DOWNTOWN MARQUEE
PORT ORCHARD, WASHINGTON

STRUCTURAL EVALUATION REPORT

CTS PROJECT NO. WA05.055.S01

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Prepared by

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A. EXECUTIVE SUMMARY

A.1 Marquee Description

Port Orchard Downtown Marquee was constructed in the early 1970's as a means of providing protection for pedestrians using the sidewalks on each side of Bay Street, from Frederick Street at the west end towards the east, where Bay Street takes a moderate turn to the north.

Sections A and B in Appendix A show a general configuration of the marquee. A corrugated metal deck serves as the roof of the marquee, and is supported by steel channels at the face of the buildings and at the edge of the sidewalks. One exception to this is at the southeast corner of Bay Street and Sidney Street intersection, where the roof of the marquee is made of wood decking supported by wood beams projecting from the building.

The steel channels at the face of the buildings are sometimes bolted to the building, but sometimes they are supported by metal posts. Most of the metal posts have steel base-plates bolted to the sidewalk, although some are embedded in the sidewalk with no visible base-plates. At the edge of the sidewalk, the steel channels are supported by round poles or build-up wood posts.

The knee braces that are located at each of the columns consist of 6x10 lumber giving the marquee its characteristic look. See Elevation C in Appendix A.

For the purposes of documenting structural conditions of the marquee, we have divided this structure into four segments named Marquee #1, #2, #3, and #4. This division can be seen in Plan 1 in Appendix A.

A.2 Structural System

The structural system to carry gravity loads, such as the weight of the materials, or snow load, consists of a corrugated metal deck, steel channels, and wood columns. The built-up wood posts are supported by small concrete pedestals, while round poles are embedded in the ground.

The lateral forces, such as wind and earthquake in the direction parallel to Bay Street, are resisted by the frame system consisting of round poles with knee braces. In the direction perpendicular to Bay Street, the corrugated metal deck acts as a diaphragm between the round poles which then resist the lateral force as cantilevers.
In our analysis we have subjected the marquee to the following loads:

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead load:</td>
<td>weight of the various materials</td>
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<tr>
<td>Basic snow load:</td>
<td>25 psf per ICBO 2003</td>
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<tr>
<td>Snow load with drift:</td>
<td>48 psf Marquee #1 and #2</td>
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<tr>
<td></td>
<td>53 psf Marquee #3 and #4</td>
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<tr>
<td>Live load:</td>
<td>75 psf per ICBO 2003</td>
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<tr>
<td>Wind load:</td>
<td>85 mph wind</td>
</tr>
<tr>
<td>Seismic load:</td>
<td>per ASCE/SEI 31-03</td>
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</table>

A.3 Deficiencies of Structural Members

The deficiencies of the structural members can be divided into two groups: a) deficiencies due to their inadequate size, and b) deficiencies due to damage.

a. Deficiencies due to Inadequate Size:

1. The maximum vertical load that can be supported by the metal deck of Marquee #1 and #2 is 39 psf. This load is more than snow load with snow drift, and less than live load of 75 psf.
2. The maximum vertical load that can be supported by the metal deck of Marquee #3 and #4 is 84 psf. For our calculations, we assumed the deck is 22 gage, this being the lightest deck produced by steel mills for some time. The reason why ICBO requires such a high load for canopies must be due to the fact that they extend over areas occupied by the public. Since this not the case in Port Orchard, we are mentioning this deficiency here only for completeness.
3. Steel channel spanning between wood columns has a capacity of 40 psf at Marquee #1 and #2, and 68 psf at Marquee #3 and #4. This capacity is more than snow load with snow drift, but less than live load of 75 psf.
4. Steel Channel supported by the steel posts next to the buildings has a capacity of 27 psf at Marquee #1 and #2, and 32 psf at Marquee #3 and #4. Both capacities are less than snow load with snow drift.
5. The round poles resist lateral forces by being embedded into the ground with support of the sidewalk at the grade level. These poles need to be embedded 8'-0" into the ground to resist wind load if the railing were to remain, and 5'-0" into the ground if the railing is removed. For seismic forces, the embedment needs to be 5'-0". During our investigation, we could not find out how deep these poles are embedded at this time.
6. The metal deck does not meet the flexible diaphragm maximum length to width ratio in order to act as a lateral load carrying element between the round poles. Again, this deficiency is only mentioned here for completeness since it will cause large deflections of the deck, but not a structural collapse. In addition, the metal deck panels are not connected at the seams, that diminishes the ability of the metal deck to act as a diaphragm even more.
7. The steel channels are connected to the round poles with two 5/8” diameter lag bolts. The capacity of this connection is 18 psf at Marquee #1 and #2, and 26 psf at Marquee #3 and #4. Neither capacity meets the snow load with snow drift or live load requirements.

8. The steel channels are connected to the buildings with one 5/8” diameter lag bolt at about 2'-9” spacing. The capacity of this connection is 49 psf at Marquee #1 and #2, and 67 psf at Marquee #3 and #4. Neither capacity meets the 75 psf requirement.

9. The knee brace connections at each end do not have the capacity to transfer lateral loads into the round pole.

b. Deficiencies due to Damage:

1. The wood members of the built-up posts suffer from dry-rot. The condition is worse in Marquee #1 and #2 than in Marquee #3 and #4, because of greater exposure to rain, sun and wind. We discounted the capacity of these columns by 50%, and even at this level, they have a capacity of 180 psf at Marquee #1 and #2, and 260 psf at Marquee #3 and #4.

2. Several of the knee braces also suffer from dry-rot, due to the same exposure conditions as stated for the posts.

3. Several 4x6 and 4x12 wood members forming the gutter, together with the steel beam, have dry-rot due to water penetration.

4. Several 4x12 wood members have severe splits.

5. Several lag bolts are loose due shrinkage of the wood members they are connecting.

A.4  Cost of Repairs

a. Immediate remedies  $ 41,000
b. Near-term repairs    $ 26,000
c. Long-term repairs    $ 10,000
d. Renovation options   $ 8,000

All costs include materials, labor, contingency, and contractor’s overhead and profit, but do not include WSST. See Section D for details of the proposed remedies and repairs.

B.  MARQUEE CONDITIONS

B.1  Marquee #1 – See Plan 2 in Appendix B and photos in Appendix C
B.2  Marquee #2 – See Plan 3 in Appendix B and photos in Appendix C
B.3  Marquee #3 – See Plan 4 in Appendix B and photos in Appendix C
C. STRUCTURAL MODELING

The marquee configuration, shown in Elevation C in Appendix A, depicts the marquee as it is viewed from the street. The round poles, also serving as lamp posts, are spaced at regular intervals of approximately 60 feet. This distance is further divided into three equal spaces by the build-up columns supporting gravity loads. The structural model of a complete marquee 60-foot bay is shown in Elevation D of Appendix A as Diagram #1.

Diagram #2 in Elevation D shows our model for resisting the lateral loads - in this case seismic load, acting in a direction parallel to Bay Street. In comparison with Diagram #1, we have removed the knee braces at the build-up posts, because if the build-up posts do not have a positive connection to the concrete base, the knee braces have a minimum role in resisting any lateral force. In the model shown in Diagram #2, the knee braces act in tension and compression.

Diagram #3 is similar to Diagram #2, except in this case only one knee brace in compression participates in the system resisting the lateral load. We have included this configuration to demonstrate that the present system has sufficient redundancy, even when some of the knee braces are incapacitated due to dry-rot, and to consider that the knee brace in tension may have insufficient connection at the top since it subjects the lag bolt to withdrawal force.

Diagram #4 shows the round pole resisting the lateral force in the direction perpendicular to the building as a cantilever.

The conclusion of our structural modeling is that the round poles have sufficient capacity to resist wind and seismic forces without knee braces provided that they have adequate embedment into the soil, and provided that the lag bolts connecting the steel channels to the these poles are increased to 1" diameter.

We have included several details in Appendix A worthy of mention. Detail E shows the build-up column configuration. The individual wood members are connected together with a few spikes and bolts. The connection is not sufficient to make these members act as one column; especially considering that the spikes have split the wood at the bottom of these posts, as demonstrated in one of the photos.

Detail F shows how the steel channel and 4x6 and 4x12 wood members, that are connected together, act as a horizontal link between the round poles. Also, all three members form a gutter that is interrupted at each round pole and slopes toward the build-up posts, because they have a downspout inside each one of them.

Detail G shows the only lag bolt serving as a connection between the horizontal link of the structural system and the knee brace.
Detail H shows condition at the joints between the metal deck panels, and a need for a shear connector at round pole locations.

D. RECOMMENDED REPAIRS

D.1 Immediate Remedies

As immediate measures to improve safety, we recommend removal of the picket fence; repairs to the gutter, replacement of the knee braces and 4x12s damaged by dry-rot; replacement of all wood members identified as having splits; and replacement of the lag bolts with larger size to account for the wood shrinkage or for size deficiency.

Cost: $41,000

D.2 Near-term Repairs

In cooperation with the building owners, replace existing flashing between the building wall and the marquee roof. Provide button punch connections between the metal deck panels. Provide shear connectors at each round pole.

Cost: $26,000

D.3 Long-term Repairs

Replace all built-up wood posts with pressure-treated lumber.

Cost: $10,000

D.4 Renovation Options

Remove knee braces at the built-up posts, where they serve only an aesthetic purpose.

Cost: $8,000

D.5 Miscellaneous Items

Maintenance Checklist:
- Leaking
- Gutter cleaning
- Minor repairs

Annual maintenance budget: $3,000 to $4,000

Business Signs
Once the picket fence is removed, the individual businesses would have the opportunity to attach their signs to the outside 4x12 fascia using the steel bolts already in place, and in case of larger signs, brace these back to the metal deck as is done at the present time.
FACE OF BUILDING

1 1/2" CORRUGATED METAL DECK

GUTTER

2 x 4 BOTTOM & TOP RAIL

1 x 6 PICKETS

2 x 4

4 x 4 POST @ 2'-0" TO 5'-0" SPACING

4 x 12

4 x 6

3 x 8 @ EA. FACE @ BUILT-UP POSTS

2 x 6 @ EA. SIDE @ BUILT-UP POSTS

RAIN LEADER CURB

SIDWALK

3" ± DIA. STEEL POST

8'-6" ± NORTH SIDE
5'-10" ± SOUTH SIDE
OF BAY STREET

DOWNTOWN MARQUEE

PORT ORCHARD, WASHINGTON 98366

STRUCTURAL INVESTIGATION

JOB NO. WA05.055.S01
DATE 10.10.05
SHEET NO. SECTION A
DOWNTOWN MARQUEE
PORT ORCHARD, WASHINGTON 98366
STRUCTURAL INVESTIGATION
2x8 BACK COVER PLATE

2x6 SIDE PLATES

BOLT (2 OR 3 TOTAL PER POST)

3x8 FRONT COVER PLATE

RAIN LEADER

SPike @ 18" to 24" SPACING (TYP.)
PUDDLE WELDS
METAL DECK
STEEL CHANNEL
5/8" DIA. LAG BOLT
6" NOMINAL
4" NOMINAL

FLASHING
2x4
CONTINUOUS 4x4 FILLER

8" DIA. M.B.
12" NOMINAL

NOTE: RAILING POSTS NOT SHOWN
NOT SHOWN
4x6
5" DIA. LAG BOLT INTO 4x6
6x10 KNEE BRACE
Metal deck panels are not connected at joints. Recommend adding button punch connection @ 24”.

Add puddle welds @ 12”.

Add shear connector L 2x2x\(\frac{3}{16}\) at each round pole.
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<td>9</td>
<td>L 00 00 00 28</td>
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</table>

Note: Please refer to the applicable code requirements or project specifications for detailed information on the listed items.

Legend:
- L: Location
- V/H: Vertical/Horizontal

This plan is intended for design and construction purposes and should be used in conjunction with other project documents.
DOWNTOWN MARQUEE
PHOTO #1
MARQUEE #1
P1 BUILT-UP POST BASE
DOWNTOWN MARQUEE
PHOTO #2
MARQUEE #1
P3 KNEE BRACE SPLIT
DOWNTOWN MARQUEE
PHOTO #3
MARQUEE #1
P4 TO C3 PICKET FENCE POST
DOWNTOWN MARQUEE
PHOTO #4
MARQUEE #1
C3 SPLIT IN 4x12
DOWNTOWN MARQUEE
PHOTO #5
MARQUEE #1
P6 FRONT COVER PLATE SPLIT
DOWNTOWN MARQUEE
PHOTO #6
MARQUEE #2
STEEL POST W/BASE, TYP @ MARQUEE #2
DOWNTOWN MARQUEE
PHOTO #7
MARQUEE #2
4x12 CORNER W/ DAMAGE
DOWNTOWN MARQUEE
PHOTO #8
MARQUEE #2
POST W/ HEAVY DAMAGE
DOWNTOWN MARQUEE
PHOTO #9
MARQUEE #2
4x6 DAMAGE
DOWNTOWN MARQUEE
PHOTO #10
MARQUEE #3
LOOSE 4x6 AND UNSAFE KNEE BRACE
DOWNTOWN MARQUEE
PHOTO #11
MARQUEE #3
OTHER SIDE OF SAME CONDITION
DOWNTOWN MARQUEE
PHOTO #12
MARQUEE #4
LAG BOLT PROJECTING OUT
DOWNTOWN MARQUEE
PHOTO #13
MARQUEE #4
4x12 SPLIT
DOWNTOWN MARQUEE
PHOTO #14
MARQUEE #4
BENT DECK AT JOINT BETWEEN C21 & P38
DECK IS RUSTY
DOWNTOWN MARQUEE
PHOTO #15
MARQUEE #2
SW CORNER WITH SEVERE PONDING
DOWNTOWN MARQUEE
PHOTO #16
MARQUEE #2
FLASHING BY BUILDING OWNER
DOWNTOWN MARQUEE
PHOTO #17
MARQUEE # N/A
4x4 RAILING POST CONNECTION
DESIGN CRITERIA

The International Building Code (IBC), 2003 Edition
ASCE Standard “Minimum Design Loads for Buildings and Other Structures”, ASCE 7-02
ASCE Standard “Seismic Evaluation of Existing Buildings”, ASCE/SEI 31-03

MATERIALS

Steel Shapes and Plates: ASTM A36, Fy = 33.0 ksi
Wood Construction Design Values: Douglas Fir-Larch No.2
Bolts: ASTM A307
Lag Bolts: NDS -2001, Appendix L.

ALLOWABLE STRESSES

ASTM A36 Structural Steel:
Bending Fb = 0.6x33.0 = 19.80 ksi (AISC Specification, Chapter F)

Wood Pole (NDS-2001, Table 6B):
Bending: Fb = 1850 psi
Compression parallel to grain: Fc = 1000 psi

Sawn Lumber, DF#2, 2” & wider (NDS-2001, Table 4A):
Bending: Fb = 900psi
Compression parallel to grain: Fc = 1350psi

Sawn Lumber, DF#2, 5”x5” and larger (NDS-2001, Table 4D):
Bending: Fb = 750psi
Tension parallel to grain: Ft = 475psi
Compression perpendicular to grain: Fc = 625psi
Compression parallel to grain: Fc = 700psi

Wood Members Allowable Stress Increase for Wood Members (NDS-2001, Table 2.3.2):
Snow Load: Cd = 1.15
Wind Load: Cd = 1.6

Seismic Evaluation Nominal Strength Multipliers (ASCE/SEI 31-03, Section 4.2.4.4):
Steel 1.7
Masonry 2.5
Wood 2.0
Lag Bolts 2.5 (FEMA 356, Chapter 8)

Allowable Lateral Soil Pressure:
S3 400 psf per each foot of depth
DESIGN LOADS

Live Load – 75psf (ASCE 7-02, Table 4-1)

Snow Load – 20 psf (IBC 2003)

Snow Drift per ASCE 7-02, Section 7.7:
Ground snow load Pg = 15psf
D = 0.13Pg + 14 = 0.13(15) + 14 = 15.95 pcf.
h_d = 0.43 x (W_b)^{1/3} x (P_g+10)^{1/4} \cdot 1.5 = 0.43 x (50)^{1/3} x (15+10)^{1/4} \cdot 1.5 = 0.43 x 3.68 x 2.24-1.5=2.05ft.
h_b = Pf/D = 15/15.95 = 0.94ft.
Wd = 4h_d = 4(2.05) = 8.20ft.
Pm = D (h_d + h_b) = 15.95(2.05 + 0.94) = 47.70psf
Snow Drift Load = 47.70 – 15.00 = 32.70psf

Wind Load: Basic Wind Speed 85mph, Exposure B, Iw = 1.0
F = qzGCf (ASCE 7-02, Sec.6.5.13), where
Cf = 1.2 (Fig. 6-20)
G = 0.85 (Section 6.5.8.1)
qz = 0.00256KzKztKdV^2l (Eq.6-15), where
Kd = 0.85 (Table 6-4)
Kzt = 1.0 (Assumed)
Kz = 0.70 (Table 6-3, Case 1
qz = 0.00256*0.70*1.0*0.85*85^2*1.0 = 11.00 psf
F = 11.00*0.85*1.2 = 11.20 psf

Seismic Load:
Site Class: D
Level of Performance: Life Safety (LSP).
Level of Seismicity (High): S_3 = 1.50g, S_1 = 0.50g.
Building Type W2: Wood Frames.

Acceptance Criteria for LSP (ASCE/SEI 31-03, 4.2.4)
Deformation-Controlled Actions (4.2.3.1):
Qu = Qg + Qe, where
Qg = Action due to Gravity Loads and EQ Forces
Qe = Action due to Gravity Forces

Force-Controlled Actions (4.2.4.3.2):
Quf = Qg + Qe/CJ, where
C = 1.3 (Table 3-4)
J = 2.5 in locations of high seismicity
ROOF DECK FOR LIVE LOAD CHECK

DESIGN LIVE LOAD = 75 psf

ASSUME 22.6 ft x 1/2 in RIBBED STEEL DECK

THE ROOF DECK SPAN VARIES FROM 5'-10" ON SOUTH SIDE TO 8'-6" ON NORTH SIDE.

ALLOWABLE UNIFORM LOAD ON ROOF DECK DUE TO

ALLOWABLE STRESSES:

FOR 5'-10" = 84 psf > LL = 75 psf

> SL = 36.37 psf (See p. 3A)

FOR 8'-6" = 39 psf < LL = 75 psf

> SL = 51.70 psf (See p. 3B)

ALLOWABLE UNIFORM LOAD ON ROOF DECK DUE TO DEFLECTIONS:

FOR 5'-10" = 55 psf < LL = 75 psf

> SL = 36.09 psf (See p. 3A)

FOR 8'-6" = 18 psf < LL = 75 psf

< SL = 50.90 psf (See p. 3B)

CONCLUSION:

METAL DECK MAX LOAD FOR 5'-10" MARQUEE IS 84 psf

METAL DECK MAX LOAD FOR 8'-6" MARQUEE IS 39 psf
ROOF DECK FOR SNOW LOAD CHECK

For 5'-10' Deck:

Uniform snow load for allowable stress:

Design = 24.45 + 23.25 * (18/15.6) = 24.45 + 11.92 = 36.37 psf < 38 psf allowable uniform load on roof deck due to allowable stresses

Uniform snow load for deflection design:

Design = 24.45 + 23.25 * (0.00652) = 24.45 + 11.64 = 36.09 psf < 55 psf allowable uniform load on roof deck due to deflections.
For 8' 6" deep:

**Uniform Snow Load for Allowable Stress**

\[ \text{Design} = 15.00 + 32.70 \times \left( \frac{8}{15.6} \right) = 15.00 + 16.70 = \]

\[ = 31.70 \text{ psf} < 39 \text{ psf Allowable Uniform Load} \]

**Due to Allowable Stresses**

**Uniform Snow Load for Deflection Design**

\[ \text{Design} = 15.00 + 32.70 \times \left( \frac{100.652}{0.01302} \right) = 15.00 + 15.40 = \]

\[ = 30.90 \text{ psf} > 18 \text{ psf Allowable Uniform Load Due to Deflections} \]
DECK SUPPORTING CHANNEL AT POLE

MARQUEE # 1

MAX. CHANNEL SPAN = 20.27 FT.

DL = (2)(18.50/2) = 9.25 LF

LL = (75)(8.50/2) = 31.9 hLF

SL = (15.00)(8.50/2) + (38.70)(8.50/2)(2/3) = 63.75 + 92.63 = 156.38 hLF

ALLOWABLE LL = 240/4.85 = 50.00 psf < 75 psf (p.5)

ALLOWABLE SNOW LOAD = 240 hLF > 156.40 hLF (p.5)

MARQUEE # 2

MAX. CHANNEL SPAN = 23.66 FT.

ALLOWABLE LL = 170/4.85 = 40.00 psf < 75 psf (p.6)

ALLOWABLE SNOW LOAD = 170 hLF > 156.40 hLF (p.6)
## Steel Beam Design

**Description:** Marquee #1 C8 Channel at Pole for Live Load check

### General Information
Calculations are designed to AISC 9th Edition ASD and 1997 UBC Requirements

<table>
<thead>
<tr>
<th>Steel Section: C8X11.5</th>
<th>Pinned-Pinned</th>
<th>Fy</th>
<th>Load Duration Factor</th>
<th>Elastic Modulus</th>
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</thead>
<tbody>
<tr>
<td>Center Span</td>
<td>20.27 ft</td>
<td>33.00 ksi</td>
<td>1.00</td>
<td>29,000 ksi</td>
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<tr>
<td>Left Cant.</td>
<td>0.00 ft</td>
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</tr>
<tr>
<td>Right Cant.</td>
<td>0.00 ft</td>
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<tr>
<td>Ly : Unbraced Length</td>
<td>0.00 ft</td>
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### Distributed Loads

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### Summary

Using: C8X11.5 section, Span = 20.27ft, Fy = 33.0 ksi
End Fixity = Pinned-Pinned, Ly = 0.00ft, LDF = 1.000

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<th>Allowable</th>
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<td>fb : Bending Stress</td>
<td>19.698 ksi</td>
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<td>fb / Fb</td>
<td>0.995 : 1</td>
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<tr>
<td>Shear</td>
<td>2.640 k</td>
<td>23.232 k</td>
</tr>
<tr>
<td>fv : Shear Stress</td>
<td>1.500 ksi</td>
<td>13.200 ksi</td>
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<td>fv / Fv</td>
<td>0.114 : 1</td>
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Max. Deflection: -1.047 in
Length/DL Defl: 2.956:1
Length/(DL+LL Defl): 232.4:1

### Force & Stress Summary

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<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear @ Left</td>
<td>2.64 k</td>
<td>0.21</td>
<td>2.64</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear @ Right</td>
<td>2.64 k</td>
<td>0.21</td>
<td>2.64</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Center Defl</td>
<td>-1.047 in</td>
<td>-0.052</td>
<td>-1.047</td>
<td>-1.047</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>Left Cant Defl</td>
<td>0.000 ln</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>Right Cant Defl</td>
<td>0.000 ln</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>...Query Name @</td>
<td>0.000 ft</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>Reaction @ Left</td>
<td>2.64 k</td>
<td>0.21</td>
<td>2.64</td>
<td>2.64</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reaction @ Rt</td>
<td>2.64 k</td>
<td>0.21</td>
<td>2.64</td>
<td>2.64</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Fa calc'd per Eq, E2.1, K' / Lp < Cc
1 Beam Passes Table B5.1, Fb per Eq. F1-1, Fb = 0.66 Fy

### Section Properties

<table>
<thead>
<tr>
<th></th>
<th>C8X11.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth</td>
<td>8.000 in</td>
</tr>
<tr>
<td>Width</td>
<td>2.260 in</td>
</tr>
<tr>
<td>Web Thick</td>
<td>0.220 in</td>
</tr>
<tr>
<td>Flange Thickness</td>
<td>0.390 in</td>
</tr>
<tr>
<td>Area</td>
<td>3.38 in²</td>
</tr>
</tbody>
</table>

- Weight: 11.48 #/ft
- r-xx: 3.106 in
- r-yy: 0.625 in

---

*Note: The calculations and data provided are based on the specifications and requirements outlined in the AISC 9th Edition ASD and 1997 UBC Requirements.*
# Steel Beam Design

## General Information

- **Calculations are designed to AISC 9th Edition ASD and 1997 UBC Requirements**
- **Steel Section**: C8X11.5
- **Fy**: 33.00 ksi
- **Load Duration Factor**: 3.00
- **Elastic Modulus**: 29,000.0 ksi

## Distributed Loads

<table>
<thead>
<tr>
<th>#1</th>
<th>#2</th>
<th>#3</th>
<th>#4</th>
<th>#5</th>
<th>#6</th>
<th>#7</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL</td>
<td>0.009 k/ft</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LL</td>
<td>0.170 k/ft</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ST</td>
<td>0.000 k/ft</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Start Location</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>End Location</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

## Summary

- **Beam OK**
- **Static Load Case Governs Stress**
- **Moment**: 13,447 k-ft, Fy = 33.00 ksi
- **Max. Deflection**: -1.421 in
- **Length/Defl**: 1,858.8 : 1
- **Length/(DL+LL Defl)**: 199.9 : 1

## Force & Stress Summary

- **Max. M +**
  - Maximum: 13.33 k-ft
  - DL Only: 1.43 k-ft
  - LL @ Center: 13.33 k-ft
- **Max. M -**
- **Max. M @ Left**
- **Max. M @ Right**
- **Shear @ Left**: 2.25 k, 0.24 k
- **Shear @ Right**: 2.25 k, 0.24 k
- **Center Defl**: -1.421 in, -1.421 in
- **Left Cant Defl**: 0.000 in, 0.000 in
- **Right Cant Defl**: 0.000 in, 0.000 in
- **...Query Defl @**: 0.000 k-ft
- **Reaction @ Left**: 2.25 k, 0.24 k
- **Reaction @ Rt**: 2.25 k, 0.24 k

## Section Properties

- **Depth**: 8.000 in
- **Width**: 0.00 in
- **Web Thick**: 0.220 in
- **Flange Thickness**: 0.390 in
- **Area**: 3.36 in²
- **Weight**: 11.48 #/ft
- **r-xx**: 3.106 in
- **r-yy**: 0.625 in
- **r-xy**: 1.32 in
- **I-xx**: 32.60 in⁴
- **I-yy**: 6.150 in³
- **S-xx**: 6.782 in³
**MARQUEE #3**

**MAX. CHANNEL SPAN = 28.80'**

\[
DL = (2)(5.83/2) = 6.04\text{ kLF}
\]

\[
LL = (75)(5.83/2) = 219.0\text{ kLF}
\]

\[
SL = (24.45)(5.83/2) + (23.25)(5.83/2)(2/3) = 71.07 + 45.18 = 116.45\text{ kLF}
\]

**ALLOWABLE LL = 200 \times 0.92 = 68.50 \text{ psf} < 75 \text{ psf} (p.8)**

**ALLOWABLE SNOW LOAD = 200 \text{ psf} > 116.45 \text{ psf} (p.8)**

**MARQUEE #4**

**MAX. CHANNEL SPAN = 19.77'**

**ALLOWABLE LL = 75 \text{ psf} (p.9)**

**ALLOWABLE SL = 167 \text{ kLF} > 116.45 \text{ kLF}**

**CONCLUSION:**

**FOR STEEL CHANNEL AT POLE:**

**MAX. LOAD FOR 8 1/2" WIDE MARQUEE AND 23 1/8" SPAN IS 40 \text{ psf}, CAPACITY OF THE STEEL CHANNEL EXCEEDS CAPACITY OF THE METAL DECK.**

**MAX. LOAD FOR 5 1/8" WIDE MARQUEE AND 22 1/2" SPAN IS 68.50 \text{ psf} AND IS LESS THAN THE CAPACITY OF THE METAL DECK.**
# Steel Beam Design

**Description:** Marquee #3 C8 Channel at Pole for Live Load and Snow Load check

### General Information

Calculations are designed to AISC 9th Edition ASD and 1997 UBC Requirements

<table>
<thead>
<tr>
<th>Steel Section: C8X11.5</th>
<th>Pinned-Pinned Load Duration Factor</th>
<th>Elastic Modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>Center Span: 22.20 ft</td>
<td>Fy = 33.00 ksi</td>
<td>29,000 ksi</td>
</tr>
<tr>
<td>Left Cant: 0.00 ft</td>
<td>Bm Wt. Added to Loads</td>
<td></td>
</tr>
<tr>
<td>Right Cant: 0.00 ft</td>
<td>LL &amp; ST Act Together</td>
<td></td>
</tr>
<tr>
<td>Lu : Unbraced Length: 0.00 ft</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Distributed Loads

<table>
<thead>
<tr>
<th>#1</th>
<th>#2</th>
<th>#3</th>
<th>#4</th>
<th>#5</th>
<th>#6</th>
<th>#7</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL</td>
<td>0.006</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LL</td>
<td>0.200</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Summary

Using: C8X11.5 section, Span = 22.20ft, Fy = 33.0ksi

End Fixity = Pinned-Pinned, Lu = 0.00ft, LDF = 1.000

#### Beam OK

Static Load Case Governs Stress

#### Force & Stress Summary

<table>
<thead>
<tr>
<th></th>
<th>Maximum</th>
<th>DL Only</th>
<th>@ Center</th>
<th>LL @ Center</th>
<th>LL+ST @ Cants</th>
<th>LL+ST @ Cants</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max. M+</td>
<td>13.40 k-ft</td>
<td>1.06</td>
<td>13.40</td>
<td></td>
<td></td>
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<tr>
<td>Max. M -</td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Max. M @ Left</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max. M @ Right</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear @ Left</td>
<td>2.41 k</td>
<td>0.19</td>
<td>2.41</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear @ Right</td>
<td>2.41 k</td>
<td>0.19</td>
<td>2.41</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>Center Def.</td>
<td>-1.257 in</td>
<td>-0.101</td>
<td>-1.257</td>
<td>-1.257 0.000</td>
<td>0.000 0.000</td>
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</tr>
<tr>
<td>Left Cant Def.</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000 0.000</td>
<td>0.000 0.000</td>
<td></td>
</tr>
<tr>
<td>Right Cant Def.</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000 0.000</td>
<td>0.000 0.000</td>
<td></td>
</tr>
<tr>
<td>...Query Def. @</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000 0.000</td>
<td>0.000 0.000</td>
<td></td>
</tr>
<tr>
<td>Reaction @ Left</td>
<td>2.41</td>
<td>0.19</td>
<td>2.41</td>
<td>2.41 2.41 2.41</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reaction @ Rt</td>
<td>2.41</td>
<td>0.19</td>
<td>2.41</td>
<td>2.41 2.41 2.41</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Fa calc'd per Eq. E2.4, K*LL < Cc
I Beam Passes Table B5.1, Fb per Eq. F1.1, Fb = 0.66 Fy

### Section Properties

<table>
<thead>
<tr>
<th>C8X11.5</th>
<th>Depth</th>
<th>Width</th>
<th>Web Thick</th>
<th>Flange Thickness</th>
<th>Area</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>8.000 in</td>
<td>2.260 in</td>
<td>0.220 in</td>
<td>0.950 in</td>
<td>3.35 in²</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>11.48 #/ft</td>
<td>S-xx</td>
<td>r-yy</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>r-xx</td>
<td>3.106 in²</td>
<td>0.625 in²</td>
</tr>
</tbody>
</table>
Steel Beam Design

Description:
Marquee #4 C8 Channel at Pole for Live Load and Snow Load check

General Information
Calculations are designed to AISC 9th Edition ASD and 1997 UBC Requirements

Steel Section: C8X11.5

Center Span: 19.77 ft
Left Cant: 0.00 ft
Right Cant: 0.00 ft
Lu: Unbraced Length: 0.00 ft
Pinned-Pinned
Bm Wt: Added to Loads
LL & ST Act Together

Fy: 33.00 ksi
Load Duration Factor: 1.00
Elastic Modulus: 29,000 ksi

Distributed Loads

<table>
<thead>
<tr>
<th>DL</th>
<th>#1</th>
<th>LL</th>
<th>#2</th>
<th>#3</th>
<th>#4</th>
<th>#5</th>
<th>#6</th>
<th>#7</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.006</td>
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<td></td>
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</tr>
</tbody>
</table>

Summary

Using: C8X11.5 section, Span = 19.77 ft, Fy = 33.00 ksi
End Fixity = Pinned-Pinned, Lu = 0.00 ft, LDF = 1.000

Moment

<table>
<thead>
<tr>
<th>fb: Bending Stress</th>
<th>Actual</th>
<th>Allowable</th>
</tr>
</thead>
<tbody>
<tr>
<td>11.54 k-ft</td>
<td>13.447 k-ft</td>
<td></td>
</tr>
<tr>
<td>17.012 ksi</td>
<td>19.800 ksi</td>
<td></td>
</tr>
</tbody>
</table>

Shear

<table>
<thead>
<tr>
<th>fv: Shear Stress</th>
<th>Actual</th>
<th>Allowable</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.338 k</td>
<td>23.232 k</td>
<td></td>
</tr>
<tr>
<td>1.328 ksi</td>
<td>13.200 ksi</td>
<td></td>
</tr>
</tbody>
</table>

Max. Deflection

Max. Deflection: -0.860 in
Length/DL Defl: 3.732 ft
Length/(DL+LL Defl): 275.9 ft

Beam OK
Static Load Case Governs Stress

Force & Stress Summary

<table>
<thead>
<tr>
<th>Maximum</th>
<th>DL Only</th>
<th>LL @ Center</th>
<th>LL=ST @ Center</th>
<th>LL @ Cants</th>
<th>LL=ST @ Cants</th>
</tr>
</thead>
<tbody>
<tr>
<td>11.55 k-ft</td>
<td>0.85</td>
<td>11.55</td>
<td>k-ft</td>
<td>k-ft</td>
<td></td>
</tr>
<tr>
<td>2.34 k</td>
<td>0.17</td>
<td>2.34</td>
<td>k-ft</td>
<td>k-ft</td>
<td></td>
</tr>
<tr>
<td>0.000 in</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>0.000 ft</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>2.34</td>
<td>0.17</td>
<td>2.34</td>
<td>k-ft</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.34</td>
<td>0.17</td>
<td>2.34</td>
<td>k-ft</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Section Properties C8X11.5

Depth: 8.00 in
Width: 2.280 in
Weight: 11.45 #/ft
r-xx: 3.106 in
Web Thick: 0.220 in
I-xx: 32.65 in4
r-yy: 0.625 in
Flange Thickness: 0.390 in
I-yy: 1.32 in4
S-xx: 8.150 in3
Area: 3.38 in2
S-yy: 0.782 in3
DECO SUPPORTING CHANNEL AT BUILDING

MARQUEE #1 & MARQUEE #2

Max. Channel Span = 28' - 0"

Allowable LL = 116/23 23 = 217 ksf < 75 ksf (p. 11)

Allowable SL = 116 psf < 156.40 psf (p. 4 & 11)

MARQUEE #3

Max. Channel Span = 27' - 0"

Allowable LL = 130 / (5.83/2) = 45 ksf < 75 ksf (p. 12)

Allowable SL = 130 psf > 116.45 psf (p. 7 & 12)

MARQUEE #4

Max. Channel Span = 31' - 6"

Allowable LL = 94 / (5.83/2) = 32 ksf < 75 ksf (p. 13)

Allowable SL = 94 psf < 116.45 psf (p. 7 & 13)

Conclusion:

Max. Load for 8' - 6" wide marquee and 28' - 0" span
is 217 ksf and is less than the capacity of metal deck.
Max. Load for 5' - 10" wide marquee and 29' - 0"
span is 32 ksf and is less than the capacity of metal deck.
# Steel Beam Design

## Description
Marquee #1 & #2 C8 Channel at Building for Live Load and Snow Load check

## General Information
Calculations are designed to AISC 9th Edition ASD and 1997 UBC Requirements

<table>
<thead>
<tr>
<th>Steel Section: C8X11.5</th>
<th>Pinned-Pinned</th>
<th>Fy: 33.00 ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Center Span: 28.00 ft</td>
<td>Bm Wt. Added to Loads</td>
<td></td>
</tr>
<tr>
<td>Left Cant: 0.00 ft</td>
<td>LL &amp; ST Act Together</td>
<td></td>
</tr>
<tr>
<td>Right Cant: 0.00 ft</td>
<td>Elastic Modulus: 29,000 ksi</td>
<td></td>
</tr>
<tr>
<td>Lu: Unbraced Length: 0.00 ft</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

## Distributed Loads

<table>
<thead>
<tr>
<th>#1</th>
<th>#2</th>
<th>#3</th>
<th>#4</th>
<th>#5</th>
<th>#6</th>
<th>#7</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL</td>
<td>0.009</td>
<td>k/ft</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LL</td>
<td>0.116</td>
<td>k/ft</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ST</td>
<td></td>
<td>k/ft</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>ft</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Start Location</td>
<td></td>
<td>ft</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>End Location</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

## Summary
Using: C8X11.5 section, Span = 28.00 ft, Fy = 33.0 ksi
End Fixity = Pinned-Pinned, Lu = 0.00 ft, LDF = 1.00

<table>
<thead>
<tr>
<th>Actual</th>
<th>Allowable</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment</td>
<td>13,375 k-ft</td>
</tr>
<tr>
<td>fb: Bending Stress</td>
<td>19.693 ksi</td>
</tr>
<tr>
<td>fb/Fb</td>
<td>0.995 : 1</td>
</tr>
<tr>
<td>Shear</td>
<td>1.911 k</td>
</tr>
<tr>
<td>fv: Shear Stress</td>
<td>1.086 ksi</td>
</tr>
<tr>
<td>fv/Fv</td>
<td>0.083 : 1</td>
</tr>
</tbody>
</table>

Max. Deflection: -1.996 in
Length/DL Defl: 1,121.5 : 1
Length/(DL+LL Defl): 168.3 : 1

Beam OK
Static Load Case Governs Stress

## Force & Stress Summary
<<-- These columns are Dead + Live Load placed as noted -->>

<table>
<thead>
<tr>
<th>Maximum</th>
<th>DL Only</th>
<th>LL @ Center</th>
<th>LL+ST @ Center</th>
<th>LL @ Cants</th>
<th>LL+ST @ Cants</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max. M+</td>
<td>13.38 k-ft</td>
<td>2.01</td>
<td>13.38</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max. M-</td>
<td>k-ft</td>
<td>k-ft</td>
<td>k-ft</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max. M @ Left</td>
<td>k-ft</td>
<td>k-ft</td>
<td>k-ft</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max. M @ Right</td>
<td>k-ft</td>
<td>k-ft</td>
<td>k-ft</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear @ Left</td>
<td>1.91 k</td>
<td>0.25</td>
<td>1.91</td>
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<td></td>
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<tr>
<td>Shear @ Right</td>
<td>1.91 k</td>
<td>0.29</td>
<td>1.91</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Center Defl</td>
<td>-1.996 in</td>
<td>-0.300</td>
<td>-1.996</td>
<td>-1.996</td>
<td>0.000</td>
</tr>
<tr>
<td>Left Cant Defl</td>
<td>0.000 in</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>Right Cant Defl</td>
<td>0.000 in</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>Query Defl @</td>
<td>0.000 ft</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>Reaction @ Left</td>
<td>1.91</td>
<td>0.29</td>
<td>1.91</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reaction @ Rt</td>
<td>1.91</td>
<td>0.29</td>
<td>1.91</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

FA calc'd per Eq, E2-1, K'Lr < Cc
Beam Passes Table B5.1, Fb per Eq, F1-1, Fb = 0.66 Fy

## Section Properties
C8X11.5

<table>
<thead>
<tr>
<th>Depth</th>
<th>Width</th>
<th>Weight</th>
<th>Weight</th>
<th>Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.000 in</td>
<td>2.260 in</td>
<td>11.48 #/ft</td>
<td>11.48 #/ft</td>
<td>11.48 #/ft</td>
</tr>
<tr>
<td>0.220 in</td>
<td>0.390 in</td>
<td>32.60 in4</td>
<td>32.60 in4</td>
<td>32.60 in4</td>
</tr>
<tr>
<td>0.220 in</td>
<td>0.390 in</td>
<td>8.150 in3</td>
<td>8.150 in3</td>
<td>8.150 in3</td>
</tr>
<tr>
<td>3.36 in2</td>
<td>3.36 in2</td>
<td>0.782 in3</td>
<td>0.782 in3</td>
<td>0.782 in3</td>
</tr>
</tbody>
</table>

Rev: 560350
User: KEN-06021209, Ver 5.6.1, 26-Oct-2002
(c)1983-2002 ENERCALC Engineering Software
Steel Beam Design

Description: Marquee #3 C8 Channel at Building for Live Load and Snow Load check

General Information

Steel Section: C8X11.5

<table>
<thead>
<tr>
<th>Pinned-Pinned</th>
<th>Fa</th>
<th>Elastic Modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load Duration Factor</td>
<td>33.00 ksi</td>
<td>29.000 ksi</td>
</tr>
</tbody>
</table>

Distributed Loads

| DL | 0.006 |
| LL | 0.130 |

Summary

Using: C8X11.5 section, Span = 27.00 ft, Fy = 33.00 ksi

<table>
<thead>
<tr>
<th>Moment</th>
<th>13.438 k-ft</th>
<th>13.447 k-ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>fb : Bending Stress</td>
<td>19.788 ksl</td>
<td>19.800 ksl</td>
</tr>
<tr>
<td>fb / Fb</td>
<td>0.999 : 1</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Shear</th>
<th>1.591 k</th>
<th>23.232 k</th>
</tr>
</thead>
<tbody>
<tr>
<td>fv : Shear Stress</td>
<td>1.131 ksl</td>
<td>13.200 ksl</td>
</tr>
<tr>
<td>fv / Fv</td>
<td>0.086 : 1</td>
<td></td>
</tr>
</tbody>
</table>

Beam OK Static Load Case Governs Stress

Max. Deflection: -1.865 in
Length/Deflt: 1.465.5 : 1
Length/(DL+LL Deflt): 173.7 : 1

Force & Stress Summary

<table>
<thead>
<tr>
<th>Maximum</th>
<th>DL Only</th>
<th>LL @ Center</th>
<th>LL+ST @ Center</th>
<th>LL @ Cants</th>
<th>LL+ST @ Cants</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max. M+</td>
<td>13.44 k-ft</td>
<td>1.59</td>
<td>13.44</td>
<td>k-ft</td>
<td></td>
</tr>
<tr>
<td>Max. M-</td>
<td>k-ft</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max. M @ Left</td>
<td>k-ft</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max. M @ Right</td>
<td>k-ft</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear @ Left</td>
<td>1.99 k</td>
<td>0.24</td>
<td>1.99</td>
<td>k</td>
<td></td>
</tr>
<tr>
<td>Shear @ Right</td>
<td>1.99 k</td>
<td>0.24</td>
<td>1.99</td>
<td>k</td>
<td></td>
</tr>
<tr>
<td>Center Defl.</td>
<td>-1.865 in</td>
<td>-0.221</td>
<td>-1.865</td>
<td>k-ft</td>
<td></td>
</tr>
<tr>
<td>Left Cant Defl.</td>
<td>0.000 in</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>Right Cant Defl.</td>
<td>0.000 in</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>...Query Defl @</td>
<td>0.000 in</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>Reaction @ Left</td>
<td>1.99</td>
<td>0.24</td>
<td>1.99</td>
<td>1.99</td>
<td>k</td>
</tr>
<tr>
<td>Reaction @ Rt</td>
<td>1.99</td>
<td>0.24</td>
<td>1.99</td>
<td>1.99</td>
<td>k</td>
</tr>
</tbody>
</table>

Fa calc'd per Eq. E2-1, KU< Cc
I Beam Passes Table B5.1, Fa per Eq. F1-1, Fb = 0.66 Fy

Section Properties

<table>
<thead>
<tr>
<th>Depth</th>
<th>8.000 in</th>
<th>Weight</th>
<th>11.48 #/ft</th>
<th>r-xx</th>
<th>3.106 in</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width</td>
<td>2.260 in</td>
<td>I-xx</td>
<td>32.60 in4</td>
<td>1.32 in4</td>
<td></td>
</tr>
<tr>
<td>Web Thick</td>
<td>0.220 in</td>
<td>I-yy</td>
<td>8.150 in3</td>
<td>0.782 in3</td>
<td></td>
</tr>
<tr>
<td>Flange Thickness</td>
<td>0.390 in</td>
<td>S-xx</td>
<td>3.106 in3</td>
<td>0.625 in</td>
<td></td>
</tr>
<tr>
<td>Area</td>
<td>3.38 in2</td>
<td>S-yy</td>
<td>11.48 #/ft</td>
<td>3.106 in</td>
<td></td>
</tr>
</tbody>
</table>
# Steel Beam Design

## General Information

Calculations are designed to AISC 9th Edition ASD and 1997 UBC Requirements.

### Steel Section: C8X11.5

<table>
<thead>
<tr>
<th>Component</th>
<th>Span</th>
<th>Load Duration Factor</th>
<th>Elastic Modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>Center Span</td>
<td>31.00 ft</td>
<td></td>
<td>29,000 ksi</td>
</tr>
<tr>
<td>Left Cant.</td>
<td>0.00 ft</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Right Cant.</td>
<td>0.00 ft</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Unbraced Length</td>
<td>0.00 ft</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

## Distributed Loads

<table>
<thead>
<tr>
<th>Load</th>
<th>#1</th>
<th>#2</th>
<th>#3</th>
<th>#4</th>
<th>#5</th>
<th>#6</th>
<th>#7</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL</td>
<td>0.006</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LL</td>
<td>0.094</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ST</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

## Summary

Using C8X11.5 section, span = 31.00 ft, Fy = 33.0 ksi, end fixity = Pinned-Pinned, Lu = 0.00 ft, LDF = 1.000

### Actual

- **Moment**: 13,392 k-ft
- **Shear**: 1,728 k

### Allowable

- **Moment**: 13,447 k-ft
- **Shear**: 23,232 k

### Static Load Case Governs Stress

- **Max. Deflection**: -2.450 in
- **Length/Def**: 968.2 / 1
- **Length/(DL + LL Def)**: 151.8 / 1

## Force & Stress Summary

<table>
<thead>
<tr>
<th>Load</th>
<th>Maximum</th>
<th>DL Only</th>
<th>@ Center</th>
<th>LL @ Center</th>
<th>LL@ Cants</th>
<th>LL+ST @ Cants</th>
<th>LL+ST @ Cants</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max. M+</td>
<td>13.39 k-ft</td>
<td>2.10</td>
<td>13.39</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max. M</td>
<td>k-ft</td>
<td></td>
<td>k-ft</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max. M @ Left</td>
<td>1.73 k</td>
<td>0.27</td>
<td>1.73</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max. M @ Right</td>
<td>1.73 k</td>
<td>0.27</td>
<td>1.73</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear @ Left</td>
<td>1.73 k</td>
<td>0.27</td>
<td>1.73</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear @ Right</td>
<td>1.73 k</td>
<td>0.27</td>
<td>1.73</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Center Defl.</td>
<td>-2.450 in</td>
<td>-0.384</td>
<td>-2.450</td>
<td>-2.450</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>Left Cant Defl</td>
<td>0.000 in</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>Right Cant Defl</td>
<td>0.000 in</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>Query Defl @</td>
<td>0.000 ft</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>Reaction @ Left</td>
<td>1.73 k</td>
<td>0.27</td>
<td>1.73</td>
<td>1.73</td>
<td>1.73</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reaction @ Rt</td>
<td>1.73 k</td>
<td>0.27</td>
<td>1.73</td>
<td>1.73</td>
<td>1.73</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Section Properties - C8X11.5

- **Depth**: 8.00 in
- **Weight**: 11.48 #/ft
- **r-xx**: 3.106 in
- **Width**: 2.280 in
- **I-xx**: 32.60 in
- **r-yy**: 0.625 in
- **Web Thick**: 0.220 in
- **I-yy**: 1.32 in
- **Flange Thickness**: 0.390 in
- **S-xx**: 8.150 in
- **Area**: 3.38 in
- **S-yy**: 0.782 in
ROOF DIAPHRAGM FLEXIBILITY LIMITATION

(IEBCO REPORTER-2767)

FLEXIBILITY FACTOR FOR 8'-6" SPAN:

\[ F = 18.25 + 181/3 = 62 \text{ (MAX)} < 70 \]

SPAN-DEPTH LIMITATION FOR FLEXIBLE WALLS,

ROTATION IN DIAPHRAGM NOT CONSIDERED - 411

DIAPHRAGM MAX. SPAN = 4' x 8.50 = 34.00 F < 60 F

FLEXIBILITY FACTOR FOR 5'-6" SPAN:

\[ F = 17.57 + 181/3 = 81 \text{ (MAX)} < 150 \]

SPAN-DEPTH LIMITATION - 75:1

DIAPHRAGM MAX. SPAN = 3' x 5.83 = 17.50 F < 60 F

CONCLUSION:
METAL DECK EXCEEDS FLEXIBLE DIAPHRAGM LIMITATIONS.
### TABLE 28 – ALLOWABLE SHEAR AND FLEXIBILITY FACTORS ON DIAPHRAGMS USING ASC3 (24) ELECTRIC-DECK® FLOOR SYSTEM WITH TRENCH HEADER1,2,3,4,5

<table>
<thead>
<tr>
<th>Deck Gage Combination</th>
<th>Base Metal Thickness (Inches)</th>
<th>Allowable Diaphragm Shear (PLF)</th>
<th>Flexibility Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Cellular ASC3(24)</td>
<td>FLUTED FLAT NON-CELLULAR 3W36</td>
<td>F₀  F₁</td>
</tr>
<tr>
<td>20/20</td>
<td>20 through 16</td>
<td>0.035 0.035 0.035 through 0.059</td>
<td>1670 0.42 0.89</td>
</tr>
<tr>
<td>20/18</td>
<td>20 through 16</td>
<td>0.035 0.047 0.035 through 0.059</td>
<td>1670 0.42 0.89</td>
</tr>
<tr>
<td>20/16</td>
<td>20 through 16</td>
<td>0.035 0.059 0.035 through 0.059</td>
<td>1670 0.42 0.89</td>
</tr>
<tr>
<td>18/20</td>
<td>20 through 16</td>
<td>0.047 0.047 0.035 through 0.059</td>
<td>1670 0.42 0.89</td>
</tr>
<tr>
<td>18/16</td>
<td>20 through 16</td>
<td>0.047 0.059 0.035 through 0.059</td>
<td>1670 0.42 0.89</td>
</tr>
<tr>
<td>16/16</td>
<td>20 through 16</td>
<td>0.059 0.059 0.035 through 0.059</td>
<td>1670 0.42 0.89</td>
</tr>
<tr>
<td>None</td>
<td>None</td>
<td>None</td>
<td>1710 0.42 1.5</td>
</tr>
</tbody>
</table>

For Si: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 psi = 6.9 N/mm, 1 psi = 6894 Pa.

1Maximum deck span is 12 feet.
2Maximum trench header width is 36 inches.
3See Figure 2 for deck support weld patterns.
4Concrete fil is 2 1/2 inches minimum above top of deck flutes. Concrete is normal-weight concrete with a minimum compressive strength of 3000 psi at 28 days and is reinforced with 6 x 6 W14 x W14 welded-wire fabric complying with ASTM A166. The fabric is centered within the concrete fill.
5Flexibility factor value is the average micro inches a diaphragm web will deflect in a span of 1 foot under a shear of 1 pound per foot. See Table 29.

\[ F₀ = \text{Flexibility factor for concrete.} \]

\[ F₁ = \text{Flexibility factor for trench.} \]

For decks with trench headers, \( \Delta \text{w} \) is equal to the calculated deflection of the trench header added to the deflection of the concrete outside of the trench.

### TABLE 29 – DIAPHRAGM FLEXIBILITY LIMITATION1,2,3,4,5

<table>
<thead>
<tr>
<th>( F )</th>
<th>Maximum Span in Feet for Masonry or Concrete Walls</th>
<th>Span-Depth Limitation</th>
</tr>
</thead>
<tbody>
<tr>
<td>More than 150</td>
<td>Not used</td>
<td>Rotation Not Considered in Diaphragm: Masonry or Concrete Walls Flexible Walls(^a)</td>
</tr>
<tr>
<td>70-150</td>
<td>200</td>
<td>2:1 or as required for deflection</td>
</tr>
<tr>
<td>10-70</td>
<td>400</td>
<td>2:1 or 1 as required for deflection</td>
</tr>
<tr>
<td>1-10</td>
<td>No limitation</td>
<td>4:1 As required for deflection</td>
</tr>
<tr>
<td>Less than 1</td>
<td>No limitation</td>
<td>As required for deflection</td>
</tr>
</tbody>
</table>

For Si: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 psi = 14.594 N/mm, 1 psi = 6894 Pa.

1Diaphragms are to be investigated regarding their flexibility and recommended span-depth limitations. Refer to above table for determination of value of \( F \).
2Diaphragms supporting masonry or concrete walls are to have their deflections limited to the following amount:

\[ \Delta \text{ Wall} = \frac{H^2 \epsilon}{0.01E} \]

Where:

- \( H \) = Unsupported height of wall in feet.
- \( \epsilon \) = Thickness of wall in inches.
- \( E \) = Modulus of elasticity of wall material for deflection determination in pounds per square inch.
- \( f_k \) = Allowable compressive strength of wall material in flexure in pounds per square inch. For concrete, \( f_k = 0.45 \epsilon \). For masonry, \( f_k = F_0 = 0.33 \epsilon \).

3The total deflection \( \Delta \) of the diaphragm may be computed from the equation: \( \Delta = \Delta_s + \Delta_w \).

Where:

\[ \Delta_s = \text{Flexural deflection of the diaphragm determined in the same manner as the deflection of beams} \]

\[ \Delta_w = \text{The web deflection may be determined by the equation:} \]

\[ \Delta_w = \frac{C_{sw} L \epsilon}{10^4} \]

Where:

- \( L \) = Distance in feet between vertical resisting element (such as shear wall) and the point to which the deflection is to be determined.
- \( C_{sw} \) = Average shear in diaphragm in pounds per foot over length \( L \).
- \( \epsilon \) = Flexibility factor. The average microinches a diaphragm web will deflect in a span of 1 foot under a shear of 1 pound per foot.

4When applying these limitations to cantilevered diaphragms, the allowable span-depth ratio will be half that shown.

5Diaphragm classification (flexible or rigid) and deflection limits shall comply with Section 2.19.4.
WIND LOAD EVALUATION

POST SECTION - SIDE VIEW

TRIBUTARY HEIGHT = 1' - 0" RAILING
= 1' - 0" * 4/12

WIND LOAD PER ROOF DIAPHRAGM =

= (11.20 * (4.00 + 1.00)) = 11.20 * (5.00) = 56.00 psi

CHECK 5' - 10" WIDE ROOF DIAPHRAGM FOR 67.20!

SPAN AT MARQUEE # 3:

\[ \text{SPAN AT MARQUEE} \]
Wind Load = 56 plf

CBU 1/2
CBU 1/2
CBU 1/2

Top Seam Weld

226 A x 1 1/2" Roof Deck

1/2" Puddle Weld @ 12" OC

(Deck to L 4 x 4)

L 4 x 4 x 1/4 @ Round Poles

1 x 1 = 2(122 + 3.38 x 34.29) = 8016 in^4

Sw = 8016 / 35 = 229 in^3

M Wind = 56000 x 67.70^2 / 8 = 32083 ft-lb

Bending Stress \( \frac{F_b}{229} = 1.83 \leq 1.33 \)

\( F_b = 1.33 \times 0.60 \times 33,000 = 26,334 \text{ psi} \)

Wind Load Deflection:

\[ \Delta \text{Wind} = \frac{5 \times 56000 \times 67.70^2 \times 123}{384 \times 29 \times 10^6 \times 8016} = 0.1114 = L \times 7.135 \]

Wind Load Reaction = 56000 / 67.70 / 2 = 1896 #

Deck Shear = 1896 / 5.83 = 325 plf
RECOMMENDED 1/2 IN DIA. Puddle Welds Parallel

To Deep Flute's Spacing (F) = \( \frac{25,000}{V} = \frac{25,000 \times 0.0299}{325} = 2.94'' \)

Use 12 in Welds spacing.

12 in Dia. Wood Pole Bending:

\[ \text{Design Wind Load} = \frac{W}{16} \]

\[ W = \frac{2252 \times 10 \times 12}{37} = 37,920 \text{ lb} \]

\[ M = \frac{37,920 \times 10}{3} = 126,400 \text{ in-lb} \]

\[ \text{Pole Average Circumference} = \frac{37.5 + 37.00}{2} = 37.25 \text{ in} \]

\[ \text{Pole Diameter at base} = \frac{37.25}{\pi} = 12.18 \text{ in} \]

Use 12 in Pole Dia.: \( A = \frac{\pi \times 12^2}{4} = 113 \text{ in}^2 \)

\[ I = \frac{\pi \times 12^4}{64} = 1,017 \text{ in}^4 \]

\[ S_m = \frac{\pi \times 12^3}{3} = 170 \text{ in}^3 \]

\[ \text{Bending Stress} \ F_b = \frac{37,920 \times 12}{170} = 2,677 \text{ psi} \]

\[ \text{Allowable Bending Stress} \ F_a = (1.6)(1,850) = 2,960 \text{ psi} \]

\( \text{(See p. 1)} \) OK

\[ \text{Pole Wind Load Deflection} = \frac{37,920 \times 10^3 \times 12^3}{3 \times 1.5 \times 10^6 \times 10^7} = 1.42 \text{ in} \]
DEPTH OF POLE EMBEDMENT REQUIRED

\[ d^2 = \frac{4.25 \times 3.792 \times 100}{3200 \times 1.0} = 7.10' \]

**Conclusion:**

12" DIA. ROUND POLES NEED TO BE EMBEDDED 8' INTO THE GROUND FOR WIND LOAD INCLUDING RAILING.

**Consider Railing to be Removed:**

TRIBUTARY HEIGHT = 11'-2"

WIND LOAD = 11.2 psf \times 1.17 = 13.10 psf

WIND LOAD PER POLE = 13.10 \times 67.2f = 900 lbs

\[ d_1 = \sqrt{\frac{4.25 \times 900 \times 10}{1800 \times 1}} = 4.60' \]

**Conclusion:**

IF THE RAILING IS REMOVED, 12" DIA. ROUND POLES NEED TO BE EMBEDDED 5'-0" INTO THE GROUND.
SCREENING PHASE - TIER 1

BENCHMARK BUILDING:

THE STRUCTURE HAS BEEN DESIGN AND CONSTRUCTED IN 1971-72, PRIOR TO UBC-1976. THEREFORE, EVOLUTION OF CHECKLISTS IS REQUIRED.

PER ASCE 31-03, TWO 3-2 CHECKLISTS REQUIRED FOR TIER 1 EVALUATION ARE:

BASIC STRUCTURAL (SEC. 3.7)
SUPPLEMENTAL STRUCTURAL (SEC. 3.7)
GEOLIC SITE HZABAR AND FOUNDATION (SEC. 3.8)
Tier 1 Analysis

Pseudo Lateral Force (Sec. 3.5.2.1)

\[ V = C \cdot S_a \cdot W \], WHERE

\[ C = 1.35 \quad (Table \ 3-4) \]

\[ S_a = \frac{S_{DI}}{T} \quad AND \quad SHALL \ NOT \ EXCEED \ T/2 \]

\[ S_{DI} = \frac{F_{V} \cdot V_{I}}{2} \]

\[ S_{AX} = \frac{F_{A} \cdot S_{a}}{2} \]

\[ S_{a} = 2 \times 1.5 \times 0.5 = 0.50 \]

\[ S_{AX} = \frac{2}{3} \times 1.0 \times 1.5 = 1.00 \]

\[ T = C \cdot h_{0} \cdot B = (0.060)(10)^{0.75} = 0.204 \]

\[ S_a = \frac{0.50}{0.204} = 1.47 \quad USE \quad S_a = S_{AX} = 1.00 \]

Weight of 67.70 Structure:

OSGC

\[ W = (2)(8.50)(67.70) + (2)(11.5)(67.70) + \]

12" O.D. Pipe

\[ = (56.5)(1785)(18.00)(2) + (36.5)(0.35)(10.00)(2) + \]

Cont. Beam

Knee Brace

\[ = (36.5)(0.52)(67.70) + (36.5)(0.37)(6.12)(8) + \]

Pausing

\[ = (36.5)(1.27)(67.70 + 0.04)(67.70) = \]

\[ = 1181 + 1.357 + 1031 + 256 + 1235 + 661 + 519 = \]

\[ = 6460 \text{ ft } / 6770 = 96 \text{ plf} \]
\[ V = 1.3 \times 1.0 \times 0.460 = 8.398 \text{ kN} \]

**Roof Diaphragm Check (See p. 16)**

**Force-Controlled Action (See p. 2)**

**EQ Force per L.F. of Diaphragm**

\[ \sqrt{V^2 + \frac{W}{M}} = \frac{8.398}{1.3 \times 0.5 \times 0.7} = 3.38 \text{ kN} \]

\[ M = 3.38 \times 0.7 \times 0.7 / 8 = 0.285 \text{ kN-m} \]

**Nominal Strength of Diaphragm in Bending**

\[ M = (19,800 \times 17)(280) / 12 = 6,423 \text{ kN-m} \]

\[ Q_{CH} = 6,423 / 2.45 = 2,656 > Q_{UF} = 0.1 \text{ kN-m} \]

**Wood Pole Bending (Transverse Direction)**

**EQ**

**Force-Controlled Action**

\[ EQ \text{ Force: } (28.20)(0.720) = 20.58 \text{ kN} \]

\[ M = 20.58 \times 10.00 = 205.8 \text{ kN-m} \]

**Pole Nominal Strength in Bending**

\[ M = (1850 \times 2)(170) / 12 = 52,416 \text{ kN-m} \]

\[ Q_{CH} = 52,416 > Q_{UF} = 25,860 \text{ kN-m} \]

A seismic force is less than wind when pailing is in place, but is more if it is removed.
Pole EQ Deflection Check:

Deformation - Controlled Action - Use the Full EQ Load (ASCE/SEI 791-03, Sec. 4.2.4.3.1).

EQ Force = \( 8398 \text{ ft} \) (p. 01)

\[ \Delta EQ = \frac{8398 \times 10^3 \times 12^3}{3 \times 1.5 \times 10^6 \times 1017} = 2.1714 \text{ in.} \]

Depth of Pole Embedment

Per ASCE/SEI 791-03, Sec. 4.2.4.3.4 use \( R_o = 8 \) for actions at foundation/soil interface for life safety.

\[ d = \sqrt{\frac{12.5 \times (8398/8) \times 10}{2000 \times 1}} = 4.76 \text{ ft} \]

Conclusion:

With railing removed, the EQ Embedment requirement is the same as for wind.
WOOD POLE BENDING (LONGITUDINAL DIRECTION):
CASE 1: CONSIDER KNEE-BRACE IN COMPRESSION ONLY

MARINER - FRONT VIEW
SCALE: 1/"=0'

(1) 5/8" #. LAG SCREW

- 11/2 5/8" #. LAG SCREWS

DESIGN LOADS:

\[ Q_D = \frac{(96)(200)}{1000} = 19.20 \text{ kN} \]

\[ Q_L = \frac{(75)(8.50)(200)(0.25)}{1000} = 3.673 \text{ kN} \]

\[ Q_C = \frac{11(1920 + 3613)}{1000} = 6.087 \text{ kN} \] (Sec. 4.2.4.2)

\[ Q_{UF} = 2.586 \times 5.69 = 14.714 \text{ kN} \] (Sec. 4.7.4.1)

\[ Q_C = (2)(1.081.56)(113) = 244.48 \text{ kN} \] (Sec. 4.13)

\[ Q_{CH} = 52.416 \text{ kN} \] (p.21)
Timber Column Design

Description: Wood Pole Compression

Calculations are designed to 1997 NDS and 1997 UBC Requirements

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Column Height</td>
<td>10.00 ft</td>
</tr>
<tr>
<td>Load Duration Factor</td>
<td>1.00</td>
</tr>
<tr>
<td>Fc</td>
<td>1,000.00 psi</td>
</tr>
<tr>
<td>Eb</td>
<td>1,850.00 psi</td>
</tr>
<tr>
<td>E - Elastic Modulus</td>
<td>1,500 ksi</td>
</tr>
<tr>
<td>Allowable (NDS-2001)</td>
<td></td>
</tr>
</tbody>
</table>

### Loads

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial Load</td>
<td>1,920.00 lbs</td>
</tr>
<tr>
<td>Eccentricity</td>
<td>0.000 in</td>
</tr>
<tr>
<td>Live Load</td>
<td>3,613.00 lbs</td>
</tr>
<tr>
<td>Short Term Load</td>
<td>0.00 lbs</td>
</tr>
</tbody>
</table>

### Summary

Using = 12.00in round column, Total Column H= 10.00ft

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>fc : Compression</td>
<td>48.92 psi</td>
</tr>
<tr>
<td>Fc : Allowable</td>
<td>1,081.56 psi</td>
</tr>
<tr>
<td>fbx : Flexural</td>
<td>0.00 psi</td>
</tr>
<tr>
<td>Fbx : Allowable</td>
<td>2,183.00 psi</td>
</tr>
<tr>
<td>Interaction Value</td>
<td>0.0452</td>
</tr>
</tbody>
</table>

### Column OK

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dl + LL</td>
<td>48.92 psi</td>
</tr>
<tr>
<td>Dl + LL + ST</td>
<td>48.92 psi</td>
</tr>
<tr>
<td>Dl + ST</td>
<td>16.98 psi</td>
</tr>
</tbody>
</table>

### Stress Details

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fc : X-X</td>
<td>1,081.56 psi</td>
</tr>
<tr>
<td>Fc : Y-Y</td>
<td>1,081.56 psi</td>
</tr>
<tr>
<td>Fc : Allowable</td>
<td>1,081.56 psi</td>
</tr>
<tr>
<td>Fc : Allow * Load Dur Factor</td>
<td>1,081.56 psi</td>
</tr>
<tr>
<td>Fbx</td>
<td>2,183.00 psi</td>
</tr>
<tr>
<td>Fbx : Allowable</td>
<td>1,081.56 psi</td>
</tr>
<tr>
<td>Fbx : Allow * Load Dur Factor</td>
<td>2,183.00 psi</td>
</tr>
</tbody>
</table>

For Bending Stress Calcs...

- Max k*Lu / d | 43.00
- Actual k*Lu/d | 22.54
- Min. Allow k*Lu / d | 9.00
- Cf:Bending | 1.00
- Rb : (Le d / b^2) ^ .5 | 14.859

For Axial Stress Calcs...

- Cf : Axial | 1.150
- Axial X-X k Lu / d | 10.00
- Axial Y-Y k Lu / d | 10.00
COMPRESSION AND BENDING COMBINED:

\[
\frac{6,087}{244,482} + \frac{14,714}{58,416} = 0.261 < 1.00 \quad \text{O.K.}
\]

KNEE-BRACE COMPRESSION CHECK:

\[
Q_{uf} = 25.36 \times 1.41 = 35.96 \text{ kN}
\]

\[
Q_{ch} = (2)(35.96) \times 52.25 = 71.58 \text{ kN}
\]

\[
Q_{ch} = 71.58 < Q_{uf} = 35.96 \quad \text{O.K.}
\]

BEARING AT 45° ANGULAR CHECK:

\[
F_\theta = \frac{700 \times 3.85}{700 \times 8.18 \times 2.25 + 675 \times 2.24 \times 2.45} = 600.18 \text{ kN}
\]

\[
Q_{uf} = 35.96 \text{ kN}
\]

\[
Q_{ch} = (2)(600.18)(73.67) = 97,844 \text{ kN}
\]

\[
Q_{ch} > Q_{uf} \quad \text{O.K.}
\]
# Timber Column Design

**Description:** Knee-Brace in Compression check

## General Information

<table>
<thead>
<tr>
<th>Wood Section</th>
<th>Total Column Height</th>
<th>Le XX for Axial</th>
</tr>
</thead>
<tbody>
<tr>
<td>6X10 FD#2</td>
<td>6.08 ft</td>
<td>6.08 ft</td>
</tr>
<tr>
<td>Load Duration Factor</td>
<td>1.00</td>
<td>Le YY for Axial</td>
</tr>
<tr>
<td>Fc</td>
<td>700.00 psi</td>
<td>Lu XX for Bending</td>
</tr>
<tr>
<td>Fb</td>
<td>750.00 psi</td>
<td>6.08 ft</td>
</tr>
<tr>
<td>E - Elastic Modulus</td>
<td>1,300 ksi</td>
<td></td>
</tr>
</tbody>
</table>

## Loads

<table>
<thead>
<tr>
<th>Axial Load</th>
<th>Dead Load</th>
<th>Live Load</th>
<th>Short Term Load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.00 lbs</td>
<td>0.00 lbs</td>
<td>3,646.00 lbs</td>
</tr>
<tr>
<td></td>
<td>0.00 in</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

## Summary

Using: 6X10 FD#2, Width= 5.50in, Depth= 9.50in, Total Column Ht= 6.08ft

<table>
<thead>
<tr>
<th></th>
<th>DL + LL</th>
<th>DL + LL + ST</th>
<th>DL + ST</th>
</tr>
</thead>
<tbody>
<tr>
<td>fcb: Compression</td>
<td>0.00 psi</td>
<td>69.78 psi</td>
<td>69.78 psi</td>
</tr>
<tr>
<td>Fc: Allowable</td>
<td>646.71 psi</td>
<td>646.71 psi</td>
<td>646.71 psi</td>
</tr>
<tr>
<td>fb: Flexural</td>
<td>0.00 psi</td>
<td>747.59 psi</td>
<td>747.59 psi</td>
</tr>
<tr>
<td>F'bx: Allowable</td>
<td>747.59 psi</td>
<td>747.59 psi</td>
<td>747.59 psi</td>
</tr>
</tbody>
</table>

**Interaction Value:** 0.0000 0.1079 0.1079

**Column OK**

## Stress Details

- **Fcb: X-X**: 684.21 psi
- **Fcb: Y-Y**: 646.71 psi
- **Fcb: Allowable**: 646.71 psi
- **F'bc: Allow * Load Dur Factor**: 646.71 psi
- **F'bx**: 747.59 psi
- **F'bx: Load Duration Factor**: 747.59 psi

**For Bending Stress Calcs...**

- Max k*Lu / d: 50.00
- Actual k*Lu/d: 28.92
- Min. Allow k*Lu / d: 11.00
- Cf:Bending: 1.000
- Rb: (Le d / b*2) ^ .5: 6.787

**For Axial Stress Calcs...**

- Cb: Axial: 1.000
- Axial X-X k Lu / d: 7.68
- Axial Y-Y k Lu / d: 13.27
KNEE-BRACE CONNECTION TO 12" DIA. POLE:

\[ Q_{UF} = 3.646 \text{ #} \ (P.25) \]

For 5/8" Ø LAG SCREW, \( B = 8 \times 0.625 = 5" \) penetration into pole, \( G = 0.50 \) (ND8-2001, Table 11.3.2A), \( Z_{m1} = 420 \# \)

For (2) 5/8" Ø LAG SCREWS PER CONNECTION:

\[ Q_{CH} = (2)(2)(420) = 1680 \# \]

\[ Q_{CH} = 1.680 \# < Q_{UF} = 3.646 \# \quad \text{N.G.} \]

KNEE-BRACE CONNECTION TO 4X6 BLOCKING:

\[ Z_{m2} = \frac{640 \times 420}{640 \times 60^2 - 450^2 + 420 \times 95^2} = 307 \# \]

The actual end distance = 1.5 in

Min. end distance req'd for full design value:

\[ AID = 4 \times 0.625 = 2.5 \text{ in} \quad \text{(ND8-2001, Table 11.5.1B)} \]

Adjustment factor = 1.5/2.5 = 0.60

\[ Q_{CH} = (0.60)(0.6)(307) = 608 \# \]

\[ Q_{CH} = 608 \# < Q_{UF} = 3.646 \# \quad \text{N.G.} \]
WOOD POLE BENDING (LONGITUDINAL DIRECTION)

CASE 2: CONSIDER ONE KNEE-BRACE IN COMPRESSION AND A SECOND KNEE BRACE IN TENSION

\[ F_{comp} = 36.46 \times 0.5 = 18.23 \, \text{lb} \]
\[ F_{tens} = 18.23 \, \text{lb} \]

**Knee Brace in Tension Check:**

\[ Q_{UF} = 18.23 \, \text{lb} \]
\[ p_{1} = 0.5 \]
\[ Q_{CH} = 2(475)(52.25) = 49,637 \, \text{lb} \]
\[ Q_{CH} = 49,637 > Q_{UF} = 18.23 \, \text{lb} \]

OK.

**Knee Brace Connection to 12" Dia. Pole:**

\[ Q_{CH} = 1,680 \, \text{lb} < Q_{UF} = 18.23 \, \text{lb} \]

NG.

**Knee Brace Connection to 4x6 Blocking:**

\[ Q_{CH} = 1,217 \, \text{lb} < Q_{UF} = 18.23 \, \text{lb} \]

NG.

**Conclusion:**

Wind per round pole w/ railing = 37.12 lb
Wind per round pole w/o railing = 900 lb
Seismic force per round pole for bending = 25.86 lb
Seismic force per round pole for embankment check = 8398/8 = 1050 lb
INTERMEDIATE POST CHECK.

DESIGN LOADS:

\[ P_{DL} = 14,920 \text{ lb} \quad \text{(p. 23)} \]
\[ P_{UL} = 36/15 \times 25 = 14,452 \text{ lb} \quad \text{(p. 23)} \]
\[ \Sigma P = 14,920 + 14,452 = 29,372 \text{ lb} \]

AREA \( A_1 \) = 1.875 \times 8.125 = 15.631 \text{ in}^2

AREA \( A_2 \) = 1.875 \times 7.687 = 14.591 \text{ in}^2

\[ \Sigma A = 15.631 + 14.591 = 30.222 \text{ in}^2 \]

DUE TO WOOD DETERIORATION

USE 50\% OF POST AREA =

\[ 0.50 \times 30.222 = 15.111 \text{ in}^2 \]

COLUMN STABILITY FACTORS:

\[ C_p = \frac{1 + \left( \frac{F_{ce}}{F_c^*} \right)^2}{2} \left[ 1 + \left( \frac{F_{ce}}{F_c^*} \right)^2 \right] = \frac{F_{ce}}{F_c^*} \]

\[ F_c^* = 10 \times 13.50 = 135.0 \text{ psi} \]

\[ F_{ce} = \frac{K_0 \cdot E^*}{(b \cdot l)^2} = \frac{0.3 \times 15 \times 10^6}{(10 \times 127)^2} = 1966 \text{ psi} \]

\[ C = 0.8 \quad \text{FOR DRY WOOD LUMBER} \]

\[ \frac{F_{ce}}{F_c^*} = \frac{1966}{1350} = 1.45 \]

\[ C_p = \frac{1 + 1.45}{2 \times 8} - \sqrt{\frac{1 + 1.45}{2 \times 8}^2 - 1.45} = 0.8 \]

\[ 1.53 - 0.73 = 0.80 \]
FOR ELEMENT 1

THE ALLOWABLE COMPRESSION

\[ \text{Force} = 0.80 \times 1350 \times 16.75 = 15.090 \text{ kips} \]

\[ \text{P} = 16.372 \text{ kips} \]

0.9

CONCLUSION:
EVEN WITH 50% OF EFFECTIVE AREA, BUILT-UP
POST HAS SUFFICIENT CAPACITY TO SUPPORT
COMBINED DEAD LOAD AND LIVE LOAD OF:

\[ \frac{18,070}{4.25} \times 23.66 = 180 \text{ psf} \]
CHANNEL C8 CONNECTION TO 12 IN. DIA. POLE

CASE 1: DL + LL ONLY

DESIGN MAX. DL + LL = 2,640 = 2,640# (75%)

FOR (2) 5/8" DIAM. X 8" = 8 x .625 = 5.14 PENETRATION

LAG SCREWS, ALLOWABLE SHEAR = \( \frac{(2)(1750)}{1.5} = 1100#\)

REQ'D: (2) - 7/8" X LAG SCREWS, ALLOWABLE

SHEAR = \( \frac{(2)(1410)}{1.5} = 2,820# \) (HDG-2001, TABL 11.1)

CASE 2: DL + LL + EDP

\[ \begin{align*}
DL &= 210# \quad (55) \\
LL &= (0.25)/(2437) = 608# \\
Q_G &= 1.1 \times (210 + 608) = 1.1 \times 818 = 900# - SHEAR \\
Q_{CN} &= 2 \times 1230 = 2,460# \\
Q_{UF} &= 2586# - WITHDRAWAL
\end{align*} \]

FOR (2) 5/8" DIAM. X 5IN PENETRATION LAG SCREWS:

\[ Q_{CN} = \frac{(2)(2)(447)(5)}{2} = 8,940# \]

DUE TO WOOD DETERIORATION USE 50% OF DESIGN VALUE FOR WITHDRAWAL.

SHEAR AND WITHDRAWAL COMBINED:

\[ \frac{900}{2560} + \frac{2586}{0.5 \times 8940} = 0.35 + 0.58 = 0.93 < 1.00 \]

O.K.
3 IN. DIA. STEEL POST CHECK

DESIGN AXIAL LOADS AT MARQUEE #3:

Deck: 6 x 11.5

P_D = (0.009)(8.5/2)(60.0/2) + (0.015)(13) = 1.50K

P_L = (0.009)(8.5/2)(60.0/2) = 5.10K

Steel post is OK - see "Enercalc" output

R. 2.23

CONCLUSION:
3" DIA. STEEL POST CAPACITY IS 230 PSF FOR 8'-6" MARQUEE.
3" DIA. STEEL POST CAPACITY IS 235 PSF FOR 10'-10" MARQUEE.

Both capacities exceed capacity of 6' x 11.5 C channel supporting by 3" dia. steel post and the capacity of metal deck supporting by channel.

CONCLUSION FOR STEEL CHANNEL CONNECTION TO THE BOUND POLE:
Capacity of 2-5/8" dia. lag screws connecting the steel channel to the bound pole is 180 PSF for 8'-6" wide marquees and 260 PSF for 10'-10" wide marquees.

If knee braces would be removed the whole loads would set the limits of marquees capacity.
Steel Column

Calculations are designed to AISC 9th Edition ASD and 1997 UBC Requirements

<table>
<thead>
<tr>
<th>General Information</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel Section: P3STD</td>
</tr>
<tr>
<td>Fy: 33.00 ksi</td>
</tr>
<tr>
<td>X-X Sidesway: Restrained</td>
</tr>
<tr>
<td>X-Y Sidesway: Restrained</td>
</tr>
<tr>
<td>Duration Factor: 1.00</td>
</tr>
<tr>
<td>Elastic Modulus: 29,000.00 ksi</td>
</tr>
<tr>
<td>Column Height: 9.000 ft</td>
</tr>
<tr>
<td>End Fixity: Pin-Pin</td>
</tr>
<tr>
<td>X-X Unbraced: 9.000 ft</td>
</tr>
<tr>
<td>Kxx: 1.000</td>
</tr>
<tr>
<td>Y-Y Unbraced: 9.000 ft</td>
</tr>
<tr>
<td>Kyy: 1.000</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial Load:</td>
</tr>
<tr>
<td>Dead Load: 1.50 k</td>
</tr>
<tr>
<td>Live Load: 5.10 k</td>
</tr>
<tr>
<td>Ecc. for X-X Axis Moments: 0.000 in</td>
</tr>
<tr>
<td>Ecc. for Y-Y Axis Moments: 0.000 in</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Summary</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section: P3STD, Height = 9.000 ft, Axial Loads: DL = 1.50, LL = 5.10, ST = 0.00 k, Ecc. = 0.000 in</td>
</tr>
<tr>
<td>Unbraced Lengths: X-X = 9.000 ft, Y-Y = 9.000 ft</td>
</tr>
<tr>
<td>Combined Stress Ratios</td>
</tr>
<tr>
<td>AISG Formula H1 - 1: 0.1740</td>
</tr>
<tr>
<td>AISG Formula H1 - 2: 0.1155</td>
</tr>
<tr>
<td>AISG Formula H1 - 3: 0.0512</td>
</tr>
</tbody>
</table>

| Column Design OK |

XX Axis: Fa calc'd per Eq. E2-1, K*L/r < Cc
YY Axis: Fa calc'd per Eq. E2-1, K*L/r < Cc

<table>
<thead>
<tr>
<th>Stresses</th>
</tr>
</thead>
<tbody>
<tr>
<td>Allowable &amp; Actual Stresses</td>
</tr>
<tr>
<td>Fa: Allowable: 13.15 ksi</td>
</tr>
<tr>
<td>fa: Actural: 0.67 ksi</td>
</tr>
<tr>
<td>Fb:xx: Allow [F3.1]: 21.78 ksi</td>
</tr>
<tr>
<td>fb:xx: Actural: 0.00 ksi</td>
</tr>
<tr>
<td>Fb:yy: Allow [F3.1]: 21.78 ksi</td>
</tr>
<tr>
<td>fb:yy: Actural: 0.00 ksi</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Analysis Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fex: DL+LL: 17,338 psi</td>
</tr>
<tr>
<td>Fey: DL+LL: 17,338 psi</td>
</tr>
<tr>
<td>Fex: DL+LL+ST: 17,338 psi</td>
</tr>
<tr>
<td>Max X-X Axis Deflection: 0.000 in at 0.000 ft</td>
</tr>
<tr>
<td>Max Y-Y Axis Deflection: 0.000 in at 0.000 ft</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Section Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter: 3.50 in</td>
</tr>
<tr>
<td>Weight: 7.57 #/ft</td>
</tr>
<tr>
<td>Area: 2.23 in²</td>
</tr>
<tr>
<td>Thickness: 0.216 in</td>
</tr>
</tbody>
</table>

---

Note: The document contains detailed calculations and requirements for structural steel columns, including load considerations, section properties, and analysis values. The calculations are based on the American Institute of Steel Construction (AISC) 9th Edition ASD and 1997 Uniform Building Code (UBC) requirements.
CHANNEL CB CONNECTION TO BUILDING

CASE 4: AT WOOD STUD WALL.

DESIGN LOADS FOR 2'- 8 5/16" SCREW SPACING

FOR 1/2" LAG SCREWS:

\[
DL + UL = \left[ \frac{19 + 40(8.50/2) + 11.50}{2} \right] \leq (270) = 219.50
\]

\[
=(219 + 11.50)(2.20) = 593 \text{ ft} (SHEAR)
\]

PE2 NDS - 2001, Table 11K:

\[
Z_{||} = 520 \text{ ft}, \; Z_{\perp} = 320 \text{ ft}
\]

\[
Z_{\perp} = 520 \times 1.15 = 593 \text{ ft} > 593 \text{ ft} \text{ (FOR SNOW LOAD)}
\]

CONCLUSION:

ASSUMING THAT 1/2" DIA. LAG SCREW IS LOADING WOOD SUPPORT PARALLEL TO GRAIN, THE CAPACITY OF THE SCREWS IS 401SF FOR LIVE LOAD FOR 8'-6" METAL DECK AND 58.0SF FOR 5'-6" DECK
3.7.2 Basic Structural Checklist for Building Type W2: Wood Frames, Commercial and Industrial

This Basic Structural Checklist shall be completed where required by Table 3-2.

Each of the evaluation statements on this checklist shall be marked Compliant (C), Non-compliant (NC), or Not Applicable (N/A) for a Tier 1 Evaluation. Compliant statements identify issues that are acceptable according to the criteria of this standard, while non-compliant statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated. For non-compliant evaluation statements, the design professional may choose to conduct further investigation using the corresponding Tier 2 Evaluation procedure; corresponding section numbers are in parentheses following each evaluation statement.

C3.7.2 Basic Structural Checklist for Building Type W2

These buildings are commercial or industrial buildings with a floor area of 5,000 square feet or more. There are few, if any, interior walls. The floor and roof framing consists of wood or steel trusses, glulam or steel beams, and wood posts or steel columns. Lateral forces are resisted by wood diaphragms and exterior stud walls sheathed with plywood, oriented strand board, stucco, plaster, straight or diagonal wood sheathing, or braced with rod bracing. Wall openings for storefronts and garages, where present, are framed by post-and-beam framing.

Building System

(C) NC N/A LOAD PATH: The structure shall contain a minimum of one complete load path for Life Safety and Immediate Occupancy for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation. (Tier 2: Sec. 4.3.1.1)

C NC (N/A) MEZZANINES: Interior mezzanine levels shall be braced independently from the main structure, or shall be anchored to the lateral-force-resisting elements of the main structure. (Tier 2: Sec. 4.3.1.3)

C NC (N/A) WEAK STORY: The strength of the lateral-force-resisting system in any story shall not be less than 80 percent of the strength in an adjacent story, above or below, for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.3.2.1)

C NC (N/A) SOFT STORY: The stiffness of the lateral-force-resisting system in any story shall not be less than 70 percent of the lateral-force-resisting system stiffness in an adjacent story above or below, or less than 80 percent of the average lateral-force-resisting system stiffness of the three stories above or below for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.3.2.2)

C NC (N/A) GEOMETRY: There shall be no changes in horizontal dimension of the lateral-force-resisting system of more than 30 percent in a story relative to adjacent stories for Life Safety and Immediate Occupancy, excluding one-story penthouses and mezzanines. (Tier 2: Sec. 4.3.2.3)

(C) NC N/A VERTICAL DISCONTINUITIES: All vertical elements in the lateral-force-resisting system shall be continuous to the foundation. (Tier 2: Sec. 4.3.2.4)

C NC (N/A) MASS: There shall be no change in effective mass more than 30 percent from one story to the next for Life Safety and Immediate Occupancy. Light roofs, penthouses, and mezzanines need not be considered. (Tier 2: Sec. 4.3.2.5)
Screening Phase (Tier 1)

| C (NC) N/A | DETERIORATION OF WOOD: There shall be no signs of decay, shrinkage, splitting, fire damage, or sagging in any of the wood members, and none of the metal connection hardware shall be deteriorated, broken, or loose. (Tier 2: Sec. 4.3.3.1) |
| C NC (N/A) | WOOD STRUCTURAL PANEL SHEAR WALL FASTENERS: There shall be no more than 15 percent of inadequate fastening such as overdriven fasteners, omitted blocking, excessive fastening spacing, or inadequate edge distance. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.3.3.2) |
| C NC (N/A) | REDUNDANCY: The number of lines of shear walls in each principal direction shall be greater than or equal to 2 for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.4.2.1.1) |
| C NC (N/A) | SHEAR STRESS CHECK: The shear stress in the shear walls, calculated using the Quick Check procedure of Section 3.5.3.3, shall be less than the following values for Life Safety and Immediate Occupancy (Tier 2: Sec. 4.4.2.7.1): |
| Structural panel sheathing | 1,000 plf |
| Diagonal sheathing | 700 plf |
| Straight sheathing | 100 plf |
| All other conditions | 100 plf |
| C NC (N/A) | STUCCO (EXTERIOR PLASTER) SHEAR WALLS: Multi-story buildings shall not rely on exterior stucco walls as the primary lateral-force-resisting system. (Tier 2: Sec. 4.4.2.7.2) |
| C NC (N/A) | GYPSUM WALLBOARD OR PLASTER SHEAR WALLS: Interior plaster or gypsum wallboard shall not be used as shear walls on buildings over one story in height with the exception of the uppermost level of a multi-story building. (Tier 2: Sec. 4.4.2.7.3) |
| C NC (N/A) | NARROW WOOD SHEAR WALLS: Narrow wood shear walls with an aspect ratio greater than 2-to-1 for Life Safety and 1.5-to-1 for Immediate Occupancy shall not be used to resist lateral forces developed in the building in levels of moderate and high seismicity. Narrow wood shear walls with an aspect ratio greater than 2-to-1 for Immediate Occupancy shall not be used to resist lateral forces developed in the building in levels of low seismicity. (Tier 2: Sec. 4.4.2.7.4) |
| C NC (N/A) | WALLS CONNECTED THROUGH FLOORS: Shear walls shall have interconnection between stories to transfer overturning and shear forces through the floor. (Tier 2: Sec. 4.4.2.7.5) |
| C NC (N/A) | HILLSIDE SITE: For structures that are taller on at least one side by more than one-half story due to a sloping site, all shear walls on the downhill slope shall have an aspect ratio less than 1-to-1 for Life Safety and 1-to-2 for Immediate Occupancy. (Tier 2: Sec. 4.4.2.7.6) |
| C NC (N/A) | CRIPPLE WALLS: Cripple walls below first-floor-level shear walls shall be braced to the foundation with wood structural panels. (Tier 2: Sec. 4.4.2.7.7) |
| C NC (N/A) | OPENINGS: Walls with openings greater than 80 percent of the length shall be braced with wood structural panel shear walls with aspect ratios of not more than 1.5-to-1 or shall be supported by adjacent construction through positive ties capable of transferring the lateral forces. (Tier 2: Sec. 4.4.2.7.8) |

**Connections**

| C (NC) N/A | WOOD POSTS: There shall be a positive connection of wood posts to the foundation. (Tier 2: Sec. 4.6.3.3) |
| C NC (N/A) | WOOD SILLS: All wood sills shall be bolted to the foundation. (Tier 2: Sec. 4.6.3.4) |
| C (NC) N/A | GIRDER/COLUMN CONNECTION: There shall be a positive connection utilizing plates, connection hardware, or straps between the girder and the column support. (Tier 2: Sec. 4.6.4.1) |
Screening Phase (Tier 1)

3.7.2S Supplemental Structural Checklist for Building Type W2: Wood Frames, Commercial and Industrial

This Supplemental Structural Checklist shall be completed where required by Table 3-2. The Basic Structural Checklist shall be completed prior to completing this Supplemental Structural Checklist.

Lateral-Force-Resisting System

C NC (N/A) HOLD-DOWN ANCHORS: All shear walls shall have hold-down anchors constructed per acceptable construction practices, attached to the end studs. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.4.2.7.9)

Diaphragms

(C) NC N/A DIAPHRAGM CONTINUITY: The diaphragms shall not be composed of split-level floors and shall not have expansion joints. (Tier 2: Sec. 4.5.1.1)

(C) NC N/A ROOF CHORD CONTINUITY: All chord elements shall be continuous, regardless of changes in roof elevation. (Tier 2: Sec. 4.5.1.3)

C NC (N/A) PLAN IRREGULARITIES: There shall be tensile capacity to develop the strength of the diaphragm at re-entrant corners or other locations of plan irregularities. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.5.1.7)

C NC (N/A) DIAPHRAGM REINFORCEMENT AT OPENINGS: There shall be reinforcing around all diaphragm openings larger than 50 percent of the building width in either major plan dimension. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.5.1.8)

C NC (N/A) STRAIGHT SHEATHING: All straight sheathed diaphragms shall have aspect ratios less than 2-to-1 for Life Safety and 1-to-1 for Immediate Occupancy in the direction being considered. (Tier 2: Sec. 4.5.2.1)

C NC (N/A) SPANS: All wood diaphragms with spans greater than 24 feet for Life Safety and 12 feet for Immediate Occupancy shall consist of wood structural panels or diagonal sheathing. Wood commercial and industrial buildings may have rod-braced systems. (Tier 2: Sec. 4.5.2.2)

C NC (N/A) UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms shall have horizontal spans less than 40 feet for Life Safety and 30 feet for Immediate Occupancy and shall have aspect ratios less than or equal to 4-to-1 for Life Safety and 3-to-1 for Immediate Occupancy. (Tier 2: Sec. 4.5.2.3)

(C) NC N/A OTHER DIAPHRAGMS: The diaphragm shall not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Tier 2: Sec. 4.5.7.1)

Connections

C NC (N/A) WOOD SILL BOLTS: Sill bolts shall be spaced at 6 feet or less for Life Safety and 4 feet or less for Immediate Occupancy, with proper edge and end distance provided for wood and concrete. (Tier 2: Sec. 4.6.3.9)
3.8 Geologic Site Hazards and Foundations Checklist

This Geologic Site Hazards and Foundations Checklist shall be completed where required by Table 3-2.

Each of the evaluation statements on this checklist shall be marked Compliant (C), Non-compliant (NC), or Not Applicable (N/A) for a Tier 1 Evaluation. Compliant statements identify issues that are acceptable according to the criteria of this standard, while non-compliant statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated. For non-compliant evaluation statements, the design professional may choose to conduct further investigation using the corresponding Tier 2 Evaluation procedure; corresponding section numbers are in parentheses following each evaluation statement.

Geologic Site Hazards

The following statements shall be completed for buildings in levels of high or moderate seismicity.

(C) NC N/A LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.7.1.1)

(C) NC N/A SLOPE FAILURE: The building site shall be sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or shall be capable of accommodating any predicted movements without failure. (Tier 2: Sec. 4.7.1.2)

(C) NC N/A SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated. (Tier 2: Sec. 4.7.1.3)

Condition of Foundations

The following statement shall be completed for all Tier 1 building evaluations.

(C) NC N/A FOUNDATION PERFORMANCE: There shall be no evidence of excessive foundation movement such as settlement or heave that would affect the integrity or strength of the structure. (Tier 2: Sec. 4.7.2.1)

The following statement shall be completed for buildings in levels of high or moderate seismicity being evaluated to the Immediate Occupancy Performance Level.

(C) NC N/A DETERIORATION: There shall not be evidence that foundation elements have deteriorated due to corrosion, sulfate attack, material breakdown, or other reasons in a manner that would affect the integrity or strength of the structure. (Tier 2: Sec. 4.7.2.2)

Capacity of Foundations

The following statement shall be completed for all Tier 1 building evaluations.

(C) NC N/A POLE FOUNDATIONS: Pole foundations shall have a minimum embedment depth of 4 feet for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.7.3.1)

The following statements shall be completed for buildings in levels of moderate seismicity being evaluated to the Immediate Occupancy Performance Level and for buildings in levels of high seismicity.

(C) NC N/A OVERTURNING: The ratio of the horizontal dimension of the lateral-force-resisting system at the foundation level to the building height (base/height) shall be greater than 0.65. (Tier 2: Sec. 4.7.3.2)
Cost Estimates

1. Immediate Remedies

Number of 4\times 6\ to be replaced = 1111111111 = 18
Number of 4\times 12\ to be replaced = 1111111 = 10
Number of 6\times 10\ to be replaced = 1111111 = 11
Number of 4\times 4\ to be replaced = (same as 4\times 6) = 18

Material costs, all pressure treated, all 20' long:

- 4\times 6 \quad \$101.08 \times 18 = \$1,819.44
- 4\times 12 \quad 228.60 \times 10 = 2,286.00
- 6\times 10 \quad 134.10 \times 11 = 1,475.10
- 4\times 4 \quad 67.40 \times 18 = 1,212.60

Total Material Costs: $6,778

Labor costs = $7,000
- Replace lag bolts = 3,000 labor + material
- Remove picket fence = 4,000 labor
- Gutter repairs = 5,000 labor + material

Total Labor Costs: $12,000

Total Costs: $25,778

Contingency 30% = $7,733

Contractor's OH + P 15% = $5,027

Total: $35,511

2. Near-term Repairs

- Flashing Repairs: $12,000
- Metal Deck Button Punch: $2,000
- Shear Connectors: $24 \times 7' long \times 10^4\ feet \times $2.00/lb = $3,360

Total: $17,360
Labor and Material

Contingency 30%  
$ 5,208

Contractor's OH+P 15%  
$ 3,385

Total  
$ 25,953

3. Long-term Repairs

Replace build-up posts

Total number of posts = 44

Total number of $2 \times 6 = 88$ (all 10'-0"

$2 \times 8 = 88$

Material costs, all pressure treated, all 10'-0" long

$2 \times 6$  
$2 \times 8$

$13.00 \times 88 = 1144$

$22.06 \times 88 = 2024$

Labor costs  
$4,000$

Total  
$7,168$

Contingency 10%  
$717$

Contractor's OH+P 15%  
$1,183$

Total  
$9,068$