Mt. Carmel Canopy Structure Report

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California Polytechnic State University, SLO
June 24, 2018
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Introduction

Purpose

To design and construct a new entry canopy for Mt. Carmel Lutheran Church using architectural and structural design concepts, along with determining the construction process and providing fabrication drawings for construction.

Existing Conditions

The old wood structure was removed several years ago, approximately five years ago fabric sails were installed as a temporary fix. The current shade installation consists of four different colored fabric triangles that are connected with steel rods on top of the existing columns.

Project Description

The client (Mt Carmel Church) wanted a durable design and low maintenance solution. Moreover, they want a new design that is more visually appealing. Therefore, our group from Cal Poly took this mission. We met with a representative of the church to get an idea of the project goals and then created design concepts and models to show the church, allowing them to pick the idea they favored. Once a design was chosen, the group performed structural calculations to determine member sizes and connections. Along with calculations, we completed a set of structural drawings. The group then contacted various material companies to procure the necessary materials at the lowest possible cost to the client. Finally the group created and compiled construction documents for future construction of the structure.
Timeline

Winter Quarter 2018

Weeks 1-6

The group met with church representative John Cook on January 6th, and discussed what the desired end product was. Discussed was the importance of the architectural aesthetic, specifically to avoid an industrial-looking installation. Also, the importance of shade, cost, upkeep, and possible rain protection and collection. We also took onsite measurements to accurately create a layout for the design. After taking input from the church, we came up with three initial designs, and crafted \(\frac{1}{4}''=1'-0''\) scale models of them along with a site model. We brought all of the models to John on February 13th which he took to present in front of the church council and allow them to pick a design they favored.

Weeks 7-8

While waiting for the church's decision on which design they wanted to go forward with, we started initial wind load calculations and deflection limits. Also during this time we started to get cost estimates on the roofing material from Home Depot, and on preliminary steel framing from B&B Steel. After getting feedback on which design they liked best, we started to determine lengths of members.

Week 9

The church contacted us and we discovered an architect at the church had previously created a design for a new entrance canopy several years ago. We met with the architect on March 6th and discussed his idea and looked at conceptual drawings. He wanted to know whether we would be willing to work with him on his design. The concept he created was abstract and still in the design process, so we decided that it would be best for our project to continue with our design and create a design proposal with a cost estimate for the church. We thought an organized design with some full scale mock-ups of the connections would be beneficial to them if they ever decided to build it in the future. We gave the church our decision to not pursue the architect’s design on March 9th.

Week 10

We determined the final design to work with and started creating a final model with set heights and angles for the structure. We also started to get a more in depth cost estimate now that we had set lengths to evaluate.
Spring Quarter 2018

Week 1
The group went out to the site to get more accurate dimensions for our connection design. Later we received an email from the church saying that they wanted us to proceed with our design.

Weeks 2-3
We contacted the church and found out they wanted us to present our design in front of the church community on April 15th and then in front of the church committee on April 19th. The architect from the church also presented his idea and based off of the presentations the church would decide which design to pursue. We presented two different versions of our idea, one that consisted of four triangles and one with three at the first meeting and determined they preferred the three triangle system. We then presented the three triangle system to the church committee. Later we heard back from the church on April 20th and they let us know that they wanted to pursue our design.

Weeks 4-7
We started to finalize the calculations. First determining a decking size that met the properties we required, specifically being able to span 17’ to prevent interior framing. We were able to get it donated by Verco Decking. The estimated ready date for pick up was Tuesday, May 22nd. When designing the framing, the initial design using HSS5x3 sizes did not work due to deflection limits. We determined HSS8x4 sizes would work. After a meeting with John on May 12th to discuss the new cost for the project because the member size changed and the fact that we would need to rent a crane to lift the triangles into place during construction, we got the go ahead to continue. On May 15th, we received a suggestion from Kevin Dong, our advisor, to see if we could get our longest member (40’ long) to span without a support because it was creating connection and construction design issues. After determining a new size that worked for that condition we increased all our sizes and got new cost estimates for HSS10x3 and HSS10x4 members to see which was more economical. Quotes were requested on May 18th and received on May 23rd. We decided to use HSS10x4 and adjust our details to account for the size increase. On May 20th contacted the Berridge Manufacturing Co. to see if we could get roofing donated.

Week 8
On May 21st we turned in our calculation package and drawing set to Kevin for review. We also contacted John with a final steel quote for the updated member sizes on May 23rd and John sent a check to B&B Steel on May 25th to order the framing.
Kevin reviewed our drawings and left comments for us to pick up. Went into the CAED shop and started to practice welding.

**Week 9**

We returned the drawing set for review again and got more accurate measurements of the site to fine-tune our details. On May 29th we picked up steel decking from Verco in Fontana, California. On May 31st we received confirmation from B&B steel that our order was processed and the steel will be on its way and should arrive by June 5th. Went out on June 2nd and got more exact measurement locations of the existing beam seats.

**Week 10**

Steel arrived on June 6th. Met with people with fabrication experience to create a timeline of what needs to happen to finish. Set up plasma cutting location. Gave presentation of senior project on June 7th. Met with Agricultural metal shop to see if we could use their facilities to cut the material. Still in negotiations with them. Confirmed on June 5th that we could get the roofing donated. Mocked up the lengths of each piece of roofing and sent to the company. Contacted companies about prices for renting a generator.

**Finals Week and Summer**

Created a cut sheet/shop drawings for the steel. Complete remaining schedule as follows: fabricate steel members, transport steel members, weld members on site, place members on existing structure, procure a crane and generator for construction purposes, pick up roofing material from Berridge Manufacturing Co., complete structure. Be ready to follow contingencies if fabrication and construction can not be finished within the timeline. Began to implement contingency to pass on the project to another group.
Initial Design
Study models with scale: ¼” = 1’

Initial Models: Design 1
This design involves removing one column in order to create a more open and inviting space.

Initial Models: Design 2
This design utilized all six columns and framed two pathways into the church. One from the parking lot and one from the daycare center.
Initial Models: Design 3

This design had four triangles angled up and pointing to the church with the purpose of leading the eye to the entrance.
Final Design

The final design for the entry structure involved the use of three triangles with peaks that formed a curve turning into the church entrance. The peaks increased in height as they approach the entry. The church committee favored this design because of the concept of three and the meaning of Holy Trinity as well as the purpose of leading the eye to the entrance.

Revit model
Physical Models
Study models with scale: ¾" = 1'
Sun Path Study

During the Spring, the structure provides shade to the window until 3 pm. The window is then exposed to the sunlight until 4 pm when the adjacent building begins to provide shade.

During the Summer, due to the sun’s overhead position, the shade provided over the service window decreases relative to Spring. However, the window receives shade during lunch hours when it is operational.
During the Fall, the shade provided by the structure is similar to that of Spring. Although it is not as effective before noon. There is still an hour between 3 and 4 pm that the service window is exposed to the sun.

During the Winter, the structure provides shade to cover the service window. The service window is exposed to the sun from 2 to 4 pm.
Rain Collection Study

The slope of the triangles help collect water to one side of the triangle. At that point, the height of the triangle connections help move the water to the corners. However the structure itself isn’t sufficient to prevent water from falling onto people walking underneath, therefore a gutter system should be implemented for better performance. Arrow represents the slope and water flow direction.
Framing Layout

Plan View

Elevation View

Detail drawings see appendix C.
Connections

Study models with scale: 1” = 1’

Detail drawings see appendix C.
Building Codes and Design References
- IBC 2015 - International Building Codes
- ASCE 7-10 - Minimum Design Loads for Buildings and other Structures
- AISC 360-10 - Steel Construction Manual

Structural Calculations Package
- See appendix B

Structural Drawings
- See appendix C
Materials

HSS Tubing
- HSS 10x4x3/16 and 5/16 Framing

Metal Deck
- 3" Corrugated Metal Deck (Verco - W3 18 ga)

Standing Seam Roofing
- Cee-lock Panel (Forrest Green)

Painting
- Church will select the paint color.
Cost

Client (church) Paid Items

Total estimate cost: $6166.80 + paint (varies)

Framing*

- Supplier: B&B Steel & Supply

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<th>Quantity</th>
<th>Extended Price</th>
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<td>11</td>
<td>$4070.00</td>
</tr>
<tr>
<td>10x4x0.312</td>
<td>$33.0 /FT</td>
<td>2</td>
<td>$1320.00</td>
</tr>
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<td>6x6x0.188</td>
<td>$16.0 /FT</td>
<td>1</td>
<td>$320.00</td>
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<td>8% Tax</td>
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<td></td>
<td>$456.80</td>
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<tr>
<td>TOTAL</td>
<td></td>
<td></td>
<td>$6166.80</td>
</tr>
</tbody>
</table>

Painting

- Selected by the church

Crane

- Local supplier: $466/day for a 4 ton crane

Donated Items

Decking* (Estimated at $4000)

- Donated by Verco Decking Company

Roofing (Estimated at $1500)

- Donated by Berridge Manufacturing Co.

Transportation

- Framing was delivered free of charge.
- Decking transportation was supported by another group from Cal Poly.
- Roofing transportation will be picked up from Fontana, CA via Uhaul.

*See appendix A for receipts
Construction

On Campus / Shop Fabrication

Acquiring Materials
- Verco - W3 18 ga (Donated)
- B&B Steel - HSS 10x4x3/16 and 5/16
- Berridge Manufacturing Co. - Roofing (Donated)
- Steel Plate - Leftover

Fabricating Connections
- Cut out angles from steel plate
- Weld steel plates to fabricate angles

Cutting Members and Decking
- Cutting in shop

Preparing Columns
- Welding angles to sides of columns

On Site Sequencing

Site Preparation
- Removing existing structure
- Grinding down beam seats
- Weld base plate atop beam seat

Framing Triangles
- Welding members together to form triangles

Placing Columns
- Weld columns down to base plates

Placing Triangles
- Determine triangle pick points
- Lift and set down triangles with a crane onto angles
- Weld into place

Screw Decking
- Lap and join with steel metal screws

Install Roofing
- Per installation instructions
Challenges

- Communication issues amongst the group created initial confusion
- Client time delays for design decisions which prolonged the schedule
- An architect intervened during the design process which brought indecision in the church
- Selecting appropriate modeling structure in RISA to match hand calculations
- Transportation of materials was difficult given the weight and size
- Inexperienced in welding and unavailability of proper training required professional welding help
- Lack of necessary construction equipment available from our department added additional costs
- Complexity in construction, such as using other department’s resources to fabricate members, created delays in the project planning

Lessons Learned

- Working with a real client can be difficult.
- Issues may arise sometimes that are out of your control.
- Get multiple opinions before making decisions and progressing with the project.
- If there are concerns voice them with solutions before it’s too late.
- Delegate tasks and set deadlines.

Ways to Improve

- Always keep in contact with the client and keep them up-to-date.
- Try to be as prepared as possible for the unexpected
- When concerns arise, come up with multiple plausible solutions to present
- Reconvene when tasks are completed to keep everyone up-to-date.
# Appendix A - Material Receipts

## Framing Receipt

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<tr>
<td>Quote #: 5554</td>
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<tr>
<td>Date: 05/22/18</td>
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<td>Page: 1</td>
</tr>
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**To:** 300000  
**Ship to:** CASH SALES (CHECKS & CASH), Vince & Cal poly

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<tr>
<th>Terms</th>
<th>Quote Expiration Date</th>
<th>Ship Terms</th>
<th>Quoted By</th>
<th></th>
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<tr>
<td>C.O.D.</td>
<td>05/25/18</td>
<td>WILL CALL</td>
<td>TIM GUIDOTTI</td>
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<td>WEIGHT: 3758</td>
<td>18.5000FT</td>
<td>4070.00</td>
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<tr>
<td>2 PC</td>
<td>10 x 4 x .312 RECT TUBE 20'</td>
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<td>33.0000FT</td>
<td>1320.00</td>
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<tr>
<td>1 PC</td>
<td>6 x 6 x .188 H.S.T. 20'</td>
<td>WEIGHT: 291</td>
<td>16.0000FT</td>
<td>320.00</td>
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- Material subject to prior sale.
- Quantity changes may cause change in pricing.
- Please review for errors or omissions.

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Decking Receipt

Nucor
Original Bill of Lading - Non Negotiable
Freight Charges are Pre-paid, Shippers Load and Count Collected.

Verco Decking, Inc. - Fontana
8333 Lime Street - Fontana CA 92335

Bill of Lading #: 600451939-0001

Combined Straight Bill of Lading

Deliveries

5006682696

Page 1 of 3

Shipping Terms: Pick up
Ship Date: May 17, 2018
Carrier: Customer Pickup - Verco
Trailer Number: CPU1

Arrival Date: 05/12/18
Driver Signature:

Mileage
Heaviest Bdl: 1,503 Tons
Drop Weight: 4,314 Lbs

05/12/18

Job No: 086-18-0004 PO# CAL POLY Cust No: 15544

Sold To: VERCO-FREMONT
FREMONT, CA

Ship To: CAL POLY DONATION
CPU - FONTANA
FONTANA, CA 92335

Phone: Phone:

2 BUNDLES

Bundle Number Job/Sequence List Description Longest Length Job Zone/Area Mark Number Qty/Units Type Finish

12071790 086-18-0004-5001D 1D1 W36 FL 18’/G60/ NONE/NONE 8’-00 0’-0"

Total: 4.80 SQS

12071801 086-18-0004-5001D 1D1 W36 FL 18’/G60/ NONE/NONE 18’-00 0’-0"

Total: 11.04 SQS

Scheduled Arrival Date Notice to Receivers: Please check each item on bill carefully. Verco will not be responsible for any shortages unless they are noted on this document. Please follow the handling recommendations on the last page.

00/00/0000

Received By

Other Products

Other Products Pieces

Joist Pieces

Joist Bundles

Bridging Bundles

Deck Bundles

Joist Sub Bundles

Accessory Bundles

0 0 0 0 0 2 0 0

This job is: Complete - Incomplete

Subject to Section 7 of applicable Bill of Lading. If this shipment is delivered to the consignee without recourse on the consignor, the consignor shall sign the following statement: “The carrier shall not make a delivery on the shipment without payment of freight and all other lawful charges.”

Signature: Nucor

WARNING! WARNING! WARNING! WARNING! WARNING!

Joists are unstable during erection. Under no circumstances are construction loads of any description to be placed on unbraced joists. Any erector who allows this to be done is in direct violation of OSHA regulations, creates significant risk of serious injury to person and property, and may be held liable for any injuries sustained. Erector is responsible for following all instructions on drawing erection notes and on the last page of this delivery ticket. Forward these sheets to the responsible steel erector.

ATTENTION: The Occupational Safety and Health Administration’s (OSHA) Hazard Communication Standard (29 CFR 1910.1200) requires manufacturers to supply their customers with SDSs for certain products. Section 313 of the Emergency Planning and Community Right-to-Know Act (“EPCRA”) also requires notification that our products may contain reportable chemicals listed under EPCRA Section 313.

Please visit http://nucor.com/products/products/matrix/ to obtain your copy of this information. If you wish to be mailed a paper copy, please notify us immediately at 8333 Lime Street - Fontana CA 92335, or call (909) 822-8079.

Please visit http://nucor.com/products/products/matrix/ as our SDS sheets have been updated as of 12/29/2017.

Rev. 04/23/2018
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<th>TYPE</th>
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Total Squares = 15.840  Total Pounds = 4314  Total Feet = 528.000  Total Bundles = 2
Total Die Sets = 0.000  Total Short Sheets = 0  Total Super Short Sheets = 0

* Clean and Pre-Treat *

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41 Pieces
* 57 FT CUTOFF AT 3 PIECES (216 IN) ON BUNDLE & 2 RUN OUT 60 IN *

SAFETY * QUALITY * PRODUCTIVITY
Appendix B - Structural Calculations Packages
Mt. Carmel Canopy Structure Structural Calculations

By:

Kyle Chase
Ricardo Gustavson
Jiaming Liu
Max Snook

California Polytechnic State University
San Luis Obispo, CA
6/21/18
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<table>
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<td>Decking</td>
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<td>Existing Footing Check</td>
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<td>Appendix: IBC Deflection Limits</td>
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CHASE 7-10 Wind

Risk Category I
- 100 mph
- k1 = 0.5
- Exposure Category B
  - k2 = 0.7
- G = 1.0

\[ q_2 = 0.00256 \times k_2 \times k_1 \times V^2 = 0.00256 \times (0.7)(1)(1.05)(100)^2 = 15.2 \text{ PSF} \]

\[ CN = 1.8 \text{ Fig 30.5-1} \]

\[ P = CN \times q_2 \times G = 1.8 \times (15.2)(1.05) \]

\[ P = 23.3 \text{ PSF} \]

Roof Live
- 20 PSF

ASCE 7-10 Table 4.1

Dead
- Roof Finish: 1 PSF
- Decking: 3.1 PSF
- Framing: 5 PSF

Total 9.1 PSF
DECKING

VERLO PLN5
18 GA MAX SPAN 17'

1/80 CAPACITY 22 PSF

\[ \Delta_1 = 20 \text{ PSF} < 22 \text{ PSF} \checkmark \]

\[ \Delta_2 = .12 (23.3) < 22 \text{ PSF} \checkmark \]

IBC 2015 1004.3

46 PSF

STRESS

\[ D + L + .6W \]

\[ S + 20 + .6(23.3) < 46 \text{ PSF} \checkmark \]

\[ = 38.9 \text{ PSF} < 46 \text{ PSF} \checkmark \]
# Type PLN3™ or HSN3™

## Allowable Uniform Loads (psf)

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</tr>
<tr>
<td>16</td>
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</tbody>
</table>

See footnotes on page 81.

www.vercodeck.com

VERCO DECKING, INC.

VR4  =  83
\[ \Delta = \frac{1}{180} \cdot 1142'' \]

\[ L = \frac{W}{60 \cdot EI} \]

\[ = \frac{(0.42)(17.135)(17.135 \times 12)^3}{60 \cdot 24,000 \cdot 33.1} \approx 0.076'' < 1.142'' \]

\[ L = \frac{(0.20 \times 17.135)(17.135 \times 12)^2}{60 \cdot 24,000 \cdot 33.1} \approx 0.1552'' < 1.142'' \]

\[ \Delta = \frac{W}{60 \cdot EI} \text{ assumes highest pt. is in the center of the member most conservative.} \]

\[ W = \frac{W}{2} \]
\( g, \frac{E}{A} = -2 \cdot 15 + 0.918(3.50 \times 1/2 (3/8) \times 3.28) + 0.2182 (11/12) (3.58 + 11/12 (1/8)) \)

\( E = 9.18 \times 10^6 \)
\( A = 1.281 \)

\( \text{Deflection} \text{ (8 x 4 x 3/16)} \)

\[ \Delta = \frac{Wl^3}{6EI} = \frac{(0.058)(15)(16 \times 2)^3}{60(29000)(33.1)} \approx 0.045 < 1'' \]

\[ L \Delta = \frac{(0.120 \times 15)(15 \times 12)}{60(29000)(33.1)} = 0.0918 < 1'' \]
\[ \text{Member 3} \]

\[ \begin{align*}
&-k_1 (22.24) + 11.12 (29.32/2)(1/2) (11.12 + 29.32) (11.12 + 1/4 (11.12)) \\
&L_1 = 1.63' \\
&L_2 = 1.63' \\
&\text{Options} \\
&\text{AISC 341} \\
&\text{Table 3-12} \\
&\text{HSS 8x4x3/16} \\
&f M_n = 33.3'' \\
&I = 33.1 in^4 \\
&\text{HSS 10x3x3/16} \\
&f M_n = 44.3'' \\
&I = 49.4 in^4 \\
&\text{HSS 10x4x3/16} \\
&f M_n = 60.2'' \\
&I = 57.8 in^4 \\
\end{align*} \]

\[ \begin{align*}
\text{Deflection Ck} &\quad (6x4x3/16) \\
\Delta &= L/H_0 = 22.24/12/180 = 0.148'' \\
\text{AISC 341} \\
\text{Table 3-23} \\
\Delta &= W/L^3 \\
\frac{60EI}{12} &= (0.586 \times 12.24)^2 \times (22.24 \times 12)^3 \\
&= 2.1 / 0.29000 / 33.1) \\
\Delta &= 0.120 \times (22.24) \\
&= 0.29000 / (33.1) \\
&= 0.41 / 1.18'' \\
&\text{Valid} \\
\end{align*} \]
Deflection calculation:

\[ \Delta = \frac{1}{150} \left( \frac{10 \times 3 \times 5}{16} \right) = \frac{1}{150} \times \frac{75}{16} = 0.4875 \text{ in} \]

\[ \Delta = \frac{W L^3}{60 E I} = \frac{(0.0695 \times 40)^3}{60 (29000)(17.63)} = 1.14" < 2.067" \]

\[ \Delta = \frac{0.140 \times (40)^3}{60 (29000)(76.3)} = 2.33" < 2.067" \]
Mt. Carmel Canopy Structure

**Membrane 2**

- **23.05** ft
- **347.5 PFL**

From Pge 8

\[ GZ = \frac{-2.07}{L_1} + \frac{0.3475}{L_2} \left( \frac{6}{12} \right) \left( \frac{6}{12} \right) + \frac{0.3475}{L_2} \left( \frac{23.05}{12} \right) \left( \frac{6 + 23.05}{12} \right) \]

**Options**

- HSS 8x1x 9/36
  - I = 56.7
  - dM = 64.7

- HSS 10x3x3/16
  - I = 49.1
  - dM = 49.1

- HSS 10x4x3/16
  - I = 57.6
  - dM = 50.2

**AISC 341**

**Table 3-12**

**Deflection (MI)**

\[ \Delta = \frac{M}{180} \cdot \frac{L}{180} = 29.05 \times 12 / 180 = 1.99 \text{ in} \]

**AISC 341**

**Table 3-23**

\[ \Delta = \frac{wL^4}{60EI} = \frac{0.068 \times 29.05 \left( 29.05 \times 12 \right)^3}{60 \left( 29.05 \right) \left( 4.9^{+} \right)} = 0.544^{+} < 1.11^{+} \]

\[ \Delta = \frac{1040 \times 29.05 \left( 29.05 \times 12 \right)^3}{60 \left( 29.05 \right) \left( 4.9^{+} \right)} = 1.12^{+} < 1.99^{+} \]
\[ \Delta = \frac{1}{180} \times 21.718 \times 12 \times 1.45 = 1.45'' \]

\[ \Delta = \frac{W \Delta L^3}{120 EI} = \frac{(0.06 \times 21.718)^3}{120 \times 60 \times 29000 \times 33.1} = 2.22'' < 1.45'' \]

\[ \Delta = \frac{(140 \times 21.718)^3}{120 \times 29000 \times 33.1} = 4.67'' < 1.45'' \]
MT. CARMEL CANOPY STRUCTURE

Triangle 3

\[ DL = 5 \text{ PSF} \]
\[ RL = 20 \text{ PSF} \]
\[ WL = 23.3 \text{ PSF} \]

Trib. Width = 7.33'

\[ DL = 37 \text{ PLF} \]
\[ RL = 147 \text{ PLF} \]
\[ WL = 171 \text{ PLF} \]

Long Compo
\[ 1.2D + 1.62L + 0.5W \]
\[ = 3177 \text{ PLF} \]

Point Load
\[ 1.2(18.16) + 1.6(18.01) + 0.5(1.81) \]
\[ = 19.89 \]

\[ \sum C = -2,122.5 + (3.77)(22.45/6)(3/22.45) + 1.9(37.70) + 1.377(15.123/2)(22.67 + 15.123/2) \]

\[ = 18,883 \]

OPTIONS

HSS 8x4x3/16
\[ I = 39.7 \text{ in}^4 \]
\[ M_n = 6.9 \text{ k-in} \]

HSS 10x3x3/16
\[ I = 49.4 \text{ in}^4 \]
\[ M_n = 44.3 \text{ k-in} \]

HSS 10x4x3/16
\[ I = 51.8 \text{ in}^4 \]
\[ M_n = 44.3 \text{ k-in} \]

Deflection

\[ \Delta = \frac{Wx}{24EI} \left( \frac{4h^2 L}{L^3} + \frac{6a^2 x}{L^3} - \frac{4a x}{L^3} - \frac{2x^3}{L^3} \right) + \frac{F_x}{6EI} \left( L + a \right) \]

\[ = 1.54'' < 1.67'' \]
\[ E = -2 \times (29.82) + 0.377 (22.62/2) (9/8) (22.62) + 7.377 (7.2) (22.62 + 72/12) \]

\[ E = 3,32 \]

\[ M = 27.91 \]

\[ V = 2.32 \]

**Deflection Check (10x3\times3/16)**

\[ \Delta = \frac{9}{180} = \frac{29.82 \times 2}{180} = 1.98'' \]

\[ 42W = \frac{WL^3}{60EI} = \frac{0.072 (29.82) (29.82 \times 12)^3}{60 (29000) (49.4)} = 572 \text{ in} < 1.98'' \]

\[ L = \frac{(147 \times 29.82)}{2} (29.82 \times 12)^3 \frac{1}{60 (29000) (49.4)} = 1.16'' = 1988''/ \]

**Options**

- HSS 8\times4\times3/8
  - \( M_o = 63.7 \text{ kip-ft} \)
  - \( I = 581.7 \text{ in}^4 \)

- HSS 10\times3\times3/16
  - \( M_o = 44.9 \text{ kip-ft} \)
  - \( I = 49.9 \text{ in}^4 \)

- HSS 10\times4\times3/16
  - \( M_o = 44.9 \text{ kip-ft} \)
  - \( I = 51.8 \text{ in}^4 \)

**AISC 360**

**Table 3.23**
\[ \gamma \Sigma = -R_1 (22.41) + 0.377 \left( \frac{15.167}{2} \right) (0.3) (15.167) + 0.377 (7.24/2) (15.167 + 72 + 72)(1/3) \]

\[ R_1 = 2.36^\circ \]
\[ R_2 = 1.86^\circ \]

**Option 1**

HSS 8 x 4 x 3/16

\[ \phi M_x = 353 k \text{ in. lb} \]
\[ I = 3314 \text{ in.}^4 \]

HSS 10 x 3 x 3/16

\[ \phi M_x = 414 k \text{ in. lb} \]
\[ I = 4794 \text{ in.}^4 \]

HSS 10 x 4 x 3/16

\[ \phi M_x = 443 k \text{ in. lb} \]
\[ I = 5738 \text{ in.}^4 \]

**Option 2**

Axle 341

Deflection \( \Delta \)

\[ \gamma_\epsilon = \Delta \]

\[ \Delta = 12 (22.41)/100 = 1.494" \]

\[ \frac{42W}{60EI} = \frac{0.072 x 22.41}{60 (29000) (33.1)} = 1.272" < 1.494" \]

\[ L = \frac{0.147 x 22.41}{60 (29000) (33.1)} < 1.494" \]

Axle 341

Table 3-12
CONNECTIONS

REACTIONS

\[
\begin{align*}
A &= 1.35k + 1.281k = 2.631k \\
B1 &= 1.13k + 1.03k = 2.16k \\
B2 &= 3.69k + 1.6k = 5.29k \\
C &= 0.918 + 1.03k = 2.55k \\
D1 &= 3.18k + 2.07k = 5.25k \\
D2 &= 3.8k + 2.36k = 5.66k \\
E1 &= 3.12k + 2.22k = 5.32k \\
E2 &= 0.12k + 2.32k = 2.44k 
\end{align*}
\]

Governed vertically

WELD CAPACITY

\[
\phi = 0.75 \\
F_{exx} = 60^\text{psi}
\]

\[
\phi R_n = C, D1
\]

D3 \rightarrow 3/16

\[
D_n = 16.4k > 5.16k
\]

\[
C_1 = 1.857 \\
D = 3 \\
L = 3'' \\
\phi = 1 \\
\phi = \phi x \frac{1}{3} = 1.25/3 \\
L = 2.84 \\
C = 2.84
\]
**Connection Angle**

**Max Force** 5.06 k @ D2

**Demand**

\[
\begin{align*}
\text{3"} & \\
\text{3"} & \\
\text{1/4"} & \\
\end{align*}
\]

**Capacity**

\[
\begin{align*}
\phi M_n &= F_j z \\
36 \times 0.1 \left( \frac{b^2}{4} \right) \\
9 \times 36 \times 0.1 \left( \frac{b (3/8\text{"})^2}{4} \right) &= 4.96k \text{"} \\
b &= 4.35\text{"} \text{ Min} \\
b &= 4.5\text{"}
\end{align*}
\]
\[ h_f = 12.8' \text{ or } 1.0' h_f \]

\[ P = 10.1' \text{ or } 1.1' \text{ CONSERVATIVE} \]

Depth:

- [Diagram]

\[ h = \frac{12.3 \times 6\sqrt{3}}{3/16} \]

Calculation:

- \[ P = 2.4' \text{ or } 2.4' \text{ CONSERVATION} \]
- \[ h = 2.4' \text{ or } 2.4' \text{ CONSERVATION} \]
- \[ C = 2.5' \text{ or } 2.5' \text{ CONSERVATION} \]
- \[ q = 2.7' \text{ or } 2.7' \text{ CONSERVATION} \]
- \[ A = 2.6' \text{ or } 2.6' \text{ CONSERVATION} \]

Conclusion:
Net Uplift of Structure

\[ \text{Area} = 900 \text{ ft}^2 \]
\[ \text{Wind} = 23.3 \text{ PSF} \]
\[ \text{Uplift} = 900 (23.3) = 20970 \text{ k} \]

Existing Column Weight

\[ \text{Conc.} = 150 \text{ PCF} \]
\[ \text{Dia.} = 2' \]
\[ \text{Ht.} = 8' \text{ on avg.} \]
\[ \text{Wt. of Column} = \pi (1^2) (8') (150) = 3770 \text{ k} \]
\[ \text{Wt. of Footing} = 3' \times 3' \times 1' \times 150 \text{ PCF} = 1350 \text{ k} \]
\[ \text{Total Weight of Col} = 5120 \text{ k} \]

6 Total Columns \times 5120 \text{ k} = 30720 \text{ k}

30720 \text{ k} > 20970 \text{ k}

Net Uplift is Ok
Existing Footing Check: Soil Pressure Load Combo (Dr. 73 + 75 (6N) + 75 LR)

\[ p = 6.71' + 3.77' = 10.48' \]

- Load from W 3.34' x 3.34' = 3.34' x 3.34' x .5 = 5.71'
- Load from C 3.34' x 3.34' x .5 = 9.25'
- Load from LR 3.34' x 3.34' x .5 = 11.35'

Total Load = 26.54'

- Footing Width = 8.11'
- Footing Height = 2.16'

- Footing Size: 8.11' x 2.16'
- Footing Thickness = 2.0'
- Footing Depth = 3.0'

- Footing Concrete: 1.500 psf
- Footing Reinforcement: 1.25 psi

\[ f_{c} = \frac{1}{2} (0.75 + 0.5) \]

- Factor of Safety = 3.23
- Eccentricity = 0.311

- Minimum Bearing Pressure:
  \[ f_{b,min} = \frac{p}{A_{ho}} \left[ 1 + \frac{e^{2}}{L^{2}} \right] \]

  \[ f_{b,min} = \frac{10.48}{9.5} \left[ 1 + \frac{0.311^{2}}{3} \right] \]

  \[ f_{b,min} = 1.87 \text{ ksf} \]

- Minimum Concrete Strength:
  \[ f_{c,min} = 0.45 \text{ ksf} \]

- Minimum Footing Depth:
  \[ d = 1.87 \text{ ksf} \]

- Minimum Footing Width:
  \[ w = 2.00 \text{ ksf} \]

- Minimum Footing Height:
  \[ h = 1.87 \text{ ksf} \]

- Footing Design:
  \[ W_{1} = \frac{171(9)}{2} \]

  \[ W_{2} = 0.75(6(1.41 + 0.973)) = 12.4 \text{ ksf} \]

- Footing Safety:
  \[ V_{s} = 1870 \text{ psf} \leq 2000 \text{ psf} \]
**Existing FTG, Check: Conk + Rein.**

\[ f_{ca} = \frac{10.91 + 3.71(1/2)}{P_{g16}} = \frac{15.43}{9} = 1.71 \text{ ksi} \]

**Shear**

\[ f_{ca} = 0.75 \left( \frac{2 \sqrt{PL}}{b_d} \right) \]
\[ = 0.75 \left( \frac{2 \sqrt{2500 \times 300 \times 9^3}}{9} \right) \]
\[ = 24.3 \text{ ksi} \]
\[ V_u = 1.71(3)(0.5) = 2.565 \text{ ksi} \]

\[ 24.3 \text{ ksi} > 2.565 \text{ ksi} \checkmark \]

**Bending**

\[ M_u = f_{ca} \left( \frac{b_d}{2} - \frac{d_1}{2} \right)^2 \left( \frac{v}{2} \right) = 1.71 \left( \frac{3}{2} - \frac{v}{2} \right)^2 \left( \frac{v}{2} \right) = 2.1375 \text{ ksi-ft} \]

\[ A_s = \frac{2.1375 \times 12}{0.9 \times 40} \left( \frac{12}{14} \right) = 0.009 \text{ in}^2/\text{ft} < 0.01 \text{ in}^2/\text{ft} \checkmark \]

**Provided**

**Shear + Bending**

\[ M_p = 7.25 \text{ ksi} \times 15.43 \times \Phi \]

\[ V_u = \left( \frac{3.3 \times 2.77}{2} + 2.77 \times 5 \right) \]
\[ = 4.55 \text{ ksi} \]
\[ N = \frac{P}{A_{so}} \left[ 1 + \left( \frac{d}{L} \right)^2 \right] = \frac{15.43}{944} \left[ 1 + \left( \frac{6.7}{3} \right)^2 \right] \]
\[ = 3.3 \text{ ksi} \times 11 \text{ ksi} \]

\[ f_{bc}^u = 3.3 \text{ ksi} \times 11 \text{ ksi} \]

\[ V_u = 24.3 \text{ ksi} > 4.55 \text{ ksi} \checkmark \]

**Moment**

\[ M_u = f_{bc} \left( \frac{b_d}{2} - \frac{d_1}{2} \right)^2 \left( \frac{v}{2} \right) = 4125 \text{ kip-ft} \]

**Consecutive**

\[ A_s = \frac{4125 \times 12}{0.9 \times 14 \times 0.9 \times 9} \left( \frac{12}{14} \right) = 0.017 \text{ in}^2/\text{ft} < 0.01 \text{ in}^2/\text{ft} \checkmark \]
materials of construction. Alternatively, buildings and other structures, and parts thereof, shall be designed and constructed to support safely the "nominal loads" in load combinations defined in this code without exceeding the appropriate specified allowable stresses for the materials of construction.

Loads and forces for occupancies or uses not covered in this chapter shall be subject to the approval of the building official.

1604.3 Serviceability. Structural systems and members thereof shall be designed to have adequate stiffness to limit deflections and lateral drift. See Section 12.12.1 of ASCE 7 for drift limits applicable to earthquake loading.

1604.3.1 Deflections. The deflections of structural members shall not exceed the more restrictive of the limitations of Sections 1604.3.2 through 1604.3.5 or that permitted by Table 1604.3.

1604.3.2 Reinforced concrete. The deflection of reinforced concrete structural members shall not exceed that permitted by ACI 318.

1604.3.3 Steel. The deflection of steel structural members shall not exceed that permitted by AISC 360, AISI S100, ASCE 8, SIJ CI, SIJ JC, SIJ K or SIJ LH/DLH, as applicable.

1604.3.4 Masonry. The deflection of masonry structural members shall not exceed that permitted by TMS 402/ACI 530/ASCE 5.

1604.3.5 Aluminum. The deflection of aluminum structural members shall not exceed that permitted by AA ADM1.

1604.3.6 Limits. The deflection limits of Section 1604.3.1 shall be used unless more restrictive deflection limits are required by a referenced standard for the element or finish material.

1604.4 Analysis. Load effects on structural members and their connections shall be determined by methods of structural analysis that take into account equilibrium, general stability, geometric compatibility and both short- and long-term material properties.

<table>
<thead>
<tr>
<th>TABLE 1604.3</th>
<th>DEFLECTION LIMITS**&lt;sup&gt;a,b&lt;/sup&gt;</th>
<th></th>
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</thead>
<tbody>
<tr>
<td>CONSTRUCTION</td>
<td>L</td>
<td>S or W</td>
<td>D + L&lt;sup&gt;e&lt;/sup&gt;</td>
<td></td>
</tr>
<tr>
<td>Roof members:&lt;sup&gt;a&lt;/sup&gt;</td>
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<td></td>
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<tr>
<td>Supporting plaster or stucco ceiling</td>
<td>1/360</td>
<td>1/360</td>
<td>1/240</td>
<td></td>
</tr>
<tr>
<td>Supporting nonplaster ceiling</td>
<td>1/240</td>
<td>1/240</td>
<td>1/180</td>
<td></td>
</tr>
<tr>
<td>Not supporting ceiling</td>
<td>1/180</td>
<td>1/180</td>
<td>1/120</td>
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<tr>
<td>Floor members</td>
<td>1/360</td>
<td>—</td>
<td>1/240</td>
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<tr>
<td>Exterior walls:</td>
<td>—</td>
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<td>1/240</td>
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<tr>
<td>With plaster or stucco finishes</td>
<td>—</td>
<td>—</td>
<td>—</td>
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<tr>
<td>With other brittle finishes</td>
<td>—</td>
<td>—</td>
<td>—</td>
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</tr>
<tr>
<td>With flexible finishes</td>
<td>—</td>
<td>1/120</td>
<td>—</td>
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<tr>
<td>Interior partitions:&lt;sup&gt;b&lt;/sup&gt;</td>
<td>1/360</td>
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<td>With plaster or stucco finishes</td>
<td>1/240</td>
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<td>With other brittle finishes</td>
<td>1/120</td>
<td>—</td>
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<tr>
<td>With flexible finishes</td>
<td>—</td>
<td>1/180</td>
<td>1/120</td>
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<tr>
<td>Farm buildings</td>
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</tr>
<tr>
<td>Greenhouses</td>
<td>—</td>
<td>—</td>
<td>1/180</td>
<td></td>
</tr>
</tbody>
</table>

For SI: 1 foot = 304.8 mm.

<sup>a</sup> For structural roofing and siding made of formed metal sheets, the total load deflection shall not exceed 1/60. For secondary roof structural members supporting formed metal roofing, the live load deflection shall not exceed 1/150. For secondary wall members supporting formed metal siding, the design wind load deflection shall not exceed 1/90. For roofs, this exception only applies when the metal sheets have no roof covering.

<sup>b</sup> Flexible, folding and portable partitions are not governed by the provisions of this section. The deflection criterion for interior partitions is based on the horizontal load defined in Section 1607.14.

<sup>c</sup> See Section 2403 for glass supports.

<sup>d</sup> The deflection limit for the D + L load combination only applies to the deflection due to the creep component of long-term dead load deflection plus the short-term live load deflection. For wood structural members that are dry at time of installation and used under dry conditions in accordance with the ANSI/AWC NDS, the creep component of the long-term deflection shall be permitted to be estimated as the immediate dead load deflection resulting from D. The value of 0.5D shall not be used in combination with ANSI/AWC NDS provisions for long-term loading.

<sup>e</sup> The above deflections do not ensure against ponding. Roofs that do not have sufficient slope or camber to ensure adequate drainage shall be investigated for ponding. See Section 1611 for rain and ponding requirements and Section 1503.4 for roof drainage requirements.

<sup>f</sup> The wind load is permitted to be taken as 0.42 times the "component and cladding" loads for the purpose of determining deflection limits herein. Where members support glass in accordance with Section 2403 using the deflection limit therein, the wind load shall be no less than 0.6 times the "component and cladding" loads for the purpose of determining deflection.

<sup>g</sup> For steel structural members, the dead load shall be taken as zero.

<sup>h</sup> For aluminum structural members or aluminum panels used in skylights and sloped glazing framing, roofs or walls of sunroom additions or patio covers not supporting edge of glass or aluminum sandwich panels, the total load deflection shall not exceed 1/60. For continuous aluminum structural members supporting edge of glass, the total load deflection shall not exceed 1/75 for each glass lite or 1/60 for the entire length of the member, whichever is more stringent. For aluminum sandwich panels used in roofs or walls of sunroom additions or patio covers, the total load deflection shall not exceed 1/120.

<sup>i</sup> For cantilever members, L shall be taken as twice the length of the cantilever.
Appendix C - Structural Drawings
Mt Carmel Entrance Canopy
Structural Drawings

June 24, 2018
1. SEE EXISTING STRUCTURAL DRAWINGS BY MAUL-STEWART ASSOCIATES INC. IN 1984 FOR EXITING CONDITIONS

NOTES

Mt Carmel Entrance Canopy

Cal Poly Senior Project
Spring 2018, Kevin Dong
Kyle Chase, Ricardo Gustavson, Jiaming Liu, Max Snook

Scale 3/16" = 1'-0"
FRAMING PLAN

3/16" = 1'-0"

NOTES
1. "1" THOUGH "6" DENOTES CONNECTION TYPE, SEE SHEETS

Cal Poly Senior Project
Spring 2018, Kevin Dong
Kyle Chase, Ricardo Gustavson, Jiaming Liu, Max Snook
ELEVATION VIEW

1/4" = 1'-0"

TOP OF COLUMN (11'-0"
TOP OF COLUMN (9'-2 1/2"
TOP OF COLUMN (9'-2 3/16"
TOP OF COLUMN (10'-6"
TOP OF COLUMN (13'-0"
TOP OF STEEL 15'-6"
TOP OF COLUMN 8'-0"
TOP OF GROUND 0'-0"

Cal Poly Senior Project
Spring 2018, Kevin Dong
Kyle Chase, Ricardo Gustavson, Jiaming Liu, Max Snook

Mt Carmel
Entrance Canopy

No. Description Date
ELEVATION VIEW

Project number: Project Number
Date: Issue Date
Drawn by: Author
Checked by: Checker
Scale As indicated
1 CONNECTION 1
1 1/2" = 1'-0"

NOTES:
1. HSS COLUMN AND HSS BEAM SIZE SEE SHEET S14-S117
2. ALL ANGLES ARE L4.5"x4.5"x0.375" W/ CUSTOM ANGLE U.N.O.
3. ANGLE DETAILS PER S.11
 NOTES:
1. HSS COLUMN AND HSS BEAM SIZE SEE SHEET S14-S117
2. ALL ANGLES ARE L4.5"X4.5"X0.375" W/ CUSTOM ANGLE U.N.O.
3. ANGLE DETAILS PER S.11
NOTES:
1. HSS COLUMN AND HSS BEAM SIZE SEE SHEET S14-S117
2. ALL ANGLES ARE L4.5"x4.5"x0.375" W/ CUSTOM ANGLE U.N.O.
3. ANGLE DETAILS PER S.11

SECTION AA

SECTION BB

CONNECTION 3

1 1/2" = 1'-0"
NOTES:
1. HSS COLUMN AND HSS BEAM SIZE SEE SHEET S14-S117
2. ALL ANGLES ARE L4.5"x4.5"x0.375" W/ CUSTOM ANGLE U.N.O.
3. ANGLE DETAILS PER S.11
NOTES:
1. HSS COLUMN AND HSS BEAM SIZE SEE SHEET S14-S117
2. ALL ANGLES ARE L4.5"x4.5"x0.375" W/CUSTOM ANGLE U.N.O.
3. ANGLE DETAILS PER S.11
TRAINGLE C PEAK

6" SQ.
STL Plate

(E) COLUMN
(E) COLUMN PLATEAU
(E) CC66

CC66 ANCHOR W/ (2) 1/2" DAIM x 15" BARS

NOTES:
1. HSS COLUMN AND HSS BEAM SIZE SEE SHEET S14-S117

1 CONNECTION 6
1" = 1'-0"
### ANGLE DETAILS

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<thead>
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<th>Description</th>
<th>Date</th>
</tr>
</thead>
<tbody>
<tr>
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<td>ANGLE CONNECTION</td>
<td>12° = 1'-0&quot;</td>
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### CONNECTION

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<th>ANGLE BOTTOM</th>
<th>ANGLE TOP</th>
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<tr>
<td>2</td>
<td>6°</td>
<td>6°</td>
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<tr>
<td>3</td>
<td>0°</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>11°</td>
<td>6°</td>
</tr>
<tr>
<td>5</td>
<td>18°</td>
<td>7°</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Appendix D - Fabrication Drawings
Mt Carmel
Entrance Canopy

6"x6"x0.188"

1

2

3

4

5

6

1 COLUMN
1/2" = 1'-0"