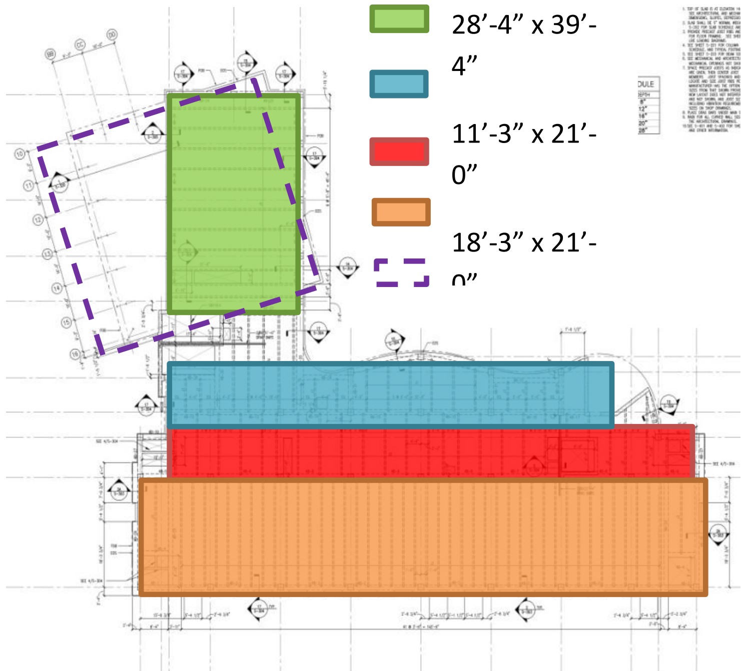


Figure 4- **Floor plan showing different bay sizes**

All the elevated floors of the J.B AC&RI are a hybrid system consisting of a precast joist ribs and soffit beam framing system with cast-in-place to unite the system. In fact, there are 5 main joists that have respectively the following depths: 8", 12', 16", 20", and 28". The entire precast joists and beam soffits are brought on site and lifted to the positions using scaffolding and then they are tied to the structure. Once the structure is erected, the formwork and the rebar reinforcing (if needed) are done then further a 5" concrete slab is casted in place to unite the system (see figure 6). As stated before, 5 different joist depths were used adequately depending on the required spans and uses. For the approximately 40' span, a 20" or J4 was used spaced at 5'-8". That area, corresponding to the green rectangle in figure 4 is typically an office area. For the orange rectangle, where the research labs reside, a J3 or 16" spaced at 5-6" was used for a span of 31'. However in the same area, J4 or 20" spaced at 3'-6" and J5 or 28" at 3'-2" were used to accommodate the PET scans and MRI components respectively (see figure 5).

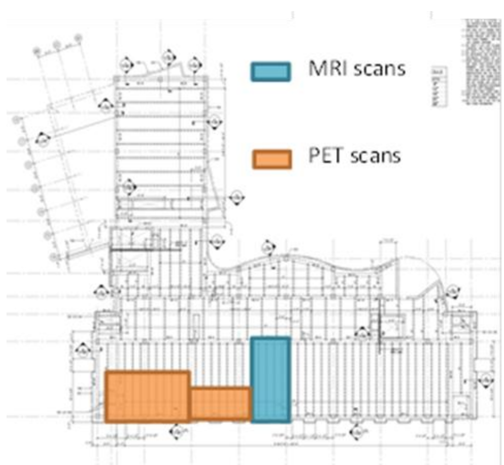
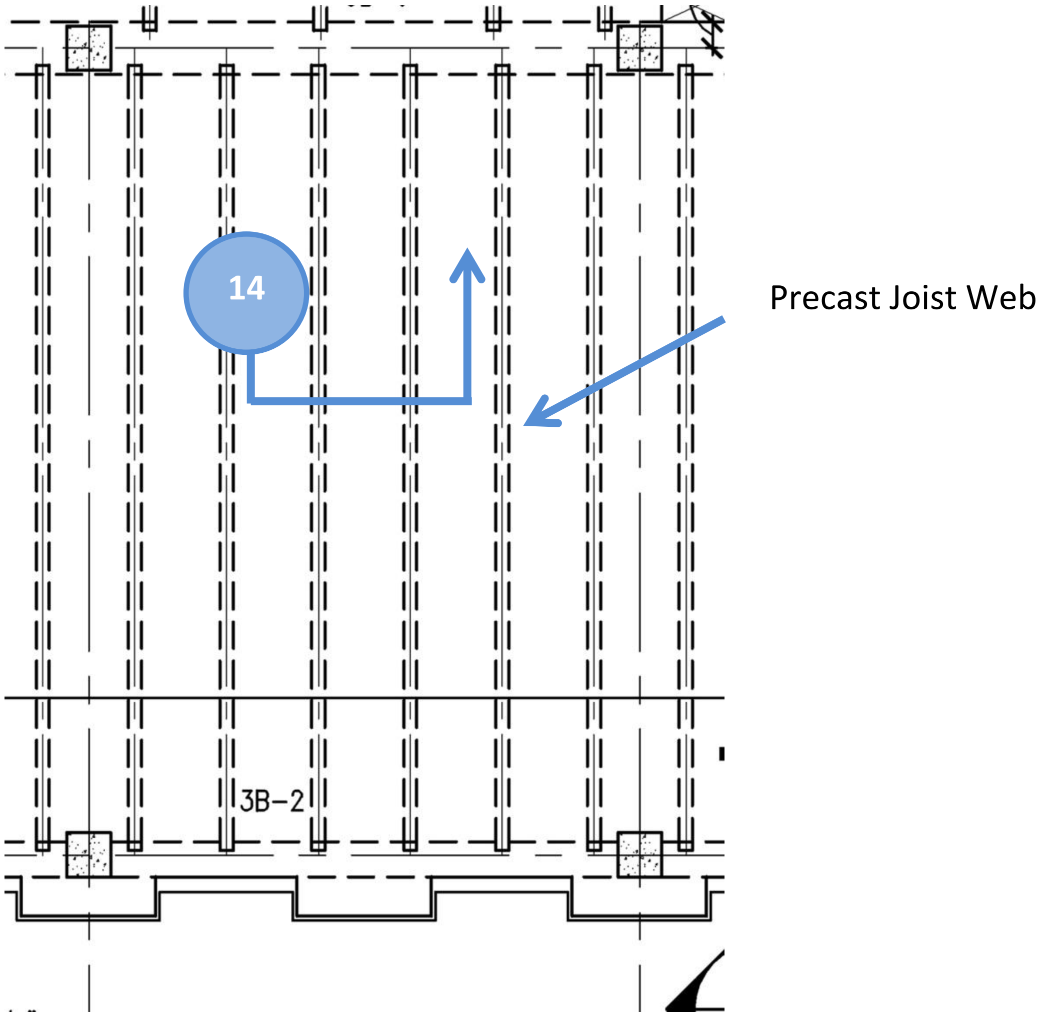


Figure 5- **2nd level floor plan showing MRI/PET scan location**



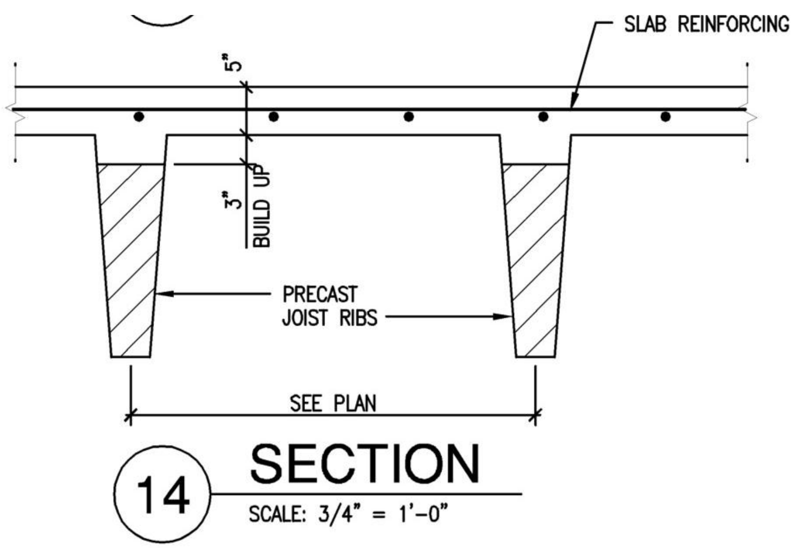


Figure 6- **Plan and section of precast joists**

Framing System

The columns in the lower 7 stories are all cast-in-place concrete. Most of the columns are square and have 4,000psi strength. However, the columns supporting the research labs where the heavy equipment exists and vibration criteria need to be attained a 6,000psi concrete columns were used at the basement and the first floor (see figure 7). All columns are about 20"x20" with reinforcing ranging from 4 to 8 bars except for a few exception that are 20"x30" with 16 bars.

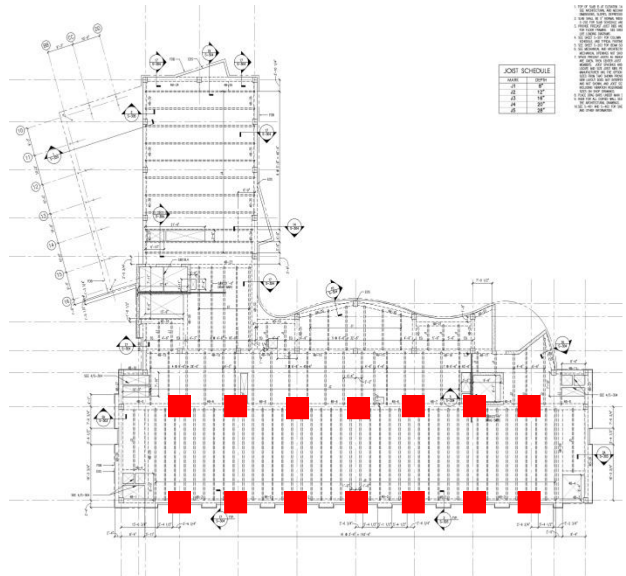


Figure 7- Floor plan showing the 6,000 psi column in basement and 1 floor

Lateral System

The lateral system is composed of concrete shear walls and moment frames. The shear walls are around the main vertical circulation at both ends of the building (see figure 8). They resist the N-S direction as well as E-W direction for best result and little torsion. All of these walls are cast-in-place and are 12" thick. All of them span from basement to the roof. They are anchored at the base by a mat slab foundation that is 3'-0" thick. An issue not investigated by this report is how much the moment frame resists the loading compared to the shear walls when loaded in both directions.

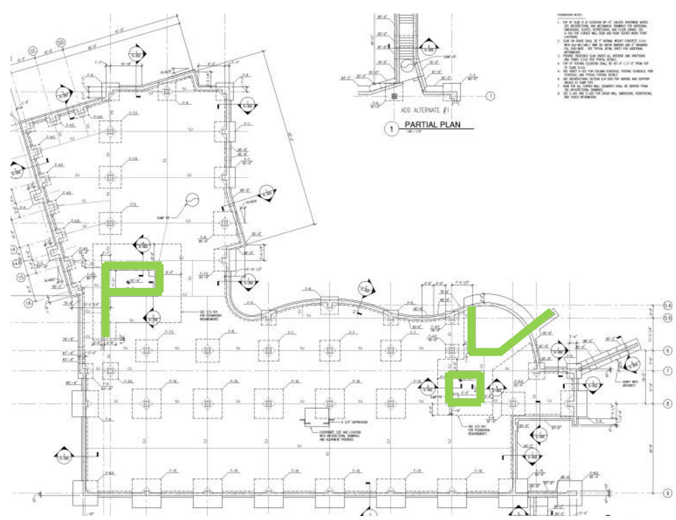


Figure 8- Floor plan showing shear walls

Roof Systems

There are two different roof levels: one on the seventh floor and the other on the mechanical level on top of that (See Figure 9). The figure shows a height from level 1 that starts at 100'0" but for simplicity only the true height is shown.

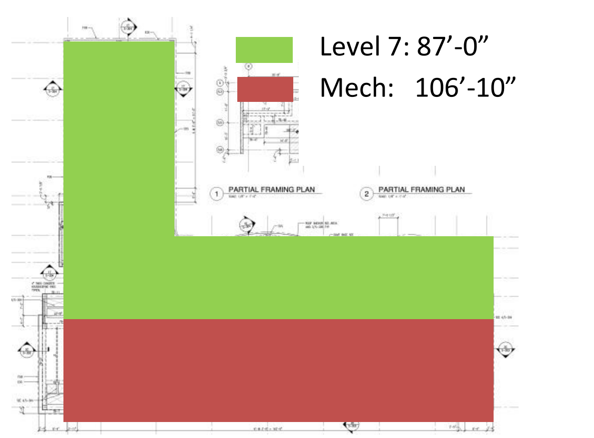


Figure 9- Showing the different roof levels on the building

This two roof structure consists of the same material and system as the floor system as they hold a great deal of load (mainly mechanical that include packaged air handlers, on-site chillers, and gas fired boilers). However, the slabs were heavily reinforced around the roof anchors. Level 7 has joist spacing of 5'8" in the green section and 3'6" under the red section. On the mechanical level a spacing of 5'-6" is used as loads are minimal. There is also the roof of the atrium cube that is not shown on this figure. That last is at height of 153'-9"and consists of trusses, angles, C shape and HSS bars. In addition to the atrium roof, a canopy at the entrance hangs at a height of 114'-6" and consists of W shape with a 1½" 18 Gage galvanized metal roof deck.

Atrium Wall Framing / Floor vibration Criteria

The atrium roof is approximately 60 feet above grade. Architectural trusses, approximately 36" deep are designed to support the exterior storefront glazing spanning this 60 feet. The trusses are designed to minimize deflections from hurricane force winds on this wall. The design wind speed for the area is 120mph which yields that the 50'- 60' range was designed at 31.3 PSF. Truss components are made from structural tubes (ASTM A500, Grade B of Fy= 46Ksi) and pipes (ASTM A53,Grade B Fy= 35Ksi) in this highly visible part of the building.

The vibration control design interfaces with the design of structural, mechanical, architectural, and electrical systems in such a way that those systems do not generate or propagate vibrations detrimental to research activities of the Florida Alzheimer's Center & Research. Vibration criteria have been developed based upon examination of vibration requirements of planned or hypothetical equipment. General labs make up the research facility, and the structure will be designed for vibration amplitude of 2000-4000 µin/s. This accommodates bench microscopes at up to 400x magnification.

Gravity Loads

Part of this technical report, dead and live loads were calculated and compared to the loads listed on the structural drawings. Snow loads however were not applicable for this project as this building exists in Tampa, Florida. Several gravity member checks were conducted. The comparisons were made by how the loads came close to the nominal strength of the members (80-85%) as opposed to information or hand calculations provided. Detailed calculations for these gravity member checks can be found in Appendix A.

Dead and Live Loads

The structural drawing S001 lists the superimposed dead loads to be used. That last is summarized in figure 10. The SP for Ceilings, lighting, plumbing, fire protection, flooring, and

HVAC for roof over mechanical levels is higher than usual because all the mechanical system that supplies the research labs that require special feed are situated in that area. These systems include packaged air handlers, on-site chillers, and gas fired boilers.

Also considered in the building weight calculation were the weights of the columns, shear walls, roofs, wall loads, precast joists and soffit beams.

SuperImposed dead loads	
Description	Load
Ceilings, lighting,plumbing, fire protection,flooring,and HVAC all	14 psf
Ceilings, lighting,plumbing, fire protection,flooring,and HVAC for roof over mechanical levels	40 psf
except mechanical	20 psf
allowance for roofing system	20 psf

Figure 10- **Superimposed Dead load on S-001**

The live loads listed below (figure 11) taken from S001 were compared to the live loads in Table 4-1 in ASCE 7-05 based on the usage of the spaces. The result came out to be the same or more than the expected minimum allowed by the code.

There was nothing about Alzheimer research labs or research labs in general hence the provision “Hospitals- Operating Rooms, Laboratories” was used for comparison. The same was done for high density file storage but with the use of two provisions one is based on "Storage-light/heavy" and the other is based on “Libraries-Stack rooms”. Both were in the range or more than the one designed with. The different live loads on each floor are on drawings S-002 and S-003 found in Appendix A. That last shows on the second level where the MRI and the PET scanner are located special loading was used. A 34kips MRI load distributed to 4 legs then each leg load to 2 joists spaced at 7’-6” apart, center in depression. Also, an 11k scanner load was considered as well as the access path to both the PET and MRI equipment.

One of the last discrepancies, the loadings on S-002 and S-003 are different than the ones stated in the table below. That is due to allow a more flexible building, more stable floors for the vibration and to take into effect the live load reductions.

Floor live loads may be reduced in accordance with the following previsions:

- For live loads not exceeding 100psf for any structural member supporting 150 sq ft or more may be reduced at the rate of 0.08% per sq ft of the area supported. Such

reduction shall not exceed 40% for horizontal members, 60% for vertical members, nor R as determined by the following formula:
R= 23.1 (1+D/L) where D=dead load and L=live load

- A reduction shall not be permitted when the live load exceeds 100psf except that the design live load for columns may be reduced by 20%.

Live Loads			
Area of the building considered	Design Load	ASCE 7-05 Live	Notes
Labratories	125psf	60 psf	Based on "Hospitals-Laboratories"
Offices	50 psf	50 psf	Based on "Office Bldg.-Offices"
Corridors, first floor	100 psf	100 psf	Based on "Office Bldg.-Corridors"
Corridors, above first floor	80 psf	80 psf	Based on "Office Bldg.-Corridors above"
Lobbies	100 psf	100 psf	Based on "Lobbies"
Storage areas	125 psf	125-250 psf	Based on "Storage- light/heavy"
High density file storage	200 psf	125-250 psf	
Mechanical spaces	150 psf	N/A	
Stairs	100 psf	100 psf	Based on "Stairs
Roof	20 psf	20 psf	Based on "Roof- Sloped"

Figure 11- **Live Load comparison to ASCE 7-05**

Snow Loads

No snow load was applicable for this project as it is located in Tampa, Florida. From this following figure 12 taken from ASCE 7-05, the ground snow loads equal zero lb/ft2.

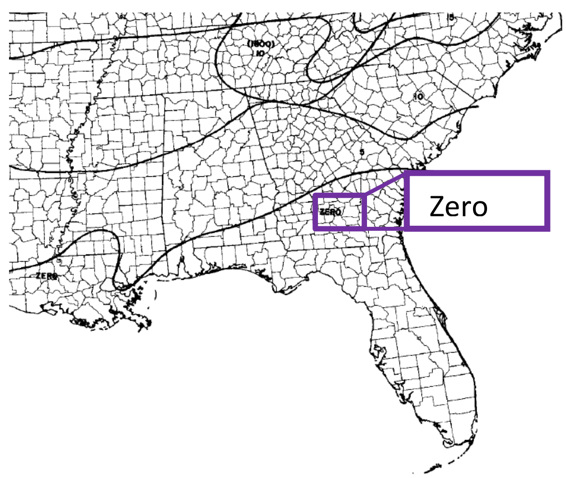


FIGURE 7-1 (continued) GROUND SNOW LOADS, p_g, FOR THE UNITED STATES (LB/FT^2)

Figure 12- **Diagram showing the ground snow load for Florida**

Column Gravity Check

The column I-8 was chosen to column gravity check. This column was chosen because it is an interior column not located near a shear wall see figure x in Appendix A. As the columns are not a part of the lateral force resisting system, lateral influences are unlikely to be a significant concern for this column, and subsequently second order effects were disregarded in this calculation. It is a 20"x20" concrete column with reinforcing changing throughout the levels as well as the concrete strength. In fact, the basement and the first floor both have strength of 6,000 psi and then changes to 4,000psi for the upper levels. It had an area of 441sq.ft. The dead loads on that area were calculated appropriately to each level and the live loads were taken from S-002 and S-003 and reduced according to the provisions in ASCE 7-05. They were chosen from the drawings instead of the ASCE 7-05 because they exceeded the minimum that the code asks for and the results were better to compare. The final check was performed at the basement level.

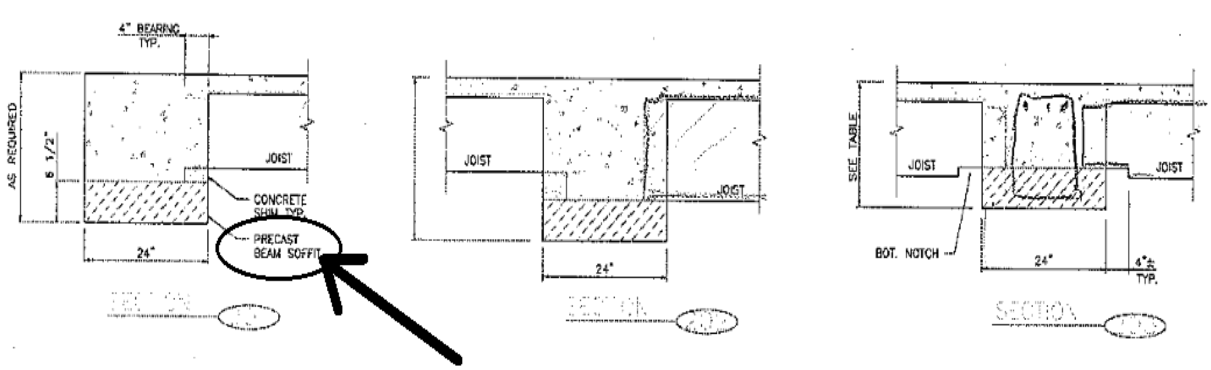
It was found that Column I-8 meet the required strength to carry the associated gravity loads. The design live loads were used as opposed to the ASCE 7-05 live loads for comparison purposes. The phi Pn calculated came to 1,641kips more than the Mu calculated of 1,337 kips. Thus the column passed with 0.815 of its capacity being used. Finally, the column meet the reinforcement ratio required according to ACI 318-08 section 10.9.1.

Beam and Joist Gravity Check

In the interest of doing a beam check, first a joist calculation was made to obtain the same size or close size as the drawing (see appendix A). The way the spot checks for the beam and joist were made is different than usual since a new precast joist and soffit beam was used on this building. This required to get the superimposed load then checked with the manufacturer's tables to choose the right joist size and spacing depending on the span. To see one of those tables go to page 35. The bay between G and H and 8 and 9 is chosen in this calculation. The loads applied were appropriate to those on the drawings. The load found was using ASD of 221.5 psf then compared to the right span in the table of 31' it was found that a Joist J3 or 16" deep at 3'-6" would suffice to carry the loads on it.

After finding the right joist size, a beam check was then in order. The beam spanning between G and H on column line 8 was chosen for this report or 5B-6. This beam spans 21'-0" and has different tributary area on each side since the bays are not uniform. The beam was designed with ACI moment coefficient since it is continuous. Checks were performed for positive moment, negative moments on both sides and shear. The supports at G and H are interior supports hence the negative moment is the same on both sides. The nominal moments as well as deflections were not computed as the manufacturer does not provide the steel areas or steel details for the precast beam soffit.

In fact, the precast manufacturer provides a block of precast concrete with the bottom reinforcing in it (it is draped pre-stressed strands also that's what they use in the precast joist webs) and casts the upper part of the beam with the floor slab (See figure 13).



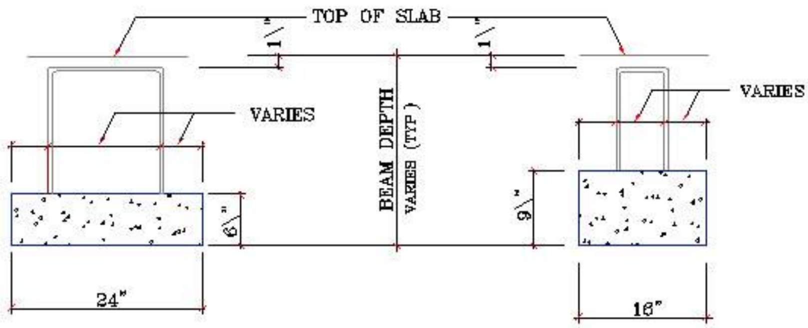


Figure 13- **Beam soffit details showing the precast and cast-in-place part**

The precast joist webs bear on this precast piece of the soffit beam so that the web is self-supporting and does not need to be shored (a cost savings). The precast manufacturer designs the bottom reinforcing based upon the moment calculated by the engineer, and then mild steel top reinforcing is placed and cast based upon the scheduled quantities provided by the engineers. Talking to the engineer the following remarks were made: "When looking at the schedule keep two things in mind. First, we may increase the moment (Mu) by 10% plus or minus, as a safety issue for us since we can't control what a the precast manufacturer actually does in his shop (i.e. I never recommend putting the exact calculated amount of reinforcing steel in a beam, but add a little extra because the steel NEVER gets placed exactly where your calculations say it should go.)" This is also stated in the notes of the schedule see figure 14.

11. PRECAST BEAM SOFFITS SHALL BE DESIGNED BY THE PRECAST MANUFACTURER FOR THE MOMENT GIVEN IN SCHEDULE. PROVIDE STIRRUPS BASED UPON SHEAR GIVEN IN SCHEDULE.

Figure 14- **Note from beam soffit schedule showing the responsibility of the precast manufacturer**

Thus, this is the reason why the deflection and the nominal moment were not calculated. However, the positive ultimate moment calculated was 182.1 k-ft with an increase of 10% as the engineer stated that number comes to 200.31. If we compare that number to that of the schedule 205k-ft (see figure 15) we get a minor discrepancy of 2.29% that could be caused to rounding throughout the calculations.

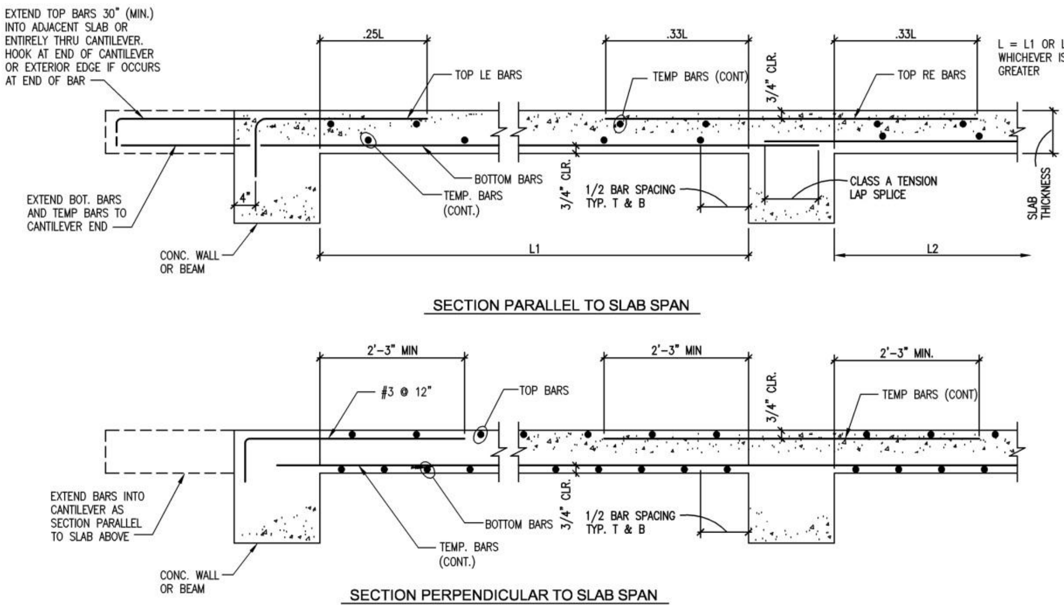
SOFFIT BEAM SCHEDULE

MARK	SIZE		MOMENT	REINFORCING				SHEAR	
	W (in)	D (in)	Mu (ft-kips)	TOP BARS				Vu LEFT(k)	Vu RIGHT(k)
				LE	FLLE	FLRE	RE		
5B-4	16	20	40		2-#6	2-#6		15	15
5B-5	24	24	270	1-#9	2-#9	2-#9	3-#9	95	120
5B-6	24	24	205			2-#9	3-#9	110	105

Figure 15- **soffit beam schedule for %B-6 showing reinforcing, Mu, and Vu.**

Slab Gravity Check

A typical one way slab was chosen to perform the calculation check in the interest that it would be applicable to most areas in the building. This check was done on the same check as the other, on column line G and H running perpendicular to the joists. For checking the minimum thickness, the longest exterior span and the longest interior span was chosen to see (worst case scenario). It turned out that the minimum slab used in the building of 5" was well above the minimum required. It also meets the minimum reinforcing for maximum moment. Those last were computed just like the beam check using ACI moment coefficients on a first interior and a second interior where the maximum moments would occur. Checks were conducted for positive moment capacity, negative moment capacity, and shear. The calculated nominal moment was greater than the Mu computed using the appropriate loads by 17%. The shear strength was also greater with 2:1 ratio.



1 TYPICAL ONE WAY SLAB DETAILS

ONE WAY SLAB NOTES:

1. LEFT AND RIGHT END OF A SLAB MARK SHALL BE DETERMINED AS THE MARK IS BEING VIEWED TO READ ON THE FRAMING PLAN.
2. WHERE SCHEDULE INDICATES 2 SETS OF TOP BARS AT SAME LOCATION (RE OF ONE SPAN AND LE OF ADJACENT SPAN), PROVIDE ONLY THE SET REQUIRING THE GREATEST CROSS-SECTIONAL AREA OF REINFORCING.

ONE WAY SLAB SCHEDULE

SLAB MARK	SLAB THICKNESS	REINFORCING				REMARKS
		BOTTOM	TOP LE	TOP RE	TEMP	
S-1	5	#4@10	#4@10	#4@10	#3@12	TOP BARS CONTINUOUS
S-2	12	#6@6	#4@6	#4@6	#4@12	
S-3	6	#4@6	#4@6	#4@6	#4@12	
S-4	4	#4@12	#4@12	#4@12	#3@12	
S-5	10	#5@6	#5@6	#5@6	#4@6	TOP BARS CONT, TEMP BARS CONT. T & B

Figure 16- One way slab details and schedule

Lateral Loads

In order to better understand the lateral systems, wind loads and seismic loads were calculated for this technical report. At this point in the evaluation of this structure, it is difficult to know exactly how much force is distributed to each shear wall because but simplifying assumptions necessary to be able to perform hand calculations. However, a more extensive analysis of the lateral system will be conducted for Technical Report 3. For Technical Report 1, the hand calculations associated with wind loading and seismic loading can be found in Appendices B and C, respectively.

Wind Loads

Wind loads were calculated with method 2 Main Wind Force Resisting System (MWRFS) procedure identified in ASCE 7-05 Chapter 6. In order to be able to use this procedure, several simplifying assumptions had to be made. First, the building was modeled with a single roof height of 107’. Next, the surface areas were projected onto North-South (N-S) and East-West (E-W) axes, and the projected lengths were used to calculate wind pressures. Using these projected lengths for the calculation of L and B would be conservative. Also, since the new projected shape looks like an L shape, it is assumed that there wouldn’t be a buildup in pressure where the shaded void is in my wind calculations (See appendix B for figure and shape used). Hence, the building will be analyzed like a rectangle.

Most calculations were performed using Microsoft Excel to simplify a potentially repetitive process. Wind pressures, including windward, leeward, sidewall, and internal pressure were found. These were then used to calculate the story forces at each level. It should be noted that the story forces include windward and leeward pressures, but not internal pressure, because internal pressure is effectively self-cancelling.

The wind pressures in the N-S direction are listed and diagramed in Figure 17. These were resolved into wind forces in the N-S direction, which are listed and diagramed in Figure 18. The resulting base shear is 682.01 k.

In addition, the wind pressures in the E-W direction are listed and diagramed in Figure 19. These were resolved into wind forces in the E-W direction, which are listed and diagramed in Figure 20. The resulting base shear is 892.22k. There was nothing to compare it to as there was no base shear provided on the drawings in either directions thus no conclusions can be drawn on discrepancies.

Desgin wind pressure for MWFRS in N-S Direction						
type	Level	Height / distance	qz/ qh	wind pressure (psf)	Net pressure	
					(+)GCPi	(-)GCPi
windward walls	1	0'	21.01	14.29	-6.14	34.72
	2	14'-6"	21.01	14.29	-6.14	34.72
	3	29'	25.51	17.35	-3.08	37.77
	4	43'-6"	28.66	19.49	-0.94	39.92
	5	58'	31.04	21.11	0.68	41.53
	6	72'-6"	33.18	22.56	2.13	42.99
	7	87'	35.06	23.84	3.41	44.27
	Roof	107'	37.14	25.26	4.83	45.68
leeward walls	All	All	37.14	-13.83	-34.25	6.60
sidewalls	All	All	37.14	-22.10	-42.53	-1.67
Roof		0-53.5	37.14	-29.93	-50.35	-9.50
		53.5-107	37.14	-27.65	-48.08	-7.23
		107-214	37.14	-16.54	-36.97	3.88

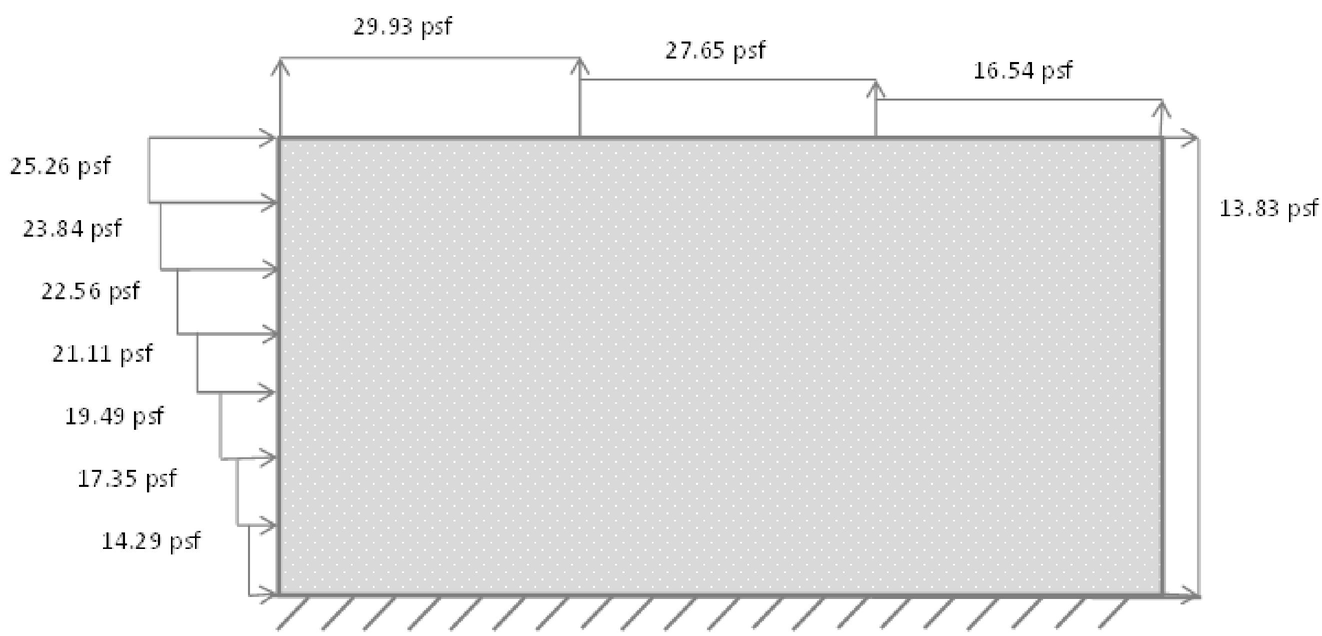


Figure 17- List and diagram showing the wind pressure on the building in N-S direction

Wind Forces- N-S Direction								
Floor level	Height / distance	Tributary below		Tributary above		Story force (K)	Story Shear (K)	Overturning Moment (k-ft)
		Height (ft)	Area (ft^2)	Height (ft)	Area (ft^2)			
1	0'	N/A	0.00	7.50	1095.00	73.49	682.01	0
2	14.5	7.00	1022.00	7.50	1095.00	76.62	608.52	1110.96
3	29	7.00	1022.00	7.50	1095.00	82.16	531.90	2382.56
4	43.5	7.00	1022.00	7.50	1095.00	86.16	449.75	3747.82
5	58	7.00	1022.00	7.50	1095.00	89.41	363.59	5185.96
6	72.5	7.00	1022.00	7.50	1095.00	92.31	274.18	6692.60
7	87	7.00	1022.00	7.50	1095.00	115.17	181.86	10019.62
Roof	107	10.00	1460.00	10.00	1460.00	66.70	66.70	7136.52
							Total base shear=	682.01 k
							Total overturning Moment=	36276.04 k-ft

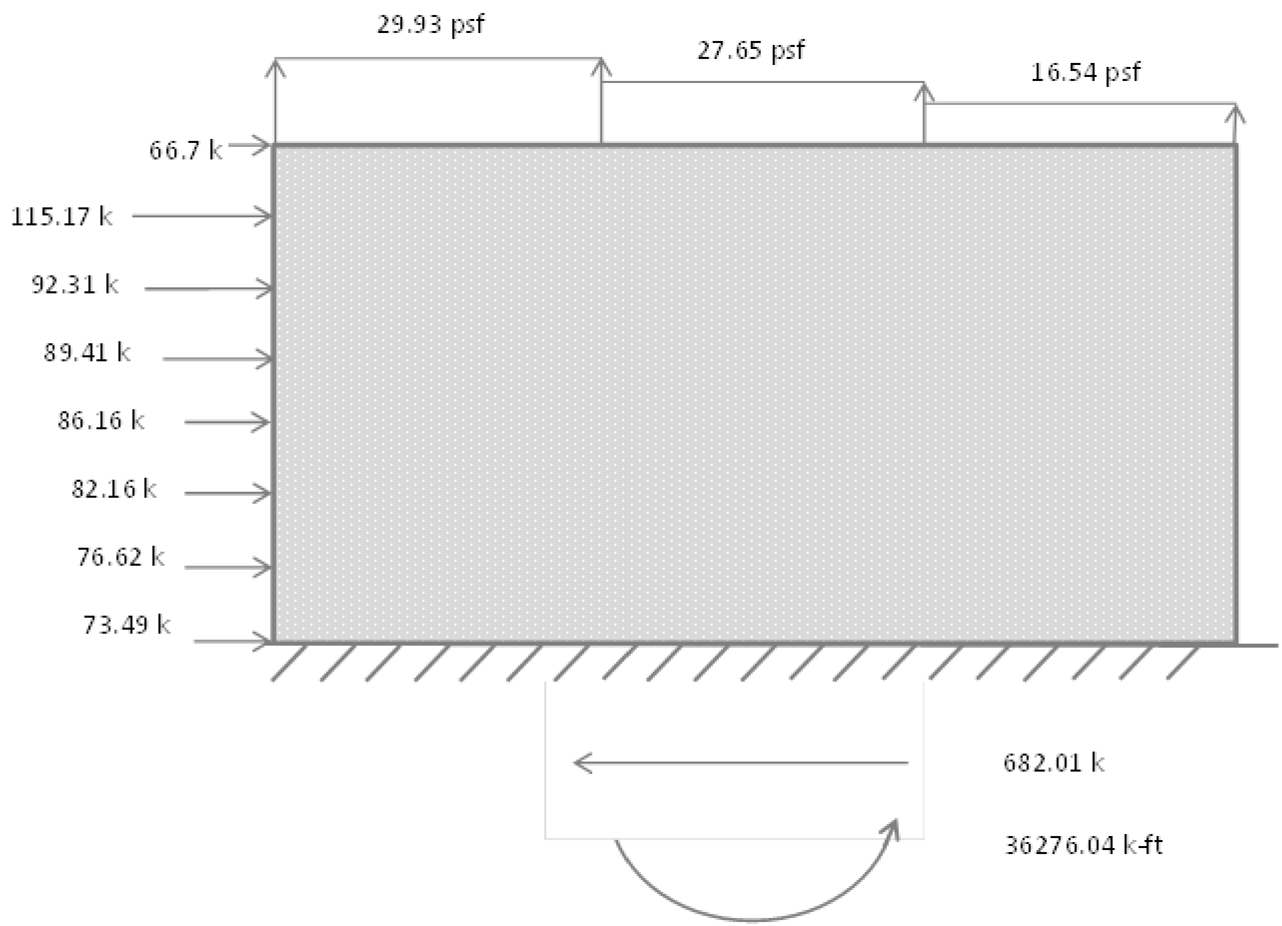


Figure 18- **List and diagram showing the wind forces on the building in the N-S direction**

Desgin wind pressure for MWFRS in E-W Direction						
type	Level	Height / distance	qz/ qh	wind pressure (psf)	Net pressure	
					(+)GCPi	(-)GCPi
windward walls	1	0'	21.01	14.29	-6.14	34.72
	2	14'-6"	21.01	14.29	-6.14	34.72
	3	29'	25.51	17.35	-3.08	37.77
	4	43'-6"	28.66	19.49	-0.94	39.92
	5	58'	31.04	21.11	0.68	41.53
	6	72'-6"	33.18	22.56	2.13	42.99
	7	87'	35.06	23.84	3.41	44.27
	Roof	107'	37.14	25.26	4.83	45.68
leeward walls	All	All	-16.54	-15.78	-36.21	4.64
sidewalls	All	All	37.14	-22.10	-42.53	-1.67
Roof		0-53.5'	37.14	-34.22	-54.65	-13.79
		53.5'-107'	37.14	-25.51	-45.93	-5.08
		107'-214'	37.14	-18.69	-39.12	1.74

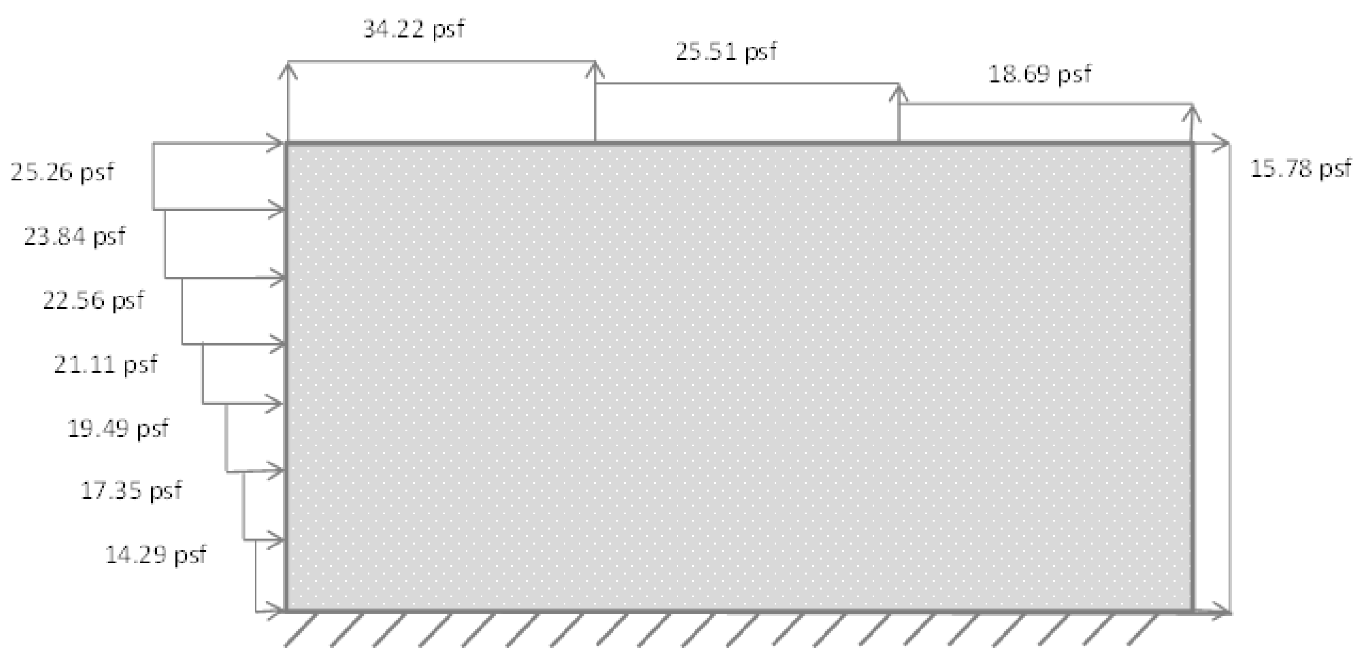


Figure 19-**List and diagram showing the wind pressure on the building in E-W direction**

Wind Forces- E-W Direction								
Floor level	Height / distance	Tributary below		Tributary above		Story force (K)	Story Shear (K)	Overturning Moment (k-ft)
		Height (ft)	Area (ft^2)	Height (ft)	Area (ft^2)			
1	0'	N/A	0.00	7.50	1432.50	96.14	892.22	0
2	14.5	7.00	1337.00	7.50	1432.50	100.23	796.08	1453.38
3	29	7.00	1337.00	7.50	1432.50	107.48	695.85	3116.92
4	43.5	7.00	1337.00	7.50	1432.50	112.71	588.37	4902.97
5	58	7.00	1337.00	7.50	1432.50	116.97	475.65	6784.37
6	72.5	7.00	1337.00	7.50	1432.50	120.76	358.68	8755.38
7	87	7.00	1337.00	7.50	1432.50	150.66	237.92	13107.85
Roof	107	10.00	1910.00	10.00	1910.00	87.25	87.25	9336.14
							Total base shear=	892.22 k
							Total overturning Moment=	47457.01 k-ft

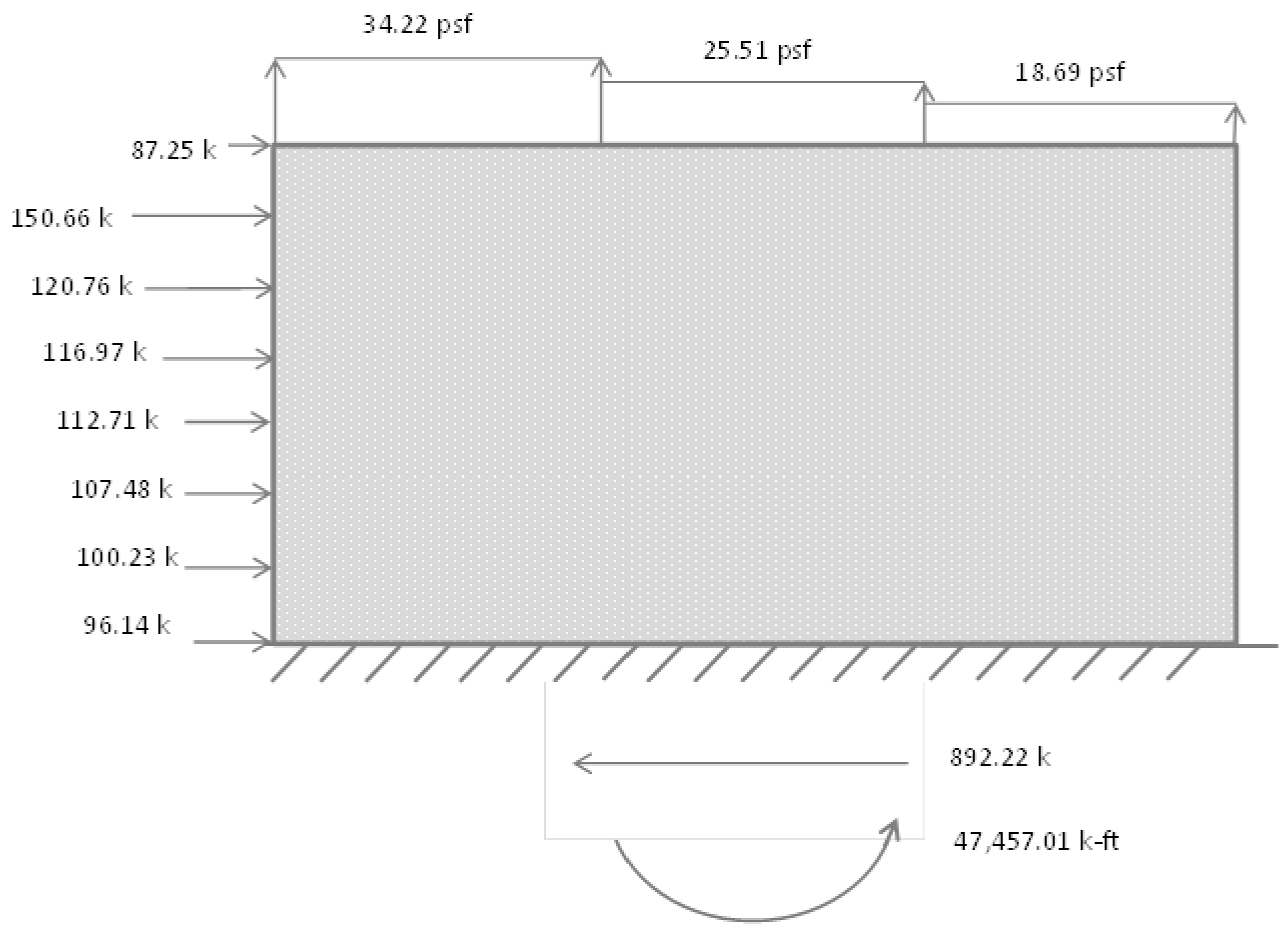


Figure 20- List and diagram showing the wind forces on the building in the E-W direction

Seismic Loads

The engineers who designed this building did not analyze the building for seismic forces as wind always controls in Tampa, Florida. However, Seismic loads were still calculated to check that statement.

Seismic loads were calculated with the Equivalent Lateral Force procedure outlined in Chapters 11 and 12 of ASCE 7-05. This procedure also assumes a simple building footprint. In fact when calculating the weight of the building,3 sections were considered to simplify the different floor joists system used. Also, an average size of beam of 24”x24”was taken to represent all sizes to simplify the calculations of each weight of the beams. Other minor assumptions are made in the calculation, see Appendix A.

The loads from seismic forces originate from the inertia of the structure itself, which is related to the mass of the structure. Most of the mass of the structure is locked in the slabs, beams, joists, and columns which are connected to the shear walls. When seismic loads are generated by a ground motion, the slabs transfer the loads directly into the shear walls, which then carry the loads down to the foundations and therefore to grade.

It was assumed that the site is classified as site class E or stiff soil. After calculating the SMs, and S1, the SD1 and SDM were computed which lead to a design category for this structure A. This means that each lateral force at every floor is the weight of the floor multiplied by 0.01. Seismic forces in the N-S direction are listed and diagramed in Figure 21. The resultant base shear in this direction is 192.99 k and the overturning moment was 10,818.64 k-ft. The calculations cannot be compared as no analysis was done.

Seismic Forces - N-S Direction					
Level	Story weight, w_x	height (ft), h_x	Story force (k) Fx=0.01, w_x	Story Shear (k)	Overturning moment (k-ft)
2nd	2895.38	14.5	28.9538	192.989	419.8301
3rd	2892.82	29	28.9282	164.035	838.9178
4th	2892.82	43.5	28.9282	135.107	1258.3767
5th	2892.82	58	28.9282	106.179	1677.8356
6th	2943.53	72.5	29.4353	77.2507	2134.05925
7th	3133.116	87	31.33116	47.8154	2725.81092
Roof	1648.42	107	16.4842	16.4842	1763.8094
	19298.906		Base Shear =		192.98906
		Total Overturning moment=			10818.63977

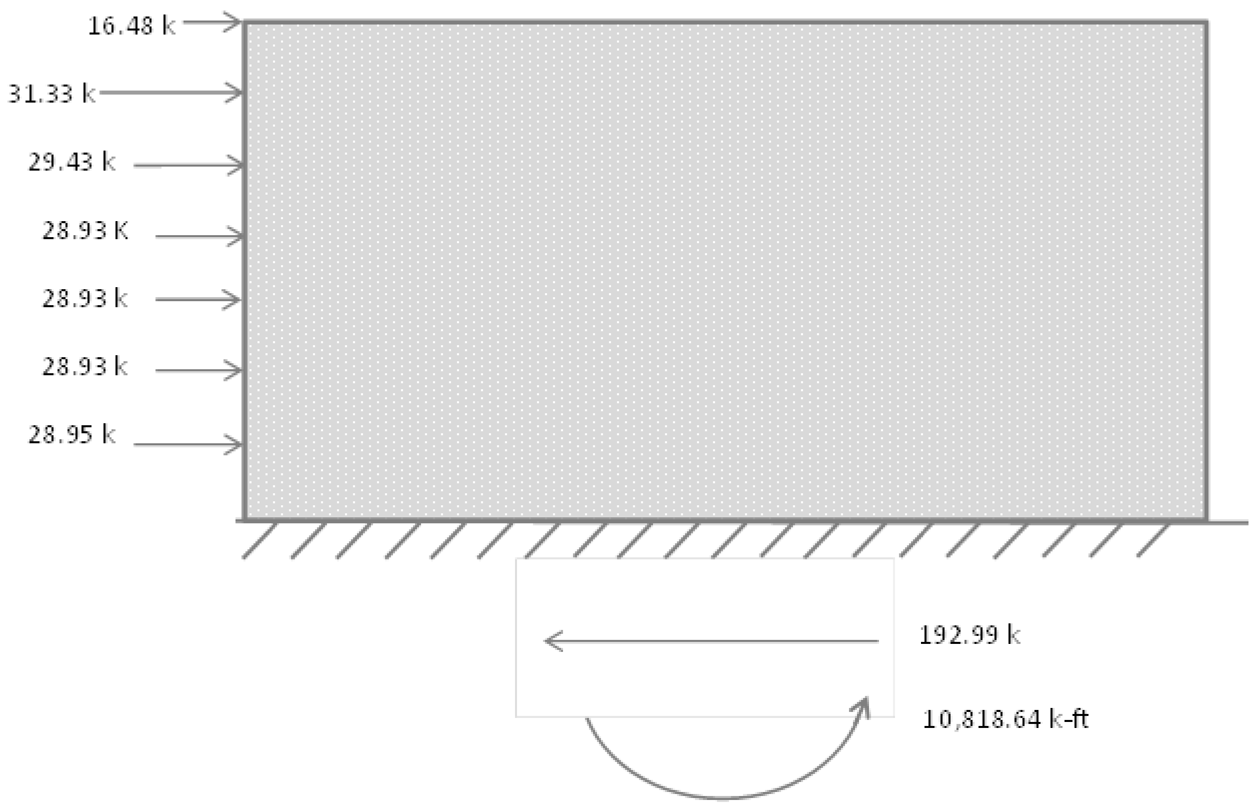


Figure 21 -List and diagram showing the Seismic forces on the building in the N-S direction

Conclusion

Technical Report 1 analyzed the existing structural conditions of the J.B.Byrd Alzheimer's Center & Research Center in Tampa, Florida. A summary of the foundations, floor systems, framing systems, lateral systems and roof systems were fully conducted with figures to describe the structure as it is presently designed.

In addition to the description and use of the systems spots checks of gravity and lateral were conducted. Spot checks for gravity included a joist, a beam, an interior column and a slab. Lateral loads for wind and seismic were conducted even though seismic for Tampa, Florida was not required.

This process relied heavily on information from ASCE 7-05, as well as the structural drawings. The use of precast joists and beam soffit construction makes this structure interesting and unique. Superimposed dead loads and live loads were tabulated and checked for since not the same codes were used at the time when the building was constructed. Discrepancies between these loads and the commonly assumed design loads are explainable. Assumptions were also made regarding calculation for wind, seismic and gravity to simplify the process. Some of those could not be compared to as the information was not provided or not applicable.

Furthermore, for gravity checks it was found that each member was adequate, but the design gravity loads could not be compared to get a margin of error. That last was compensated using the live loads given by drawings instead of those in ASCE 7-05. Thus the strength of each member came close to the designed loads in a margin of 10% or less.

In addition to gravity checks, wind and seismic loads were calculated. Wind loads on this structure were found to control, and no requirement for earthquake. Seismic loads were extremely less than the wind loads: 3.6 times less in the East-West direction and 2.5 times less than the wind loads in the North-South direction. Thus, wind controls the lateral design of this building. This is likely due to the wind speed in that region of 120mph. Both lateral loads could not be compared to any number as the information for both was not provided.

Appendices start on the next page

Appendix A: Gravity Load Calculations

PRESTRESS SYSTEMS OF FLORIDA, INC.

16603 ~~OLD~~ US 41 FORT MYERS, FLORIDA 33912

PHONE: 239-437-0660 **www.psfjoist.com** FAX 239-437-0697

SUPERIMPOSED LOAD CAPACITY

FIRE	DECK	JOIST SIZE	JOIST SPACING	18	20	22	24	26	28	30	32	34	36	38	40	42	44	46	System Weight PSF
2 HOUR	4 3/4"	12"	2'-6"	-	-	-	-	-	-	-	222	202	172	147	138	119	102	87	84
			3'-6"	-	-	-	-	245	201	170	145	120	107	97	86	72	59	-	76
			4'-6"	-	-	-	225	195	162	130	113	94	78	63	52	-	-	-	72
			5'-6"	-	-	220	195	160	125	102	86	70	56	-	-	-	-	-	69
			6'-6"	-	217	175	154	125	100	80	69	54	-	-	-	-	-	-	67
		16"	2'-6"	-	-	-	-	-	-	-	-	-	-	245	214	192	173	160	95
			3'-6"	-	-	-	-	-	-	-	-	-	211	180	157	135	118	102	84
			4'-6"	-	-	-	-	-	-	-	219	185	158	133	117	100	86	72	78
			5'-6"	-	-	-	-	-	225	197	174	145	122	103	86	74	60	52	74
			6'-6"	-	-	-	-	241	201	157	138	115	102	85	69	62	50	-	72

Notes: Spans shown are clear (Face-to-face of supports). For design conditions not addressed please contact PSF

Joist table 1/2

PRESTRESS SYSTEMS OF FLORIDA, INC.
16603 OLD US 41 FORT MYERS, FLORIDA 33912
PHONE: 239-437-0660 www.psfjoist.com FAX 239-437-0697

FIRE	DECK	JOIST		SUPERIMPOSED LOAD CAPACITY															System Weight PSF
				SPAN															
		SIZE	SPACING	40	42	44	46	48	50	52	54	56	58	60	62	64	66	68	
2 HOUR	4 3/4"	20"	4'-8"	-	-	209	191	169	154	136	117	103	98	82	-	-	-	-	90
			5'-8"	213	193	169	153	134	123	108	92	80	-	-	-	-	-	-	84
			6'-8"	178	161	139	127	110	100	87	75	-	-	-	-	-	-	-	80
			7'-8"	149	133	115	104	90	80	67	-	-	-	-	-	-	-	-	77
			8'-8"	118	106	90	80	68	-	-	-	-	-	-	-	-	-	-	75
			10'-0"	101	90	76	65	-	-	-	-	-	-	-	-	-	-	-	72
		24"	4'-8"	-	-	-	-	-	218	202	173	168	150	135	125	109	101	87	95
			5'-8"	-	-	-	216	199	177	163	145	134	119	105	98	84	78	67	88
			6'-8"	-	-	206	180	166	147	135	119	110	97	85	78	67	62	50	84
			7'-8"	-	190	173	152	138	122	112	98	90	78	67	57	52	-	-	80
			8'-8"	-	-	139	121	110	95	87	76	66	57	50	-	-	-	-	78
			10'-0"	-	-	-	98	89	80	71	58	54	45	-	-	-	-	-	75
		28"	4'-8"	-	-	-	-	-	-	-	-	219	198	178	166	146	131	118	102
			5'-8"	-	-	-	-	-	-	228	199	178	160	144	133	116	100	90	94
			6'-8"	-	-	-	-	-	209	191	167	149	133	118	109	95	84	72	89
			7'-8"	-	-	-	-	-	176	161	139	124	110	97	86	76	67	59	85
			8'-8"	-	-	-	-	-	142	129	111	97	81	72	66	57	50	-	82
			10'-0"	-	-	-	-	-	-	111	94	82	72	60	52	46	-	-	78

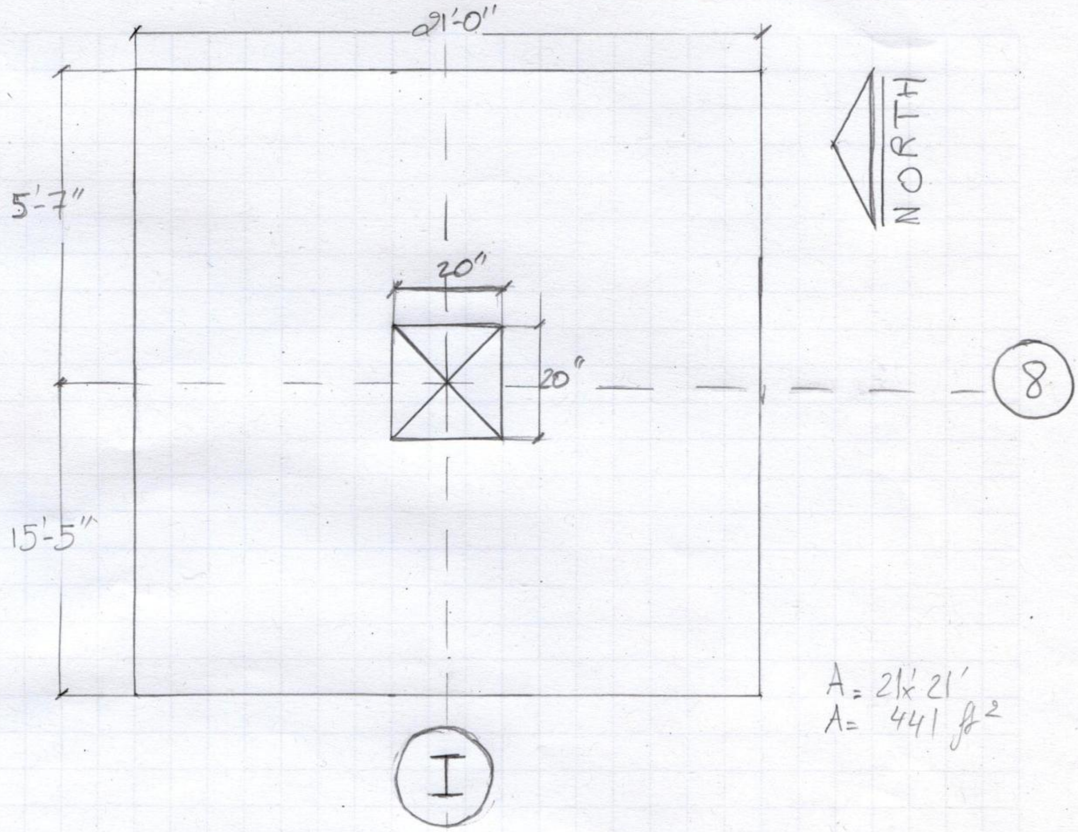
Notes: Spans shown are clear (Face-to-face of supports). For design conditions not addressed please contact PSF

Joist table 2/2

http://psfjoist.com/loadtables4.html 5/25/2004

HDR | ONE COMPANY *Many Solutions*

Project: Tech 1		Computed:	Date:
Subject: Gravity Check		Checked:	Date:
Task: Column		Page: 1	of: 5
Job #:		No:	



PARTIAL PLAN OF COL I-8

NTS

Live loads will be taken from S-002 and S-003 for each floor and will be reduced accordingly

Normal weight of concrete is 150 psf

Joist weights + beam soffits will be taken from tables provided by manufacturer

Column size doesn't change through out the height of building

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Project:		Computed:	Date:
Subject:		Checked:	Date:
Task:	Column Check	Page: 2	of: 5
Job #:		No:	

The height of each floor is 14'-6" except for the roof of 20'-0"

Roof level: D: 5" NW slab
J2 @ 5'-6' on both sides
SD: 20psf Roofing from S001
L_r: 80 psf from S003

K_{LL} for column = 4

$4 \times A_{TT} = 4 \times 441 > 400 ft^2 \Rightarrow$ Live can be reduced

using ASCE 7-05 section 4.8

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right) = 80\left(0.25 + \frac{15}{\sqrt{4 \times 441}}\right)$$

$$L = 48.57 > 0.6 \times 80 = 48 \Rightarrow OK!$$

using provisions made by the engineer for vertical members

D: $\frac{5}{12} \times 150 = 62.5$ psf
J2 @ 5'-6" for deck of 4 3/4", a depth of 12"
weight = 69 psf
SP = 20psf
} = 151.5psf

$P_D = 66.81$ K $\quad P_L = 21.42^K$

level 7 62.5 for slab
SP = 40psf for lighting, plumbing, fire protection and HVAC for Roof over mech.
for 16" deep of J3 @ 3'-6" for long side $\Rightarrow \omega = 84$ psf
for 12" deep of J2 @ 3'-6" for short side $\Rightarrow \omega = 76$ psf

$D_{J2} = (5'-7") \times (21') \times 76 psf = 8.911$ K
$D_{J3} = (15'-5") \times (21') \times 84 psf = 27.195$ K
} 36.106 K

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Project:
Subject: Column check
Task:
Job #:
Computed:
Checked:
Page: 3 of: 5
Date:
Date:
No:

$D = (62.5 + 40)\ 441 = 45.2025$
$+ 36.106$
$P_D = 81.308\,K$

L = 150 psf for I-8 level 7, according to provisions (see live loads in report) design live load for columns may be reduced by 20% ⇒ L = 120 psf

$P_L = 120 \times 441 = 52.92\ K$

checking ASCE 7-05; the same provision is mentioned as long as the column holds two or more floors thus

$L = 120 \times (0.607) = 72.84 \Rightarrow P_L = 32.122\ K$

level 6

62.5 psf × 441 = 27.5625 K
for long side J3 @ 3'-6" ⇒ 27.195 K from level 7
$P_D = 77.61\,K$ ⇐ for short site J2 @ 6'-6" ⇒ 67 psf
(5'-7") × (21') × 67 = 7.855 K
SP = 14 psf from lighting, plumbing, fire protection and HVAC for Roof over all levels and 20 psf for partitions ⇒ $P_{sp} = 34 \times 441 = 14.994\ K$

L = 125 psf ⇒ reduced L = 125 × 0.8 (see level 7 provisions)
L = 100
$P_L = 44.1\ K$

level 5

27.5625 + 14.994 = 42.557 K from (slab + SP)
Joists: J3 @ 3'-6" = 27.195 K long side
J2 @ 6'-6" = 7.855 K

$P_D = 77.61\ K$

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Project:		Computed:	Date:
Subject: Column Check		Checked:	Date:
Task:		Page: 4	of: 5
Job #:		No:	

$P_L = 44.1$ K as $L = 125$ psf same as level 6

level 4 same as level 5 and 6 $P_D = 77.61$ K
$P_L = 44.1$ K

level 3 same as level 4 → $P_D = 77.61$ K
$P_L = 44.1$ K

level 2 same loading as before → $P_D = 77.61$ K
$P_L = 44.1$ K

level 1 D: from slab and SP = 42.557 K
for long side, J3 @ 3'-6" d = 16"
→ w = 84 psf
= 27.195 K (see level 7)

for short side J2 @ 4'-6"
d = 12" w = 72 psf

$72 \times (5'-7'') \times (21') = 8.442$ K

$P_D = 78.194$ K

$P_L = \underline{250} \times 0.8 = 200$ psf ⇒ $P_L = 88.2$ K

2 live loads exist here, a 250psf and 50psf
the 250 was chosen to be more conservative

P_D, loads = 614.362 K $P_L = 340.822$ K $P_{LR} = 21.42$

From columns, $P_D = \left[\frac{20\times20}{144} \times 14.5 \times 150\right] 6 + \left(\frac{20\times20}{144}\right) \times 20 \times 150 = 36.26$ K

$P_u = 1.2(650.622) + 1.6(340.822) + 0.5(21.42)$

$P_u = 1336.77$ K

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Project:		Computed:	Date:
Subject:	Column check.	Checked:	Date:
Task:		Page: 5	of: 5
Job #:		No:	

Checking column reinforcing:

Column is checked for pure compression only:

from the column schedual, column I-8 is 20"x20" and has 16 #10 and a $f'_c = 6000$ psi @ the basement level

$$A_s = 16 \times (1.27) = 20.32 \text{ in}^2 \qquad A_c = (20 \times 20) - 20.32 = 379.68 \text{ in}^2$$

$$\phi P_o = \phi\,(0.85 f'_c A_c + A_s f_y)$$
$$= 0.65\left[(0.85(6)(379.68)) + (20.32)(60)\right]$$

$$\phi P_o = 2{,}051 \text{ K}$$

Must include α for min eccentricity

$$\phi P_n = \alpha\,\phi P_o = 0.8\,(2{,}051) = 1{,}641 \text{ K}$$

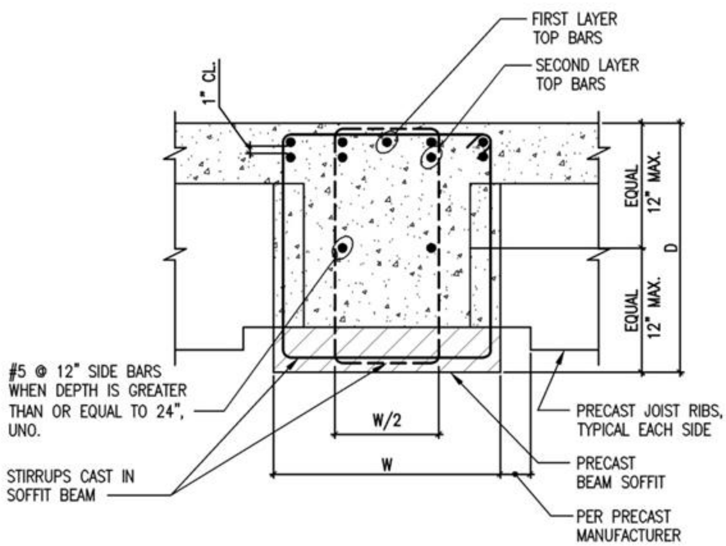
$$\phi P_n > P_u = 1{,}337 \text{ K} \Rightarrow \text{Okay!}$$

Check ρ

$$\rho = \frac{A_s}{(b)\cdot(d)} = \frac{20.32}{20 \times 20} = 0.057 > 0.01 \text{ or } 1\%$$

According to ACI 318-08 section 10.9.1 for minimum allowable ρ.

16" JOIST WITH 3" COMPOSITE SLAB (P.S.F.)								
Joist Spacing	DESIGN SPAN (Feet)							
	26	28	30	32	34	36	38	40
3'-61/4"			282	253	222	196	172	150
4'-61/4"			212	190	170	150	132	114
5'-61/4"	200	184	168	150	132	115	101	87
6'-61/4"	166	152	138	122	107	93	80	68



2 TYPICAL BEAM SECTION

HDR | ONE COMPANY *Many Solutions*

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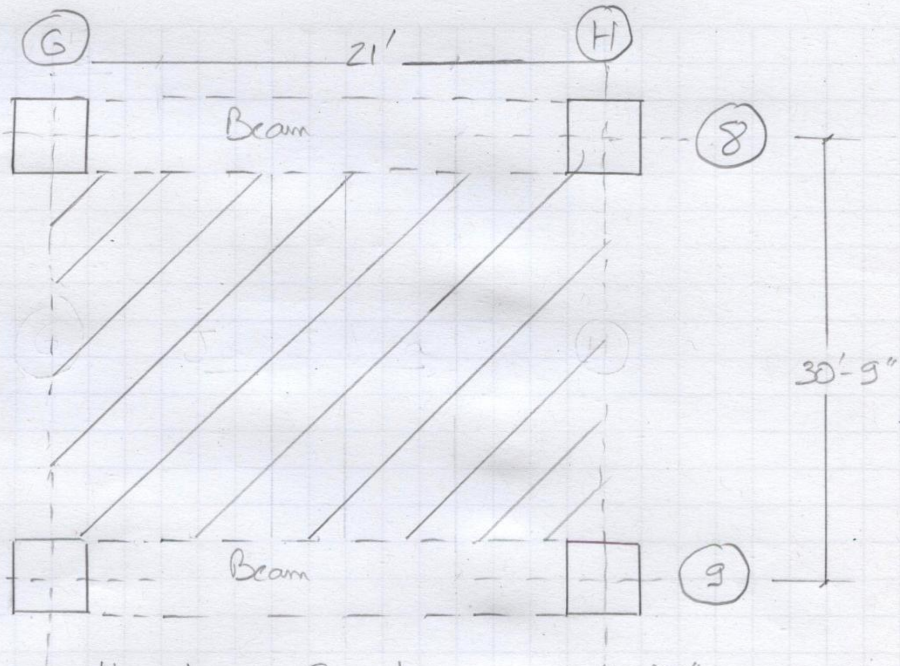
Subject: Gravity Check		Checked:	Date:
Task: Joist		Page: 1	of: 1
Job #:		No:	



All columns @ ends are 20" x 20"

Slab + SP = 62.5 + 14 + 20 = 96.5 psf

L = 125 psf

D+L = 125 + 96.5 = 221.5

Using the precast tables, for a span of 32' @ a spacing of 3'-6" to be uniform $\left(\frac{21'}{3.5'} = 6\right)$ a 16" deep joist of J3 would have a capacity 253 psf thus for a span of 31' a J3 @ 3'-6" would suffice

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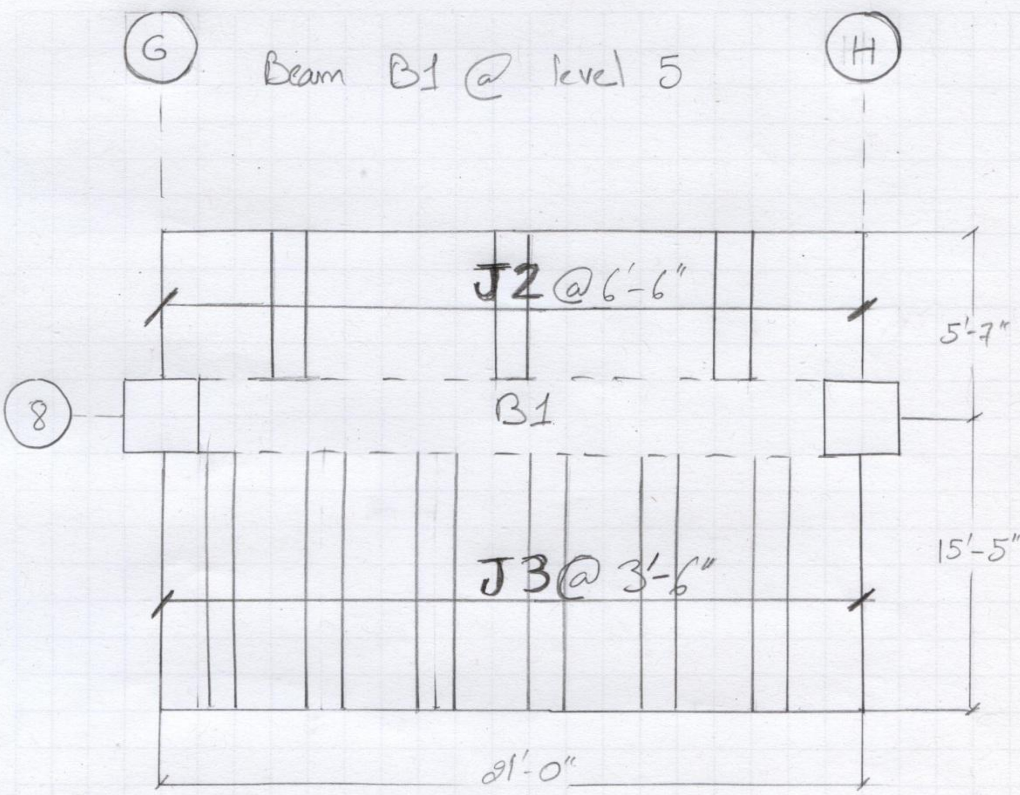
Project: Tech 1 Report		Computed:	Date:
Subject: Gravity check		Checked:	Date:
Task: Beam check.		Page: 1	of: 3
Job #:		No:	



Joist schedule: Assume deck is 4 3/4"

	depth:		weight	
J2	12"	@ 6'-6"	w = 67 psf	values taken from manufacturer
J3	16"	@ 3'-6"	w = 84 psf	

5" NW slab $\left(\frac{5}{12}\right)(150) = 62.5$ psf

SP 14 psf lighting, plumbing, ceiling, HVAC
20 psf partition load

Assume beam is continuous as there are moment frames

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Project: Computed: Date:
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Task: Beam check Page: 2 of: 3
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short direction : $(5'-7")(67) = 0.374$ KlF

long direction : $(15'-5")(84) = 1.295$ KlF

Slab + SP : $(62.5 + 14 + 20)\left(\frac{15'-5" + 5'-7"}{21'}\right) = 2.027$ KlF

total live : 125 psF

reduced by 20% see provisions

100 psf (21') = 2.1 KlF

$w_u = 1.2D + 1.6L = 1.2(2.027 + 1.295 + 0.374) + 1.6(2.1)$

$w_u = 7.795$ KlF

$$M_u^+ = \frac{7.795\ \text{KlF}\left(21' - \frac{20}{12}\right)^2}{16} = 182.1\ \text{K-ft}$$

Positive moments for interior spans.

$$M_u^- = \frac{7.795\ \text{KlF}\left(21' - \frac{20}{12}\right)^2}{11} = 264\ \text{K-ft}$$

on both sides

using coefficient in continuous beams for negative moment @ other faces of interior supports

B1 is a soffit beam of 5B-6, looking @ the beam soffit schedual, it is 24" x 24"

$M_u = 205$ $V_{u\,left} = 110$

w/ 2 #9 Full Length
3 #9 @ Right End

$V_{u\,right} = 105$

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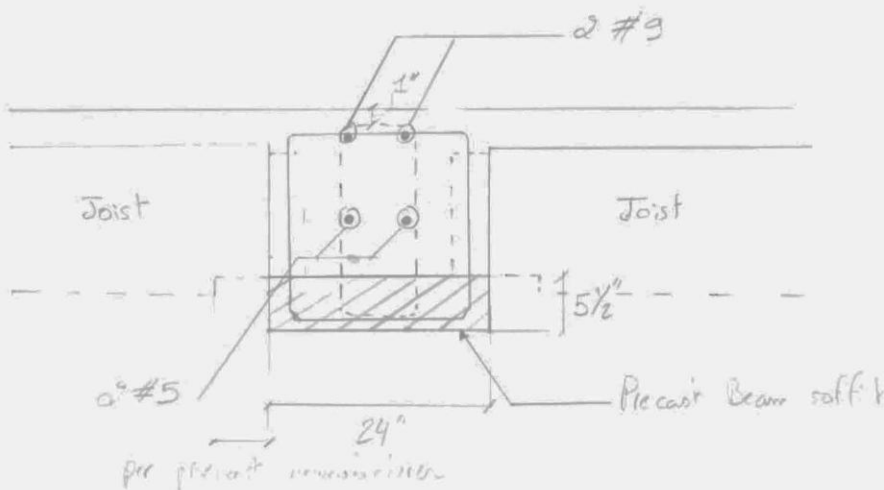
Project:		Computed:	Date:
Subject:		Checked:	Date:
Task:	Beam check	Page: 3	of: 3
Job #:		No:	



Note: the bottom precast beam soffit has reinforcement but the sizes are not mentioned by the manufacturer. However they are draped prestressed strands (same as the precast joist ones).

$$V_u = \frac{w_u \ell_n}{2} \qquad \frac{7.755\left(21 - \frac{8.10}{12}\right)}{2}$$

$$= 75.35\,k \qquad +10\%$$

The 10% added as a safety factor by the engineer since the loads are provided by the manufacturer. (for more details see beam check in Report)

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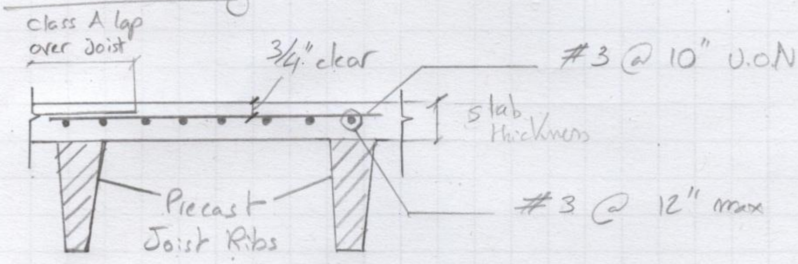
Project: Tech 1	Computed:	Date:
Subject: Gravity Check	Checked:	Date:
Task: Slab check	Page: 1	of: 3
Job #:	No:	

Slab One way :



Typical slab Reinforcing detail

- Check slab thickness

minimum slab thickness (table 9.5 (a))

Exterior Bay : $\frac{l}{24}$ for length of 39'-4" (maximum)

$\rightarrow \frac{39'-4''}{24} = 1.64''$

For interior Bay : $\frac{l}{28} = \frac{21'}{28} = 0.75'$

Design uses a min of 5" > 1.64" $\Rightarrow$ OK

- Check Reinforcement for max moment

Find W_u : $W_D = \left(\frac{5}{12} \times 150\right) + \underset{L, P, HVAC}{14\,psf} + \underset{\text{partition dead load}}{20\,psf}$

$W_D = 96.5\,psf$

$W_L = 50\,psf$ (office since thats where the typical was) + cannot be reduced

$W_u = 1.2D + 1.6\,W_L = 195.8\,psf \approx 196\,psf$

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Project:		Computed:	Date:
Subject:		Checked:	Date:
Task:	Slab check	Page: 2	of: 3
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Since $w_L < 3 w_D$ then we can use ACI moment coefficient

First interior: $M_u = \dfrac{-w_u l_n^2}{10}$

$= \dfrac{196 \times \left(19.5 - \frac{20}{12}\right)^2 \times 1ft}{10}$

$= 6,233$ lb-ft/ft

Second Interior $M_u = \dfrac{-w_u l_n^2}{11} = \dfrac{[illegible] \left(21 - \frac{20}{12}\right)^2 \times 1ft}{11}$

$M_u = 6,660$ lb-ft/ft

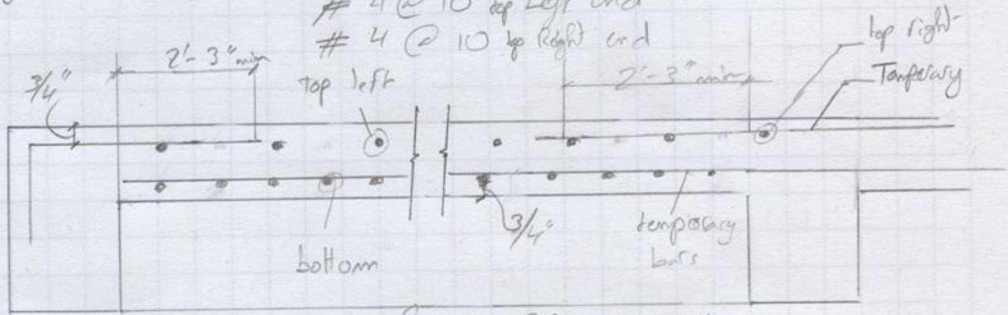
the second controls,

from the one way slab schedule an S-1 or 5" slab

that is typical would have #4@10 bottom

#4 @ 10 top Left end

#4 @ 10 top Right end



$l_n = 21 - \frac{20}{12} = 19.33'$

$\dfrac{19.33' \times 12}{10} = 23.2 \Rightarrow 23$

Top bars: since right end and left end can be combined

23 × #4

bottom 23 × #4 $\Big\} \Rightarrow 23 \times 2 \times 0.2 = 9.2\ in^2$

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Project:		Computed:	Date:
Subject:		Checked:	Date:
Task: slab check		Page: 3	of: 3
Job #:		No:	

$$A_s \text{ (per foot)} = \frac{9.2}{19.33'} = 0.476 \text{ in}^2/\text{ft}$$

M_n:

Assume $f_s > f_y$

$$a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{0.476(60{,}000)}{0.85(4000)(12)} = 0.7''$$

per foot

$$d = 5'' - \left(0.75 + \frac{0.5}{2}\right) = 4''$$

$$c = a/\beta_1 = \frac{0.7}{0.85} = 0.824''$$

Check $\varepsilon_s > \varepsilon_y$: $\varepsilon_s = \frac{\varepsilon_{cu}}{c}(d-c) = \frac{0.003}{0.824}(4-0.824)$

$$\varepsilon_s = 0.0115 \gg \varepsilon_y \Rightarrow \phi = 0.9$$

$$\phi M_n = 0.9\left(A_s f_y \left(d - \frac{a}{2}\right)\right) = 0.9\left(0.476(60{,}000)\left(4 - \frac{0.7}{2}\right)\right)$$

$$\phi M_n = 93{,}820 \text{ lb-in} = 7{,}818 \text{ lb-ft}$$

$$\phi M_n > M_u = 6{,}660 \text{ lb-ft} \Rightarrow \text{Okay!}$$

Check for shear:

for end members @ face of first support

$$V_u = 1.5 \frac{w_u \ell_n}{2} \quad (\text{ACI Sc } 8.3.3)$$

$$V_u = 1.5 \frac{196 \text{ psf} \left(21 - \frac{20}{12}\right)}{2} = 2{,}179 \text{ lb/ft of slab}$$

$$\phi V_c = 0.75\left(2\lambda\sqrt{f'_c}\, b_w d\right)$$

$$= 0.75\left(2(1)\sqrt{4000}(12)(4)\right) = 4{,}554 \text{ lb/ft} > V_u \Rightarrow \text{OK!}$$

Appendix B: Wind Load Calculations

General Wind Load Design Criteria	
Design Wind Speed	120mph
Directionality Factor (Kd)	0.85
Importance Factor (Iw)	1
Exposure Category	B
Internal pressure coefficient	0.55

Velocity Pressure, qz			
Level	Height	Kz	$q_z=0.00256K_zK_{zt}V^2I$
1	0'	0.57	21.012
2	14'-6"	0.57	21.012
3	29'	0.692	25.510
4	43'-6"	0.7775	28.662
5	58'	0.842	31.039
6	72'-6"	0.9	33.178
7	87'	0.951	35.058
Roof	107'	1.0075	37.140

Wind calculations start on next page.

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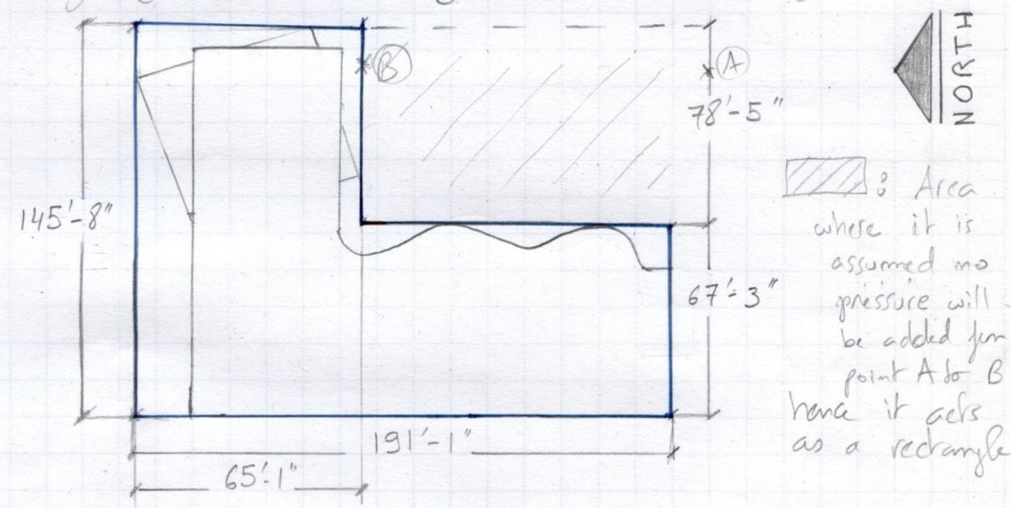
Project: Tech Report 1 Computed: Date:
Subject: Wind Analysis Checked: Date:
Task: Page: 1 of: 3
Job #: No:

Wind Analysis :

To simplify the calculations, a basic shape enclosing that of my original building will be taken as reference.

Figure



N-S Direction

L = 191'
B = 146'

} Rounding for simplicity

E-W Direction

L = 146'
B = 191'

There are two roof heights, the upper roof height will be taken as the only roof height for simplicity thus, 107'

Wind loads will be established based upon ASCE 7-05

Use method 2 since building meets criteria of 6.5.1 & 6.5.2

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Project: Tech 1		Computed:	Date:
Subject: Wind Analysis		Checked:	Date:
Task:		Page: 2	of: 3
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- This building is partially enclosed, and is not a low rise building hence, $GC_{pi} = \pm 0.55$
- Basic wind speed using figure 6-1c, $V = 120$ mph
- Site exposure is B, with wind importance factor, $I = 1.0$ since building category is office ⇒ II
- $K_{zt} = 1.0$ (Assumed)
- Wind directionality factor, $K_d = 0.85$ using table 6-4

velocity pressure q_z shall be determined by the following equation

$$q_z = 0.00256\, K_z K_{zt} K_d V^2 I$$
$$q_h = 0.00256\, K_h K_{zt} K_d V^2 I$$

See Excell spread-sheets

* Gust effect factor:

Rigidity of structure

$$T_a = C_t h^x = 0.02 (107)^{.75} = 0.665$$

$$1/T_a = 1.5\ Hz > 1.0 \Rightarrow \text{Rigid} \Rightarrow G = 0.85$$

Design wind pressures for MWFRS is determined by

$$p = qGC_p - q_i (GC_{pi})$$

$q_i = 37.14$ psf, $GC_{pi} = \pm 0.55$, $G = 0.5$

see table

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Project: Tech 1	Computed:	Date:
Subject: Wind	Checked:	Date:
Task:	Page: 3	of: 3
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N-S direction:

$L/B = \frac{191}{146} = 1.31$

Windward: $C_p = 0.8$

Leeward: $C_p = -0.438$ by interpolation

Side Wall: $C_p = -0.7$

E-W direction:

$L/B = 0.764$

$C_p = 0.8$

$C_p = -0.5$

$C_p = -0.7$

See Excell table

for $\theta = 0°$

$h/L = 107/191 = 0.56$

between 0.5 and 1.0 ⇔ interpolation ⇔ between 0.5 and 1.0

$h/L = \frac{107}{146} = .73$

$0 - h/2 \Rightarrow C_p = -0.948$ | $C_p = -1.084$

$h/2$ to $h \Rightarrow C_p = -0.876$ | $C_p = -0.808$

h to $2h \Rightarrow C_p = -0.524$ | $C_p = -0.592$

Appendix C: Seismic Load Calculations

Project:	Tech 1	Computed:		Date:	
Subject:	Seismic	Checked:		Date:	
Task:		Page:	1	of:	8
Job #:		No:			

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Calculating weight of building

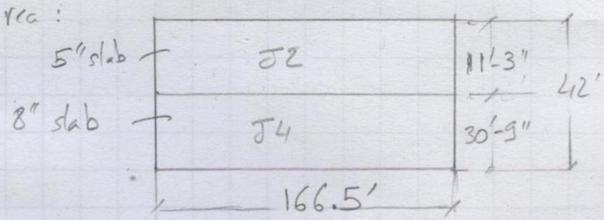
Roof level:

Framing: Joists J2 (12" deep) @ 5'-6" w = 69 psf

J4 (20" deep) @ 5'-6" w = 83 psf

Area:

total: 1,183.3

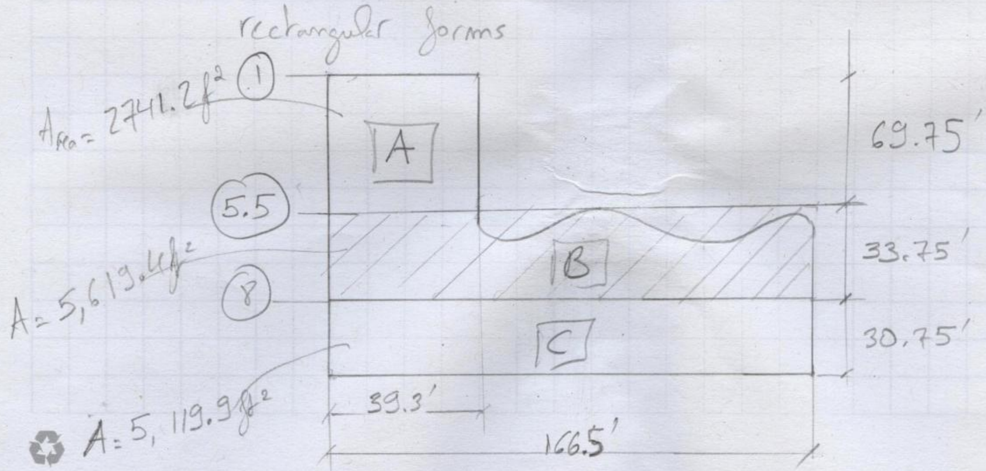


$$\left[69 + \left(\frac{5}{12} \times 150\right)\right](11'\text{-}3")(166.5) = 246.315 \text{ K}$$

$$\left[83 + \left(\frac{8}{12} \times 150\right)\right](30'\text{-}9")(166.5) = 936.937 \text{ K}$$

level 7:

For simplicity, the building is divided into 3 rectangular forms



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Project: Computed: Date:
Subject: Seismic Checked: Date:
Task: Page: 2 of: 8
Job #: No:

Notation (7)-(A)
level building Part

in 7-A 5" slab, J4 @ 5'-8" w = 84 psf
(67.5 + 84 psf)(2741.2) = 401.586 K

7-B 6" slab, J2 @ 3'-6" w = 76 psf
(75 + 76 psf)(5,619.4) = 848.529 K

7-C 6" slab, J3 @ 3'-6" w = 84 psf
(75 + 84)(5,119.9) = 814.064 K
+ 4" House Keeping pad
of (11'-3")(4")(14'-6") * 150 = 8.193 K

Total 7th = 2,072.372 Kips

Note this number is without columns & shear walls because they will be added later.

Also beam size are 20x30" or 24"x24" or 16"x24" or 16"x20" or 24"x30"
thus, an assumption of average size for simplification of weight calculation, the typical size 24"x24" will be taken.

Since this acumption is taken weight of beams will also be added at the end

Also, assume that where joists are closer together compensate for the voids that exist in the framing.

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Project: Computed: Date:
Subject: Seismic Checked: Date:
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level 6: 6-A 6" slab, J4 @ 4'-8' w = 90 psf
$(75+90)(2741.2) = 452.298$ K

6-B 5" slab, J2 @ 6'-6" w = 67 psf
Total: $(62.5+67)(5619.4) = 663.027$ K

1,865.39 K 6-C 5" slab, J3 @ 3'-6" w = 84 psf
$(62.5+84)(5119.9) = 750.065$ K

level 5: 5-A 5" slab, J4 @ 5'-8" w = 84 psf
$(62.5+84)(2741.2) = 401.586$

Total: 5-B 5" slab, J2 @ 6'-6" ⇒ 663.027 K

1,814.68 K

5-C 5" slab J3 @ 3'-6" ⇒ 750.065 K.

level 4: Same as 5

Total: 1,814.68 K

level 3: same as 5

Total: 1,814.68 K

level 2: 2-A Same as 5-A = 401.586 K

2-B Same as 5-B = 663.027 K

Total: 1,817.24 K 2-C ½ is J5 @ 4'-8" w = 95 psf

½ is J3 @ 5'-6" w = 74 psf

This ½ and ½ assumption because of magnet installation can be taken since where J5 exist @ closer spacing, J4 can compensate for the weight.

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Project:		Computed:	Date:
Subject:		Checked:	Date:
Task: Seismic		Page: 4	of: 8
Job #:		No:	

2-C . has 5" slab

$(62.5)(5119.9) = 319.994$

$\frac{95}{2}(5119.9) = 243.195K$ $\frac{74}{2}(5119.9) = 189.436K$

$\Rightarrow 752.625K$

Beam weight of 24×24

$w_{Beam} = \frac{24\times24}{144} \times 150 = 600$ plf

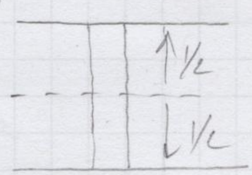
For each floor there is

# of B ×	L	= Kips
27 beams	@ 21'	340.2 K
2	@ 30'-9"	36.9
6	@ 11'-3"	40.5 K
1	@ 6'-4"	3.8
1	@ 16'-8"	10
1	@ 12'-6"	7.5
2	@ 31'	37.2
1	@ 22'	13.2
4	@ 12'-9"	30.6
4	@ 15'-9"	37.8
		557.7
		≈ 558 K

Columns

All columns are 20×20 and 14'-6" in height except for the roof

the weight of each column is divided by 1/2 to each floor



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Project:		Computed:	Date:
Subject:		Checked:	Date:
Task:	Seismic	Page: 5	of: 8
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thus $\frac{20 \times 20}{144} \times 14'\text{-}6" \times 150 = 6.041\ K$ for each column

there are 43 columns for each level except for roof level $\Rightarrow 43 \times 6.041 = 259.792\ K$

Roof —— ½ of —— $\frac{20 \times 20}{144} \times 20 \times 150 = 8.33\ K$

7 —— $\frac{1}{2}\ 259.792 + \frac{1}{2}\left(\underbrace{27 \text{ columns @ } 20' \text{ height}}_{112.5}\right) = 242.396\ K$

6 —— 259.792 K

5 —— ″

4 —— ″

3 —— ″

2 —— ″

1 ——

Thus total from each floor = 1,653.82 K

total for beam from each floor :

$(558 \times 6) + (\underbrace{24 @ 21',\ 2 @ 11'\text{-}3",\ 2 @ 30'\text{-}9"}_{302.22\ +\ 13.5\ +\ 36.9})$

Total = 3,700.62 K

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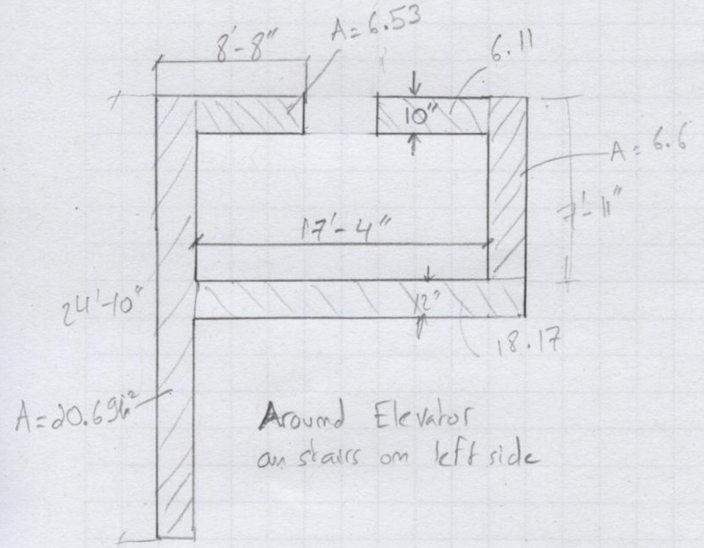
Project:		Computed:	Date:
Subject:		Checked:	Date:
Task:	Seismic	Page: 6	of: 8
Job #:		No:	

For shearwall,



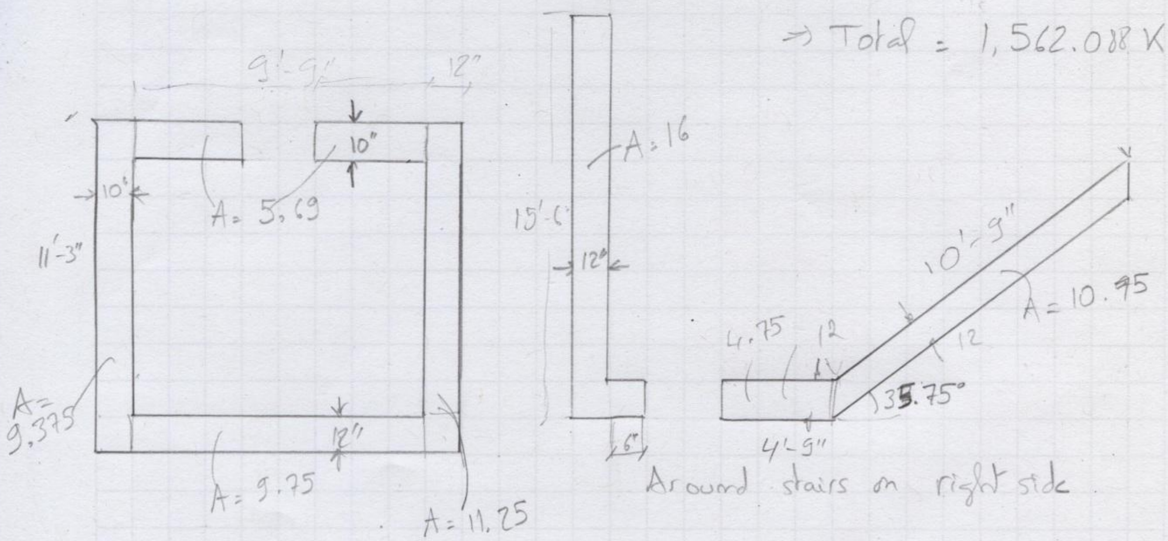
Around Elevator
on stairs on left side

Adding the Areas of all shear walls

58.1 + 30.1 + 31.5
= 119.7

119.7 × 150 = 17,955 plf
× story height
of 14.5
260.348
for floor
2, 3, 4, 5, 6 7

⇒ Total = 1,562.088 K



Around stairs on right side

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Project:		Computed:		Date:	
Subject:		Checked:		Date:	
Task:	Weight / Seismic	Page:	7	of:	8
Job #:		No:			

Total weight of building:

levels : 1,183.3 + 2,072.37 + 1,865.39 + (1,814.68 × 3) + 1,812.24 =

Columns : 1,653.82 K shearwalls = 1,562.09 K
Beams : 3,700.62 K

⇒ total ≈ 19,300 Kips

Design Spectral Response

Figure 22.1 and 22.2 of the ASCE 7-05 show S_s and S_1 respectively. the following are for J.B. Byrd, Tampa, FL

S_s = 7.8 % = 0.078
S_1 = 3.2 % = 0.032

Assume site class D for Tampa, Florida

⇒ a stiff soil since it has limestone

For site class E and S_s < 0.25 , F_a = 1.6

$S_{MS} = F_a \cdot S_s = 1.6 \times 0.078 = 0.125$

For site class E and S_1 < 0.1 then F_v = 2.4

$S_{M1} = F_v S_1 = 2.4 \times 0.032 = 0.0768$

$S_{DS} = 2/3\, S_{MS} \Rightarrow S_{DS} = 0.0832$
$S_{D1} = 2/3\, S_{M1} \Rightarrow S_{D1} \doteq 0.0512$

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Project:	Computed:	Date:	
Subject:	Checked:	Date:	
Task: Seismic	Page: 8	of: 8	
Job #:	No:		

$S_{DS} = 0.0832 < 0.167$ for building category II

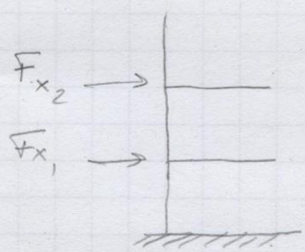
⇒ it is seismic design category A

and $S_{D1} = 0.0512 < 0.067$ for building category II

⇒ it is in seismic category A

For structures in that category then lateral forces are calculated as follows.

$F_x = 0.01 w_x$



For the computation of each please see the seismic table

Appendix D: Typical Plans

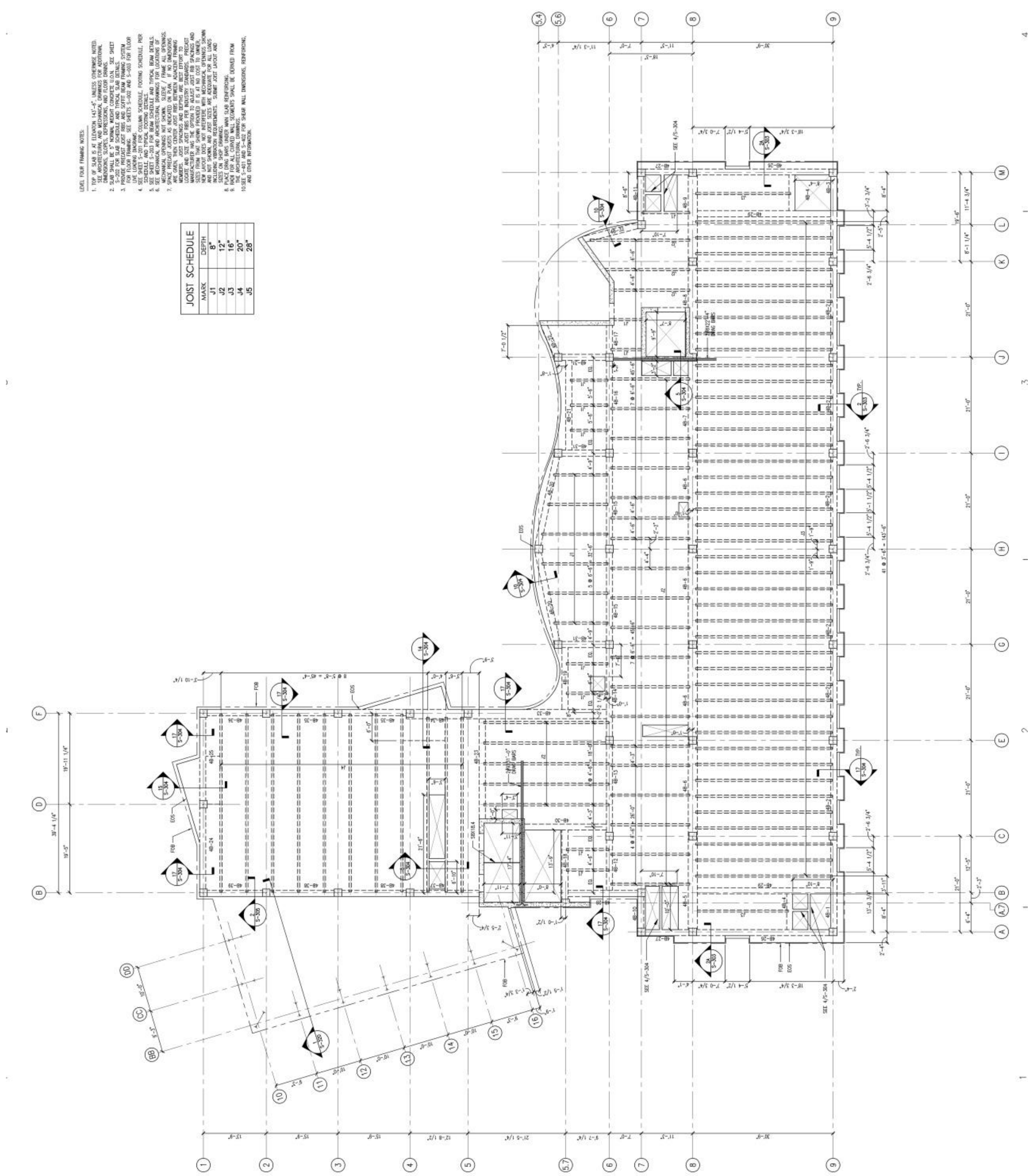


Figure 22 - Typical floor plan taken from S-104

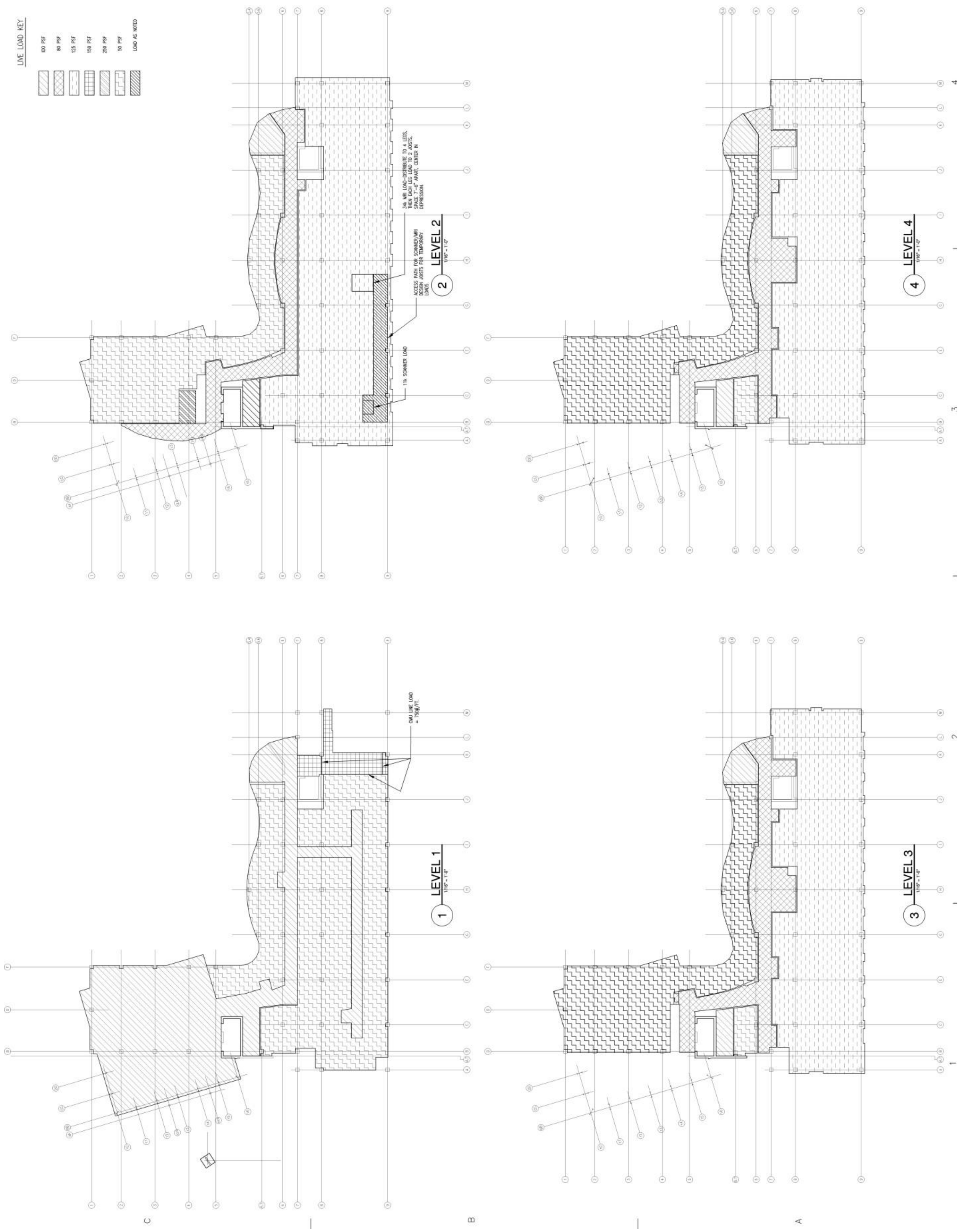


Figure 23 - Live Load diagram from S-002 (live load used in calculations)

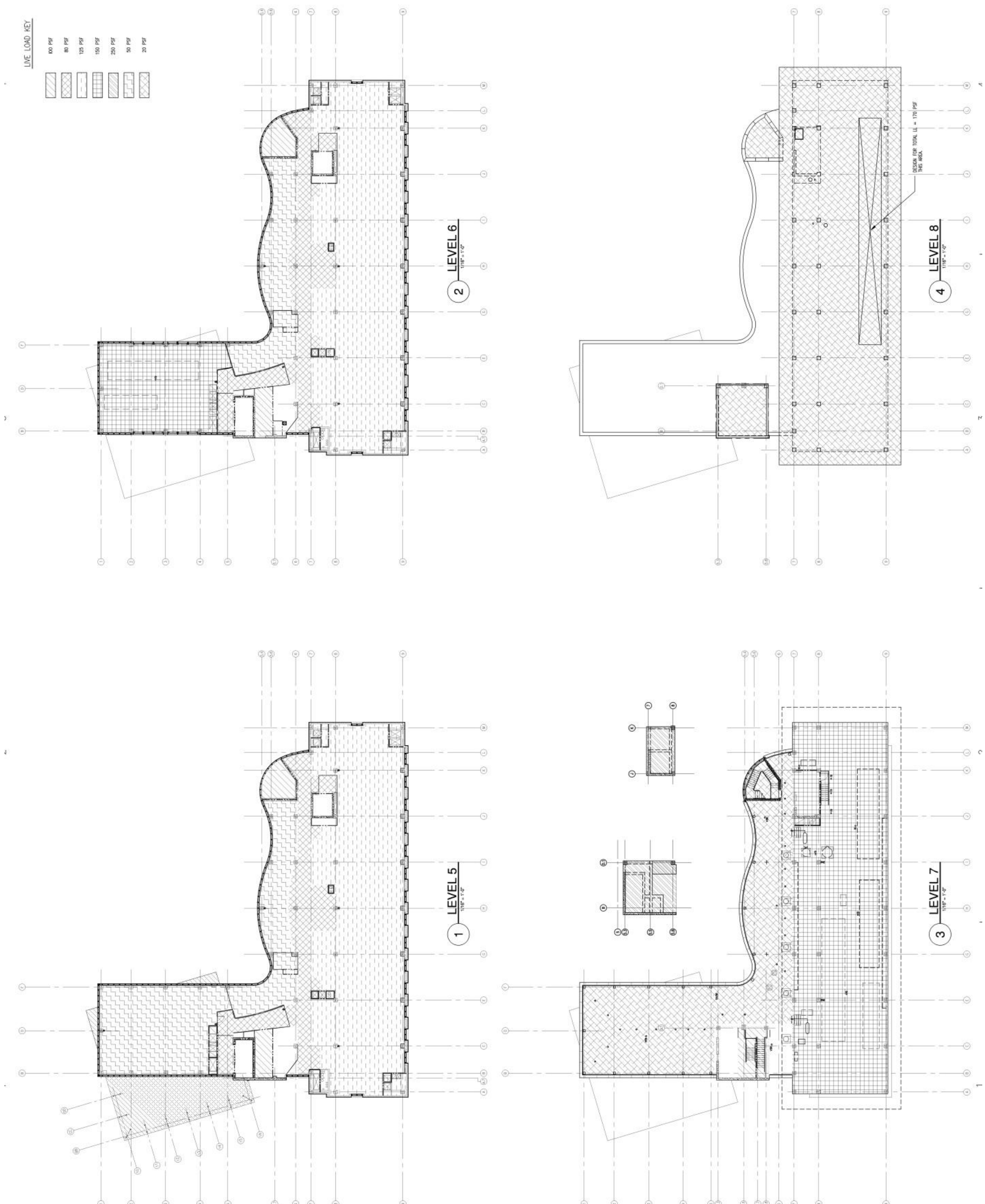


Figure 24- Live load diagram from S-003 (live load used in calculations)

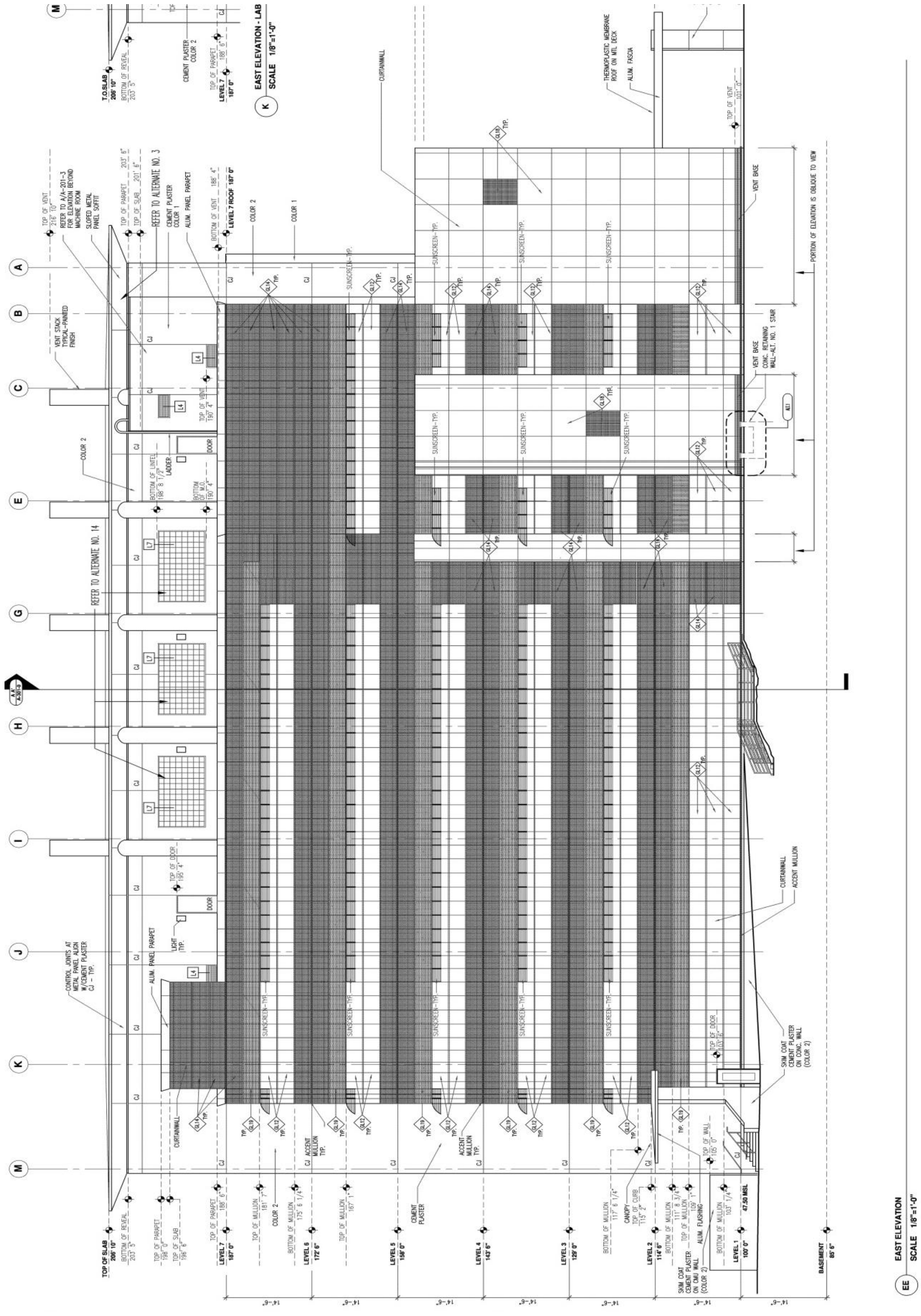


Figure 25 - Elevation of the building showing the different floor heights from A -201- 0