3. Structural Depth

3.1. Existing Structural System

3.1.1 – Foundation System

The site for 101 Eola Drive is located in the Central Floridian Aquifer region. The soil in this area can generally be described in terms of three sedimentary layers. The superficial layer is composed of fine sands. This is underlain with a layer of clay, clayey sand, phosphate, and limestone which are locally referred to as the “Hawthorn Group.” This layer is underlain with limestone bedrock. The results of ten test borings, drilled 50 feet to 100 feet in depth, indicated the subsurface soils would be suitable to support shallow spread footings after subsoils had been improved with vibro-replacement stone columns, a process where a probe is advanced into the soil and gradually removed while backfilling the space in lifts with crushed stone, then vibrated to densify the soil/stone matrix.

Footings are placed on top of groups of the stone columns, which are grouped and positioned according to the size of the footing they support. Footings are either a spread design (shear walls, core, and columns) or a continuous strip footing (east wall and retaining wall). Depths range from 12” to 39”. Footings are designed with an allowable load of 8000psf and f’c = 4000psi.

A slab on grade is used as the ground floor level with a minimum thickness of 5”, typical, and a 8” slab on grade in the loading dock area, with expansion joints no more than 15 feet apart. Material strength is 4000psi.
3.1.2 – Typical Floor Plans and Framing

There are several variations in the floor framing of 101 Eola, depending on their location and usage characteristics. Most floor elements consist of precast double tee beam or hollowcore planks.

The ground floor consists of slab on grade construction and areas of exposed soil behind precast retaining walls. 6 inch equipment pads are placed beneath mechanical and electrical equipment on top of this slab. Additionally, ramping for the parking structure begins on this level with precast double tee beams supported by precast walls on either end.

Floors 2 thru 4 comprise an open air parking facility. Construction is precast double tee on precast walls are stepped and rotated to eliminate need for blocking in order to support sloped garage ramps. Average span is 62 feet E-W, but spans range from 20 feet to 68 feet in length depending on location. Floor 3 also adds precast flat slabs above the loading dock (used for storage) with span of around 23 feet. Floor 4 is typical of floor 2 with the exception of 12 foot wide precast single tee beams spanning 23 feet to support a pool and hot tub located on the 5th floor roof. These tees are also covered with a 6” structural concrete topping slab.

At floor 5, the entire building profile steps back an average 10 feet and the upper levels take shape. An accessible roof with pool and hot tub is located on the south side roof. Roof area construction is a mixture of precast flat slabs, single tee beams, and hollowcore planks with a C.I.P. topping sloped to roof drains. Interior floor makeup is 8” thick hollowcore slabs. These slabs span N-S with length 25’-5” to 26”-7”, and are supported by unique precast trusses 45’-0” in length that attach to the lateral shear wall system to transfer shear loads (refer to figure 3.1.2.1). These truss beams have a height of 12’-0”, and are placed on every other floor (levels 5,7,9,11) in what is called a skip truss system. E-W edge planks are supported on the exterior wall by precast beams 2’-8” in width. Precast flat slabs form balconies on the exterior edges of the structure, spanning between the shear walls that run up the building’s exterior.

Shear walls around the exterior of the building act as the main gravity load carrying system as well as the lateral system. The load is carried from the floor system (double tee beams or hollowcore/flat slabs) to the precast trusses and edge beams, and then into the shear wall system or the central core. In addition to these elements, there are miscellaneous columns in key places in the structure. One 24”x24” column in each corner of the west elevation runs vertically until reaching the 5th floor step-back in order to carry the corner load. Corner loads on the opposite side of the building is carried by precast walls that run monolithically up to the 5th story as part of the system that carries the pool and hot tub loads. 2 columns of the same size assist in carrying load surrounding a large garage door on this elevation also. The two main supporting columns in the structure are at either end of the central core strip. Each precast column is 36”x48”

Please refer to figures 3.1.2.2 thru 3.1.2.7 on the following pages for typical floor plans and building sections illustrating the system layout.
Material Properties:
\[ f'c = 4000 \text{psi} \]
\[ f'c = 6000 \text{psi} \]
\[ f_y = 60,000 \text{psi} \]
\[ f_{pre} = 270,000 \text{psi} \]
\[ \frac{1}{2}" \text{Ø strand} \]
\[ 8" \text{ Thick, 145psf} \]
Figure 3.1.2.6 – East / West Building Section

Figure 3.1.2.7 – North / South Building Section
3.1.3 – Lateral Load Resisting System

Lateral system elements are highlighted in figure 3.1.3.1 below. The system consists of precast concrete shear walls around the exterior of the building, and a cast in place concrete core. These elements are tied together with precast truss (inverted “T” shape). The plan below is on one of the upper floors. Floors G-4 have much larger shear walls.

After calculations and analysis it was determined that 101 Eola’s lateral force resisting system is controlled by wind forces in the short (E-W) direction (long side perpendicular to wind), and by seismic forces in the long (N-S) direction.

Figure 3.1.3.1 – Lateral Force Resisting System
3.1.4 - Codes and Referenced Standards

Structural Concrete: American Concrete Institute 2002 edition (ACI 318-02)
                    Prestressed Concrete Institute (PCI MNL 116,120,129)
Mechanical Code: Florida Mechanical Code (2004 w/ 2005 revisions)
Electrical Code: NFPA 70, National Electrical Code
           Florida Statutes and Administrative Code
           Orlando City Code
Building Design Loads: American Society of Civil Engineers (ASCE-7) 2002 edition
Materials Standards: American Society for Testing and Materials (ASTM)
Testing Methods: American Society for Testing and Materials (ASTM)

3.1.5 – Material Strength Requirements

Cast in place concrete (normal weight 145pcf)
Footings................................. 4,000 psi
Column Piers.............................. 5,000 psi
Grade Beams.............................. 4,000 psi
Columns.................................. 5,000 psi
Walls..................................... 4,000 psi
Slab on grade (interior)..................... 4,000 psi
Stairs, landings, lobbies................... 4,000 psi
Tee pour strips........................... 5,000 psi
All other................................ 4,000 psi

Precast Concrete (normal weight 145pcf)
Shear Walls, Trusses, Beams............... 7,000 psi
All Other Types.......................... 6,000 psi

Other Concrete
Columns dry base pack........................ 6,000 psi
N.S.N.S. grout................................ 6,000 psi

Reinforcing and Connection Steel
Welded bars............................... ASTM A706 .................. 60,000 psi
All bars u.n.o............................. ASTM A615 .................. 60,000 psi
Welded Wire Fabric (smooth).............. ASTM A185 .................. 60,000 psi
Prestressing strand....................... ASTM A416............... $(f_{pu})$ 270,000 psi
Coil bolts and coil rods u.n.o............. 65,000 psi
Deformed bar anchors..................... ASTM A496 ............ 70,000 psi
Headed anchor studs....................... ASTM A108 ............ 50,000 psi

Structural Steel
Structural W Shapes........................ ASTM A992 ............ 50,000 psi
Structural HSS (shaped)................... ASTM A500 Gr B ........ 46,000 psi
3.1.6 – Load Cases and Combinations

The following load combinations were considered. Snow loads were not included in this analysis due to building location (Orlando, Florida).

1. 1.4(D)
2. 1.2(D) + 1.6(L) + 0.5(Lr)
3. 1.2D + 1.6(Lr) + (L or 0.8W)
4. 1.2D + 1.6W + L + 0.5(Lr)
5. 1.2D + 1.0E + L
6. 0.9D + 1.6W
7. 0.9D + 1.0E

3.1.7 – Dead Loads

Material weights used to determine the dead loads are taken from the LRFD Manual, ASCE 7-02, and accepted practice values, or were specified by the designers.

<table>
<thead>
<tr>
<th>Supported parking and drive areas</th>
<th>40 psf</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concentrated wheel load (on 20 sq. inches)</td>
<td>3,000 lb.</td>
</tr>
<tr>
<td>Bumper impact load, over 1 foot square, located 18” above finished floor, ultimate...</td>
<td>10,000 lb.</td>
</tr>
<tr>
<td>Superimposed dead loads</td>
<td></td>
</tr>
<tr>
<td>Condominiums</td>
<td></td>
</tr>
<tr>
<td>Partitions</td>
<td>10 psf</td>
</tr>
<tr>
<td>Mechanical, Electrical, Misc</td>
<td>10 psf</td>
</tr>
<tr>
<td>Roof</td>
<td></td>
</tr>
<tr>
<td>Superimposed</td>
<td>20 psf</td>
</tr>
<tr>
<td>Mechanical, Electrical, Misc</td>
<td>20 psf</td>
</tr>
<tr>
<td>Slabs on grade</td>
<td>50 psf</td>
</tr>
<tr>
<td>Stairs, landings, lobbies</td>
<td>100 psf</td>
</tr>
</tbody>
</table>
3.1.8 – Live Loads

Live Loads are determined in accordance with Florida Building Code, ASCE 7-02 or designer’s specification.

Superimposed live loads
Condominiums
Condominiums................................................................. 40 psf
Corridors................................................................. 80 psf
Stairs, lobbies, balconies.................................................. 100 psf
Roof........................................................................... 20 psf

3.1.9 – Wind Load Criteria

Major assumptions used in the determination of 101 Eola’s wind load are listed below. Wind loads were calculated in accordance with ASCE 7-02. 101 Eola is located in downtown Orlando, Florida, which is a hurricane prone region.

Basic Wind Speed.......................................................... 110 mph
Exposure Category......................................................... B
Enclosure Category....................................................... Enclosed
Occupancy Category....................................................... III
Wind Directionality Factor (Kd)................................. 0.85
Importance Factor (I)..................................................... 1.15
Topographic Factor (Kz)............................................. 1.0
Gust Effect Factor (G).................................................. 0.85
Internal Pressure Coefficient
  Parking Garage......................................................... 0
  Enclosed rooms and elevator......................... ±0.18

3.1.10 – Seismic Load Criteria

Major assumptions used in the determination of 101 Eola’s seismic response load are listed below. Seismic loads were calculated using the equivalent lateral force method in accordance with ASCE 7-02. Soil information came from the geotechnical report for the site, performed by ESC-Florida, LLC, dated February 2006. Because of the site location, and based on calculations, seismic loads will control only in the short direction.

Seismic Use Group......................................................... II
Occupancy Importance Factor (IE)................. 1.25
Site Class................................................................. D
Soil Profile................................................................. Stiff Soil Profile
Mapped Spectral Response Accelerations
  \( S_S = 0.10 \)
  \( S_I = 0.04 \)
Site Class Factors
  \( F_A = 1.6 \)
  \( F_V = 2.4 \)
<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_{MS}$</td>
<td>0.160g</td>
</tr>
<tr>
<td>$S_{SM}$</td>
<td>0.096g</td>
</tr>
<tr>
<td>$S_{DS}$</td>
<td>0.107g</td>
</tr>
<tr>
<td>$S_{SD}$</td>
<td>0.064g</td>
</tr>
<tr>
<td>Seismic Design Category</td>
<td>A</td>
</tr>
<tr>
<td>Building Frame</td>
<td>“Ordinary Reinforced Concrete Shear Walls”</td>
</tr>
<tr>
<td>Response Modification Factor ($R$)</td>
<td>5</td>
</tr>
<tr>
<td>Over-Strength Factor ($W_0$)</td>
<td>2.5</td>
</tr>
<tr>
<td>Deflection Amplification Factor ($C_D$)</td>
<td>4.5</td>
</tr>
<tr>
<td>Period Coefficient ($x$)</td>
<td>0.75</td>
</tr>
<tr>
<td>Seismic Response Coefficient ($C_S$)</td>
<td>0.01</td>
</tr>
<tr>
<td>Period Exponent ($k$)</td>
<td>1.135</td>
</tr>
</tbody>
</table>
3.2 – Rationale for Redesign

3.2.1 – From Concrete to Steel

When 101 Eola was originally designed in 2005, precast concrete was chosen as the system of choice due to the high quality of the appearance, and the cost savings the system presented over cast-in-place concrete. Also, concrete is the predominant structural system in the Orlando area, and is a familiar material. This is especially true for specialized companies such as the structural engineers, Finfrock Industries. It was for these and other reasons that a precast concrete structural system was selected for the building.

Although this system saved a large sum of money over a cast-in-place system, it can be improved. The concrete skip truss system that is responsible for carrying two floors worth of precast floor planks and superimposed condominium loading over a 45'-o" span (refer to photo at right). Even for a pre-tensioned member, this is a very long distance to span while carrying such a high loading. In fact, the trusses were designed especially for this project. Never before has such a system been used in the precast industry. However, since column-free tenant spaces are important to the marketability of the building, it would not have been acceptable to shorten the span.

A steel truss could be more effective at carrying the load over the span required by the project. Steel is extremely good in tension as well as compression, whereas concrete is excellent in compression, but requires large amounts of steel embeds to develop tension control. The concrete trusses that are the object of this thesis have (6) ½" diameter pre-stressing strands in both the top and bottom, in addition to #11 bars running longitudinally in both the top and bottom, and shear stirrups as close together as 5" (refer to piece ticket in appendix). They also weigh over 54,000lb each. The amount of labor necessary to prepare such a piece is quite a lot. With 48 of these trusses in the building, material and labor costs can add up quickly. Steel on the other hand requires no additions to allow it to reach such spans, and will result in a smaller, lighter, cheaper solution than using precast concrete. Therefore the truss will be redesigned using a steel solution and analyzed to determine if it is a viable solution. The design process is shown in section 3.3.1.

3.2.2 – Impact of Redesign

It is expected that changing a portion of the structure will affect other portions of the building. Besides potentially being cheaper, skinnier, and more forgiving of penetration needs, a steel truss will certainly be a much lighter system. This will inevitably affect the foundation design, so this impact will be determined. Refer to section 3.4.1 for this data.
3.3 – Proposed Changes to Structural System

3.3.1 – Skip Truss System: From Concrete to Steel

3.3.1.1 – Introduction

A skip truss system is not something you would normally find in the design of most buildings. This system is very similar to what is called a staggered truss system, in which trusses are “staggered” in the elevation of the building. The trusses in this system are normally placed at mid-span of the floor diaphragm resting in the trusses below. In a skip truss system however, the trusses line up above one another, but skip every other floor. Therefore each truss is responsible for carrying two floors worth of loading; one floor resting on the bottom chord, one resting on the top, allowing every other floor to be column and wall free if desired. Refer to figures 3.3.1.1.1 and 3.3.1.1.2 for clarification between these two systems.

3.3.1.2 – Design Process

The following process was used in the redesign of the precast concrete skip truss system:

1. Determine factored loading due to floor system and superimposed loads for typical bay. Use of LRFD process is used.
2. Use AISC Design Guide #14 (Staggered Truss Framing Systems) for the preliminary design of trusses. Use of W-shape chords and HSS web members chosen. 2 designs considered.
3. Analyze truss designs using hand calculations and method of joint equilibrium to determine forces in members.
4. Estimate member sizes based on factored axial forces.
5. Recheck designs using self-weight addition.
6. Resize members as necessary.
7. Enter designs into STAAD Pro. Check designs for member adequacy.
8. Check deflections of truss assemblies.
9. Redesign members as necessary.
10. Enter designs into SAP2000. Recheck designs against STAAD and hand calculation results.
11. Redesign members for adequacy, serviceability, and economy.
12. Design connection options based on AISC Design Specifications for HSS and AISC Design Guide #14
13. Consult with steel fabricators on connection design and truss design. Choose connection and truss design to further the construction management breadth’s success (cost and feasibility based).
14. Account for 1” spray applied fire resistive materials (SFRM) to achieve 2hr fire rating.
15. Finalize Design.

3.3.1.3 – Material Selection

Based on suggestions and examples in AISC Design Guide #14 chapter 3 (staggered truss framing systems), a box type truss was chosen for design. A typical box truss is shown below in figure 3.3.1.2.1 for reference.

![Figure 3.3.1.3.1 – Box Truss](image)

AISC Design Guide #14 states that most common trusses today are designed with W-shape chords and HSS web members connected with gusset plates. To allow the precast planks to have adequate bearing, a minimum width of 6” is required, with the smallest typical chords being W10x33. Figure 3.3.1.3.2, located below, is a material guide for the steel members selected for this design.

<table>
<thead>
<tr>
<th>Column and Truss Chords</th>
<th>Section</th>
<th>ASTM</th>
<th>Fy (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>W-Shape Chords</td>
<td>Web Flange</td>
<td>A992</td>
<td>50</td>
</tr>
<tr>
<td>Web Members (Vertical and Diagonal)</td>
<td>Hollow Structural Section</td>
<td>A500 grade B</td>
<td>46 (not rectangular)</td>
</tr>
<tr>
<td>Gusset Plates</td>
<td>Plates</td>
<td>A36</td>
<td>36</td>
</tr>
</tbody>
</table>

![Figure 3.3.1.3.2 – Material Guide](image)
3.3.1.4 – Truss Design Options

It was decided early on that a box truss with single diagonal braces would be used for the redesign. Factors considered in the design included panel width, brace direction, and economy considerations. Two designs were chosen for further analysis. They are shown below in figures 3.3.1.4.1 and 3.3.1.4.2.

![Figure 3.3.1.4.1 - 7 Panel Design](image1)

 ![Figure 3.3.1.4.2 - 5 Panel Design](image2)

It was determined though analysis and an economy study that the 5 panel solution would be the best solution for the project. The central panel's brace direction is determined by the direction the wind force is applied. These models are for trusses on the left (looking from the thinner end of the building). Trusses on the opposite side of the building are identical but reversed. Axial forces as a result of analysis are shown in figure 3.3.1.4.3.

After many design iterations, the final truss design sections are shown in figure 3.3.1.4.4. Deflection is calculated to be 0.3962 inches which is well within the code restriction of L/240, which equates to 2.25 inches.

![Figure 3.3.1.4.3 - Truss Force Diagram](image3)
3.3.1.5 – Connection Design Options

Two main connection options exist when considering typical truss construction using HSS members. The first involves slotting the HSS and fitting it over a single ½” gusset plate, welded to the center of the chord member and to the HSS on both sides with a ¼” fillet weld. This option is shown below in figure 3.3.1.5.1. The second option is to cut the HSS at an angle (for diagonal members), and weld all around directly to the top of the chord’s flange. This option is shown below in figure 3.3.1.5.2.

After consulting with steel fabricators, the gusset plate connection was chosen. The connection is much easier and cost effective to fabricate than the direct flange connection. The direct connection also presents a problem when the precast plank is considered, since the flange must be wider than in the first option to achieve proper bearing width. The HSS simply takes up too much space when welded directly to the chord member. Final design is a ½” A36 steel gusset plate with ¼” fillet welds applied to both sides. Weld lengths vary by HSS size, but range from 4” to 8”.

3.3.1.6 – Final Design
The final design of the trusses is shown below compared to the concrete trusses they replace. Trusses are assumed to be connected to the structure in the same fashion as their concrete counterparts, by welding at the bottom flange of the bottom chord. It is to be recognized that this connection in no way transfers any significant moment to the shear walls or core, and is conservatively assumed to be a pin-pin action. The trusses will be erected and fastened in the same way as the concrete trusses they replace, by sitting in the shear wall pockets designed for the concrete trusses. Final steel truss design is shown in figure 3.3.1.6.1 while the replaced concrete truss is shown in figure 3.3.1.6.2.

![Steel Truss Design](image1)

**Figure 3.3.1.6.1 – Steel Truss Design**

![Concrete Truss Design](image2)

**Figure 3.3.1.6.2 – Concrete Truss Design**
3.4 – Impacts of Redesign

3.4.1 – Foundation Impact

The effects of redesigning the concrete trusses reach far beyond structural performance alone. As illustrated in figure 3.4.1.1 below, the weight of the replacement steel trusses is significantly less than that of the concrete trusses. This makes sense since the steel is lightweight and the HSS are hollow, whereas the concrete truss, even with the open blockouts, is an extremely massive piece of structure. Weight savings per steel truss is close to 50,000lb over the concrete trusses they replace, while still providing the same structural function. Please see appendix for detailed calculations regarding the data below.

<table>
<thead>
<tr>
<th>Structural System</th>
<th>System Components</th>
<th>Weight of System</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Skip Truss</td>
<td>Concrete Truss</td>
<td>59.21 kips per truss</td>
</tr>
<tr>
<td>Steel Skip Truss</td>
<td>Steel Truss (w/ plates and welds)</td>
<td>8.55 kips per truss</td>
</tr>
<tr>
<td></td>
<td>Sprayed Fire-Resistive Materials</td>
<td>+ 1.14 kips per truss</td>
</tr>
<tr>
<td></td>
<td></td>
<td>9.69 kips per truss</td>
</tr>
</tbody>
</table>
|                         | **Savings of** 49.52 kips per truss **|**

Figure 3.4.1.1 – System Weight Comparison

When a force diagram is constructed to follow the path of forces down to the foundations, it is found that an average of 100 kips (for a typical bay) is diverted from each foundation beneath the shear walls. Because the forces on the foundations decreased, no change in the original design is necessary. It is not expected that this small of a decrease in gravity load would be enough to decrease the size of the footing so they will remain as designed.

3.4.2 – Other Impacts

3.4.2.1 – Fireproofing

Converting this truss system to steel also requires that a fireproofing solution be administered to the structure. The building code and project specifications require a 2 hr fire rating on structural components. No fireproofing was necessary when considering the concrete construction due to the physical properties of the material, which is inherently fireproof. Concrete achieves its fire rating though thickness rather than a covering.

Fireproofing solutions for steel include many different options. The most common are intumescent paint coating systems, and a gypsum-based cementitious spray-applied fire resistive material (SFRM). For this project, I have chosen to apply SFRM since it is the most economical method when applied to structures that will be unexposed after construction. One inch minimum thickness will be applied to all exposed steel surfaces to achieve the 2 hr required fire rating required by code.
3.4.2.2 – Wall Width

Converting this truss system to steel has decreased the overall wall width significantly. The original concrete truss has a width of 13” between floors. In addition, another 4 ½” of hat channel and gypsum wallboard are attached to provide the finished wall surface. The finalized steel truss has a maximum vertical/horizontal member width of 6”. This has effectively decreased the total width of the wall by 7” total, which equates to 3.5” of additional floor space on either side of the wall, assuming the hat channel and GWB construction remain to provide a smooth wall surface. Refer to figures 3.4.2.2.1 and 3.4.2.2.2 below for wall section details. To put things in perspective, an additional 3.5” on either side of the wall means if a typical interior condominium is chosen from the plan, with trusses on either side, the room will increase in size by roughly 26 square feet of floor space.

Figure 3.4.2.2.1 – Wall Section with Concrete Truss

Figure 3.4.2.2.2 – Wall Section with Steel Truss