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

Tampa, Florida

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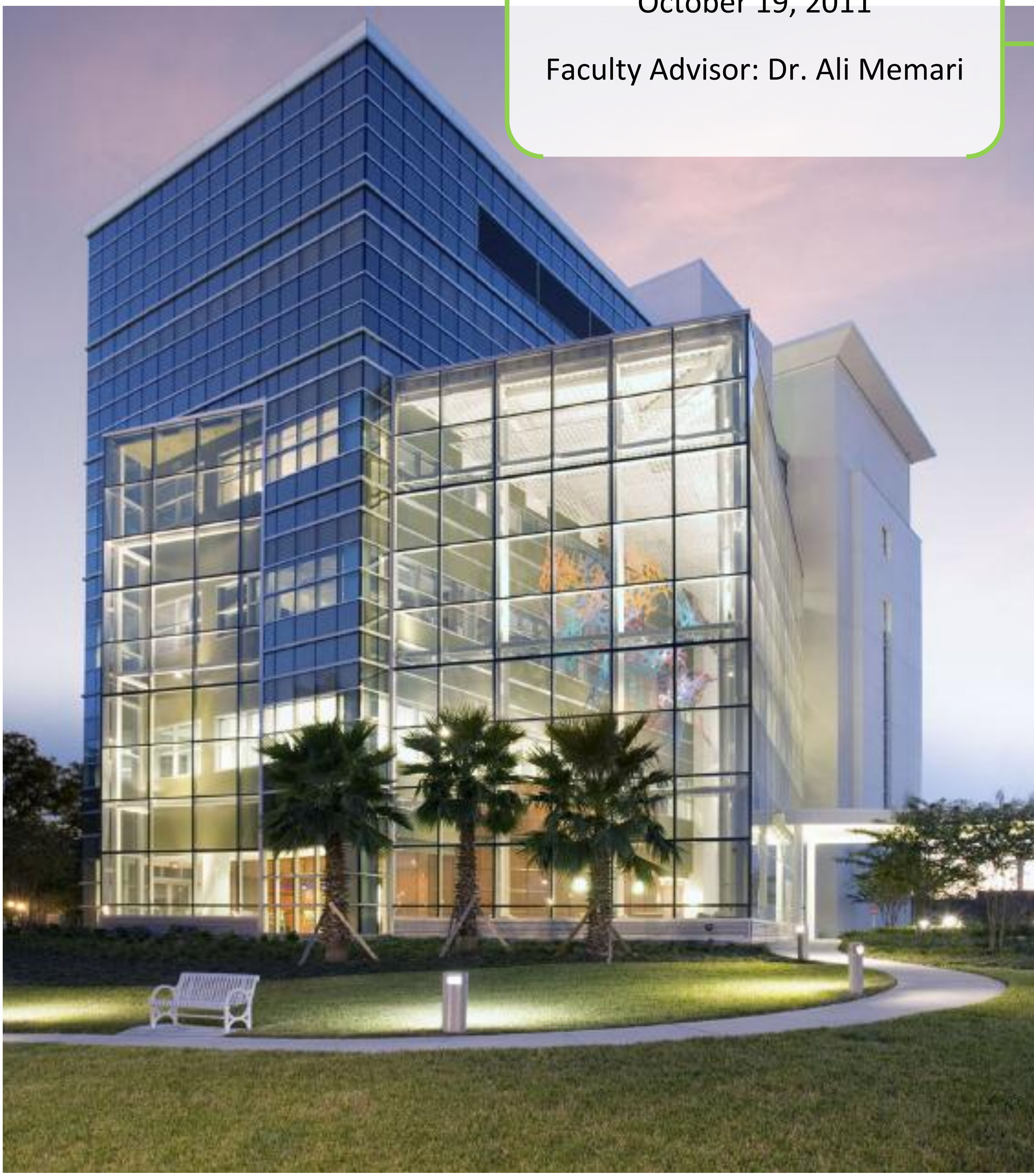

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Executive Summary

The purpose of Technical Report 2 is to design three alternative floor systems and compare them to the analysis performed on the existing structural system of the J.B.Byrd Alzheimer's Center & Research Institute in Tampa, Florida. This is accomplished through both hand and computer-aided calculations performed on a typical laboratory 30'-9"x21'-0" exterior bay spanning in the East-West direction from column lines E to G and in the North-South direction from column lines 8 to 9. The systems were compared on the basis of general conditions (weight, cost per square foot, and structural depth), architectural conditions (fire rating and other impacts), structural conditions (foundation impact and lateral system impact), serviceability conditions (maximum deflection and vibration control) and construction concerns (additional fire protection required, schedule impact, and constructability). The bay size and columns dimensions were kept the same as not to change the architecture and use of the building. The existing floor system is a 5" concrete slab with precast joists and soffit beams. The three systems designed in this report include:

- Composite Steel Framing with Composite Steel Deck
- Flat Plate with mild reinforcing 60ksi steel
- One-way slab with continuous beams

The design of the composite steel system results in 4 ½" concrete topping on 2" Vulcraft 2VL20 composite deck. The framing is W18x55 infill beams spanning 30'-9" with W21x62 girders spanning 21'. This system has less weight of the existing system, and has a comparable cost. It receives its strongest benefit from its additional constructability as well as the potential to reduce the required foundations. Its largest flaw is the addition of structural depth, the additional vibration precautions and the requirement for fireproofing that would probably necessitate a drop ceiling.

The second alternative, 12" flat plate system with mild reinforcement was the least viable. The system had deflection control issues, future expansions problems since floor drilling is not an option with a punching shear controlling design, a higher cost than the existing system, increase construction schedule, span restrictions in other areas of the building (i.e. next to the atrium), and finally a heavy structure that will not suit the existing foundations and may be rejected by the geo-tech as the site of the building sits on a potential sinkhole and requires a relatively light structure. The flat plate had to be rejected.

The last alternative selected for this report is the one-way cast-in-place concrete. The 4" slab with 20"x20" beams and girders came to be 15% close to the original weight of the building as well as the cheapest option of them all. The weight can be reduced significantly by reducing the width of the beams by 6 to 8 inches. That should also decrease the cost of the building. This should be done in later reports if the option is chosen. Additionally, it responds great to vibration, heavy live loads and future expansion. It is deemed great for research centers and hospitals. However, this may delay the construction schedule of the building. This is deemed to be the most competitive, even an alternative system to the existing precast joist and soffit beams if the cost is cheaper.

Building Introduction

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 the orange lines on 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 as a 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 108,054 sq ft, 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.

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

The Owner/Client of the project is Johnnie B. Byrd Alzheimer's Center & Research Institute. 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 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"per foot slope with Aluminum parapet for architectural reasons.

Structural Overview

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.

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 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. 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 9^{th} ED.
- AISC Manual of Steel Construction, Load Resistance Factor Design (LRFD) 1^{st} 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.

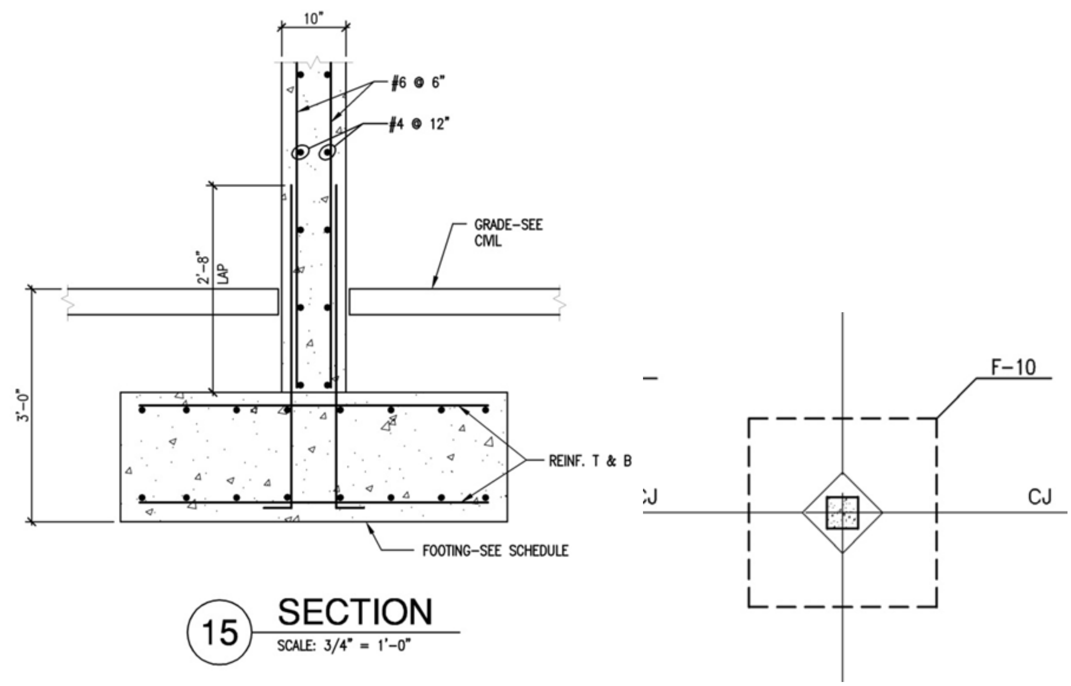


Figure 3- **Foundation section and plan showing footing-column connection and size**

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.

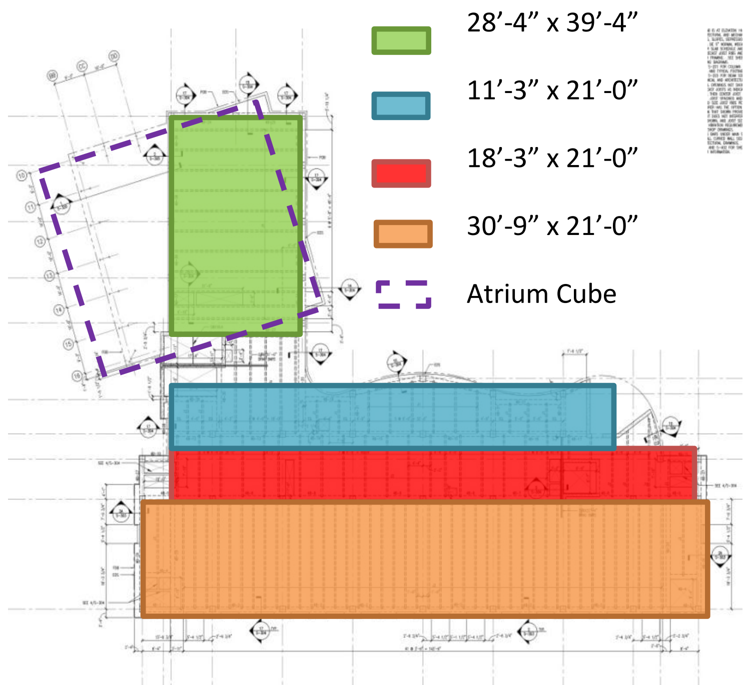


Figure 4- **Floor plan showing different bay sizes**

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).

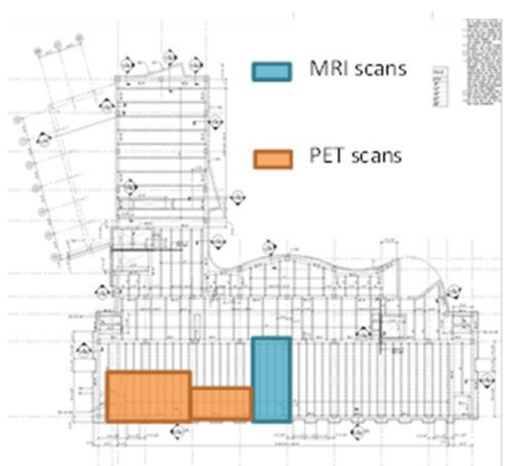
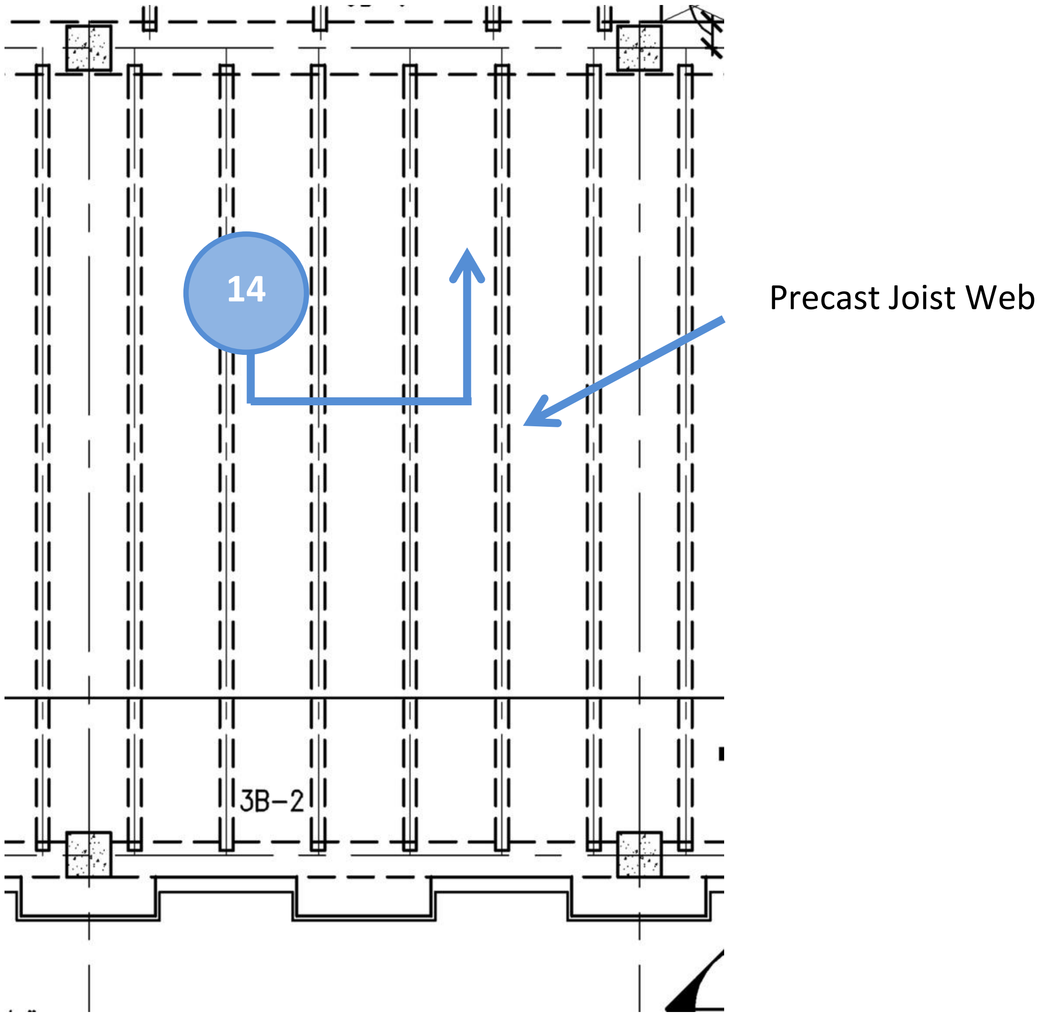


Figure 5- **2nd level floor plan showing MRI/PET scan location**



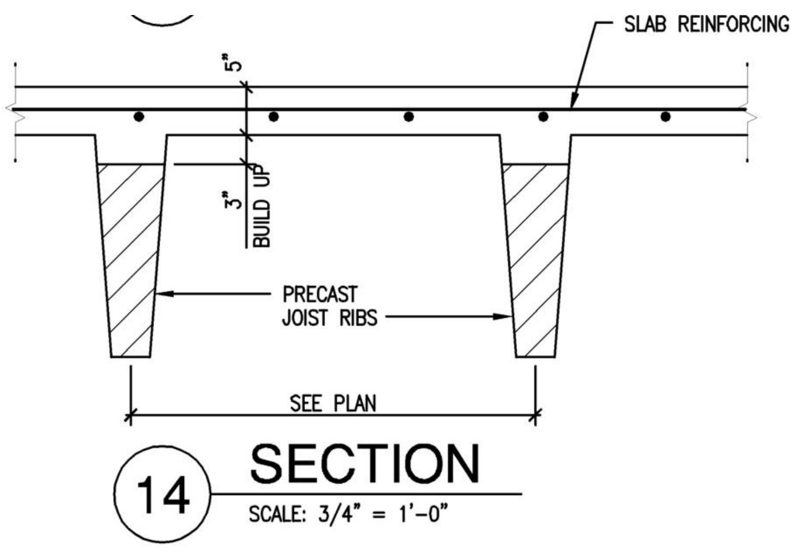


Figure 6- **Plan and section of precast joists**

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.

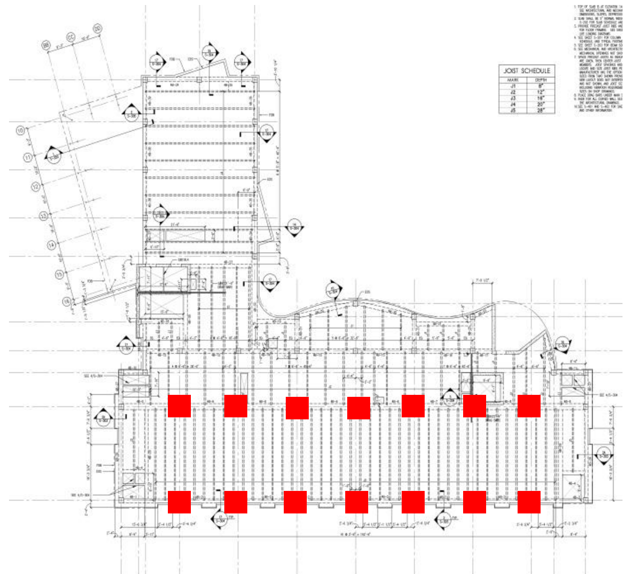


Figure 7- Floor plan showing the 6,000 psi column in basement and 1 floor

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.

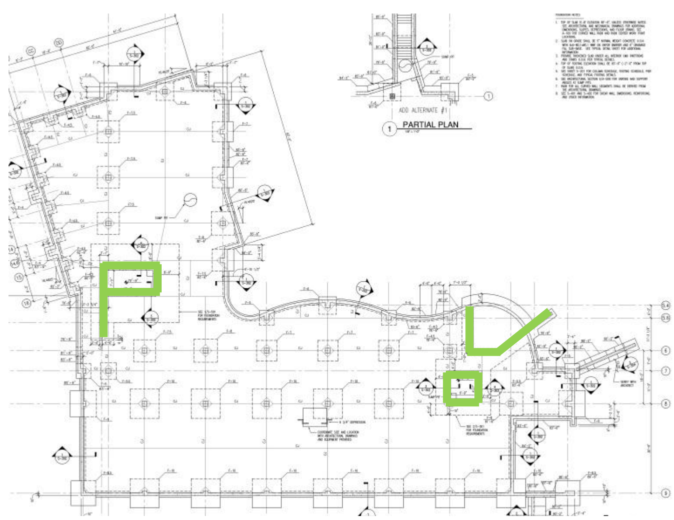


Figure 8- Floor plan showing shear walls

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. This last will play a significant role in choosing the members of the system as well as the systems themselves.

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. 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.

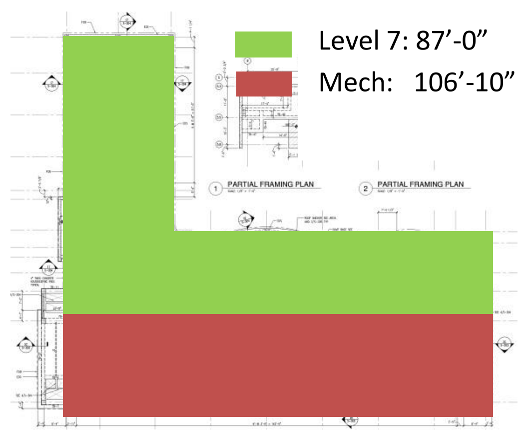


Figure 9- **Showing the different roof levels on the building**

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. 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.

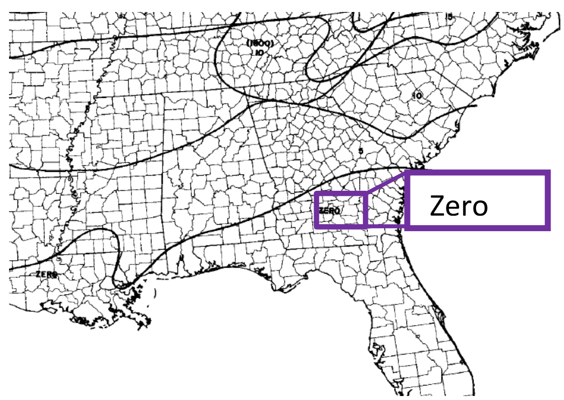


Figure 12- Diagram showing the ground snow load for Florida

Floor Systems

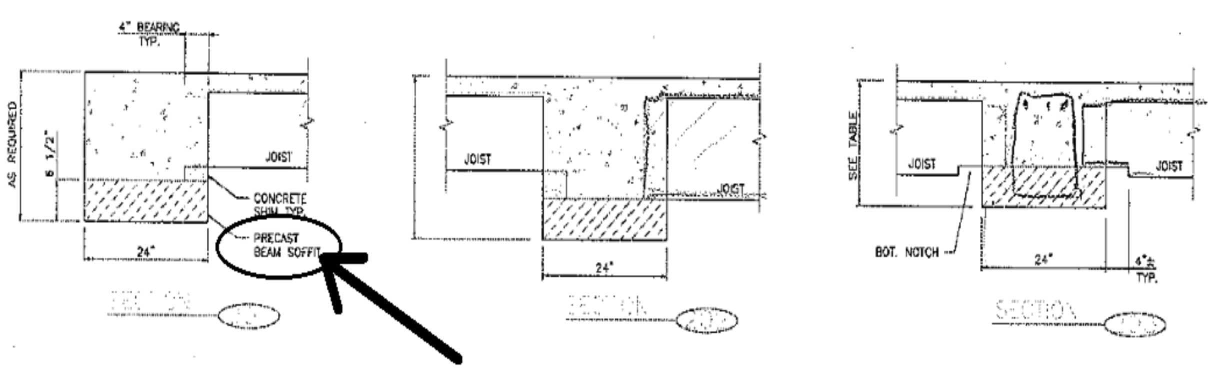
Precast Joists and Soffit Beams (Existing)

Joist and Beam Spot 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).



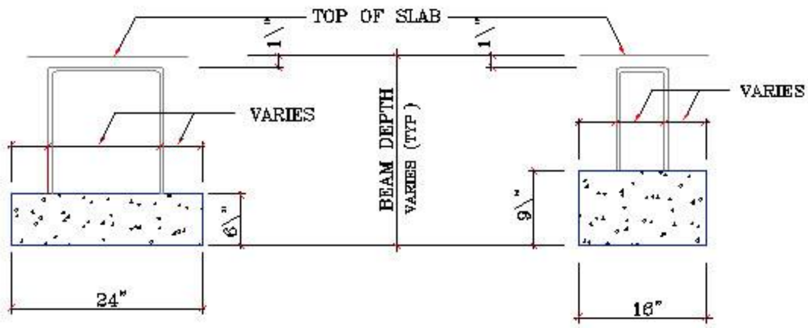


Figure 13- **Beam soffit details showing the precast and cast-in-place part**

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.

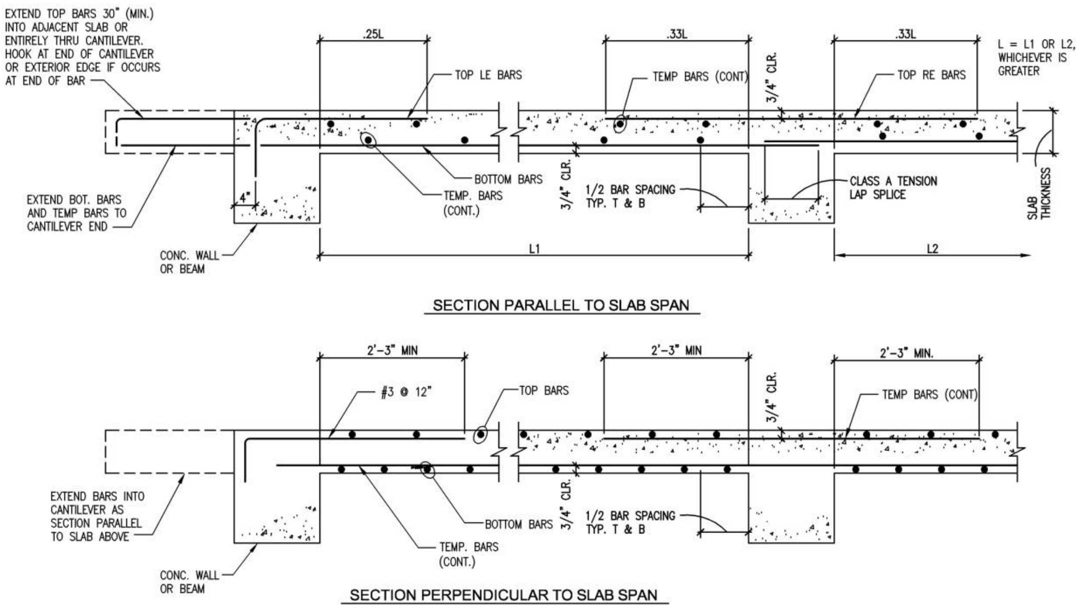
SOFFIT BEAM SCHEDULE

MARK	SIZE		MOMENT	REINFORCING				SHEAR	
	W	D	Mu	TOP BARS				Vu	Vu
	(in)	(in)	(ft-kips)	LE	FLLE	FLRE	RE	LEFT(k)	RIGHT(k)
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**

<u>General</u>

The price of this system is undetermined but in the process as the company that did this project is no longer in business. However it is safe to assume that it is relatively cheap (cheaper than the composite system) as it was chosen by the owner and general contractor for economic reasons.

<u>Architectural</u>

This system achieves all the requirements for fire rating, needs no fire proofing, can have cheap architectural ceiling finishes, less combustible materials in lab such as a suspended ceiling and creates more ceiling spaces for the labs. It should be noted that there are several locations in the building where the bottom of the structure was left exposed, which was made possible by the smooth surface of the precast concrete.

Structural

This system has a small or equal weight compared to the other systems. This translates to the light foundation system chosen and thus makes the building more economical. It satisfies all the structural requirements. This system would also have little or no effect on the lateral system, since concrete shear walls make the most sense for a structure that will be cast-in-place concrete.

Serviceability

Deflections were not calculated for this system, nor flexure requirements as all were already done by the precast company that provides the joists and beam soffits. However, the sizes with their respectable Mu were checked using the tables provided in appendix B. Also, this system was not analyzed for vibration but it meets the required owner's vibration requirements since it is known that post-tensioned joists and beams also tend to perform very well under vibration loading, and thus serviceability is not likely to be a concern for this system.

Construction

This system was given a constructability rating of "good", because although it only involves a cast-in-place concrete, that concrete crew is knowledgeable in erecting the precast joists and soffit. The erecting of the precast members only takes a day or two making the process really quick. No additional fire proofing is required to achieve the required rating. This system caused no delays in construction mainly everything went according to plan.

System Pro-Con Analysis

Pros:

- Low cost per square foot
- Low deflections and vibrations
- Maximizes ceiling use
- Easy to construct
- No fire proofing needed
- Specialized practice in Florida

Cons:

- Heavy scaffolding and temporary shoring
-

Composite Steel

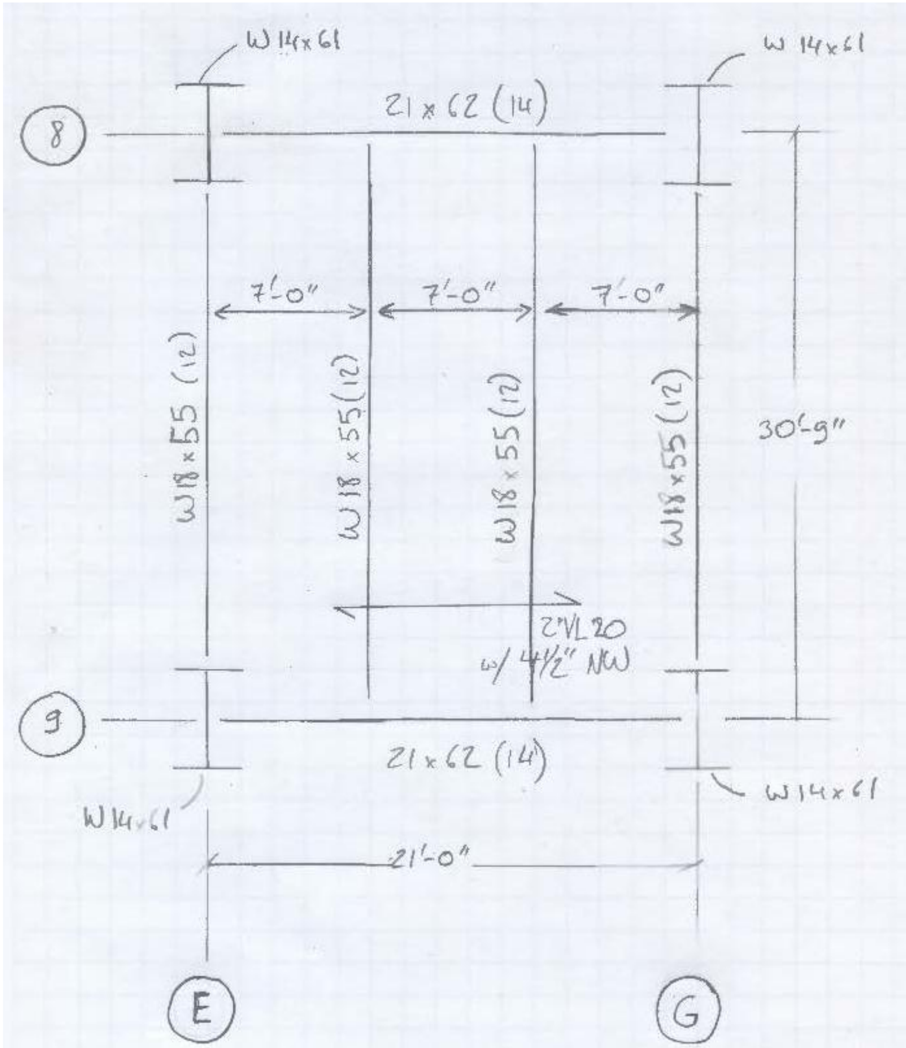


Figure 17- **Result of final Composite system after hand calculations and vibration analysis**

This system was chosen because of the relatively long spans and heavy live loads. The resulting system shown above is derived through hand calculations as well as the use of Microsoft Excel to develop a spreadsheet for repetitive calculations. That last was made for vibration analysis caused by humans for sensitive equipment existing in the lab such as heavy microscopes and PT and MRI scans. The detailed calculations and the results of the spreadsheet are shown in appendix C. The beams are topped with a 2" Vulcraft 2VL 20 galvanized composite metal deck with a 4 ½" normal weight concrete topping. That depth was chosen for a 2hr fire rating as well as a heavy floor for vibration purposes as well.

The layout of the two beams cutting the bay size into three was a result of the short girder span and long beam span that would benefit the one way load bearing system. In fact, because of the long span of the beam a short spacing equal to the third of the girder's span of 7'-0" was selected. This resulted in the beams and the girder being the same size in the preliminary stage of 21x44. That result would have been ideal for construction as pieces are the same but have different lengths and studs. Furthermore, a deeper analysis of the floor vibration as it should meet the required 2000 u-in/sec proved the system not good for serviceability. After several reiterations of the vibration calculations found on the spread sheet that are not shown here but available upon request, new framing was chosen. The layout, the metal deck and the topped stayed the same for simplicity and testing reasons however the beam and girder sizes changed. The beams decreased in size but increased its weight and the result was an 18x55 with 12 studs. On the other hand, the girders stayed the same size but increased weight to result in a 21x62 with 14 studs.

General

Total thickness of 6 ½" deck and the beams was found to weigh 77 pounds per square foot. This system costs about 14.53$. This estimate is taken from RSMeans CostWorks online program by choosing the closest dimensions, loads and deck thickness. This cost includes the precast production, transportation, and installation, the steel framing (including the columns) and erection, the concrete topping, and fireproofing for the steel, but no schedule or foundation impacts.

Architectural

The composite structure may be less volume than the existing concrete structure that may open the space and bring more light and relief to the space. However, being a steel structure it has to meet a 2 hour fire requirements thus the beams, girders or columns may be sprayed with fire proofing material. The most economical solution for this system to meet the required fireproofing is to provide a drop ceiling. That could result in a decrease in ceiling height or increase in overall building height to keep the same open ceiling space for the labs.

Structural

This system is almost the weight of the existing system. This was achieved by the use of steel and the weight of the steel joists compared (76 psf) to the precast joists (70-90 psf). This light system would benefit the structure as it sits on a potential sink holes and light foundations are needed. Since the existing system is in place this would have zero impact on the foundations. Additionally, since seismic is not an issue a lighter structure may not play a heavy role in decision making. However, being a steel frame building the use of shear walls could still be used or braces can be used instead that could reduce the cost and building schedule.

Furthermore, as the building is 18 miles from the Gulf of Mexico is relatively close to Lake Magdalene, the corrosion of structural steel may need to be addressed.

Serviceability

This system was mainly affected by deflections and vibration criteria. In fact, the beam and girder sizes were changed in the hand calculations to reduce deflections. After the right sizes were chosen according to gravity and deflection checks they were tested in the spreadsheet done in appendix C. This spreadsheet is not shown here in detail but is available upon request to see formulas. The live load of 11psf and 4psf were used in this case to represent the maximum loads from the Steel Design Guide series 11- Floor vibrations due to human activity. The results are compared to the moderate walking

pace. As this is a lab space it is safe to assume that the adjacent bays will see no fast pace walking steps per minute. Knowing that vibrations are a concern in steel, the result came in upsizing the members of the girders by giving them more mass. More depth would have helped but for competing with the high ceiling from the existing precast joists and soffit beams they were kept to a minimal.
Additionally, future drilling is not a problem as the building could have future expansions.

Construction

As structural steel needs to cased or sprayed with fire-proofing that could impact the cost and the construction schedule. The erection of steel is however quicker than the previous system since no reinforcements is needed to tie it with the slab like the existing precast joists and soffit beams. Thus, this could balance out the schedule even reduce it as steel construction is given a rating of “very good”.

System Pro-Con Analysis

Pros:

- Less Weight
- Easy to construct
- May shorten construction schedule
- Future expansions and floor drilling

Cons:

- Deflections and vibrations
- Fireproofing
- Corrosion issues
- Height limitations in labs
- Higher Cost than existing

Flat Plate with Mild Reinforcement

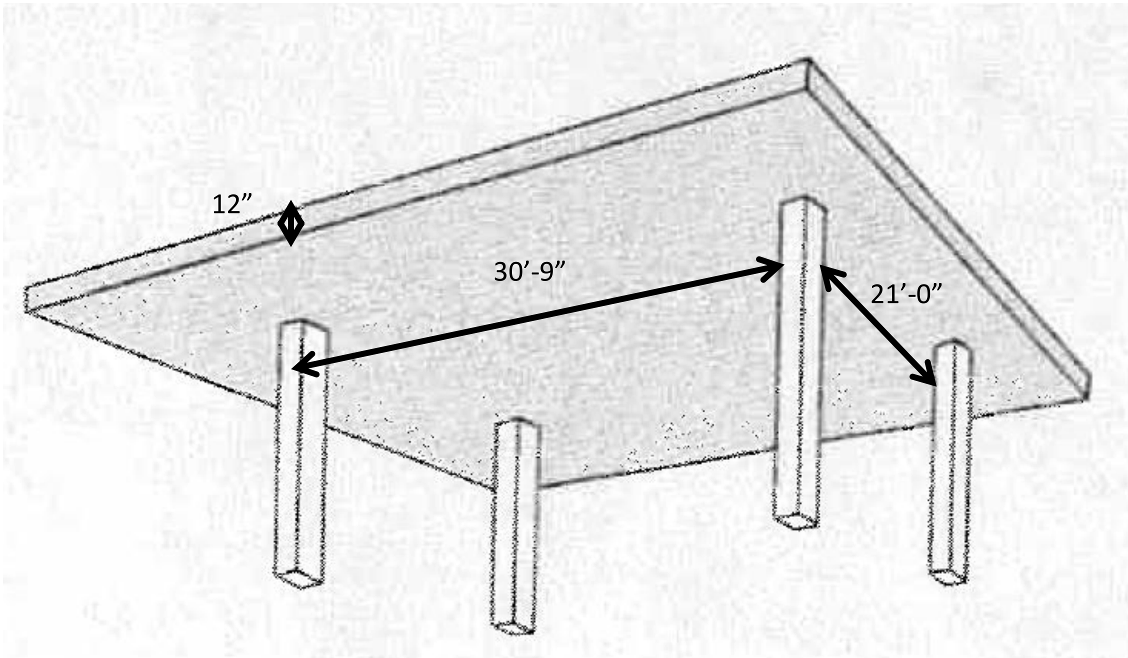


Figure 18 - **Resulting two-way reinforced Flat Plate System**

The second alternative floor system chosen is a two-way reinforced flat plate. This was chosen to keep a high ceiling usage since no beams exist as well as open space for the labs. Even though this system is limited to 25’x25’ bays, the bay size was chosen to be kept the same as to keep the long spans for the labs’ comfort and use. The plate also kept the same concrete strength of 4,000 psi normal weight and 60,000 psi for steel reinforcements.
The plate was designed using the direct design method from ACI 318-08. Please note for the simplicity of the calculations that last was used even though not all of the requirements were satisfied. Upon completion of the design calculations it was determined that a 12 in. slab would suffice with top and bottom reinforcing. As that is a heavy and thick slab a higher strength concrete could have been used however for cost, availability and comparison the 4,000 psi was kept. Also, heavy reinforcement such as 12 number 8 bars were used around the columns. To see reinforcement detail please see page 9 of the calculations of the flat plate system found in appendix D.

General

Total thickness of 12”slab was found to weigh 150 pounds per square foot. This system costs about 15.28$. This estimate is taken from RSMeans CostWorks online program by choosing the closest dimensions, loads and deck thickness. This system weighs more than the existing system and is more expensive.

Architectural

The flat plate structure may be less volume than the existing concrete structure that may open the space and bring more light and relief to the space. That could result in a decrease in ceiling height or decrease in overall building height keeping the same open ceiling space for the labs. Thus the owner than save money or can have more square footage. Also, the flat plate eliminates the need for a ceiling finish due to the aesthetically pleasing smooth surface that is the bottom of the slab. Furthermore, the concrete possesses a two hour fire rating making additional fire protection unnecessary.

Structural

This system is almost 1.5 the weight of the existing system. This system would have significant effects on the foundations that may require drilled piers. Additionally, since the system weighs more, the mass would help in the overturning moment of the structure. This system needs a lot of reinforcing around the columns as it is vulnerable to punching shear. However, the calculations shown in appendix D, took in considerations deflection control, punching shear and wide beam action making the 12" slab adequate to support and resist all of the above.

Serviceability

This system was mainly affected by deflections and punching shear. In fact, the slab's thickness was controlled by punching shear. Vibrations were assumed not an issue as the slab is 12" thick with heavy reinforcements would satisfy the 2000 u-in/sec required for the labs. If this system should be later used then additional vibration analysis would be done. Additionally, future drilling is a problem for this kind of system.

Construction

This system was given a constructability rating of "good" because although it only involves a cast-in-place concrete, the formwork is very simple and uniform throughout the building. This would not decrease the price even though formwork is the most expensive since additional reinforcement is applied. This system is not as quick in erection as the other and may increase the schedule of construction.

System Pro-Con Analysis

Pros:

- Overall building height may be decreased
- Thin Structure
- Simple Formwork
- No fire proofing is needed
- No ceiling finish is needed

Cons:

- Deflections Control
- Future expansions and floor drilling
- Higher Cost than existing
- Span restrictions in other areas of the building
- Heavy structure that may change foundations
- Increase Construction schedule

One Way Slab with Beams

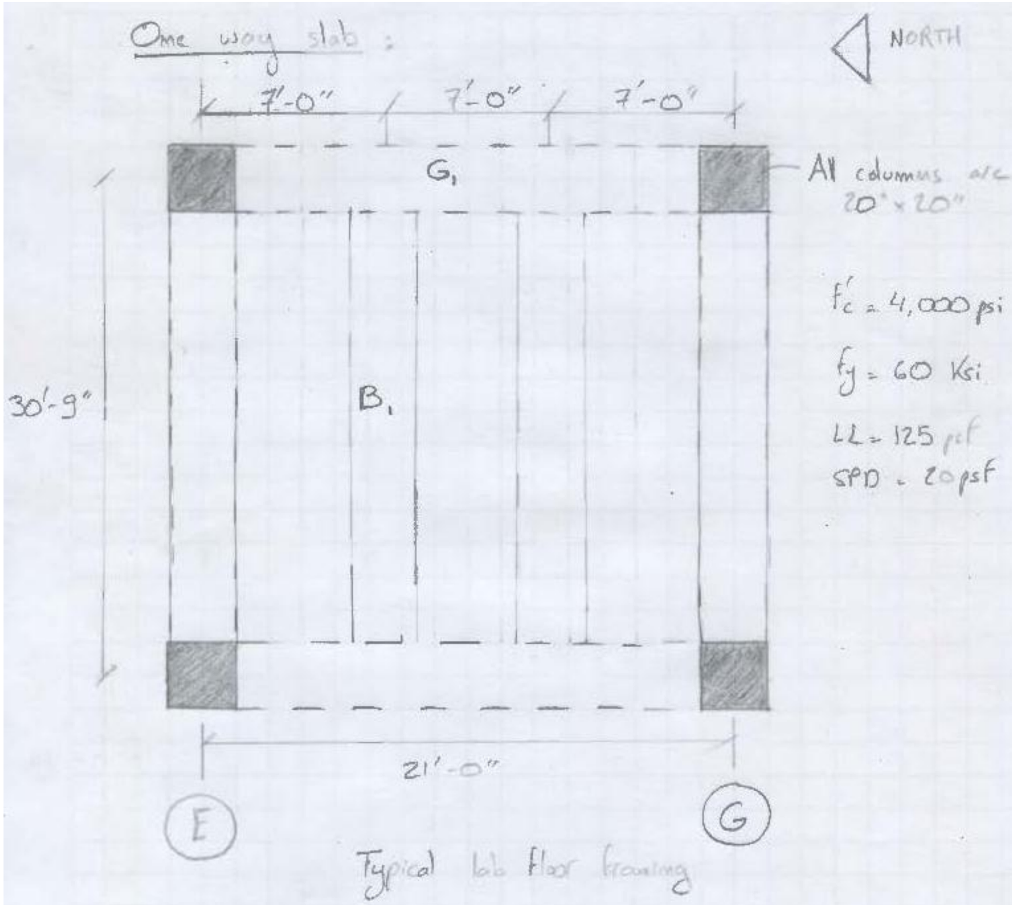


Figure 19 - **Layout chosen for a typical laboratory bay**

The third alternative floor system chosen is a one-way slab. This was chosen to compare how a typical cast-in-place system would perform instead of the existing one. The layout above was chosen to minimize the slab thickness in order to minimize the weight, and keep beams and girders the same sizes. A total of 4"thick slab on top of 20"x20" beams to fit girder size for formwork reasons spaced at 7'-0". The slab also kept the same concrete strength of 4,000 psi normal weight and 60,000 psi for steel reinforcements.

The slab beams and girders were designed using the ACI coefficient from ACI 318-08. Please note for the simplicity of the calculations that last was used even though not all of the requirements were satisfied. Upon completion of the design calculations it was determined that the slab was designed to have #4 at 12" on center for flexure, shrinkage and temperature. The beam spanning the 30'-9" had large negative moments which required more reinforcements. Also, since the bay is at the edge of the building the beam was analyzed at the supports and mid-span totaling 3 zones. The following reinforcements were designed starting from the edge going to the interior of the building: 2 #9, 3 #9 and 4 #9. The girder had 1 #9 at mid-span and 4 #9 at the supports. All of the members had a # 4 stirrup. The detailed calculations for the one-slab system can be found in Appendix E.

General

Total thickness of 4"slab was found to weigh 50 pounds per square foot. And the beam was found to weigh 59 pounds per square foot a total of 110 slightly heavier than the existing system (Note that it is possible to make the beams 4 to 8" thinner than 20" lowering the weight of the structure). This system costs about 14.25$. This estimate is taken from RSMeans CostWorks online program by choosing the closest dimensions, loads and deck thickness. However, since the beams and girders both have the same size and number of bars and the uniformity of the building the system cost should decrease.

Architectural

This system does not provide architecturally pleasing ceiling finish as the beams are heavily exposed. However, the voids between the beams could provide mechanical equipment since they span the long way thus minimizing the ceiling height. With careful construction practices, a smooth underside of the structure could be achieved, which would then allow the structure to be left exposed. However, this may be more costly than the basic costs that were evaluated in this report. Furthermore, the concrete possesses a two hour fire rating making additional fire protection unnecessary.

Structural

This system would have negligible effects on the foundations and lateral system of the building. Additionally, since the system weighs the same, the mass would not be an issue in the overturning moment of the structure. This system needed to be checked on several locations since the bay is an edge bay. The calculations shown in appendix E, took in considerations flexure, shear and deflection control.

Serviceability

Vibrations were assumed not an issue since this system is inherent in vibration resistance and would satisfy the 2000 u-in/sec required for the labs, thus no calculations were done. If this system should be later used then additional vibration analysis would be done. Additionally, since this system deals well with high live loads and core drilling it is good for future renovations

Construction

This system was given a constructability rating of "medium" because it only involves a cast-in-place concrete, the complex formwork and shoring. The uniformity of the beams and girders would decrease the price since formwork is the most expensive. This system requires a lot of time for construction thus it may increase the schedule of construction.

System Pro-Con Analysis

Pros:

- Heavy live loads
- Future expansions
- No fire proofing is needed
- Inherent vibration resistance
- Relatively cheap

Cons:

- Construction schedule delay
- Complex formwork
- Labor extensive (however labor is relatively cheap in Florida)

Summary of Systems

Consideration		Precast Joist and Soffit Beams (Existing)	Composite Steel	Flat Plate with mild Reinforcements	One-Way Slab
		System			
General	Weight (psf)	90	75	150	109
General	Cost ($/SF)*	cheap (unknown)	14.53	15.28	14.25
General	Floor Depth	5 slab/ 16-24 beams	4.5 slab/ 21 girders	12 slab	4 slab/ 20 beam and girders
Architectural	Fire Rating	2 hr	2 hr	2 hr	2 hr
Architectural	Other Impacts	Structure is hidden but left exposed in some locations	Drop ceiling must be provided and decreases floor to floor	Can be left exposed and creates higher ceiling for labs	Minimizes ceiling height and structure cannot be left exposed
Structural	Foundation Impacts	Existing Cast-in-place footings and mat slabs	May reduce required foundations	Heavy structure that may not be good for a potential sinkhole site	Zero to negligible effect on foundations
Structural	Lateral System Impact	Existing Cast-in-place shear walls	Steel braced/ moment frames	Shear walls would remain	Shear walls would remain
Serviceability	Maximum Defelection (inches)	N/A	0.93	1.343	1.186
Serviceability	Vibration Control	Very Good	Average but analyzed in report	Average	Very Good
Construction	Additional Fire Protection Required	None	Will have spray-on	Will likely have none	Will likely have none
Construction	Schedule Impact	N/A	Likely have no delay	Likely delay shedule	Likely delay shedule
Construction	Constructability	Good	Good	Medium	Medium
Feasibility		N/A	**Yes**	**No**	**Yes**

* All costs are taken from RSMeans CostWorks online program which carries an error of +/- 15% by choosing the closest dimensions (25'x30'), loads (SP=20-40psf) and deck/slab thickness. This cost includes the precast production, transportation, and installation, the steel framing (including the columns) and erection, the concrete topping, and fireproofing for the steel, but no schedule or foundation impacts.

Figure 20 **Summary chart of this report's different framing systems**

Conclusion

Technical Report 2 analyzed the existing floor system of the J.B.Byrd Alzheimer's Center & Research Institute in Tampa, Florida and compared it to three additional floor systems, all of which were also designed as a part of the technical report. The analysis/design of all systems was performed at a typical laboratory bay which happens to be an exterior bay. Major factors in the comparison of the systems were cost, weight, structural depth, constructability and architectural impact, although several other considerations were also included. It was desirable to keep the weight of the building without adversely affecting the cost or structural depth.

The existing 5" slab with precast joists and soffit beams remains the least expensive until further analysis on how much the existing system costs will be available. It is the second lightest after steel or the lightest in concrete even though the one way slab system can be reduced in weight.

Composite steel was found to be slightly more expensive but significantly lighter than all the systems. However, it has several negative impacts on the building architecture, such as the potential of increased height (due to higher structural depth) and the inability to leave the structure exposed. Similarly, it needs additional fire proofing such as a spray-on that is not included in the cost and its effect on the construction schedule. Steel structure is also not the best in vibration requirement for sensitive equipment that is a major design in the J.B.Byrd Alzheimer's Center & Research Institute. Despite these concerns, the system has a great deal of inherent flexibility, and it is possible that with further refinement (with a detailed vibration analysis), these concerns could be resolved. It also can utilize either a braced frame or moment frame lateral system, which provides additional opportunities to adjust the design to suit the building. For these reasons, it was deemed to be a viable alternative.

The second alternative, the flat plate system with mild reinforcement was the least viable. Even though the flat plate had great structural responses and would provide more space in the ceilings it had to be rejected. The system had deflection control issues, future expansions problems since floor drilling is not an option with a punching shear controlling design, a higher cost than the existing system, increase construction schedule, span restrictions in other areas of the building (i.e. next to the atrium), and finally a heavy structure that will not suit the existing foundations and may be rejected by the geo-tech as the site of the building sits on a potential sinkhole and requires a relatively light structure.

The most competitive system - yet not better than the existing except if cheaper - that was found is the one-way cast-in-place concrete. The 4" slab with 20"x20" beams and girders came to be 15% close to the original weight of the building as well as the cheapest option of them all. The weight can be reduced significantly by reducing the width of the beams by 6 to 8 inches. That should also decrease the cost of the building. This should be done in later reports if the option is chosen. Additionally, it responds great to vibration, heavy live loads and future expansion. It is deemed great for research centers and hospitals. However, this may delay the construction schedule of the building.

Appendices

Appendix A: Typical Plans

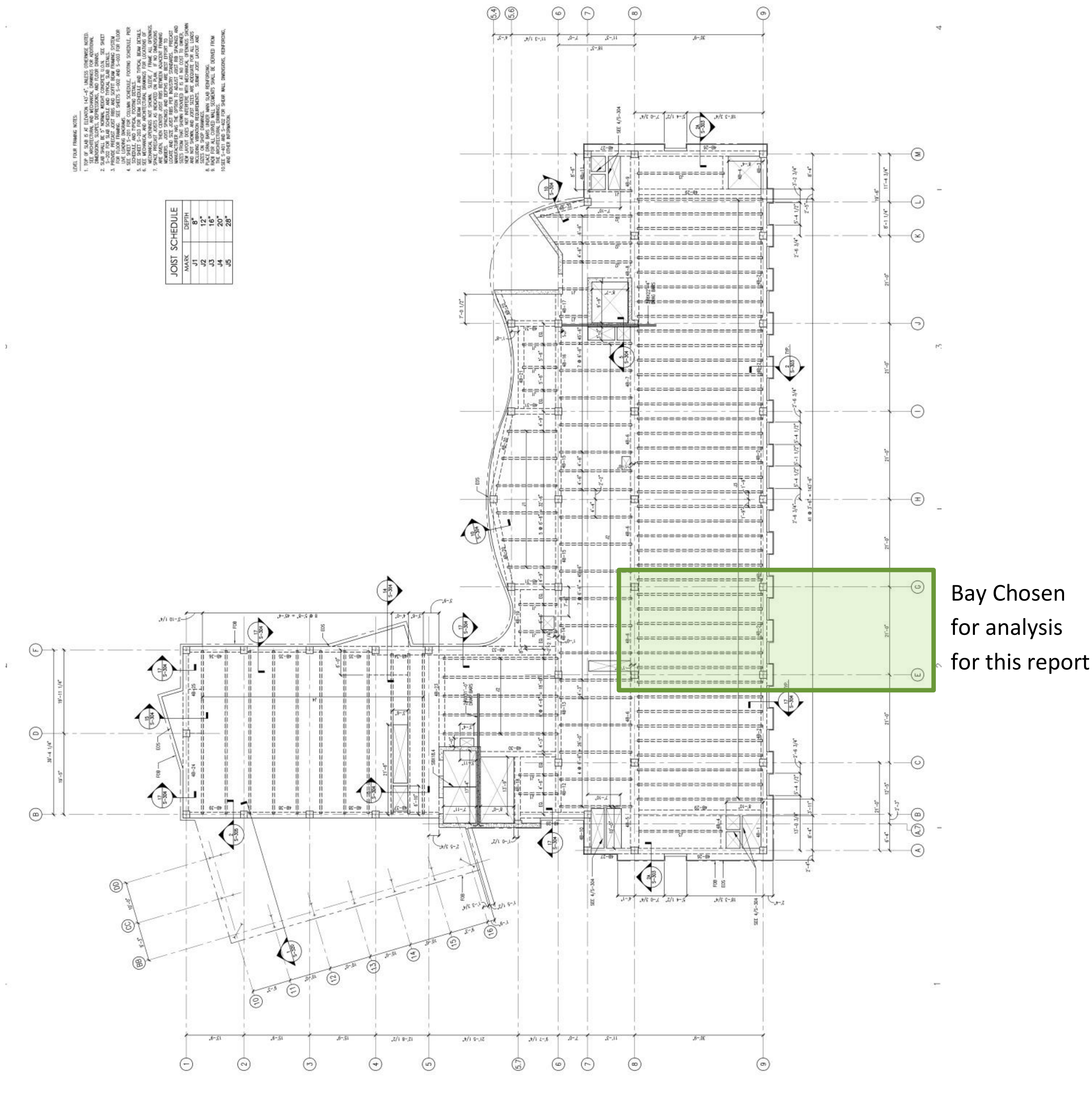


Figure 21 - **Typical floor plan taken from S-104**

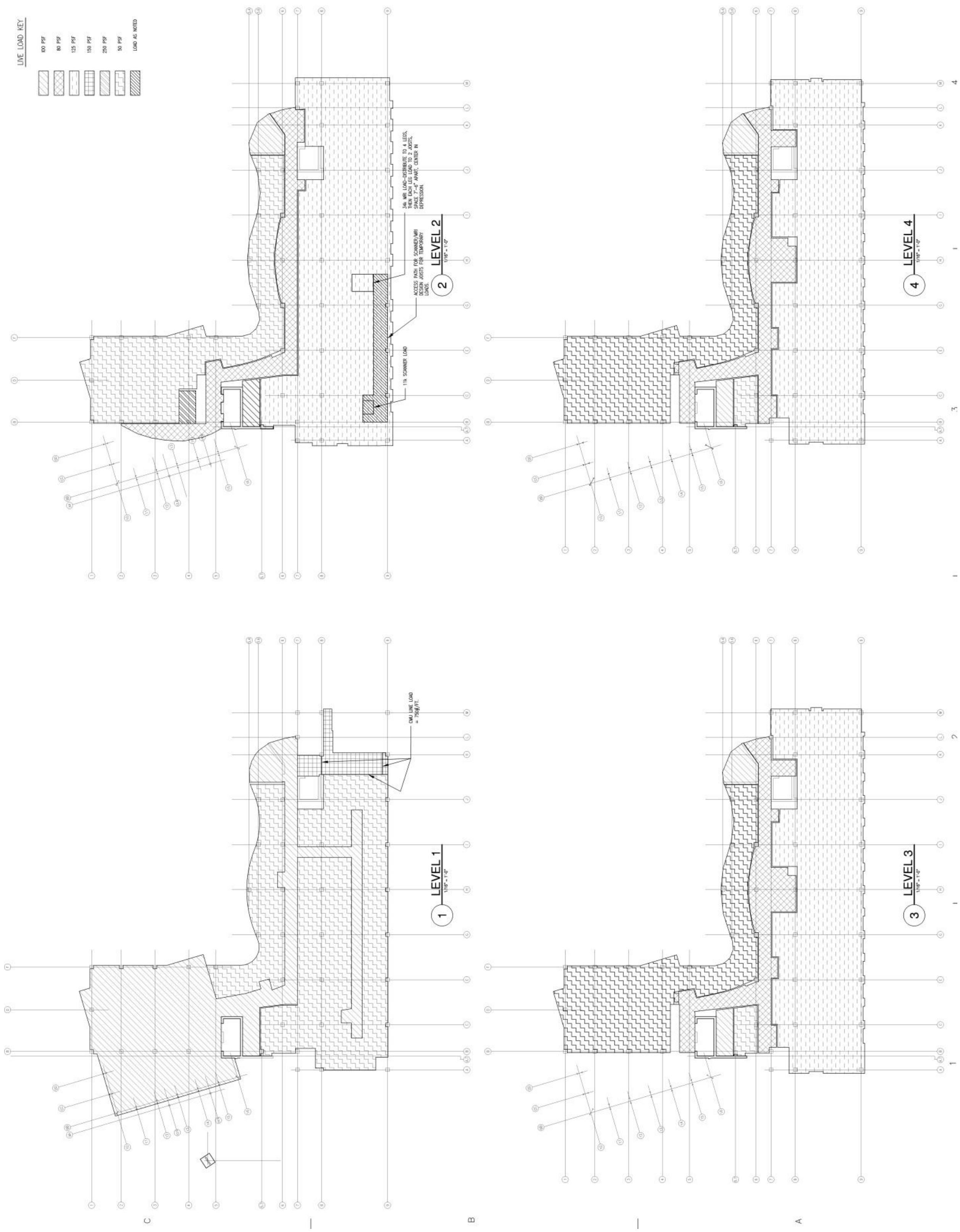


Figure 22 - Live Load diagram from S-002 (live load used in calculations)

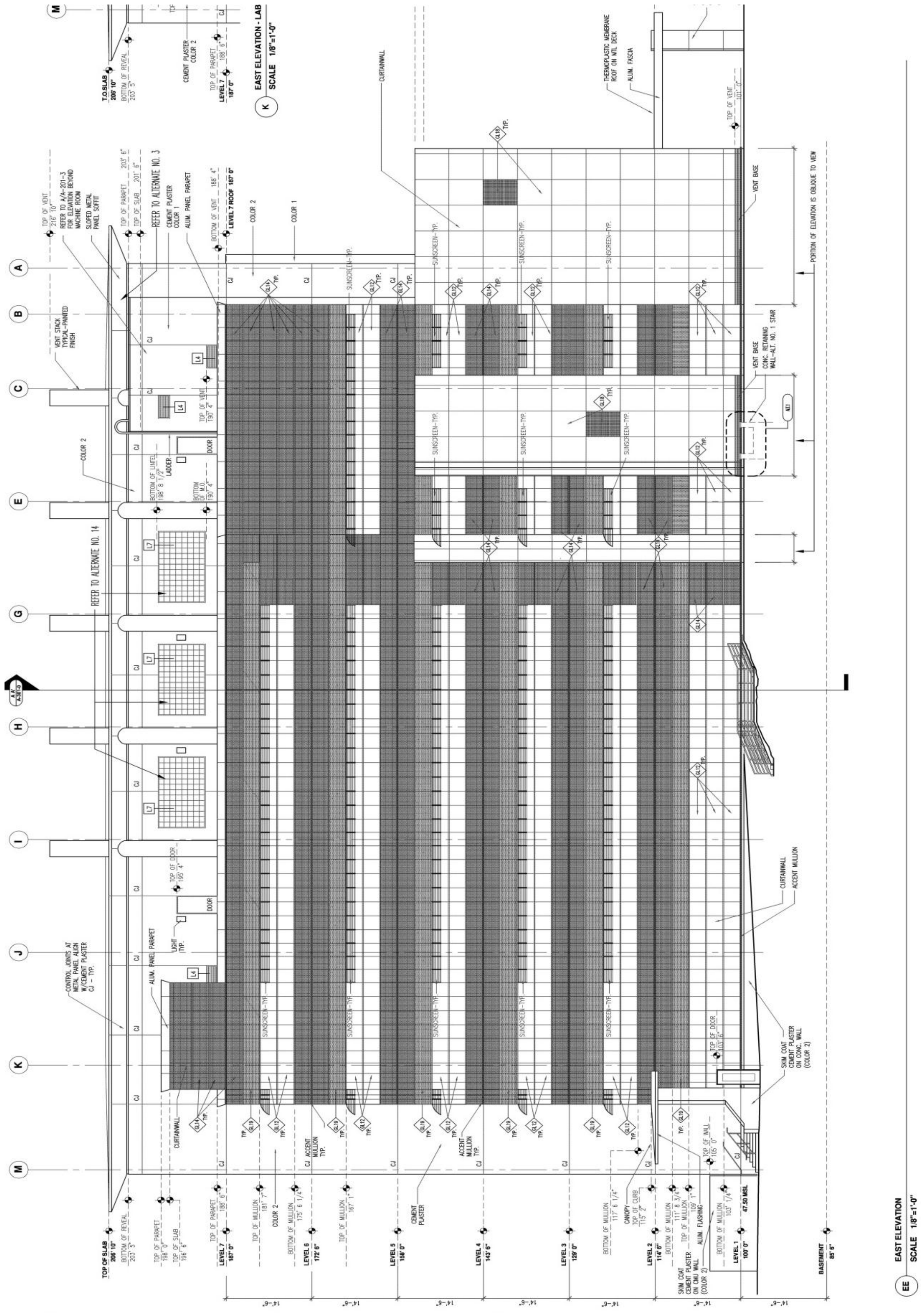


Figure 23 - **Elevation of the building showing the different floor heights from A -201- 0**

Appendix B: Existing: Precast Joists and Soffit Beams

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		SPAN															System Weight PSF
		SIZE	SPACING	18	20	22	24	26	28	30	32	34	36	38	40	42	44	46	
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

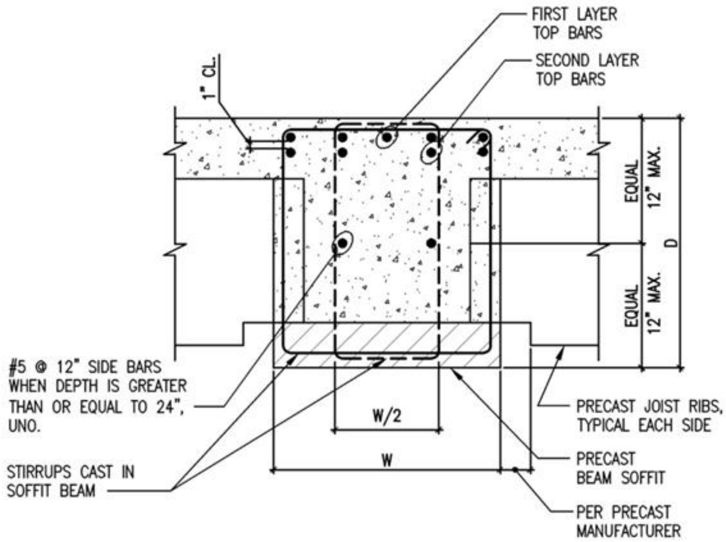
PRESTRESS SYSTEMS OF FLORIDA, INC. 16603 OLD US 41 FORT MYERS, FLORIDA 33912 PHONE: 239-437-0660 www.psfjoist.com FAX 239-437-0697																			
FIRE	DECK	JOIST		SUPERIMPOSED LOAD CAPACITY SPAN															System Weight PSF
		SIZE	SPACING	40	42	44	46	48	50	52	54	56	58	60	62	64	66	68	
2 HOUR	4 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

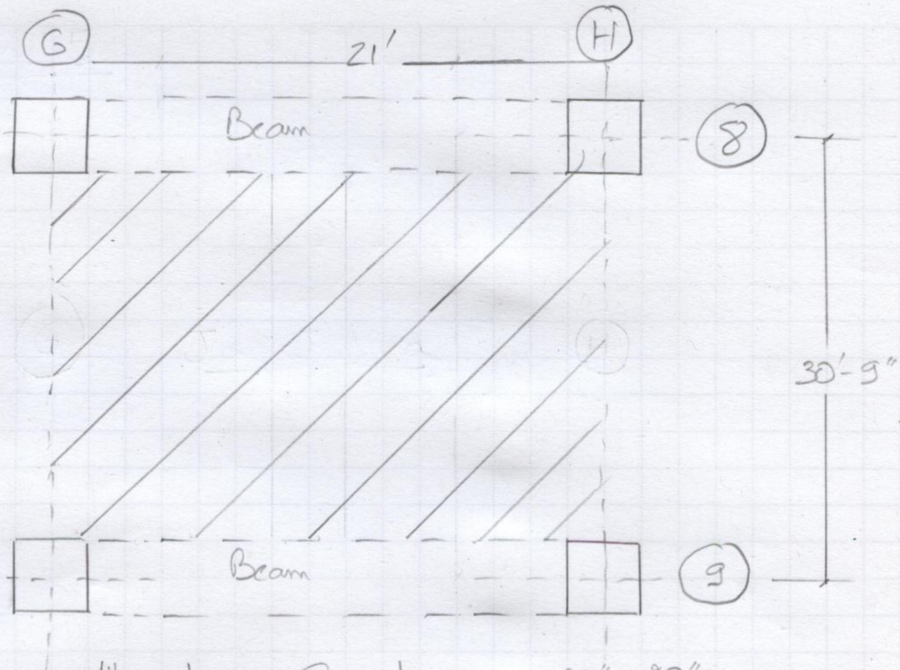
16" JOIST WITH 3" COMPOSITE SLAB (P.S.F.)								
Joist Spacing	DESIGN SPAN (Feet)							
	26	28	30	32	34	36	38	40
3'-6 1/4"			282	253	222	196	172	150
4'-6 1/4"			212	190	170	150	132	114
5'-6 1/4"	200	184	168	150	132	115	101	87
6'-6 1/4"	166	152	138	122	107	93	80	68



2 TYPICAL BEAM SECTION

HDR | ONE COMPANY *Many Solutions*

Project: Tech 1		Computed:	Date:
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

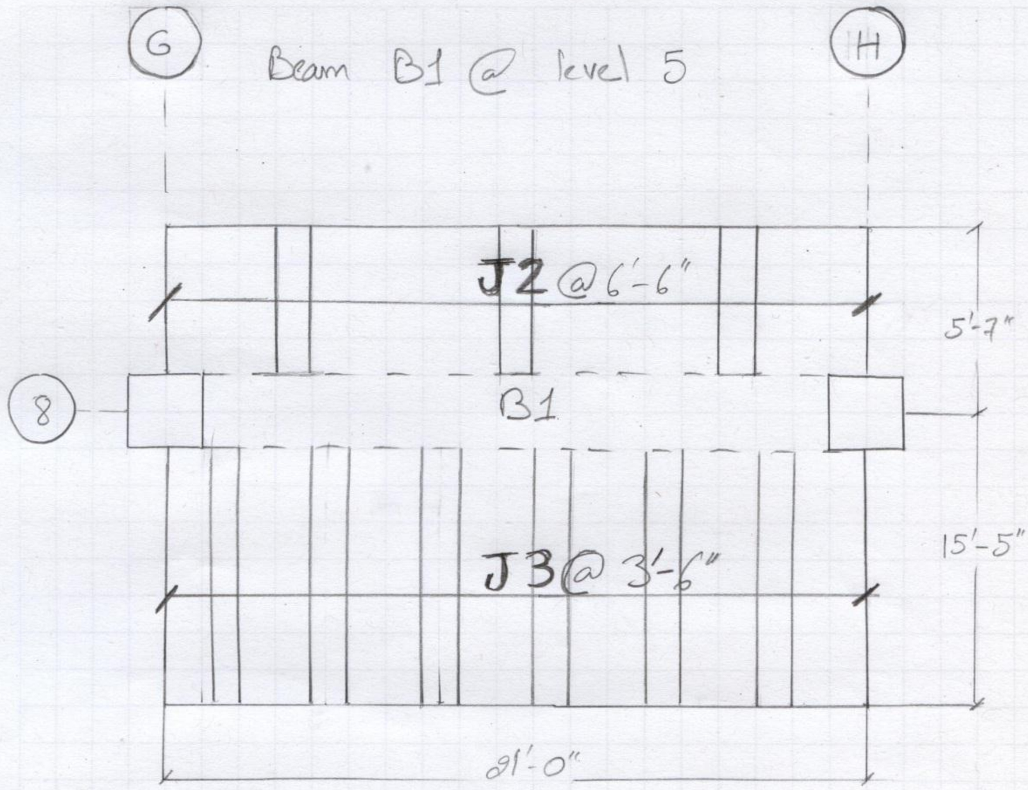
D+L = 125 + 96.5 = 221.5

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 or J3 would have a capacity 253 psf thus for a span of 31' a J3 @ 3'-6" would suffice

HDR | ONE COMPANY *Many Solutions*

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

Beam B1 @ level 5



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

Project:		Computed:	Date:
Subject:		Checked:	Date:
Task:	Beam check	Page: 2	of: 3
Job #:		No:	

HDR | ONE COMPANY *Many Solutions*

short direction : $(5'\text{-}7'')(67) = 0.374$ KlF

long direction : $(15'\text{-}5'')(84) = 1.295$ KlF

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

total live : 125 psF

reduced by 20% see provisions

$100 \text{ psf}(21') = 2.1$ KlF

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

$\omega_u = 7.795$ KlF

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

Positive moments for interior spans.

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

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

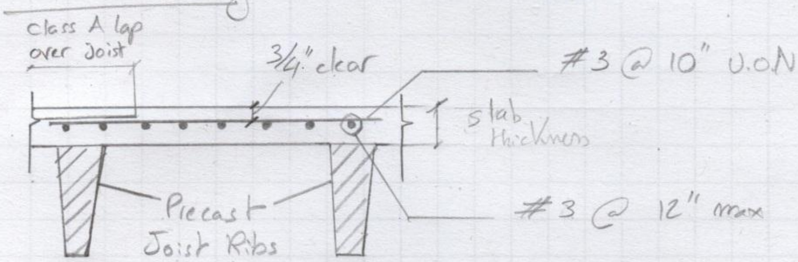
w/ 2 #9 Full Length
3 #9 @ Right End

$V_{u\,right} = 105$

HDR | ONE COMPANY *Many Solutions*

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 that's 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:
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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: $M_u = \dfrac{-w_u l_n^2}{10}$

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

$$= 6{,}233 \text{ lb-ft/ft}$$

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

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

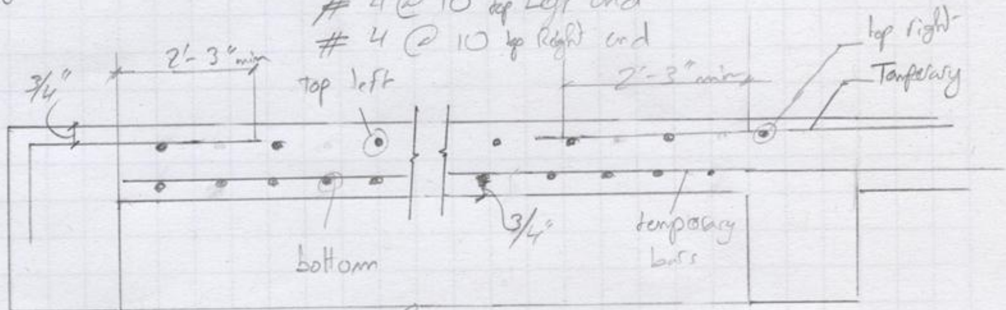
the second controls,

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

$\Rightarrow 23 \times 2 \times 0.2 = 9.2 \text{ in}^2$

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Project: Computed: Date:
Subject: Checked: Date:
Task: slab check Page: 3 of: 3
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$$A_s \text{ (per foot)} = \frac{9.2}{19'.33'} = 0.476 \text{ in}^2/\text{ft}$$

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

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

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

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

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

Check for shear:

for end members @ face of first support

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

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

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

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

Appendix C: Composite Steel Calculations

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Project: Tech 2		Computed:	Date:
Subject: Composite steel beam		Checked:	Date:
Task: with composite metal deck		Page: 1	of: 12
Job #:		No:	

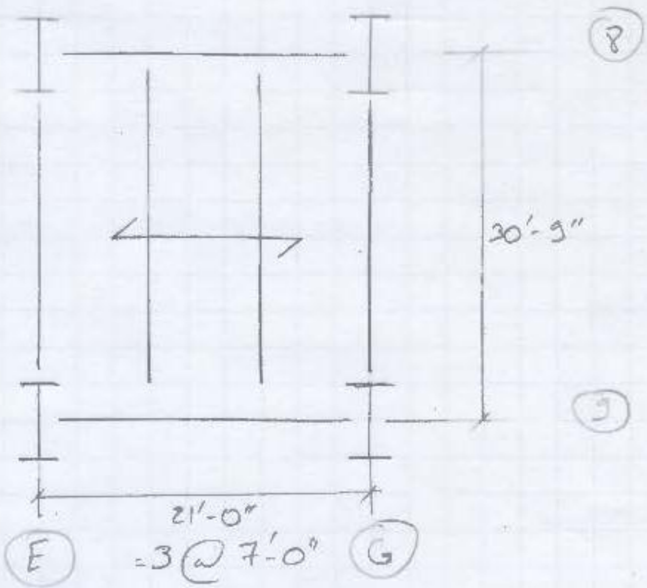
COMPOSITE STEEL:

Used Vulcraft Manual for deck (v. 2008)

2 hour fire rating for project thus a 4 1/2" N.W. concrete topping or fire proofing spray should be used

Not changing span lengths, design for 3 span minimum condition

TYPICAL BAY



b_{eff} beam = $\dfrac{30.75 \times 12}{8} + \frac{1}{2}(7')(12) = 88.125$

min $\dfrac{30.75 \times 12}{8}, \frac{1}{2}(7)(12) = 88.125$

$\Rightarrow b_{eff\ beam} = 88.1$

b_{eff} girder = $\dfrac{21 \times 12}{8} + \frac{1}{2}(30.75)(12) = 216$

min $\dfrac{21 \times 12}{8} + (2'3)(12) = 59.1$

$\Rightarrow b_{eff}$ girder $= 59.1$

Loods: Super imposed dead = 20psf
Live load = 125 psf
Total = 145 psf
w/ 4.5" N.W concrete for fire resisting purposes (2HR)

⇒ use 2 VL 20 with max 3 span = 8'-4" > 7'-0" ok!

@ 7'-0" 337 psf > 145 psf
Deck weight = 1.97 psf
total weight = 69 psf , height = 6.5

The reason for over designing the deck is for vibration purposes that will be checked accordingly for sensitive equipment later.

Design of beams:

height consideration: 14'-6" } Max depth = 5'-5" (12")
ceiling height : 9'-1" } − 6.5"
59.5"

Assume simply supported ⇒ No issue size
Add 5psf for self-weight smalled to fit mechanical equipment

$w_D = (20 + 69 + 5)(7') = 658$ lb/ft.

$LL_r = 0.25 + \frac{15}{\sqrt{(2)(30.75)(7)}} = 0.97$ ⇒ assume No reduction close to 1.0

$w_L = (125)(7) = 875$ lb/ft

$w_u = 1.2D + 1.6L = 1.2(.658) + 1.6(.875)$
$= 2.19$ K/ft

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Project:		Computed:	Date:
Subject: Composite steel		Checked:	Date:
Task:		Page: 3	of: 12
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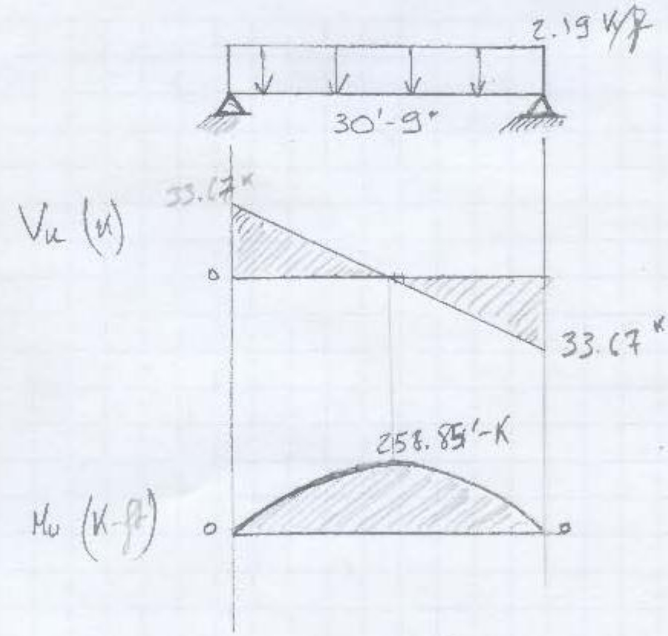

$V_u = 2.19\left(\frac{30.75}{2}\right) = 33.67$

$M_u = \frac{w\ell^2}{8} = \frac{2.19\,(30.75)^2}{8}$

$M_u^+ = 258.85'\text{-K}$

$\underline{Q_n}$: Assume 1 stud/Rib, 3/4" ⌀ studs, $f'_c = 4000$ psi

Deck ⊥ to beam, weak position

$Q_n = 17.2^K$ (per AISC table 3-21)

<u>Deflections</u>:

Δ_{LL} allowance $= \frac{\ell}{360} = \frac{(30.75)(12)}{360} = 1.025"$

$\Delta_{L} = \frac{5 w_L \ell^4 \,(1728)}{384\, E\, I_B} \Rightarrow I_B = \frac{5 w_L \ell^4\,(1728)}{384\, E\, \Delta_{LL}}$

$I_B = \frac{5\,(.875)(30.75)^4(1728)}{384 \cdot 29{,}000 \cdot 1.025} = 592.2 \text{ in}^4$ <u>minimum</u>

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Project:		Computed:	Date:
Subject: Composite steel		Checked:	Date:
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Assume $a = 1'' \Rightarrow 6.5'' - \left(\frac{1''}{2}\right) = 6''$

TRIAL SIZES USING AISC TABLE 3-19 & 3-20:

W 18x40 Most eco for $I_x = 612 > 592.2 \text{ in}^4$

$\Sigma Q_n = 147 \qquad \phi M_n = 433'\text{-K} \quad \Rightarrow \#\text{ of studs} = \frac{147}{17.2}$

Economy: $40(30.75) + 10(10) = 1330$ lbs of steel $\qquad N = 8.5 \rightarrow 10$

$$\Delta_{LL} = \frac{5(.875)(30.75)^4(1728)}{384 \times 29{,}000 \times 612} = 0.992'' < 1.025'' \text{ OK!}$$

$$\Delta_{TL} = \frac{5(.658 + .875)(30.75)^4(1728)}{384 \times 29{,}000 \times 612} = 1.74'' < \frac{l}{240} = 1.5$$

$\Rightarrow$ N.G

TRY 21x44

$$\Delta_{TL} = \frac{5(.658 + .875)(30.75)^4(1728)}{384 \times 29{,}000 \times 843} = 1.263 \rightarrow \text{OK!}$$

Unshored strength: W 21x44 $\quad \phi_b M_p = 358'\text{-K}$

$W_u = 1.4(69\text{ psf})(7') + 1.4(44) = 737.8 \text{ plf}$

Or $W_u = 1.2(69)(7) + 1.2(44) + 1.6(20)(7) = 961.8 \text{ plf}$

$$M_u = \frac{0.962(30.75)^2}{8} = 113.7 < 358'\text{-K} \Rightarrow \text{OK!}$$

HDR ONE COMPANY *Many Solutions*			
Project:		Computed:	Date:
Subject: Composite steel		Checked:	Date:
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Wet concrete deflection:

$W_{wc} = 69(7') + 44 = .527$ KF

$\Delta_{wc} = \dfrac{5(.527)(30.75)^4(1728)}{384(29{,}000)(843)} = 0.433'' <$ W_{wc} allowable

$\Rightarrow$ OK!

use $\boxed{21 \times 44 \text{ w/ } 10 \text{ studs}}$

still need to check for vibration

Design of girder:

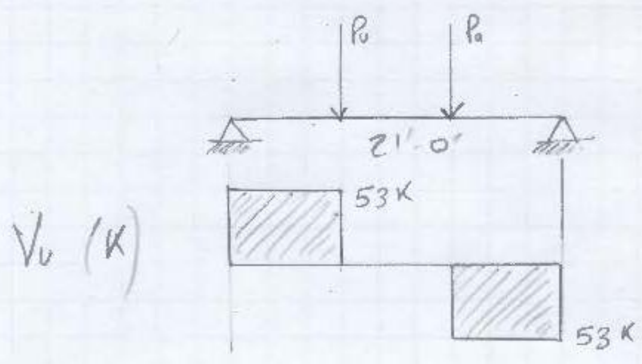
Assume simply supported
Add 1^K to each point load for girder self-weight

$P_D = (0.658)(30.75) = 20.23^K$

$LL_r = 0.25 + \dfrac{15}{\sqrt{2(30.75)(21)}} = 0.667$

$P_L = (0.667)(0.875)(30.75) = 17.95^K$

$P_u = 1.2 P_D + 1.6 P_L = 52.99 = 53^K$



$V_u = \dfrac{(2)(53)}{2} = 53^K$

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Project: Computed: Date:
Subject: Composite steel Checked: Date:
Task: Page: 6 of: 12
Job #: No:

M_u (ft-K)

$M_u = (7')(53) = 371'\text{-K}$

Assume:

Q_n: // to girder, weak position, 1 stud/rib, 3/4" ∅ stud

$\frac{w_r}{h_r} = \frac{5}{2} = 2.5 > 1.5$, $f'_c = 4{,}000$, NW

$\Rightarrow Q_n = 21.5$ (from t. 3-21 AISC)

$\Delta_{LL} = \frac{\ell}{360} = \frac{(21 \times 12)}{360} = 0.7'$

$\Delta_L = \frac{P\ell^3}{28EI}$ for $a = \frac{\ell}{3}$ ⇒ OK.

$I_{LB} = \frac{PL^3}{28\,E\,\Delta_{LL}} = \frac{(17.35)(21)^3(1728)}{(28)(29{,}000)(0.7)} = 505 \text{ in}^4$ min

TRIAL SIZES:

table 3-19 & 3-20: $a = 1"$ ⇒ $Y_2 = 6"$

W 18x35 w/ $\Sigma Q_n = 194$ $\phi M_n = 419'\text{-K}$

$N = \frac{194}{21.5} = 9 \approx 10$ → weight $= 35(21) + 10(10) = 835$ lbs

W 16x26 w/ $\Sigma Q_n = 337$ $\phi M_n = 377'\text{-K}$

$N = \frac{337}{21.5} = 16$ → weight $= 26(21) + 16(10) = 706$ lbs

$I < I_{min}$ ⇒ N.G

W 18x40 w/ $\Sigma Q_n = 147$ $\phi M_n = 433'\text{-K}$

$N = \frac{147}{21.5} \Rightarrow 7$ → weight $= 40(21) + 7(10) = 910$ lbs

Project:		Computed:	Date:
Subject:	Composite steel	Checked:	Date:
Task:		Page: 7	of: 12
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Unshored issue :

$P_u = \left[1.2\,(69 \times 7' + W) + 1.6\,(20 \times 7')\right] 21'$

$P_u = [579.6 + 1.2W + 224]\,21 = \dfrac{16{,}876 + 25.2W}{1000\ lb/K}$

$M_u = [16.876 + 0.0252W](7')$

For $18 \times 40 \Rightarrow M_u = 144.3'\text{-}K < 294'\text{-}K \Rightarrow$ OK!

$\Delta_{LL} = \dfrac{(17.95)(21)^3(1728)}{28\,(29{,}000)(612)} = 0.577'' < 0.7'' \Rightarrow$ OK!

$\Delta_{TL} = \dfrac{(21)(12)}{240} = 1.05''$

$\Delta_{TL} = \dfrac{(20.23 + 1 + 17.95)(21)^3(1728)}{28\,(29{,}000)(612)} = 1.26'' \Rightarrow$ N.G

$\Rightarrow$ Use $W\,21 \times 44 \Rightarrow \Delta_{TL} = 0.93'' \Rightarrow$ OK!

works for unshored too!

$\Rightarrow W\,21 \times 44$ w/ 8 studs

Design interior column :

$P_u = 2(53) + 2(33.17) = 173^K$ @ $14'\text{-}8''$ unbraced length

$K = 2.0$ (conservative approach)

$KL = 29' \Rightarrow$ from AISC table 4-1, $W\,12 \times 58$ or $\boxed{W\,14 \times 61}$ work page: 8/12

From Steel design guide series (11) - Floor vibrations due to human activity (1997)

Vibration check Calculation for the Composite frame calculated by hand to meet 2000 u-in/sec for labs.

Steel Design Guide 11 - Table 4.1

	Building Type and usage	Constant Force P_o	Damping Ratio β	Acceleration Limit $a_o/g \times 100\%$
1	Offices, Residences, Churches	65	.02-.05*	0.5%
2	Shopping Malls	65	0.02	1.5%
3	Footbridges - Indoor	92	0.01	1.5%
4	Footbridges - Outdoor	92	0.01	5.0%

Recommended Values of Parameters in Equation (4.1) and ao / g limits

* 0.02 for floors with few non-structural components (ceilings, ducts, partitions, etc.) as can occur in open work areas and churches
0.03 for floors with non-structural components and furnishings, but with only small demountable partitions, typical of many modular office areas
0.05 for full height partitions between floors

Input

Type of building from table 4.1: (page 1) 1 — Building type 1 is used as worst case scenario for labs

Design Vibrational Velocity= 2000

Vibration Characteristics of Interior Bay of Structural Steel framing with composite concrete slab

Steel Beam Properties					Deck Properties			
Left Girder:	W21X44	A =	13	in^2	Concrete	wc =	145	pcf
Length:	21 ft	Ix =	843	in^4		f_c' =	4000	psi
		d =	20.66	in	Composite Slab above deck =		4.5	in
Right Girder	W21X44	A =	13	in^2				
Length	21 ft	Ix =	843	in^4				
		d =	20.66	in	Metal Deck depth =		2	in
Beam:	W21X44	A =	13	in^2	Gage=		20	
Length:	30.75 ft	Ix =	843	in^4	Slab weight =		69.0	psf
Spacing:	7 ft	d =	20.66	in	Metal Deck weight =		1.97	psf
					Slab + Deck Weight =		71.0	psf

Actual Live Load = 11 psf
Actual SDL = 4 psf

Effective Slab Width = 84 inches
Uniform Dist. Load = 545.79 plf

Beam Mode Properties

Ec = $w^{1.5}$*sqrt(fc')		3492 ksi
n = modular ratio = Es/1.35Ec =		6.15
Composite Transformed Inertial Axis, y =		0.296 in
Moment of Inertia of Composite section, I_j =		3228 in^4
Deflection under actual Dead + Live Load, Δ_j =		0.139 in
Beam mode fundamental Frequency =		9.49 Hz
Transformed slab moment of inertia per unit width, Ds =		27.05 in^4 / ft
Transformed beam moment of inertia per unit width =		461.09 in^4 / ft
Cj= 2.0	2.0 for joists or beams in most areas; 1.0 for joists or beams parallel to an interior edge	
Effective beam panel width is min of		30.266
2/3 of 3 times the girder span for an interior bay		42 ft → 30.27 ft
factor of continuity 1.5	1.5 for Interior Bay	
Effective weight of beam panel accounting for continuity =		129 kips

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Girder Mode Properties

Left Girder

Effective Slab width = 100.8 in
Slab width Meets requirements
Average Concrete depth = 5.5 in
Composite Transformed Inertial Axis, y = -0.975 in
Moment of Inertia of Composite section, Ig = 3323 in^4
Equivalent uniform loading = 2880.86 plf
Deflection under actual Dead + Live Load, Δ_j = 0.131 in
Girder Mode fundamental Frequency = 9.78 Hz
Transformed beam moment of inertia per unit width, Dj = 461.09 in^4 / ft
Transformed girder moment of inertia per unit width, Dg = 108.05 in^4 / ft
Cg= 1.8 | 1.6 for girders supporting joists connected to girder flange
1.8 for girders supporting beams connected to girder web
Effective width for girder panel mode = min of 54.33 | 54.33 ft
2/3 of 3 times the girder span for an interior bay 61.50
Girder Panel Weight, Wg = 107 kips
Check before reducing Δg and using $\Delta g'$ | Ok to use formula

Maximum deflection of girder due to weight, Δ_g' = 0.091 in

Right Girder

Effective Slab width = 100.8 in
Slab width Meets requirements
Average Concrete depth = 5.5 in
Composite Transformed Inertial Axis, y = -0.975 in
Moment of Inertia of Composite section, Ig = 3323 in^4
Equivalent uniform loading = 2880.86 plf
Deflection under actual Dead + Live Load, Δ_j = 0.131 in
Girder Mode fundamental Frequency = 9.78 Hz
Transformed beam moment of inertia per unit width, Dj = 461.09 in^4 / ft
Transformed girder moment of inertia per unit width, Dg = 108.05 in^4 / ft
Cg= 1.8 | 1.6 for girders supporting joists connected to girder flange
1.8 for girders supporting beams connected to girder web
Effective width for girder panel mode = 54.33 | 54.33 ft
2/3 of 3 times the girder span for an interior bay 61.50
Girder Panel Weight, Wg = 107 kips
Check before reducing Δg and using $\Delta g'$ | Ok to use formula

Maximum deflection of girder due to weight, Δ_g' = 0.091 in

Taking average of both girders to calculate a uniform $\Delta g'$ in the floor frequency 0.091

Floor Fundamental Frequency, f_n = 7.38 Hz
W = 120 kips
For office occupancy without full height partitions, β = 0.03
Acceleration Limit = 0.5 % g
βW = 3.60 kips
Po= 65
ap/g = 0.14 % g
Satisfactory

Table 6.2 Values of Footfall Impulse Parameters		
Walking Pace Steps/minute	f_o	Uv (lb.Hz^2)
100 (fast)	5	25000
75 (moderate)	2.5	5500
50 (slow)	1.4	1500

Deflection factors to be used is 1/ 96 | 48 for simple span beams
96 for beams with built-in ends

Mid-span flexibility of beam= Deloj= 5.59E-06 in /lb
Mid-span flexibility of left girder= DelgP= 1.73E-06 in /lb
Mid-span flexibility of right girder= DelgP= 1.73E-06 in /lb
The effective number of tee-beams= Neff= max of 2.77 | 1 | 2.77

Mid-bay flexibility= DelP= 2.88E-06 in /lb

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Maximum expected velocity(50 step/min.)	V=	5.86E-04 in/sec	586 u-in/sec OK
f_n/f_{o} 5.272 >1			
Maximum expected velocity(75 step/min.)	V=	2.15E-03 in/sec	2149 u-in/sec NG → > 2000
f_n/f_{o} 2.952 >1			
Maximum expected velocity(100 step/min)	V=	9.77E-03 in/sec	9766 u-in/sec NG
f_n/f_{o} 1.476 >1			

Re-check framing system

The framing used does not pass the moderate level for 2000 u-in thus a new framing system will be chosen that meets the required vibrations for laboratories.

Input

Type of building from table 4.1: (page 1) 1

Design Vibrational Velocity= 2000

Vibration Characteristics of Interior Bay of Structural Steel framing with composite concrete slab

Steel Beam Properties					Deck Properties			
Left Girder: **W21X62**	A =	18.3	in^2		Concrete:	wc =	**145**	pcf
Length **21 ft**	Ix =	1330	in^4			f'_c =	**4000**	psi
	d =	20.99	in		Composite Slab above deck =		**4.5**	in
Right Girder **W21X62**	A =	18.3	in^2					
Length **21 ft**	Ix =	1330	in^4					
	d =	20.99	in		Metal Deck depth =		**2**	in
Beam **W18X55**	A =	16.2	in^2		Gage=		**20**	
Length **30.75 ft**	Ix =	890	in^4		Slab weight =		69.0	psf
Spacing **7 ft**	d =	18.11	in		Metal Deck weight =		**1.97**	psf
					Slab + Deck Weight =		71.0	psf

Actual Live Load = **11** psf
Actual SDL = **4** psf

Effective Slab Width = 84 inches
Uniform Dist. Load = 656.79 plf

Beam Mode Properties

Ec = $w^{1.5}$*sqrt(fc')	3492 ksi
n = modular ratio = Es/1.35Ec =	6.15
Composite Transformed Inertial Axis, y =	0.526 in
Moment of Inertia of Composite section, Ij =	3263 in^4
Deflection under actual Dead + Live Load, Δ_j =	0.140 in
Beam mode fundamental Frequency =	9.46 Hz
Transformed slab moment of inertia per unit width, Ds =	27.05 in^4 / ft
Transformed beam moment of inertia per unit width =	466.16 in^4 / ft

Cj= 2.0 | 2.0 for joists or beams in most areas
1.0 for joists or beams parallel to an interior edge

Effective beam panel width is min of: 30.183 / 42 ft | 30.18 ft
2/3 of 3 times the girder span for an interior bay
factor of continuity 1.5 | 1.5 for Interior Bay

Effective weight of beam panel accounting for continuity = 131 kips

Girder Mode Properties

Left Girder

Effective Slab width =	100.8 in
	Slab width Meets requirements
Average Concrete depth =	5.5 in
Composite Transformed Inertial Axis, y =	-0.346 in
Moment of Inertia of Composite section, Ig =	4644 in^4
Equivalent uniform loading =	2947.18 plf
Deflection under actual Dead + Live Load, Δ_j =	0.096 in
Girder Mode fundamental Frequency =	11.43 Hz
Transformed beam moment of inertia per unit width, Dj =	466.16 in^4 / ft

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Cg=	1.8	1.6	for girders supporting joists connected to girder flange	
		1.8	for girders supporting beams connected to girder web	
Effective width for girder panel mode = min of				50.10 / 61.50 → 50.10 ft
	2/3 of 3 times the girder span for an interior bay			
Girder Panel Weight, Wg =				101 kips
	Check before reducing Δg and using $\Delta g'$		Ok to use formula	
Maximum deflection of girder due to weight, Δ_g' =				0.067 in

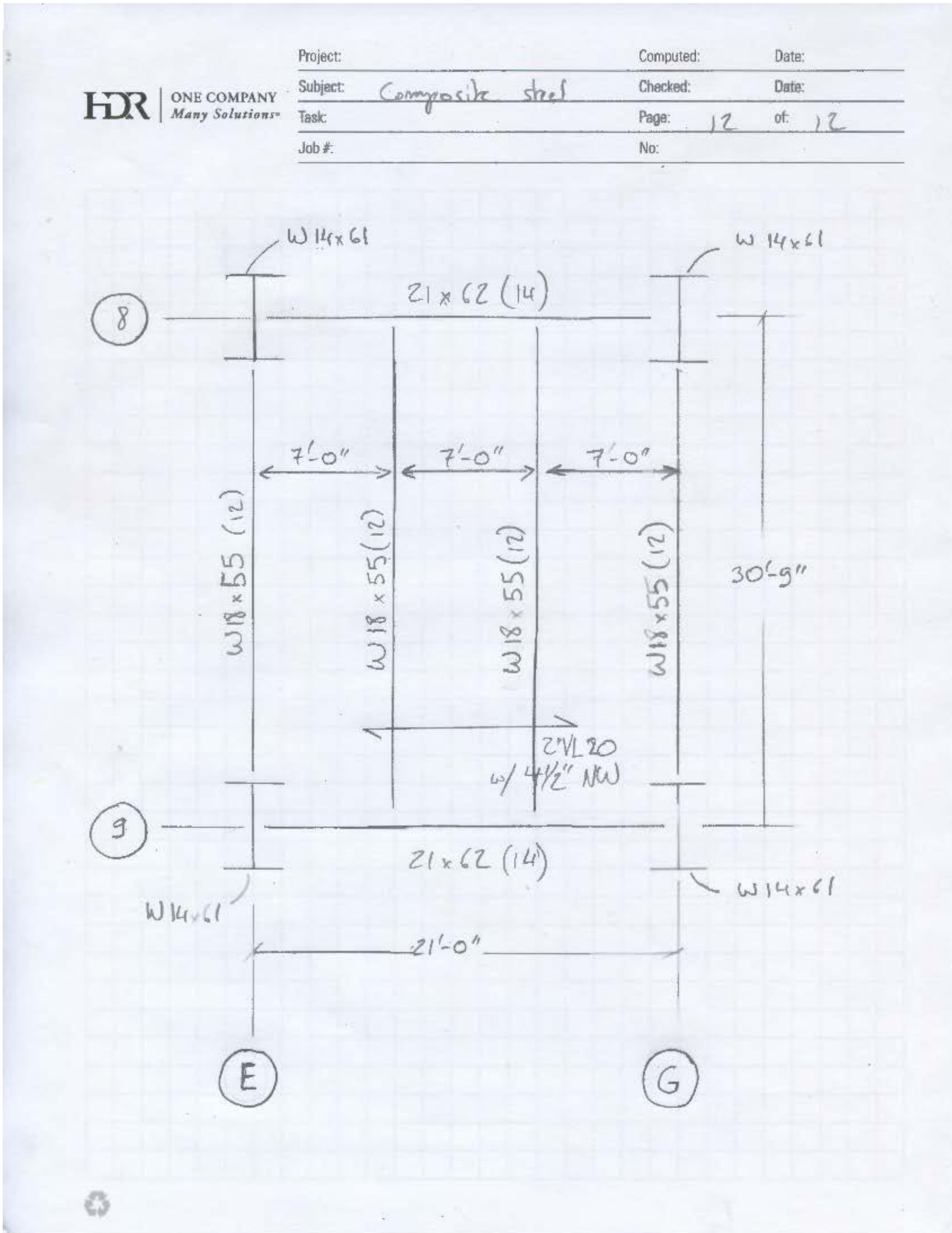
Right Girder

Effective Slab width =			100.8 in
		Slab width Meets requirements	
Average Concrete depth =			5.5 in
Composite Transformed Inertial Axis, y =			-0.346 in
Moment of Inertia of Composite section, Ig =			4644 in^4
Equivalent uniform loading =			2947.18 plf
Deflection under actual Dead + Live Load, Δ_j =			0.096 in
Girder Mode fundamental Frequency =			11.43 Hz
Transformed beam moment of inertia per unit width, Dj =			466.16 in^4 / ft
Transformed girder moment of inertia per unit width, Dg =			151.02 in^4 / ft
Cg= 1.8	1.6	for girders supporting joists connected to girder flange	
	1.8	for girders supporting beams connected to girder web	
Effective width for girder panel mode =			50.10 / 61.50 → 50.10 ft
2/3 of 3 times the girder span for an interior bay			
Girder Panel Weight, Wg =			101 kips
Check before reducing Δg and using $\Delta g'$		Ok to use formula	
Maximum deflection of girder due to weight, Δ_g' =			0.067 in
Taking average of both girders to calculate a uniform $\Delta g'$ in the floor frequency			0.067
Floor Fundamental Frequency, f_n =			7.79 Hz
		W =	121 kips
For office occupancy without full height partitions, β =			**0.03**
Acceleration Limit =			**0.5** % g
		βW =	3.63 kips
		Po=	65
		ap/g =	0.12 % g
		Satisfactory	

Table 6.2 Values of Footfall Impulse Parameters		
Walking Pace Steps/minute	f_0	Uv (lb.Hz^2)
100 (fast)	5	25000
75 (moderate)	2.5	5500
50 (slow)	1.4	1500

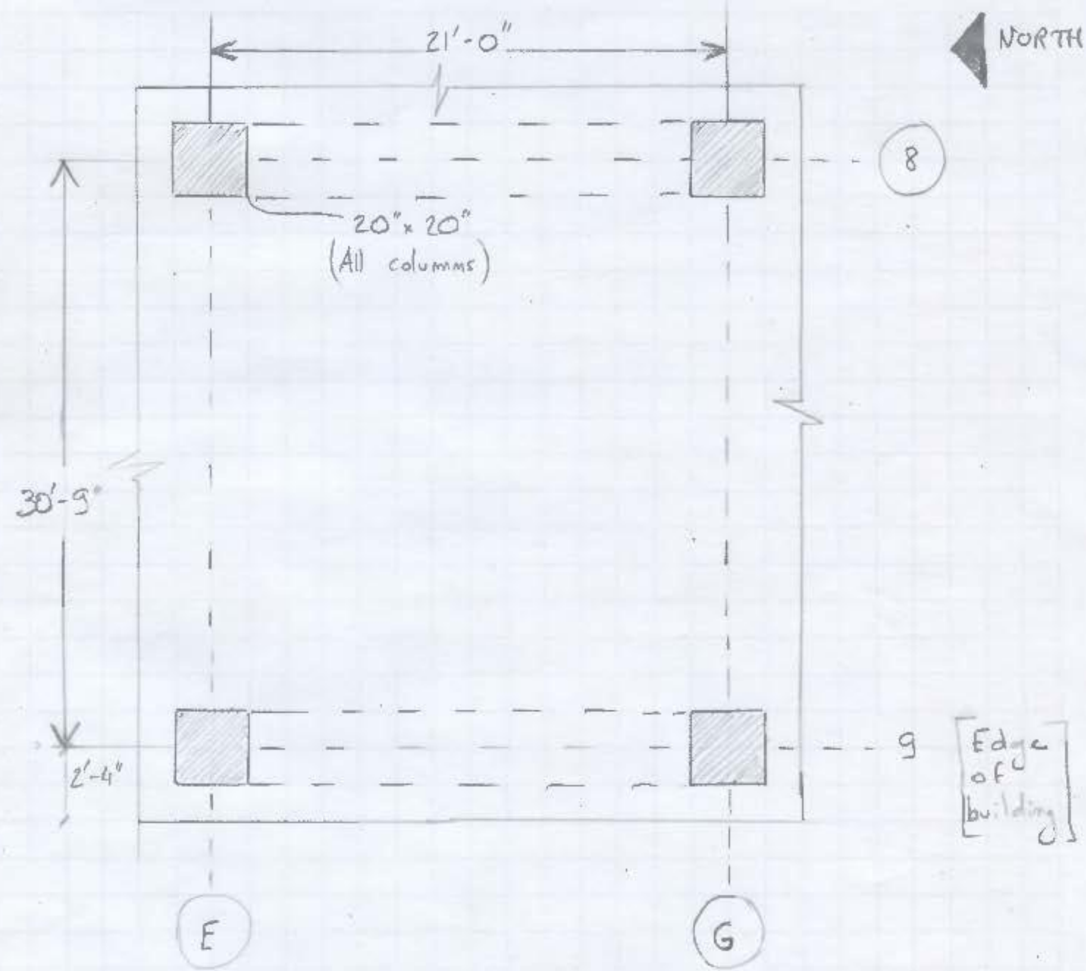
Deflection factors to be used is 1/		96	48	for simple span beams
			96	for beams with built-in ends
Mid-span flexibility of beam=	Deloj=		5.53E-06 in /lb	
Mid-span flexibility of left girder=	DelgP=		1.24E-06 in /lb	
Mid-span flexibility of right girder=	DelgP=		1.24E-06 in /lb	
The effective number of tee-beams= Neff= max of		2.77 / 1	2.77	
Mid-bay flexibility=	DelP=		2.62E-06 in /lb	
Maximum expected velocity(50 step/min.)	V=		5.04E-04 in/sec	504 u-in/sec OK
f_n/f_0= 5.562 >1				
Maximum expected velocity(75 step/min.)	V=		1.85E-03 in/sec	1848 u-in/sec OK < 2000
f_n/f_0= 3.115 >1				
Maximum expected velocity(100 step/min)	V=		8.40E-03 in/sec	8399 u-in/sec NG
f_n/f_0= 1.557 >1				

→ OK to use


Project:
Computed:
Date:
HDR ONE COMPANY Many Solutions
Subject: Composite steel
Checked:
Date:
Task:
Page: 12
of: 12
Job #:
No:
W 14x61
W 14x61
21 x 62 (14)
8
7'-0"
7'-0"
7'-0"
W18 x 55 (12)
W18 x 55 (12)
W18 x 55 (12)
W18 x 55 (12)
30'-9"
2"VL 20
w/ 4 1/2" NW
9
21 x 62 (14)
W 14x61
W14x61
21'-0"
E
G

Appendix D: Flat Plate Calculations

Project:		Computed:	Date:
Subject:	Flat plate w/ mild	Checked:	Date:
Task:	reinforcing.	Page: 1	of: 12
Job #:		No:	



$F'_c = 4{,}000$ psi story height: $h = 14'-6"$

$F_y = 60{,}000$ psi w_L L = 125 psf

Even though this in an exterior bay, it was chosen as typical since all bays are irregular in the East-West but one regular in the North-South. This is a typical lab bay.

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Project:		Computed:	Date:
Subject: flat plate		Checked:	Date:
Task:		Page: 2	of: 12
Job #:		No:	

Note:

The direct design method will be chosen to design this typical bay even though it does not match all the requirements. However, for simple calculation in a schematic design this step will be done.

Using ACI 318-08, table 9.5(c)

Minimum thickness of slabs without interior beams and without drop panels, for exterior panel, without edge beam

$$t_{min,\ mild\ reinforcement} = \frac{\ell_n}{30} = \frac{\left[(30.75') - \left(\frac{20''}{12}\right)\right] \times 12}{30} = 11.63'' \Rightarrow \text{use } 12'' \text{ for constructibility and punching shear}$$

Direct Design Method:

1. 3 continuous spans in each direction X

does not apply in the E-W direction but see Note above

2. Panel ratio ≤ 2 $\quad \frac{\ell_2}{\ell_1} = \frac{30.75}{21} = 1.46 < 2 \Rightarrow \underline{OK!}$

3. $\ell_1 \geq \frac{2}{3}\ell_2$ $\quad \ell_1 = 21' \geq \frac{2}{3}(30.75) = 20.5' \Rightarrow OK!$

4. Can't have a column offset of more than 10% length $\Rightarrow OK!$

5. $w_L \leq 2 w_D$ $\qquad w_L = 125$ psf

$w_D = \left(\frac{11''}{12''}\right)(150\ \text{lb/ft}^3) = 137.5$ psf
$w_{DSI} = 20$ psf

$\Big\}\ w_D = 157.5$ psf $\Rightarrow OK$

$\Rightarrow$ OK to use Direct design method.

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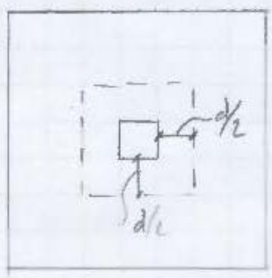
Project:		Computed:	Date:
Subject:	Flat plate	Checked:	Date:
Task:		Page: 3	of: 12
Job #:		No:	

Punching shear (two-way) action :

$V_u = W_u \times \text{Area}$

$V_u = .389 \times \left[(21')(30.75) - \left(\frac{20 + \overset{d_{avg}}{10.5}}{144}\right)^2 \right] = 248.6\ \text{K}$

shear strength :



$b_o = (20 + 10.5)\,4 = 122''$

$\frac{b_o}{d} = \frac{122}{10.5} = 11.62$

$\beta_c = 1 < 2$ for equivalent square

$V_c = \left[\left(\frac{\alpha_s}{b_o/d}\right) + 2\right] \sqrt{f'c}\ b_o \cdot d$ (controls)

$\alpha_s = 40$ for interior columns

$V_c = \left(\frac{40}{11.62} + 2\right) \sqrt{4{,}000}\ (122)(10.5) = 440.9\ \text{K}$

$\phi V_c = 0.75\,(440.9) = 330\ \text{K} > 248.6\ \text{K}$

$\Rightarrow$ ok!

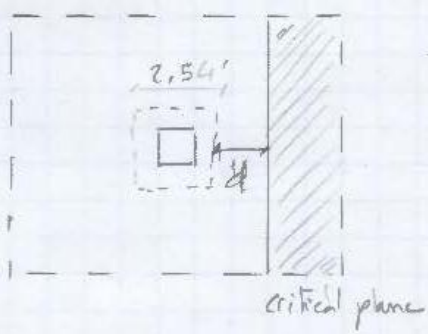
For edge columns , $\alpha_s = 30$

$\phi V_c = .75\,(371.2) = 278.4 > 248.6\ \text{K}$

$\rightarrow$ OK!

Project:		Computed:	Date:
Subject: flate plate		Checked:	Date:
Task:		Page: 4	of: 12
Job #:		No:	

Wide beam action:



$d_{avg} = 10.5''$

$x = 21/2 - 2.54' - \left(\frac{10.5}{12}\right)$

$x = 7.08'$

$V_n = V_c = 2\sqrt{f'_c}\, b_w\, d$

$= 2\sqrt{4000}\,(30.75 \times 12)(10.5)$

$V_n = 490.1\text{K}$

$\phi V_n = 0.75 \times 490.1 = 367.6$

$V_u = w_u \times 7.08 \times (30.75) = 0.389 \times 7.08 \times 30.75$

$V_u = 84.69$

$\phi V_n > V_u \Rightarrow$ OK!

Same for long direction since the one calculated is the worst case scenario.

The slab depth passes both shear failures thus continue with $t = 12''$ for Direct Design Method.

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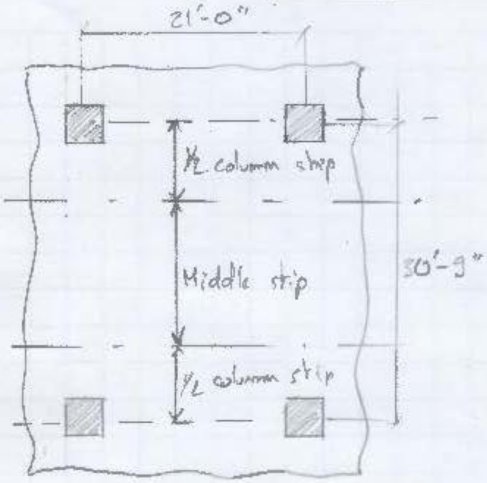
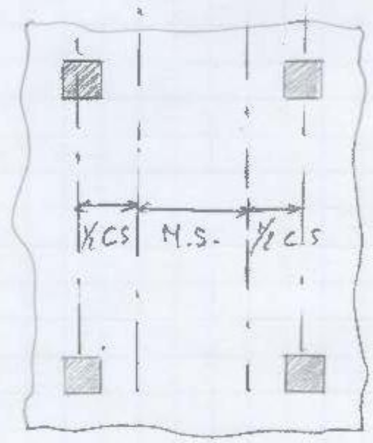
Project: Computed: Date:
Subject: flat plate Checked: Date:
Task: Page: 5 of: 12
Job #: No:





Frame A

$\frac{1}{2}$ column strip $= \frac{30'\text{-}9"}{4} = 7.688'$

Middle strip $= \frac{30'\text{-}9"}{2} = 15.375'$

Moment: $M_o = \frac{w_u \ell_2 \ell_n^2}{8}$

Frame B

$\frac{1}{2}$ C.S $= \frac{21'\text{-}0"}{4} = 5.25'$

M.S $= \frac{21'\text{-}0"}{2} = 10.5'$

$w_u = 1.2D + 1.6L = 1.2(157.5) + 1.6(125) = 389$ psf

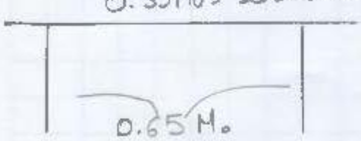
$M_o = \frac{(389)(21)(30.75 - \frac{20"}{12})^2}{8}$

$M_o = 863.7$ ft-K

$M_o = \frac{(389)(30.75)(21 - \frac{20}{12})^2}{8}$

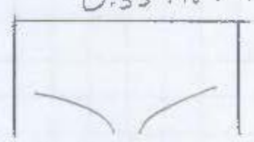
$M_o = 559$ ft-K

$0.35 M_o = 302.3$ ft-K



$0.65 M_o = -561.4$ ft-K

$0.35 M_o = 195.6$ ft-K



$0.65 M_o = -363.4$ ft-K

[Using ACI-318-08 § 13.6.3.3 for exterior edge fully restrained]

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Project:		Computed:	Date:
Subject: Flat plate		Checked:	Date:
Task:		Page: 6	of: 17
Job #:		No:	

Frame A:

-561.4 → 75% to C.S = -421.1 ft-K → 100% slab
-561.4 → 25% to M.S = -140.3 ft-K

+302.3 → 60% to C.S = 181.4 ft-K → 100% slab
+302.3 → 40% to M.S = 120.9 ft-K

Frame B:

-363.4 → 75% to C.S = -272.55 ft-K → 100% slab
-363.4 → 25% to M.S = -90.85 ft-K

+195.6 → 60% to C.S = 117.4 ft-K → 100% slab
+195.6 → 40% to M.S = 78.2 ft-K

Summary:

Total M.	-561.4	+302.3	-561.4
Moment in C.S	-421.1	+181.4	-421.1
Moment in M.S	-140.3	+120.9	-140.3

Frame B

-363.4	+195.6	-363.4
-272.5	+117.4	-272.5
-90.15	+78.2	-90.75

Frame A: width: 30'-9" Frame B: width: 21'-0"

Since Moments in the long direction are larger than those in the short direction the larger d is assigned to the long direction

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Project: Computed: Date:
Subject: flt. plate Checked: Date:
Task: Page: 7 of: 12
Job #: No:

Middle strip:

Description	Frame A M^-	Frame A M^+	Frame B M^-	Frame B M^+
1) Moment M_u (K-ft)	-140.3	+120.9	-90.85	+78.2
2) width of Middle Strip	184.5"	184.5"	126"	126"
3) Effective depth $d = t - .75 - .75/2$ Assuming #6 bars	10.875"	10.875"	10.125"	10.125"
4) $M_n = M_u/\phi$	-155.9	+134.3	-100.9	+86.9
5) $R = \frac{M_n \times 1200}{b d^2}$	85.74	73.86	84.4	72.65
6) ρ (table A.5a Wilson)	0.0014	0.0013	0.0014	0.0013
7) $A_s = \rho . b . d$	2.81 in^2	2.61 in^2	1.79 in^2	1.66 in^2
8) $A_{s,min} = 0.0018\, b t$	3.99 in^2	3.99 in^2	2.72 in^2	2.72 in^2
9) $N = \frac{\text{larger of 7 or 8}}{0.44 \text{ Area of \#6}}$	9.1 ≈ 10	10	6.18 ≈ 7	7
10) $N_{min} = \frac{\text{width of strip}}{2t}$	7.68 ≈ 8	8	5.25 ≈ 6	6
USE ⇒	10	10	7	7

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Project:
Subject: flat plate
Task:
Job #:
Computed: Date:
Checked: Date:
Page: 8 of: 12
No:

Column strip: Frame A / Frame B

Description	Frame A M^-	Frame A M^+	Frame B M^-	Frame B M^+
1) Moment M_u (ft-k)	-421.1	+181.4	-272.5	117.4
2) Width of column strip	184.5"	184.5"	126"	126"
3) Effective depth $d = t - .75 - \frac{1}{2}$ Assuming # 8 bars	10.75"	10.75"	9.75"	9.75"
4) $M_n = M_u/\phi$	467.9	201.6	302.8	130.4
5) $R = \frac{M_n \times 1200}{b d^2}$	263.3	102.1	273	117.6
6) ρ (table A.5a)	0.0045	0.0014	0.0047	0.0015
7) $A_s = \rho \cdot b \cdot d$	8.92 in^2	2.78 in^2	5.77 in^2	1.84 in^2
8) $A_{s,min} = 0.0018\,bt$	3.99 in^2	3.99 in^2	2.72 in^2	2.72 in^2
9) $N = \frac{\text{larger of 7 or 8}}{0.79}$ #8	11.29 ≈ 12	5.05 ≈ 6	7.3 ≈ 8	7
10) $N_{min} = \frac{\text{width of slab}}{2t}$	8	8	6	6
USE →	12	8	8	7

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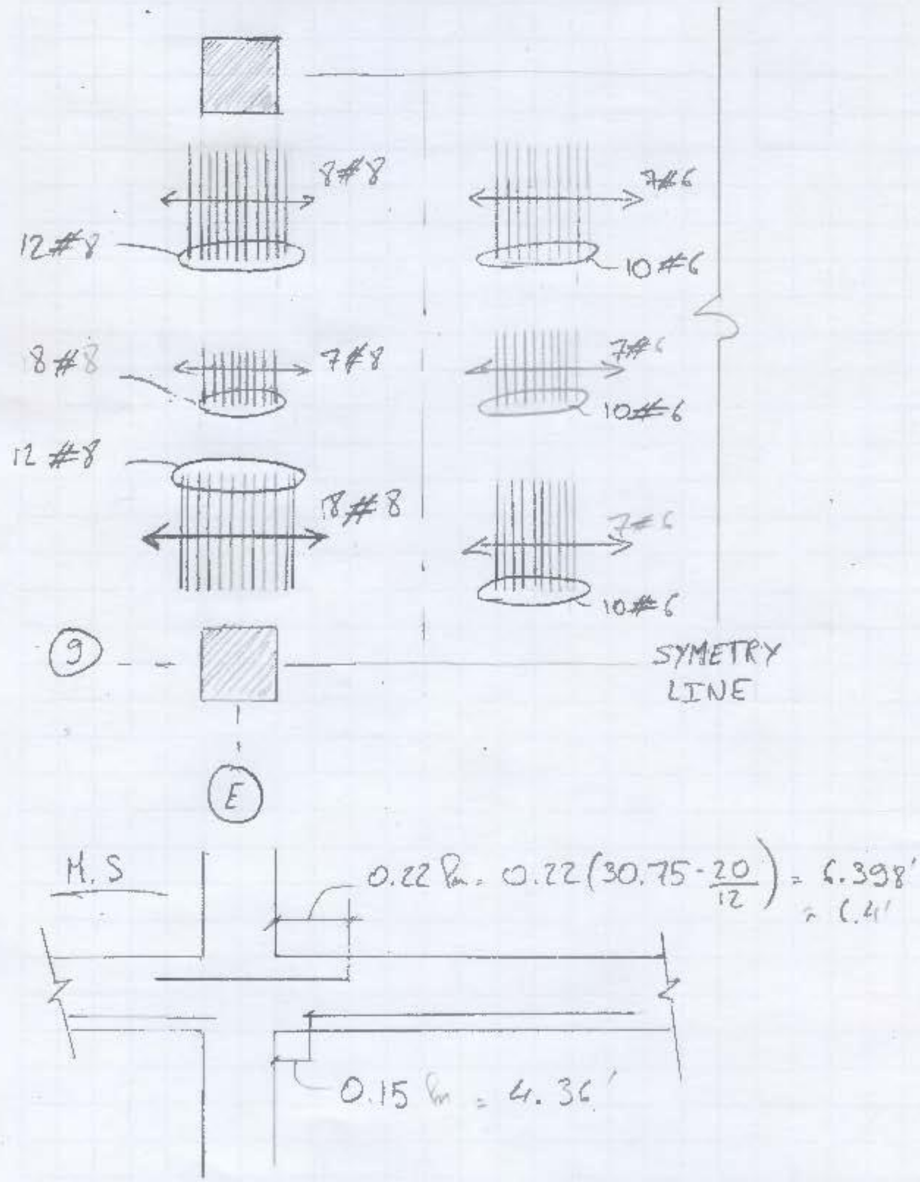
Project:		Computed:	Date:
Subject: flat plate		Checked:	Date:
Task:		Page: 9	of: 12
Job #:		No:	



Reinforcement Detail

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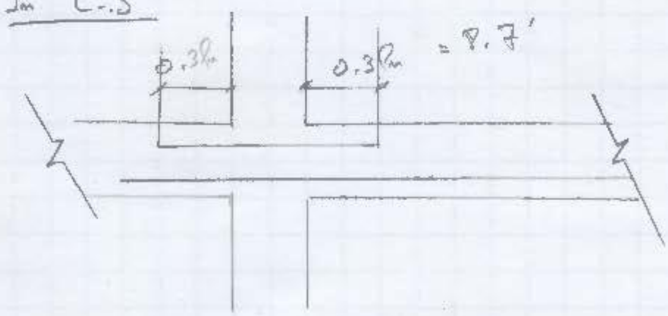
Project:		Computed:	Date:
Subject: flat plate		Checked:	Date:
Task:		Page: 10	of: 12
Job #:		No:	

In C.S



Reinforcement Detail

Deflection check:

Assumed (weighted Average) = 67.5 % of Moment to column strip

$F_y = 60{,}000\,psi$ $\quad f'_c = 4000\,psi$ 32.5 % of Moment to middle strip

$t = 12''$

Intermediate deflection due to dead load:

in C.S $\quad w_D = (.158)(30.75)(.675) = 3.279\ k/ft$

$$I_g = \frac{(184.5)(12)^3}{12} = 26{,}568\ in^4$$

$$E_c = 57000\sqrt{4000} = 3605\ ksi$$

$$\Delta_D\,(max) = \frac{0.0026\,(3.279)(30.75)^4(12)^3}{3605\,(26{,}568)} = 0.137''$$

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Project: | Computed: | Date:
Subject: Flat plate | Checked: | Date:
Task: | Page: 11 | of: 12
Job #: | No:

in M.S $\omega_D = (.158)(30.75)(.325) = 1.579 \text{ K/ft}$

$I_g = 26,568 \text{ in}^4$; $E_c = 3,605 \text{ Ksi}$

$$\Delta_D(\max) = \frac{0.0026(1.579)(30.75)^4(12)^3}{3605 \times 26,568} = 0.066''$$

Total immediate Δ due to DL total $= 0.066 + 0.137'' = 0.203''$

Immediate Deflection due to total live load:

Column strip: $W_L = (.125)(30.75)(.675) = 2.594 \text{ K/ft}$

$$\Delta_L(\max) = \frac{0.0048(2.594)(30.75)^4(12)^3}{(3605)(26,568)} = 0.201''$$

Middle strip: $W_L = (.125)(30.75)(.325) = 1.249 \text{ K/ft}$

$$\Delta_L(\max) = \frac{0.0048(1.249)(30.75)^4(12)^3}{(3605)(26,568)} = 0.097''$$

Total immediate due to Live $= 0.201 + 0.097 = 0.298''$

Additional DL Δ after a long time due to DL total

Assume $\lambda = 3.0$ $\quad \Delta_D(\max) = 3.0\left[0.203'' + 0.25(0.298)\right] = 0.833''$

Live load deflection check: from ACI 318-08 table 9.5(b)

$\ell/360$ (floors not supporting or attached to non structural elements likely to be damaged by large deflection)

$$\ell/360 = \frac{30.75 \times 12}{360} = 1.025'' > 0.298''$$

HDR ONE COMPANY *Many Solutions*	Project:	Computed:	Date:
	Subject: flat plate	Checked:	Date:
	Task:	Page: 12	of: 12
	Job #:	No:	

Total Deflection After partitions = 0.1 (0.203) + 0.298 + 1.025"

Δ_{max} for partitions = 1.343"

ACI 318-08 $\ell/240 = \frac{30.75 \times 12}{240} = 1.538" > 1.343"$

$\Rightarrow$ OK!

Thus the design system works for flexure, shear and deflection and since t = 12" and deflection have F.S of 3 we can assume that this floor system will meet the floor vibration requirements for the Alzheimers center.

Appendix F: One Way Slab Calculations

Project:	Tech (2)	Computed:		Date:	
Subject:	One way slab	Checked:		Date:	
Task:	w/ continuous beams	Page:	1	of:	10
Job #:		No:			

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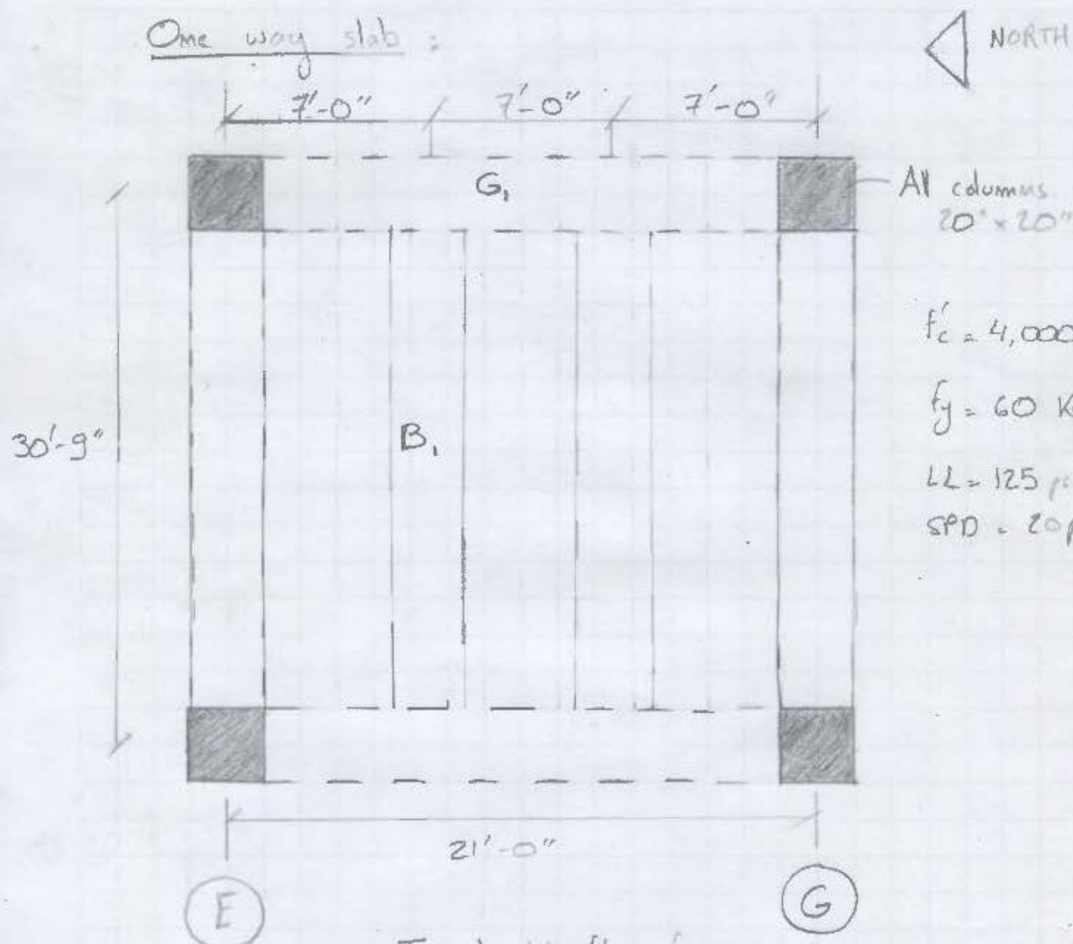
One way slab :



All columns a/c 20" x 20"

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

$f_y = 60$ Ksi

LL = 125 psf

SPD = 20 psf

Typical lab floor framing

Minimum thickness of slab from table 9.5 (a) ACI 318-08

Exterior Bay = $\frac{\ell}{24} = \frac{(7')(12")}{24} = 3.5"$

⇒ Use a thickness of 4" for constructability and vibrations and fire proofing

Project:		Computed:	Date:
Subject: One way		Checked:	Date:
Task:		Page: 2	of: 10
Job #:		No:	

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d:

Assume #4 bars to calculate d, and clear cover of 3/4"

$$d = h - 3/4" - d_b/2 = 4" - 3/4" - \frac{0.5}{2} = 3"$$

Load: $w_D = \left(\frac{4}{12}\right)(150) = 50$ psf

Superimposed: 20 psf

$w_{D\,total} = 70$ psf

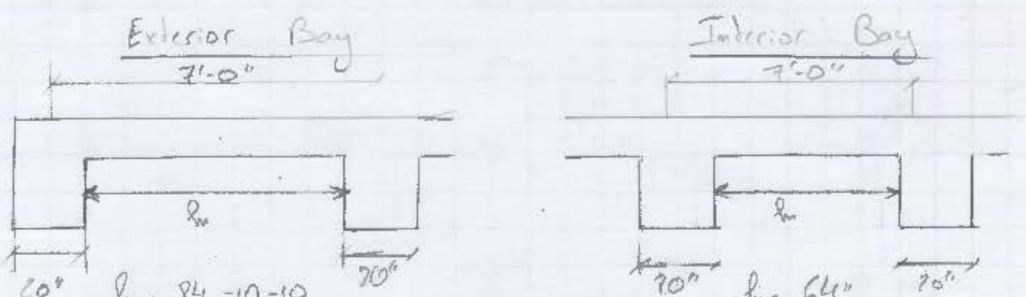
$w_L = 125$ psf unreducible

$$w_u = 1.2D + 1.6L = 1.2(70) + 1.6(125) = 284 \text{ psf}$$

Assume: the section is tension controlled → $\phi = 0.9$

$b = 20"$ for constructibility and form work costs

$w_L = 125 < 3 w_D = 70 \times 3 = 210$ psf → we can use ACI Moment coefficients.



$l_n = 84 - 10 - 10$

$l_n = 64"$

→ $l_{n\,avg} = 64"$

Note: Using ACI moment coefficient

For simple calculations and preliminary schematic design, this method will be used even though the spans in the E-W direction are not the same. However, only the one bay will be used.

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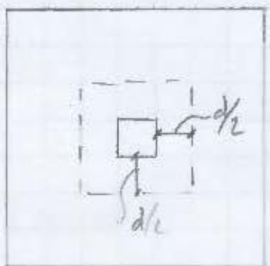
Project:		Computed:	Date:
Subject: Flat plate		Checked:	Date:
Task:		Page: 3	of: 12
Job #:		No:	

Punching shear (two-way) action :

$V_u = W_u \times \text{Area}$

$V_u = .389 \times \left[(21')(30.75) - \dfrac{(20 + \overset{d \text{ avg}}{10.5})^2}{144} \right] = 248.6 \text{ K}$

shear strength :



$b_o = (20 + 10.5)\, 4 = 122''$

$\dfrac{b_o}{d} = \dfrac{122}{10.5} = 11.62$

$\beta_c = 1 < 2$ for equivalent square

$V_c = \left[\left(\dfrac{\alpha_s}{b_o/d} \right) + 2 \right] \sqrt{f'_c}\; b_o \cdot d$ (controls)

$\alpha_s = 40$ for interior columns

$V_c = \left(\dfrac{40}{11.62} + 2 \right) \sqrt{4{,}000}\, (122)(10.5) = 440.9 \text{ K}$

$\phi V_c = 0.75\,(440.9) = 330 \text{ K} > 248.6 \text{ K}$

$\Rightarrow$ OK!

For edge columns, $\alpha_s = 30$

$\phi V_c = .75\,(371.2) = 278.4 > 248.6 \text{ K}$

$\Rightarrow$ OK!

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Project: Computed: Date:
Subject: One way Checked: Date:
Task: Page: 4 of: 10
Job #: No:

Design of Reinforcement:

Description	External Support	Exterior Midspan	First Interior	Interior Midspan
1. l_n (ft)	64"	64"	64"	64"
2. $w_u l_n^2$ (K-ft/ft)	8.02	8.02	8.02	8.02
3. M coefficient	$-1/24$	$1/14$	$-1/10$	$1/14$
4. M_u (K-ft/ft)	-0.334	0.573	-0.802	0.573
5. A_s (required)	0.025	0.043	0.061	0.043
6. $A_{s,min}$ in²/ft	0.086	0.086	0.086	0.086
7. Bars selected	No 4 @ 12"	→	→	→
8. Final A_s	0.2 in²	→	→	→

Check for spacing:

$$s = 15\left(\frac{40{,}000}{f_s}\right) - 2.5\,c_c \leq 12\left(\frac{40{,}000}{f_s}\right)$$

$f_s = 2/3 f_y$ clear cover

$s = 15 - 2.5(0.75) \leq 12$
$s = 13.125 > 12 \Rightarrow$ OK!

Transverse direction:

A_s for shrinkage and temperature $= 0.0018\,(12)(4) = 0.086$

Maximum spacing $\leq 5h = 20"$
$\leq 18 \Rightarrow$ controls

Slab detail:

4" slab No 4 @ 12" for top and bottom steel
No 4 @ 18" for transverse steel.

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Project:		Computed:	Date:
Subject:	One way	Checked:	Date:
Task:		Page: 5	of: 10
Job #:		No:	

Beam Design:

$b = 20"$ $\quad h \approx \frac{\ell}{12}$ to $\frac{\ell}{18} \approx 30.75"$ to $20.5"$

select $h = 24$ in (between two values)

$w_{\text{beam web}} = \left(\frac{(24-4)(20)}{144}\right) \times 150 \approx 417$ lb/ft $= .417$ K/ft

$w(\text{slab and SDL}) = (70\,\text{psf} \times 12) = 840$ lb/ft $= .840$ K/ft

$w_u = 1.2\,w_D + 1.6\,w_L = 1.2(.840 + .417) + 1.6(1.5) = 3.91$ K/ft

$1.4(1.257) = 1.76$ K/ft

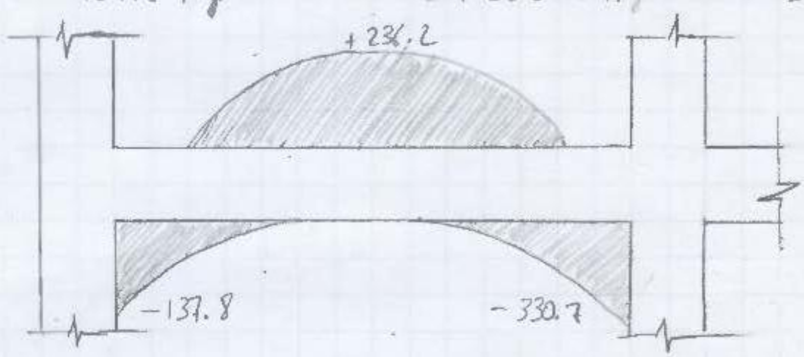
1.5 — unreduced for laboratory > 100 psf

First ext: $M_u = -\frac{w_u \ell_n^2}{24} = -\frac{3.91(30.75 - \frac{20}{12})^2}{24}$

$M_u = -137.8$ K-ft

Interior: $+\frac{w_u \ell_n^2}{14} = +\frac{3.91(29.08)^2}{14} = +236.2$ K-ft

First int: $\frac{-w_u \ell_n^2}{10} = -\frac{3.91(29.08)^2}{10} = -330.7$ K-ft



$b_{eff} =$ min of:

- 1/4 span length $= \frac{1}{4}(30.75)(12) = 92.25"$
- $b_w + 16\,h_f = 20 + 16(4) = 84"$
- $b_w + 2(\frac{1}{2}$ clear distance$) = 20 + 2(\frac{1}{2}(7' - \frac{20}{12})) = 84"$

$\Rightarrow b_{eff} = 84"$

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Assume $f'_c = 4{,}000$ use #4 stirrups

Description:	First exterior	Midspan	First Interior
1. M_u (k-ft)	−137.8	+236.2	−330.7
2. $A_s = \frac{M_u}{4 \cdot d}$ (in²)	$-137.8/4(21) = 1.64$	$236.2/4(21) = 2.81$	$330.7/4(21) = 3.94$
3. A_s (chosen)	2 #9 = 2.0 in²	3 #9 = 3.0 in²	4 #9 = 4.0 in²
4. $\rho = \frac{A_s}{b \cdot d} < 0.025$	0.0048	0.0071	0.0055
5. d	$24'' - 1.5 - .5 - \frac{1.128}{2} = 21.44$	21.44	21.44
6. b_{eff}	84"	84"	84"
7. $M_{u,TBM} = \phi\, 0.85 f'_c b_{eff} h_f (d - \frac{h_f}{2})$	$[0.9(.85)(4)(84)(4)(21.44'' - \frac{4}{2})]/12 = 1666$ '-K > Mu Treat as rectangular beam		
8. $a = \frac{A_s f_y}{0.85 f'_c b}$	1.765	2.65	3.53
9. $c = a/\beta_1 (.85)$	2.08	3.117	4.15
10. $\varepsilon_s = \frac{(d-c)\varepsilon_u}{c}$	$\left(\frac{21.44 - 2.08}{2.08}\right).003 = 0.028$	0.0176	0.0125
9. if $\varepsilon_s > 0.005$ then $\phi = 0.9$	→ ϕ = 0.9	→ 0.9	→ 0.9
10. $\phi M_n = [\phi A_s f_y (d - \frac{a}{2})]/12$	185 '-K > Mu → OK	271.5 '-K > Mu → OK	354.15 '-K > Mu → OK!
11. $A_{s,min} = \frac{3\sqrt{f'_c}\, b d}{f_y}$	$3\sqrt{4{,}000}(20)(21.44)/60{,}000 = 1.356$ in²		
max $\frac{200\, b d}{f_y}$	$200(20)(21.44)/60{,}000 = 1.43$ in²	$A_{s,min} = 1.43$ in²	
12. Spacing requirement	Meets spacing requirements → OK!		
13. $V_u = w_u l_n / 2$	$[3.91(30.75 - (20/12))]/2 = V_u = 56.86$ K		
14. $V_c = 2\sqrt{f'_c}\, b \cdot d$	$2\sqrt{4{,}000}(20)(21.44) = 54.24$ K		
15. $V_s = 2(A_s) f_y (\frac{d}{12})$	$2(.2)(60)(\frac{21.44}{12})$ #4 stirrup $V_s = 42.88$ K		
16. $\phi V_n = \phi(V_c + V_s)$	$\phi V_n = 0.75(54.24 + 42.88) = 72.84$; $\phi V_n > V_u$ → OK!		

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Deflection: (Assume simply supported)

$$\Delta_{LL} = \frac{5 w_L \ell^4 (12)^3}{384 E I} = \frac{5(1.5)(30.75)^4(1728) \times 1000}{384(57{,}000\sqrt{4{,}000})\left(\frac{20(24)^3}{12}\right)} = 0.363"$$

Δ_{LL} OK!

$$\Delta_{LL\,max} = \frac{\ell}{480} = \frac{30.75 \times 12}{480} = 0.769$$

$$\Delta_{TL} = \frac{5(1.257+1.5)(30.75)^4(1728) \times 1000}{384(57{,}000\sqrt{4{,}000})\left(\frac{20(24)^3}{12}\right)} = 0.668"$$

$$\Delta_{TL\,max} = \frac{\ell}{280} = \frac{30.75 \times 12}{280} = 1.319"$$

$\Rightarrow \Delta_{TL}$ OK!

Long term deflection:

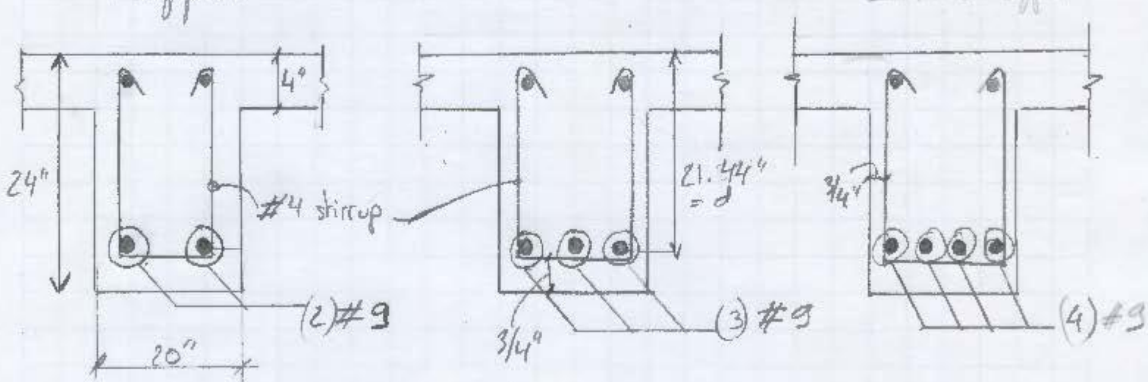
$$w_{TL} = 3(1.257 + .25(1.5)) = 4.896$$

$$\Delta_{TL} = \frac{5(4.896)(30.75)^4(1728) \times 1000}{384(57{,}000\sqrt{4{,}000})\left(\frac{20(24)^3}{12}\right)} = 1.186" < \Delta_{TL\,max} \Rightarrow \text{OK!}$$

Beam @ Exterior Support

@ Midspan

@ First Interior support



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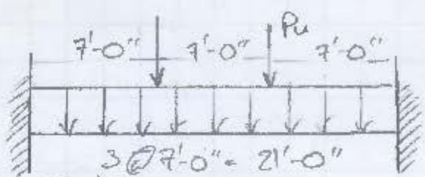
Girder Design:

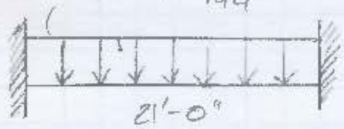
Assume $b = 20''$ to match columns and beams
and $h = 24''$ to match beams for constructability, cost, uniformity



$P_u = 56.86^K$ (from V_u previous)

$w_{self\ weight} = \frac{(24-4)(20)}{144} \cdot .150 = .417\ k/ft$

$=$



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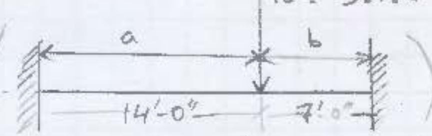

$\Big)$

$M^- = \frac{w\ell^2}{12} = \frac{.417(21')^2}{12} = 15.32\ ft\text{-}K$

$M^- = \frac{P a^2 b}{\ell^2}$ — when $a = 7$: $= 88.45'\text{-}K$ | when $a = 14$: $176.9'\text{-}K$

$M^+_{@x} = \frac{w}{12}(6\ell x - \ell^2 - 6x^2)$

$= \frac{.417}{12}\left(6(21)(14') - (21)^2 - 6(14)^2\right)$

$M^+_{@14'} = 5.12\ ft\text{-}K$

$V = \frac{w\ell}{2} = \frac{.417(21)}{2} = 4.38^K$

$M^+_{@load} = \frac{2Pa^2b^2}{\ell^3}$ $\quad [M^+_{@x}]$ $\quad 16.85K\text{-}ft$

$V = \frac{Pa^2}{\ell^3}(a+3b)$ $\quad 14.74^K \quad 42.12^K$

$[M^+ @ x = 7] = R_1 x - \frac{Pab^2}{\ell^2}$

$14.74(7) - \frac{56.86(14)(7)^2}{(21)^2}$

$M^+_{@x} = 14.74$

$V_{max} = 4.38^K + 14.74 + 42.12 = 61.24^K$

$M^+_{max} = 5.12 + 16.85 + 14.74 = 36.71'\text{-}K$

$M^-_{max} = 15.32 + 176.9 + 88.45 = 280.67'\text{-}K$

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Description:	@ Midspan	@ Supports
1. M_u (K-ft)	36.71'-K	280.67'-K
2. $A_s = \frac{M_u}{4d}$	$\frac{36.71}{4(21)} = 0.44 in^2$	3.34 in^2
3. A_s, chosen	(1) #9 = 0.6 in^2	(4)#9 4 in^2
4. $\rho = \frac{A_s}{b \cdot d}$	$\frac{1.0}{20(21)} = 0.0024$	0.0095

5. b_{eff} = | $1/4\,(21')(12) = 63''$ ⟹ controls
| $20 + 16(4) = 84''$
min | $20 + 2(1/2[21'(12)-20)] = 252''$

6. $M_{u,TBM} = [\phi(0.85)(f'_c)(b_{eff})(h_f)(d - \frac{h_f}{2})]/12 = \frac{0.9(.85)(4)(63)(4)(4-\frac{d}{2})}{12}$

7. $d = 24 - 1.5 - \underset{\#4\ stirrup}{0.5''} - \frac{1.128}{2} = 21.44$ ⟹ $M_{u,TBM} = 1249'\text{-}K > M_u$

TREAT AS RECTANGULAR BEAM

8. $a = \frac{A_s f_y}{0.85 f'_c\, b}$	0.88	3.53
9. $c = a/\beta_1$	1.04	4.15
10. $\varepsilon_s = \left(\frac{d-c}{c}\right).003$	$\left(\frac{21.44-1.04}{1.04}\right).003 = 0.059 > .005$ ⟹ $\phi = 0.9$	0.0125 > 0.005 ⟹ $\phi = 0.9$

11. $A_{s,min}$ = from previous since same dimensions $A_{s,min} = 1.43 in^2$

12. Spacing Requirements is the same

13. From previous $\phi V_n = \phi(V_c + V_s) = 72.84$

$V_u = V_{max} = 61.24^K$

⟹ $\phi V_n > V_u$ ⟹ OK!

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Deflection:

$$\Delta_{max} = \frac{PL^3}{28EI} + \frac{5w_L l^4 (12)^3}{384EI}$$

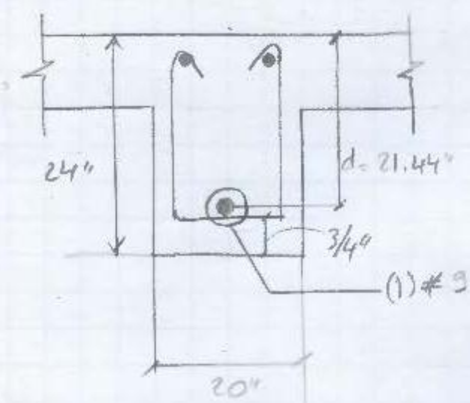
$$LL: \frac{23.06(21)^3 1728}{28(57{,}000\sqrt{4{,}000})\left(\frac{20(24)^3}{12}\right)} + 0 = 0.391'' < \frac{l}{480} = 0.525''$$

$\Rightarrow$ OK!

$$TL: \frac{(23.06+19.33)(21)^3(1728)}{28(57{,}000\sqrt{4000})\left(\frac{20(24)^3}{12}\right)} + \frac{5(417)(21)^4(1728)}{384(57{,}000)\sqrt{4000}\left(\frac{20(24)^3}{12}\right)}$$

$$\Delta_{TL} = 0.718'' + 0.022 = 0.74'' < \frac{l}{240} = 1.05''$$

Girder @ Midspan



Girder @ Supports