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Executive Summary

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

Gravity loads were calculated or verified for the building, including the total weight of the structure using simplifying assumptions. These were investigated by spot checks of four gravity members: an interior column, a slab panel, a joist framing system and a beam. The members were chosen as typical as possible to mimic the entire building structure. They were all found to be satisfactory; hence the assumptions made were then verified to a degree.

Lateral load calculations were performed in accordance with ASCE 7-05 procedures. It was found that wind loads will control over seismic by a factor of about 3.6 in the East-West direction and 2.5 in the North-South direction. The design base shear in the North-South direction was calculated to be 682.01 k, and in the East-West direction was calculated to be 892.22 k. These loads were not compared to a design base shears as they were not listed on the structural drawings. It was also found that seismic was not required for this region since wind control most of times. Further lateral analysis will be performed in Technical Report 3.

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

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

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

The façade of the building is mainly divided into two parts. The east side consist of curtain wall glazing and Aluminum panels. The curtain wall glazing consists of: Clear Tempered, insulating laminated spandrel glass, clear insulating laminated glass, insulated fritted glass 30% silkscreen coverage pattern, insulating fritted glass 50% silkscreen coverage pattern, sunscreens and louvers. The west side consists of cement plaster with the same curtain wall like glazing and...
decorative grille with louver at the top. As for the roof the use of Thermoplastic Membrane roofing was chosen with ½”/ft slope with Aluminum parapet for architectural reasons.

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

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

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

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

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

- 2001 Florida Building Code with 2003 updates
- 2001 Florida Building Mechanical Code with 2003 updates
- 2001 Florida Building Plumbing Code with 2003 updates
- 2001 Florida Building Fuel Gas Code with 2003 updates
- 2001 Florida Building Accessibility Code as Ch.11 and Energy Code as Ch.13
- 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</th>
<th>Usage</th>
<th>Weight</th>
<th>Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spread footing</td>
<td>Normal</td>
<td></td>
<td>3000</td>
</tr>
<tr>
<td>Mat slab foundation</td>
<td>Normal</td>
<td></td>
<td>3000</td>
</tr>
<tr>
<td>Precast Joist Webs and soffit beams</td>
<td>Normal</td>
<td></td>
<td>5000</td>
</tr>
<tr>
<td>Cast-in-place slab</td>
<td>Normal</td>
<td></td>
<td>4000</td>
</tr>
<tr>
<td>Columns, typical</td>
<td>Normal</td>
<td></td>
<td>4000</td>
</tr>
<tr>
<td>Columns, as noted</td>
<td>Normal</td>
<td></td>
<td>6000</td>
</tr>
<tr>
<td>Precast Masonary Lintels</td>
<td>Normal</td>
<td></td>
<td>5000</td>
</tr>
<tr>
<td>Housekeeping Pads</td>
<td>Normal</td>
<td></td>
<td>4000</td>
</tr>
<tr>
<td>General Structure Concrete</td>
<td>Normal</td>
<td></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

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

Note: Welding Electrodes used were E70XX

<table>
<thead>
<tr>
<th>Masonary</th>
<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>
<td></td>
</tr>
<tr>
<td>Mortar</td>
<td>ASTM C270, M</td>
<td>f'_{c} = 2500</td>
<td></td>
</tr>
<tr>
<td>Mortar</td>
<td>ASTM C270, S</td>
<td>f'_{c} = 1800</td>
<td></td>
</tr>
<tr>
<td>Grout</td>
<td>ASTM C476</td>
<td>f'_{c} = 3000</td>
<td></td>
</tr>
<tr>
<td>Joint Reinforcement</td>
<td>ASTM A82, Truss Type</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 2 - Material Used in building: Concrete, Steel, Masonary
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](image-url)
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 4,000 psi strength. However, the columns supporting the research labs where the heavy equipment exists and vibration criteria need to be attained a 6,000 psi 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.

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.

**Atrium Wall Framing / Floor vibration Criteria**

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

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

**Gravity Loads**

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

**Dead and Live Loads**

The structural drawing S001 lists the superimposed dead loads to be used. That last is summarized in figure 10. The SP for Ceilings, lighting, plumbing, fire protection, flooring, and
HVAC for roof over mechanical levels is higher than usual because all the mechanical system that supplies the research labs that require special feed are situated in that area. These systems include packaged air handlers, on-site chillers, and gas fired boilers.

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

<table>
<thead>
<tr>
<th>Description</th>
<th>Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ceilings, lighting, plumbing, fire protection, flooring, and HVAC all</td>
<td>14 psf</td>
</tr>
<tr>
<td>Ceilings, lighting, plumbing, fire protection, flooring, and HVAC for roof over mechanical levels</td>
<td>40 psf</td>
</tr>
<tr>
<td>except mechanical</td>
<td>20 psf</td>
</tr>
<tr>
<td>allowance for roofing system</td>
<td>20 psf</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:
\[ R = 23.1 (1+D/L) \]
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%.

<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>125psf</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>

**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.

**Column Gravity Check**

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

---

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

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

---

**Figure 7-1 (continued) GROUND SNOW LOADS, \( p_s \), FOR THE UNITED STATES (LB/FT²)**
It was found that Column I-8 meet the required strength to carry the associated gravity loads. The design live loads were used as opposed to the ASCE 7-05 live loads for comparison purposes. The phi Pn calculated came to 1,641 kips more than the Mu calculated of 1,337 kips. Thus the column passed with 0.815 of its capacity being used. Finally, the column meet the reinforcement ratio required according to ACI 318-08 section 10.9.1.

**Beam and Joist Gravity Check**

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

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

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

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.
Lateral Loads

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

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

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

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

In addition, the wind pressures in the E-W direction are listed and diagramed in Figure 19. These were resolved into wind forces in the E-W direction, which are listed and diagramed in Figure 20. The resulting base shear is 892.22k. There was nothing to compare it to as there was no base shear provided on the drawings in either directions thus no conclusions can be drawn on discrepancies.
### Table 1: Design wind pressure for MWFRS in N-S Direction

<table>
<thead>
<tr>
<th></th>
<th>Level</th>
<th>Height / distance</th>
<th>qz/ qh</th>
<th>Wind pressure (psf)</th>
<th>Net pressure (+GCPI)</th>
<th>Net pressure (-GCPI)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>windward walls</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>0'</td>
<td>21.01</td>
<td>14.29</td>
<td>-6.14</td>
<td>34.72</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>14'-6&quot;</td>
<td>21.01</td>
<td>14.29</td>
<td>-6.14</td>
<td>34.72</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>29'</td>
<td>25.51</td>
<td>17.35</td>
<td>-3.08</td>
<td>37.77</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>43'-6&quot;</td>
<td>28.66</td>
<td>19.49</td>
<td>-0.94</td>
<td>39.92</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>58'</td>
<td>31.04</td>
<td>21.11</td>
<td>0.68</td>
<td>41.53</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>72'-6&quot;</td>
<td>33.18</td>
<td>22.56</td>
<td>2.13</td>
<td>42.99</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>87'</td>
<td>35.06</td>
<td>23.84</td>
<td>3.41</td>
<td>44.27</td>
<td></td>
</tr>
<tr>
<td><strong>Roof</strong></td>
<td>107&quot;</td>
<td>37.14</td>
<td>25.26</td>
<td>4.83</td>
<td>45.68</td>
<td></td>
</tr>
<tr>
<td><strong>leeward walls</strong></td>
<td>All</td>
<td>All</td>
<td>37.14</td>
<td>-13.83</td>
<td>-34.25</td>
<td>6.60</td>
</tr>
<tr>
<td><strong>sidewalls</strong></td>
<td>All</td>
<td>All</td>
<td>37.14</td>
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<td>-42.53</td>
<td>-1.67</td>
</tr>
<tr>
<td><strong>Roof</strong></td>
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<td>-29.93</td>
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<tr>
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<td>53.5-107</td>
<td>37.14</td>
<td>-27.65</td>
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<td>-7.23</td>
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</tr>
<tr>
<td></td>
<td>107-214</td>
<td>37.14</td>
<td>-16.54</td>
<td>-36.97</td>
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<td></td>
</tr>
</tbody>
</table>

**Figure 17**: List and diagram showing the wind pressure on the building in N-S direction.
<table>
<thead>
<tr>
<th>Floor level</th>
<th>Height / distance</th>
<th>Tributary below</th>
<th>Tributary above</th>
<th>Story force (K)</th>
<th>Story Shear (K)</th>
<th>Overturning Moment (k-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0' N/A</td>
<td>0.00</td>
<td>7.50</td>
<td>1095.00</td>
<td>73.49</td>
<td>682.01 k ft</td>
</tr>
<tr>
<td>2</td>
<td>14.5</td>
<td>7.00</td>
<td>1022.00</td>
<td>7.50</td>
<td>1095.00</td>
<td>608.52 k ft</td>
</tr>
<tr>
<td>3</td>
<td>29</td>
<td>7.00</td>
<td>1022.00</td>
<td>7.50</td>
<td>1095.00</td>
<td>1110.96 k ft</td>
</tr>
<tr>
<td>4</td>
<td>43.5</td>
<td>7.00</td>
<td>1022.00</td>
<td>7.50</td>
<td>1095.00</td>
<td>3747.82 k ft</td>
</tr>
<tr>
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<td>58</td>
<td>7.00</td>
<td>1022.00</td>
<td>7.50</td>
<td>1095.00</td>
<td>5185.96 k ft</td>
</tr>
<tr>
<td>6</td>
<td>72.5</td>
<td>7.00</td>
<td>1022.00</td>
<td>7.50</td>
<td>1095.00</td>
<td>6692.60 k ft</td>
</tr>
<tr>
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<td>87</td>
<td>7.00</td>
<td>1022.00</td>
<td>7.50</td>
<td>1095.00</td>
<td>10019.62 k ft</td>
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<tr>
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<td>107</td>
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<td>1460.00</td>
<td>10.00</td>
<td>1460.00</td>
<td>7136.52 k ft</td>
</tr>
</tbody>
</table>

Total base shear= 682.01 k
Total overturning Moment= 36276.04 k-ft

Figure 18: List and diagram showing the wind forces on the building in the N-S direction
Table showing the design wind pressure for MWFRS in E-W Direction.

<table>
<thead>
<tr>
<th>Type</th>
<th>Level</th>
<th>Height / distance</th>
<th>qz/ qh</th>
<th>Wind pressure (psf)</th>
<th>Net pressure (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward walls</td>
<td>1</td>
<td>0'</td>
<td>21.01</td>
<td>14.29</td>
<td>-6.14</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>14'-6&quot;</td>
<td>21.01</td>
<td>14.29</td>
<td>-6.14</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>29'</td>
<td>25.51</td>
<td>17.35</td>
<td>-3.08</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>43'-6&quot;</td>
<td>28.66</td>
<td>19.49</td>
<td>-0.94</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>58'</td>
<td>31.04</td>
<td>21.11</td>
<td>0.68</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>72'-6&quot;</td>
<td>33.18</td>
<td>22.56</td>
<td>2.13</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>87'</td>
<td>35.06</td>
<td>23.84</td>
<td>3.41</td>
</tr>
<tr>
<td></td>
<td>Roof</td>
<td>107'</td>
<td>37.14</td>
<td>25.26</td>
<td>4.83</td>
</tr>
<tr>
<td>Leeward walls</td>
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<td>All</td>
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</tr>
<tr>
<td>Sidewalls</td>
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<td>All</td>
<td>37.14</td>
<td>-22.10</td>
<td>-42.53</td>
</tr>
<tr>
<td>Roof</td>
<td>0-53.5'</td>
<td>37.14</td>
<td>-34.22</td>
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</tr>
<tr>
<td></td>
<td>53.5'-107'</td>
<td>37.14</td>
<td>-25.51</td>
<td>-45.93</td>
<td>-5.08</td>
</tr>
<tr>
<td></td>
<td>107'-214'</td>
<td>37.14</td>
<td>-18.69</td>
<td>-39.12</td>
<td>1.74</td>
</tr>
</tbody>
</table>

Figure 19: List and diagram showing the wind pressure on the building in E-W direction.
### Wind Forces - E-W Direction

<table>
<thead>
<tr>
<th>Floor level</th>
<th>Height / distance</th>
<th>Tributary below</th>
<th>Tributary above</th>
<th>Story force (K)</th>
<th>Story Shear (K)</th>
<th>Overturning Moment (k-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0'</td>
<td>N/A</td>
<td>7.50</td>
<td>1432.50</td>
<td>96.14</td>
<td>892.22</td>
</tr>
<tr>
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<td>14.5</td>
<td>7.00</td>
<td>7.50</td>
<td>1432.50</td>
<td>100.23</td>
<td>796.08</td>
</tr>
<tr>
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<td>29</td>
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<td>1432.50</td>
<td>107.48</td>
<td>695.85</td>
</tr>
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<td>1432.50</td>
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<td>475.65</td>
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<td>6</td>
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<td>7.50</td>
<td>1432.50</td>
<td>120.76</td>
<td>358.68</td>
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<tr>
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<td>87</td>
<td>7.00</td>
<td>7.50</td>
<td>1432.50</td>
<td>150.66</td>
<td>237.92</td>
</tr>
<tr>
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<td>10.00</td>
<td>1910.00</td>
<td>87.25</td>
<td>87.25</td>
</tr>
</tbody>
</table>

Total base shear = 892.22 k

Total overturning Moment = 47457.01 k-ft

---

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

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

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

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

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

Seismic Forces - N-S Direction

<table>
<thead>
<tr>
<th>Level</th>
<th>Story weight, ( w_x )</th>
<th>height (ft), ( h_x )</th>
<th>Story force (k), ( F_x=0.01 \times w_x )</th>
<th>Story Shear (k)</th>
<th>Overturning moment (k-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2nd</td>
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<td>28.9538</td>
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<tr>
<td>3rd</td>
<td>2892.82</td>
<td>29</td>
<td>28.9282</td>
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<td>838.9178</td>
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<td>4th</td>
<td>2892.82</td>
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<td>28.9282</td>
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<td>1677.8356</td>
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<td>6th</td>
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<tr>
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<tr>
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<td>107</td>
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<td>16.4842</td>
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<tr>
<td></td>
<td>19298.906</td>
<td></td>
<td>Base Shear = 192.98906</td>
<td>Total overturning moment = 10818.63977</td>
<td></td>
</tr>
</tbody>
</table>
Conclusion

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

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

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

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

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

Appendices start on the next page
### Appendix A: Gravity Load Calculations

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<th>Firedeck</th>
<th>Joist Spacing</th>
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</tr>
<tr>
<td></td>
<td>6'-6&quot;</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>241</td>
<td>201</td>
<td>157</td>
<td>138</td>
<td>115</td>
<td>102</td>
<td>85</td>
<td>69</td>
<td>62</td>
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</table>

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

<table>
<thead>
<tr>
<th>Size/Spacing</th>
<th>20&quot;</th>
<th>24&quot;</th>
<th>28&quot;</th>
</tr>
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<tbody>
<tr>
<td>4'8&quot;</td>
<td>90</td>
<td>84</td>
<td>88</td>
</tr>
<tr>
<td>5'8&quot;</td>
<td>90</td>
<td>84</td>
<td>88</td>
</tr>
<tr>
<td>6'8&quot;</td>
<td>75</td>
<td>77</td>
<td>75</td>
</tr>
<tr>
<td>7'8&quot;</td>
<td>67</td>
<td>72</td>
<td>-</td>
</tr>
<tr>
<td>8'8&quot;</td>
<td>68</td>
<td>75</td>
<td>-</td>
</tr>
<tr>
<td>10'0&quot;</td>
<td>65</td>
<td>72</td>
<td>-</td>
</tr>
</tbody>
</table>

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

---

http://psfjoist.com/loadtables4.html

5/25/2004
Partial Plan of Col 1-8

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

Normal weight of concrete is 150 psf.

Joint weights and beam solids will be taken from tables provided by manufacturer.

Column size doesn’t change through at the height of building.
The height of each floor is 14'-6" except for the roof of 20'-0"

Roof level:
- D: 5' NW slab
- S2 @ 5'-6" on both sides
- SD: 20 psf Roofing Gum 5001
- L: 80 psf wood 5003

kL for column = 4

Using ASCE 7-05
\[ L = L_0 \left( 0.25 + \frac{15}{kL \cdot A_T} \right) = 80 \left( 0.25 + \frac{15}{4 \times 4.441} \right) \]
\[ L = 48.57 > 0.6 \times 80 = 48 \text{ ft. OK!} \]

Using provisions made by the engineer for vertical members

\[ D = \frac{5}{12} \times 150 = 62.5 \text{ psf} \]
\[ S2 @ 5'-6" for dock of 4 3/8" & a depth of 12" \]
\[ S2 = 69 \text{ psf} \]
\[ S4 = 20 \text{ psf} \]
\[ P_D = 66.81 \times \quad P_L = 81.42 \times \]

Level 7:
- 62.5 for slab
- SP = 48 psf for lighting, plumbing, fire protection
- HVAC for roof over reach
- 16" deep or \( \frac{S3}{2} \) for long side \( w = 84 \text{ psf} \)
- 12" deep or \( \frac{S3}{2} \) for short side \( w = 76 \text{ psf} \)

\[ D_{S3} = (5\frac{1}{2}\text{f}) \times (81') \times 36 \text{psf} = 8.911 \times 36.102 \]
\[ D_{S2} = (15\frac{1}{2}\text{f}) \times (21') \times 84 \text{psf} = 87.195 \times \]
\[ D = (69.5 + 40) \times 441 = 45,025 \]
\[ P_D = 81.38 \times 10^6 \]

\[ L = 150 \text{ psf for I-8 level 7, according to provisions (see live loads in report)}, \]

\[ P_L = 120 \times 441 = 52,928 \text{ kN} \]

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

\[ L = 120 \times (0.607) = 72.84 \Rightarrow P_L = 32,122 \text{ kN} \]

Level 6

\[ 62.5 \text{ psf} \times 441 = 27,562.5 \text{ kN} \]

For long side: \[ 32 @ 6'6" = 46 \text{ psf} \]

\[ \text{SP} = 14 \text{ psf} \text{ for lighting, plumbing, fire protection, and HVAC for Roof and all levels 0 and 1}\]

\[ P_D = 77.61 \text{ kN} \]

Level 7

\[ 42 \text{ psf} \times 441 = 4,633 \text{ kN} \]

For short side: \[ 32 @ 6'6" = 47 \text{ psf} \]

\[ \text{SP} = 14 \text{ psf} \text{ for lighting, plumbing, fire protection, and HVAC for Roof and all levels 0 and 1}\]

\[ P_D = 77.61 \text{ kN} \]

Level 5

\[ 27.5625 \times 14.994 = 419,554 \text{ kN} \text{ (see level 7 provisions)} \]

Joists: \[ 52 @ 3'6" = 27.195 \text{ kN} \text{ (long side)} \]

\[ 52 @ 6'6" = 7.855 \text{ kN} \text{ (short side)} \]

\[ P_D = 77.61 \text{ kN} \]
Project: Column Check

Subject: Column Check

Task: Page: 41 of 5

Job #: No.

\( P_L = 44.1 \text{ kips} \) as \( L = 125 \text{ psi} \) same as level 6

level 4

same as level 5 and 6 \( \Rightarrow P_D = 44.1 \text{ kips} \)

level 3

same as level 4 \( \Rightarrow P_D = 44.1 \text{ kips} \)

level 2

same loading as before \( \Rightarrow P_D = 44.1 \text{ kips} \)

level 1

D. from slab and \( SP = 42.55 \text{ kips} \)

for long side \( 83 \times 6\text{"} \times 84 \text{ psf} \)

\( = 27.195 \text{ kips} \) (see level 7)

for short side \( 52 \times 6\text{"} \times 72 \text{ psf} \)

\( 4.2 \times (51.7\) \( \times 21) = 9.442 \text{ kips} \)

\( P_D = 78.194 \text{ kips} \)

\( P_L = 250 \times 0.8 = 200 \text{ psf} \) \( \Rightarrow P_L = 88.2 \text{ kips} \)

2 live loads exist here, a 850psf and 50psf

He 250 was chosen to be more conservative

\( P_D, \text{ live} = 161.362 \text{ kips} \)

\( P_L = 340.822 \text{ kips} \)

From columns, \( P = \left[ \frac{600 \times 125 \times 150}{144} \right] + \left( 0.05 \times 150 \right) = 36.26 \text{ kips} \)

\( P_L = 1.2(650.672) + 1.6(340.822) + 0.5(31.412) \)

\( L = 1336.77 \text{ kips} \)
Checking column reinforcing.

Column is checked for pure compression only.

From the column schedule, column 1-8 is 20\times20", and has 16 #10 and a $f'_c = 6000$ psi. At the basement level,

\[
\begin{align*}
A_s &= 16 \times 1.27 = 20.32 \text{ in}^2 \\
A_c &= (20 \times 20) - 20.32 = 329.68 \text{ in}^2 \\
\phi b &= \phi \left( \frac{0.85 f'_c A_c + A_s f_y}{3.05} \right) \\
&= 0.65 \left[ \frac{0.85 (6000) (349.68) + (20.32)(0)}{3.05} \right] \\
\phi b &= 2.051 \text{ in} \\
\end{align*}
\]

Must include $d$ for min eccentricity

\[
\phi P_m = \alpha \phi b = 0.8 (2.051) = 1.641 \text{ in} \\
\phi P_m > P_u = 1.357 \text{ in} \Rightarrow \text{OK!}
\]

Check $f$

\[
f = \frac{A_s}{b_1 d} = \frac{20.32}{20 \times 20} = 0.0570.01 \leq 0.011$

According to ACS 318-08 section 10.9.1

For minimum allowable $f$. 
## 16" JOIST WITH 3" COMPOSITE SLAB (P.S.F.)

<table>
<thead>
<tr>
<th>Joist Spacing</th>
<th>26</th>
<th>28</th>
<th>30</th>
<th>32</th>
<th>34</th>
<th>36</th>
<th>38</th>
<th>40</th>
</tr>
</thead>
<tbody>
<tr>
<td>3'-61/4&quot;</td>
<td>282</td>
<td>253</td>
<td>222</td>
<td>196</td>
<td>172</td>
<td>150</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4'-61/4&quot;</td>
<td>212</td>
<td>190</td>
<td>170</td>
<td>150</td>
<td>132</td>
<td>114</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5'-61/4&quot;</td>
<td>200</td>
<td>184</td>
<td>168</td>
<td>150</td>
<td>132</td>
<td>115</td>
<td>101</td>
<td>87</td>
</tr>
<tr>
<td>6'-61/4&quot;</td>
<td>166</td>
<td>152</td>
<td>138</td>
<td>122</td>
<td>107</td>
<td>93</td>
<td>80</td>
<td>68</td>
</tr>
</tbody>
</table>

### TYPICAL BEAM SECTION

- First Layer Top Bars
- Second Layer Top Bars
- Stirrups Cast in Soffit Beam
- #5 @ 12" Side Bars when depth is greater than or equal to 24" ung.
- Precast Joist Ribs, Typical Each Side
- Precast Beam Soffit
- Per Precast Manufacturer

- W/2
- W

---

September, 23rd 2011             J.B. Byrd Alzheimer’s Center & Research Institute | Tampa, FL
All columns at 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 present table, for a span of 32' @ a spacing of 3' 6" to be uniform (41' = 6)
a 16" deep joist of #3 would have a capacity 253 psf thus for a span of 31'
a #3 @ 3' 6" would suffice
Project: Tech 1 Report
Subject: Gravity check
Task: Gravity check

Beam B1 @ level 5

Joist schedule: Assume deck is 4 3/4"
depth
J2 12" @ 6'-6" w = 67 psf (values taken from manufacturer)
J3 16" @ 3'-6" w = 84 psf

5" NW slab (5/12) (150) = 62.5 psf
SP 14 psf lighting, plumbing, wiring, HVAC
20 psf pedestrian load

Assume beam is continuous as there are moment frames.
short direction: \((5\,\text{ft})(17) = 0.374 \text{ kIf}\)

long direction: \((15\,\text{ft})(84) = 1.295 \text{ kIf}\)

S lab + Sp: \((62.5 + 14 + 20)(15\,\text{ft})(5\,\text{ft}) = 0.027 \text{ kF}\)

total line: \(125 \mu\text{F}\)

Reduced by 80% see provisions

\[ 100 \mu\text{F} \times (81) = 2.1 \text{ kIf}\]

\[ W_0 = 1.2 D + 1.6 L = 1.2(0.027 + 1.275 + 0.374) + 1.1(81) \]

\[ W_0 = 7.795 \text{ kIf} \]

\[ M_u = 7.795 \text{ kIf} \left(81 - \frac{80}{12}\right)^2 = 182.1 \text{ k-Ft} \]

Positive moments for interior spans:

\[ M_u = 7.795 \text{ kIf} \left(81 - \frac{80}{12}\right)^2 = 264 \text{ k-Ft} \]

On both sides using coefficient in continuous beams for negative moment @ other face of interior supports

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

\[ M_u = 205 \quad W_u \text{ k-Ft} = 110 \]

\[ W_{right} = 105 \]

4 \# 5 Full Length

3 \# 5 @ Right End
Note: the bottom prestressed beam still has reinforcement but its sizes are not mentioned in the drawings. However, they are typical prestressed strands (same as the general notes in the report).

\[ V_u = \frac{V}{2} = \frac{475 \times (0.1 - 0.12)}{12} \]

\[ = 75.35 \text{ kips} + 10\% \]

The 10% added as a safety factor in the engineering since the loads were provided to the manufacturer. For more details, see beam check in (report).
Typical slab reinforcing detail.

Check slab thickness

Minimum slab thickness (Table 3-5(a))

Exterior Bay: $\frac{5}{24}$ for length of 39'4" (maximum)

$\Rightarrow \frac{39'4''}{24} = 1.64''$

For interior Bay: $\frac{1}{28}$ = 0.75''

Design uses a min of 5'' > 1.64'' = OK

Check reinforcement for max moment

Find WU = $W_D = \left( \frac{5}{12} \times 150 \right) + 14 \text{ psf} + 20 \text{ psf}$

$W_D = 96.5 \text{ psf}$

For L/H/M/C: $W = 50 \text{ psf}$ (office since it's less than the typical use) cannot be reduced

$W_U = 1.20 + 1.0 W = 195.8 \text{ psf} = 196 \text{ psf}$
Since \( w < 3 \) we then we can use ACI moment coefficient.

**First interior:**

\[
M_u = \frac{wL}{L^2} = \frac{196 \times (18.5 - \frac{20}{12})^2}{10} = 6,283 \text{ lb-ft/ft}
\]

**Second interior:**

\[
M_u = \frac{wL}{L^2} = \frac{102 \times (21 - \frac{20}{12})^2}{11} = 1,660 \text{ lb-ft/ft}
\]

We select 6 in. A4320 bars in the second column,

From the one-way slab schedule at 5′ or 5′ 5″ slab,

that is typical would have #4 @ 10′ for the #21 or #22.

Bottom:

\[
6.35 \times 12 = 76.2 \Rightarrow 28
\]

Top bars: since high and left end can be combined

\[
23 \times \#4 \Rightarrow 23 \times 8 \times 0.2 = 8.2 \text{ in}^2
\]
As (per sq in) = \frac{3.2}{18.33'} = 0.176 \text{ in}^2/\text{ft}

\[ H_a = 6 \text{ in} \]

Assume \( f_s > f_y \)

\[ a = \frac{As \times f_s}{0.85 \times f_y} = \frac{0.476 \times (60,000)}{0.85 \times (4000) \times (12)} = 0.7'' \]

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

\[ \varepsilon = \frac{d}{0.85} = 0.7 = 0.824'' \]

Check: \( E_s > E_y \):

\[ E_s = \frac{E_{cu} \times (d - c)}{0.85} = \frac{0.0533 (4 - 0.824)}{0.85} \]

\[ E_s = 0.0195 >> E_y \Rightarrow \phi = 0.9 \]

\[ \phi M_w = 0.8 \left( \frac{As_f y (d - \frac{c}{2})}{A_c} \right) = 0.8 \left( \frac{0.476 \times (60,000) \times (4 - 0.2)}{2} \right) \]

\[ \phi M_w = 33,820 \text{ lb-ft} = 7,818 \text{ lb-ft} \]

\[ \phi M_m > M_u = 6,660 \text{ lb-ft} \Rightarrow \text{OK!} \]

Check for shear:

\[ V_u = 1.5 \times \frac{W_u \times h_m}{2} (ACI\ & \phi = 0.83) \]

\[ V_u = 1.5 \times \frac{12 \times \cos (81 - 20)}{2} = 18,199 \text{ lb/sq in} \]

\[ \psi V_c = 0.75 \left( \frac{2 \times \\sqrt{f_y}}{h_m} \right)^2 \]

\[ = 0.75 \left( \frac{2 \times 14000 \times (12)}{12} \right) = 4,554 \text{ lb/sq in} > V_u \Rightarrow \text{OK!} \]
Appendix B: Wind Load Calculations

<table>
<thead>
<tr>
<th>General Wind Load Design Criteria</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Wind Speed</td>
<td>120mph</td>
</tr>
<tr>
<td>Directionality Factor (Kd)</td>
<td>0.85</td>
</tr>
<tr>
<td>Importance Factor (Iw)</td>
<td>1</td>
</tr>
<tr>
<td>Exposure Category</td>
<td>B</td>
</tr>
<tr>
<td>Internal pressure coefficient</td>
<td>0.55</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Velocity Pressure, qz</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Level</td>
<td>Height</td>
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<td>1</td>
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</tr>
<tr>
<td>2</td>
<td>14'-6&quot;</td>
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<tr>
<td>3</td>
<td>29'</td>
</tr>
<tr>
<td>4</td>
<td>43'-6&quot;</td>
</tr>
<tr>
<td>5</td>
<td>58'</td>
</tr>
<tr>
<td>6</td>
<td>72'-6&quot;</td>
</tr>
<tr>
<td>7</td>
<td>87'</td>
</tr>
<tr>
<td>Roof</td>
<td>107'</td>
</tr>
</tbody>
</table>

Wind calculations start on next page.
Wind Analysis:

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

Figure

\[ 145' - 8" \]
\[ 191' - 1" \]
\[ 67' - 3" \]
\[ 98' - 5" \]

N-S Direction

L = 191'
B = 146'  
Rounding for simplicity

E-W Direction

L = 146'
B = 191'

There are two roof heights, the upper roof height will be taken as the only roof height for simplicity. Thus, 107' wind loads will be established based upon ASCE 7-05. Use method 2 since building meets criterion of 6.5.1 & 6.5.2.
This building is partially enclosed, and is of low rise.

- Basic wind speed using figure 6.12, $V = 120$ mph
- Site exposure is $B$, with wind importance factor $I = 1.0$
- Since building category is office $= II$
- Wind directionality factor $V_d = 0.75$ using $x=6.11$

Velocity pressure $q_v$ shall be determined by the following equation:

$$q_v = 0.00256 \times \frac{V_e}{K_v} \times V_d \times V^2 \times I$$

See Excel spread sheets.

- Gust effect factor:
  
  Rigidity of structure

$$f_a = 0.65$$

$$T_a = 1.5 \times \frac{h}{T_a} = 1.5 \times \frac{1}{0.75} = 2.0$$

Design wind pressure for MUERS is determined by

$$p = q \times GCP - q_i \times (GCP_i)$$

$$q_i = 37.14 \text{ psf}, \quad GCP_i = 0.55, \quad G = 0.5$$

see table.
N-S direction:
\[
\frac{h}{L} = \frac{109}{191} = 0.56
\]
Windward: \( C_p = 0.8 \)
Leeward: \( C_p = -0.438 \) by interpolation
Side wall: \( C_p = -0.7 \)

E-W direction:
\[
\frac{h}{L} = \frac{0.7}{4} = 0.175
\]
Windward: \( C_p = 0.8 \)
Leeward: \( C_p = -0.5 \)
Side wall: \( C_p = -0.7 \)

For \( \theta = 0^\circ \):
\[
\frac{h}{L} = \frac{109}{191} = 0.56
\]
\[
\frac{h}{L} = \frac{107}{146} = 0.73
\]
between 0.5 and 1.0 \( C_p \) is interpolated between 0.5 and 1.0
\[
0 - \frac{h}{L} \Rightarrow C_p = 0.948
\]
\[
\frac{h}{L} \leq 0.5\Rightarrow C_p = 0.838
\]
\[
h + 2h = C_p = 0.524
\]
\[
h + 2h = C_p = 0.532
\]
Appendix C: Seismic Load Calculations

Calculating weight of building

Roof level:
Framing: Joists J2 (12” deep) @ 5’-6” w = 63 psf
J4 (20’ deep) @ 5’-6” w = 83 psf

Area:
6’ x 3’ = 18’
8’ x 3’ = 24’

Total:
1183.3

\[
\frac{6^2 + \left(\frac{5}{12} \times 150\right)}{2} \times (11’-3”) (16’-5”) = 246.315 \times \frac{2}{3}
\]
\[
\frac{8^2 + \left(\frac{5}{12} \times 150\right)}{2} \times (30’-3”) (16’-5”) = 936.937 \times \frac{2}{3}
\]

level 7:

For simplicity, the building is divided into 3 rectangular forms

1
\[
A_1 = \frac{27.41 + 1.25^2}{2} = 63.75’
\]

5.5
\[
A_2 = 5.5 \times 13.4’ = 73.75’
\]

3
\[
A_3 = 3 \times 13.4’ = 30.75’
\]

Total:

1183.3
Notes:

(7-A)

Level building port

in 7-A 5” slab, \( \frac{54}{\pi} \) @ 5’-8” w = 74 psf
\[
(27.5 + 84 \text{ psf}) (2741.2) = 401,586 \text{ kips}
\]

7-B 6” slab, J3 @ 3’-6” w = 74 psf
\[
(47.5 + 76 \text{ psf}) (5,619.4) = 848,523 \text{ kips}
\]

7-C 6” slab, J3 @ 3’-6” w = 84 psf
\[
(47.5 + 84) (5,119.5) = 814,064 \text{ kips}
\]

\( \text{E} (11.3) (4.4) (14 - 2') \times 150 = 8,193 \text{ kips} \)

Total 7th = 2,072,872 kips

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

Also, beam size are 20” x 30” or 24” x 24” or 16” x 30”

Thus, an assumption of average size for simplification of weight calculation, the typical size 24” x 24” will be taken.

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

Also, assume that where joists are closer together compensate for the voids that exist in the framing.
level 6:  
6-A 6" slab, 34 @ 4'-8" w = 30 psf  
\[(75 + 90)(2741.2)\] = 4,522.38 k  
6-B 5" slab, 34 @ 5'-6" w = 67 psf  
\[(62.5 + 48)(2741.2)\] = 663.027 k  
Total: \[1,865.39\] k  
6-C 5" slab, 32 @ 2'-6" w = 84 psf  
\[(62.5 + 84)(2741.2)\] = 750.065 k  
level 5:  
5-A 5" slab, 34 @ 5'-8" w = 84 psf  
\[(62.5 + 84)(2741.2)\] = 4101.58 k  
Total: \[1,814.68\] k  
5-C 5" slab, 33 @ 3'-6" w = 750.065 k  
level 4: same as 5  
Total: \[1,814.68\] k  
level 3: same as 5  
Total: \[1,814.68\] k  
level 2:  
2-A same as 5-A = 1401.586 k  
2-B same as 5-B = 663.027 k  
Total: \[1,874.24\] k  
2-C \(\frac{1}{2}\) is 55 @ 4'-8" w = 95 psf  
\(\frac{1}{2}\) is 53 @ 5'-6" w = 74 psf  
This \(\frac{1}{2}\) and \(\frac{1}{2}\) assumption can be taken since where 55 exist @ closer spacing 54 can compensate for the weight.
2-C - Beam 5" slab

\[
\frac{62.5 \times 519.9}{10} = 319.934
\]

\[
\frac{95 \times (519.9)}{2} = 243.195\text{k}
\]

\[
\frac{94 \times (519.9)}{2} = 189.436\text{k}
\]

Beam weight of 24x24

\[
\omega_{\text{min}} = \frac{44 \times 24 \times 150}{144} = 600\text{ plf}
\]

For each floor there is \( \frac{27}{2} \) beams @ 10'

Columns
All columns are 20x20 and 14' - 6" in height
except for the roof
The weight of each column is \( \frac{1}{2} \) divided by \( \frac{1}{2} \) to each floor

\[
559.7 \text{ k}
\]

\[
\approx 558\text{ k}
\]
Thus \( \frac{30 \times 30}{144} \times 14\frac{6}{12} \times 150 = 6.041 \) k for each column

There are 43 columns for each level except for roof level: \( 43 \times 6.041 = 259.732 \) k

Roof: \( \frac{1}{2} \times \frac{30 \times 30}{144} \times 30 \times 150 = 8.33 \) k

7: \( \frac{1}{2} \times 259.732 \) k

6: \( \frac{1}{2} \times 259.732 \) k

5: \( \frac{1}{2} \times 259.732 \) k

4: \( \frac{1}{2} \times 259.732 \) k

3: \( \frac{1}{2} \times 259.732 \) k

2: \( \frac{1}{2} \times 259.732 \) k

Thus total from each floor: \( 1,453.82 \) k

Total for beam from each floor:

\[
(558 \times 6) + \left( \frac{24 @ 81^\circ, 2 @ 11^\circ, 2 @ 30^\circ}{} \right)
\]

Total = 3,700.62 k
For shear wall:

\[ A = 6.53 \]

\[ A = 6.6 \]

\[ A = 6.5 \]

\[ A = 6.5 \]

Adding the Areas of all shear walls:

\[ 58.1 + 30.1 + 31.5 = 119.7 \]

\[ 119.7 \times 150 = 17,955 \text{ ft}^2 \]

\[ 5.7 \times 150 = 855 \text{ ft}^2 \]

\[ h = 14.5 \]

\[ \frac{260.348}{14.5} = 18 \]

\[ 3, 4, 5, 6, 7 \]

Total: 1,562.0 ft²

Around Elevator:

\[ A = 20.63 \]

\[ A = 20.63 \]

\[ A = 16 \]

Around stairs on left side:

\[ A = 10.95 \]

\[ A = 10.95 \]

\[ 4-75 \]

\[ 35.75^\circ \]

Around stairs on right side:
Total weight of building:

levels: 1,183.3 + 2,072.3 + 1,865.3 + (1,814.68 x 3)
+ 1,817.24 =

Columns: 1,553.82 k
Beams: 3,700.22 k

D total = 13,300 kips

Design Spectral Response

Figure 22.1 and 22.2 of the ASCE 7-05 show S5 and S1, respectively. The following are for J.B. Byrd, Tampa, FL:

S5 = 4.8% = 0.078
S1 = 3.2% = 0.032

Assume site class D for Tampa, Florida

= D a stiff soil since it has limestone

For site class E and S5 < 0.25, Fa = 1.6

Sms = Fa. S5 = 1.6 x 0.078 = 0.125

For site class E and S1 < 0.11 then Fv = 2.4

Smi = Fv. S1 = 2.4 x 0.032 = 0.0768

Sd5 = 3/5 Sms = 0.0832
Sd1 = 2/3 Smi = 0.0512
For the computation of each please see the seismic table.
Appendix D: Typical Plans

Figure 22 - Typical floor plan taken from S-104
Figure 23 - Live Load diagram from S-002 (live load used in calculations)
Figure 24: Live load diagram from S-003 (live load used in calculations)
Figure 25 - Elevation of the building showing the different floor heights from A -201- 0