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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.

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
- Building code requirements for reinforced concrete (ACI 318)
- AISC Manual of Steel Construction, Allowable Stress Design 9th ED.
- AISC Manual of Steel Construction, Load Resistance Factor Design (LRFD) 1st ED.
- American Welding Society (AWS), D1.1, D1.3, D1.4
- Minimum Design Loads for Buildings and Other Structures (ASCE 7-98)
- Masonry Construction for Buildings (ACI 530-99 AND ACI 530.1-99)

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

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

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).

<table>
<thead>
<tr>
<th>Concrete Usage</th>
<th>Weight</th>
<th>Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spread footing</td>
<td>Normal</td>
<td>3000</td>
</tr>
<tr>
<td>Mat slab foundation</td>
<td>Normal</td>
<td>3000</td>
</tr>
<tr>
<td>Precast Joist Webs and soffit beams</td>
<td>Normal</td>
<td>5000</td>
</tr>
<tr>
<td>Cast-in-place slab</td>
<td>Normal</td>
<td>4000</td>
</tr>
<tr>
<td>Columns, typical</td>
<td>Normal</td>
<td>4000</td>
</tr>
<tr>
<td>Columns, as noted</td>
<td>Normal</td>
<td>6000</td>
</tr>
<tr>
<td>Precast Masonary Lintels</td>
<td>Normal</td>
<td>5000</td>
</tr>
<tr>
<td>Housekeeping Pads</td>
<td>Normal</td>
<td>4000</td>
</tr>
<tr>
<td>General Structure Concrete</td>
<td>Normal</td>
<td>4000</td>
</tr>
</tbody>
</table>

Note: Normal weight concrete is at 28 day compressive strength.
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

---

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

<table>
<thead>
<tr>
<th>Usage</th>
<th>Standard</th>
<th>Grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforcing Steel</td>
<td>ASTM A615</td>
<td>60</td>
</tr>
<tr>
<td>Reinforcing Steel (welded)</td>
<td>ASTM A706</td>
<td>60</td>
</tr>
<tr>
<td>Welded Wire Fabric</td>
<td>ASTM A185</td>
<td>70</td>
</tr>
<tr>
<td>Prestressing Tendons</td>
<td>ASTM A416</td>
<td>270</td>
</tr>
<tr>
<td>Wide Flange, S and Tee shapes</td>
<td>ASTM A992</td>
<td>50</td>
</tr>
<tr>
<td>Angles Channels and Plates</td>
<td>ASTM A36</td>
<td>36</td>
</tr>
<tr>
<td>Tubes</td>
<td>ASTM A500 B</td>
<td>46</td>
</tr>
<tr>
<td>Pipes</td>
<td>ASTM A53 B</td>
<td>35</td>
</tr>
<tr>
<td>Bolts</td>
<td>ASTM A325</td>
<td>36</td>
</tr>
<tr>
<td>Glavanized Roof deck</td>
<td>ASTM A653</td>
<td>33</td>
</tr>
</tbody>
</table>

Note: Welding Electrodes used were E70XX

<table>
<thead>
<tr>
<th>Usage</th>
<th>Standard</th>
<th>Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Masonary Units</td>
<td>ASTM C-90</td>
<td>f’m = 1500</td>
</tr>
<tr>
<td>Mortar</td>
<td>ASTM C270, M</td>
<td>f’c= 2500</td>
</tr>
<tr>
<td>Mortar</td>
<td>ASTM C270, S</td>
<td>f’c= 1800</td>
</tr>
<tr>
<td>Grout</td>
<td>ASTM C476</td>
<td>f’c= 3000</td>
</tr>
<tr>
<td>Joint Reinforcement</td>
<td>ASTM A82, Truss Type</td>
<td></td>
</tr>
</tbody>
</table>

---

**Steel**

<table>
<thead>
<tr>
<th>Usage</th>
<th>Standard</th>
<th>Grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforcing Steel</td>
<td>ASTM A615</td>
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<tr>
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</tr>
<tr>
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<td>ASTM A36</td>
<td>36</td>
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<tr>
<td>Tubes</td>
<td>ASTM A500 B</td>
<td>46</td>
</tr>
<tr>
<td>Pipes</td>
<td>ASTM A53 B</td>
<td>35</td>
</tr>
<tr>
<td>Bolts</td>
<td>ASTM A325</td>
<td>36</td>
</tr>
<tr>
<td>Glavanized Roof deck</td>
<td>ASTM A653</td>
<td>33</td>
</tr>
</tbody>
</table>

Note: Welding Electrodes used were E70XX
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.

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 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.

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.

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.

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 sofit beams.

<table>
<thead>
<tr>
<th>Superimposed dead loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>Description</td>
</tr>
<tr>
<td>Ceilings, lighting, plumbing, fire protection, flooring, and HVAC all</td>
</tr>
<tr>
<td>Ceilings, lighting, plumbing, fire protection, flooring, and HVAC for roof over mechanical levels</td>
</tr>
<tr>
<td>except mechanical</td>
</tr>
<tr>
<td>allowance for roofing system</td>
</tr>
</tbody>
</table>

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:
\[ R = 23.1 \times (1 + \frac{D}{L}) \]
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%.

<table>
<thead>
<tr>
<th>Area of the building considered</th>
<th>Design Load</th>
<th>ASCE 7-05 Live</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Laboratories</td>
<td>125 psf</td>
<td>60 psf</td>
<td>Based on &quot;Hospitals-Laboratories&quot;</td>
</tr>
<tr>
<td>Offices</td>
<td>50 psf</td>
<td>50 psf</td>
<td>Based on &quot;Office Bldg.-Offices&quot;</td>
</tr>
<tr>
<td>Corridors, first floor</td>
<td>100 psf</td>
<td>100 psf</td>
<td>Based on &quot;Office Bldg.-Corridors&quot;</td>
</tr>
<tr>
<td>Corridors, above first floor</td>
<td>80 psf</td>
<td>80 psf</td>
<td>Based on &quot;Office Bldg.-Corridors above&quot;</td>
</tr>
<tr>
<td>Lobbies</td>
<td>100 psf</td>
<td>100 psf</td>
<td>Based on &quot;Lobbies&quot;</td>
</tr>
<tr>
<td>Storage areas</td>
<td>125 psf</td>
<td>125-250 psf</td>
<td>Based on &quot;Storage- light/heavy&quot;</td>
</tr>
<tr>
<td>High density file storage</td>
<td>200 psf</td>
<td>125-250 psf</td>
<td></td>
</tr>
<tr>
<td>Mechanical spaces</td>
<td>150 psf</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>Stairs</td>
<td>100 psf</td>
<td>100 psf</td>
<td>Based on &quot;Stairs&quot;</td>
</tr>
<tr>
<td>Roof</td>
<td>20 psf</td>
<td>20 psf</td>
<td>Based on &quot;Roof- Sloped&quot;</td>
</tr>
</tbody>
</table>

**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.

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.
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.
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.

**Architectural**
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.

---

**One Way Slab Details**

**One Way Slab Schedule**

<table>
<thead>
<tr>
<th>SLAB MARK</th>
<th>SLAB THICKNESS</th>
<th>REINFORCING</th>
<th>REMARKS</th>
</tr>
</thead>
<tbody>
<tr>
<td>S-1</td>
<td>5</td>
<td>#4@10</td>
<td>#4@10</td>
</tr>
<tr>
<td>S-2</td>
<td>12</td>
<td>#4@8</td>
<td>#4@8</td>
</tr>
<tr>
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<td>6</td>
<td>#4@10</td>
<td>#4@10</td>
</tr>
<tr>
<td>S-4</td>
<td>4</td>
<td>#4@10</td>
<td>#4@10</td>
</tr>
<tr>
<td>S-5</td>
<td>10</td>
<td>#4@8</td>
<td>#4@8</td>
</tr>
</tbody>
</table>
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

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 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

![Composite Steel Diagram](image-url)
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 spreadsheet 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

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 times 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

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

<table>
<thead>
<tr>
<th>Consideration</th>
<th>Precast Joist and Soffit Beams (Existing)</th>
<th>Composite Steel</th>
<th>Flat Plate with mild Reinforcements</th>
<th>One-Way Slab</th>
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</thead>
<tbody>
<tr>
<td>Weight (psf)</td>
<td>90</td>
<td>75</td>
<td>150</td>
<td>109</td>
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<tr>
<td>Cost ($/SF)*</td>
<td>cheap (unknown)</td>
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<td>15.28</td>
<td>14.25</td>
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<tr>
<td>Floor Depth</td>
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<td>4.5 slab/ 21 girders</td>
<td>12 slab</td>
<td>4 slab/ 20 beam and girders</td>
</tr>
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<td>Fire Rating</td>
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<td>2 hr</td>
<td>2 hr</td>
<td>2 hr</td>
</tr>
<tr>
<td>Other Impacts</td>
<td>Structure is hidden but left exposed in some locations</td>
<td>Drop ceiling must be provided and decreases floor to floor</td>
<td>Can be left exposed and creates higher ceiling for labs</td>
<td>Minimizes ceiling height and structure cannot be left exposed</td>
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</table>

<table>
<thead>
<tr>
<th>Structural</th>
<th>Foundation Impacts</th>
<th>Existing Cast-in-place footings and mat slabs</th>
<th>May reduce required foundations</th>
<th>Heavy structure that may not be good for a potential sinkhole site</th>
<th>Zero to negligible effect on foundations</th>
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<tbody>
<tr>
<td>Lateral System Impact</td>
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<td>Steel braced/ moment frames</td>
<td>Shear walls would remain</td>
<td>Shear walls would remain</td>
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<tr>
<th>Serviceability</th>
<th>Maximum Deflection (inches)</th>
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<th>1.343</th>
<th>1.186</th>
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<tr>
<td>Vibration Control</td>
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<td>Average but analyzed in report</td>
<td>Average</td>
<td>Very Good</td>
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<table>
<thead>
<tr>
<th>Construction</th>
<th>Additional Fire Protection Required</th>
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<th>Will have spray-on</th>
<th>Will likely have none</th>
<th>Will likely have none</th>
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<tr>
<td>Schedule Impact</td>
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<td>Likely have no delay</td>
<td>Likely delay shedule</td>
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<table>
<thead>
<tr>
<th>Feasibility</th>
<th>Yes</th>
<th>Good</th>
<th>Medium</th>
<th>Medium</th>
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</table>

*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

![Table Image](image-url)

### SUPERIMPOSED LOAD CAPACITY

<table>
<thead>
<tr>
<th>JOIST SIZE</th>
<th>SPAN</th>
<th>System Weight PSF</th>
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<tbody>
<tr>
<td>2'-6&quot;</td>
<td>-</td>
<td>84</td>
</tr>
<tr>
<td>3'-6&quot;</td>
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<td>76</td>
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<td>4'-6&quot;</td>
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<td>72</td>
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<td>74</td>
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<tr>
<td>6'-0&quot;</td>
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Notes: Spans shown are clear (Face-to-face of supports). For design conditions not addressed please contact PSF
### SUPERIMPOSED LOAD CAPACITY

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<th>JOIST SIZE</th>
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<tr>
<td></td>
<td>10'0''</td>
<td>101</td>
</tr>
<tr>
<td>4 3/4''</td>
<td>4'8''</td>
<td>-</td>
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<td>10'0''</td>
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Notes: Spans shown are clear (Face-to-face of supports). For design conditions not addressed please contact PSF.

http://psfjoist.com/loadtables4.html
### 16” JOIST WITH 3” COMPOSITE SLAB (P.S.F.)

<table>
<thead>
<tr>
<th>Joist Spacing</th>
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<th>30</th>
<th>32</th>
<th>34</th>
<th>36</th>
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<td>222</td>
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<td>4'-6 1/4”</td>
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<td>5'-6 1/4”</td>
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<td>138</td>
<td>122</td>
<td>107</td>
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</table>

#### TYPICAL BEAM SECTION

- **FIRST LAYER TOP BARS**
- **SECOND LAYER TOP BARS**
- **#5 @ 12” SIDE BARS** when depth is greater than or equal to 24”, U.N.C.
- **STIRRUPS CAST IN SOFTT BEAM**
- **PRECAST JOIST RIBS, TYPICAL EACH SIDE**
- **PRECAST BEAM SOFTT**
- **PER PRECAST MANUFACTURER**

# W/2
**Project:** Tech 1  
**Subject:** Grafix  
**Task:** Joint  
**Job #:**

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<tr>
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</thead>
<tbody>
<tr>
<td>Tech 1</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Subject</th>
<th>Checked</th>
<th>Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grafix</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Task</th>
<th>Page</th>
<th>of</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joint</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>

---

All columns @ ends are 2x10 x 20

\[
\text{Slab + SP} = 62.5 + 14 + 20 = 96.5 \text{ psf}
\]

\[
L = 125 \text{ psf}
\]

\[
D + L = 125 + 96.5 = 221.5
\]

Using the present tables, for a span of 32' @ a spacing of 3'-6" to be uniform \( \left( \frac{31}{3.5'} = 6 \right) \), a 12" deep joist or \( T.3 \) would have a capacity 253 psf that for a span of 31'

a T3 @ 3'-6" would suffice
Beam B1 @ level 5

J2 @ 6'-6"

J3 @ 3'-6"

Joist schedule: Assume deck is 4'3/4"

J2 12" @ 6'-6" w = 67 psf
J3 16" @ 3'-6" w = 84 psf

5" NW slab \( \left( \frac{5}{12} \right) (150) = 62.5 \) psf
SP 14 psf Lighting, plumbing,ailing, HVAC
20 psf pedestrian load

Assume beam is continuous as there are moment frames.
short direction: \((5'7'') (17) = 0.374 \text{ klf}\)

long direction: \((15'5'') (84) = 1.935 \text{ klf}\)

Sleb + Sp: \((42.5 + 14 + 20) (15'5' + 5'7'') = 2.027 \text{ klf}\)

total klf: \(125 \text{ klf}\)

reduced by 80% see provisions

\(100 \mu F (81') = 2.1 \text{ klf}\)

\(W_0 = 1.2 D + 1.6 L = 1.2 (2.027 + 1.235 + 0.394) + 1.1 (81')\)

\(W_0 = 7.795 \text{ klf}\)

\(M_{u+} = 7.795 \text{ klf} \left(\frac{81' - 80}{12}\right)^2 = 1.821 \text{ k-ft}\)

Positive moments for interior spans:

\(M_{u-} = 7.795 \text{ klf} \left(\frac{81' - 80}{12}\right)^2 = 2.64 \text{ k-ft}\)

At both sides:

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

\(B_1\) is a soffit beam of 5B-6, looking at the beam soffit schedule, it is \(24'' \times 24''\)

\(\# 2\) at Full Length

\(\# 3\) at Right End

\(M_u = 205\)

\(V_u \text{ klf} = 110\)

\(V_u \text{ right} = 105\)
Typical slab reinforcing detail

Check slab thickness

Minimum slab thickness (Table 3.5 (a5))

Exterior Bay: \( \frac{5}{24} \) for length of 39.4' (maximum)

\[ \frac{39.4'}{24} = 1.64' \]

For interior Bay: \( \frac{1}{2} = \frac{21'}{28} = 0.75' \)

Design uses a rim of 5' > 1.64' Ω OK

Check reinforcement for max moment

Find \( W_u \): \( W_d = \left( \frac{5}{12} \times 150 \right) + 14 \text{ psf} + 20 \text{ psf} \)

\[ W_d = 36.5 \text{ psf} \]

\( W = 50 \text{ psf} \) (office floor loads where the typical wood cannot be reduced)

\[ W_d = 1.20 + 1.5 W = 195.8 \text{ psf} = 194 \text{ psf} \]
Since \( w < 3 \) we then we can use ACI around concrete:

First interior:
\[
\mu_u = \frac{W_u}{A_u} = \frac{156 \times (13.5 - \frac{20}{12})^2}{10} \times \text{ft} = 6,233 \text{ lb-ft/ft}
\]

Second interior:
\[
\mu_u = \frac{W_u}{A_u} = \frac{111 (21 - \frac{20}{12})^2}{11} \times \text{ft} = 6,670 \text{ lb-ft/ft}
\]

The second condition:

From the one way slab schedule on S-1 5" slab

That is typical would have

- #4 @ 10" bottom
- #4 @ 10" by left end
- #4 @ 10" by right end

Top:

\[
\begin{align*}
13.35'' \times 12 &= 19.2'' = 2.32'' \\
10 &= 23
\end{align*}
\]

Top has: since right end and left end can be combined
\[
23 \times \#4 \Rightarrow 23 \times 4 	imes 0.2 = 9.2 \text{ in}^2
\]
As (per sec) = \frac{3}{13.33} = 0.476 \text{ in}^2/\text{ft}

Assume \( f_s > f_y \)

\[ a = \frac{A_s}{A_f} = \frac{0.476 (60,000)}{0.85 (4000) (12)} = 0.7" \]

\[ d = 5" - \left( \frac{0.75 + 0.5}{2} \right) = 4" \]

\[ e = \frac{a}{2} = \frac{0.7}{2} = 0.35" \]

\[ \varepsilon_2 = \frac{0.724}{0.85} = 0.860" \]

Check: \( \varepsilon_2 > \varepsilon_y \) : \[ \varepsilon_2 = \frac{E_2}{E_y} (d - c) = \frac{0.0023}{0.0021} (4 - 0.824) \]

\[ \varepsilon_2 = 0.0175 >> \varepsilon_y = 0.3 \]

\[ \phi M_{xx} = 0.9 \left( \frac{A_s f_y (d - a)}{2} \right) = 0.9 \left( \frac{0.476 (60,000) (4 - 0.724)}{2} \right) \]

\[ \phi M_{xx} = 93.820 \text{ lb - ft} = 7.18 \text{ lb - ft} \]

\[ \phi M_{yy} > M_0 = 6660 \text{ lb - ft} \Rightarrow \text{okay!} \]

Check for shear:

\[ \phi V_c = \frac{0.75 \left( \frac{1}{2} f_y (d - a) \right)^2}{2} \]

\[ = 0.75 \left( \frac{1}{2} \frac{1}{4000} (12) (4) \right)^2 = 4.554 \text{ lb/ft} > V_u \Rightarrow \text{OK!} \]
Appendix C: Composite Steel Calculations

**COMPOSITE STEEL:**

*Used Volume Manual for deck (v. COO)*

2 hour fire rating for project from a 4½" thick concrete topping or fire proofing spray should be used

Not changing span lengths, design for 3 span minimum condition

**TYPICAL BAY**

$$\begin{align*}
\text{beam} &= \frac{30.75 \times 12}{8} + \frac{1}{2} (3') (12) = 88.125 \\
\text{min} &= \frac{30.75 \times 12}{8} + \frac{1}{2} (3') (12) = 88.125 \\
\text{girder} &= \frac{21 \times 12}{8} + \frac{1}{6} (30.75) (12) = 216 \\
\end{align*}$$

$$\Rightarrow \text{beam} = 88.1$$

$$\Rightarrow \text{girder} = 53.1$$
Lords: Super imposed dead = 20 psf
Live load = 125 psf
Total = 145 psf

w/4.5" New concrete for fire resisting purpose (CHRR)

Use 2 Y. 20 with or 3 span = 8'-4" > 7'-0"

@ 7'-0" 337 psf > 145 psf
Deck weight = 1.5 psf
Dowel weight = 65 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:

Weight consideration: 14'-6" 3 Max depth = 5'-5" (12')
Ceiling height = 9'-1"

Assume simply supported
Add 5 psf for self-weight smaller to fit mechanical equipment

\[ w_d = \left( \frac{20 + 69 + 5}{7} \right) = 6.5 \text{ psf} \]

\[ LL_t = 0.25 + \frac{15}{(2)(20.75)} = 0.39 = \text{assume} \text{ no reduction close to 1.0} \]

\[ w_{le} = 125(7) = 875 \text{ psf} \]

\[ w_o = 12.5 + 1(1.2 (1.58) + 1.6 (1.875)) = 2.13 \text{ psf} \]
Subject: Composite steel

Task: Page: 3

V_u (k) = 33.67 [5]

\[ V_u = 2.19 \left( \frac{30.95}{2} \right) = 33.67 \]

H_u = \frac{wP L}{8} = 2.19 \left( \frac{30.95}{2} \right) \]

H_u = 258.85 kN

Q_u = 17.2 kN (per AISC table 3-21)

Deflections:

\[ \Delta_u \text{ allowable} = \frac{P}{360} = \frac{(30.95)(12)}{360} = 1.025'' \]

\[ \Delta_u = \frac{5.4 W}{384 E I_{el}} (17.28) \]

\[ I_{el} = \frac{5W L^4}{384 E} (17.28) \]

T = 5.875 \left( \frac{30.95}{2} \right) (17.28) = 532.2 kN \]

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Assume $a = 1'' = 0.5\cdot (\frac{1''}{2}) = 0.5''$

TRIAL SIZES USING ASSC TABLE 3-12 & 3-20:

$W_{19} = 40$ $W_{40} = 80$ $I = 12 > 52.2$ $L$

$Z_{Ah} = 14.4$ $\phi_{Ah} = 483-K$ $\Rightarrow$ # of steps $= 14.4$

$Economy: 10(30.75) + 10(10) = 1530$ $\Rightarrow$ # of sizes

$\Delta I = 5 \cdot (30.75)(30.75)^{4}/(1928) = 0.098^\circ < 1.0^\circ$

$\Delta I = 5 \cdot (30.75)(30.75)^{4}/(1928) = 1.74^\circ < \frac{2}{200}$

$\Rightarrow N = 6$

Try $21\times 44$

$\Delta I = 5 \cdot (30.75)(30.75)^{4}/(1928) = 1.24 < 0.1$

Unshaped strength:

$W_{19} = 21 \times 44$ $\phi_{19} = 358-K$

$W_{u} = 1.4(65lbs)(7) + 1.4(44) = 737.8$ $\Rightarrow$

or $W_{u} = 1.2(9)(7) + 1.2(44) + 1.5(20)(7) = 981.8$ $\Rightarrow$

$H_{u} = 0.922(30.75)^{2} = 113.7 < 358-K = 0$ $\Rightarrow$ OK!
**Concrete Deflection:**

\[ W_{cd} = 0.4(4) + 44 = 527 \text{ kN} \]

\[ \Delta_{we} = \frac{5(527)(20.75)^4(1288)}{384(20,000)(843)} = 0.483'' > \text{allowable} \]

Use 2\(\times\)44 U10 studs.

Still need to check for vibration.

**Design of Girder:**

Assume simply supported. Add 1\(\times\) to each point load for girder self-weight.

\[ P_0 = 0.658(30.75) = 20.23\text{ kN} \]

\[ |L_1| = 0.25 + \frac{15}{\sqrt{2}(30.75)(0.67)} = 0.667 \]

\[ P_L = (0.667)(0.875)(30.75) = 17.35\text{ kN} \]

\[ P_0 = 1.12P_0 = 1.12(20.23) = 53\text{ kN} \]

**Diagram:**

\[ V_u = \frac{53 \times 2}{2} = 53\text{ kN} \]
\( M_u (4-K) = 3714 - K \)

Assume:

- \( W \) girder, weak position, 1 std/rib, \( 3/8 " \) std
- \( \frac{W}{N} = \frac{5}{2} > 1.5 \)
- \( f_c = 4,000 \) NW
- \( \frac{W}{N} = 21.5 \)
  \[ \frac{1}{3} (\frac{f_m + 1.3 - 0.5}{1.3}) \] AISC

\[ \Delta L = \frac{6}{360} = \frac{6 (12)}{360} = 0.01 \]

\[ \Delta L = \frac{P a}{28 E I} \] for \( a = \frac{9}{3} = 3 \)

\[ I_{lb} = \frac{PL^3}{24 E I} = \frac{17.35 (21)^3 (172)}{28 (20000) (0.7)} = 50.5 \text{ in}^4 \text{ min} \]

**TRIAL SIZES:**

- \( W_{19} x 25 \)
- \( W_{16} x 26 \)
- \( W_{18} x 40 \)

<table>
<thead>
<tr>
<th>Section</th>
<th>Width</th>
<th>Depth</th>
<th>Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>19</td>
<td>25</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>16</td>
<td>26</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>18</td>
<td>40</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Weight calculations:

- \( W_{19} x 25 \)
  - \( N = \frac{19}{21.5} = 0.9 \)
  - \( \text{Weight} = 35 (21) + 10 (10) = 835 \text{ lb} \)

- \( W_{16} x 26 \)
  - \( N = \frac{227}{21.5} = 10 \)
  - \( \text{Weight} = 26 (2) + 16 (10) = 406 \text{ lb} \)

- \( W_{18} x 40 \)
  - \( N = \frac{147}{21.5} = 7 \)
  - \( \text{Weight} = 40 (2) + 7 (10) = 510 \text{ lb} \)
Unshared issue:

\[ P_u = \left( \frac{1.2 (60 \times 7' + W) + 1.5 (20 \times 7')} {2} \right) \]

\[ P_u = \left( \frac{570.6 + 1.23 \times 224} {2} \right) = 16.976 + 25.2 W \]

\[ H_a = \left[ 16.876 + 0.0256 W \right] (7) \]

For \( 18' \times 40' \) \( H_a = 144' \), \( 8' < 154' \times 8' < 184' \)

\[ \Delta L = \frac{(12.25)(12)} {29.000} = 0.597' < 0.7' = \text{OK!} \]

\[ \Delta T_L = \frac{21}(12) = 1.05'' \]

\[ \Delta T_L = \frac{(20 - 13 + 17.35)(21)(728)} {28 (25'000)} (6.12) = 1.22'' = \text{OK!} \]

So use \( W 21' \times 44' \) \( \Delta T_L = 0.93'' = \text{OK!} \)

Works fine unshared too!

Design interior columns:

\[ P_u = 2 \left( \frac{53 \times 224} {2} \right) = 173' \]

\( k = 2.0 \) (conservative approach)

\[ k = 20' \text{ from AISC LRK 4-1, W 12 X 58 or } \]

\[ k = 14 X 61 \text{ work} \]
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

<table>
<thead>
<tr>
<th>Building Type and Usage</th>
<th>Recommended Values of Parameters in Equation (4.1) and ae / g limits</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Constant Force</td>
</tr>
<tr>
<td>-------------------------------</td>
<td>----------------</td>
</tr>
<tr>
<td>1 Office, Residences, Churches</td>
<td>85</td>
</tr>
<tr>
<td>2 Shopping Malls</td>
<td>65</td>
</tr>
<tr>
<td>3 Footbridges - Indoor</td>
<td>92</td>
</tr>
<tr>
<td>4 Footbridges - Outdoor</td>
<td>92</td>
</tr>
</tbody>
</table>

* 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.

0.06 for full height partitions between floors.

Type of building from table 4.1 (page 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 slabs

Steel Beam Properties

- Left Girder: W21X44
  - A = 13 in²
  - l = 843 in²
  - d = 20.66 in

- Right Girder: W21X44
  - A = 13 in²
  - l = 843 in²
  - d = 20.66 in

- Beam: W21X44
  - A = 13 in²
  - l = 843 in²
  - d = 20.66 in

Deck Properties

- Concrete:
  - wc = 415 psf
  - ft = 4,960 psi

- Composite Slab above deck:
  - 4.6 in

- Metal Deck depth = 2 in
- Gage = 20
- Slab weight = 69.0 psf
- Metal Deck weight = 1.97 psf
- Slab + Deck Weight = 71.0 psf

Effective Slab Width = 84 inches
Uniform Dist. Load = 645.79 psf

Beam Mode Properties

- $E_a = \frac{w^{1.5} \text{kip}^{0.5}}{l}$
- $n = \text{modular ratio} = E_a l^{1.5} E_c = 0.15$
- Composite Transformed Inertia Axis, $y = 0.250$ in
- Moment of Inertia of Composite section, $I = 3228$ in⁴
- Deflection under actual Dead + Live Load, $\delta_d = 0.130$ in
- Beam mode fundamental Frequency = 0.49 Hz
- Transformed slab moment of inertia per unit width, $D_s = 27.05$ in⁴/ft
- Transformed beam moment of inertia per unit width = 481.09 in⁴/ft

- $C_p = 2.0$ for posts or beams in most areas
- $C_p = 1.0$ for posts or beams parallel to an interior edge

Effective beam panel width = 15 min of
- $21/3$ times the girder span for an interior bay = 42 ft
- $21/3$ times the girder span for an interior bay = 30.27 ft
- $\delta_c = 1.5$ for Interior Bay

Effective weight of beam panel accounting for continuity = 159 kips
### Girdar Mode Properties

**Left Girder**
- Effective Slab width: 100.8 in
- Slab width Meets requirements
- Average Concrete depth: 5.5 in
- Composite Transformed Initial Axis, y = 0.973
- Moment of Inertia of Composite section, Iy = 5323 in^4
- Equivalent uniform loading: 2865.86 psf
- Deflection under actual Dead + Live Load, Δ1 = 0.131 in
- Girdar Mode fundamental Frequency = 9.78 Hz
- Transformed beam moment of inertia per unit width, δj = 106.06 in^3 / ft
- Transformed girdar moment of inertia per unit width, δg = 1.6
- Girdar Mode fundamental Frequency = 9.78 Hz
- Effective width for girdar panel mode = 64.5 ft
- Effective width for girdar panel mode = 54.33 ft
- Girdar Panel Weight, Wg = 107 kips
- Check before reducing, Δg and using δg

**Right Girder**
- Effective Slab width: 190.6 in
- Slab width Meets requirements
- Average Concrete depth: 5.5 in
- Composite Transformed Initial Axis, y = 0.973
- Moment of Inertia of Composite section, Iy = 5323 in^4
- Equivalent uniform loading: 2865.86 psf
- Deflection under actual Dead + Live Load, Δ1 = 0.131 in
- Girdar Mode fundamental Frequency = 9.78 Hz
- Transformed beam moment of inertia per unit width, δj = 106.06 in^3 / ft
- Transformed girdar moment of inertia per unit width, δg = 1.6
- Girdar Mode fundamental Frequency = 9.78 Hz
- Effective width for girdar panel mode = 64.33 ft
- Effective width for girdar panel mode = 54.33 ft
- Girdar Panel Weight, Wg = 107 kips
- Check before reducing, Δg and using δg

### Table 6.2 Values of Footfall Impulse Parameters

<table>
<thead>
<tr>
<th>Walking Pace</th>
<th>Steps/Minute</th>
<th>Uv (lb-ft/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 (fast)</td>
<td>5</td>
<td>25000</td>
</tr>
<tr>
<td>75 (moderate)</td>
<td>2.8</td>
<td>5500</td>
</tr>
<tr>
<td>30 (slow)</td>
<td>1.4</td>
<td>1500</td>
</tr>
</tbody>
</table>

Deflection factors to be used is 1/64 for simple open beams
95 for beams with built-in ends
6.59E-06 in dB
1.75E-06 in dB
1.75E-06 in dB

The effective number of tip-beams: NoEff max of 2.77
Mid-bay flexibility = DefP = 2.08E-06 in dB

---

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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.

<table>
<thead>
<tr>
<th>Steel Beam Properties</th>
<th>Dock Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>Left Gird A W21X62</td>
<td>A = 18.3 in²</td>
</tr>
<tr>
<td>Length 21 ft</td>
<td>b = 1300 in⁴</td>
</tr>
<tr>
<td></td>
<td>d = 26.99 in</td>
</tr>
<tr>
<td>Right Gird B W21X62</td>
<td>A = 18.2 in²</td>
</tr>
<tr>
<td>Length 21 ft</td>
<td>b = 1300 in⁴</td>
</tr>
<tr>
<td></td>
<td>d = 26.99 in</td>
</tr>
<tr>
<td>Beam W18X58</td>
<td>A = 16.2 in²</td>
</tr>
<tr>
<td>Length 30.75 ft</td>
<td>b = 020 in⁴</td>
</tr>
<tr>
<td>Spacing 7 ft</td>
<td>d = 14.11 in</td>
</tr>
</tbody>
</table>

| Actual Live Load = 11 pcf |
| Actual 30L = 4 pcf         |

Effective Slab Width = 34 inches
Uniform Distr. Load = 656.79 pcf

**Beam Mode Properties**

<table>
<thead>
<tr>
<th>w = w_d [k/ft]</th>
<th>3452 lbs</th>
</tr>
</thead>
<tbody>
<tr>
<td>n = modular ratio = 641.366</td>
<td>6.15</td>
</tr>
<tr>
<td>Composite Transformed Inertia Axis, y = 0.3268 in</td>
<td></td>
</tr>
<tr>
<td>Moment of Inertia of Composite section, I_y = 3253 in⁴</td>
<td></td>
</tr>
<tr>
<td>Deflection under actual Dead + Live Load, δ = 0.014 in</td>
<td></td>
</tr>
<tr>
<td>Beam mode fundamental Frequency = 9.46 Hz</td>
<td></td>
</tr>
<tr>
<td>Transformed slab moment of inertia per unit width, D_y = 27.05 in³/ft</td>
<td></td>
</tr>
<tr>
<td>Transformed beam moment of inertia per unit width = 456.16 in³/ft</td>
<td></td>
</tr>
<tr>
<td>C_y = 2.0</td>
<td>2.0 for joists or beams in most areas</td>
</tr>
<tr>
<td>1.6 for joists or beams parallel to an interior edge</td>
<td></td>
</tr>
<tr>
<td>Effective beam panel width is min of 30 1/8 in for interior spans</td>
<td></td>
</tr>
<tr>
<td>30 1/8 in for bay with factor of continuity 1.5</td>
<td></td>
</tr>
<tr>
<td>Effective weight of beam panel accounting for continuity = 31 lbs</td>
<td></td>
</tr>
</tbody>
</table>

**Girder Mode Properties**

<table>
<thead>
<tr>
<th>Left Gird A</th>
<th>Effective Slab width = 100 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Concrete depth = 5.5 in</td>
<td></td>
</tr>
<tr>
<td>Composite Transformed Inertia Axis, y = 0.348 in</td>
<td></td>
</tr>
<tr>
<td>Moment of Inertia of Composite section, I_y = 4644 in⁴</td>
<td></td>
</tr>
<tr>
<td>Equivalent uniform loading = 23.4756 pcf</td>
<td></td>
</tr>
<tr>
<td>Deflection under actual Dead + Live Load, δ = 0.086 in</td>
<td></td>
</tr>
<tr>
<td>Girder mode fundamental Frequency = 11.43 Hz</td>
<td></td>
</tr>
<tr>
<td>Transformed beam moment of inertia per unit width, D_y = 4861.6 in³/ft</td>
<td></td>
</tr>
</tbody>
</table>
Effective width for girders supporting joists connected to girders flange
for girders supporting beams connected to girders web
(2/3 of times the girder span for an interior bay)
Girder Panel Weight, Wg =
Check before reducing $\Delta y$ and using $\Delta y'$

Maximum deflection of girders due to weight, $\Delta y'$ =

Effective Slab width =
Slab width meets requirements

Girder panel moment of inertia per unit width, $I_g$ =
Deflection under actual Dead + Live Load, $\Delta y$ =
Equivalent uniform loading =
Girder Mode Fundamental Frequency =
Transformed girder moment of inertia per unit width, $I_g'$ =

$C_g$ =
for girders supporting joists connected to girders flange
for girders supporting beams connected to girders web

Effective width for girders panel mode =
(2/3 of times the girder span for an interior bay)
Girder Panel Weight, Wg =
Check before reducing $\Delta y$ and using $\Delta y'$

Maximum deflection of girders due to weight, $\Delta y'$ =

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

Floor Fundamental Frequency, $f$ =

For office occupancy without full-height partitions, $\mu$

Acceleration Limit =

<table>
<thead>
<tr>
<th>Walking Pace</th>
<th>Steps/minute</th>
<th>$\mu$</th>
<th>$U_c$ (lb-ft)$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 (fast)</td>
<td>5</td>
<td></td>
<td>25000</td>
</tr>
<tr>
<td>73 (moderate)</td>
<td>2.9</td>
<td></td>
<td>5000</td>
</tr>
<tr>
<td>20 (slow)</td>
<td>1.4</td>
<td></td>
<td>1500</td>
</tr>
</tbody>
</table>

Table 6.2 Values of Footfall Impulse Parameters

Deflection factors to be used is 1/

Mid-span flexural of beam =
Mid-span flexural of left girders =
Mid-span flexural of right girders =
The effective number of tee beams =
The effective number of tee beams =
Mid-span flexural =

Maximum expected velocity (50 step/min.)

Maximum expected velocity (75 step/min.)

Maximum expected velocity (100 step/min.)

Satisfactory

4.8 for simple span beams
96 for beams with built-in ends

5.53 E-06 in/ft
1.24 E-06 in/ft
1.24 E-06 in/ft
2.77
2.77
2.62 E-06 in/ft
5.04 E-04 in/sec
1.85 E-03 in/sec
8.40 E-03 in/sec
504 u-in/sec OK
1848 u-in/sec OK
< 2000

OK | 102
Appendix D: Flat Plate Calculations

Even though this is 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 leg bay.
The direct design method will be chosen to design this typical beam 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 beams:

\[ t_{min, min} = \frac{8m}{30} - \frac{(30.75)}{30} = 11.63 \text{ in} \]

Use 18 in for constructability and pourability

Direct Design Method:

1. 3 continuous spans in each direction X does not apply in W E-W direction but see notes above

2. Panel ratio \( P_e \leq 2 \) \( P_e = \frac{30.75}{21} \leq 1.5 < 2 \Rightarrow \text{OK} \)

3. \( P_e \geq \frac{30.75}{21} \Rightarrow \frac{W_2}{3} (30.75) > 80.5 \Rightarrow \text{OK} \)

4. Can’t have a column offset of more than 10% length \( \Rightarrow \text{OK} \)

5. \( W_c \leq 2 W_d \) \( W_c = 125 \text{ lb} \)

\[ W_d = (11.63') (150 \text{ lb}') = 137.5 \text{ lb} \text{F} \]

\[ W_d = 157.5 \text{ lb} \text{F} \Rightarrow \text{OK} \]

\[ W_d = 20 \text{ lb} \text{F} \]

\( \Rightarrow \text{OK} \) to use direct design method.
Punching shear (two-way) action:

\[ V_u = w_u \times \text{Area} \]

\[ V_u = 0.389 \times \left[ (2.1)^2 (30.75) \right] = 248.6 \text{ kN} \]

Shear strength:

\[ b_o = (20 + 10.5) \times 4 = 122^\circ \]

\[ \frac{b_o}{d} = \frac{122}{10.5} = 11.62 \]

\[ f_c = 1 < 2 \text{ for equivalent square.} \]

\[ V_c = \left[ \frac{(K_p)}{b_o/d} + 2 \right] \sqrt{f_c} \text{ byeq} \]

\[ a_s = 40 \text{ for internal columns.} \]

\[ V_c = \left( \frac{41}{11.62} + 2 \right) \sqrt{40,000} \left( 122 \times 10.5 \right) = 440.9 \text{ kN} \]

\[ \phi V_c = 0.75 (440.9) = 330 \text{ kN} > 248.6 \text{ kN} \]

For edge columns, \( a_s = 30 \)

\[ \phi V_c = 0.75 (371.2) = 278.4 \text{ kN} > 248.6 \text{ kN} \]

\[ \Rightarrow \text{OK!} \]
Wide beam action:

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

\[ z = 7.08' \]

\[ V_u = V_c = 2 \frac{f_c}{b_d} b_w d \]

\[ V_r = 490.1 \text{ kN} \]

\[ V_u = \frac{0.75 \times 490.1 \times 1}{30.75} = 307.6 \]

\[ V_u = W_0 \times 7.08' \times (30.75) = 0.385 \times 7.08 \times 30.75 \]

\[ V_r = 841.69 \]

\[ V_u > V_r \Rightarrow OK! \]

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.
Frame A

$\frac{1}{2}$ column strip: $\frac{30^\circ - 9^\circ}{2} = 7.68''$

Middle strip: $\frac{30^\circ - 9^\circ}{2} = 15.375''$

Moment: $M_o = \frac{W_o \times d_o}{8}$

$W_o = 1.2 \times 1.6 \times 3 = 1.2(5.75) + 1.6(125) = 589$ psf

$M_o = (589)(21)(30.75 - \frac{20}{12})^2$

$M_o = 843.7$ ft-k

$0.35 M_o = 292.3$ ft-k

$0.65 M_o = -501.4$ ft-k

Using ACI - 318-05 § 13.6.3.3 for exterior edge gusset reinforcement

Frame B

$\frac{1}{2}$ column strip: $\frac{21^\circ - 0^\circ}{4} = 5.25''$

Moment: $M_o = \frac{W_o \times d_o}{8}$

$W_o = 1.2 \times 1.6 \times 3 = 1.2(5.75) + 1.6(125) = 589$ psf

$M_o = (589)(21)(30.75 - \frac{20}{12})^2$

$M_o = 559$ ft-k

$0.35 M_o = 195.6$ ft-k

$0.65 M_o = -368.4$ ft-k

Using ACI - 318-05 § 13.6.3.3 for exterior edge gusset reinforcement
Frame A:
- 561.4 <
  75% to C.S. = -421.1 ft.k → 100% slab
  25% to H.S. = -140.3 ft.k
+ 302.3 <
  60% to C.S. = 181.4 ft.k → 100% slab
  40% to H.S. = 120.9 ft.k
- 363.4 <
  75% to C.S. = -272.5 ft.k → 100% slab
  25% to H.S. = -90.85 ft.k
+ 195.6 <
  60% to C.S. = 117.4 ft.k → 100% slab
  40% to H.S. = 78.2 ft.k

Summary:

Frame B:

<table>
<thead>
<tr>
<th>Total H.</th>
<th>-561.4</th>
<th>+302.3</th>
<th>-561.4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment in C.S.</td>
<td>-421.1</td>
<td>+181.4</td>
<td>-421.1</td>
</tr>
<tr>
<td>Moment in H.S.</td>
<td>-140.3</td>
<td>+120.9</td>
<td>-140.3</td>
</tr>
</tbody>
</table>

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.
### Middle Strip:

<table>
<thead>
<tr>
<th>Description</th>
<th>Frame A</th>
<th>Frame B</th>
</tr>
</thead>
<tbody>
<tr>
<td>1) Moment Mu (K-ft)</td>
<td>$-110.3$</td>
<td>$+120.9$</td>
</tr>
<tr>
<td>2) Width of Middle Strip</td>
<td>$184.5^\circ$</td>
<td>$184.5^\circ$</td>
</tr>
<tr>
<td>3) Effective depth</td>
<td>$10.875^\circ$</td>
<td>$10.875^\circ$</td>
</tr>
<tr>
<td>4) $M_n = Mu/\phi$</td>
<td>$-155.9$</td>
<td>$+184.3$</td>
</tr>
<tr>
<td>5) $R = \frac{N_m \times 1200}{b_2 d^2}$</td>
<td>$85.74$</td>
<td>$73.86$</td>
</tr>
<tr>
<td>6) $f'(\text{table A.5a, Wilson})$</td>
<td>$0.0014$</td>
<td>$0.0013$</td>
</tr>
<tr>
<td>7) $A_s = \frac{1}{b_1 d}$</td>
<td>$2.91 \text{ in}^2$</td>
<td>$2.61 \text{ in}^2$</td>
</tr>
<tr>
<td>8) $A_{s, min} = 0.0018 b_1 d$</td>
<td>$3.93 \text{ in}^2$</td>
<td>$3.93 \text{ in}^2$</td>
</tr>
<tr>
<td>9) $N_o = \frac{f' d_1}{0.44 \text{ Area of } \pi d^2}$</td>
<td>$3.1 \approx 10$</td>
<td>$6.1 = 7$</td>
</tr>
<tr>
<td>10) $N_{min} = \frac{\text{Width of strip}}{2}$</td>
<td>$7.67 \approx 8$</td>
<td>$5.25 \approx 6$</td>
</tr>
</tbody>
</table>

**USE** ⇒ 10, 10, 7, 7
<table>
<thead>
<tr>
<th>Description</th>
<th>Frame A</th>
<th>Frame B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment Mx (ft-k)</td>
<td>-421.1</td>
<td>+181.4</td>
</tr>
<tr>
<td>Width of column strip</td>
<td>184.5</td>
<td>184.5</td>
</tr>
<tr>
<td>Effective depth</td>
<td>10.75</td>
<td>10.75</td>
</tr>
<tr>
<td>d = t - 0.75 - %2</td>
<td>12&quot;</td>
<td>12&quot;</td>
</tr>
<tr>
<td>Assume #8 bars</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mx = Mx / 1200</td>
<td>263.3</td>
<td>102.1</td>
</tr>
<tr>
<td>f (table 4.5a)</td>
<td>0.00415</td>
<td>0.0014</td>
</tr>
<tr>
<td>As = f . b . d</td>
<td>8.32 in²</td>
<td>2.78 in²</td>
</tr>
<tr>
<td>As, min = 0.001 in²</td>
<td>3.93 in²</td>
<td>3.93 in²</td>
</tr>
<tr>
<td>N = larger of 7 x 8</td>
<td>11.25 m</td>
<td>5.05 m</td>
</tr>
<tr>
<td>N, min = width of slab / 2 t</td>
<td>8</td>
<td>8</td>
</tr>
</tbody>
</table>

Use: → 12  8  8  7
Deflection check:

Assumed (weighted average) = 67.5 % Moment to column,

E = 60,000 ksi for 40 ksi

t = 12".

Intermediate deflection due to dead load:

\[ w_0 = (0.158)(10.75)(0.675) = 3.279 \text{ kips} \]

\[ I_y = \left(\frac{194.5 \times 12}{12}\right)^3 = 2656 \text{ in}^4 \]

\[ E = \frac{50000 \times 1000}{3005 \times 10} = 3005 \text{ ksi} \]

\[ \Delta_p = \frac{0.0026 \times (3.279)(30.75)^4(12)^3}{3005 (2.578^8)} = 0.137^\circ \]
\[ W_0 = (0.158) (30.75) (0.725) = 1.579 \text{ k/ft} \]
\[ l_g = 26.568 \text{ in}^{2} \quad E_t = 3.605 \text{ ksi} \]
\[ D_{\text{D}} = \frac{0.0026 (1.579) (30.75)^{4} (12)}{3605} = 0.066'' \]

Total immediate \( D \) due to DL + WL = 0.066 + 0.137'' = 0.203''

Immediate Deflection due to total live load:
- Column strip: \( W_1 = (125) (30.75) (0.725) = 2.594 \text{ k/ft} \)
  \[ \Delta_{l}^{\text{max}} = \frac{0.0048 (2.594) (30.75)^{4} (12)}{(3605) (26.568)} = 0.201'' \]
- Middle strip: \( W_2 = (125) (30.75) (0.375) = 1.249 \text{ k/ft} \)
  \[ \Delta_{l}^{\text{max}} = \frac{0.0048 (1.249) (30.75)^{4} (12)}{(3605) (26.568)} = 0.097'' \]

Total immediate due to live = 0.201 + 0.097 = 0.298''

Additional DL \( \Delta \) after a long time due to DL load:
Assume \( \lambda = 3.0 \)
\[ \Delta_3^{\text{max}} = 3.0 \left[ 0.203'' + 0.25 (0.231) \right] = 0.883'' \]

Live load deflection check: from ACI 318-08 Table 9.5(b)
\[ \frac{8}{360} \text{ (beams not supporting or attached to non-structural elements likely to be damaged by large deflections)} \]
\[ \frac{8}{360} = \frac{30.75 \times 12}{360} = 1.025'' > 0.238'' \]
Total Deflection After partitions = 0.1 (0.203) + 0.298 + 1.075"

\[ \Delta \text{def} \text{ for partitions} = 1.343^\circ \]

\[ \text{ACT} \ 318.08 \ \%240 = \frac{30.75 \times 12}{240} = 1.538^\circ > 1.343^\circ \]

\[ \rightarrow \text{OK!} \]

Thus the design system works for flexure, shear and deflection and since \( \theta = 12^\circ \) and deflection have

F.S of 3 we can assume that this floor system will meet the floor vibration requirements for the

Alzheimer's Center.
Appendix E: One Way Slab Calculations

Minimum thickness of slab from table 3.5(a) ACI 318-08

Exterior Bay: \[
\frac{f_c}{f_y} = \frac{4,000}{60} = 66.67
\]

- Use a thickness of 4” for durability and fire protection
Assume #4 bars to calculate \( d \) and clear cover of \( 3/4" \):
\[
d = h - \frac{3}{4} - \frac{d_{cv}}{2} = 4 - \frac{3}{4} - 0.5 = 3\ 
\]

Load: \( W_D = \left( \frac{4L}{12} \right) (150) = 50 \text{ p.f.f.} \)

Superimposed: \( 20 \text{ p.f.f.} \)

\( W_e = 125 \text{ p.f.f.} \) uneducable

\( W_e = 1.2D + 1.6L = 1.2 (70) + 1.6 (125) = 284 \text{ p.f.f.} \)

Assume: The section is torsion (confined) \( \phi = 0.9 \)

\( b = 20" \) for constructability and form work costs

\( W_e = 125 < 3(W_D) = 180 \text{ p.f.f.} \) we can use ACI

Moment coefficients

**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 way will be used.
Punching shear (two-way) action:

\[ V_d = W_{ax} \times \text{Area} \]
\[ V_d = 385 \times \left( \frac{21}{14} \right) \left( \frac{30.75}{20 + 0.5} \right)^2 \]
\[ V_d = 248.6 \text{ kN} \]

Shear strength:

\[ b_o = (20 + 0.5) \times 4 = 122" \]
\[ \frac{b_o}{d} = \frac{122}{10.5} = 11.62 \]
\[ \phi = 1 < 2 \text{ for equivalent square} \]
\[ V_c = \left[ \frac{\phi d}{\phi} \right] + 2 \left[ \frac{\phi d}{\phi} \right] \left( \frac{b_o}{d} \right) \text{ kN} \]
\[ \phi = 0.75 \text{ for internal columns} \]
\[ V_c = 0.75 \times 440.9 = 330 \text{ kN} > 248.6 \text{ kN} \]

For edge columns, \( d_s = 30" \)
\[ V_c = 0.75 \times 278.4 = 209.4 \text{ kN} \]
\[ = \text{OK!} \]
Design of Reinforcement:

<table>
<thead>
<tr>
<th>Description</th>
<th>Exterior Support</th>
<th>Exterior Midspan</th>
<th>First Midspan</th>
<th>Interior Midspan</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. % Reinforcement</td>
<td>6.4&quot;</td>
<td>6.4&quot;</td>
<td>6.4&quot;</td>
<td>6.4&quot;</td>
</tr>
<tr>
<td>2. w = x(x-1/4)</td>
<td>8.02</td>
<td>8.02</td>
<td>8.02</td>
<td>8.02</td>
</tr>
<tr>
<td>3. H = 2x(x/2-1/4)</td>
<td>-1/24</td>
<td>1/14</td>
<td>-1/14</td>
<td>-1/14</td>
</tr>
<tr>
<td>4. Mmax = (x/2)</td>
<td>0.573</td>
<td>-0.802</td>
<td>0.573</td>
<td>0.573</td>
</tr>
<tr>
<td>5. As (required)</td>
<td>0.075</td>
<td>0.043</td>
<td>0.061</td>
<td>0.043</td>
</tr>
<tr>
<td>6. As, min in %</td>
<td>0.086</td>
<td>0.086</td>
<td>0.086</td>
<td>0.086</td>
</tr>
<tr>
<td>7. Bars selected</td>
<td>No. 4 @ 12&quot;</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8. Final As.</td>
<td>0.2&quot;</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Check for spacing:

\[ s = 15 \left( \frac{40,000}{35} \right) - 2.5 \geq \frac{12}{\left( \frac{40,000}{85} \right)} \]

\[ s = 15 - 2.5(0.75) \geq 12 \]

\[ s = 12.75 \geq 12 \Rightarrow OK! \]

Transverse direction:

As for shrinkage and temperature = 0.0018 (12)(14) = 0.086

Maximum spacing \( \leq 5h = 20" \)

Slab detail:

\( 7" \) slab

No. 4 @ 12" for top and bottom slabs

No. 4 @ 18" for transverse steel.
Beam Design:

\[ b = 20^\prime, \quad h = \frac{8}{12} = 6.5' \]

Select \( h = 24 \) in (between two values)

\[ W_{beam} = \frac{(24 - 21)(20)}{144} \times 150 = 417 \text{ lb/ft} = 417 \text{ kN/m} \]

\[ U_{wet} = 0.8 \text{ psf} 	imes 12 = 8.4 \text{ kips} = 8.4 \text{ kN/m} \]

\[ W_u = 1.2 \text{ Wd} + 1.0 \text{ Wl} = 1.2(4.440) + 1.0(1.5) = 3.91 \text{ kips} \]

First Set:

\[ M_u = \frac{W_u h^2}{24} = \frac{3.91(30.75 - 20)^2}{24} = 3.91(29.08)^2 \]

\[ M_u = -3.91 \text{ kips ft} \]

First Set:

\[ M_u = \frac{-3.91(29.08)}{14} = -236.2 \text{ kips ft} \]

\[ b_x = \frac{1}{4} \text{ span length} = \sqrt{\frac{4}{12} (30.75)(42)} = 92.75 \text{ in} \]

\[ b_x + 16 \text{ ft} = 20 + 16(4) = 84 \text{ in} \]

\[ b_x \pm \frac{1}{2} (2 \text{ clear distance}) = 20 \pm \frac{1}{2} (26.25) = 84 \text{ in} \]
Assume $g = 4,000$ lb, 2.4 in.

Description:

<table>
<thead>
<tr>
<th>First exterior</th>
<th>Midspan</th>
<th>First interior</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mu $(f_0 A)$</td>
<td>$-137.8$</td>
<td>$+236.2$</td>
</tr>
<tr>
<td>$A_d = \frac{f_0 (f_0 - 2)}{4}$</td>
<td>$1.64$</td>
<td>$236.2/4 = 59.1$</td>
</tr>
<tr>
<td>As (chosen)</td>
<td>$24.5% &gt; 20$</td>
<td>$3% &gt; 30$</td>
</tr>
<tr>
<td>$f_d / f_0 &lt; 0.025$</td>
<td>$0.0048$</td>
<td>$0.0048$</td>
</tr>
<tr>
<td>$d$</td>
<td>$0.85 ft$</td>
<td>$0.85 ft$</td>
</tr>
<tr>
<td>$H_u$ (top)</td>
<td>$166.6 ft \cdot k$</td>
<td>$166.6 ft \cdot k$</td>
</tr>
<tr>
<td>$a = \frac{H_t}{0.85 f_0 b}$</td>
<td>$1.765$</td>
<td>$2.65$</td>
</tr>
<tr>
<td>$c = c/\sqrt{f_0}$</td>
<td>$2.08$</td>
<td>$3.117$</td>
</tr>
<tr>
<td>$L_5 = (d-e) / c$</td>
<td>$0.043$</td>
<td>$0.043$</td>
</tr>
<tr>
<td>$f_5 &gt; 0.05$</td>
<td>$0.0176$</td>
<td>$0.0176$</td>
</tr>
<tr>
<td>$H_u$ (top)</td>
<td>$271.5 ft \cdot k$</td>
<td>$271.5 ft \cdot k$</td>
</tr>
<tr>
<td>$&gt; H_u$ (OK)</td>
<td>$&gt; H_u$ (OK)</td>
<td>$&gt; H_u$ (OK)</td>
</tr>
<tr>
<td>$d_{min} = \sqrt{3V_b / b_d}$</td>
<td>$750$</td>
<td>$750$</td>
</tr>
<tr>
<td>$\alpha = 0.0005$</td>
<td>$0.0005$</td>
<td>$0.0005$</td>
</tr>
<tr>
<td>$V_u = \alpha (d-e)(2\cdot h)$</td>
<td>$3.51 (30.95 - 20/19) / 6 = 55.84 k$</td>
<td>$55.84 k$</td>
</tr>
<tr>
<td>$2\sqrt{V_u (2\cdot h) / (2 \cdot h)}$</td>
<td>$54.24 k$</td>
<td>$54.24 k$</td>
</tr>
<tr>
<td>$V_5 = 42.88 k$</td>
<td>$42.88 k$</td>
<td>$42.88 k$</td>
</tr>
<tr>
<td>$V_u &gt; V_u = 0.35 (54.24 + 42.88) = 72.84 k$</td>
<td>$72.84 k$</td>
<td>$72.84 k$</td>
</tr>
</tbody>
</table>
Deflection (Assume simply supported)

\[ D_{UL} = \frac{5wL^4}{384EI} = \frac{5(1.5)(30.75)^4(1728)}{384(57,000)(144)(20(24)^2)} = 0.363'' \]

\[ D_{UL} = \frac{8}{480} = \frac{30.75 \times 12}{480} = 0.769'' \]

\[ D_{UL} = \frac{5(1.257 + 1.5)(30.75)^4(1728)}{384(57,000)(144)(20(24)^2)} = 0.664'' \]

\[ D_{UL} = \frac{8}{280} = \frac{30.75 \times 12}{280} = 1.315'' \]

Long term deflection:

\[ w_{UL} = \frac{5(1.257 + 1.5)(15)}{24} = 4.756'' \]

\[ D_{UL} = \frac{5(4.756)(30.75)^4(1728)}{384(57,000)(144)(20(24)^2)} = 1.186'' < D_{UL} \text{ max} = \text{OK!} \]
Girder Design

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

$$P_a = 56.86k (from V_a)$$

$$P_b = 56.86k$$

$$M = \frac{P_a \cdot b}{2}$$

$$V = \frac{P_b}{2}$$

$$M = \frac{w \cdot b^2}{12}$$

$$V = \frac{w \cdot d}{2}$$

$$M = \frac{w \cdot (2h - x^2)}{12}$$

$$V = \frac{w \cdot (2h - x)}{2}$$

$$M_{max} = 14.74k + 5.12k = 19.86k$$

$$M_{min} = 15.32k + 170.3k + 88.45k - 280.67k$$
Description:

1. $M_1 = (4.91)$
2. $A_2 = \frac{M_2}{d}$
3. $A_3$, chosen
4. $f = \frac{A_3}{b}$
5. $l_f = \frac{M_2 (112) (12)}{20 + 2 \left( \frac{M_2 (112) - 20}{2} \right)} = 2.52$
6. $M_{1b} = \left( 0.85 \left( \frac{8}{112} \right) \left( \frac{d - \frac{b}{2}}{2} \right) \right) = 0.9 (0.85) (8) (112) (d - \frac{b}{2})$
7. $d = 24.15 - 0.5 - 1.128 = 21.44 \Rightarrow M_{1b} = 12.49 \text{ kN} > M$
8. $\alpha = \frac{A_3 b}{b} = 0.88$
9. $c = \frac{g}{b} = 1.04$
10. $\epsilon_5 = \left( \frac{d - c}{c} \right) 0.030 = 0.053 \geq 0.025 \geq 0.005$
11. Assume same previous same dimensions $A_{min} = 1.143 \text{ m}^2$
12. Spectral Requirements is the same
13. $V_u = V_{max} = 61.74 \text{ kN}$
   $\phi V_u > V_{u \phi} \text{ OK!}$
Deflection:

\[ \Delta = \frac{PL^3}{24EI} + \frac{54L}{5704} \]

\[ LL = \frac{23.06(21)^2(778)}{28(57,000 + 4,000)(10(14))^3} + 0.35' < \frac{R}{410} = 0.55' \]

\[ TL = \frac{(23.06 + 19.33)(21)(778)}{28(57,000 + 1,000)(10(14))^3} + \frac{5(412)(21)(778)}{384(57,000 + 4,000)(10(14))^3} \]

\[ \text{Restrain} = 0.718' + 0.022 = 0.74' < \frac{R}{240} = 1.05' \]

Guides at Midspan

Guides at Supports