SCHEMATIC STRUCTURAL DESIGN

FOR

Center for Centering (C4C)

Swanton Ranch, CA

FOR

Cynthia Campoy Brophy & Marcos Lutyens

SUBMITTED BY

Jonathan Oberemopt, Faith Sharp, Sam Buckman
# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project Description</td>
<td>3</td>
</tr>
<tr>
<td>Design Team</td>
<td>5</td>
</tr>
<tr>
<td>Chapter 1: A Walk Through the Center</td>
<td>W1</td>
</tr>
<tr>
<td>Chapter 2: Visualization of the C4C</td>
<td>V1</td>
</tr>
<tr>
<td>Chapter 3: Structural Process</td>
<td>S1</td>
</tr>
<tr>
<td>Chapter 4: Calculations</td>
<td>C1</td>
</tr>
<tr>
<td>Appendix</td>
<td>A1</td>
</tr>
</tbody>
</table>
Project Description

The Center for Centering (C4C) is a meditative center for individuals to reflect and discover their own creativity. Based on a spoke and wheel concept, the center integrates social, sensory, environmental, and inductive spaces, allowing for individuals to discover themselves in different settings, while appearing as a striking sculpture conceived in a spiral gesture integrated in the local landscape.

The location for this structure is in Santa Cruz, CA, on Swanton Ranch. The building is built on mostly flat terrain surrounded by sparse trees and an overlooking view of the ocean.

The initial concept for the Center for Centering is inspired by the naturally layered structure of a succulent. This project is intended to be a place where residents can self-reflect and discover their own creativity in the direct contact of nature.

The C4C structural dome houses four different experiential spaces; a social space within the dome, for connecting with others; surrounded by warm timber elements and shaded by the shadows of the woven dome, with a light breeze blowing past you and through the building. An inductive space, representing a place for solitude; for meditation; for quiet reflection. A shallow indent on the floor will be filled with small clay balls, providing a space for visitors to lay back and relax, gazing up to the open sky above that will be visible through a ceiling opening resembling a human shape, reflecting in spirit the person lying down. A sensory space; a space with a large gong-like bowl affixed to one wall, inviting visitors to touch it, to engage with the bowl to emit sound that will resonate in the space, in the visitors’ ears, as well as through their body. And lastly, an environmental space; incorporating movement by inviting visitors to wander through a floor-level labyrinth.
The main goal of the senior project design team was to provide two clients, Marcos Lutyens and Cynthia Campoy Brophy, with a practical schematic design that can be used to present the project to possible donors. By joining ARCH 551 for the quarter, the team was tasked to the “structures group”, whose goal would be to absorb architectural ideas and feedback from architects, professors, clients, and others, then assess the structural viability of them. Through the experience of the design team, the team was able to give the clients a more engineering-focused viewpoint.

The structure of the building is composed of discontinuous timber pieces that form the main diagrid form of the main dome. The members are pin-connected, with a compression ring at the tip and where the dome meets the walls of the sensory spaces. The bottom of the dome is supported by concrete columns that curve in-plane to allow the form of the dome structure to be fully realized to the foundation. The pathway is made up of a series of three-hinged arches, also made with discontinuous timber pieces. The three spaces at the Northern face of the dome are designed with concrete, to allow for the free-form shapes which the architecture demands.
DESIGN TEAM

Bryan Garcia    Jonathan Oberempt    Sam Buckman    Faith Sharp

Saksham Tikekar    Elijah Williams    Nishat Subah Peau    Nicole Svilich

Eric Jeschelnik    Deniz Karadere    Laetitia Herya Wallimann

Thomas Fowler & Kevin Dong ARCH 551 Studio 2020-21

Fourth-year ARCE students Jonathan Oberempt, Sam Buckman, and Faith Sharp joined in Spring Quarter 2021 for their senior project, advised by Kevin Dong.
CHAPTER 1

A WALK THROUGH THE CENTER

(Credit: Marcos Lutyens & ARCH 551 Students)
A WALK THROUGH THE CENTER (Credit: Marcos Lutyens & ARCH 551 Students)

“I approach the Center following a couple of signs that lead me down a pathway through the trees. It’s quiet enough to hear the flap of a bird’s wings in the distance. The building before me stands out but still feels like it belongs in the space.”

“I realize that in the middle of this undulating landscape there is an entryway made of basketlike material. It conjures up feelings of rootedness, as a kind of weaving together of feelings, ideas, purpose. As I walk down this passageway, the space opens up to a large woven space through which the light filters, as if I was in a palm grove.”

“At the end of the walkway there seems to be a central congregating space. The woven structure that I first encountered seems to emanate from this lower area. It seems that there is a sense of integration between the different parts of this building. It all seems woven together in its design and feeling.”
“I turn to my right and see a small space with curved walls. Intrigued, I walk inside, take off my shoes, and lay on the soft clay balls. The human-shaped void in the roof makes me feel close and intimate with the sky. I feel able to close my eyes and connect with myself in a way I haven’t experienced before.”

“I go into the second room, where I find a gong. The roof has disappeared, and I see only the clouds above. I feel inclined to use the drumstick nearby to hit the gong. Before I know it, the room fills with soothing vibrations and sounds. I let my body experience the fullness of the sound through all of my senses.”

I explore the third and last room - a labyrinth made of bark. I follow this winding pathway, until the outside wall completely tapers to the ground, releasing me into the landscape. I feel completely immersed and disconnected from the hustle and bustle of modern life.
CHAPTER 2

VISUALIZATION OF THE C4C
The Center for Centering is located in Swanton Ranch, in the Santa Cruz area. The surrounding trees and foliage are sparse due to a recent forest fire in 2020. However, the location still provides beautiful views of the ocean and a deep feeling of being separated from urbanization.
VISUALIZATION OF THE C4C

The rendering shown provides a visualization of the structure beneath the architectural weave. The dome structure incorporates discontinuous timber members held together by an axial bolted connection, which are carried to the ground through columns. The pathway includes a variation of the classic “three-hinged arch” that provides stability for the low lateral forces that may be developed.

The final plan was developed after many weeks of working with the design. Originally much larger in scale, the project was scaled down to reduce costs and overall structural demands. The intention is for visitors to follow the “spiral” layout; first, one enters the woven pathway which leads to the woven dome. The visitors would then enter the spaces from left to right, starting in the relaxing inductive space, moving on to the sensory space, and finally exiting to the landscape in the environmental space.
CHAPTER 3

STRUCTURAL PROCESS
STRUCTURAL PROCESS

At the beginning of the quarter, the structure was developed from the idea of continuing the diagrid structure to the ground via braces. This proved to be a cumbersome method, and a lot of time was spent trying to remedy the fact that a passerby would “bonk” their head on the bracing. Eventually, the curved glulam column design came to be, remedying this and maintaining the curved form. At this point in the project, the scale was much, much larger. As shown, there was a second floor being modeled and the diameter of the dome was nearly 70 feet.
STRUCTURAL PROCESS

Originally, we discussed having continuous members from the bottom of the dome to the very top, which would mean double curved glulams. The more we looked at this design, the less viable it became. It took a long time to even find an example of a double curved glulam used in real life. The photographs show that it is possible, however very expensive due to many factors including difficulty of assembly, transportation, and constructibility. So, the design was reimagined. Here we have a comparison between having the full double curvature in our dome structure and having discontinuous straight members. As you can see, there is not much of a difference. This was a critical point in our structural design, as simplifying it into this form allowed for ease of member design, connection design, and construction.
STRUCTURAL PROCESS

The primary designer of the dome was an ARCE grad student, Bryan Garcia. The first ETABS model (top right) was made using continuous, double-curved members, whereas the final model (bottom right) used a series of straight members. We found that the new model performed much better. With minimal difference in the way the structure looked between the two versions, as discussed previously, we were eager to move forward with the new version. This would also allow us to use sawn lumber rather than curved glulams, saving the clients’ money in the long run. The results from Bryan’s computer analysis are shown below, and demonstrate the importance of this discovery.

<table>
<thead>
<tr>
<th></th>
<th>Old Model</th>
<th>New Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max Axial</td>
<td>19.2</td>
<td>11.4 (kips)</td>
</tr>
<tr>
<td>Max Shear</td>
<td>0.45</td>
<td>0.1 (kips)</td>
</tr>
<tr>
<td>Max Moment</td>
<td>3.1</td>
<td>1.5 (kip*ft)</td>
</tr>
</tbody>
</table>
STRUCTURAL PROCESS

A large portion of our structural design process involved a second floor within our dome structure. As stated previously, the design process was iterative and constantly adjusting with the client’s evolving vision. Initially, we considered having a second floor “nirvana” space – the literal center space for centering. We looked at a disc-like floor held up by a tree-like collection of columns. The collection of columns at the base ended up taking up about 5 feet of space, in diameter. This quickly became an undesirable design.

We then looked at the opposite configuration, a saturn ring walkway. The biggest issue with this was stability. The curved cantilevering columns naturally experienced large moment demands. We played around with creating a way to resolve moments using timber, but demands were too high.

Ultimately, the clients decided to bring down the building scale. In doing this, we found it impractical to maintain roof access, thus eliminating the desire for a second level within the dome. This naturally cut down a significant amount of the overall cost of the project, also making the clients significantly happier. The dome now represents the social space in the Center for Centering.
STRUCTURAL PROCESS

Another important component of this project was the walkway leading towards the dome. We pulled the woven pattern from the dome onto the pathway, as a way of form finding. This evolved into a roof-truss design, made up of a collection of three-hinged arches. This served as a lateral system in each direction, allowing us to maintain the timber structure we desired. The opening of the pathway to the parking lot spans 18 ft in width, and 10 ft in height. The width tapers to about 14 ft where the pathway meets the dome. Primary members in this structure came to be 4x4 members, using pressure treated douglas-fir.
STRUCTURAL PROCESS

The connections proved to be a bit difficult to visualize at first, since the members were originally long and curved in two directions. However, as the design of the dome was simplified the connections began to take shape. As stated many times before, this schematic design process was very iterative in nature. Shown below are three iterations of what our design process went through. These connections were designed using the NDS and provide axial strength between each of the timber members. The demands were found using ETABS by our grad student Bryan Garcia. This connection would be typical throughout the dome structure as well as the pathway, with the ability to change the angle depending on the needs of each joint.
STRUCTURAL PROCESS

Some time was spent looking at the more intricate and unique connections, such as the very tip of the structure. This design was done using the same process as was used for the typical connections. Having fourteen members connecting at one point is quite difficult. Sadly, it is also quite difficult to find a piece of wood that has such a tight radius as the ring shown. We found that a steel tube ring would allow for a very robust connection without taking too much away from the timber structure. This would also allow the very top of the structure to be more architectural in nature and for the form to be fully realized into a tip.
STRUCTURAL PROCESS

The structure of the sensory rooms were initially designed to be timber construction. It was finally decided that concrete would best suit our clients’ needs. The latest iteration of the sensory rooms involved curvilinear walls and floors, which is represented in the section below. The architectural vision for this room led to a few undesirable structural designs, such as the large gap in the curved roof, which is not something the design team had seen before.

The inductive room spans 30 feet on the outside perimeter, 15 feet where the wall meets the dome, and 18 feet in width. The roof slab has a human-shaped void (assuming 1:1 human scale), meaning that the slab is cantilevering from the surrounding walls, all around. Due to the given span and cantilever condition, the roof slab is estimated to be a minimum of 14” thick, per ACI 318-19 table 8.3.1.1 per thk = ln/33. The walls, in order to satisfy the out of plane moment demand from the roof slab, had to be 14” thick. As this causes a large increase in total project cost, it is advised that the roof is eliminated so the walls can become much thinner. Also, it is recommended that the tapering of the roof slab be removed. See calculations in appendix section.
STRUCTURAL PROCESS

The gong room represents the **sensory** space. The labyrinth represents the **environmental** space. In an effort to retain congruence, the outer walls were kept at the same width, even though they could have been significantly thinner otherwise due to having no roof. The wall of the labyrinth room begins at the same height as the other two rooms, but diminishes in height as the labyrinth comes to an end. This allows occupants to feel liberated into the environment as the labyrinth is completed.

As stated previously, these rooms were previously meant to be timber framed to remain homogeneous with the dome structure. However, it was decided that concrete would allow for a more desirable form that the clients felt would amplify the sensory experience.
STRUCTURAL PROCESS

We took a closer look at the connection between the timber beams that form the compression ring at the base of the woven dome and the concrete walls of the sensory spaces. The concrete walls serve as the primary lateral-force resisting system for the dome, so we felt that it was essential that we look at this connection.

The timber beams forming the tension ring are 6x6 pressure-treated sawn lumber, based on the governing lateral load of 11.5 kips. Steel angles are to be used on one side of the beam to provide lateral and uplift resistance with (4)-½” diameter lag screws and one 1 ½” anchor bolt. To separate the timber members from the concrete wall for moisture control, we recommend using a steel plate in between the two elements, or creating a substantial gap between the elements via steel connectors.
STRUCTURAL PROCESS

Upon review of our design and input from Michelle Kam-Biron, Professor Dong, and Professor Fowler, it became apparent that our columns would be insufficient. At this point, it was difficult to update renders and plans due to the studio course being completed already and our architects done for the quarter. However, the columns were reimagined as concrete to establish a moment capacity. The sizing (using LRFD) was found to be ACI code minimum. Four #4 bars were to be used longitudinally in order to allow for shear stirrups to be placed at 8” on center, the maximum allowable spacing due to the column dimensions.

This change was welcomed, as there was some uncertainty in the back of the design team’s minds about the capacity of these (previously glulam) columns. Now that the column design had been reimagined, and the curved wooden pieces had become non structural in nature, the design is certainly more realistic and achievable.
STRUCTURAL PROCESS

Conclusion

The Center for Centering provides a very unique experience for those who wish for an escape from the hustle and bustle of modern life. The building provides a chance for introspection and an opportunity to access senses that one may take for granted under normal circumstances.

As visitors walk through the weave they experience the natural light dancing and shining through the weave in a playful manner. Entering the dome they feel a sense of magnificence as the timber peak climbs to the sky. The inductive space provides a one of a kind opportunity to relax and connect with oneself. Lying on the clay balls, visitors can relax and fix their gaze upon the blue sky above. The sensory room allows visitors to access a primal urge to strike the large gong and feel the vibrations throughout their body. Finally, the environmental room opens one up to the beautiful landscape surrounding them.

The unique opportunity given to the structural design team this quarter was to use these architectural visions of both the client as well as the architecture team from the quarters prior and provide an analysis of the structure behind them. Through changes in scale, materiality, and program, the team has ultimately provided a concept and schematic design that they can be proud of. By providing their hard work and dedication, the team hopes that our clients, Marcos and Cynthia, can utilize this concept and fully realize its potential.
STRUCTURAL PROCESS

Challenges

Of the challenges faced, the primary challenge was keeping up with the clients’ evolving vision for the center. Alterations to the building plan, scale, program, and materiality were common throughout our time on this project. The main dome changed drastically; it started out as a 70-foot diameter space with a second level and ended as a 50-foot diameter space with only a ground floor level. The initial vision was for the spaces surrounding the dome to have green roofs, which visitors had access to and could even follow a pathway from the roof onto the second level. Naturally, as the team, including the clients, decided to reduce the building scale, there was no longer a need for a second level or roof access. We then felt that the green-roof would add complications that were not in the clients' best interest, so the green-roof was also removed. Additionally, the inductive rooms were originally envisioned as wood-construction, when it was decided that these spaces would instead be made up of concrete. In summary, change was a major theme of our time spent working on this project; this meant that we were able to help the clients get closer to their goal, but the depth of engineering was limited to light schematic design in order to simply give the clients a better sense of scale and practicality moving forward.
Reflections

Jonathan Oberempt:
One important takeaway was that when working collaboratively with clients and architects, it is important to be malleable in your design. Another takeaway is that no matter how many changes the design might go through, it will always be for the better, so trust the process. I learned a lot about how architects can help to find form in structure and the collaborative nature of design. The sensorial approach to design was something new but not unwelcome. I was extremely excited at the beginning of the quarter to be working with real clients, as it is something I had never experienced. It proved to be everything I had hoped for and more. They were very clear about what they wanted in their design and were always happy to give feedback, both positive and negative. It was certainly an experience that I will carry with me for the rest of my time in the workforce.

Faith Sharp:
This project brought my ARCE undergrad experience at Cal Poly full-circle. We essentially started out as architecture students our first year, then focused on the technical in the following three years. Working on the C4C design team challenged me to tap into my creative side again, while staying quick on my feet to come up with practical and economical solutions for our clients. We learned a lot from the clients as well. They helped us explore the ways we can think about and interact with architecture. The importance of sensory experience is something that will stick with me throughout my career. Further, I learned how difficult it can be to make a design perfectly in alignment with the vision of the client and the architect. Aside from the design aspects of this project, I have gained more perspective and respect for the architect-client relationship.

Sam Buckman:
With Center for Centering being the first project I have gotten to interact with clients and architects, I learned a lot about the structure of relationships between clients, architects, engineers, and investors. Now, I am able to better appreciate the rationale behind design decisions that I never could have before, and I feel as though I have more empathy for architects and clients alike. I now understand why effective communication is such a pivotal aspect of design, and I got to see first-hand what happens when communication is lacking. One thing I loved, however, was the opportunity I got to work alongside other engineers. The completion of this project makes me excited for what is to come, and I am very glad I had the opportunity to do so.
CHAPTER 4

CALCULATIONS
PROJECT INFORMATION

Project: Center For Centering (C4C)
Location: Swanton Ranch, CA
Architect: 
Owner: 
Jurisdiction: Santa Cruz, CA
ASCE 7-16
ACI 318-19
2018 NDS
2018 SDPWS

Structural Systems:
Vertical/Lateral
Diagrid Frame System
Trussed Three-Hinged-Arch Frames
Concrete Walls
Concrete Spread Footing Foundation
Concrete Slab - on - Grade

Building Description:
Construction Type: IV
Risk Category: II
Sprinkler System: No

Other:
Soils Engineer: 
Soils Report No.: 
Soils Report Date: 
Soil Bearing: D + L: 1500 psf, assumed
# LOADING - VERTICAL

## Dome Dead Load Take Off (PSF)

<table>
<thead>
<tr>
<th>Component</th>
<th>Description</th>
<th>Load (PSF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Facade</td>
<td>LW Arch’l Weaving</td>
<td>1.0</td>
</tr>
<tr>
<td>Beams</td>
<td>6x6 PTDF-L</td>
<td>2.5</td>
</tr>
<tr>
<td>Misc.</td>
<td></td>
<td>1.0</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td><strong>4.5</strong></td>
</tr>
</tbody>
</table>

Horizontal Proj. Conversion: 25/25

4.5 PSF (Horiz. Proj.)

## Columns

<table>
<thead>
<tr>
<th>Component</th>
<th>Description</th>
<th>Load (PSF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>(allowance)</td>
<td>3.0</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td><strong>7.5</strong></td>
</tr>
</tbody>
</table>

7.5 PSF (columns/ftgs)

## Shear Walls

<table>
<thead>
<tr>
<th>Component</th>
<th>Description</th>
<th>Load (PSF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>(allowance)</td>
<td>15</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td><strong>22.5</strong></td>
</tr>
</tbody>
</table>

22.5 PSF (walls/ftgs)

## Pathway Dead Load Take Off (PSF)

<table>
<thead>
<tr>
<th>Component</th>
<th>Description</th>
<th>Load (PSF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beams</td>
<td>4x4 PTDF-L</td>
<td>1.5</td>
</tr>
<tr>
<td>Facade &amp; Misc</td>
<td>LW Arch’l Weaving</td>
<td>0.5</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td><strong>2.0</strong></td>
</tr>
</tbody>
</table>

## Live Load (PSF)

<table>
<thead>
<tr>
<th>Component</th>
<th>Description</th>
<th>Load (PSF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ordinary Roof</td>
<td></td>
<td>20</td>
</tr>
<tr>
<td>Sensory Rooms</td>
<td></td>
<td>75</td>
</tr>
</tbody>
</table>

20 PSF (reducible)

75 PSF (for slab-on-grade)

### Notes:

1. Straight timber elements are Pressure Treated Douglas-Fir Larch (PTDF-L).
LOADING - LATERAL

RISK CAT. : II

BASIC WIND SPEED : 90 MPH

\( K_d = 0.85 \)

EXP. CAT. : C

\( K_{et} = 1.0 \) (ASSUMED)

\( K_e = 1.0 \)

\( G = 0.85 \)

\( K_r = 0.85 \)

\( q_n = 0.00250 \) psi

\( \frac{K_{et}}{K_e} \times \frac{G}{0.85} \times \frac{K_r}{1.0} \times V^2 = 15 \) PSF

\( C_H = 0.85 \)

\[
\begin{align*}
P &= g_c C_H (27.3 - 2) \\
&= 15 (0.85)(0.5) \\
P &= 0.4 \text{ PSF} \quad \text{WIND PRESSURE @ WALKWAY}
\end{align*}
\]

HORIZONTAL LOAD TO FRAME DUE TO WIND

\[
(\text{trib} \times \text{trib} \times \text{height} \times \text{wind pressure}) = 320 \#
\]

\[
\begin{align*}
\text{trib} &= 28.28 \text{ in}^2 = 12.4 \text{ ft}^2 \quad \text{<< f_c = 1350 psi \text{ (OK)}}
\end{align*}
\]

SEISMIC

WEIGHT = 35 PSF \( \left\{ \left( \text{vert. mem} \times \text{horz. mem} \times \left( 10 \% \times 4 + \left( 16 \% \times 2 \right) \right) \right) \right\} = 182 \#
\]

\( C_S = 1.472 \) (SEISMIC MAPS.ORG - SWANTON RANCH)

\( I_c = 1.5 \)

\( I_e = 1.0 \)

\( C_S = \text{EXPECTING WILL REDUCE SEISMIC WEIGHT} \times \text{SEISMIC WEIGHT} = 182 \# < \text{WIND LOAD} = 320 \#
\]

\( \text{WIND GOVERNS} \)
LOADING - LATERAL

Lateral - Dome
Wind Load

Risk Category: II
Enclosure Classification: Open
Exposure Category: C
Wind Speed: 90 MPH
Directionality factor $K_d = 1.0$
Topographic factor $K_t = 1.0$
Ground elevation factor $K_e = 1.0$
Gust factor $C = 0.85$

Internal pressure coefficient $C_{p_i} = 0$ (open)
Velocity pressure exposure coefficient $K_v = 0.96$

Velocity pressure

$$P_h = 0.00256 K_v K_t K_e V^2$$

$$= 0.00256 \cdot 0.96 \cdot 1.0 \cdot 1.0 \cdot 90^2 = 19.91 \text{ psi}$$

Net pressure coefficients:

- ASCE 7 provides no $C_{p}$ for open domes
- Model as pitched free roof, clear wind flow (diagram works in both directions)

Load Case A: $C_{N,L} = 1.3$, $C_{W,L} = 0.6$ $\theta = 0^\circ$

Load Case B: $C_{N,L} = 0.7$, $C_{W,L} = 0.6$ $\theta = 90^\circ$

Net Design Pressure

$$P = P_h C_{W,L}$$

Local Case A (max lateral load)
Design Wind Load Case 1 governs, $\theta = 0^\circ$

$$P_{W,L} = 19.91 \cdot 0.85 \cdot 1.3 = 22.0 \text{ psi}$$

$$P_{L,L} = 19.91 \cdot 0.85 \cdot 6 = 12.2 \text{ psi}$$

$$\Sigma P_x = 7.37 \text{ psi}$$

Local Case B (max uplift)
Design Wind Load Case 1 governs

$$P_{W,Y} = 19.91 \cdot 0.85 \cdot -0.2 = -3.38 \text{ psi}$$

$$P_{L,Y} = 19.91 \cdot 0.85 \cdot -6 = -10.2 \text{ psi}$$

$$\Sigma P_y = (-3.38 -10.2) \cos 38.7^\circ = -10.6 \text{ psi}$$
LOADING - LATERAL

Lateral - Dome

Load Case A (Max torsional moment)
Wind Load Case 2 governs

\[ M_T = 0.75 \left( P_{wx} + P_{lx} \right) B_x e_x \]
\[ e_x = 0.15 B_x = 0.15 \cdot 50' = 7.5' \]
\[ M_T = 0.75 \left( 7.37 \right) 50' \cdot 7.5' = 2.1 k \cdot ft \]

Max horizontal force: 7.37 psf \((20' \cdot 50') = 7.37 \times \)
Max uplift force: 19.6 psf \((20' \cdot 50') = 19.6 k \)
Max. \( M_T = 2.1 k \cdot ft \)

Seismic Load

Weight Dome = 41.5 psf \cdot Area Dome
Assume dome is spherical \( A_{Dome} = \frac{4}{3} \pi r^2 = \frac{4}{3} \pi \cdot 25^2 = 7854 \frac{4}{2} = 3927 \)

\[ W_{Dome} = 41.5 \text{ psf} \cdot 3927 = 17671.5 \text{ lb} \]

\[ V = C_5 W \]

Find \( C_5 \)

\[ T_a = C_6 h_a = 0.02 \cdot 30 \cdot 75 = 0.256 \]
\[ T_L = 12 \]
\[ C_5 = \frac{S_{d5}}{T_{d5}} \left( \frac{5}{I_{d5}} \right) = \frac{1.472}{(4/1)} = 0.368 \]

but need not exceed
\[ C_5 = \frac{S_{d1}}{T_{d1}} \left( \frac{5}{I_{d1}} \right) = \frac{.717}{.256 (4/1)} = 1.700 \]

\( C_5 = 0.368 \)

\[ V = C_5 W = 0.368 \cdot 17671.5 = 6.5 k \text{ Does not govern} \]
TYPICAL COLUMN DESIGN

Estimating demand...

\[ \text{Area} = \frac{1}{2} (4\pi r^2) = \frac{1}{2} (4 \pi (25')^2) = 3927 \text{ ft}^2 \]

DL columns: 7.5 PSF  LL columns: 20 PSF  \( \rightarrow \) 112 (7.5 PSF) + 1.6 (20 PSF) = 41 PSF

\[ \frac{P_{col}}{7 \text{ columns}} = 23.0^\circ \text{ (VERY low)} \]

Select trial size...

\[ A_g\text{trial} = \frac{23.0^\circ}{0.40(f'_c + f'_yB)} = \frac{23.0^\circ}{0.40(4500 + 60(0.015))} = 11.7 \text{ in}^2 \]

A 4\"x4\" column would not be good, use minimum of 8\"x8\" column.

\[ A_g = 64 \text{ in}^2 \]

Design reinforcement...

\[ f = 0.6 \]

\[ \frac{P_u}{bh} = \frac{26.4^\circ}{(8" \times 8")} = 0.4125 \]

Using P-M diagram to determine \( \rho_g \) required. (Wight p.1094)

Use \( \rho = 0.01 \) since our moment demand is very low

\[ A_s = \rho_g A_g = 0.64 \text{ in}^2 \]

We need a minimum of 4 bars for rectangular columns

Use (4) #4 BARS \( (A_s = 0.80 \text{ in}^2) \)

\[ \rho_g = \frac{0.80 \text{ in}^2}{64 \text{ in}^2} = 0.0125 \]

Shear reinforcement... (#3 TIES)

\[ S_{max} = \begin{cases} 1.64 \\ 4HD_2 = 18" \\ \text{min col. dim. = 8"} \end{cases} \rightarrow \text{CONTROL} \]

\[ V_c = (2 \frac{\sqrt{f'_c}}{\rho_g} + \frac{N_v}{6})_{\text{min}} = (2)(1.0)(\sqrt{4500}) + \frac{(26.4^\circ)}{6(64 \text{ in}^2)}(8\")(5.875\") = 9^\circ \]

Provide #3 TIES @ 8" o.c.
DOME LOWER RING BEAM DESIGN

Try DF-L No. 1 6x6

Check Tension

\[ f_t = \frac{P}{A} = \frac{11.5}{\frac{6.5 \times 5.5}{2}} = 380.2 \text{ #} \]

\[ F_t = F_c C_0 C_m C_e C_f C_i = 225 \times 1.6 \times 1.0 \times 1.0 \times 1.0 = 1320 \text{ #} > 380.2 \text{ #} \checkmark \]

Check Bending

\[ f_b = \frac{M}{5} = \left( \frac{47.5^2}{8} \right) \left( \frac{225}{27.73} \right) = 265.8 \text{ psi} \]

\[ \omega = 7.37 \text{ f/50' } = 147.4 \text{ ft} \]

\[ l = 20' \]

\[ F_b = 1350 \times 1.6 \times 1.0 \times 1.0 \times 1.0 \times 1.0 = 2160 \text{ psi} \] \[ f_b \checkmark \]

Check Shear

\[ V = \frac{47.5}{2} = 147.4 \text{ #} \]

\[ f_v = 1.5 \frac{47.5}{32.25} = 73 \text{ psi} \]

\[ f' = 170 \text{ psi} \] \[ f_v \checkmark \]

DF-L No. 1 6x6 is adequate
PATHWAY MEMBER SIZING

BEAMS

LOAD TAKE-OFF

PRIMARY FRAME MEMBERS: TRY 4" x 4" (A = 12.25 in²)

UNIT LOAD: 35 psf (CF-L)

TRIB. WIDTH: ~ 2' (allows for 20' (10' x 2') = 412# PT LOAD)

SELF WEIGHT: (12.25 in²/144.35) / 12' = 1.5 PSF

FLEXURE (SELF-WEIGHT)

\[ V' = \frac{P}{L} = \frac{1.5}{12} = 0.125 \text{ in} \]

DEMAND

WARCH' & MISC: DL = 2.0 PSF (ASSUMED)

1.2(D - 1.2(1.5)) = 1.8 PSF

\[ W = 1.8 (2') = 3.6 \text{ PLF} \]

\[ V_{\text{MAX}} = 3.6 (10.3') / 2 = 18.5 \text{#} \]

\[ M_{\text{MAX}} = 3.6 (10.3')^2 / 8 = 47.7 \text{#} \\text{in} \]

\[ f_p = \frac{V}{b} = \frac{47.7 (18)}{7.15 \text{ in}^2} = 30.1 \text{ psi} \]

\[ M_{\text{CAP}} = 63 \text{#} \text{ in} < 47.7 \text{#} \text{ in} \]

\[ f_p = \frac{M_{\text{CAP}}}{1.5} = 30.1 \text{ psi} \]

CHECK

WHERE HAVE MOMENT, HAVE SHEAR.

MOMENT CAPACITY MORE THAN SUFFICIENT FOR

SELF-WEIGHT, ASSUMING SAME FOR SHEAR.

Note: Span at entrance is assumed to be 18',
adjusted from the architect's request for 20'.

See NDS 14A.
For RISA, assumed DL_{total} = 4.0 PSF

\[ 1.2D + 1.6L = 1.2(4) + 1.6(10) = 20.0 \text{ PSF} \]

PT. LOAD @ MIDDLE PIN = 36.8(0.3)(2) = 758 #
SKY 0.8k

\[ \text{PT. LOAD @ CORNERS} = 758/2 = 379 # \]
SKY 0.4k

Uses throughout length of pathway, where max = 10.3" OK, to assume 10.5" in RISA.

**FBD Joint A**

\[ 2F_y = 0 : \quad 0.8 = 2(F_y / 3)(0.73) \]
\[ \Rightarrow F_y = 1.740 \text{ k} \]

FROM RISA2D

\[ \checkmark \text{RISA looks good.} \]

See A1 for RISA documentation

**Axial**

\[ N_{max} = 1.740 \text{ k (see RISA outputs)} \]

\[ \sigma = \frac{N}{A} = \frac{F_y}{1350} = 1.29 \text{ in}^2 < 12.35 \text{ in}^2 \]

\[ \text{Where } F_y = 1350 \text{ PSF} \]

**Serviceability**

\[ 0.5D + L : \text{ Allow} = \frac{L}{120} = 10.3(12) = 1.03" > 0.13" \checkmark \]

Due to

\[ 1.2D + 1.6L \]

More than OK

\[ L \text{ ONLY : Allow} = \frac{L}{180} = 10.3(12) = 0.69" > 0.13" \checkmark \]

**Columns**

\[ P_{cr} = \frac{\text{Allow} \cdot A}{F_y} = 1350(1.1)(0.8)(12.25 \text{ in}^2) = 14.6 \text{ k} \]

\[ N_{max} = 2(N_{UM_{max}}) = 2(0.73) = 1.5k < < 14.6k \checkmark \]

Use 4x4 Pressure-Treated Douglas Fir-Larch for all framing members.
ROOF SLAB - INDUCTIVE ROOM

Estimate thickness per The Architect's Studio Companion.

Span range of two-way slabs: 40' maximum / 
Per ACI Table 8.3.1.1

\[ \text{thk}_{\text{min}} = \frac{20 \times 12}{33} = \frac{240}{33} = 11'' \text{ thk} \]

Because slab is sloped and cantilevered, this further increases minimum thickness. As this creates a large moment on the supporting walls, it is advised that roof opening is removed from design or changed in design. This would divide wall thickness by approximately two and reduce project cost significantly.

For rest of wall design for sensory rooms, will assume that roof is removed from inductive room.

WALL DESIGN - OUTER WALL SENSORY ROOMS

Per table 11.3.1 - Minimum wall thickness = 4' or \( \frac{3}{25} \) where

\[ L = \text{unsupported height or length} \]

\[ \text{wall thk} = \frac{10 \times 12}{25} = 4.8 \rightarrow 5'' \text{ wall} \]

WALL DESIGN - INNER WALL

Lateral force

\[ L = 500'' \text{ for beam-connection requirement} \]

\[ P_u = 1.2 \times (150 \times 0.5) \times 10 \times 50' = 67.5k \]

\[ V_u = 1.6 \times (7.37') = 11.8k \]

\[ (11.5.3.1) \]

\[ \phi P_n = 40.55 \times c A_g \left[ 1 - \left( \frac{k L_c}{324} \right)^2 \right] \]

\[ = 9.55 \times 4000 \times 5400 \left[ 1 - \left( \frac{72.6 \times 12}{324} \right)^2 \right] = 12000k \]

\[ d P_n = 10520k > P_u \checkmark \]

\[ \text{in-plane shear} \]

\[ \frac{V_n}{(\phi c \sqrt{f_c} + \rho f_y t)} = \frac{205k}{711.8k} \checkmark \]

\[ \text{t 11.4.1} \]

Min reinforcement:

\[ \min p = \text{longitudinal} = 0.0015, \min p = \text{transverse} = 0.0025 \]

Transverse: use (2) #4 bars: \[ P_e = A_s f_y = 0.04 \times 7,000 = 28k \]

Longitudinal: use #4 bars at 24" o.c.: \[ P_e = 0.070 \times 0.025 = 0.0025 \]

\[ \checkmark \]
Out-of-Plane Shear

22.5.5 \[ V_c = \left[ 2\pi \sqrt{F_c} \cdot \frac{N_n}{k_n} \right] b_1 d \]

\[ = \left[ 2.1.9 \cdot 4000 + \frac{675.00}{6 (4.5512)} \right] \left( \frac{9}{12} \right) (10.12) \]

\[ = 13.6^k \]

\[ \phi V_c = 17.3^k \quad \Rightarrow \quad V_U = 11.8^k \]

USE 9" thk inner wall
TYPICAL COLUMN FOOTING

\[ f'_c = 4 \text{ ksf} \]
\[ f_y = 60 \text{ ksf} \]
\[ F_b = 7000 \text{ PSF} \]

\[ P_u = 23.0^k \]

\[ A_{req} = \frac{P_u}{F_b} = \frac{23.0^k}{2.0 \text{ ksf}} = 11.5 \text{ ft}^2 \]

TRY 3'-8" sq. ft. (13.44 ft²)

\[ A_{req} = 0.0018 \text{ kbf} = 0.0018 (3.67^r)(1.25') = 0.00826 \text{ ft}^2 = 1.19 \text{ m}^2 \]

USE (6) #4 BARS \( (A_s = 1.2 \text{ in}^2) \) \( (A_s = 32.7 \text{ in}^2/ft) \)

LD for #4 BARS = 12" \( \checkmark \)

STRENGTH:

\[ f_{bu} = 26.4^k \]

\[ f_{bu} / 13.44 \text{ ft}^2 = 1.96 \text{ ksf} < 2 \text{ ksf} \]

\[ \checkmark \]

FLEXURE:

\[ M_0 = \frac{f_y}{3} (1.96 \text{ ksf})(1.4)^3 = 1.92 \text{ kips/ft} \]

\[ \alpha = \frac{(0.327 \text{ in}^3/ft)(60 \text{ ksf})}{(0.85)(4 \text{ ksf})(12')} = 0.481'' \]

\[ \phi M_h = (0.9)(0.327 \text{ in}^3/ft)(60 \text{ ksf})(15' - 4'' - \frac{4.81''}{2})(\frac{15''}{12''}) = 60.81 \text{ kips/ft} > M_0 \]

\[ \checkmark \]
Foundations - Pathway

Gravity

Col. Load = 1.7+1.2 (2.5+0.8) = 2.5k

[Conservatively Using Max Axial Demand in Frame]

1.5 + 1.0 = 1.0 12k

Overturning

\[ \text{EM}_1 = D \times \text{EM}_{10} = C \times 155^\circ < 3.5k \]

Size for Gravity

Footprint:

\[ \frac{2300}{1500} = 1.53 \rightarrow \sim 1\text{-ft sq. fig} \]

or

1\text{-ft circular}

Very low demand

\[ \rightarrow \text{Assume min depth} \]

\[ 0.001B^2 = 0.001 \times (12\text{"})(18\text{"}) = 0.40 \text{ in}^2 \rightarrow (2) \# 4\text{"} W-12\text{"} \]

Use 1\text{-ft sq. fig w/ 12\text{"} depth, min}

\& (2)\# 4\text{"} W-12\text{"} @ 6\text{"} col.
SLAB DESIGN, SENSORY ROOMS

Loading:

\[ DL \]
Assume 8" slab
\[ DL_{slab} = \left( \frac{8}{12} \right) \cdot 150 \text{pcf} = 100 \text{ psf} \]
use 105 psf for design

Typ. Section (worst case)

\[ 20' - 0" \]

\[ LL \]
- 75 psf (recreational use)

\[ TL \] (factored)
\[ 1.2D + 1.6L = 1.2 (105 \text{ psf}) + 1.6 (75 \text{ psf}) = 245 \text{ psf} \]
(1800 psf service)

Find required thick. for slab to remain uncracked at midspan
this allows max deflection: \( \frac{5.2L^4}{384EI} \), \( I = \frac{bt^3}{12} \), use of service loads,
and uncracked modes of rupture

Modulus of Rupture: \( f_r = 7,500 \sqrt{f_c} = 7,500 \cdot \sqrt{4000} = 4,743 \text{ psi} \)

Modulus of Elasticity: \( E = 57 \sqrt{4000} = 3600 \text{ ksi} \)

\[ M_{\text{midspan}} = 0.04wL^2 - \frac{wX(L-x)}{2} = 0.04wL^2 - 0.12wL^2 \]
\[ = 0.04(180 \text{ psf})(20')^2 - 0.12(180 \text{ psf})(20')^2 = -5.8 \text{ k-ft} \]

ACI limit: \( M_{r}^{max} = \frac{wL^2}{11} = \frac{180 \cdot 20^2}{11} = 6.5 \text{ k-ft} > 5.8 \text{ k-ft} \)

Find required thick per foot of slab

\[ s = \frac{bt^2}{6} \]
\[ s = \frac{M_{r}}{f_r} \] (uncracked assumption)

\[ t = \sqrt{\frac{6s}{12f_r}} = \sqrt{\frac{6 \cdot 5.8 \text{ k-ft}}{12 \cdot 4743}} = 8.56 \rightarrow 9" \text{ thick slab} \]

Deflection Check

Assume uncracked \( \Delta_{\text{midspan}} = \frac{5wL^4}{384EI} \)
Assume simply supported \( \Delta_{\text{actual}} = \frac{5wL^4}{384EI} \) (conservative)

\[ \Delta = \frac{5wL^4}{384EI} = \left( 5 \cdot 100 \text{ psf} \cdot (20')^2 \cdot 1728 \right) / 384 \cdot 3600 \text{ ksi} \cdot (12 \cdot 9^2) \]
\[ = 0.247" \]
Proportion Slab so concrete resists 100% of shear

\[ V_u = 1.15 \left( \frac{w \cdot b}{2} \right)^{\frac{3}{2}} = 1.15 \left( \frac{0.245 \cdot 20}{2} \right) = 2.82 \text{kips} \]
\[ V_{u \text{max}} = \frac{S}{b} \cdot w \cdot L = \frac{S}{b} \left( \frac{V_{u \text{max}}}{2} \right) = 3.76 \text{kips} \Rightarrow V_u \checkmark \]

**SLAB SHEAR CAPACITY:**

\[ \phi V_c = \frac{\phi f' c}{g} bd \Rightarrow \frac{V_u - 1000}{\phi f' c/b} = \frac{2.82 \cdot 2 \cdot 1000}{2 \cdot 75 \cdot 5/12} = 2.2 \]

\[ q'' = t_{\text{slab}} \]

Must be greater than 2d (Since tension face switches)

\[ q'' \geq d + d = 4.4'' \]

**USE A 4.4'' THK CONC. SLAB**

**Determine Flexural steel required**

\[ \phi V_c = \frac{1}{6} \frac{w \cdot b^2}{f' c} \Rightarrow \frac{1}{6} \left( \frac{0.245 \cdot 20}{2} \right) = 8.9 \text{ kips} \]
\[ M_v = \frac{1}{10} \frac{w \cdot b^2}{f' c} = \frac{1}{10} \left( \frac{0.245 \cdot 20}{2} \right) = 9.8 \text{ kips} \]

**Estimate Steel Tension**

\[ A_s = \frac{V_u - \phi V_c}{f_y g' c d} = (7.8 - 12) \quad \frac{9.60 \cdot 3^2}{9.9 - 60} = 0.81 \text{ in}^2 / \text{ft} \]

Try \( 1 \# 7 \) @ 6'' o.c. (\( A_s = 1.20 \text{ in}^2 \))

\[ p = \frac{A_s}{b d} = \frac{1.20}{9.12} = 0.131 > 0.018 \checkmark \]

**Check Flexural Capacity**

\[ a = \frac{A_s f_y}{g'b c} = \frac{(1.20 \cdot 60) \cdot (0.85 \cdot 4 \cdot 3)}{(0.85 \cdot 4 \cdot 3)} = 2.35'' \]
\[ \phi M_a = 0.9 \left( 1.20 \cdot 60 \cdot 3^2 \right) \left( 3'' - \frac{2.35''}{2} \right) \left( \frac{1}{12} \right) = 9.9 \text{ kips} \]
\[ 9.9 \text{kips} > 9.8 \text{kips} \checkmark \]

\[ p = 0.111 \]
Shear Check

Try using #4 @ 12" o.c.

\[ \phi V_c = \phi \frac{2f_{c}'}{\sqrt{\frac{1}{60}}} = 0.75 \cdot 1.0 \cdot \frac{2 \sqrt{4000}}{\sqrt{18 \cdot 30}} = 51.2 \text{k} \]

\[ \phi V_s = \phi A_v f_y \frac{d}{s} = \frac{1.75 \cdot 2 \text{in} \cdot 60 \text{psi} \cdot 3(\frac{3}{8})}{12} = 22.5 \text{k} \]

\[ V_u = 1.15 \left[ \frac{w \cdot L^2}{2} \right] = 1.15 \left[ \frac{551 \cdot 20^2}{2} \right] = 60.9 \text{k} \]

\[ \phi V_c + \phi V_s = 51.2 \text{k} + 22.5 \text{k} = 73.7 \text{k} \rightarrow V_u = 60.9 \text{k} \]

USE #4 @ 12" o.c.

W_u, worst case

D = 120 psf, L = 75 psf

\[ \begin{align*}
V &= 265 \text{ psf} \\
W &= 265 \text{ psf} \cdot (20') = 5.3 \text{k}f_e
\end{align*} \]
FOUNDATION DESIGN - SENSORY ROOMS

Continuous footing:

\[ f'c = 4 \text{k} \]  
\[ f'y = 60 \text{k} \]  
\[ F_b = 1500 \text{k} \]

Find \( P_v \)

\[ P_v = 1.2 \left[ (10 \cdot 150 \text{in}^2 \cdot (6/12)) - \left( (75 \text{ft}^2 \cdot 10' \right) \right] + 1.6 \left[ \frac{75 \text{in}^2 \cdot 10'}{1500 \text{k}} \right] = 3.8 \text{k} \]

\[ \text{Array} = \frac{P_v}{F_b} = \frac{3.8 \text{k}}{1500 \text{k}} = \text{TRY 2"-6" cont. footing} \]

Determine distance "c"

\[ c = 4 \left( 12.5 - 1.17 \right) = 5.7 \]
\[ d = 2.2 \sqrt{\frac{P_v}{A_s} \cdot c^2} = 2.2 \sqrt{\frac{3.8}{2.5} \cdot 5.7^2} = 14.8'' \]
\[ h = d + 4 = 19'' \]

\[ A_s = 0.0012 \text{ in}^2 \cdot 2.5 \cdot 12.19'' = 1.0 \text{ in}^2 \]

Try \( S = 4 \) bars

\[ A_s = 5 \left( 0.20 \text{ in}^2 / \text{bar} \right) = 1.0 \text{ in}^2 \geq 1.0 \text{ in}^2 \]  
\( (A_s = 0.003 \text{ in}^2 / \text{bar}) \]

\[ L_4 = 12'' \text{ for } #4 \text{ bars} \]

Check strength

\[ f_{bu} = 4.1 \text{k} \]  
\[ f_{bu} / 2.5'' = 1.64 \text{k} \]

Check beam shear

\[ V_u = f_{bu} \cdot c = 1.64 \cdot 5.7 = 9.35 \text{k} \]
\[ dV_{u} = 0.7 \left[ 2 \sqrt{450} \cdot 12 \cdot 15 \cdot \frac{1}{1000} \right] = 17 \text{k} \]

Check Moment

\[ M_u = \frac{1}{2} \cdot 1.64 \cdot 5.7^2 = 26.6 \]
\[ \phi M_u = 0.9 \cdot 10 \cdot \sqrt[1]{2 \cdot 10 \cdot 5.7 \cdot 15 - 4.5 \cdot 122 \left( \frac{1}{2} \right)} = 67 \text{k} \]
**Connection Geometry**

\( \frac{1}{2} \) Dia. Bolts

Minimum End \( D = 7D \) for tension = 3.5"

Minimum \( S = 4D = 2" \)

Minimum Edge \( D = 1.5D \) or \( 1" \geq 1" \)

Minimum \( S_{rows} \)

\( \mu/D = 6^\circ/5^\circ = 12" \rightarrow S_{rows} = 2.5" \)

\[ C_y = 0.89 \quad (6 \text{ bolts}) \Rightarrow Z_n = 12,870 \text{#} \]

\[ C_y = 0.80 \quad (8 \text{ bolts}) \Rightarrow Z_n = 15424 \text{#} \]
DOUBLE SHEAR

$G = 0.50$

$\frac{1}{2}''$ DIA. BOLT

8'' BOLT w/ 7/16'' THREAD

$Z_{11} = 2410\#$
WALL - DOME CONNECTION

Must resist lateral force = 7.37 k, uplift force = 10.6 k

Length of concrete wall = 55' 0"
Uplift force (k LF) = 10.6 k / 55' = 0.19 k LF
Lateral force = 7.37 k / 55' = 0.13 k LF

4 Log Screws
Zn = 2000 lb (lateral, will not govern)
Zn = 1280 lb (must resist uplift)
W = 1512 lb (lateral, governs)
Spacing between rows = 2.5 (d) = 1.25" (Min.)
Spacing of holes = 4D = 2" (Min.)

Lag screws will govern, by inspection

Find spacing of connection
Lateral:
\[
W \cdot \# \text{ of connections} = \text{Lateral force} \times 1.6
\]
\[
1.51 \times n = 7.37 \times 1.6
\]
\[
n = 7.8 \approx 8 \text{ connections}
\]
Spacing: 55' / 8 = 7' 0" o.c.

Uplift:
\[
Zn \cdot \# \text{ of connections} = \text{Uplift} \times 1.6
\]
\[
1280 \times n = 16.96 k
\]
\[
n = 13.25 \approx 14 \text{ connections (5 = 4' 0")}
\]

Use connection at 4' 0" o.c.

Check:
\[
Zn = 1.28 \times 14 = 18 > 10.6 \times 1.6 = 17 \checkmark
\]
\[
W = 1.5 \times 14 = 21 > 7.37 \times 1.6 = 11.8 \checkmark
\]
PATHWAY CONNECTIONS

@ EA. MEMBER END (TRANSFER AXIAL) $N_{Ax} = 1.74\psi k$

(22) 10# SINKERS ($\psi_i = 82#/$mil) w/ 10 ga steel fl

or

(2) ½" 4 bolts ($\psi_i = 830#/$bolt) w/ ¼ Ax00 steel plate

@ COLUMN BASE
RECOMMEND SET COL. CAP OR COVER FOR PROTECTION FROM MOISTURE.
FROM SAP2000:

Tentative max load = 1K (compression)
Prelim size: 3/8" x 6"
Material: 24F-V8

Need $F'_c$

$F'_c = C_D C_M C_f C_p$

$C_D = 0.9$ (dead)
$C_M = 1.0$ (dry)
$C_f = 1.0$ (normal temp)
$C_p = \frac{1 + \frac{F_{ce}/F_{c,\text{a}}}{2e}}{2} - \sqrt{\left(1 + \frac{F_{ce}/F_{c,\text{a}}}{2e}\right)^2 - \frac{F_{ce}/F_{c,\text{a}}}{e}}$

$F_{ce} = \frac{0.822 (0.95 \times 10^4)}{(15.1 / (0.115/12))^2} = 232.3$

$F_{c,\text{a}} = 0.9 (1650) = 1485$

$F_c = 1485 (0.158) = 227 \text{ PSI}$

$f = \frac{P/A}{(3.173 \times 6)} = 53.3 \text{ PSI}$

53.3 PSI < 227 PSI ✓
CURVATURE MEMBER (L2→L3)

R = 49'
arc length = 27.22'
t = 1/2''
t/R = 0.00278

\[ C_c = 1 - 2000 \left( \frac{t}{R} \right)^2 = 0.985 \]
\[ P = 0.9 \]

\[ M = 0.8 \text{k-in} \]

\[ M_{L2} = 0.5 \text{k-in} \]

\[ V_{1/2} = 0.1 \text{k} \]

\[ V_L = 0.1 \text{k} \]

\[ F_c' = 227 \text{ psi} \]

\[ F_b' = C_D C_M C_S C_V f_{f0} C_c \]

\[ C_D = 0.9 \]

\[ C_M = 1.0 \]

\[ C_V = \left( \frac{21}{28.27} \right) \left( \frac{12}{0.1} \right) \left( \frac{6}{3.125} \right) = 0.909 \]

\[ C_c = 0.985 \]

\[ F_b' = 2400 \text{ psi} \times (0.9)(0.909)(0.985) = 1934 \text{ psi} \]

\[ F_V = C_D C_M C_S C_V \]

\[ C_D = 0.9 \]

\[ F_V = 280 \text{ psi} \times (0.9) = 252 \text{ psi} \]

\[ f_{b_{33}} = \frac{0.8 \text{ k-in}}{30.75 \text{ in}^2} = 26.0 \text{ psi} \]

\[ f_{b_{22}} = \frac{0.5 \text{ k-in}}{26.27 \text{ in}^3} = 19.0 \text{ psi} < 1934 \text{ psi} \checkmark \]

\[ f_{V_{33}} = \frac{3V}{2A} = \frac{3(0.1 \text{k})}{2(30.75 \text{ in}^2)} = 4.88 \text{ psi} \]

\[ f_{V_{22}} = 4.88 \text{ psi} < 207 \text{ psi} \checkmark \]

\[ f_c = \frac{500}{30.75 \text{ in}^2} = 16.3 \text{ psi} < 227 \text{ psi} \checkmark \]
INNER CORE - ALL COLUMN

- 8 COLUMNS
- 2 2ND FLOOR = 5D'

LOADS
DEAD: 20 PSF (ESTIMATE)
LIVE: 80 PSF

FLOOR PT. LOAD ON COL:
\( F \left( \frac{2D^2}{8} \right) = 25k \) SEE COORD. DOC.

SELF WEIGHT = 20 PSF

PT. LOAD @ TOP = 20PSF(1')(20') = 0.5 KIPS

CURVED RADIUS = 18''-0''

DEMAND: AXIAL: 37k
SHEAR: 29 k
MOMENT: 158 KFT

SIZE MEMBER

\( f_u = 2400 \text{ ksi} \) (NDS table 5A)
\( f_u = 125 \rightarrow S_{myx} = M \frac{f_u}{f_y} = \frac{158(1000)(2)}{2400} = 790 \text{ in}^3 \)

\( \text{TRY A 10 3/4'' x 21'' (s = 790.1 \text{ in}^3)} \)

\( f_y = \frac{158(1000)(2)}{790.1} = 2399.7 \text{ ksi} \)

CHECK FLEXURE

\( F_f = \frac{F_0 C_c C_{vc} C_{f} C_{d} C_{t} C_{s}}{L^0} \)

\( C_c (5.8.8) \)
\( 0.8/(1.5) = t = 3/4'' \rightarrow USE 3/4'' \\
LAMS \ (12\% 3/8'') \)

\( t/f = \frac{9/4''}{18'(2)} = 0.00947 < \frac{1}{20} = 0.008 \sqrt{5} \)

\( C_c = 1 - \frac{2000 (0.00947)^2}{18(2)} = 0.9940 \rightarrow 0.9759 \text{ LAM.} \)
\[
\begin{align*}
C_l &= \frac{3 \cdot 3.3}{3} \\
\therefore \quad \frac{l_u}{L_u} &= 1.81 \quad \Rightarrow \quad \frac{L_u}{L_c} &= 1.87 \\
R_B &= \frac{33.6(2)(21))}{\sqrt{(10.75)^2}} = 8.528 \quad (\text{in.})
\end{align*}
\]

\[
\begin{align*}
F_{pe} &= \frac{1.20(E_i)}{R_B^2} = 1.20(0.936c) = 1555.8
\end{align*}
\]

\[
\begin{align*}
F_p &= 2400(1.25)(0.98) = 2940
\end{align*}
\]

\[
\begin{align*}
C_l &= \frac{1 + \left(\frac{15558}{2940}\right)}{1.9} = \sqrt{\left(\frac{1 + \left(\frac{15558}{2940}\right)}{1.9}\right) - \left(\frac{15558}{2940}\right)}
\end{align*}
\]

\[
\begin{align*}
C_l &= 0.989
\end{align*}
\]

\[
\begin{align*}
C_v &= 1.0 \quad (\text{assumed}) \quad \Rightarrow \quad \text{USE } C_l
\end{align*}
\]

\[
\begin{align*}
F_v &= 2940(0.989) = 2907 \quad \text{PSI} > 2399 \quad \text{PSI}
\end{align*}
\]

**CHECK COMPRESSION/TENSION (SAME SINCE USING BALANCED)**

\[
\begin{align*}
f_c &= \frac{37.5(1000)}{225.8} = 164 \quad \text{PSI}
\end{align*}
\]

\[
\begin{align*}
f_c' &= \frac{C_v \times C_l \times F_c}{1.25} = 20625 \quad \text{PSI} > 104 \quad \text{PSI}
\end{align*}
\]

**Radial Stress**

\[
\begin{align*}
\text{Vary } \text{later}
\end{align*}
\]

**Shear cheek**

\[
\begin{align*}
f_v &= \frac{F_c}{A} = \frac{2940(1000)}{225.8} = 193 \quad \text{PSI} < 331 \quad \text{PSI}
\end{align*}
\]

**ONLY WORKS IF ADD BENDING**

\[
\text{USE } 10\frac{3}{4} " \times 2" \quad \text{(BALANCED)}
\]

\[
\begin{align*}
L &= \frac{d^2}{6} = \frac{17^3}{6} = 818.8 \quad \text{in.}^3 > 790.1 \quad \text{in.}^3
\end{align*}
\]

\[
\begin{align*}
\text{OK.}
\end{align*}
\]
$P = 43 + 8.5 \times 2 = 60" = 5'0"

$c = 190" \rightarrow d = 43"

$c = 60" \times 2 = 120"

$c = 60" \times 2 = 120"

$c = 60" \times 2 = 120"

Footprint of bundle of columns @ base

Footprint of bundle of columns @ base
INNER CORE - OPTION I (SAM'S) W/LAT. BRACING

8 COLUMNS

2ND FLOOR = 50'

LOADS

DEAD: 20 PSF (ESTIMATE)
LIVE: 80 PSF

FLOOR PT LOAD ON COL:

\[ T \left( \frac{93^2}{2} \right) / 8 (100 \text{ PSF}) = 25k \]

SE coordinate doc.

SELF WEIGHT = 20 PSF

\[ \approx 0.5 \text{ kips} \]

CURVED RADIUS = 5'-0''

DEMAND:

AXIAL: 39 k (\( \frac{47}{12} \)) = 65.5 K
SHEAR: 8.5 k (\( \frac{47}{12} \)) = 14.28 K
MOMENT: 65 KFT (\( \frac{47}{12} \)) = 109. KFT

SIZE MEMBER

\[ f_b = 2400 \text{ psi} \quad (\text{NDS table 5A}) \]

\[ f_b = 115 \rightarrow S_{vot} = \frac{M}{f_b} = \frac{95 \text{ kip} \times 12}{2400} = 54.5 \text{ in}^3 \]

\[ \text{Try } A 3\frac{3}{4}'' \times 19\frac{1}{2}'' (S = 557.5 \text{ in}^3) \]

\[ f_b = \frac{109 \times 1000 \times (12)}{557.5} = 2358.9 \text{ psi} \]

\[ S = 557.5 \text{ in}^3 \]

CHECK FLEXURE

\[ P'_f = \frac{f_c \times C_D \times C_E \times C_a \times C_v \times C_T}{2 \times 1.25} \]

\[ C_D = 1.0 \]

\[ C_E = 1.0 \]

\[ C_a = 1.0 \]

\[ C_T = 1 \]

\[ C_V = 1 \]

\[ f_c = 5.8 \text{ksi} \]

\[ R = 125t \]

\[ 5''(12''/4) = t \approx 0.25'' \rightarrow USE \frac{3}{8}'' \text{ LAMs} \]

\[ t_K = \frac{3}{8}'' = 0.00375 \times 125 = 0.008 \sqrt{8} \]

\[ C_v = 1 - 2000 \left( \frac{3}{8}'' \right)^2 = 0.922 \]
\[ c_L = (3.3) \quad \Rightarrow \quad k_e = 1.84, L_e = 33.60' \]

\[ R_B = \sqrt{\frac{33.6(12)(19.2)}{(8.92^2)}} = 10.13 \]

\[ F_{BE} = \frac{1.20(E_{min})}{R_B^2} = 1.20(0.95,E_{cs}) = 1101 \]

\[ F_{P*} = 2400(1.25)(0.92) = 2760 \]

\[ c_L = \frac{1 + \left( \frac{1101}{2760} \right)}{1.9} = \sqrt{\frac{1 + \left( \frac{1105}{2760} \right)}{1.9} - \frac{11101}{2760} - 0.95} \]

\[ c_L = 0.984 \]

\[ c_L = 1.0 \text{ (assumed)... verify later} \]

\[ \text{USE } c_L \]

\[ F_1 = 2940(0.984) = 2893 \text{ psi } > 2360 \text{ psi}. \checkmark \]

Check Compression

\[ f_c = \frac{6000(1000)}{170.6 \text{ in}^2} = 331.9 \text{ psi} \]

\[ F_1 = 6000 \times \frac{1}{1.0} \times \frac{1.0}{1.0} = 2062.5 \text{ psi } > 331.9 \text{ psi } \checkmark \]

Radial Stress

\[ \text{Verify later} \]

Shear Stress

\[ F_s = F_v \cdot G \cdot \frac{c}{t} \quad F_w = (265 \text{ psi}) (1.25) = 331.3 \text{ psi} \]

\[ F_v = \frac{1}{2} \left( \frac{54(100)}{170.6 \text{ in}^2} \right) = 132 \text{ psi } < 331 \text{ psi} \checkmark \]

\[ \text{Only works if add reinf} \]
If \( A = 170.6 \text{ in}^2 \), equivalent circular cross section:

\[ s = \frac{d^2}{6} = \frac{15^2}{6} = 56.25 > 55.75 \checkmark \text{ in}^2 \]

38” + 7.5” (c) = 4.4’,

ROUND TO 4’-4”

\[ C = 120” \Rightarrow \theta = \frac{120}{7.5} = 16\pi \]

38” + 15” = 53” (≈ 4.5’)

\[ \therefore C = 53” (\pi) \]

\[ = 167” \ (≈ 14’) \]
**DONUT MBR ESTIMATES**

**COLUMNS & BEAM (ALL CANTILEVERS)**

- 24F-E4 (psi)
  - Fe = 2400
  - FeLx = 1450
  - FeLx = 600
  - Fux = 205
  - Exmp = 1.35E0

**DEMANDS**

- MBR 1: V: 24k
  - M: 25.5 kip
  - N: 2K

- MBR 2: V: 6.8 k
  - M: 25.5 kip
  - N: 4.5 k

- MBR 3: V: 6.8 k
  - M: 5.4 kip

**SHEAR**

- 14 BAYS
- 6” WIDE DONUT PATHWAY

**CHECK BUCKLING**

- MBR 1
  - \( P_{cr} = \frac{\pi^2 E}{(kL)^2} \)
  - \( \sigma = \frac{P_{cr}}{A} \)
  - \( A = 4\times4 = 6.15 \text{ in}^2 \)
  - \( 2\times2, \text{ use } 4\times4 \)

- MBR 2
  - \( S = \frac{2.5 \times 1000 \times 12}{2400} = 127.5 \text{ in}^2 \)
  - \( \text{try } 5\frac{1}{8}'' \times 13\frac{1}{8}'' \) (Say 5 1/8” x 6” glue or 5 1/8”x5 1/2”)

- MBR 3
  - SHEAR
  - MIGHT GOVERN BUT NELM SIZE BASE ON M
  - \( S = \frac{5.4 \times 12 \times 1000}{2400} = 27.0 \text{ in}^2 \)
  - \( \text{try } 5\frac{1}{8}'' \times 4'' \) (Say 20.76 in²)
Air-Bnb Calcs

Load Takeoff

DL
Sub-Purlins
Purlins
Girders
4' x 12'
6' x 6'
4' x 4'
4' x 6'
Pond Load

LL
20 psf

Key Plan - Air-Bnb's

Try DF-L Select Structural 2x4 at 24" oc

Check Bending

\[
f_b = \frac{M}{S} = \frac{17200 \times 4}{3.06 \text{ in}^2} = 2553 \text{ ft-lb}
\]

\[
F'_b = F_b \times C_b \times C_n \times C_l \times C_f \times C_u \times C_t
\]

\[
= 1500 \times 1.0 \times 1.0 \times 1.5 \times 1.0 \times 1.15 = 2588 \text{ in-lb}
\]

\[
D/C = 0.91
\]

Check Deflection

By inspection, L only governs

\[
\Delta = \frac{5wL^4}{384EI} = \frac{5.24 \times 10^3 \times 1228}{384 \times 1100000 \times 5.359} = 0.508"\]

Per 18C 1604.3,

\[
\Delta_{allow} = \frac{L}{240} = \frac{10.24}{240} = 0.50"\]

\[
D/C = 1.02 \approx 1.0 \checkmark
\]

Check Shear

\[
f_v = 1.5 \frac{f_a}{A} = 1.5 \frac{240}{5.25} = 68.6 \text{ ft-lb}
\]

\[
F'_v = F_v \times C_b \times C_n \times C_l \times C_t
\]

\[
= 110 \times 1.0 \times 1.0 \times 1.0 \times 1.0 = 110 \text{ in-lb}
\]

\[
D/C = 0.38 \leq 1.0 \checkmark
\]

USE DF-L Sel. Structural 2x4 at 24" oc
Purlins

Try DF-L Select structural 4x10

Check Bending
\[ f_b = \frac{M}{I} = 8.7750 \frac{\text{in}^2}{\text{in}^4} = 1758.2 \text{ psi} \]
\[ F_b = F_v C_d C_n C_t C_r C_f C_r \]
\[ = 1500 \cdot 1.0 \cdot 0.5 \cdot 1.0 \cdot 1.0 \cdot 1.0 \cdot 1.0 = 1500 \text{ psi} \]
\[ f / f_b = 0.98 \checkmark \]

Check Deflection
\[ \Delta_{actual} = \frac{5L^4}{384EI} = \frac{5 \cdot 240 \cdot 13}{384 \cdot 1900000 \cdot 233.8} = 0.67" \]
\[ \Delta_{allow} = \frac{L^2}{240} = \frac{13^2}{240} = 0.75" \]
\[ \frac{\Delta}{\Delta_{allow}} = 0.87 \checkmark \]

Check Shear
\[ f_v = 1.5 \frac{V}{A} = 1.5 \cdot \frac{180}{32.31} = 90.3 \text{ psi} \]
\[ F_v = F_v C_d C_n C_t C_r \]
\[ = 180 \text{ psi} \]
\[ f / f_b = 0.50 \checkmark \]

USE Select Structural DF-L 4x10
Girders

<table>
<thead>
<tr>
<th>1900</th>
<th>2400</th>
</tr>
</thead>
<tbody>
<tr>
<td>1215</td>
<td>1215</td>
</tr>
</tbody>
</table>

\[ 1215 \times 975 = 110700 \text{ N}\cdot\text{m} \]

\[ \text{Moment} \]

Try DF-L #1 6 x 12

Check Bending

\[ f_b = M / S = \frac{151400}{1212} = -1084 \text{ psi} \]

\[ F_b = 1350 \times 1.0 \times 1.0 \times 1.0 \times 1.0 = 1350 \text{ psi} \]

\[ \text{D/C} = 0.80 \]

Check Deflection

\[ \Delta_{\text{actual}} = \frac{6wl^2}{384EI} + \frac{pl^3}{48EI} = \frac{6 \times 24 \times 20^2 \times 1778}{384 \times 160000 \times 697.1} + \frac{1950 \times 20^2 \times 147}{48 \times 160000 \times 697.1} = 0.12'' \]

\[ \Delta_{\text{allow}} = \frac{8}{240} = 1'' \]

\[ \text{D/C} = 0.12 \]

Check Shear

\[ f_v = 1.5 \times \frac{12.15}{63.75} = 28.8 \text{ psi} \]

\[ F_v = 170 \text{ psi} \]

\[ \text{D/C} = 0.17 \sqrt{\checkmark} \]

USE DF-L #1 6 x 12
Sensory Room Cales

Load Takeoff

\[ \frac{DL}{Joists} + \frac{Girders}{6 \text{ psf} + 11 \text{ psf}} \]

\[ \frac{LL}{100 \text{ sf}^2} \text{ (green roof)} \]

Girders - G1

1387.5 SF

31219 lb

45 ft

31219 lb

31219 lb

Moment

\[ M_{\text{max}} = \frac{wL^2}{8} = 4,214 \text{ k-in} \]

Try 2 1/2" x 36 DF/DF

Check Deflection

\[ \Delta_{\text{act}} = \frac{5wL^4}{384EI} = \frac{5 \times 1387.5 \times 45^4 \times 1728}{384 \times 1.2 \times 10^5 \times 41,800} = 1.70" \]

\[ D_{\text{act}} = \frac{5L^2}{240} = \frac{45^2}{240} = 2.25" \]

D/C = 0.76

Check Bending

\[ f_b = \frac{M}{S} = \frac{4,214,000}{2,322} = 1,819 \text{ psi} \]

\[ f_b = 2,400 \times 1.2 \times 1.0 \times 0.77 \times 1.0 \times 1.0 \times 1850 \text{ psi} \]

D/C = 0.98

Shear is adequate per inspection

Use 2 1/2" x 36 DF/DF
Joints - J-1

\[
\frac{212}{16} \quad \frac{25}{\text{L}}
\]

\[
2650 \quad \frac{19875}{\text{in}}
\]

Try 24F-14 6 3/4 x 12 DF/DF

Check Bending

\[
\begin{align*}
P_b &= \frac{19875}{16} = 1233 \text{ psi} \\
F &= 2400 \cdot 1.0 \cdot 1.0 = 2400 \\
F &= 0.7 \cdot 1.0 \cdot 1.0 = 2325 \\
C_v &= \left(\frac{21}{25}\right)^{1/2} \left(\frac{172}{16}\right)^{1/2} \left(\frac{5.125}{6.75}\right)^{1/2} \\
D/C &= 0.53 \leq 1.0 \\
D/C &= 0.97
\end{align*}
\]

Check Deflection

\[
\begin{align*}
\Delta_{\text{actual}} &= \frac{5 \cdot 212 \cdot 25 \cdot 1729}{384 \cdot 1.8 \cdot 10^4 \cdot 972^2} = 1.06\" \\
\Delta_{\text{allow}} &= \frac{1.25}{240} = 0.01\" \\
D/C &= 0.85 \leq 1.0 \\
D/C &= 0.85 \leq 1.0
\end{align*}
\]

Check Shear

\[
\begin{align*}
P_v &= 1.5 \cdot \frac{2650}{81} = 49 \text{ psi} \\
1' &= 2650 \text{ psi} \\
D/C &= 0.11 \leq 1.0
\end{align*}
\]

Use 24F-14 6 3/4 x 12 DF/DF.