

LEIGH & O'KANE L.L.C. – STRUCTURAL CALCULATIONS FOR
NEW STRUCTURE AND EXISTING RENOVATION

JKV | EMS BUILDING

506 NW MURRY RD., LEE'S SUMMIT, MISSOURI 64061



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6-16-2025

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Revision:	Date:	Sheet No. i
Project Name: JKV-EMS		

TABLE OF CONTENTS	
GENERAL DESIGN CONSIDERATIONS	1
REFERENCED DESIGN STANDARDS	1
BUILDING LOADING	3
BUILDING DESIGN	45
MASONRY WALL SECTION DESIGN - WP17	117
MASONRY WALL SECTION DESIGN - WP18	132
MASONRY WALL SECTION DESIGN - WP19	147
MASONRY WALL SECTION DESIGN - WP20	162
STEEL LINTEL AT APP BAY DOOR	176
MASONRY LINTEL AT APP BAY WINDOW	179
ROOF OUTRIGGER DESIGN AND ANALYSIS - UNIFORM LOAD	181
ROOF OUTRIGGER DESIGN AND ANALYSIS - POINT LOAD	185
ROOF EDGE ANGLE DESIGN	189
COMBINED FOOTING - INSIDE CORNER COLUMN	193
CONNECTIONS	203
EXISTING BUILDING	240
SNOW LOAD ON CANOPY	245
CANOPY ROOF JOIST	246
CANOPY ROOF GIRDER	255
CANOPY ROOF LEDGER	259
CANOPY LEDGER CONNECTION	269
CANOPY IN WALL GIRDER COLUMN	272
CANOPY COLUMN	274
CANOPY CONCRETE COLUMN	276
CANOPY FOOTING	279
NEW ROOF TRUSS TOP CHORD ESTENSION AND OUTLOOKER	286
NEW TOWER OUTRIGGER	294
NEW TOWER JOIST	303
NEW TOWER JOIST GIRDER	314
EXISTING STEEL COLUMN - END SUPPORT - GRID LINE B-5	323
NEW STEEL BEAM SUPPORT IN EXISTING - GRID LINE B - NO POINT LOAD	329
NEW STEEL BEAM SUPPORT IN EXISTING - GRID LINE B - WITH POINT LOAD	333
NEW STEEL COLUMN - END SUPPORT - GRID LINE E-6	336
NEW STEEL COLUMN - END SUPPORT - GRID LINE B-7.5	338
NEW STEEL COLUMN - MID SUPPORT - GRID LINE B.4-6	340
FOOTING AT EXISTING COLUMN - GRID LINE B-5	342
FOOTING AT NEW COLUMN - GRID LINE B.4-6	349
FOOTING AT NEW COLUMN - GRID LINE B-7.5	356
FOOTING AT NEW COLUMN - GRID LINE E-6	363
MECH SCREEN FRAMING	370
BRICK SUPPORT CORBEL ON EXISTING	376



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Revision:	Date:	Sheet No. 1
Project Name: JKV-EMS		

GENERAL DESIGN CONSIDERATIONS

The following structural calculations are for a new ambulance apparatus building adjacent to an existing structure on site along with limited modifications to the existing structure. The ambulance bay will be constructed of load bearing CMU walls and a structural steel framed roof. Lateral loads will be resisted by ordinary CMU shear walls and concentric steel braced frames. The foundations are designed to bear on assumed 1500 psf allowable bearing capacity soil. The steel column footings will share load with the existing foundations where the new apparatus bay is directly adjacent to the existing structure, otherwise the two buildings are structurally independent of one another.

The modifications to the existing structure do not increase the vertical or lateral load to the building, hence the existing structure has not been analyzed for current wind and seismic loading. There are locations around the exterior of the building where the existing roof trusses will be replaced with rectangular wood roof trusses. There are locations in the interior of the building where new steel beams and columns will be installed to support the existing roof structure while allowing the interior floor plan to fit the proposed design.

The structural design for this project utilizes analysis and design software along with hand calculations. The majority of the apparatus bay uses Tekla Structural Designer. Other software used is Tekla Tedds modular calculation software and Idea Statica for connection design.

REFERENCED DESIGN STANDARDS

- IBC 2018 – International Building Code 2018 Edition
 - ACI 318-14 – American Concrete Institute Building Code Requirements for Structural Concrete
 - AISC 360-16 – American Institute of Steel Construction Specification for Structural Steel Buildings
 - ASCE 7-16 – American Society of Civil Engineers Minimum Design Loads and Associated Criteria for Buildings and Other Structures
-
- A. Overall Building Classification
 - a. Risk Category III
 - b. Show Importance Factor 1.10
 - c. Seismic Importance Factor 1.25
 - B. Roof Dead and Live Loads
 - a. Dead Load 15 psf
 - b. Mechanical Dead Load 15 psf
 - c. Live Load 20 psf
 - C. Roof Snow Loads
 - a. Ground Snow Load 20 psf
 - b. Flat Roof Snow Load 15.4 psf
 - c. Snow Exposure Factor 1.0
 - d. Thermal Factor 1.0
 - e. Drifting Per Code
 - D. Floor Dead and Live Loads
 - a. Dead Load Actual Structure
 - b. Floor Live Load 100 psf



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Revision:	Date:	Sheet No. 2
Project Name: JKV-EMS		

E. Wind Loading

a. Basic Wind Speed	117 mph
b. Exposure Category	C
c. Internal Pressure Coefficient	+/- 0.18
d. Components and Cladding	Per ASCE 7-16

F. Seismic Loads

a. S_s	0.099 g
b. S_1	0.068 g
c. Site Class	D (default assumed)
d. S_{DS}	0.1056 g
e. S_{D1}	0.1088 g
f. Seismic Design Category	B
g. Seismic Force Resisting System	Ordinary Reinforced Masonry Shear Walls
h. Design Base Shear	$C_s W$
i. Design Response Coefficient	0.044
j. Response Modification Coefficient	2.0
k. Analysis Procedure Used	Equivalent Lateral Force

G. Roof Rain Loads

a. 60-min. Duration/100 Year Rain Intensity	3.92 in./hr.
b. 15-min. Duration/100 Year Rain Intensity	8.31 in./hr.



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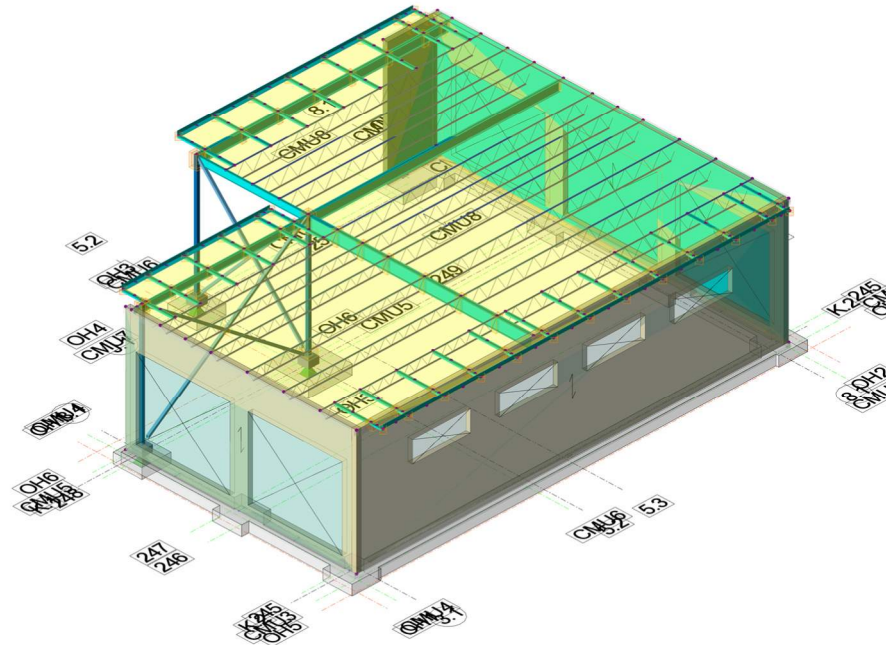
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Revision:	Date:	Sheet No. 3
Project Name: JKV-EMS		

BUILDING LOADING



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Revision:	Date:	Sheet No. 4
Project Name: JKV-EMS		



Structure

Structure

Action Codes

General Loading ASCE7 (2016)
Wind Loading ASCE7 (2016)
Snow Loading ASCE7 (2016)
Seismic Loading ASCE7 (2016)
Combinations ASCE7 (2016)

Resistance Codes

Steel Design AISC 360/341 LRFD (2016)
Concrete Design ACI 318 (2014)
Composite Design AISC 360/341 LRFD (2016)
Timber Design NDS LRFD (2018)
Masonry Design TMS 402/602 ASD (2016)
Foundation Design No Design Code
Seismic Design and Detailing ASCE7 (2016)
Steel Fire Design No Design Code



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Revision:	Date:	Sheet No. 5
Project Name: JKV-EMS		

Combinations

Name	Class	Active	Strength	Service
1 Effective Seismic Weight	Modal Mass	•	•	
2 LRFD ₁ -1.4D	Gravity	•	•	•
3 LRFD ₂ -1.2D+1.6L	Gravity	•	•	•
4 LRFD ₃ -1.2D+1.6L+0.5Lr	Gravity	•	•	•
5 LRFD _{4,1} -1.2D+1.6L+0.5S	Gravity	•	•	•
6 LRFD _{4,2} -1.2D+1.6L+0.5S	Gravity	•	•	•
7 LRFD _{4,3} -1.2D+1.6L+0.5S	Gravity	•	•	•
8 LRFD _{4,4} -1.2D+1.6L+0.5S	Gravity	•	•	•
9 LRFD ₅ -1.2D+L+1.6Lr	Gravity	•	•	•
10 LRFD _{6,1} -1.2D+L+1.6S	Gravity	•	•	•
11 LRFD _{6,2} -1.2D+L+1.6S	Gravity	•	•	•
12 LRFD _{6,3} -1.2D+L+1.6S	Gravity	•	•	•
13 LRFD _{6,4} -1.2D+L+1.6S	Gravity	•	•	•
14 LRFD _{7,1} -1.2D+1.6Lr+0.5W	Lateral	•	•	•
15 LRFD _{7,2} -1.2D+1.6Lr+0.5W	Lateral	•	•	•
16 LRFD _{7,3} -1.2D+1.6Lr+0.5W	Lateral	•	•	•
17 LRFD _{7,4} -1.2D+1.6Lr+0.5W	Lateral	•	•	•
18 LRFD _{7,5} -1.2D+1.6Lr+0.5W	Lateral	•	•	•
19 LRFD _{7,6} -1.2D+1.6Lr+0.5W	Lateral	•	•	•
20 LRFD _{7,7} -1.2D+1.6Lr+0.5W	Lateral	•	•	•
21 LRFD _{7,8} -1.2D+1.6Lr+0.5W	Lateral	•	•	•
22 LRFD _{8,1} -1.2D+1.6S+0.5W	Lateral	•	•	•
23 LRFD _{8,2} -1.2D+1.6S+0.5W	Lateral	•	•	•
24 LRFD _{8,3} -1.2D+1.6S+0.5W	Lateral	•	•	•
25 LRFD _{8,4} -1.2D+1.6S+0.5W	Lateral	•	•	•
26 LRFD _{8,5} -1.2D+1.6S+0.5W	Lateral	•	•	•
27 LRFD _{8,6} -1.2D+1.6S+0.5W	Lateral	•	•	•
28 LRFD _{8,7} -1.2D+1.6S+0.5W	Lateral	•	•	•
29 LRFD _{8,8} -1.2D+1.6S+0.5W	Lateral	•	•	•
30 LRFD _{8,9} -1.2D+1.6S+0.5W	Lateral	•	•	•
31 LRFD _{8,10} -1.2D+1.6S+0.5W	Lateral	•	•	•
32 LRFD _{8,11} -1.2D+1.6S+0.5W	Lateral	•	•	•
33 LRFD _{8,12} -1.2D+1.6S+0.5W	Lateral	•	•	•
34 LRFD _{8,13} -1.2D+1.6S+0.5W	Lateral	•	•	•
35 LRFD _{8,14} -1.2D+1.6S+0.5W	Lateral	•	•	•
36 LRFD _{8,15} -1.2D+1.6S+0.5W	Lateral	•	•	•
37 LRFD _{8,16} -1.2D+1.6S+0.5W	Lateral	•	•	•
38 LRFD _{8,17} -1.2D+1.6S+0.5W	Lateral	•	•	•
39 LRFD _{8,18} -1.2D+1.6S+0.5W	Lateral	•	•	•
40 LRFD _{8,19} -1.2D+1.6S+0.5W	Lateral	•	•	•
41 LRFD _{8,20} -1.2D+1.6S+0.5W	Lateral	•	•	•
42 LRFD _{8,21} -1.2D+1.6S+0.5W	Lateral	•	•	•
43 LRFD _{8,22} -1.2D+1.6S+0.5W	Lateral	•	•	•
44 LRFD _{8,23} -1.2D+1.6S+0.5W	Lateral	•	•	•
45 LRFD _{8,24} -1.2D+1.6S+0.5W	Lateral	•	•	•
46 LRFD _{8,25} -1.2D+1.6S+0.5W	Lateral	•	•	•
47 LRFD _{8,26} -1.2D+1.6S+0.5W	Lateral	•	•	•
48 LRFD _{8,27} -1.2D+1.6S+0.5W	Lateral	•	•	•
49 LRFD _{8,28} -1.2D+1.6S+0.5W	Lateral	•	•	•



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Revision:	Date:	Sheet No. 6
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Name	Class	Active	Strength	Service
50 LRFD _{8.29} -1.2D+1.6S+0.5W	Lateral	•	•	•
51 LRFD _{8.30} -1.2D+1.6S+0.5W	Lateral	•	•	•
52 LRFD _{8.31} -1.2D+1.6S+0.5W	Lateral	•	•	•
53 LRFD _{8.32} -1.2D+1.6S+0.5W	Lateral	•	•	•
54 LRFD _{9.1} -1.2D+L+0.5Lr+W	Lateral	•	•	•
55 LRFD _{9.2} -1.2D+L+0.5Lr+W	Lateral	•	•	•
56 LRFD _{9.3} -1.2D+L+0.5Lr+W	Lateral	•	•	•
57 LRFD _{9.4} -1.2D+L+0.5Lr+W	Lateral	•	•	•
58 LRFD _{9.5} -1.2D+L+0.5Lr+W	Lateral	•	•	•
59 LRFD _{9.6} -1.2D+L+0.5Lr+W	Lateral	•	•	•
60 LRFD _{9.7} -1.2D+L+0.5Lr+W	Lateral	•	•	•
61 LRFD _{9.8} -1.2D+L+0.5Lr+W	Lateral	•	•	•
62 LRFD _{10.1} -1.2D+L+0.5S+W	Lateral	•	•	•
63 LRFD _{10.2} -1.2D+L+0.5S+W	Lateral	•	•	•
64 LRFD _{10.3} -1.2D+L+0.5S+W	Lateral	•	•	•
65 LRFD _{10.4} -1.2D+L+0.5S+W	Lateral	•	•	•
66 LRFD _{10.5} -1.2D+L+0.5S+W	Lateral	•	•	•
67 LRFD _{10.6} -1.2D+L+0.5S+W	Lateral	•	•	•
68 LRFD _{10.7} -1.2D+L+0.5S+W	Lateral	•	•	•
69 LRFD _{10.8} -1.2D+L+0.5S+W	Lateral	•	•	•
70 LRFD _{10.9} -1.2D+L+0.5S+W	Lateral	•	•	•
71 LRFD _{10.10} -1.2D+L+0.5S+W	Lateral	•	•	•
72 LRFD _{10.11} -1.2D+L+0.5S+W	Lateral	•	•	•
73 LRFD _{10.12} -1.2D+L+0.5S+W	Lateral	•	•	•
74 LRFD _{10.13} -1.2D+L+0.5S+W	Lateral	•	•	•
75 LRFD _{10.14} -1.2D+L+0.5S+W	Lateral	•	•	•
76 LRFD _{10.15} -1.2D+L+0.5S+W	Lateral	•	•	•
77 LRFD _{10.16} -1.2D+L+0.5S+W	Lateral	•	•	•
78 LRFD _{10.17} -1.2D+L+0.5S+W	Lateral	•	•	•
79 LRFD _{10.18} -1.2D+L+0.5S+W	Lateral	•	•	•
80 LRFD _{10.19} -1.2D+L+0.5S+W	Lateral	•	•	•
81 LRFD _{10.20} -1.2D+L+0.5S+W	Lateral	•	•	•
82 LRFD _{10.21} -1.2D+L+0.5S+W	Lateral	•	•	•
83 LRFD _{10.22} -1.2D+L+0.5S+W	Lateral	•	•	•
84 LRFD _{10.23} -1.2D+L+0.5S+W	Lateral	•	•	•
85 LRFD _{10.24} -1.2D+L+0.5S+W	Lateral	•	•	•
86 LRFD _{10.25} -1.2D+L+0.5S+W	Lateral	•	•	•
87 LRFD _{10.26} -1.2D+L+0.5S+W	Lateral	•	•	•
88 LRFD _{10.27} -1.2D+L+0.5S+W	Lateral	•	•	•
89 LRFD _{10.28} -1.2D+L+0.5S+W	Lateral	•	•	•
90 LRFD _{10.29} -1.2D+L+0.5S+W	Lateral	•	•	•
91 LRFD _{10.30} -1.2D+L+0.5S+W	Lateral	•	•	•
92 LRFD _{10.31} -1.2D+L+0.5S+W	Lateral	•	•	•
93 LRFD _{10.32} -1.2D+L+0.5S+W	Lateral	•	•	•
102 LRFD _{12.1} -0.9D+W	Lateral	•	•	•
103 LRFD _{12.2} -0.9D+W	Lateral	•	•	•
104 LRFD _{12.3} -0.9D+W	Lateral	•	•	•
105 LRFD _{12.4} -0.9D+W	Lateral	•	•	•
106 LRFD _{12.5} -0.9D+W	Lateral	•	•	•
107 LRFD _{12.6} -0.9D+W	Lateral	•	•	•



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Revision:	Date:	Sheet No. 7
Project Name: JKV-EMS		

Name	Class	Active	Strength	Service
108 LRFD _{12.7} -0.9D+W	Lateral	•	•	•
109 LRFD _{12.8} -0.9D+W	Lateral	•	•	•
118 LRFD _{11.1} -1.2D+L+0.2S+E	Seismic	•	•	•
119 LRFD _{11.2} -1.2D+L+0.2S+E	Seismic	•	•	•
120 LRFD _{11.3} -1.2D+L+0.2S+E	Seismic	•	•	•
121 LRFD _{11.4} -1.2D+L+0.2S+E	Seismic	•	•	•
122 LRFD _{11.5} -1.2D+L+0.2S+E	Seismic	•	•	•
123 LRFD _{11.6} -1.2D+L+0.2S+E	Seismic	•	•	•
124 LRFD _{11.7} -1.2D+L+0.2S+E	Seismic	•	•	•
125 LRFD _{11.8} -1.2D+L+0.2S+E	Seismic	•	•	•
126 LRFD _{13.1} -0.9D+E	Seismic	•	•	•
127 LRFD _{13.2} -0.9D+E	Seismic	•	•	•
128 LRFD _{13.3} -0.9D+E	Seismic	•	•	•
129 LRFD _{13.4} -0.9D+E	Seismic	•	•	•
130 LRFD _{13.5} -0.9D+E	Seismic	•	•	•
131 LRFD _{13.6} -0.9D+E	Seismic	•	•	•
132 LRFD _{13.7} -0.9D+E	Seismic	•	•	•
133 LRFD _{13.8} -0.9D+E	Seismic	•	•	•

1 Effective Seismic Weight

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.000	1.000
3 Dead	1.000	1.000
6 MEP	1.000	1.000
2 Slab self weight	1.000	1.000

2 LRFD₁-1.4D

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.400	1.000
2 Slab self weight	1.400	1.000
3 Dead	1.400	1.000
6 MEP	1.400	1.000

3 LRFD₂-1.2D+1.6L

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.600	1.000

4 LRFD₃-1.2D+1.6L+0.5Lr

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000



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Revision:	Date:	Sheet No. 8
Project Name: JKV-EMS		

Loadcase Title	Strength	Service
6 MEP	1.200	1.000
4 Live	1.600	0.750
5 Roof Live	0.500	0.750

5 LRFD_{4,1}-1.2D+1.6L+0.5S

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.600	0.750
7 Minimum Snow Load	0.500	0.750

6 LRFD_{4,2}-1.2D+1.6L+0.5S

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.600	0.750
8 Balanced Snow Load	0.500	0.750
11 Rain on Snow Surcharge	0.500	0.750

7 LRFD_{4,3}-1.2D+1.6L+0.5S

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.600	0.750
9 Unbalanced Snow Load 1	0.500	0.750

8 LRFD_{4,4}-1.2D+1.6L+0.5S

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.600	0.750
10 Drift Snow Load 1	0.500	0.750

9 LRFD₅-1.2D+L+1.6Lr

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000



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Revision:	Date:	Sheet No. 9
Project Name: JKV-EMS		

Loadcase Title	Strength	Service
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.000
5 Roof Live	1.600	1.000

10 LRFD_{6.1}-1.2D+L+1.6S

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.000
7 Minimum Snow Load	1.600	1.000

11 LRFD_{6.2}-1.2D+L+1.6S

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.000
8 Balanced Snow Load	1.600	1.000
11 Rain on Snow Surcharge	1.600	1.000

12 LRFD_{6.3}-1.2D+L+1.6S

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.000
9 Unbalanced Snow Load 1	1.600	1.000

13 LRFD_{6.4}-1.2D+L+1.6S

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.000
10 Drift Snow Load 1	1.600	1.000

14 LRFD_{7.1}-1.2D+1.6Lr+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000



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Revision:	Date:	Sheet No. 10
Project Name: JKV-EMS		

Loadcase Title	Strength	Service
3 Dead	1.200	1.000
6 MEP	1.200	1.000
12 Wind +X',GCpi 0.18,-Cp	0.500	0.500
5 Roof Live	1.600	1.000

15 LRFD_{7,2}-1.2D+1.6Lr+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
13 Wind +X',GCpi 0.18,+Cp	0.500	0.500
5 Roof Live	1.600	1.000

16 LRFD_{7,3}-1.2D+1.6Lr+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
14 Wind +Y',GCpi 0.18,-Cp	0.500	0.500
5 Roof Live	1.600	1.000

17 LRFD_{7,4}-1.2D+1.6Lr+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
15 Wind +Y',GCpi 0.18,+Cp	0.500	0.500
5 Roof Live	1.600	1.000

18 LRFD_{7,5}-1.2D+1.6Lr+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
16 Wind -X',GCpi 0.18,-Cp	0.500	0.500
5 Roof Live	1.600	1.000

19 LRFD_{7,6}-1.2D+1.6Lr+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000



209 SE DOUGLAS
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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 11
Project Name: JKV-EMS		

Loadcase Title	Strength	Service
6 MEP	1.200	1.000
17 Wind -X',GCpi 0.18,+Cp	0.500	0.500
5 Roof Live	1.600	1.000

20 LRFD_{7,7}-1.2D+1.6Lr+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
18 Wind -Y',GCpi 0.18,-Cp	0.500	0.500
5 Roof Live	1.600	1.000

21 LRFD_{7,8}-1.2D+1.6Lr+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
19 Wind -Y',GCpi 0.18,+Cp	0.500	0.500
5 Roof Live	1.600	1.000

22 LRFD_{8,1}-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
12 Wind +X',GCpi 0.18,-Cp	0.500	0.500
7 Minimum Snow Load	1.600	1.000

23 LRFD_{8,2}-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
13 Wind +X',GCpi 0.18,+Cp	0.500	0.500
7 Minimum Snow Load	1.600	1.000

24 LRFD_{8,3}-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 12
Project Name: JKV-EMS		

Loadcase Title	Strength	Service
14 Wind +Y',GCpi 0.18,-Cp	0.500	0.500
7 Minimum Snow Load	1.600	1.000

25 LRFD_{8.4}-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
15 Wind +Y',GCpi 0.18,+Cp	0.500	0.500
7 Minimum Snow Load	1.600	1.000

26 LRFD_{8.5}-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
16 Wind -X',GCpi 0.18,-Cp	0.500	0.500
7 Minimum Snow Load	1.600	1.000

27 LRFD_{8.6}-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
17 Wind -X',GCpi 0.18,+Cp	0.500	0.500
7 Minimum Snow Load	1.600	1.000

28 LRFD_{8.7}-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
18 Wind -Y',GCpi 0.18,-Cp	0.500	0.500
7 Minimum Snow Load	1.600	1.000

29 LRFD_{8.8}-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000



209 SE DOUGLAS
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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 13
Project Name: JKV-EMS		

Loadcase Title	Strength	Service
19 Wind -Y',GCpi 0.18,+Cp	0.500	0.500
7 Minimum Snow Load	1.600	1.000

30 LRFD_{8,9}-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
12 Wind +X',GCpi 0.18,-Cp	0.500	0.500
8 Balanced Snow Load	1.600	1.000
11 Rain on Snow Surcharge	1.600	1.000

31 LRFD_{8,10}-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
13 Wind +X',GCpi 0.18,+Cp	0.500	0.500
8 Balanced Snow Load	1.600	1.000
11 Rain on Snow Surcharge	1.600	1.000

32 LRFD_{8,11}-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
14 Wind +Y',GCpi 0.18,-Cp	0.500	0.500
8 Balanced Snow Load	1.600	1.000
11 Rain on Snow Surcharge	1.600	1.000

33 LRFD_{8,12}-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
15 Wind +Y',GCpi 0.18,+Cp	0.500	0.500
8 Balanced Snow Load	1.600	1.000
11 Rain on Snow Surcharge	1.600	1.000

34 LRFD_{8,13}-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 14
Project Name: JKV-EMS		

Loadcase Title	Strength	Service
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
16 Wind -X',GCpi 0.18,-Cp	0.500	0.500
8 Balanced Snow Load	1.600	1.000
11 Rain on Snow Surcharge	1.600	1.000

35 LRFD_{8.14}-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
17 Wind -X',GCpi 0.18,+Cp	0.500	0.500
8 Balanced Snow Load	1.600	1.000
11 Rain on Snow Surcharge	1.600	1.000

36 LRFD_{8.15}-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
18 Wind -Y',GCpi 0.18,-Cp	0.500	0.500
8 Balanced Snow Load	1.600	1.000
11 Rain on Snow Surcharge	1.600	1.000

37 LRFD_{8.16}-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
19 Wind -Y',GCpi 0.18,+Cp	0.500	0.500
8 Balanced Snow Load	1.600	1.000
11 Rain on Snow Surcharge	1.600	1.000

38 LRFD_{8.17}-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
12 Wind +X',GCpi 0.18,-Cp	0.500	0.500
9 Unbalanced Snow Load 1	1.600	1.000



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 15
Project Name: JKV-EMS		

39 LRFD_{8.18}-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
13 Wind +X',GCpi 0.18,+Cp	0.500	0.500
9 Unbalanced Snow Load 1	1.600	1.000

40 LRFD_{8.19}-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
14 Wind +Y',GCpi 0.18,-Cp	0.500	0.500
9 Unbalanced Snow Load 1	1.600	1.000

41 LRFD_{8.20}-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
15 Wind +Y',GCpi 0.18,+Cp	0.500	0.500
9 Unbalanced Snow Load 1	1.600	1.000

42 LRFD_{8.21}-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
16 Wind -X',GCpi 0.18,-Cp	0.500	0.500
9 Unbalanced Snow Load 1	1.600	1.000

43 LRFD_{8.22}-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
17 Wind -X',GCpi 0.18,+Cp	0.500	0.500
9 Unbalanced Snow Load 1	1.600	1.000

44 LRFD_{8.23}-1.2D+1.6S+0.5W



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 16
Project Name: JKV-EMS		

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
18 Wind -Y',GCpi 0.18,-Cp	0.500	0.500
9 Unbalanced Snow Load 1	1.600	1.000

45 LRFD_{8.24}-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
19 Wind -Y',GCpi 0.18,+Cp	0.500	0.500
9 Unbalanced Snow Load 1	1.600	1.000

46 LRFD_{8.25}-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
12 Wind +X',GCpi 0.18,-Cp	0.500	0.500
10 Drift Snow Load 1	1.600	1.000

47 LRFD_{8.26}-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
13 Wind +X',GCpi 0.18,+Cp	0.500	0.500
10 Drift Snow Load 1	1.600	1.000

48 LRFD_{8.27}-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
14 Wind +Y',GCpi 0.18,-Cp	0.500	0.500
10 Drift Snow Load 1	1.600	1.000

49 LRFD_{8.28}-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 17
Project Name: JKV-EMS		

Loadcase Title	Strength	Service
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
15 Wind +Y',GCpi 0.18,+Cp	0.500	0.500
10 Drift Snow Load 1	1.600	1.000

50 LRFD_{8.29}-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
16 Wind -X',GCpi 0.18,-Cp	0.500	0.500
10 Drift Snow Load 1	1.600	1.000

51 LRFD_{8.30}-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
17 Wind -X',GCpi 0.18,+Cp	0.500	0.500
10 Drift Snow Load 1	1.600	1.000

52 LRFD_{8.31}-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
18 Wind -Y',GCpi 0.18,-Cp	0.500	0.500
10 Drift Snow Load 1	1.600	1.000

53 LRFD_{8.32}-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
19 Wind -Y',GCpi 0.18,+Cp	0.500	0.500
10 Drift Snow Load 1	1.600	1.000

54 LRFD_{9.1}-1.2D+L+0.5Lr+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 18
Project Name: JKV-EMS		

Loadcase Title	Strength	Service
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
12 Wind +X',GCpi 0.18,-Cp	1.000	0.450
5 Roof Live	0.500	0.750

55 LRFD_{9,2}-1.2D+L+0.5Lr+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
13 Wind +X',GCpi 0.18,+Cp	1.000	0.450
5 Roof Live	0.500	0.750

56 LRFD_{9,3}-1.2D+L+0.5Lr+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
14 Wind +Y',GCpi 0.18,-Cp	1.000	0.450
5 Roof Live	0.500	0.750

57 LRFD_{9,4}-1.2D+L+0.5Lr+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
15 Wind +Y',GCpi 0.18,+Cp	1.000	0.450
5 Roof Live	0.500	0.750

58 LRFD_{9,5}-1.2D+L+0.5Lr+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
16 Wind -X',GCpi 0.18,-Cp	1.000	0.450
5 Roof Live	0.500	0.750



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 19
Project Name: JKV-EMS		

59 LRFD_{9.6}-1.2D+L+0.5Lr+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
17 Wind -X',GCpi 0.18,+Cp	1.000	0.450
5 Roof Live	0.500	0.750

60 LRFD_{9.7}-1.2D+L+0.5Lr+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
18 Wind -Y',GCpi 0.18,-Cp	1.000	0.450
5 Roof Live	0.500	0.750

61 LRFD_{9.8}-1.2D+L+0.5Lr+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
19 Wind -Y',GCpi 0.18,+Cp	1.000	0.450
5 Roof Live	0.500	0.750

62 LRFD_{10.1}-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
12 Wind +X',GCpi 0.18,-Cp	1.000	0.450
7 Minimum Snow Load	0.500	0.750

63 LRFD_{10.2}-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 20
Project Name: JKV-EMS		

Loadcase Title	Strength	Service
13 Wind +X',GCpi 0.18,+Cp	1.000	0.450
7 Minimum Snow Load	0.500	0.750

64 LRFD_{10.3}-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
14 Wind +Y',GCpi 0.18,-Cp	1.000	0.450
7 Minimum Snow Load	0.500	0.750

65 LRFD_{10.4}-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
15 Wind +Y',GCpi 0.18,+Cp	1.000	0.450
7 Minimum Snow Load	0.500	0.750

66 LRFD_{10.5}-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
16 Wind -X',GCpi 0.18,-Cp	1.000	0.450
7 Minimum Snow Load	0.500	0.750

67 LRFD_{10.6}-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
17 Wind -X',GCpi 0.18,+Cp	1.000	0.450
7 Minimum Snow Load	0.500	0.750

68 LRFD_{10.7}-1.2D+L+0.5S+W



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 21
Project Name: JKV-EMS		

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
18 Wind -Y',GCpi 0.18,-Cp	1.000	0.450
7 Minimum Snow Load	0.500	0.750

69 LRFD_{10.8}-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
19 Wind -Y',GCpi 0.18,+Cp	1.000	0.450
7 Minimum Snow Load	0.500	0.750

70 LRFD_{10.9}-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
12 Wind +X',GCpi 0.18,-Cp	1.000	0.450
8 Balanced Snow Load	0.500	0.750
11 Rain on Snow Surcharge	0.500	0.750

71 LRFD_{10.10}-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
13 Wind +X',GCpi 0.18,+Cp	1.000	0.450
8 Balanced Snow Load	0.500	0.750
11 Rain on Snow Surcharge	0.500	0.750

72 LRFD_{10.11}-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750



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Revision:	Date:	Sheet No. 22
Project Name: JKV-EMS		

Loadcase Title	Strength	Service
14 Wind +Y',GCpi 0.18,-Cp	1.000	0.450
8 Balanced Snow Load	0.500	0.750
11 Rain on Snow Surcharge	0.500	0.750

73 LRFD_{10.12}-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
15 Wind +Y',GCpi 0.18,+Cp	1.000	0.450
8 Balanced Snow Load	0.500	0.750
11 Rain on Snow Surcharge	0.500	0.750

74 LRFD_{10.13}-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
16 Wind -X',GCpi 0.18,-Cp	1.000	0.450
8 Balanced Snow Load	0.500	0.750
11 Rain on Snow Surcharge	0.500	0.750

75 LRFD_{10.14}-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
17 Wind -X',GCpi 0.18,+Cp	1.000	0.450
8 Balanced Snow Load	0.500	0.750
11 Rain on Snow Surcharge	0.500	0.750

76 LRFD_{10.15}-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
18 Wind -Y',GCpi 0.18,-Cp	1.000	0.450
8 Balanced Snow Load	0.500	0.750
11 Rain on Snow Surcharge	0.500	0.750



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 23
Project Name: JKV-EMS		

77 LRFD_{10.16}-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
19 Wind -Y',GCpi 0.18,+Cp	1.000	0.450
8 Balanced Snow Load	0.500	0.750
11 Rain on Snow Surcharge	0.500	0.750

78 LRFD_{10.17}-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
12 Wind +X',GCpi 0.18,-Cp	1.000	0.450
9 Unbalanced Snow Load 1	0.500	0.750

79 LRFD_{10.18}-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
13 Wind +X',GCpi 0.18,+Cp	1.000	0.450
9 Unbalanced Snow Load 1	0.500	0.750

80 LRFD_{10.19}-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
14 Wind +Y',GCpi 0.18,-Cp	1.000	0.450
9 Unbalanced Snow Load 1	0.500	0.750

81 LRFD_{10.20}-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 24
Project Name: JKV-EMS		

Loadcase Title	Strength	Service
6 MEP	1.200	1.000
4 Live	1.000	0.750
15 Wind +Y',GCpi 0.18,+Cp	1.000	0.450
9 Unbalanced Snow Load 1	0.500	0.750

82 LRFD_{10.21}-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
16 Wind -X',GCpi 0.18,-Cp	1.000	0.450
9 Unbalanced Snow Load 1	0.500	0.750

83 LRFD_{10.22}-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
17 Wind -X',GCpi 0.18,+Cp	1.000	0.450
9 Unbalanced Snow Load 1	0.500	0.750

84 LRFD_{10.23}-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
18 Wind -Y',GCpi 0.18,-Cp	1.000	0.450
9 Unbalanced Snow Load 1	0.500	0.750

85 LRFD_{10.24}-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
19 Wind -Y',GCpi 0.18,+Cp	1.000	0.450
9 Unbalanced Snow Load 1	0.500	0.750

86 LRFD_{10.25}-1.2D+L+0.5S+W



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 25
Project Name: JKV-EMS		

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
12 Wind +X',GCpi 0.18,-Cp	1.000	0.450
10 Drift Snow Load 1	0.500	0.750

87 LRFD_{10.26}-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
13 Wind +X',GCpi 0.18,+Cp	1.000	0.450
10 Drift Snow Load 1	0.500	0.750

88 LRFD_{10.27}-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
14 Wind +Y',GCpi 0.18,-Cp	1.000	0.450
10 Drift Snow Load 1	0.500	0.750

89 LRFD_{10.28}-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
15 Wind +Y',GCpi 0.18,+Cp	1.000	0.450
10 Drift Snow Load 1	0.500	0.750

90 LRFD_{10.29}-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
16 Wind -X',GCpi 0.18,-Cp	1.000	0.450
10 Drift Snow Load 1	0.500	0.750



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 26
Project Name: JKV-EMS		

91 LRFD_{10.30}-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
17 Wind -X',GCpi 0.18,+Cp	1.000	0.450
10 Drift Snow Load 1	0.500	0.750

92 LRFD_{10.31}-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
18 Wind -Y',GCpi 0.18,-Cp	1.000	0.450
10 Drift Snow Load 1	0.500	0.750

93 LRFD_{10.32}-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 MEP	1.200	1.000
4 Live	1.000	0.750
19 Wind -Y',GCpi 0.18,+Cp	1.000	0.450
10 Drift Snow Load 1	0.500	0.750

102 LRFD_{12.1}-0.9D+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.900	0.600
2 Slab self weight	0.900	0.600
3 Dead	0.900	0.600
6 MEP	0.900	0.600
12 Wind +X',GCpi 0.18,-Cp	1.000	0.600

103 LRFD_{12.2}-0.9D+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.900	0.600
2 Slab self weight	0.900	0.600
3 Dead	0.900	0.600
6 MEP	0.900	0.600
13 Wind +X',GCpi 0.18,+Cp	1.000	0.600



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 27
Project Name: JKV-EMS		

104 LRFD_{12.3}-0.9D+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.900	0.600
2 Slab self weight	0.900	0.600
3 Dead	0.900	0.600
6 MEP	0.900	0.600
14 Wind +Y',GCpi 0.18,-Cp	1.000	0.600

105 LRFD_{12.4}-0.9D+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.900	0.600
2 Slab self weight	0.900	0.600
3 Dead	0.900	0.600
6 MEP	0.900	0.600
15 Wind +Y',GCpi 0.18,+Cp	1.000	0.600

106 LRFD_{12.5}-0.9D+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.900	0.600
2 Slab self weight	0.900	0.600
3 Dead	0.900	0.600
6 MEP	0.900	0.600
16 Wind -X',GCpi 0.18,-Cp	1.000	0.600

107 LRFD_{12.6}-0.9D+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.900	0.600
2 Slab self weight	0.900	0.600
3 Dead	0.900	0.600
6 MEP	0.900	0.600
17 Wind -X',GCpi 0.18,+Cp	1.000	0.600

108 LRFD_{12.7}-0.9D+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.900	0.600
2 Slab self weight	0.900	0.600
3 Dead	0.900	0.600
6 MEP	0.900	0.600
18 Wind -Y',GCpi 0.18,-Cp	1.000	0.600

109 LRFD_{12.8}-0.9D+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.900	0.600
2 Slab self weight	0.900	0.600
3 Dead	0.900	0.600



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 28
Project Name: JKV-EMS		

Loadcase Title	Strength	Service
6 MEP	0.900	0.600
19 Wind -Y', GCpi 0.18,+Cp	1.000	0.600

118 LRFD_{11.1}-1.2D+L+0.2S+E

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.221	1.015
2 Slab self weight	1.221	1.015
3 Dead	1.221	1.015
6 MEP	1.221	1.015
4 Live	1.000	0.000
8 Balanced Snow Load	0.200	0.000
11 Rain on Snow Surcharge	0.200	0.000
20 Seismic Dir1	1.000	0.700
21 Seismic Dir2	0.300	0.210

119 LRFD_{11.2}-1.2D+L+0.2S+E

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.221	1.015
2 Slab self weight	1.221	1.015
3 Dead	1.221	1.015
6 MEP	1.221	1.015
4 Live	1.000	0.000
8 Balanced Snow Load	0.200	0.000
11 Rain on Snow Surcharge	0.200	0.000
20 Seismic Dir1	1.000	0.700
21 Seismic Dir2	-0.300	-0.210

120 LRFD_{11.3}-1.2D+L+0.2S+E

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.221	1.015
2 Slab self weight	1.221	1.015
3 Dead	1.221	1.015
6 MEP	1.221	1.015
4 Live	1.000	0.000
8 Balanced Snow Load	0.200	0.000
11 Rain on Snow Surcharge	0.200	0.000
20 Seismic Dir1	-1.000	-0.700
21 Seismic Dir2	0.300	0.210

121 LRFD_{11.4}-1.2D+L+0.2S+E

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.221	1.015
2 Slab self weight	1.221	1.015
3 Dead	1.221	1.015
6 MEP	1.221	1.015
4 Live	1.000	0.000
8 Balanced Snow Load	0.200	0.000



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 29
Project Name: JKV-EMS		

Loadcase Title	Strength	Service
11 Rain on Snow Surcharge	0.200	0.000
20 Seismic Dir1	-1.000	-0.700
21 Seismic Dir2	-0.300	-0.210

122 LRFD_{11.5}-1.2D+L+0.2S+E

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.221	1.015
2 Slab self weight	1.221	1.015
3 Dead	1.221	1.015
6 MEP	1.221	1.015
4 Live	1.000	0.000
8 Balanced Snow Load	0.200	0.000
11 Rain on Snow Surcharge	0.200	0.000
20 Seismic Dir1	0.300	0.210
21 Seismic Dir2	1.000	0.700

123 LRFD_{11.6}-1.2D+L+0.2S+E

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.221	1.015
2 Slab self weight	1.221	1.015
3 Dead	1.221	1.015
6 MEP	1.221	1.015
4 Live	1.000	0.000
8 Balanced Snow Load	0.200	0.000
11 Rain on Snow Surcharge	0.200	0.000
20 Seismic Dir1	-0.300	-0.210
21 Seismic Dir2	1.000	0.700

124 LRFD_{11.7}-1.2D+L+0.2S+E

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.221	1.015
2 Slab self weight	1.221	1.015
3 Dead	1.221	1.015
6 MEP	1.221	1.015
4 Live	1.000	0.000
8 Balanced Snow Load	0.200	0.000
11 Rain on Snow Surcharge	0.200	0.000
20 Seismic Dir1	0.300	0.210
21 Seismic Dir2	-1.000	-0.700

125 LRFD_{11.8}-1.2D+L+0.2S+E

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.221	1.015
2 Slab self weight	1.221	1.015
3 Dead	1.221	1.015
6 MEP	1.221	1.015
4 Live	1.000	0.000



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 30
Project Name: JKV-EMS		

Loadcase Title	Strength	Service
8 Balanced Snow Load	0.200	0.000
11 Rain on Snow Surcharge	0.200	0.000
20 Seismic Dir1	-0.300	-0.210
21 Seismic Dir2	-1.000	-0.700

126 LRFD_{13.1}-0.9D+E

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.879	0.585
2 Slab self weight	0.879	0.585
3 Dead	0.879	0.585
6 MEP	0.879	0.585
20 Seismic Dir1	1.000	0.700
21 Seismic Dir2	0.300	0.210

127 LRFD_{13.2}-0.9D+E

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.879	0.585
2 Slab self weight	0.879	0.585
3 Dead	0.879	0.585
6 MEP	0.879	0.585
20 Seismic Dir1	1.000	0.700
21 Seismic Dir2	-0.300	-0.210

128 LRFD_{13.3}-0.9D+E

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.879	0.585
2 Slab self weight	0.879	0.585
3 Dead	0.879	0.585
6 MEP	0.879	0.585
20 Seismic Dir1	-1.000	-0.700
21 Seismic Dir2	0.300	0.210

129 LRFD_{13.4}-0.9D+E

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.879	0.585
2 Slab self weight	0.879	0.585
3 Dead	0.879	0.585
6 MEP	0.879	0.585
20 Seismic Dir1	-1.000	-0.700
21 Seismic Dir2	-0.300	-0.210

130 LRFD_{13.5}-0.9D+E

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.879	0.585
2 Slab self weight	0.879	0.585



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 31
Project Name: JKV-EMS		

Loadcase Title	Strength	Service
3 Dead	0.879	0.585
6 MEP	0.879	0.585
20 Seismic Dir1	0.300	0.210
21 Seismic Dir2	1.000	0.700

131 LRFD_{13.6}-0.9D+E

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.879	0.585
2 Slab self weight	0.879	0.585
3 Dead	0.879	0.585
6 MEP	0.879	0.585
20 Seismic Dir1	-0.300	-0.210
21 Seismic Dir2	1.000	0.700

132 LRFD_{13.7}-0.9D+E

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.879	0.585
2 Slab self weight	0.879	0.585
3 Dead	0.879	0.585
6 MEP	0.879	0.585
20 Seismic Dir1	0.300	0.210
21 Seismic Dir2	-1.000	-0.700

133 LRFD_{13.8}-0.9D+E

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.879	0.585
2 Slab self weight	0.879	0.585
3 Dead	0.879	0.585
6 MEP	0.879	0.585
20 Seismic Dir1	-0.300	-0.210
21 Seismic Dir2	-1.000	-0.700

Wind Data

General

Method
ASCE/SEI 7-16 : Directional Procedure Part 1

Site Details

Site Ground Level	0"	ft, in
Orientation of Principal Axes relative to Global axes	0.0000	Â°
Mean Roof Height, h	17' 65/128"	ft, in
Level of Highest Opening in Building (Use Mean Roof Height)	17' 65/128"	ft, in
Overall Building X' Dimension	59' 4 3/4"	ft, in
Overall Building Y' Dimension	51' 6 5/8"	ft, in



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 32
Project Name: JKV-EMS		

Torsion Design Pressure Factor	75.00	%
Torsion Eccentricity	15.00	%
Wind Loading		
Basic Wind Speed, V	117.00	mph
Directionality Factor, K _d	0.850	
Ground Elevation above Sea Level, z _g	0	ft
Ground Elevation Factor, K _e	1.000	
Enclosure Classification	Enclosed	
Gust Effect factor, G	0.850	
Principal Axis	+X'	
Exposure Category	C	
Topographic Feature	None	
Principal Axis	+Y'	
Exposure Category	C	
Topographic Feature	None	
Principal Axis	-X'	
Exposure Category	C	
Topographic Feature	None	
Principal Axis	-Y'	
Exposure Category	C	
Topographic Feature	None	

Intermediate Factors

+X'

Height above ground, z	15' 0"	ft, in
Exposure Coefficient, K _z	0.849	
Topographic Factor, K _{zt}	1.000	
Height above ground, z	17' 65/128"	ft, in
Exposure Coefficient, K _z	0.872	
Topographic Factor, K _{zt}	1.000	

+Y'

Height above ground, z	15' 0"	ft, in
Exposure Coefficient, K _z	0.849	
Topographic Factor, K _{zt}	1.000	
Height above ground, z	17' 65/128"	ft, in
Exposure Coefficient, K _z	0.872	
Topographic Factor, K _{zt}	1.000	

-X'

Height above ground, z	15' 0"	ft, in
Exposure Coefficient, K _z	0.849	
Topographic Factor, K _{zt}	1.000	
Height above ground, z	17' 65/128"	ft, in
Exposure Coefficient, K _z	0.872	
Topographic Factor, K _{zt}	1.000	



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 33
Project Name: JKV-EMS		

-Y'

Height above ground, z 15' 0" ft, in
Exposure Coefficient, K_z 0.849
Topographic Factor, K_{zt} 1.000
Height above ground, z 17' 65/128" ft, in
Exposure Coefficient, K_z 0.872
Topographic Factor, K_{zt} 1.000

Velocity Pressures

+X'

Height above ground, z 15' 0" ft, in
Velocity Pressure, q_z 25.3 psf
Height above ground, z 17' 65/128" ft, in
Velocity Pressure, q_z 26.0 psf

+Y'

Height above ground, z 15' 0" ft, in
Velocity Pressure, q_z 25.3 psf
Height above ground, z 17' 65/128" ft, in
Velocity Pressure, q_z 26.0 psf

-X'

Height above ground, z 15' 0" ft, in
Velocity Pressure, q_z 25.3 psf
Height above ground, z 17' 65/128" ft, in
Velocity Pressure, q_z 26.0 psf

-Y'

Height above ground, z 15' 0" ft, in
Velocity Pressure, q_z 25.3 psf
Height above ground, z 17' 65/128" ft, in
Velocity Pressure, q_z 26.0 psf

Generated Loadcases

12 Wind +X', GCpi 0.18, -Cp

Wall Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft ²]	Applied Load [kip]
WI 15	Zero	0.0	10	0.0
	S	-20.1	127	-2.6
WI 14	Zero	0.0	64	0.0
	L	-15.0	823	-12.4
WI 13	Zero	0.0	79	0.0
	S	-20.1	1012	-20.4



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 34
Project Name: JKV-EMS		

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
WI 19	Zero	0.0	42	0.0
	W15	12.5	476	6.0
	W17	13.0	65	0.8

Average Wall Loads

Wind 1
Leew -
Left -
Right -

Roof Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
RI 3	1	-24.5	14	-0.3
	2	-24.5	14	-0.3
	3	-15.7	28	-0.4
	4	-11.3	36	-0.4
	1	-24.5	264	-6.5
	2	-24.5	264	-6.5
	3	-15.7	527	-8.3
	4	-11.3	783	-8.8
	1	-24.5	27	-0.7
	2	-24.5	27	-0.7
	3	-15.7	11	-0.2
	1	-24.5	131	-3.2
	2	-24.5	131	-3.2
	3	-15.7	261	-4.1
	4	-11.3	15	-0.2
	1	-24.5	31	-0.8
	2	-24.5	31	-0.8
	3	-15.7	59	-0.9

Lateral Loads by Level

Level	Total [kip]		Center [ft, in]	
	X	Y	X	Y
St. DF (Dropped Footing)	0.0	0.0	-	-
St. FF (First Floor)	9.6	-8.9	24' 5 15/16"	19' 9 7/32"
St. TOS HB (TOS High Bearing)	9.6	-8.9	24' 5 15/16"	19' 9 5/128"
Total	19.2	-17.8	-	-

Total Loads

1
-
-



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 35
Project Name: JKV-EMS		

13 Wind +X', GCpi 0.18, +Cp

Wall Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
WI 15	Zero	0.0	10	0.0
	S	-20.1	127	-2.6
WI 14	Zero	0.0	64	0.0
	L	-15.0	823	-12.4
WI 13	Zero	0.0	79	0.0
	S	-20.1	1012	-20.4
WI 19	Zero	0.0	42	0.0
	W15	12.5	476	6.0
	W17	13.0	65	0.8

Average Wall Loads

Wind 1
Leew -
Left -
Right -

Roof Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
RI 3	1	-8.6	14	-0.1
	2	-8.6	14	-0.1
	3	-8.6	28	-0.2
	4	-8.6	36	-0.3
	1	-8.6	264	-2.3
	2	-8.6	264	-2.3
	3	-8.6	527	-4.6
	4	-8.6	783	-6.8
	1	-8.6	27	-0.2
	2	-8.6	27	-0.2
	3	-8.6	11	-0.1
	1	-8.6	131	-1.1
	2	-8.6	131	-1.1
	3	-8.6	261	-2.3
	4	-8.6	15	-0.1
	1	-8.6	31	-0.3
	2	-8.6	31	-0.3
	3	-8.6	59	-0.5

Lateral Loads by Level

Level	Total [kip]		Center [ft, in]	
	X	Y	X	Y
St. DF (Dropped Footing)	0.0	0.0	-	-
St. FF (First Floor)	9.6	-8.9	24' 5 15/16"	19' 9 7/32"
St. TOS HB (TOS High Bearing)	9.6	-8.9	24' 5 15/16"	19' 9 5/128"



209 SE DOUGLAS
LEE'S SUMMIT, MO 64063
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Made by: WNH

Date: 06/16/2025

Job No. 25-003

Revision:

Date:

Sheet No. 36

Project Name: JKV-EMS

Level	Total [kip]		Center [ft, in]	
	X	Y	X	Y
Total	19.2	-17.8	-	-

Total Loads

1
-
-

14 Wind +Y', GCpi 0.18, -Cp

Wall Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
WI 15	Zero	0.0	10	0.0
	L	-15.7	127	-2.0
WI 14	Zero	0.0	64	0.0
	S	-20.1	823	-16.6
WI 13	Zero	0.0	79	0.0
	W15	12.5	891	11.1
	W17	13.0	121	1.6
WI 19	Zero	0.0	42	0.0
	S	-20.1	541	-10.9

Average Wall Loads

Wind 1
Leew -
Left -
Right -

Roof Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
RI 3	1	-24.5	13	-0.3
	2	-24.5	13	-0.3
	3	-15.7	26	-0.4
	4	-11.3	21	-0.2
	1	-24.5	476	-11.7
	2	-24.5	476	-11.7
	3	-15.7	983	-15.4
	4	-11.3	582	-6.6
	1	-24.5	17	-0.4
	2	-24.5	17	-0.4
	3	-15.7	28	-0.4

Lateral Loads by Level



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 37
Project Name: JKV-EMS		

Level	Total [kip]		Center [ft, in]	
	X	Y	X	Y
St. DF (Dropped Footing)	0.0	0.0	-	-
St. FF (First Floor)	2.8	7.3	31' 9 13/64"	38' 6 13/16"
St. TOS HB (TOS High Bearing)	2.8	7.4	31' 8 235/256"	38' 6 13/16"
Total	5.7	14.7	-	-

Total Loads

5
1
-

15 Wind +Y', GCpi 0.18,+Cp

Wall Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
WI 15	Zero	0.0	10	0.0
	L	-15.7	127	-2.0
WI 14	Zero	0.0	64	0.0
	S	-20.1	823	-16.6
WI 13	Zero	0.0	79	0.0
	W15	12.5	891	11.1
	W17	13.0	121	1.6
WI 19	Zero	0.0	42	0.0
	S	-20.1	541	-10.9

Average Wall Loads

Wind 1
Leew -
Left -
Right -

Roof Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
RI 3	1	-8.6	13	-0.1
	2	-8.6	13	-0.1
	3	-8.6	26	-0.2
	4	-8.6	21	-0.2
	1	-8.6	476	-4.1
	2	-8.6	476	-4.1
	3	-8.6	983	-8.5
	4	-8.6	582	-5.0
	1	-8.6	17	-0.1
	2	-8.6	17	-0.1
	3	-8.6	28	-0.2



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 38
Project Name: JKV-EMS		

Lateral Loads by Level

Level	Total [kip]		Center [ft, in]	
	X	Y	X	Y
St. DF (Dropped Footing)	0.0	0.0	-	-
St. FF (First Floor)	2.8	7.3	31' 9 13/64"	38' 6 13/16"
St. TOS HB (TOS High Bearing)	2.8	7.4	31' 8 235/256"	38' 6 13/16"
Total	5.7	14.7	-	-

Total Loads

5
1
-

16 Wind -X', GCpi 0.18, -Cp

Wall Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
WI 15	Zero	0.0	10	0.0
	S	-20.1	127	-2.6
WI 14	Zero	0.0	64	0.0
	W15	12.5	724	9.1
	W17	13.0	99	1.3
WI 13	Zero	0.0	79	0.0
	S	-20.1	1012	-20.4
WI 19	Zero	0.0	42	0.0
	L	-15.0	541	-8.1

Average Wall Loads

Wind 1
Leew -
Left -
Right -

Roof Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
RI 3	1	-24.5	14	-0.3
	2	-24.5	14	-0.3
	3	-15.7	25	-0.4
	1	-24.5	27	-0.7
	2	-24.5	27	-0.7
	3	-15.7	11	-0.2
	1	-24.5	411	-10.1
	2	-24.5	411	-10.1
	3	-15.7	822	-12.9
	4	-11.3	798	-9.0
	1	-24.5	14	-0.3



209 SE DOUGLAS
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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 39
Project Name: JKV-EMS		

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
	2	-24.5	14	-0.3
	3	-15.7	28	-0.4
	4	-11.3	36	-0.4

Lateral Loads by Level

Level	Total [kip]		Center [ft, in]	
	X	Y	X	Y
St. DF (Dropped Footing)	0.0	0.0	-	-
St. FF (First Floor)	-9.2	-8.9	24' 5 15/16"	19' 139/256"
St. TOS HB (TOS High Bearing)	-9.3	-8.9	24' 5 15/16"	19' 189/256"
Total	-18.5	-17.8	-	-

Total Loads

-
-
-

17 Wind -X', GCpi 0.18, +Cp

Wall Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
WI 15	Zero	0.0	10	0.0
	S	-20.1	127	-2.6
WI 14	Zero	0.0	64	0.0
	W15	12.5	724	9.1
	W17	13.0	99	1.3
WI 13	Zero	0.0	79	0.0
	S	-20.1	1012	-20.4
WI 19	Zero	0.0	42	0.0
	L	-15.0	541	-8.1

Average Wall Loads

Wind 1
Leew -
Left -
Right -

Roof Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
RI 3	1	-8.6	14	-0.1
	2	-8.6	14	-0.1
	3	-8.6	25	-0.2
	1	-8.6	27	-0.2



209 SE DOUGLAS
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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 40
Project Name: JKV-EMS		

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
	2	-8.6	27	-0.2
	3	-8.6	11	-0.1
	1	-8.6	411	-3.6
	2	-8.6	411	-3.6
	3	-8.6	822	-7.1
	4	-8.6	798	-6.9
	1	-8.6	14	-0.1
	2	-8.6	14	-0.1
	3	-8.6	28	-0.2
	4	-8.6	36	-0.3

Lateral Loads by Level

Level	Total [kip]		Center [ft, in]	
	X	Y	X	Y
St. DF (Dropped Footing)	0.0	0.0	-	-
St. FF (First Floor)	-9.2	-8.9	24' 5 15/16"	19' 139/256"
St. TOS HB (TOS High Bearing)	-9.3	-8.9	24' 5 15/16"	19' 189/256"
Total	-18.5	-17.8	-	-

Total Loads

-
-
-

18 Wind -Y', GCpi 0.18, -Cp

Wall Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
WI 15	Zero	0.0	10	0.0
	W15	12.5	112	1.4
	W17	13.0	15	0.2
WI 14	Zero	0.0	64	0.0
	S	-20.1	823	-16.6
WI 13	Zero	0.0	79	0.0
	L	-15.7	1012	-15.9
WI 19	Zero	0.0	42	0.0
	S	-20.1	541	-10.9

Average Wall Loads

Wind 1
Leew -
Left -
Right -

Roof Zone Loads



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 41
Project Name: JKV-EMS		

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
RI 3	1	-24.5	17	-0.4
	2	-24.5	17	-0.4
	3	-15.7	28	-0.4
	1	-24.5	174	-4.3
	2	-24.5	174	-4.3
	3	-15.7	348	-5.5
	4	-11.3	34	-0.4
	1	-24.5	17	-0.4
	2	-24.5	17	-0.4
	3	-15.7	30	-0.5
	1	-24.5	10	-0.2
	2	-24.5	10	-0.2
	3	-15.7	20	-0.3
	4	-11.3	16	-0.2
	1	-24.5	276	-6.8
	2	-24.5	276	-6.8
	3	-15.7	552	-8.7
	4	-11.3	566	-6.4
	1	-24.5	13	-0.3
	2	-24.5	13	-0.3
	3	-15.7	26	-0.4
	4	-11.3	21	-0.2

Lateral Loads by Level

Level	Total [kip]		Center [ft, in]	
	X	Y	X	Y
St. DF (Dropped Footing)	0.0	0.0	-	-
St. FF (First Floor)	2.8	-8.7	30' 7 17/128"	38' 6 13/16"
St. TOS HB (TOS High Bearing)	2.8	-8.8	30' 7 43/128"	38' 6 13/16"
Total	5.7	-17.5	-	-

Total Loads

5
-
-

19 Wind -Y', GCpi 0.18, +Cp

Wall Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
WI 15	Zero	0.0	10	0.0
	W15	12.5	112	1.4
	W17	13.0	15	0.2
WI 14	Zero	0.0	64	0.0
	S	-20.1	823	-16.6
WI 13	Zero	0.0	79	0.0
	L	-15.7	1012	-15.9
WI 19	Zero	0.0	42	0.0



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 42
Project Name: JKV-EMS		

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
	S	-20.1	541	-10.9

Average Wall Loads

Wind 1
Leew -
Left -
Right -

Roof Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
RI 3	1	-8.6	17	-0.1
	2	-8.6	17	-0.1
	3	-8.6	28	-0.2
	1	-8.6	174	-1.5
	2	-8.6	174	-1.5
	3	-8.6	348	-3.0
	4	-8.6	34	-0.3
	1	-8.6	17	-0.1
	2	-8.6	17	-0.1
	3	-8.6	30	-0.3
	1	-8.6	10	-0.1
	2	-8.6	10	-0.1
	3	-8.6	20	-0.2
	4	-8.6	16	-0.1
	1	-8.6	276	-2.4
	2	-8.6	276	-2.4
	3	-8.6	552	-4.8
	4	-8.6	566	-4.9
	1	-8.6	13	-0.1
	2	-8.6	13	-0.1
	3	-8.6	26	-0.2
	4	-8.6	21	-0.2

Lateral Loads by Level

Level	Total [kip]		Center [ft, in]	
	X	Y	X	Y
St. DF (Dropped Footing)	0.0	0.0	-	-
St. FF (First Floor)	2.8	-8.7	30' 7 17/128"	38' 6 13/16"
St. TOS HB (TOS High Bearing)	2.8	-8.8	30' 7 43/128"	38' 6 13/16"
Total	5.7	-17.5	-	-

Total Loads

5
-
-



209 SE DOUGLAS
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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 43
Project Name: JKV-EMS		

Structure

Seismic Loading Summary

General			
Method	ASCE7		
Structure details			
Height to highest level	16' 0"	ft, in	
Ignore seismic in floor (and below)	St. FF (First		
Number of storeys	1		
Two-Period Spectrum			
Max earthquake spectral response			
S _s short (0.2 s) period (mapped)	9.90	% g	
S ₁ 1.0 s period (mapped)	6.80	% g	
Site Class	D - Stiff soil		ASCE7-16 Table
Risk Category	III		ASCE7-16 Table 1.5-
Design spectral response			
S _{DS} short (0.2 s) period	10.56	% g	ASCE7-16 Clause
S _{D1} 1.0 s period	10.88	% g	ASCE7-16 Clause
Seismic Importance Factor, I _e	1.250		ASCE7-16 Table 1.5-
Seismic Design Category	B		ASCE7-16 Clause
T _L long period transition period	12.000	sec	
T _S (S _{D1} /S _{DS})	1.030	sec	
Effective Seismic Weight, W	155.2	kip	ASCE7-16 Clause
Structure Type			ASCE7-16 Table
Direction Dir1	All other structural systems		
Direction Dir2	All other structural systems		
Basic seismic force resisting system			ASCE7-16 Table
Direction Dir1	H. Steel Systems not Specifically detailed for Seismic		
Direction Dir2	H. Steel Systems not Specifically detailed for Seismic		
	Direction	Direction	
Response Modification Factor, R	3.000	3.000	
System Over-Strength Factor, Ω ₀	3.000	3.000	
Deflection Amplification Factor, Cd	3.000	3.000	
Redundancy Factor, p	1.000	1.000	ASCE7-16 Clause
Approximate fundamental period,	0.160	0.160	ASCE7-16 Clause
Exponent related to structural	1.000	1.000	
Seismic response coefficient, C _s	0.044	0.044	ASCE7-16 Clause
Seismic base shear, V [kip]	6.8	6.8	
Scaling Factor for Forces (1.00	-	-	ASCE7-16 Clause
Scaling Factor for Drifts	-	-	ASCE7-16 Clause
Structure Plan Irregularities - User Defined			ASCE7-16 Table
Plan irreg 1a - torsion	No		
Plan irreg 1b - extreme torsion	No		
Plan irreg 2 - re-entrant corners	No		
Plan irreg 3 - diaphragm	No		
Plan irreg 4 - out of plane	No		
Plan irreg 5 - Non parallel systems	No		
Structure Vertical Irregularities - User Defined			ASCE7-16 Table
Vert irreg 1a - soft story	No		
Vert irreg 1b - extreme soft story	No		
Vert irreg 2 - weight mass	No		
Vert irreg 3 - geometric	No		



209 SE DOUGLAS
LEE'S SUMMIT, MO 64063
816.444.3144
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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 44
Project Name: JKV-EMS		

Vert irreg 4 - in plane No
Vert irreg 5a - weak story No
Vert irreg 5b - extreme weak story No

Equivalent Lateral Force Procedure is permitted (ASCE7-16 12.8).

With orthogonal seismic combination procedure (ASCE7-16 12.5.3).

Reference	Level [ft, in]	Weight [kip]	Direction Dir1			Direction Dir2		
			C _v	F [kip]	Ecc [ft, in]	C _v	F [kip]	Ecc [ft, in]
St. TOS HB (TOS High Bearing)	17' 65/128"	155.2	1.000	6.8	0"	1.000	6.8	0"

Analysis procedure to be used:

Equivalent Lateral Force Procedure

Structure

Snow Data

General

Met A

Basic data

Ground Snow Load,	20.0	ASCE7-16
Terrain Category	C (open terrain scattered	
Exposure	Partially exposed	
Exposure Factor, C _e	1.000	ASCE7-16
Thermal Condition	All except below	
Thermal Factor, C _t	1.000	ASCE7-16
Risk Category	III	
Snow Importance	1.100	
Flat Roof Snow	15.4	ASCE7-16

Snow load cases

Minimum Snow Load	Y	
Minimum snow load,	2	ASCE7-16
Balanced Snow Load	Y	
Unbalanced Snow	Y	
Drift Snow Load	Y	
Rain on Snow	Y	
Rain on snow	5	ASCE7-16



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Revision:	Date:	Sheet No. 45
Project Name: JKV-EMS		

BUILDING DESIGN



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Revision:	Date:	Sheet No. 46
Project Name: JKV-EMS		

Steel

Beam Design Summary

Static

Member Reference	Group Ref.	Span	Section	Grade	Length [ft, in]	No. Connectors	Camber	Utilization	Status
TOS HBB55	SBR15	1	W 12x14	A992-50	15' 4"		0.00	0.600	✓ Pass
TOS HBB97	SBR9	1	HSS 5x2-1/2x1/4	A500B-46	1' 7 5/8"		0.00	0.143	✓ Pass
TOS HBB97	SBR9	2	HSS 5x2-1/2x1/4	A500B-46	5' 0"		0.00	0.224	✓ Pass
TOS HBB98	SBR9	1	HSS 5x2-1/2x1/4	A500B-46	1' 7 5/8"		0.00	0.177	✓ Pass
TOS HBB98	SBR9	2	HSS 5x2-1/2x1/4	A500B-46	5' 0"		0.00	0.410	✓ Pass
TOS HBB99	SBR9	1	HSS 5x2-1/2x1/4	A500B-46	1' 7 5/8"		0.00	0.184	✓ Pass
TOS HBB99	SBR9	2	HSS 5x2-1/2x1/4	A500B-46	5' 0"		0.00	0.470	✓ Pass
TOS HBB103	SBR9	1	HSS 5x2-1/2x1/4	A500B-46	1' 7 5/8"		0.00	0.066	✓ Pass
TOS HBB103	SBR9	2	HSS 5x2-1/2x1/4	A500B-46	5' 0"		0.00	0.376	✓ Pass
TOS HBB54	SBR20	1	W 12x14	A992-50	22' 10 1/2"		0.00	0.620	✓ Pass
TOS HBB41	SBR7	1	HSS 5x2-1/2x1/4	A500B-46	4' 0"		0.00	0.227	✓ Pass
TOS HBB41	SBR7	2	HSS 5x2-1/2x1/4	A500B-46	3' 1 3/4"		0.00	0.234	✓ Pass
TOS HBB107	SBR9	1	HSS 5x2-1/2x1/4	A500B-46	1' 7 5/8"		0.00	0.198	✓ Pass
TOS HBB107	SBR9	2	HSS 5x2-1/2x1/4	A500B-46	5' 0"		0.00	0.334	✓ Pass
TOS HBB108	SBR9	1	HSS 5x2-1/2x1/4	A500B-46	1' 7 5/8"		0.00	0.071	✓ Pass
TOS HBB108	SBR9	2	HSS 5x2-1/2x1/4	A500B-46	5' 0"		0.00	0.132	✓ Pass
TOS HBB29	SBR2	1	HSS 5x2-1/2x1/4	A500B-46	1' 7 5/8"		0.00	0.060	✓ Pass
TOS HBB29	SBR2	2	HSS 5x2-1/2x1/4	A500B-46	5' 0"		0.00	0.093	✓ Pass
TOS HBB102	SBR9	1	HSS 5x2-1/2x1/4	A500B-46	1' 7 5/8"		0.00	0.087	✓ Pass
TOS HBB102	SBR9	2	HSS 5x2-1/2x1/4	A500B-46	5' 0"		0.00	0.502	✓ Pass
TOS HBB101	SBR9	1	HSS 5x2-1/2x1/4	A500B-46	1' 7 5/8"		0.00	0.140	✓ Pass
TOS HBB101	SBR9	2	HSS 5x2-1/2x1/4	A500B-46	5' 0"		0.00	0.517	✓ Pass
TOS HBB105	SBR9	1	HSS 5x2-1/2x1/4	A500B-46	1' 7 5/8"		0.00	0.198	✓ Pass
TOS HBB105	SBR9	2	HSS 5x2-1/2x1/4	A500B-46	5' 0"		0.00	0.512	✓ Pass
TOS HBB106	SBR9	1	HSS 5x2-1/2x1/4	A500B-46	1' 7 5/8"		0.00	0.242	✓ Pass
TOS HBB106	SBR9	2	HSS 5x2-1/2x1/4	A500B-46	5' 0"		0.00	0.418	✓ Pass
TOS HBB119	SBR19	1	W 16x26	A992-50	35' 5/8"		0.00	0.525	✓ Pass



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Revision:	Date:	Sheet No. 47
Project Name: JKV-EMS		

Member Reference	Group Ref.	Span	Section	Grade	Length [ft, in]	No. Connectors	Camber	Utilization	Status
TOS HBB100	SBR9	1	HSS 5x2-1/2x1/4	A500B-46	1' 7 5/8"		0.00	0.121	✓ Pass
TOS HBB100	SBR9	2	HSS 5x2-1/2x1/4	A500B-46	5' 0"		0.00	0.585	✓ Pass
TOS HBB118	SBR7	1	HSS 5x2-1/2x1/4	A500B-46	4' 0"		0.00	0.077	✓ Pass
TOS HBB118	SBR7	2	HSS 5x2-1/2x1/4	A500B-46	3' 1 3/4"		0.00	0.082	✓ Pass
TOS HBB117	SBR7	1	HSS 5x2-1/2x1/4	A500B-46	4' 0"		0.00	0.156	✓ Pass
TOS HBB117	SBR7	2	HSS 5x2-1/2x1/4	A500B-46	3' 1 3/4"		0.00	0.188	✓ Pass
TOS HBB116	SBR7	1	HSS 5x2-1/2x1/4	A500B-46	4' 0"		0.00	0.082	✓ Pass
TOS HBB116	SBR7	2	HSS 5x2-1/2x1/4	A500B-46	3' 1 3/4"		0.00	0.093	✓ Pass
TOS HBB115	SBR4	1	HSS 5x2-1/2x1/4	A500B-46	4' 0"		0.00	0.158	✓ Pass
TOS HBB115	SBR4	2	HSS 5x2-1/2x1/4	A500B-46	3' 1 3/4"		0.00	0.195	✓ Pass
TOS HBB114	SBR3	1	HSS 5x2-1/2x1/4	A500B-46	4' 0"		0.00	0.253	✓ Pass
TOS HBB114	SBR3	2	HSS 5x2-1/2x1/4	A500B-46	3' 1 3/4"		0.00	0.253	✓ Pass
TOS HBB133	SBR10	1	W 8x10	A992-50	1' 5 5/8"		0.00	0.019	✓ Pass
TOS HBB60	SBR1	1	W 21x44	A992-50	30' 11 3/8"		0.00	0.835	✓ Pass
TOS HBB56	SBR19	1	W 16x26	A992-50	35' 5/8"		0.00	0.630	✓ Pass
TOS HBB53	SBR18	1	L 6x6x3/8	A36-36	5' 0"		0.00	0.148	✓ Pass
TOS HBB53	SBR18	2	L 6x6x3/8	A36-36	5' 0"		0.00	0.237	✓ Pass
TOS HBB53	SBR18	3	L 6x6x3/8	A36-36	5' 0"		0.00	0.281	✓ Pass
TOS HBB53	SBR18	4	L 6x6x3/8	A36-36	5' 0"		0.00	0.281	✓ Pass
TOS HBB53	SBR18	5	L 6x6x3/8	A36-36	5' 0"		0.00	0.281	✓ Pass
TOS HBB53	SBR18	6	L 6x6x3/8	A36-36	5' 0"		0.00	0.439	✓ Pass
TOS HBB53	SBR18	7	L 6x6x3/8	A36-36	2' 4 3/4"		0.00	0.439	✓ Pass
TOS HBB52	SBR17	1	L 6x6x3/8	A36-36	3' 6"		0.00	0.139	✓ Pass
TOS HBB52	SBR17	2	L 6x6x3/8	A36-36	5' 0"		0.00	0.301	✓ Pass
TOS HBB52	SBR17	3	L 6x6x3/8	A36-36	5' 0"		0.00	0.336	✓ Pass
TOS HBB52	SBR17	4	L 6x6x3/8	A36-36	5' 0"		0.00	0.336	✓ Pass
TOS HBB52	SBR17	5	L 6x6x3/8	A36-36	1' 10 3/4"		0.00	0.336	✓ Pass
TOS HBB50	SBR16	1	L 6x6x3/8	A36-36	3' 6"		0.00	0.097	✓ Pass
TOS HBB50	SBR16	2	L 6x6x3/8	A36-36	5' 0"		0.00	0.155	✓ Pass
TOS HBB50	SBR16	3	L 6x6x3/8	A36-36	5' 0"		0.00	0.155	✓ Pass
TOS HBB50	SBR16	4	L 6x6x3/8	A36-36	5' 0"		0.00	0.155	✓ Pass
TOS HBB50	SBR16	5	L 6x6x3/8	A36-36	5' 0"		0.00	0.155	✓ Pass
TOS HBB50	SBR16	6	L 6x6x3/8	A36-36	5' 0"		0.00	0.155	✓ Pass
TOS HBB50	SBR16	7	L 6x6x3/8	A36-36	5' 0"		0.00	0.155	✓ Pass
TOS HBB50	SBR16	8	L 6x6x3/8	A36-36	5' 0"		0.00	0.155	✓ Pass
TOS HBB50	SBR16	9	L 6x6x3/8	A36-36	5' 0"		0.00	0.155	✓ Pass
TOS HBB50	SBR16	10	L 6x6x3/8	A36-36	5' 0"		0.00	0.155	✓ Pass
TOS HBB50	SBR16	11	L 6x6x3/8	A36-36	5' 0"		0.00	0.155	✓ Pass
TOS HBB50	SBR16	12	L 6x6x3/8	A36-36	2' 4 3/4"		0.00	0.155	✓ Pass



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Revision:	Date:	Sheet No. 48
Project Name: JKV-EMS		

Member Reference	Group Ref.	Span	Section	Grade	Length [ft, in]	No. Connectors	Camber	Utilization	Status
TOS HBB49	SBR15	1	HSS 5x2-1/2x1/4	A500B-46	4' 0"		0.00	0.163	✓ Pass
TOS HBB49	SBR15	2	HSS 5x2-1/2x1/4	A500B-46	2' 0"		0.00	0.389	✓ Pass
TOS HBB49	SBR15	3	HSS 5x2-1/2x1/4	A500B-46	1' 7 5/8"		0.00	0.389	✓ Pass
TOS HBB120	SBR8	1	HSS 5x2-1/2x1/4	A500B-46	4' 0"		0.00	0.278	✓ Pass
TOS HBB120	SBR8	2	HSS 5x2-1/2x1/4	A500B-46	3' 7 5/8"		0.00	0.286	✓ Pass
TOS HBB132	SBR14	1	HSS 5x2-1/2x1/4	A500B-46	4' 0"		0.00	0.431	✓ Pass
TOS HBB132	SBR14	2	HSS 5x2-1/2x1/4	A500B-46	2' 0"		0.00	0.991	✓ Pass
TOS HBB132	SBR14	3	HSS 5x2-1/2x1/4	A500B-46	1' 7 5/8"		0.00	0.985	✓ Pass
TOS HBB131	SBR6	1	HSS 5x2-1/2x1/4	A500B-46	4' 0"		0.00	0.074	✓ Pass
TOS HBB131	SBR6	2	HSS 5x2-1/2x1/4	A500B-46	3' 7 5/8"		0.00	0.097	✓ Pass
TOS HBB130	SBR6	1	HSS 5x2-1/2x1/4	A500B-46	4' 0"		0.00	0.159	✓ Pass
TOS HBB130	SBR6	2	HSS 5x2-1/2x1/4	A500B-46	3' 7 5/8"		0.00	0.164	✓ Pass
TOS HBB129	SBR6	1	HSS 5x2-1/2x1/4	A500B-46	4' 0"		0.00	0.197	✓ Pass
TOS HBB129	SBR6	2	HSS 5x2-1/2x1/4	A500B-46	3' 7 5/8"		0.00	0.197	✓ Pass
TOS HBB104	SBR9	1	HSS 5x2-1/2x1/4	A500B-46	1' 7 5/8"		0.00	0.170	✓ Pass
TOS HBB104	SBR9	2	HSS 5x2-1/2x1/4	A500B-46	5' 0"		0.00	0.394	✓ Pass
TOS HBB128	SBR6	1	HSS 5x2-1/2x1/4	A500B-46	4' 0"		0.00	0.214	✓ Pass
TOS HBB128	SBR6	2	HSS 5x2-1/2x1/4	A500B-46	3' 7 5/8"		0.00	0.214	✓ Pass
TOS HBB127	SBR5	1	HSS 5x2-1/2x1/4	A500B-46	4' 0"		0.00	0.234	✓ Pass
TOS HBB127	SBR5	2	HSS 5x2-1/2x1/4	A500B-46	3' 7 5/8"		0.00	0.242	✓ Pass

Column Design Summary

Static

Member Reference	Group Ref.	Stack	Section	Grade	Length [ft, in]	Utilization	Status
C248	SCR1	1	HSS 5x5x3/8	A500B-46	18' 65/128"	0.095	✓ Pass
C244	SCR2	1	HSS 5x5x3/8	A500B-46	18' 65/128"	0.246	✓ Pass
C245	SCR3	1	HSS 5x5x3/8	A500B-46	18' 65/128"	0.669	✓ Pass

Brace Design Summary

Static



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Revision:	Date:	Sheet No. 49
Project Name: JKV-EMS		

Member Reference	Group Ref.	Span	Section	Grade	Length [ft, in]	Utilization	Status
SBR4	SBrR2	1	HSS 4x4x0.25	A500B-46	29' 1 39/64"	0.240	✓ Pass
SBR3	SBrR2	1	HSS 4x4x0.25	A500B-46	29' 1 39/64"	0.000	Not required
SBR2	SBrR1	1	HSS 4x4x0.25	A500B-46	23' 8 17/128"	0.000	Not required
SBR1	SBrR1	1	HSS 4x4x0.25	A500B-46	23' 8 17/128"	0.000	Not required

Concrete

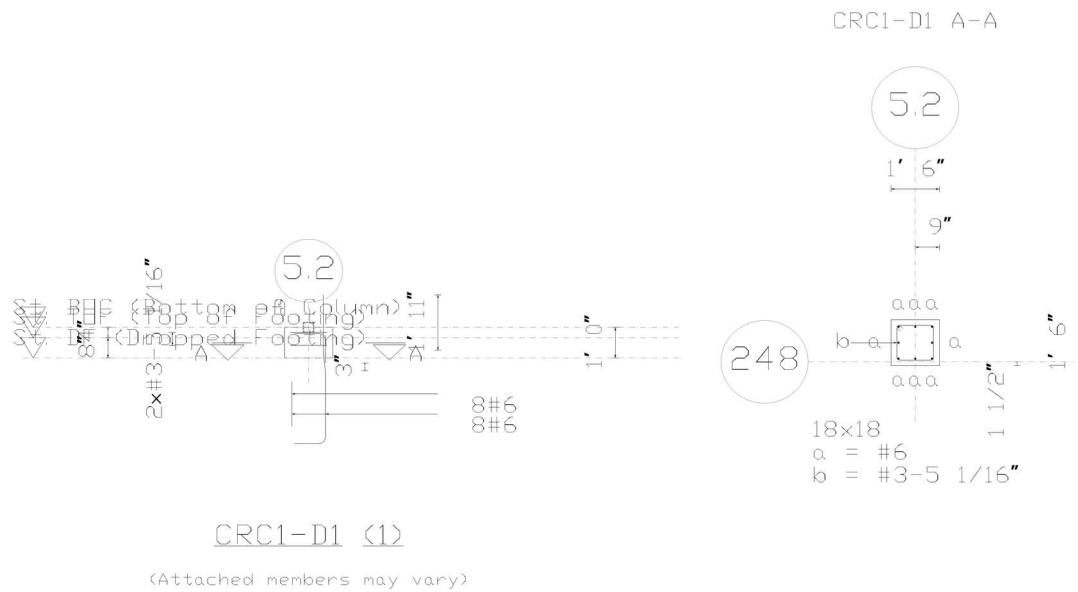
Column Group Design Summary

Static

Reference	No. in Group	Critical Member	Stacks	Critical Stack	Section	Grade	Max. UR	Status
CRC1	2	CC250	1	1	18"x18"	4000 psi	0.071	✓ Pass

Reference	Stacks	Critical Stack	Section	Grade	Max. UR	Status
CC249	2	1	18"x18"	4000 psi	0.626	✓ Pass

CC250



Static Design Summary

Longitudinal Bars Summary

Stack	Section	Longitudinal Bars	Analysis	Combination	Critical position	Ratio	Status
1	18"x18"	8#6	Grillage Chase Down	10	Top	0.071	✓ Pass

Ties Summary



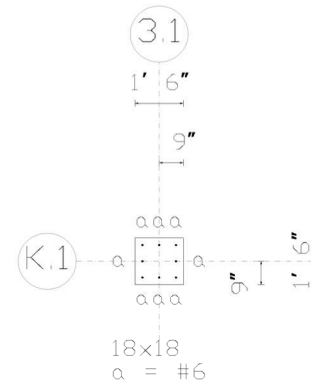
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Revision:	Date:	Sheet No. 50
Project Name: JKV-EMS		

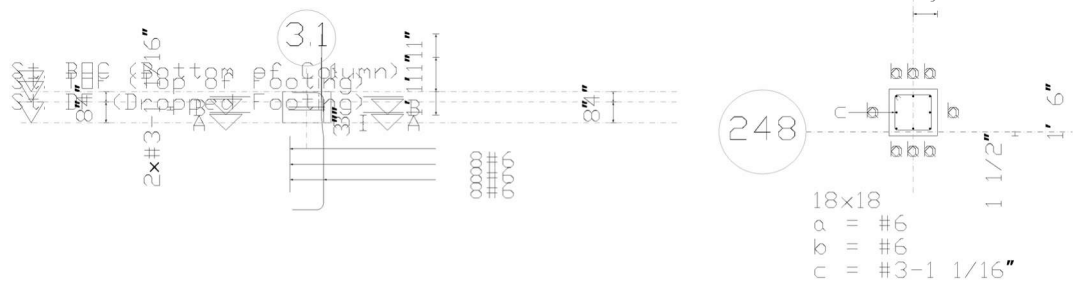
Stack	Section	Top support ties	Span ties	Bottom support ties	Analysis	Combination	Ratio	Status
1	18"x18"	-	1#3-8	-	3D Building Analysis	2	0.000	✓ Pass

CC249

CC249 B-B



CC249 A-A



CC249

Static Design Summary

Longitudinal Bars Summary

Stack	Section	Longitudinal Bars	Analysis	Combination	Critical position	Ratio	Status
2	18"x18"	8#6	3D Building Analysis	29	Bottom	0.011	✓ Pass
1	18"x18"	8#6	3D Building Analysis	17	Top	0.200	✓ Pass

Ties Summary



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Revision:	Date:	Sheet No. 51
Project Name: JKV-EMS		

Stack	Section	Top support ties	Span ties	Bottom support ties	Analysis	Combination	Ratio	Status
2	18"x18"	-	1#3-6	-	3D Building Analysis	69	0.206	✓ Pass
1	18"x18"	-	1#3-6	-	3D Building Analysis	17	0.626	✓ Pass

Slab/Mat Panels

Mat: MF 1, Panel: SI 2

Static Design Summary

Reinforcement Design Summary

Reinforcement Layer	Analysis	Combination	Reinforcement Provided	Area Provided [in ² /ft]	Area Required [in ² /ft]	Utilization
Bottom-X	3D Building Analysis	19 LRFD _{7.6} -1.2D+1.6Lr+0.5W	#6-1' 0"-B1	0.44	0.05	0.110
Bottom-Y	3D Building Analysis	27 LRFD _{8.6} -1.2D+1.6S+0.5W	#6-1' 0"-B2	0.44	0.04	0.088
Top-X	3D Building Analysis	27 LRFD _{8.6} -1.2D+1.6S+0.5W	#6-1' 0"-T1	0.44	0.04	0.093
Top-Y	FE Chase Down	10 LRFD _{6.1} -1.2D+L+1.6S	#6-1' 0"-T2	0.44	0.06	0.144

Bearing Pressure Summary

Bearing pressure checks	Max. value [ksf]	Limit [ksf]	Status	Utilization
Pressure	1.15	1.50	✓ Pass	0.766

Mat: MF 1, Panel: SI 3

Static Design Summary

Reinforcement Design Summary

Reinforcement Layer	Analysis	Combination	Reinforcement Provided	Area Provided [in ² /ft]	Area Required [in ² /ft]	Utilization
Bottom-X	3D Building Analysis	67 LRFD _{10.6} -1.2D+L+0.5S+W	#6-1' 0"-B1	0.44	0.03	0.074
Bottom-Y	3D Building Analysis	67 LRFD _{10.6} -1.2D+L+0.5S+W	#6-1' 0"-B2	0.44	0.05	0.103
Top-X	3D Building Analysis	27 LRFD _{8.6} -1.2D+1.6S+0.5W	#6-1' 0"-T1	0.44	0.04	0.088
Top-Y	3D Building Analysis	27 LRFD _{8.6} -1.2D+1.6S+0.5W	#6-1' 0"-T2	0.44	0.05	0.107

Bearing Pressure Summary



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Revision:	Date:	Sheet No. 52
Project Name: JKV-EMS		

Bearing pressure checks	Max. value [ksf]	Limit [ksf]	Status	Utilization
Pressure	1.27	1.50	✓ Pass	0.844
Tension	-0.12	0.00	✓ Pass	

Mat: MF 1, Panel: SI 4

Static Design Summary

Reinforcement Design Summary

Reinforcement Layer	Analysis	Combination	Reinforcement Provided	Area Provided [in ² /ft]	Area Required [in ² /ft]	Utilization
Bottom-X	3D Building Analysis	27 LRFD _{8.6} -1.2D+1.6S+0.5W	#6-1' 0"-B1	0.44	0.04	0.088
Bottom-Y	FE Chase Down	9 LRFD ₅ -1.2D+L+1.6Lr	#6-1' 0"-B2	0.44	0.06	0.127
Top-X	3D Building Analysis	27 LRFD _{8.6} -1.2D+1.6S+0.5W	#6-1' 0"-T1	0.44	0.04	0.087
Top-Y	FE Chase Down	9 LRFD ₅ -1.2D+L+1.6Lr	#6-1' 0"-T2	0.44	0.08	0.173

Bearing Pressure Summary

Bearing pressure checks	Max. value [ksf]	Limit [ksf]	Status	Utilization
Pressure	0.95	1.50	✓ Pass	0.635

Mat: MF 1, Panel: SI 5

Static Design Summary

Reinforcement Design Summary

Reinforcement Layer	Analysis	Combination	Reinforcement Provided	Area Provided [in ² /ft]	Area Required [in ² /ft]	Utilization
Bottom-X	FE Chase Down	9 LRFD ₅ -1.2D+L+1.6Lr	#6-1' 0"-B1	0.44	0.05	0.102
Bottom-Y	FE Chase Down	9 LRFD ₅ -1.2D+L+1.6Lr	#6-1' 0"-B2	0.44	0.07	0.152
Top-X	3D Building Analysis	83 LRFD _{10.22} -1.2D+L+0.5S+W	#6-1' 0"-T1	0.44	0.01	0.021
Top-Y	3D Building Analysis	106 LRFD _{12.5} -0.9D+W	#6-1' 0"-T2	0.44	0.00	0.011

Bearing Pressure Summary

Bearing pressure checks	Max. value [ksf]	Limit [ksf]	Status	Utilization
Pressure	1.12	1.50	✓ Pass	0.743



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 53
Project Name: JKV-EMS		

Mat: MF 1, Panel: SI 6

Static Design Summary

Reinforcement Design Summary

Reinforcement Layer	Analysis	Combination	Reinforcement Provided	Area Provided [in ² /ft]	Area Required [in ² /ft]	Utilization
Bottom-X	3D Building Analysis	17 LRFD _{7.4} -1.2D+1.6Lr+0.5W	#6-1' 0"-B1	0.44	0.04	0.085
Bottom-Y	3D Building Analysis	17 LRFD _{7.4} -1.2D+1.6Lr+0.5W	#6-1' 0"-B2	0.44	0.06	0.133
Top-X	3D Building Analysis	56 LRFD _{9.3} -1.2D+L+0.5Lr+W	#6-1' 0"-T1	0.44	0.01	0.014
Top-Y	3D Building Analysis	21 LRFD _{7.8} -1.2D+1.6Lr+0.5W	#6-1' 0"-T2	0.44	0.03	0.068

Bearing Pressure Summary

Bearing pressure checks	Max. value [ksf]	Limit [ksf]	Status	Utilization
Pressure	1.01	1.50	✓ Pass	0.671

Mat: MF 1, Panel: SI 7

Static Design Summary

Reinforcement Design Summary

Reinforcement Layer	Analysis	Combination	Reinforcement Provided	Area Provided [in ² /ft]	Area Required [in ² /ft]	Utilization
Bottom-X	FE Chase Down	9 LRFD ₅ -1.2D+L+1.6Lr	#6-1' 0"-B1	0.44	0.04	0.099
Bottom-Y	FE Chase Down	9 LRFD ₅ -1.2D+L+1.6Lr	#6-1' 0"-B2	0.44	0.07	0.156
Top-X	3D Building Analysis	17 LRFD _{7.4} -1.2D+1.6Lr+0.5W	#6-1' 0"-T1	0.44	0.01	0.015
Top-Y	FE Chase Down	9 LRFD ₅ -1.2D+L+1.6Lr	#6-1' 0"-T2	0.44	0.00	0.011

Bearing Pressure Summary

Bearing pressure checks	Max. value [ksf]	Limit [ksf]	Status	Utilization
Pressure	1.12	1.50	✓ Pass	0.747

Mat: MF 1, Panel: SI 8

Static Design Summary

Reinforcement Design Summary



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Revision:	Date:	Sheet No. 54
Project Name: JKV-EMS		

Reinforcement Layer	Analysis	Combination	Reinforcement Provided	Area Provided [in ² /ft]	Area Required [in ² /ft]	Utilization
Bottom-X	3D Building Analysis	25 LRFD _{8.4} -1.2D+1.6S+0.5W	#6-1' 0"-B1	0.44	0.05	0.108
Bottom-Y	FE Chase Down	9 LRFD ₅ -1.2D+L+1.6Lr	#6-1' 0"-B2	0.44	0.05	0.108
Top-X	FE Chase Down	9 LRFD ₅ -1.2D+L+1.6Lr	#6-1' 0"-T1	0.44	0.04	0.085
Top-Y	3D Building Analysis	21 LRFD _{7.8} -1.2D+1.6Lr+0.5W	#6-1' 0"-T2	0.44	0.08	0.189

Bearing Pressure Summary

Bearing pressure checks	Max. value [ksf]	Limit [ksf]	Status	Utilization
Pressure	0.97	1.50	✓ Pass	0.646

Mat: MF 1, Panel: SI 9

Static Design Summary

Reinforcement Design Summary

Reinforcement Layer	Analysis	Combination	Reinforcement Provided	Area Provided [in ² /ft]	Area Required [in ² /ft]	Utilization
Bottom-X	3D Building Analysis	25 LRFD _{8.4} -1.2D+1.6S+0.5W	#6-1' 0"-B1	0.44	0.03	0.072
Bottom-Y	3D Building Analysis	125 LRFD _{11.8} -1.2D+L+0.2S+E	#6-1' 0"-B2	0.44	0.02	0.051
Top-X	3D Building Analysis	124 LRFD _{11.7} -1.2D+L+0.2S+E	#6-1' 0"-T1	0.44	0.04	0.098
Top-Y	3D Building Analysis	19 LRFD _{7.6} -1.2D+1.6Lr+0.5W	#6-1' 0"-T2	0.44	0.07	0.154

Bearing Pressure Summary

Bearing pressure checks	Max. value [ksf]	Limit [ksf]	Status	Utilization
Pressure	1.12	1.50	✓ Pass	0.745

Mat: MF 1, Panel: SI 10

Static Design Summary

Reinforcement Design Summary

Reinforcement Layer	Analysis	Combination	Reinforcement Provided	Area Provided [in ² /ft]	Area Required [in ² /ft]	Utilization
Bottom-X	3D Building Analysis	125 LRFD _{11.8} -1.2D+L+0.2S+E	#6-1' 0"-B1	0.44	0.05	0.115
Bottom-Y	3D Building Analysis	125 LRFD _{11.8} -1.2D+L+0.2S+E	#6-1' 0"-B2	0.44	0.05	0.109



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Revision:	Date:	Sheet No. 55
Project Name: JKV-EMS		

Reinforcement Layer	Analysis	Combination	Reinforcement Provided	Area Provided [in ² /ft]	Area Required [in ² /ft]	Utilization
Top-X	3D Building Analysis	124 LRFD _{11.7} -1.2D+L+0.2S+E	#6-1' 0"-T1	0.44	0.04	0.098
Top-Y	3D Building Analysis	125 LRFD _{11.8} -1.2D+L+0.2S+E	#6-1' 0"-T2	0.44	0.08	0.174

Bearing Pressure Summary

Bearing pressure checks	Max. value [ksf]	Limit [ksf]	Status	Utilization
Pressure	1.30	1.50	✓ Pass	0.866

Mat: MF 1, Panel: SI 11

Static Design Summary

Reinforcement Design Summary

Reinforcement Layer	Analysis	Combination	Reinforcement Provided	Area Provided [in ² /ft]	Area Required [in ² /ft]	Utilization
Bottom-X	3D Building Analysis	27 LRFD _{8.6} -1.2D+1.6S+0.5W	#6-1' 0"-B1	0.44	0.03	0.074
Bottom-Y	3D Building Analysis	125 LRFD _{11.8} -1.2D+L+0.2S+E	#6-1' 0"-B2	0.44	0.02	0.052
Top-X	3D Building Analysis	125 LRFD _{11.8} -1.2D+L+0.2S+E	#6-1' 0"-T1	0.44	0.04	0.098
Top-Y	3D Building Analysis	125 LRFD _{11.8} -1.2D+L+0.2S+E	#6-1' 0"-T2	0.44	0.08	0.174

Bearing Pressure Summary

Bearing pressure checks	Max. value [ksf]	Limit [ksf]	Status	Utilization
Pressure	1.28	1.50	✓ Pass	0.856

Mat: MF 1, Panel: SI 12

Static Design Summary

Reinforcement Design Summary

Reinforcement Layer	Analysis	Combination	Reinforcement Provided	Area Provided [in ² /ft]	Area Required [in ² /ft]	Utilization
Bottom-X	3D Building Analysis	25 LRFD _{8.4} -1.2D+1.6S+0.5W	#6-1' 0"-B1	0.44	0.04	0.090
Bottom-Y	3D Building Analysis	55 LRFD _{9.2} -1.2D+L+0.5Lr+W	#6-1' 0"-B2	0.44	0.04	0.099
Top-X	3D Building Analysis	9 LRFD ₅ -1.2D+L+1.6Lr	#6-1' 0"-T1	0.44	0.03	0.067



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Project Name: JKV-EMS		

Reinforcement Layer	Analysis	Combination	Reinforcement Provided	Area Provided [in ² /ft]	Area Required [in ² /ft]	Utilization
Top-Y	3D Building Analysis	19 LRFD _{7.6} -1.2D+1.6Lr+0.5W	#6-1' 0"-T2	0.44	0.08	0.183

Bearing Pressure Summary

Bearing pressure checks	Max. value [ksf]	Limit [ksf]	Status	Utilization
Pressure	0.85	1.50	✓ Pass	0.566

Mat: MF 1, Panel: SI 13

Static Design Summary

Reinforcement Design Summary

Reinforcement Layer	Analysis	Combination	Reinforcement Provided	Area Provided [in ² /ft]	Area Required [in ² /ft]	Utilization
Bottom-X	3D Building Analysis	9 LRFD ₅ -1.2D+L+1.6Lr	#6-1' 0"-B1	0.44	0.03	0.074
Bottom-Y	3D Building Analysis	9 LRFD ₅ -1.2D+L+1.6Lr	#6-1' 0"-B2	0.44	0.06	0.136
Top-X	3D Building Analysis	17 LRFD _{7.4} -1.2D+1.6Lr+0.5W	#6-1' 0"-T1	0.44	0.01	0.012
Top-Y	3D Building Analysis	19 LRFD _{7.6} -1.2D+1.6Lr+0.5W	#6-1' 0"-T2	0.44	0.00	0.009

Bearing Pressure Summary

Bearing pressure checks	Max. value [ksf]	Limit [ksf]	Status	Utilization
Pressure	0.83	1.50	✓ Pass	0.554

Mat: MF 1, Panel: SI 14

Static Design Summary

Reinforcement Design Summary

Reinforcement Layer	Analysis	Combination	Reinforcement Provided	Area Provided [in ² /ft]	Area Required [in ² /ft]	Utilization
Bottom-X	3D Building Analysis	57 LRFD _{9.4} -1.2D+L+0.5Lr+W	#6-1' 0"-B1	0.44	0.04	0.085
Bottom-Y	3D Building Analysis	17 LRFD _{7.4} -1.2D+1.6Lr+0.5W	#6-1' 0"-B2	0.44	0.06	0.130
Top-X	3D Building Analysis	17 LRFD _{7.4} -1.2D+1.6Lr+0.5W	#6-1' 0"-T1	0.44	0.11	0.249
Top-Y	3D Building Analysis	17 LRFD _{7.4} -1.2D+1.6Lr+0.5W	#6-1' 0"-T2	0.44	0.22	0.499



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Project Name: JKV-EMS		

Bearing Pressure Summary

Bearing pressure checks	Max. value [ksf]	Limit [ksf]	Status	Utilization
Pressure	0.72	1.50	✓ Pass	0.483
Tension	-0.01	0.00	✓ Pass	

Steel

Beams

Non-Composite

Rolled

TOS HBB29

{\H0.5x;\A1;\A2;\A3;\A4;\A5;\A6;\A7;\A8;\A9;\A10;\A11;\A12;\A13;\A14;\A15;\A16;\A17;\A18;\A19;\A20;\A21;\A22;\A23;\A24;\A25;\A26;\A27;\A28;\A29;\A30;\A31;\A32;\A33;\A34;\A35;\A36;\A37;\A38;\A39;\A40;\A41;\A42;\A43;\A44;\A45;\A46;\A47;\A48;\A49;\A50;\A51;\A52;\A53;\A54;\A55;\A56;\A57;\A58;\A59;\A60;\A61;\A62;\A63;\A64;\A65;\A66;\A67;\A68;\A69;\A70;\A71;\A72;\A73;\A74;\A75;\A76;\A77;\A78;\A79;\A80;\A81;\A82;\A83;\A84;\A85;\A86;\A87;\A88;\A89;\A90;\A91;\A92;\A93;\A94;\A95;\A96;\A97;\A98;\A99;\A100;\A101;\A102;\A103;\A104;\A105;\A106;\A107;\A108;\A109;\A110;\A111;\A112;\A113;\A114;\A115;\A116;\A117;\A118;\A119;\A120;\A121;\A122;\A123;\A124;\A125;\A126;\A127;\A128;\A129;\A130;\A131;\A132;\A133;\A134;\A135;\A136;\A137;\A138;\A139;\A140;\A141;\A142;\A143;\A144;\A145;\A146;\A147;\A148;\A149;\A150;\A151;\A152;\A153;\A154;\A155;\A156;\A157;\A158;\A159;\A160;\A161;\A162;\A163;\A164;\A165;\A166;\A167;\A168;\A169;\A170;\A171;\A172;\A173;\A174;\A175;\A176;\A177;\A178;\A179;\A180;\A181;\A182;\A183;\A184;\A185;\A186;\A187;\A188;\A189;\A190;\A191;\A192;\A193;\A194;\A195;\A196;\A197;\A198;\A199;\A200;\A201;\A202;\A203;\A204;\A205;\A206;\A207;\A208;\A209;\A210;\A211;\A212;\A213;\A214;\A215;\A216;\A217;\A218;\A219;\A220;\A221;\A222;\A223;\A224;\A225;\A226;\A227;\A228;\A229;\A230;\A231;\A232;\A233;\A234;\A235;\A236;\A237;\A238;\A239;\A240;\A241;\A242;\A243;\A244;\A245;\A246;\A247;\A248;\A249;\A250;\A251;\A252;\A253;\A254;\A255;\A256;\A257;\A258;\A259;\A260;\A261;\A262;\A263;\A264;\A265;\A266;\A267;\A268;\A269;\A270;\A271;\A272;\A273;\A274;\A275;\A276;\A277;\A278;\A279;\A280;\A281;\A282;\A283;\A284;\A285;\A286;\A287;\A288;\A289;\A290;\A291;\A292;\A293;\A294;\A295;\A296;\A297;\A298;\A299;\A300;\A301;\A302;\A303;\A304;\A305;\A306;\A307;\A308;\A309;\A310;\A311;\A312;\A313;\A314;\A315;\A316;\A317;\A318;\A319;\A320;\A321;\A322;\A323;\A324;\A325;\A326;\A327;\A328;\A329;\A330;\A331;\A332;\A333;\A334;\A335;\A336;\A337;\A338;\A339;\A340;\A341;\A342;\A343;\A344;\A345;\A346;\A347;\A348;\A349;\A350;\A351;\A352;\A353;\A354;\A355;\A356;\A357;\A358;\A359;\A360;\A361;\A362;\A363;\A364;\A365;\A366;\A367;\A368;\A369;\A370;\A371;\A372;\A373;\A374;\A375;\A376;\A377;\A378;\A379;\A380;\A381;\A382;\A383;\A384;\A385;\A386;\A387;\A388;\A389;\A390;\A391;\A392;\A393;\A394;\A395;\A396;\A397;\A398;\A399;\A400;\A401;\A402;\A403;\A404;\A405;\A406;\A407;\A408;\A409;\A410;\A411;\A412;\A413;\A414;\A415;\A416;\A417;\A418;\A419;\A420;\A421;\A422;\A423;\A424;\A425;\A426;\A427;\A428;\A429;\A430;\A431;\A432;\A433;\A434;\A435;\A436;\A437;\A438;\A439;\A440;\A441;\A442;\A443;\A444;\A445;\A446;\A447;\A448;\A449;\A450;\A451;\A452;\A453;\A454;\A455;\A456;\A457;\A458;\A459;\A460;\A461;\A462;\A463;\A464;\A465;\A466;\A467;\A468;\A469;\A470;\A471;\A472;\A473;\A474;\A475;\A476;\A477;\A478;\A479;\A480;\A481;\A482;\A483;\A484;\A485;\A486;\A487;\A488;\A489;\A490;\A491;\A492;\A493;\A494;\A495;\A496;\A497;\A498;\A499;\A500;\A501;\A502;\A503;\A504;\A505;\A506;\A507;\A508;\A509;\A510;\A511;\A512;\A513;\A514;\A515;\A516;\A517;\A518;\A519;\A520;\A521;\A522;\A523;\A524;\A525;\A526;\A527;\A528;\A529;\A530;\A531;\A532;\A533;\A534;\A535;\A536;\A537;\A538;\A539;\A540;\A541;\A542;\A543;\A544;\A545;\A546;\A547;\A548;\A549;\A550;\A551;\A552;\A553;\A554;\A555;\A556;\A557;\A558;\A559;\A560;\A561;\A562;\A563;\A564;\A565;\A566;\A567;\A568;\A569;\A570;\A571;\A572;\A573;\A574;\A575;\A576;\A577;\A578;\A579;\A580;\A581;\A582;\A583;\A584;\A585;\A586;\A587;\A588;\A589;\A590;\A591;\A592;\A593;\A594;\A595;\A596;\A597;\A598;\A599;\A600;\A601;\A602;\A603;\A604;\A605;\A606;\A607;\A608;\A609;\A610;\A611;\A612;\A613;\A614;\A615;\A616;\A617;\A618;\A619;\A620;\A621;\A622;\A623;\A624;\A625;\A626;\A627;\A628;\A629;\A630;\A631;\A632;\A633;\A634;\A635;\A636;\A637;\A638;\A639;\A640;\A641;\A642;\A643;\A644;\A645;\A646;\A647;\A648;\A649;\A650;\A651;\A652;\A653;\A654;\A655;\A656;\A657;\A658;\A659;\A660;\A661;\A662;\A663;\A664;\A665;\A666;\A667;\A668;\A669;\A670;\A671;\A672;\A673;\A674;\A675;\A676;\A677;\A678;\A679;\A680;\A681;\A682;\A683;\A684;\A685;\A686;\A687;\A688;\A689;\A690;\A691;\A692;\A693;\A694;\A695;\A696;\A697;\A698;\A699;\A700;\A701;\A702;\A703;\A704;\A705;\A706;\A707;\A708;\A709;\A710;\A711;\A712;\A713;\A714;\A715;\A716;\A717;\A718;\A719;\A720;\A721;\A722;\A723;\A724;\A725;\A726;\A727;\A728;\A729;\A730;\A731;\A732;\A733;\A734;\A735;\A736;\A737;\A738;\A739;\A740;\A741;\A742;\A743;\A744;\A745;\A746;\A747;\A748;\A749;\A750;\A751;\A752;\A753;\A754;\A755;\A756;\A757;\A758;\A759;\A760;\A761;\A762;\A763;\A764;\A765;\A766;\A767;\A768;\A769;\A770;\A771;\A772;\A773;\A774;\A775;\A776;\A777;\A778;\A779;\A780;\A781;\A782;\A783;\A784;\A785;\A786;\A787;\A788;\A789;\A790;\A791;\A792;\A793;\A794;\A795;\A796;\A797;\A798;\A799;\A800;\A801;\A802;\A803;\A804;\A805;\A806;\A807;\A808;\A809;\A810;\A811;\A812;\A813;\A814;\A815;\A816;\A817;\A818;\A819;\A820;\A821;\A822;\A823;\A824;\A825;\A826;\A827;\A828;\A829;\A830;\A831;\A832;\A833;\A834;\A835;\A836;\A837;\A838;\A839;\A840;\A841;\A842;\A843;\A844;\A845;\A846;\A847;\A848;\A849;\A850;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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 58
Project Name: JKV-EMS		

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
sub-beam	5' 0"	•	1.000		1.000		1.000		1.000
support	6' 7 5/8"	•		•		•		•	

Static Design Summary

Summary HSS 5x2-1/2x1/4

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	2	Non-slender	-	-	-	✓ Pass
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	123	-0.3	49.8	kip	0.007	✓ Pass
Shear Minor	No Significant Forces			kip	-	Not required
Flexure Major	123	-0.6	16.7	kip ft	0.036	✓ Pass
Flexure Minor	10	-0.8	10.2	kip ft	0.075	✓ Pass
Axial Tension	133	-0.2	130.0	kip	0.001	✓ Pass
Axial Compression	122	0.4	102.0	kip	0.004	✓ Pass
Combined Forces	29	-	-	-	0.093	✓ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Dead	118	0.003	0.109	in	0.031	✓ Pass
Deflection Live	9	0.002	0.109	in	0.021	✓ Pass
Deflection Wind	109	0.003	0.109	in	0.029	✓ Pass
Deflection Total	123	0.010	0.164	in	0.060	✓ Pass

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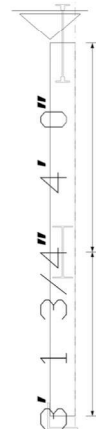
Date:

Sheet No. 59

Project Name: JKV-EMS

{\H0.5x;\A210407}\A1;

16' 10"



St. TOS HB (TOS High Bearing):
HSS 5x2-1/2x1-1/4 A500B-46

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	4' 0"	•	1.000		1.000		1.000		1.000
support	4' 0"	•		•		•		•	
sub-beam	3' 1 3/4"	•	1.000		1.000		1.000		1.000
support	7' 1 3/4"	•		•		•		•	

Static Design Summary

Summary HSS 5x2-1/2x1-1/4

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	2	Non-slender	-	-	-	✓ Pass
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	120	-0.2	49.8	kip	0.003	✓ Pass
Shear Minor	10	0.7	20.8	kip	0.035	✓ Pass
Flexure Major	120	-0.7	16.7	kip ft	0.039	✓ Pass
Flexure Minor	10	-2.2	10.2	kip ft	0.220	✓ Pass
Axial Tension	No Significant Forces			kip	-	Not required
Axial Compression	10	0.3	118.1	kip	0.002	✓ Pass



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Revision:	Date:	Sheet No. 60
Project Name: JKV-EMS		

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Combined Forces	10	-	-	-	0.234	✓ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Dead	118	0.004	0.210	in	0.021	✓ Pass
Deflection Live	9	0.003	0.210	in	0.014	✓ Pass
Deflection Wind	109	0.005	0.210	in	0.024	✓ Pass
Deflection Total	120	0.018	0.315	in	0.056	✓ Pass

TOS HBB49

{\H0.5x;\G411\A1;

16' 10"

4' 0"
4' 0"
7' 5 7/8"

St. TOS HB (TOS High Bearing):
HSS 5x2 1/2x1/4 A500B-46

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	4' 0"	•	1.000		1.000		1.000		1.000
support	4' 0"	•		•		•		•	
sub-beam	2' 0"	•	1.000		1.000		1.000		1.000
support	6' 0"	•		•		•		•	
sub-beam	1' 7 5/8"	•	1.000		1.000		1.000		1.000



Job No. 25-003

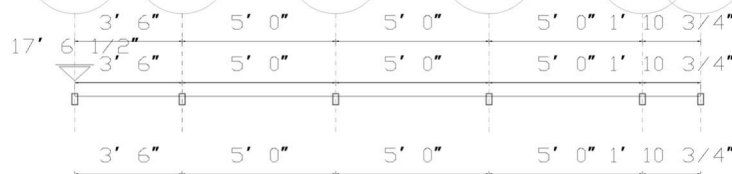
Sheet No. 62

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	33' 6"	●		●		●		●	
sub-beam	5' 0"		1.000		1.000		1.000		1.000
support	38' 6"	●		●		●		●	
sub-beam	5' 0"		1.000		1.000		1.000		1.000
support	43' 6"	●		●		●		●	
sub-beam	5' 0"		1.000		1.000		1.000		1.000
support	48' 6"	●		●		●		●	
sub-beam	5' 0"		1.000		1.000		1.000		1.000
support	53' 6"	●		●		●		●	
sub-beam	2' 4 3/4"		1.000		1.000		1.000		1.000
support	55' 10 3/4"	●		●		●		●	

Summary L 6x6x3/8

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	2	Slender	-	-	-	✓ Pass
Flexural Classification	2	Non-compact	-	-	-	✓ Pass
Shear Major	122	-0.7	43.7	kip	0.016	✓ Pass
Shear Minor	10	-0.4	43.7	kip	0.008	✓ Pass
Flexure Major	122	1.8	23.6	kip ft	0.056	✓ Pass
Flexure Minor	122	0.3	11.0	kip ft	0.122	✓ Pass
Axial Tension	133	-5.1	141.9	kip	0.036	✓ Pass
Axial Compression	122	8.4	113.2	kip	0.074	✓ Pass
Combined Forces	122	-	-	-	0.155	✓ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Self weight	118	0.001	-	in	-	-
Deflection Dead	118	-0.001	0.117	in	0.010	✓ Pass
Deflection Live	14	-0.001	0.117	in	0.007	✓ Pass
Deflection Wind	104	0.001	0.210	in	0.003	✓ Pass
Deflection Total	123	-0.003	0.175	in	0.018	✓ Pass

{\H0.5\;NA.5;1034210586\A8015386\A015392A2A10390}\A1;



St. IOS SUB KONSPEKT HIGH TMS HIGH TMS HIGH TMS HIGH TMS HIGH TMS HIGH TMS
L 6x6x3/86 A36-36 A36-36 A36-36 A36-36 A36-36 A36-36

Restraints



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Project Name: JKV-EMS		

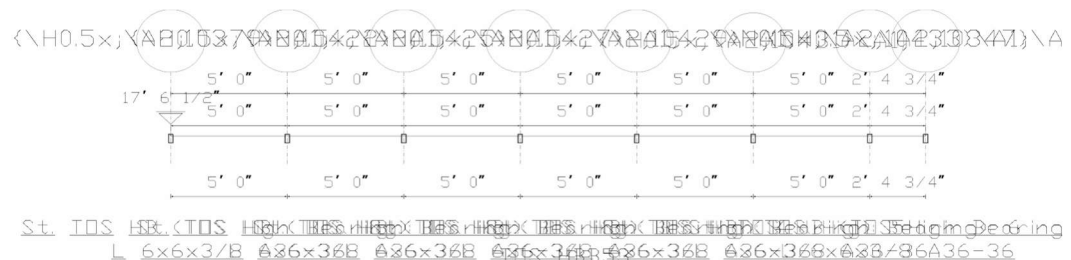
Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	3' 6"		1.000		1.000		1.000		1.000
support	3' 6"	•		•		•		•	
sub-beam	5' 0"		1.000		1.000		1.000		1.000
support	8' 6"	•		•		•		•	
sub-beam	5' 0"		1.000		1.000		1.000		1.000
support	13' 6"	•		•		•		•	
sub-beam	5' 0"		1.000		1.000		1.000		1.000
support	18' 6"	•		•		•		•	
sub-beam	1' 10 3/4"		1.000		1.000		1.000		1.000
support	20' 4 3/4"	•		•		•		•	

Static Design Summary

Summary L 6x6x3/8

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	2	Unknown	-	-	-	Not required
Flexural Classification	2	Non-compact	-	-	-	✓ Pass
Shear Major	10	-0.9	43.7	kip	0.020	✓ Pass
Shear Minor	10	-0.3	43.7	kip	0.007	✓ Pass
Flexure Major	10	4.3	22.6	kip ft	0.210	✓ Pass
Flexure Minor	10	1.9	11.0	kip ft	0.127	✓ Pass
Axial Tension	29	-0.7	141.9	kip	0.005	✓ Pass
Axial Compression	No Significant Forces			kip	-	Not required
Combined Forces	10	-	-	-	0.336	✓ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Self weight	118	0.006	-	in	-	-
Deflection Dead	118	0.019	0.167	in	0.117	✓ Pass
Deflection Live	9	0.013	0.167	in	0.077	✓ Pass
Deflection Wind	108	-0.009	0.300	in	0.032	✓ Pass
Deflection Total	10	0.039	0.250	in	0.158	✓ Pass

TOS HBB53



Restraints



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Revision:	Date:	Sheet No. 64
Project Name: JKV-EMS		

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	5' 0"		1.000		1.000		1.000		1.000
support	5' 0"	•		•		•		•	
sub-beam	5' 0"		1.000		1.000		1.000		1.000
support	10' 0"	•		•		•		•	
sub-beam	5' 0"		1.000		1.000		1.000		1.000
support	15' 0"	•		•		•		•	
sub-beam	5' 0"		1.000		1.000		1.000		1.000
support	20' 0"	•		•		•		•	
sub-beam	5' 0"		1.000		1.000		1.000		1.000
support	25' 0"	•		•		•		•	
sub-beam	5' 0"		1.000		1.000		1.000		1.000
support	30' 0"	•		•		•		•	
sub-beam	2' 4 3/4"		1.000		1.000		1.000		1.000
support	32' 4 3/4"	•		•		•		•	

Static Design Summary

Summary L 6x6x3/8

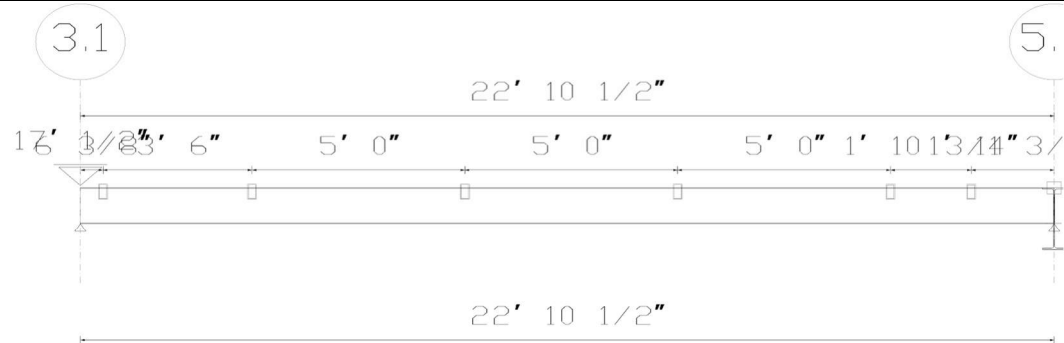
Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	2	Slender	-	-	-	✓ Pass
Flexural Classification	2	Non-compact	-	-	-	✓ Pass
Shear Major	10	1.6	43.7	kip	0.036	✓ Pass
Shear Minor	23	0.7	43.7	kip	0.015	✓ Pass
Flexure Major	10	-5.5	23.6	kip ft	0.225	✓ Pass
Flexure Minor	10	-2.0	11.0	kip ft	0.221	✓ Pass
Axial Tension	102	-0.7	141.9	kip	0.005	✓ Pass
Axial Compression	29	0.8	113.2	kip	0.007	✓ Pass
Combined Forces	10	-	-	-	0.439	✓ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Self weight	118	0.006	-	in	-	-
Deflection Dead	118	0.016	0.167	in	0.093	✓ Pass
Deflection Live	14	0.010	0.167	in	0.061	✓ Pass
Deflection Wind	108	-0.008	0.300	in	0.027	✓ Pass
Deflection Total	10	0.033	0.250	in	0.131	✓ Pass

TOS HBB54



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Project Name: JKV-EMS		



St. TOS HB (TOS High Bearing): TOS HBB54 =
W 12x14 A992-50

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	6 3/8"	•	1.000		1.000		1.000		1.000
member	6 3/8"	•						•	
sub-beam	3' 6"	•	1.000		1.000		1.000		1.000
member	4' 3/8"	•						•	
sub-beam	5' 0"	•	1.000		1.000		1.000		1.000
member	9' 3/8"	•						•	
sub-beam	5' 0"	•	1.000		1.000		1.000		1.000
member	14' 3/8"	•						•	
sub-beam	5' 0"	•	1.000		1.000		1.000		1.000
member	19' 3/8"	•						•	
sub-beam	1' 10 3/4"	•	1.000		1.000		1.000		1.000
member	20' 11 1/8"	•						•	
sub-beam	1' 11 3/8"	•	1.000		1.000		1.000		1.000
support	22' 10 1/2"	•		•		•		•	

Static Design Summary

Summary W 12x14

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	2	Slender	-	-	-	✓ Pass
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	10	5.7	64.3	kip	0.089	✓ Pass
Shear Minor	123	-0.4	48.2	kip	0.008	✓ Pass
Flexure Major	10	25.1	65.2	kip ft	0.385	✓ Pass
Flexure Minor	123	-0.7	7.1	kip ft	0.098	✓ Pass
Axial Tension	69	-2.9	187.2	kip	0.016	✓ Pass
Axial Compression	63	1.8	39.4	kip	0.046	✓ Pass
Combined Forces	10	-	-	-	0.466	✓ Pass
Torsion	No Significant Forces			-	-	Not required

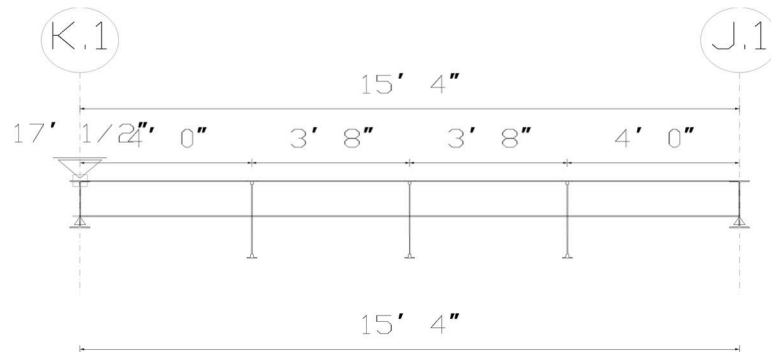


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Revision:	Date:	Sheet No. 66
Project Name: JKV-EMS		

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Deflection Self weight	118	0.093	-	in	-	-
Deflection Dead	118	0.361	0.763	in	0.474	✓ Pass
Deflection Live	9	0.237	0.763	in	0.311	✓ Pass
Deflection Wind	108	-0.175	1.373	in	0.127	✓ Pass
Deflection Total	10	0.709	1.144	in	0.620	✓ Pass

TOS HBB55



St. TOS HB (TOS High Bearing): TOS HBB55 =
W 12x14 A992-50

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	4' 0"	•	1.000		1.000		1.000		1.000
member	4' 0"	•						•	
sub-beam	3' 8"	•	1.000		1.000		1.000		1.000
member	7' 8"	•						•	
sub-beam	3' 8"	•	1.000		1.000		1.000		1.000
member	11' 4"	•						•	
sub-beam	4' 0"	•	1.000		1.000		1.000		1.000
support	15' 4"	•		•		•		•	

Static Design Summary

Summary W 12x14

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	9	Slender	-	-	-	✓ Pass
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	9	7.9	64.3	kip	0.123	✓ Pass
Shear Minor	No Significant Forces			kip	-	Not required
Flexure Major	9	38.0	65.2	kip ft	0.583	✓ Pass
Flexure Minor	123	-0.2	7.1	kip ft	0.029	✓ Pass
Axial Tension	104	-0.2	187.2	kip	0.001	✓ Pass

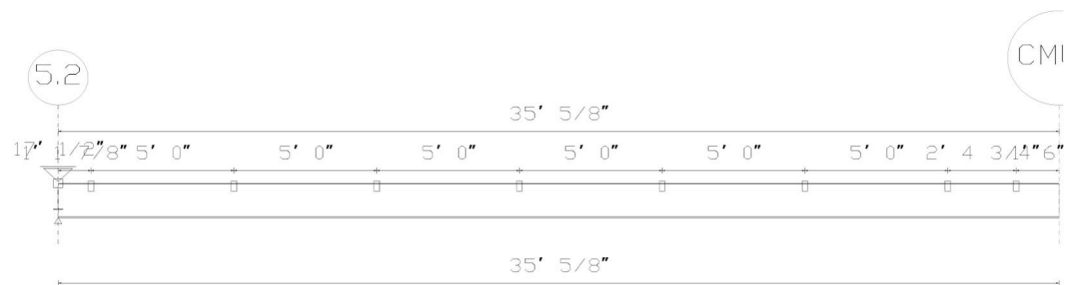


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Revision:	Date:	Sheet No. 67
Project Name: JKV-EMS		

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Compression	124	0.3	52.9	kip	0.006	✓ Pass
Combined Forces	9	-	-	-	0.600	✓ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Self weight	118	0.020	-	in	-	-
Deflection Dead	118	0.259	0.511	in	0.506	✓ Pass
Deflection Live	9	0.175	0.511	in	0.342	✓ Pass
Deflection Wind	108	-0.109	0.920	in	0.118	✓ Pass
Deflection Total	9	0.450	0.767	in	0.587	✓ Pass

TOS HBB56



St. TOS HB (TOS High Bearing): TOS HBB56 = 1
W 16x26 A992-50

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	1' 1 7/8"	•	1.000		1.000		1.000		1.000
member	1' 1 7/8"	•						•	
sub-beam	5' 0"	•	1.000		1.000		1.000		1.000
member	6' 1 7/8"	•						•	
sub-beam	5' 0"	•	1.000		1.000		1.000		1.000
member	11' 1 7/8"	•						•	
sub-beam	5' 0"	•	1.000		1.000		1.000		1.000
member	16' 1 7/8"	•						•	
sub-beam	5' 0"	•	1.000		1.000		1.000		1.000
member	21' 1 7/8"	•						•	
sub-beam	5' 0"	•	1.000		1.000		1.000		1.000
member	26' 1 7/8"	•						•	
sub-beam	5' 0"	•	1.000		1.000		1.000		1.000
member	31' 1 7/8"	•						•	
sub-beam	2' 4 3/4"	•	1.000		1.000		1.000		1.000
member	33' 6 5/8"	•						•	
sub-beam	1' 6"	•	1.000		1.000		1.000		1.000
support	35' 5/8"	•		•		•		•	

Static Design Summary



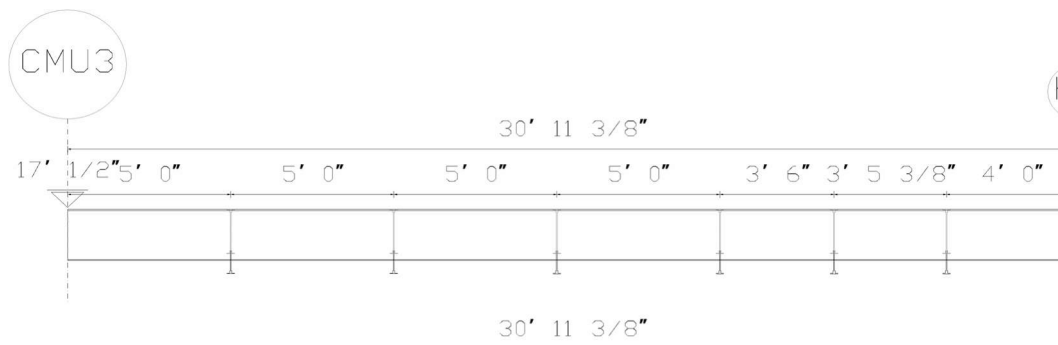
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Project Name: JKV-EMS		

Summary W 16x26

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	2	Slender	-	-	-	✓ Pass
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	10	8.2	106.0	kip	0.077	✓ Pass
Shear Minor	29	-0.8	102.5	kip	0.008	✓ Pass
Flexure Major	25	-13.6	23.4	kip ft	0.580	✓ Pass
Flexure Minor	29	1.2	20.5	kip ft	0.056	✓ Pass
Axial Tension	69	-0.7	345.6	kip	0.002	✓ Pass
Axial Compression	106	0.6	74.9	kip	0.008	✓ Pass
Combined Forces	29	-	-	-	0.630	✓ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Self weight	118	0.129	-	in	-	-
Deflection Slab	118	0.002	1.753	in	0.001	✓ Pass
Deflection Dead	118	0.444	1.168	in	0.380	✓ Pass
Deflection Live	9	0.292	1.168	in	0.250	✓ Pass
Deflection Wind	108	-0.239	2.103	in	0.113	✓ Pass
Deflection Total	10	0.888	1.753	in	0.507	✓ Pass

TOS HBB60



St. TOS HB (TOS High Bearing): TOS HBB60 = 1
W 21x44 A992-50

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	5' 0"	•	1.000		1.000		1.000		1.000
member	5' 0"	•						•	
sub-beam	5' 0"	•	1.000		1.000		1.000		1.000
member	10' 0"	•						•	
sub-beam	5' 0"	•	1.000		1.000		1.000		1.000
member	15' 0"	•						•	
sub-beam	5' 0"	•	1.000		1.000		1.000		1.000
member	20' 0"	•						•	



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Revision:	Date:	Sheet No. 69
Project Name: JKV-EMS		

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
sub-beam	3' 6"	•	1.000		1.000		1.000		1.000
member	23' 6"	•						•	
sub-beam	3' 5 3/8"	•	1.000		1.000		1.000		1.000
member	26' 11 3/8"	•						•	
sub-beam	4' 0"	•	1.000		1.000		1.000		1.000
support	30' 11 3/8"	•		•		•		•	

Static Design Summary

Summary W 21x44

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	2	Slender	-	-	-	✓ Pass
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	10	-28.3	217.4	kip	0.130	✓ Pass
Shear Minor	123	0.4	158.0	kip	0.002	✓ Pass
Flexure Major	10	250.3	357.7	kip ft	0.700	✓ Pass
Flexure Minor	123	1.3	38.2	kip ft	0.035	✓ Pass
Axial Tension	130	-1.1	585.0	kip	0.002	✓ Pass
Axial Compression	125	1.2	154.1	kip	0.008	✓ Pass
Combined Forces	10	-	-	-	0.715	✓ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Self weight	118	0.077	-	in	-	-
Deflection Dead	118	0.712	1.032	in	0.690	✓ Pass
Deflection Live	9	0.468	1.032	in	0.453	✓ Pass
Deflection Wind	104	-0.279	1.857	in	0.150	✓ Pass
Deflection Total	10	1.292	1.547	in	0.835	✓ Pass

TOS HBB97



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Job No. 25-003

Revision:

Date:

Sheet No. 70

Project Name: JKV-EMS

{\H0.5x;\CMU3
42450293}\A1;

16' 10"

5' 0"

1' 7 5/8"

1' 7 5/8"

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1' 7 5/8"

1' 7 5/8"

St. TOS HB (TOS High Bearing):
HSS 5x2-1/2x1/4 A500B-46

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	1' 7 5/8"	•	1.000		1.000		1.000		1.000
support	1' 7 5/8"	•		•		•		•	
sub-beam	5' 0"	•	1.000		1.000		1.000		1.000
support	6' 7 5/8"	•		•		•		•	

Static Design Summary

Summary HSS 5x2-1/2x1/4

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	120	Non-slender	-	-	-	✓ Pass
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	122	2.1	49.8	kip	0.041	✓ Pass
Shear Minor	10	0.4	20.8	kip	0.020	✓ Pass
Flexure Major	122	-1.9	16.7	kip ft	0.114	✓ Pass
Flexure Minor	10	-2.0	10.2	kip ft	0.194	✓ Pass
Axial Tension	122	-0.2	130.0	kip	0.002	✓ Pass
Axial Compression	123	0.2	126.7	kip	0.001	✓ Pass

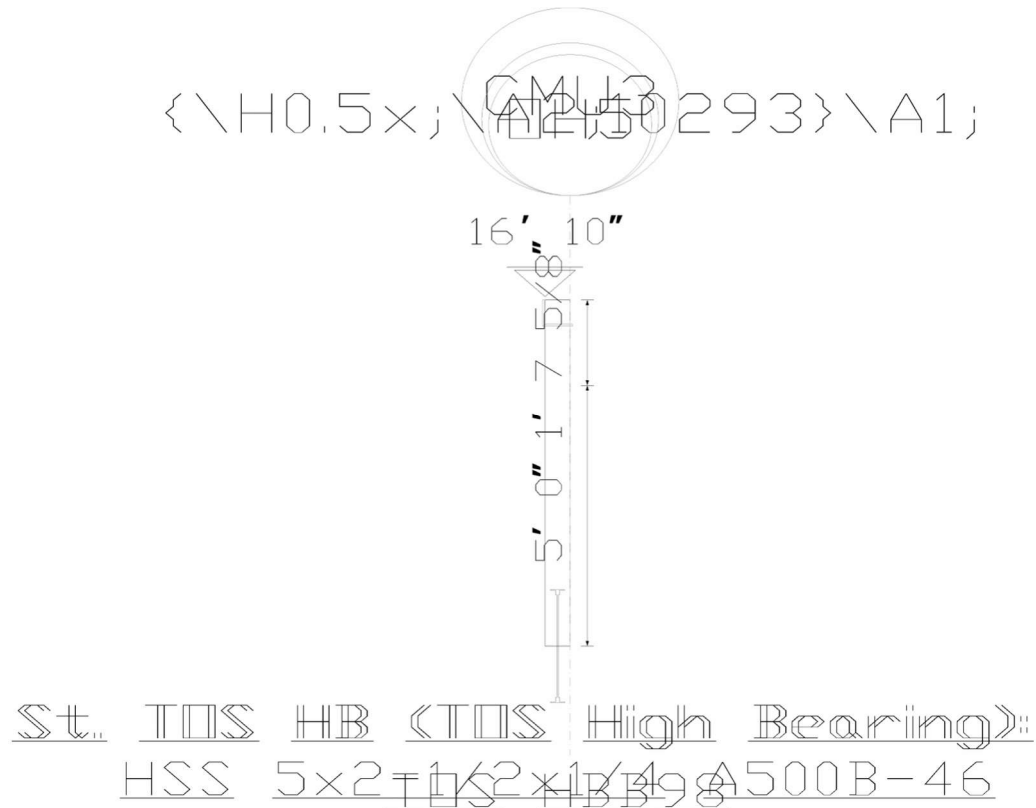


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Revision:	Date:	Sheet No. 71
Project Name: JKV-EMS		

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Combined Forces	10	-	-	-	0.224	✓ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Dead	118	0.003	0.109	in	0.030	✓ Pass
Deflection Live	9	0.002	0.109	in	0.020	✓ Pass
Deflection Wind	109	0.002	0.109	in	0.021	✓ Pass
Deflection Total	123	0.009	0.164	in	0.058	✓ Pass

TOS HBB98



Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	1' 7 5/8"	•	1.000		1.000		1.000		1.000
support	1' 7 5/8"	•		•		•		•	
sub-beam	5' 0"	•	1.000		1.000		1.000		1.000
support	6' 7 5/8"	•		•		•		•	

Static Design Summary

Summary HSS 5x2-1/2x1/4



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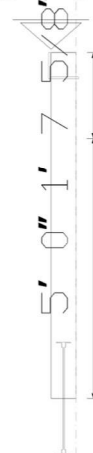
Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 72
Project Name: JKV-EMS		

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	2	Unknown	-	-	-	Not required
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	123	2.4	49.8	kip	0.048	✓ Pass
Shear Minor	10	0.8	20.8	kip	0.039	✓ Pass
Flexure Major	123	-2.3	16.7	kip ft	0.140	✓ Pass
Flexure Minor	10	-3.9	10.2	kip ft	0.382	✓ Pass
Axial Tension	No Significant Forces			kip	-	Not required
Axial Compression	No Significant Forces			kip	-	Not required
Combined Forces	10	-	-	-	0.410	✓ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Dead	118	0.003	0.109	in	0.026	✓ Pass
Deflection Live	9	0.002	0.109	in	0.017	✓ Pass
Deflection Wind	109	0.001	0.109	in	0.013	✓ Pass
Deflection Total	123	0.009	0.164	in	0.053	✓ Pass

TOS HBB99

{\H0.5x;\A1;}

16' 10"



St. TOS HB (TOS High Bearing):
HSS 5x2 1/2x1/4 A500B-46

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	



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Revision:	Date:	Sheet No. 73
Project Name: JKV-EMS		

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
sub-beam	1' 7 5/8"	•	1.000		1.000		1.000		1.000
support	1' 7 5/8"	•		•		•		•	
sub-beam	5' 0"	•	1.000		1.000		1.000		1.000
support	6' 7 5/8"	•		•		•		•	

Static Design Summary

Summary HSS 5x2-1/2x1/4

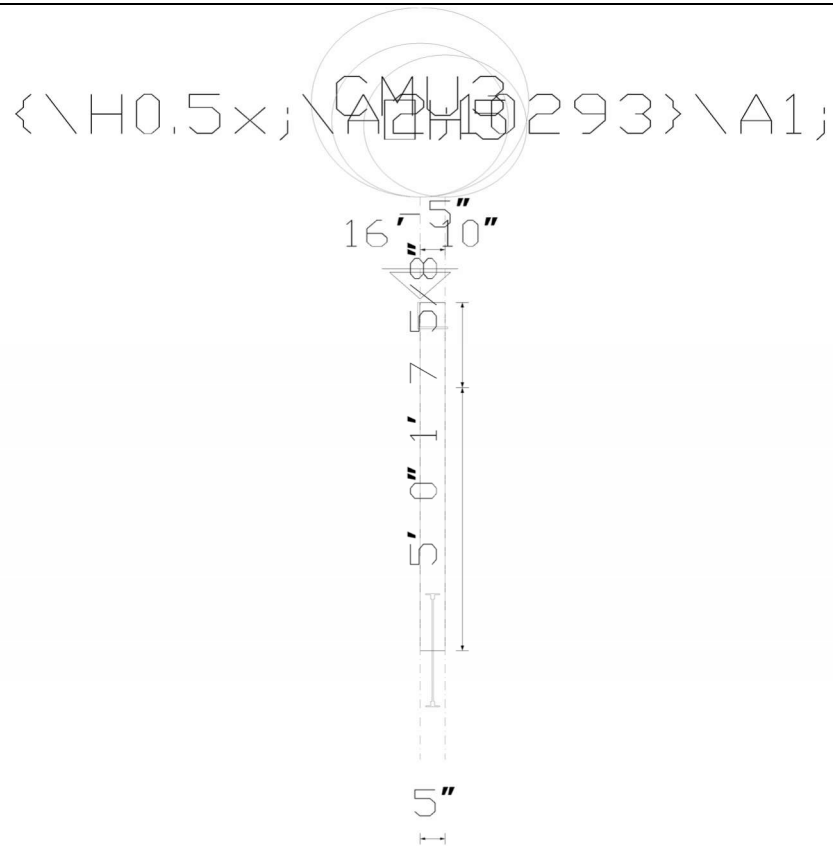
Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	2	Unknown	-	-	-	Not required
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	122	1.1	49.8	kip	0.021	✓ Pass
Shear Minor	10	1.0	20.8	kip	0.047	✓ Pass
Flexure Major	123	-1.9	16.7	kip ft	0.116	✓ Pass
Flexure Minor	10	-4.8	10.2	kip ft	0.470	✓ Pass
Axial Tension	No Significant Forces			kip	-	Not required
Axial Compression	No Significant Forces			kip	-	Not required
Combined Forces	124	-	-	-	0.315	✓ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Dead	118	0.002	0.109	in	0.022	✓ Pass
Deflection Live	9	0.002	0.109	in	0.014	✓ Pass
Deflection Wind	109	0.001	0.109	in	0.007	✓ Pass
Deflection Total	123	0.008	0.164	in	0.050	✓ Pass

TOS HBB100



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Project Name: JKV-EMS		



St. TOS HB (TOS High Bearing)
HSS 5x2-1/2x1/4 A500B-46

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	1' 7 5/8"	•	1.000		1.000		1.000		1.000
support	1' 7 5/8"	•		•		•		•	
sub-beam	5' 0"	•	1.000		1.000		1.000		1.000
support	6' 7 5/8"	•		•		•		•	

Static Design Summary

Summary HSS 5x2-1/2x1/4

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	2	Unknown	-	-	-	Not required
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	123	1.7	49.8	kip	0.034	✓ Pass

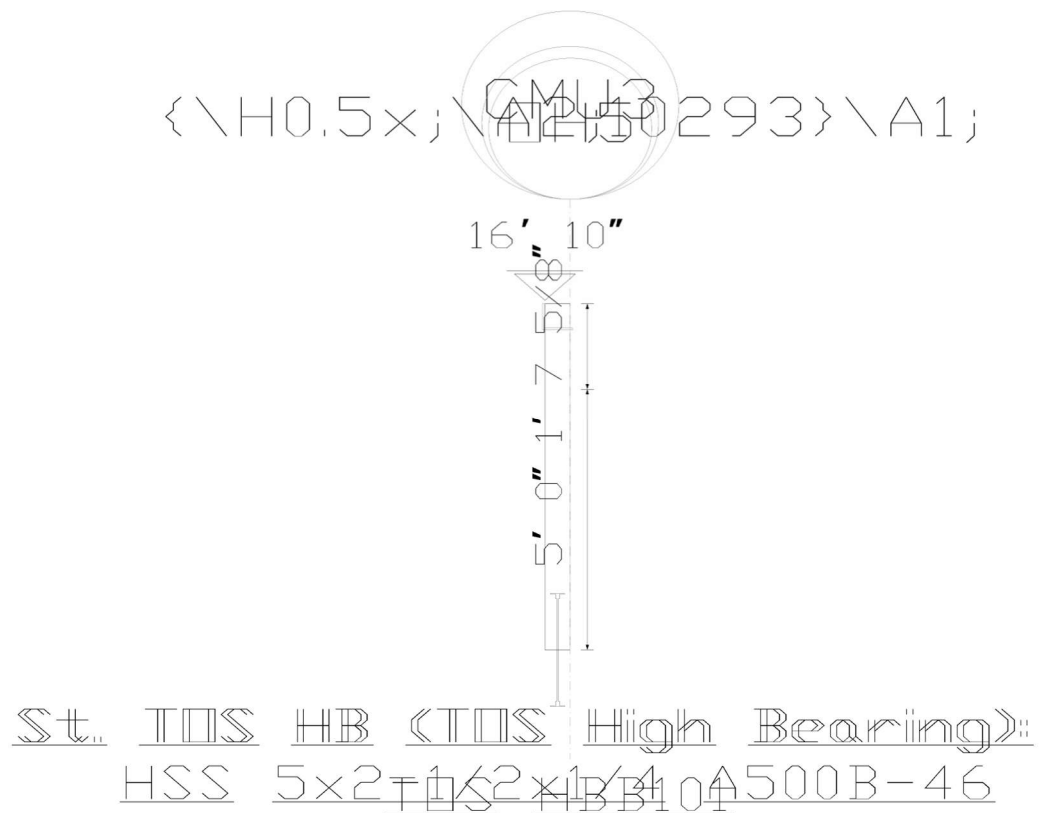


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Revision:	Date:	Sheet No. 75
Project Name: JKV-EMS		

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Shear Minor	10	1.2	20.8	kip	0.058	✓ Pass
Flexure Major	123	-1.7	16.7	kip ft	0.104	✓ Pass
Flexure Minor	10	-5.9	10.2	kip ft	0.578	✓ Pass
Axial Tension	10	-0.3	130.0	kip	0.002	✓ Pass
Axial Compression	No Significant Forces			kip	-	Not required
Combined Forces	10	-	-	-	0.585	✓ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Dead	118	0.001	0.109	in	0.009	✓ Pass
Deflection Live	9	0.001	0.109	in	0.006	✓ Pass
Deflection Wind	107	0.000	0.109	in	0.004	✓ Pass
Deflection Total	124	-0.006	0.164	in	0.036	✓ Pass

TOS HBB101



Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	1' 7 5/8"	•	1.000		1.000		1.000		1.000
support	1' 7 5/8"	•		•		•		•	
sub-beam	5' 0"	•	1.000		1.000		1.000		1.000



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Revision:	Date:	Sheet No. 76
Project Name: JKV-EMS		

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	6' 7 5/8"	•		•		•		•	

Static Design Summary

Summary HSS 5x2-1/2x1/4

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	124	Non-slender	-	-	-	✓ Pass
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	122	0.7	49.8	kip	0.014	✓ Pass
Shear Minor	10	1.1	20.8	kip	0.052	✓ Pass
Flexure Major	123	-1.1	16.7	kip ft	0.066	✓ Pass
Flexure Minor	10	-5.3	10.2	kip ft	0.517	✓ Pass
Axial Tension	130	-0.1	130.0	kip	0.001	✓ Pass
Axial Compression	125	0.2	126.7	kip	0.001	✓ Pass
Combined Forces	124	-	-	-	0.360	✓ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Wind	106	0.001	0.109	in	0.006	✓ Pass
Deflection Total	119	-0.006	0.164	in	0.035	✓ Pass

TOS HBB102

{\H0.5x;\CMU3
4250293}\A1;
16' 10"

5' 0" 1' 7 5/8"

St. TOS HB (TOS High Bearing):
HSS 5x2-1/2x1/4 A500B-46



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Revision:	Date:	Sheet No. 77
Project Name: JKV-EMS		

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	1' 7 5/8"	•	1.000		1.000		1.000		1.000
support	1' 7 5/8"	•		•		•		•	
sub-beam	5' 0"	•	1.000		1.000		1.000		1.000
support	6' 7 5/8"	•		•		•		•	

Static Design Summary

Summary HSS 5x2-1/2x1/4

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	122	Non-slender	-	-	-	✓ Pass
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	124	-1.1	49.8	kip	0.022	✓ Pass
Shear Minor	10	1.0	20.8	kip	0.049	✓ Pass
Flexure Major	124	1.0	16.7	kip ft	0.063	✓ Pass
Flexure Minor	10	-5.0	10.2	kip ft	0.488	✓ Pass
Axial Tension	125	-0.4	130.0	kip	0.003	✓ Pass
Axial Compression	130	0.3	102.0	kip	0.003	✓ Pass
Combined Forces	10	-	-	-	0.502	✓ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Dead	118	-0.002	0.109	in	0.016	✓ Pass
Deflection Live	9	-0.001	0.109	in	0.010	✓ Pass
Deflection Wind	106	0.001	0.109	in	0.007	✓ Pass
Deflection Total	119	-0.006	0.164	in	0.035	✓ Pass

TOS HBB103

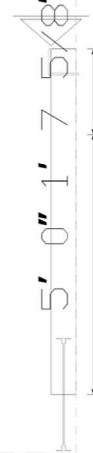


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Revision:	Date:	Sheet No. 78
Project Name: JKV-EMS		

{\H0.5x;\CMU3
42450293}\A1;

16' 10"



St. TOS HB (TOS High Bearing):
HSS 5x2-1/2x1/4 A500B-46

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	1' 7 5/8"	•	1.000		1.000		1.000		1.000
support	1' 7 5/8"	•		•		•		•	
sub-beam	5' 0"	•	1.000		1.000		1.000		1.000
support	6' 7 5/8"	•		•		•		•	

Static Design Summary

Summary HSS 5x2-1/2x1/4

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	2	Non-slender	-	-	-	✓ Pass
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	122	-0.4	49.8	kip	0.008	✓ Pass
Shear Minor	10	0.8	20.8	kip	0.036	✓ Pass
Flexure Major	118	0.5	16.7	kip ft	0.029	✓ Pass
Flexure Minor	10	-3.6	10.2	kip ft	0.358	✓ Pass
Axial Tension	130	-1.2	130.0	kip	0.009	✓ Pass
Axial Compression	125	1.4	102.0	kip	0.013	✓ Pass
Combined Forces	10	-	-	-	0.376	✓ Pass

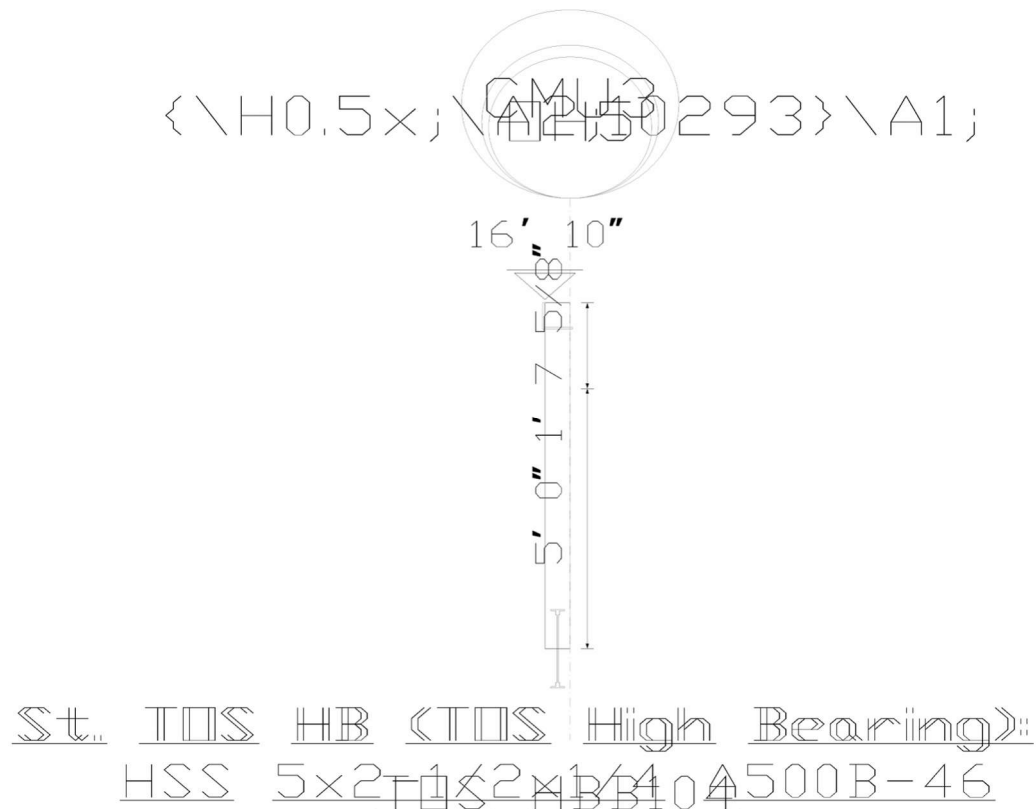


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Revision:	Date:	Sheet No. 79
Project Name: JKV-EMS		

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Torsion		No Significant Forces		-	-	Not required
Deflection Dead	118	-0.002	0.109	in	0.018	✓ Pass
Deflection Live	9	-0.001	0.109	in	0.012	✓ Pass
Deflection Wind	103	-0.001	0.109	in	0.006	✓ Pass
Deflection Total	118	-0.006	0.164	in	0.037	✓ Pass

TOS HBB104



Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	1' 7 5/8"	•	1.000		1.000		1.000		1.000
support	1' 7 5/8"	•		•		•		•	
sub-beam	5' 0"	•	1.000		1.000		1.000		1.000
support	6' 7 5/8"	•		•		•		•	

Static Design Summary

Summary HSS 5x2-1/2x1/4



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Date: 06/16/2025

Job No. 25-003

Revision:

Date:

Sheet No. 80

Project Name: JKV-EMS

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	124	Non-slender	-	-	-	✓ Pass
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	123	-1.1	49.8	kip	0.021	✓ Pass
Shear Minor	10	0.8	20.8	kip	0.040	✓ Pass
Flexure Major	122	1.9	16.7	kip ft	0.116	✓ Pass
Flexure Minor	10	-4.0	10.2	kip ft	0.394	✓ Pass
Axial Tension	130	-0.1	130.0	kip	0.001	✓ Pass
Axial Compression	125	0.1	126.7	kip	0.001	✓ Pass
Combined Forces	29	-	-	-	0.379	✓ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Self weight	118	0.001	-	in	-	-
Deflection Dead	118	-0.002	0.109	in	0.016	✓ Pass
Deflection Live	9	-0.001	0.109	in	0.011	✓ Pass
Deflection Wind	103	-0.001	0.109	in	0.011	✓ Pass
Deflection Total	122	-0.007	0.164	in	0.042	✓ Pass

TOS HBB105

{\H0.5x;\CMU3
4150293}\A1;

16' 10"

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16' 10"

St. TOS HB (TOS High Bearing):
HSS 5x2T1/4x1/4 A500B-46

Restraints



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Revision:	Date:	Sheet No. 81
Project Name: JKV-EMS		

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	1' 7 5/8"	•	1.000		1.000		1.000		1.000
support	1' 7 5/8"	•		•		•		•	
sub-beam	5' 0"	•	1.000		1.000		1.000		1.000
support	6' 7 5/8"	•		•		•		•	

Static Design Summary

Summary HSS 5x2-1/2x1/4

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	2	Unknown	-	-	-	Not required
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	122	-2.7	49.8	kip	0.055	✓ Pass
Shear Minor	10	1.0	20.8	kip	0.050	✓ Pass
Flexure Major	122	2.8	16.7	kip ft	0.166	✓ Pass
Flexure Minor	10	-5.0	10.2	kip ft	0.493	✓ Pass
Axial Tension	10	-0.2	130.0	kip	0.002	✓ Pass
Axial Compression	No Significant Forces			kip	-	Not required
Combined Forces	10	-	-	-	0.512	✓ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Self weight	118	0.001	-	in	-	-
Deflection Dead	118	-0.002	0.109	in	0.020	✓ Pass
Deflection Live	9	-0.001	0.109	in	0.014	✓ Pass
Deflection Wind	103	-0.001	0.109	in	0.013	✓ Pass
Deflection Total	122	-0.009	0.164	in	0.053	✓ Pass

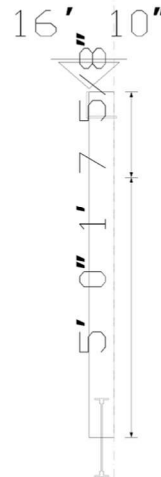
TOS HBB106



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Revision:	Date:	Sheet No. 82
Project Name: JKV-EMS		

{\H0.5x;\CMU3
42450293}\A1;
16' 10"



St. TOS HB (TOS High Bearing):
HSS 5x2-1/2x1/4 A500B-46

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	1' 7 5/8"	•	1.000		1.000		1.000		1.000
support	1' 7 5/8"	•		•		•		•	
sub-beam	5' 0"	•	1.000		1.000		1.000		1.000
support	6' 7 5/8"	•		•		•		•	

Static Design Summary

Summary HSS 5x2-1/2x1/4

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	2	Unknown	-	-	-	Not required
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	123	-1.6	49.8	kip	0.032	✓ Pass
Shear Minor	10	0.9	20.8	kip	0.042	✓ Pass
Flexure Major	122	2.9	16.7	kip ft	0.176	✓ Pass
Flexure Minor	10	-4.3	10.2	kip ft	0.418	✓ Pass
Axial Tension	No Significant Forces			kip	-	Not required
Axial Compression	No Significant Forces			kip	-	Not required
Combined Forces	9	-	-	-	0.406	✓ Pass



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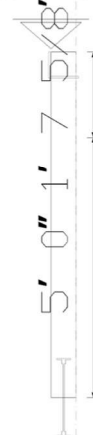
Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 83
Project Name: JKV-EMS		

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Torsion		No Significant Forces		-	-	Not required
Deflection Self weight	118	0.002	-	in	-	-
Deflection Dead	118	-0.003	0.109	in	0.030	✓ Pass
Deflection Live	9	-0.002	0.109	in	0.020	✓ Pass
Deflection Wind	109	-0.002	0.109	in	0.015	✓ Pass
Deflection Total	122	-0.011	0.164	in	0.068	✓ Pass

TOS HBB107

{\H0.5x;\CMU3\A2450293}\A1;

16' 10"



St. TOS HB (TOS High Bearing):

HSS 5x2-1/2x1/4 A500B-46

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	1' 7 5/8"	•	1.000		1.000		1.000		1.000
support	1' 7 5/8"	•		•		•		•	
sub-beam	5' 0"	•	1.000		1.000		1.000		1.000
support	6' 7 5/8"	•		•		•		•	

Static Design Summary

Summary HSS 5x2-1/2x1/4



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Date: 06/16/2025

Job No. 25-003

Revision:

Date:

Sheet No. 84

Project Name: JKV-EMS

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	9	Non-slender	-	-	-	✓ Pass
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	122	-3.0	49.8	kip	0.061	✓ Pass
Shear Minor	10	0.6	20.8	kip	0.031	✓ Pass
Flexure Major	122	2.8	16.7	kip ft	0.169	✓ Pass
Flexure Minor	10	-3.0	10.2	kip ft	0.299	✓ Pass
Axial Tension	133	-0.3	130.0	kip	0.002	✓ Pass
Axial Compression	122	0.4	126.7	kip	0.003	✓ Pass
Combined Forces	10	-	-	-	0.334	✓ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Self weight	118	0.002	-	in	-	-
Deflection Dead	118	-0.003	0.109	in	0.032	✓ Pass
Deflection Live	9	-0.002	0.109	in	0.021	✓ Pass
Deflection Wind	109	-0.002	0.109	in	0.018	✓ Pass
Deflection Total	122	-0.011	0.164	in	0.068	✓ Pass

TOS HBB108

{\H0.5x;\CMU3
A510293}\A1;

16.5"
10"
5' 0" 1' 7 5/8"
5"
1

St. TOS HB (TOS High Bearing):
HSS 5x2T142x1/4 A500B-46
TOS HBB108



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Revision:	Date:	Sheet No. 85
Project Name: JKV-EMS		

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	1' 7 5/8"	•	1.000		1.000		1.000		1.000
support	1' 7 5/8"	•		•		•		•	
sub-beam	5' 0"	•	1.000		1.000		1.000		1.000
support	6' 7 5/8"	•		•		•		•	

Static Design Summary

Summary HSS 5x2-1/2x1/4

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	4	Non-slender	-	-	-	✓ Pass
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	122	0.4	49.8	kip	0.008	✓ Pass
Shear Minor	10	0.3	20.8	kip	0.013	✓ Pass
Flexure Major	122	0.7	16.7	kip ft	0.042	✓ Pass
Flexure Minor	10	-1.2	10.2	kip ft	0.116	✓ Pass
Axial Tension	132	-0.1	130.0	kip	0.001	✓ Pass
Axial Compression	123	0.3	126.7	kip	0.002	✓ Pass
Combined Forces	10	-	-	-	0.132	✓ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Self weight	118	0.002	-	in	-	-
Deflection Dead	118	-0.004	0.109	in	0.034	✓ Pass
Deflection Live	9	-0.002	0.109	in	0.023	✓ Pass
Deflection Wind	109	-0.002	0.109	in	0.022	✓ Pass
Deflection Total	122	-0.012	0.164	in	0.071	✓ Pass

TOS HBB114

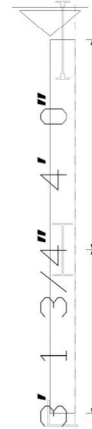


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Revision:	Date:	Sheet No. 86
Project Name: JKV-EMS		

{\H0.5x;\A210407}\A1;

16' 10"



St. TOS HB (TOS High Bearing):
HSS 5x2-1/2x1/4 4500B-46

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	4' 0"	•	1.000		1.000		1.000		1.000
support	4' 0"	•		•		•		•	
sub-beam	3' 1 3/4"	•	1.000		1.000		1.000		1.000
support	7' 1 3/4"	•		•		•		•	

Static Design Summary

Summary HSS 5x2-1/2x1/4

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	2	Non-slender	-	-	-	✓ Pass
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	123	-0.2	49.8	kip	0.004	✓ Pass
Shear Minor	10	0.8	20.8	kip	0.036	✓ Pass
Flexure Major	123	-0.7	16.7	kip ft	0.045	✓ Pass
Flexure Minor	10	-2.3	10.2	kip ft	0.227	✓ Pass
Axial Tension	123	-0.5	130.0	kip	0.004	✓ Pass



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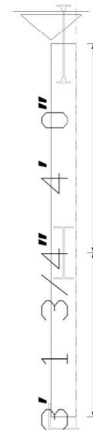
Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 87
Project Name: JKV-EMS		

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Compression	132	0.4	111.3	kip	0.004	✓ Pass
Combined Forces	10	-	-	-	0.253	✓ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Dead	118	0.004	0.210	in	0.021	✓ Pass
Deflection Live	9	0.003	0.210	in	0.014	✓ Pass
Deflection Wind	104	-0.001	0.210	in	0.005	✓ Pass
Deflection Total	123	0.016	0.315	in	0.051	✓ Pass

TOS HBB115

{\H0.5x;\A210407}\A1;

16' 10"



St. TOS HB (TOS High Bearing):

HSS 5x2 1/2x1/4 A500B-46

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	4' 0"	•	1.000		1.000		1.000		1.000
support	4' 0"	•		•		•		•	
sub-beam	3' 1 3/4"	•	1.000		1.000		1.000		1.000
support	7' 1 3/4"	•		•		•		•	

Static Design Summary



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Project Name: JKV-EMS		

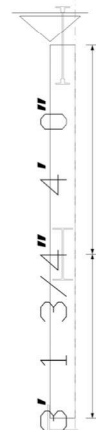
Summary HSS 5x2-1/2x1/4

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	120	Non-slender	-	-	-	✓ Pass
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	29	-0.5	49.8	kip	0.009	✓ Pass
Shear Minor	10	0.5	20.8	kip	0.022	✓ Pass
Flexure Major	29	-1.0	16.7	kip ft	0.059	✓ Pass
Flexure Minor	10	-1.4	10.2	kip ft	0.136	✓ Pass
Axial Tension	132	-0.2	130.0	kip	0.001	✓ Pass
Axial Compression	123	0.2	111.3	kip	0.002	✓ Pass
Combined Forces	10	-	-	-	0.195	✓ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Dead	118	0.004	0.210	in	0.021	✓ Pass
Deflection Live	9	0.003	0.210	in	0.014	✓ Pass
Deflection Wind	109	0.001	0.210	in	0.005	✓ Pass
Deflection Total	123	0.016	0.315	in	0.051	✓ Pass

TOS HBB116

{\H0.5x;\A210407}\A1;

16' 10"



St. TOS HB (TOS High Bearing):
HSS 5x2-1/2x1/4 A500B-46

Restraints



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Revision:	Date:	Sheet No. 89
Project Name: JKV-EMS		

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	4' 0"	•	1.000		1.000		1.000		1.000
support	4' 0"	•		•		•		•	
sub-beam	3' 1 3/4"	•	1.000		1.000		1.000		1.000
support	7' 1 3/4"	•		•		•		•	

Static Design Summary

Summary HSS 5x2-1/2x1/4

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	2	Unknown	-	-	-	Not required
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	29	-0.2	49.8	kip	0.003	✓ Pass
Shear Minor	10	0.2	20.8	kip	0.011	✓ Pass
Flexure Major	120	-0.6	16.7	kip ft	0.038	✓ Pass
Flexure Minor	10	-0.7	10.2	kip ft	0.067	✓ Pass
Axial Tension	10	-0.1	130.0	kip	0.001	✓ Pass
Axial Compression		No Significant Forces		kip	-	Not required
Combined Forces	10	-	-	-	0.093	✓ Pass
Torsion		No Significant Forces		-	-	Not required
Deflection Dead	118	0.004	0.210	in	0.021	✓ Pass
Deflection Live	9	0.003	0.210	in	0.014	✓ Pass
Deflection Wind	109	0.002	0.210	in	0.011	✓ Pass
Deflection Total	120	0.016	0.315	in	0.052	✓ Pass

TOS HBB117



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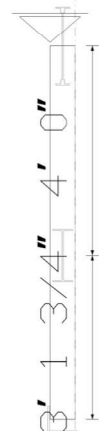
Date:

Sheet No. 90

Project Name: JKV-EMS

{\H0.5x;\A210407}\A1;

16' 10"



St. TOS HB (TOS High Bearing):
HSS 5x2-1/2x1/4 A500B-46

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	4' 0"	•	1.000		1.000		1.000		1.000
support	4' 0"	•		•		•		•	
sub-beam	3' 1 3/4"	•	1.000		1.000		1.000		1.000
support	7' 1 3/4"	•		•		•		•	

Static Design Summary

Summary HSS 5x2-1/2x1/4

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	2	Unknown	-	-	-	Not required
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	29	0.4	49.8	kip	0.008	✓ Pass
Shear Minor	10	0.5	20.8	kip	0.024	✓ Pass
Flexure Major	10	0.7	16.7	kip ft	0.041	✓ Pass
Flexure Minor	10	-1.5	10.2	kip ft	0.147	✓ Pass
Axial Tension	119	-0.2	130.0	kip	0.002	✓ Pass
Axial Compression	No Significant Forces			kip	-	Not required



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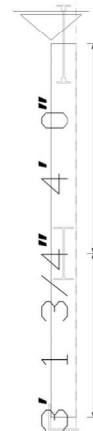
Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 91
Project Name: JKV-EMS		

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Combined Forces	10	-	-	-	0.188	✓ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Dead	118	0.004	0.210	in	0.020	✓ Pass
Deflection Live	9	0.003	0.210	in	0.013	✓ Pass
Deflection Wind	109	0.005	0.210	in	0.022	✓ Pass
Deflection Total	120	0.017	0.315	in	0.055	✓ Pass

TOS HBB118

{\H0.5x;\A210407}\A1;

16' 10"



St. TOS HB (TOS High Bearing):
HSS 5x2 1/4x1/4 A500B-46

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	4' 0"	•	1.000		1.000		1.000		1.000
support	4' 0"	•		•		•		•	
sub-beam	3' 1 3/4"	•	1.000		1.000		1.000		1.000
support	7' 1 3/4"	•		•		•		•	

Static Design Summary



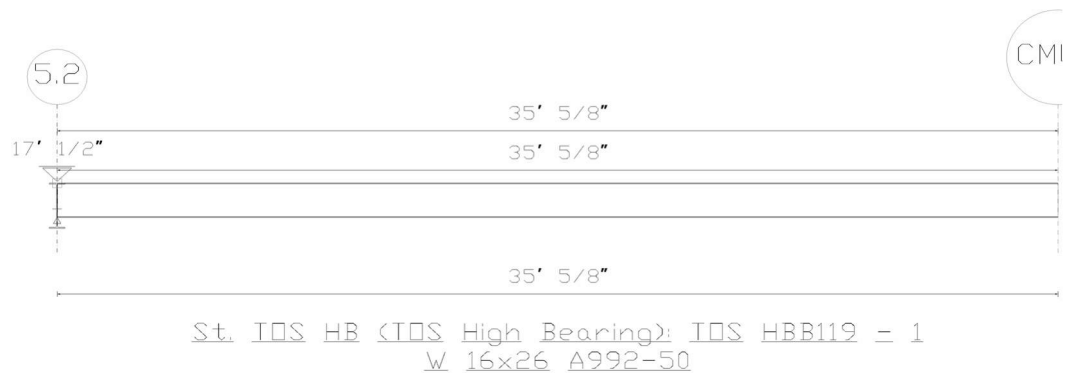
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Revision:	Date:	Sheet No. 92
Project Name: JKV-EMS		

Summary HSS 5x2-1/2x1/4

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	2	Unknown	-	-	-	Not required
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	10	0.2	49.8	kip	0.004	✓ Pass
Shear Minor	10	0.2	20.8	kip	0.011	✓ Pass
Flexure Major	120	-0.6	16.7	kip ft	0.037	✓ Pass
Flexure Minor	10	-0.6	10.2	kip ft	0.062	✓ Pass
Axial Tension	10	-0.1	130.0	kip	0.001	✓ Pass
Axial Compression	No Significant Forces			kip	-	Not required
Combined Forces	10	-	-	-	0.082	✓ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Dead	118	0.004	0.210	in	0.020	✓ Pass
Deflection Live	9	0.003	0.210	in	0.013	✓ Pass
Deflection Wind	109	0.003	0.210	in	0.016	✓ Pass
Deflection Total	120	0.017	0.315	in	0.053	✓ Pass

TOS HBB119



Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	35' 5/8"	•	1.000		1.000		1.000		1.000
support	35' 5/8"	•		•		•		•	

Static Design Summary

Summary W 16x26

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	9	Slender	-	-	-	✓ Pass
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	10	5.5	106.0	kip	0.052	✓ Pass
Shear Minor	No Significant Forces			kip	-	Not required



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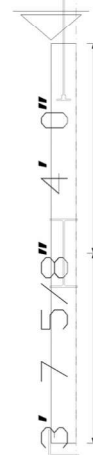
Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 93
Project Name: JKV-EMS		

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Flexure Major	10	48.6	165.7	kip ft	0.293	✓ Pass
Flexure Minor	No Significant Forces			kip ft	-	Not required
Axial Tension	126	-0.4	345.6	kip	0.001	✓ Pass
Axial Compression	121	0.5	12.3	kip	0.040	✓ Pass
Combined Forces	10	-	-	-	0.293	✓ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Self weight	118	0.104	-	in	-	-
Deflection Dead	118	0.479	1.168	in	0.410	✓ Pass
Deflection Live	9	0.315	1.168	in	0.269	✓ Pass
Deflection Wind	106	-0.188	2.103	in	0.089	✓ Pass
Deflection Total	10	0.921	1.753	in	0.525	✓ Pass

TOS HBB120

{\H0.5x;\A2140359}\A1;

16' 10"



St. TOS HB (TOS High Bearing):
HSS 5x2 1/2x1/4 A500B-46

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	



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Revision:	Date:	Sheet No. 94
Project Name: JKV-EMS		

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
sub-beam	4' 0"	•	1.000		1.000		1.000		1.000
support	4' 0"	•		•		•		•	
sub-beam	3' 7 5/8"	•	1.000		1.000		1.000		1.000
support	7' 7 5/8"	•		•		•		•	

Static Design Summary

Summary HSS 5x2-1/2x1/4

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	2	Non-slender	-	-	-	✓ Pass
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	104	-0.1	49.8	kip	0.002	✓ Pass
Shear Minor	10	0.8	20.8	kip	0.038	✓ Pass
Flexure Major	104	-0.5	16.7	kip ft	0.029	✓ Pass
Flexure Minor	10	-2.8	10.2	kip ft	0.278	✓ Pass
Axial Tension	No Significant Forces			kip	-	Not required
Axial Compression	10	0.2	114.3	kip	0.002	✓ Pass
Combined Forces	10	-	-	-	0.286	✓ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Dead	118	-0.003	0.242	in	0.014	✓ Pass
Deflection Live	9	-0.002	0.242	in	0.009	✓ Pass
Deflection Wind	104	0.007	0.242	in	0.028	✓ Pass
Deflection Total	123	-0.009	0.364	in	0.024	✓ Pass

TOS HBB127

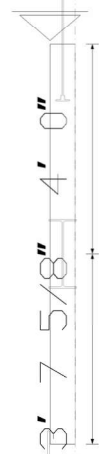


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Revision:	Date:	Sheet No. 95
Project Name: JKV-EMS		

{\H0.5x;\A1;}

16' 10"



St. TOS HB (TOS High Bearing):
HSS 5x2-1/2x1/4 A500B-46

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	4' 0"	•	1.000		1.000		1.000		1.000
support	4' 0"	•		•		•		•	
sub-beam	3' 7 5/8"	•	1.000		1.000		1.000		1.000
support	7' 7 5/8"	•		•		•		•	

Static Design Summary

Summary HSS 5x2-1/2x1/4

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	2	Unknown	-	-	-	Not required
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	63	-0.2	49.8	kip	0.005	✓ Pass
Shear Minor	10	0.7	20.8	kip	0.032	✓ Pass
Flexure Major	104	-0.4	16.7	kip ft	0.026	✓ Pass
Flexure Minor	10	-2.3	10.2	kip ft	0.228	✓ Pass



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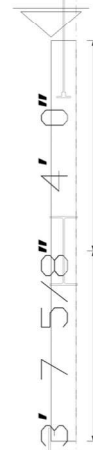
Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 96
Project Name: JKV-EMS		

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Tension	23	-0.1	130.0	kip	0.001	✓ Pass
Axial Compression	No Significant Forces			kip	-	Not required
Combined Forces	10	-	-	-	0.242	✓ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Dead	118	-0.003	0.242	in	0.014	✓ Pass
Deflection Live	9	-0.002	0.242	in	0.009	✓ Pass
Deflection Wind	104	0.007	0.242	in	0.028	✓ Pass
Deflection Total	122	-0.009	0.364	in	0.024	✓ Pass

TOS HBB128

{\H0.5x;\A2140359}\A1;

16' 10"



St. TOS HB (TOS High Bearing):
HSS 5x2 1/2x1/4 500B-46

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	4' 0"	•	1.000		1.000		1.000		1.000
support	4' 0"	•		•		•		•	
sub-beam	3' 7 5/8"	•	1.000		1.000		1.000		1.000
support	7' 7 5/8"	•		•		•		•	



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Revision:	Date:	Sheet No. 97
Project Name: JKV-EMS		

Static Design Summary

Summary HSS 5x2-1/2x1/4

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	2	Unknown	-	-	-	Not required
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	78	-0.2	49.8	kip	0.003	✓ Pass
Shear Minor	10	0.6	20.8	kip	0.028	✓ Pass
Flexure Major	122	0.4	16.7	kip ft	0.024	✓ Pass
Flexure Minor	10	-2.0	10.2	kip ft	0.200	✓ Pass
Axial Tension	No Significant Forces			kip	-	Not required
Axial Compression	No Significant Forces			kip	-	Not required
Combined Forces	10	-	-	-	0.214	✓ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Dead	118	-0.003	0.242	in	0.014	✓ Pass
Deflection Live	9	-0.002	0.242	in	0.009	✓ Pass
Deflection Wind	104	0.007	0.242	in	0.028	✓ Pass
Deflection Total	122	-0.009	0.364	in	0.024	✓ Pass

TOS HBB129

{\H0.5x;\A110359}\A1;

16' 10"

3' 7 5/8" 4' 0"

St. TOS HB (TOS High Bearing):
HSS 5x2-1/2x1/4 A500B-46



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Revision:	Date:	Sheet No. 98
Project Name: JKV-EMS		

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	4' 0"	•	1.000		1.000		1.000		1.000
support	4' 0"	•		•		•		•	
sub-beam	3' 7 5/8"	•	1.000		1.000		1.000		1.000
support	7' 7 5/8"	•		•		•		•	

Static Design Summary

Summary HSS 5x2-1/2x1/4

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	2	Unknown	-	-	-	Not required
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	102	-0.1	49.8	kip	0.003	✓ Pass
Shear Minor	10	0.5	20.8	kip	0.025	✓ Pass
Flexure Major	122	0.5	16.7	kip ft	0.029	✓ Pass
Flexure Minor	10	-1.8	10.2	kip ft	0.177	✓ Pass
Axial Tension	No Significant Forces			kip	-	Not required
Axial Compression	No Significant Forces			kip	-	Not required
Combined Forces	10	-	-	-	0.197	✓ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Dead	118	-0.003	0.242	in	0.014	✓ Pass
Deflection Live	9	-0.002	0.242	in	0.009	✓ Pass
Deflection Wind	104	0.007	0.242	in	0.028	✓ Pass
Deflection Total	122	-0.009	0.364	in	0.024	✓ Pass

TOS HBB130

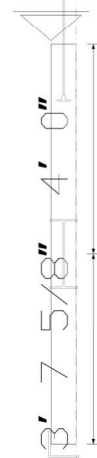


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Project Name: JKV-EMS		

{\H0.5x;\A140359}\A1;

16' 10"



St. TOS HB (TOS High Bearing):
HSS 5x2-1/2x1-1/4 A500B-46

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	4' 0"	•	1.000		1.000		1.000		1.000
support	4' 0"	•		•		•		•	
sub-beam	3' 7 5/8"	•	1.000		1.000		1.000		1.000
support	7' 7 5/8"	•		•		•		•	

Static Design Summary

Summary HSS 5x2-1/2x1/4

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	2	Unknown	-	-	-	Not required
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	29	0.3	49.8	kip	0.006	✓ Pass
Shear Minor	10	0.4	20.8	kip	0.018	✓ Pass
Flexure Major	122	0.6	16.7	kip ft	0.035	✓ Pass
Flexure Minor	10	-1.3	10.2	kip ft	0.129	✓ Pass



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Revision:	Date:	Sheet No. 100
Project Name: JKV-EMS		

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Tension	10	-0.1	130.0	kip	0.001	✓ Pass
Axial Compression	No Significant Forces			kip	-	Not required
Combined Forces	10	-	-	-	0.164	✓ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Dead	118	-0.003	0.242	in	0.014	✓ Pass
Deflection Live	9	-0.002	0.242	in	0.009	✓ Pass
Deflection Wind	108	0.007	0.242	in	0.029	✓ Pass
Deflection Total	122	-0.009	0.364	in	0.025	✓ Pass

TOS HBB131

{\H0.5x;\A1;}

16' 10"

4' 0"
3' 7 5/8"

St. TOS HB (TOS High Bearing):
HSS 5x2 1/2x1/4 500B-46

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	4' 0"	•	1.000		1.000		1.000		1.000
support	4' 0"	•		•		•		•	
sub-beam	3' 7 5/8"	•	1.000		1.000		1.000		1.000
support	7' 7 5/8"	•		•		•		•	



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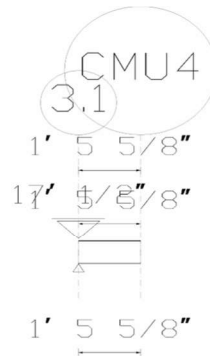
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Revision:	Date:	Sheet No. 101
Project Name: JKV-EMS		

Static Design Summary

Summary HSS 5x2-1/2x1/4

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	2	Unknown	-	-	-	Not required
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	10	0.5	49.8	kip	0.010	✓ Pass
Shear Minor		No Significant Forces		kip	-	Not required
Flexure Major	10	1.0	16.7	kip ft	0.058	✓ Pass
Flexure Minor	10	-0.4	10.2	kip ft	0.037	✓ Pass
Axial Tension	10	-0.2	130.0	kip	0.002	✓ Pass
Axial Compression		No Significant Forces		kip	-	Not required
Combined Forces	10	-	-	-	0.097	✓ Pass
Torsion		No Significant Forces		-	-	Not required
Deflection Dead	118	-0.003	0.242	in	0.014	✓ Pass
Deflection Live	9	-0.002	0.242	in	0.009	✓ Pass
Deflection Wind	108	0.007	0.242	in	0.030	✓ Pass
Deflection Total	122	-0.009	0.364	in	0.025	✓ Pass

TOS HBB133



St. TOS HB (TOS High Bearing): TOS HBB133 =
W 8x10 A992-50

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	1' 5 5/8"	•	1.000		1.000		1.000		1.000
support	1' 5 5/8"	•		•		•		•	

Static Design Summary

Summary W 8x10



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Job No. 25-003

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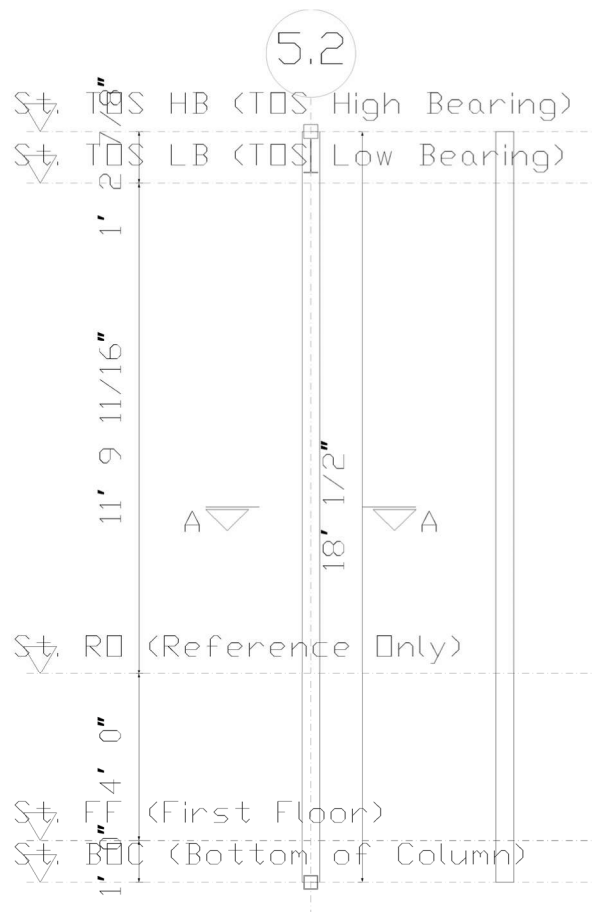
Project Name: JKV-EMS

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	14	Slender	-	-	-	✓ Pass
Flexural Classification	2	Unknown	-	-	-	Not required
Shear Major	10	0.1	40.2	kip	0.003	✓ Pass
Shear Minor		No Significant Forces		kip	-	Not required
Flexure Major		No Significant Forces		kip ft	-	Not required
Flexure Minor		No Significant Forces		kip ft	-	Not required
Axial Tension	104	-2.5	133.2	kip	0.019	✓ Pass
Axial Compression	63	1.6	125.3	kip	0.013	✓ Pass
Combined Forces		No Significant Forces		-	-	Not required
Torsion		No Significant Forces		-	-	Not required

Columns

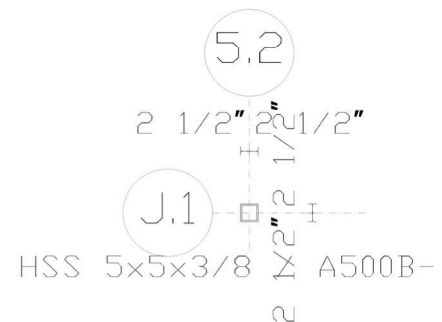
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C244



C244

C244 A-A



Lateral Restraints



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Revision:	Date:	Sheet No. 103
Project Name: JKV-EMS		

Level	Source	Distance / Length	Face A restrained / Sub-stack continuous	Face A factor	Face C restrained / Sub-stack continuous	Face C factor
2	floor	18' 65/128"	Yes		Yes	
	sub-beam	18' 65/128"	No	1.000	No	1.000
1	floor	0"	Yes		Yes	

Strut Restraints

Level	Source	Distance / Length	Major restrained / Sub-stack continuous	Major factor	Minor restrained / Sub-stack continuous	Minor factor
2	floor	18' 65/128"	Yes		Yes	
	sub-beam	18' 65/128"	No	1.000	No	1.000
1	floor	0"	Yes		Yes	

Static Design Summary

Summary HSS 5x5x3/8 (A500B-46)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	2	Non-slender	-	-	-	✓ Pass
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	No	Significant	Forces	kip	-	Not required
Shear Minor	No	Significant	Forces	kip	-	Not required
Flexure Major	9	3.0	36.6	kip ft	0.082	✓ Pass
Flexure Minor	10	-3.4	36.6	kip ft	0.093	✓ Pass
Axial Tension	No	Significant	Forces	kip	-	Not required
Axial Compression	9	15.6	103.7	kip	0.150	✓ Pass
Combined Forces	9	-	-	-	0.246	✓ Pass

C245



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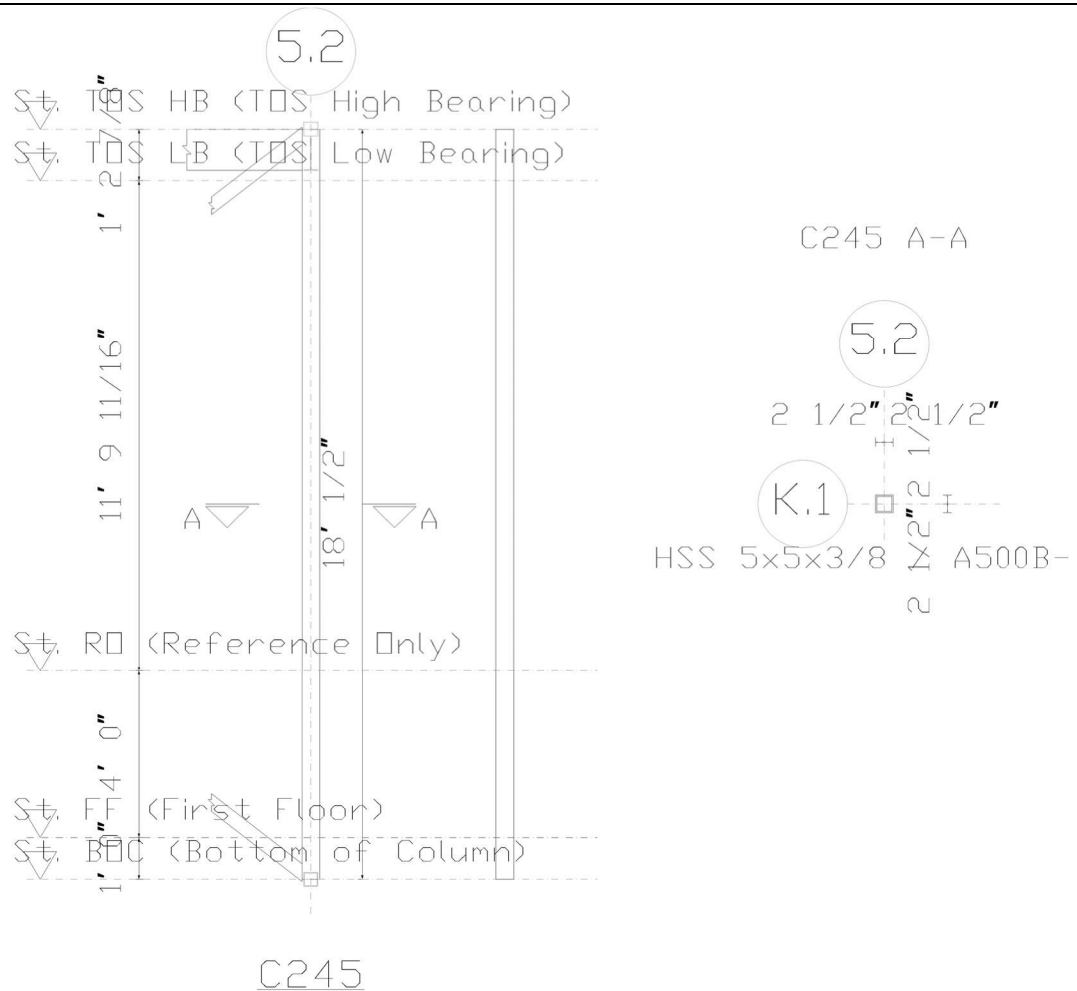
Job No. 25-003

Revision:

Date:

Sheet No. 104

Project Name: JKV-EMS



Lateral Restraints

Level	Source	Distance / Length	Face A restrained / Sub-stack continuous	Face A factor	Face C restrained / Sub-stack continuous	Face C factor
2	floor	18' 65/128"	Yes		Yes	
	sub-beam	18' 65/128"	No	1.000	No	1.000
1	floor	0"	Yes		Yes	

Strut Restraints

Level	Source	Distance / Length	Major restrained / Sub-stack continuous	Major factor	Minor restrained / Sub-stack continuous	Minor factor
2	floor	18' 65/128"	Yes		Yes	
	sub-beam	18' 65/128"	No	1.000	No	1.000
1	floor	0"	Yes		Yes	

Static Design Summary

Summary HSS 5x5x3/8 (A500B-46)

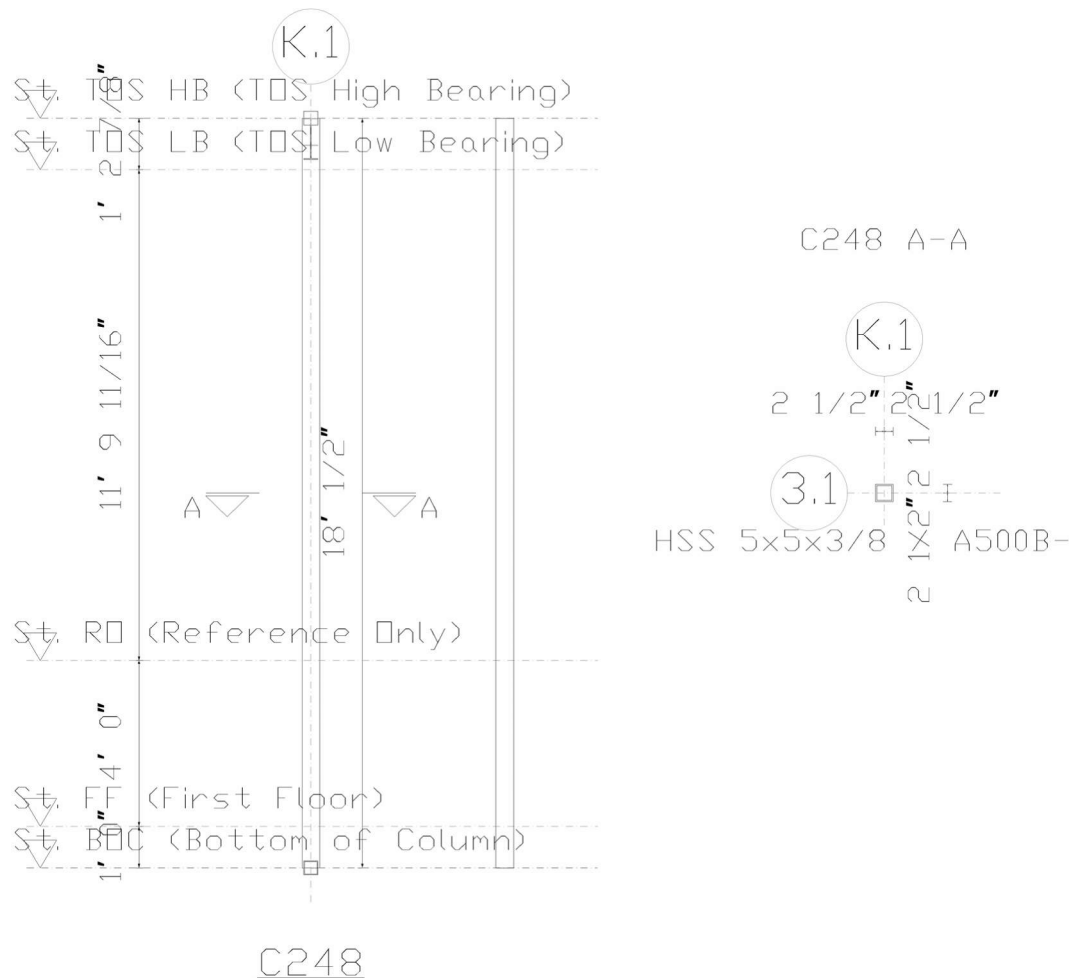


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Revision:	Date:	Sheet No. 105
Project Name: JKV-EMS		

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	2	Non-slender	-	-	-	✓ Pass
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	No	Significant	Forces	kip	-	Not required
Shear Minor	No	Significant	Forces	kip	-	Not required
Flexure Major	10	8.7	36.6	kip ft	0.237	✓ Pass
Flexure Minor	68	-0.2	36.6	kip ft	0.004	✓ Pass
Axial Tension	No	Significant	Forces	kip	-	Not required
Axial Compression	10	47.5	103.7	kip	0.458	✓ Pass
Combined Forces	10	-	-	-	0.669	✓ Pass

C248



Lateral Restraints

Level	Source	Distance / Length	Face A restrained / Sub-stack continuous	Face A factor	Face C restrained / Sub-stack continuous	Face C factor
2	floor	18' 65/128"	No		No	
	sub-beam	18' 65/128"	No	1.000	No	1.000



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Revision:	Date:	Sheet No. 106
Project Name: JKV-EMS		

Level	Source	Distance / Length	Face A restrained / Sub-stack continuous	Face A factor	Face C restrained / Sub-stack continuous	Face C factor
1	floor	0"	Yes		Yes	

Strut Restraints

Level	Source	Distance / Length	Major restrained / Sub-stack continuous	Major factor	Minor restrained / Sub-stack continuous	Minor factor
2	floor	18' 65/128"	Yes		No	
	sub-beam	18' 65/128"	No	1.000	No	1.000
1	floor	0"	Yes		Yes	

Static Design Summary

Summary HSS 5x5x3/8 (A500B-46)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	2	Non-slender	-	-	-	✓ Pass
Flexural Classification	2	Compact	-	-	-	✓ Pass
Shear Major	No	Significant	Forces	kip	-	Not required
Shear Minor	No	Significant	Forces	kip	-	Not required
Flexure Major	10	2.3	36.6	kip ft	0.064	✓ Pass
Flexure Minor	No	Significant	Forces	kip ft	-	Not required
Axial Tension	No	Significant	Forces	kip	-	Not required
Axial Compression	10	6.5	103.7	kip	0.062	✓ Pass
Combined Forces	10	-	-	-	0.095	✓ Pass

Braces

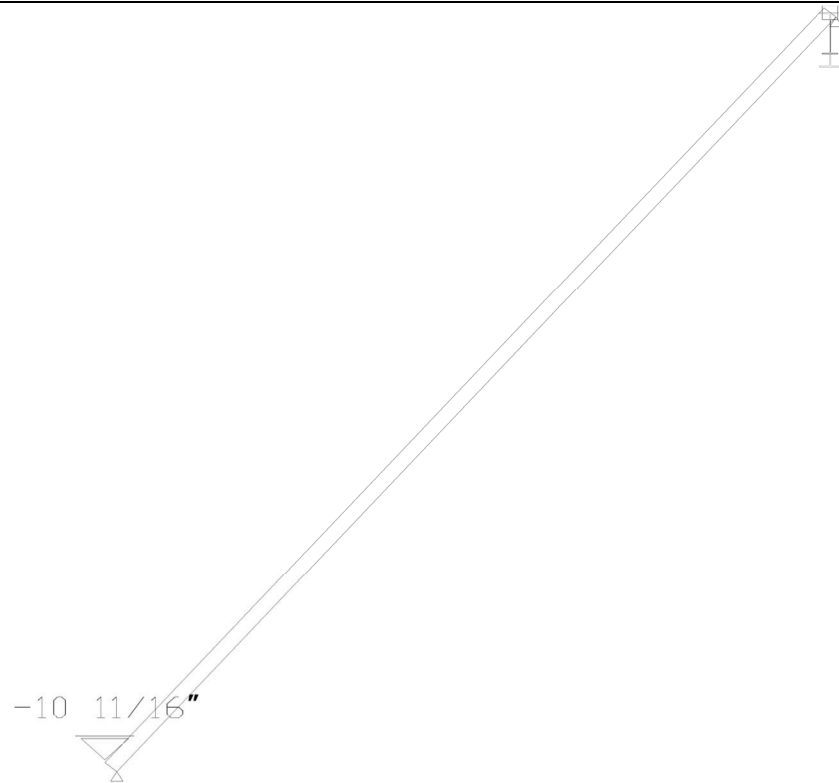
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SBR1



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Revision:	Date:	Sheet No. 107
Project Name: JKV-EMS		



FRM 5.2: SBR1 = 1 (HSS 4x4x0.25 A500B-4)

Static Design Summary

Summary HSS 4x4x0.25(A500B-46)

Design Condition	Combination Name	Design Value	Design Capacity	Units	U.R.	Status
Classification	2	Unknown	-	-	-	Not required
Axial Compression	2	No Axial Compression		kip	-	Not required
Axial Tension	2	No Axial Tension		kip	-	Not required

SBR2



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Project Name: JKV-EMS		

17' 1 13/16"



FRM 5.2: SBR2 - 1 (HSS 4x4x0.25 A500B-4)

Static Design Summary

Summary HSS 4x4x0.25(A500B-46)

Design Condition	Combination Name	Design Value	Design Capacity	Units	U.R.	Status
Classification	2	Unknown	-	-	-	Not required
Axial Compression	2	No Axial Compression		kip	-	Not required
Axial Tension	2	No Axial Tension		kip	-	Not required

SBR3



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Revision:	Date:	Sheet No. 109
Project Name: JKV-EMS		

-10 7/16"

FRM K.1: SBR3 - 1 (HSS 4x4x0.25 A500B-4

Static Design Summary

Summary HSS 4x4x0.25(A500B-46)

Design Condition	Combination Name	Design Value	Design Capacity	Units	U.R.	Status
Classification	2	Unknown	-	-	-	Not required
Axial Compression	2	No Axial Compression		kip	-	Not required
Axial Tension	2	No Axial Tension		kip	-	Not required

SBR4



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Project Name: JKV-EMS		

17' 2 1/16"



FRM K.1: SBR4 - 1 (HSS 4x4x0.25 A500B-4

Static Design Summary

Summary HSS 4x4x0.25(A500B-46)

Design Condition	Combination Name	Design Value	Design Capacity	Units	U.R.	Status
Classification	2	Non-slender	-	-	-	✓ Pass
Axial Compression	69	3.5	14.4	kip	0.240	✓ Pass
Axial Tension	127	-2.9	139.5	kip	0.021	✓ Pass

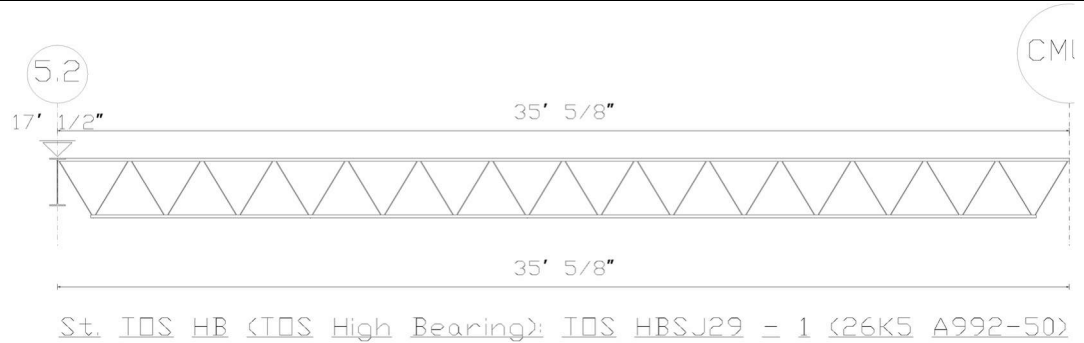
Steel Joists

TOS HBSJ29



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Revision:	Date:	Sheet No. 111
Project Name: JKV-EMS		

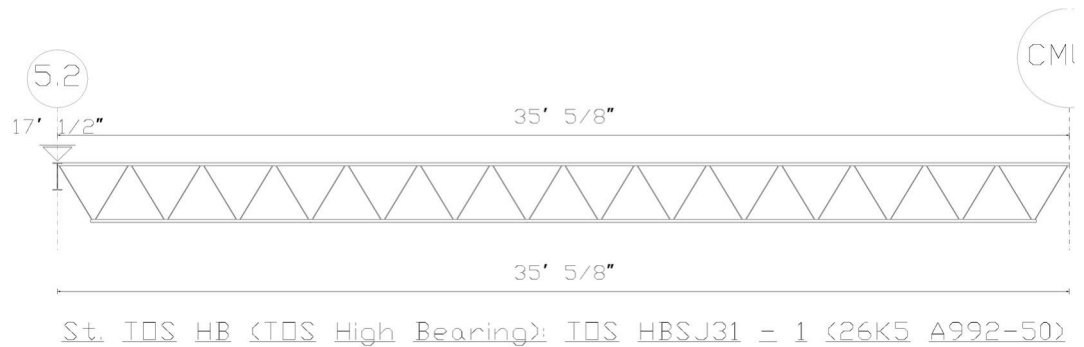


Static Design Summary

Summary 26K5 (A992-50)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Min. Joist Depth	-	2' 2"	1' 5 135/256"	ft, in	0.674	✓ Pass
Strength	10	0.27	0.44	klf	0.619	✓ Pass
Deflection Dead	-	-	-	klf	-	Not required
Deflection Live	10	0.08	0.22	klf	0.379	✓ Pass
Deflection Total	10	0.20	0.32	klf	0.622	✓ Pass

TOS HBSJ31



Static Design Summary

Summary 26K5 (A992-50)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Min. Joist Depth	-	2' 2"	1' 5 135/256"	ft, in	0.674	✓ Pass
Strength	10	0.28	0.44	klf	0.637	✓ Pass
Deflection Dead	-	-	-	klf	-	Not required
Deflection Live	10	0.08	0.22	klf	0.390	✓ Pass
Deflection Total	10	0.21	0.32	klf	0.640	✓ Pass

TOS HBSJ32



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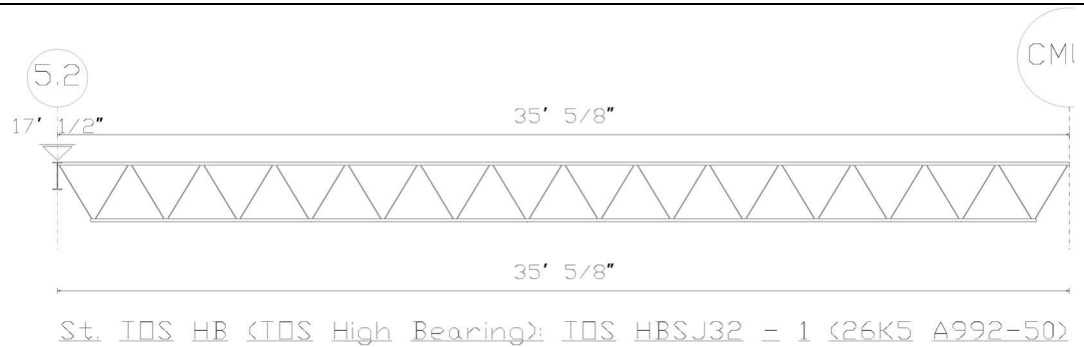
Job No. 25-003

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Date:

Sheet No. 112

Project Name: JKV-EMS

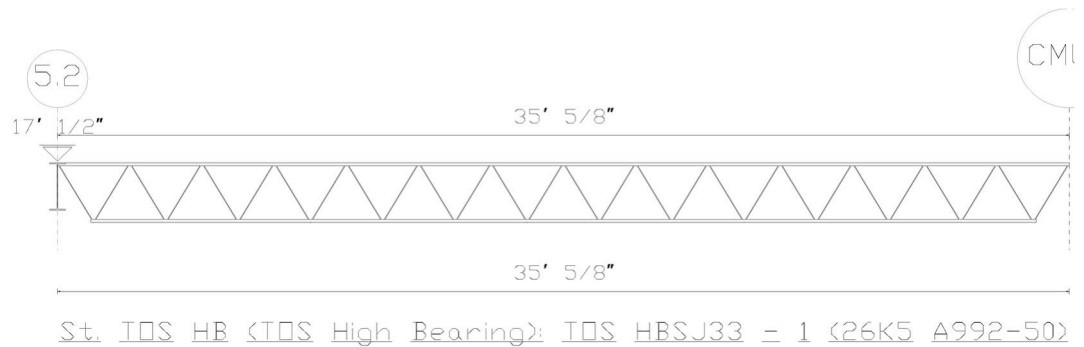


Static Design Summary

Summary 26K5 (A992-50)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Min. Joist Depth	-	2' 2"	1' 5 135/256"	ft, in	0.674	✓ Pass
Strength	10	0.27	0.44	klf	0.610	✓ Pass
Deflection Dead	-	-	-	klf	-	Not required
Deflection Live	10	0.08	0.22	klf	0.373	✓ Pass
Deflection Total	10	0.20	0.32	klf	0.613	✓ Pass

TOS HBSJ33



Static Design Summary

Summary 26K5 (A992-50)

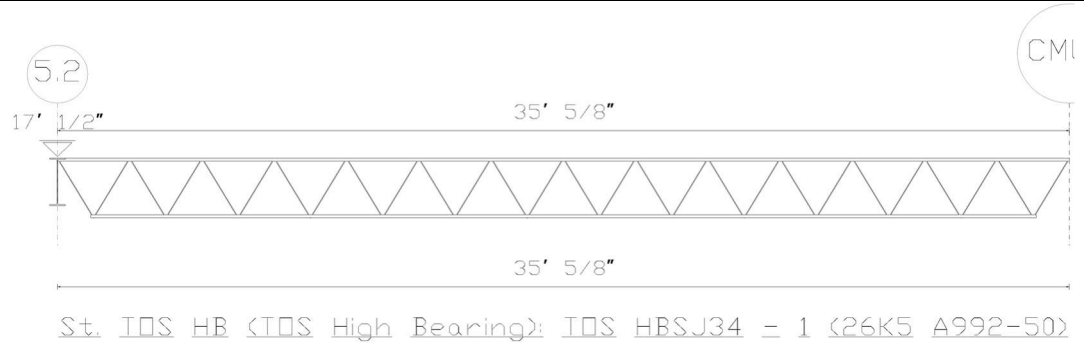
Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Min. Joist Depth	-	2' 2"	1' 5 135/256"	ft, in	0.674	✓ Pass
Strength	10	0.37	0.44	klf	0.824	✓ Pass
Deflection Dead	-	-	-	klf	-	Not required
Deflection Live	10	0.11	0.22	klf	0.509	✓ Pass
Deflection Total	10	0.27	0.32	klf	0.827	✓ Pass

TOS HBSJ34



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Revision:	Date:	Sheet No. 113
Project Name: JKV-EMS		

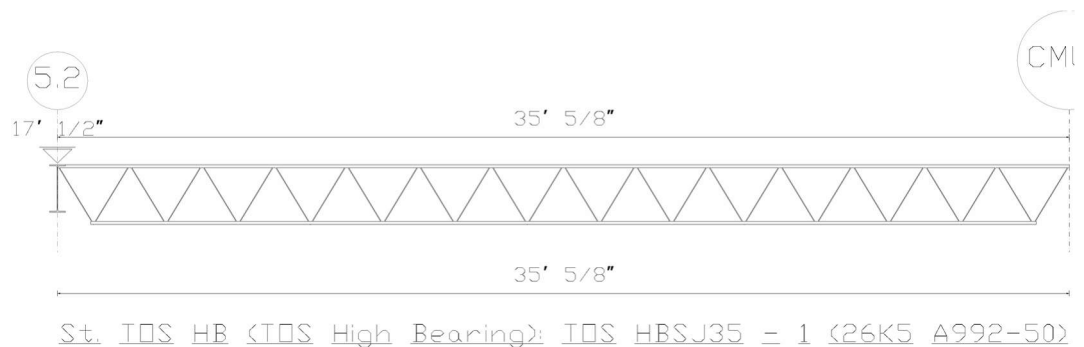


Static Design Summary

Summary 26K5 (A992-50)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Min. Joist Depth	-	2' 2"	1' 5 135/256"	ft, in	0.674	✓ Pass
Strength	10	0.37	0.44	klf	0.824	✓ Pass
Deflection Dead	-	-	-	klf	-	Not required
Deflection Live	10	0.11	0.22	klf	0.509	✓ Pass
Deflection Total	10	0.27	0.32	klf	0.827	✓ Pass

TOS HBSJ35



Static Design Summary

Summary 26K5 (A992-50)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Min. Joist Depth	-	2' 2"	1' 5 135/256"	ft, in	0.674	✓ Pass
Strength	10	0.31	0.44	klf	0.704	✓ Pass
Deflection Dead	-	-	-	klf	-	Not required
Deflection Live	10	0.09	0.22	klf	0.433	✓ Pass
Deflection Total	10	0.23	0.32	klf	0.707	✓ Pass

TOS HBSJ37



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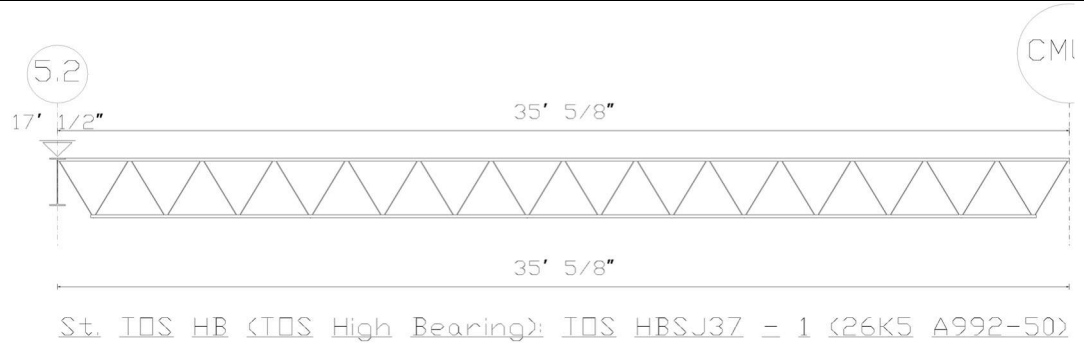
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Revision:

Date:

Sheet No. 114

Project Name: JKV-EMS

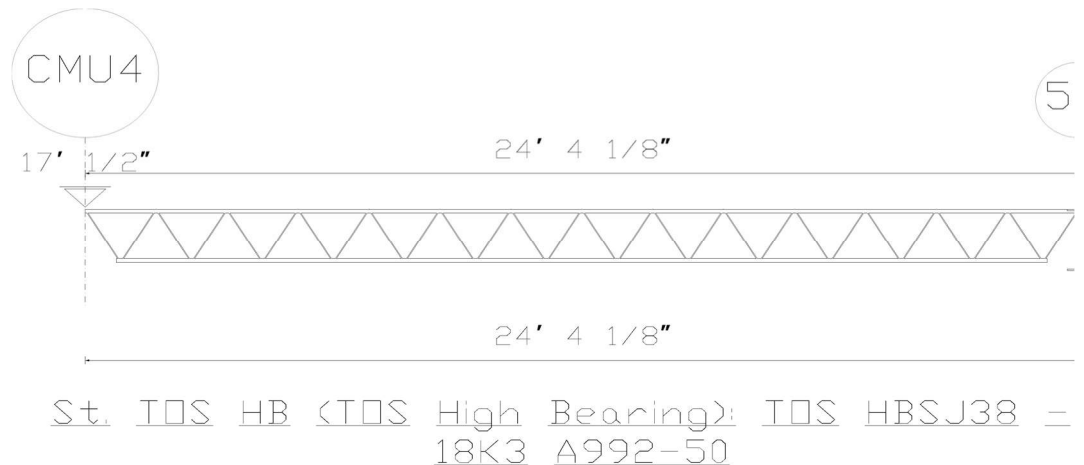


Static Design Summary

Summary 26K5 (A992-50)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Min. Joist Depth	-	2' 2"	1' 5 135/256"	ft, in	0.674	✓ Pass
Strength	10	0.26	0.44	klf	0.579	✓ Pass
Deflection Dead	-	-	-	klf	-	Not required
Deflection Live	10	0.08	0.22	klf	0.354	✓ Pass
Deflection Total	10	0.19	0.32	klf	0.582	✓ Pass

TOS HBSJ38



Static Design Summary

Summary 18K3 (A992-50)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Min. Joist Depth	-	1' 6"	1' 11/64"	ft, in	0.676	✓ Pass
Strength	10	0.36	0.47	klf	0.779	✓ Pass
Deflection Dead	-	-	-	klf	-	Not required
Deflection Live	10	0.11	0.23	klf	0.473	✓ Pass
Deflection Total	10	0.27	0.35	klf	0.764	✓ Pass



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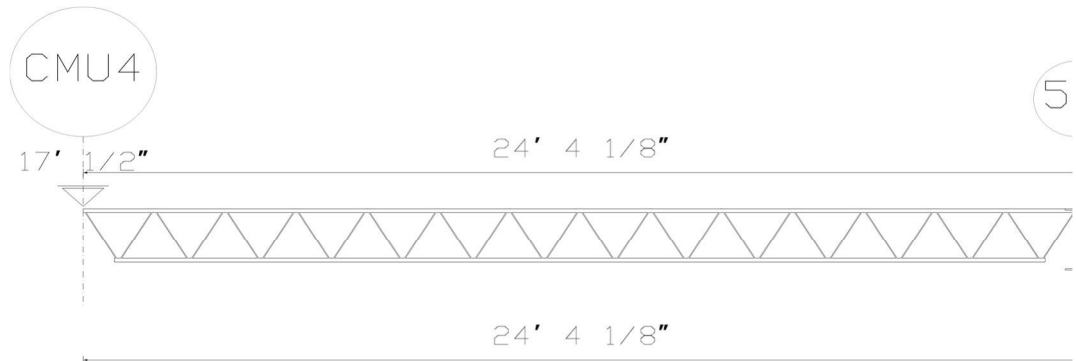
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Date:

Sheet No. 115

Project Name: JKV-EMS

TOS HBSJ39



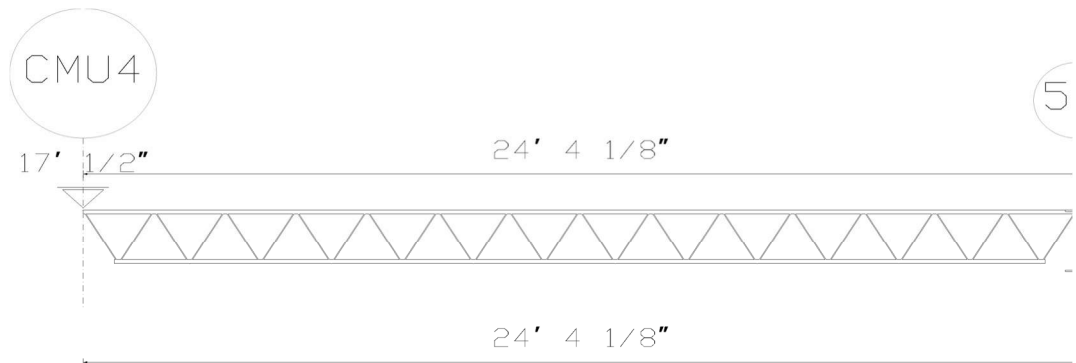
St. TOS HB (TOS High Bearing): TOS HBSJ39 =
18K3 A992-50

Static Design Summary

Summary 18K3 (A992-50)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Min. Joist Depth	-	1' 6"	1' 11/64"	ft, in	0.676	✓ Pass
Strength	10	0.36	0.47	klf	0.779	✓ Pass
Deflection Dead	-	-	-	klf	-	Not required
Deflection Live	10	0.11	0.23	klf	0.473	✓ Pass
Deflection Total	10	0.27	0.35	klf	0.764	✓ Pass

TOS HBSJ40



St. TOS HB (TOS High Bearing): TOS HBSJ40 =
18K3 A992-50

Static Design Summary

Summary 18K3 (A992-50)

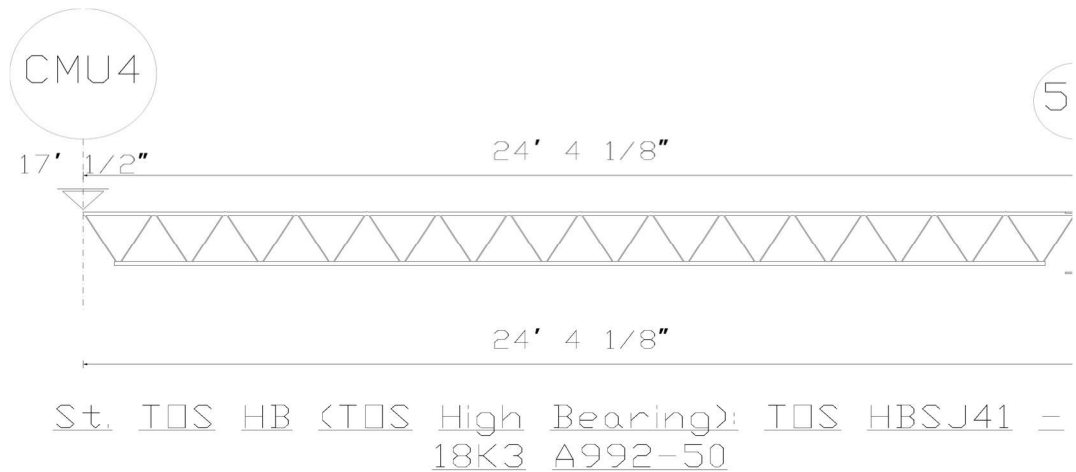


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Revision:	Date:	Sheet No. 116
Project Name: JKV-EMS		

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Min. Joist Depth	-	1' 6"	1' 11/64"	ft, in	0.676	✓ Pass
Strength	10	0.31	0.47	klf	0.665	✓ Pass
Deflection Dead	-	-	-	klf	-	Not required
Deflection Live	10	0.09	0.23	klf	0.402	✓ Pass
Deflection Total	10	0.23	0.35	klf	0.652	✓ Pass

TOS HBSJ41



Static Design Summary

Summary 18K3 (A992-50)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Min. Joist Depth	-	1' 6"	1' 11/64"	ft, in	0.676	✓ Pass
Strength	10	0.26	0.47	klf	0.547	✓ Pass
Deflection Dead	-	-	-	klf	-	Not required
Deflection Live	10	0.08	0.23	klf	0.329	✓ Pass
Deflection Total	10	0.19	0.35	klf	0.537	✓ Pass



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Revision:	Date:	Sheet No. 117
Project Name: JKV-EMS		

MASONRY WALL SECTION DESIGN - WP17

In accordance with TMS-16 using the allowable stress design method

Tedds calculation version 1.0.06

Design summary

Overall design status; PASS
Overall design utilization; 0.418

Top: Max axial

Description	Unit	Provided	Required	Utilization	Result
Out-of-plane axial force	kips/ft	22.1	1.4	0.065	PASS
Out-of-plane axial stress	psi	357.8	21.3	0.060	PASS
Out-of-plane flexural strength	kip_ft/ft	2.250	0.857	0.381	PASS
Out-of-plane shear stress	psi	46.5	7.8	0.167	PASS
In-plane axial force	kips	283.6	10.7	0.038	PASS
In-plane axial stress	psi	615.2	21.3	0.035	PASS
In-plane flexural strength	kip_ft	222.7	16.9	0.076	PASS
In-plane shear stress	psi	96.6	10.5	0.109	PASS
In-plane shear friction	psi	101.3	10.5	0.104	PASS
Biaxial bending				0.388	PASS

Top: Min axial

Description	Unit	Provided	Required	Utilization	Result
Out-of-plane axial force	kips/ft	22.1	0.2	0.011	PASS
Out-of-plane axial stress	psi	357.8	3.6	0.010	PASS
Out-of-plane flexural strength	kip_ft/ft	2.097	0.857	0.409	PASS
Out-of-plane shear stress	psi	42.9	7.8	0.182	PASS
In-plane axial force	kips	283.6	1.8	0.006	PASS
In-plane axial stress	psi	615.2	3.6	0.006	PASS
In-plane flexural strength	kip_ft	196.4	16.9	0.086	PASS
In-plane shear stress	psi	93.0	10.5	0.113	PASS
In-plane shear friction	psi	86.8	10.5	0.121	PASS
Biaxial bending				0.418	PASS

Mid-fifth: Max axial

Description	Unit	Provided	Required	Utilization	Result
Out-of-plane axial force	kips/ft	22.1	2.6	0.119	PASS
Out-of-plane axial stress	psi	357.8	39.2	0.110	PASS
Out-of-plane flexural strength	kip_ft/ft	2.548	0.161	0.063	PASS
Out-of-plane shear stress	psi	50.2	1.8	0.036	PASS



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Revision:	Date:	Sheet No. 118
Project Name: JKV-EMS		

Description	Unit	Provided	Required	Utilization	Result
In-plane axial force	kips	283.6	19.7	0.069	PASS
In-plane axial stress	psi	615.2	39.2	0.064	PASS
In-plane flexural strength	kip_ft	247.9	33.2	0.134	PASS
In-plane shear stress	psi	75.0	10.0	0.134	PASS
In-plane shear friction	psi	77.7	10.0	0.129	PASS
Biaxial bending				0.148	PASS

Mid-fifth: Min axial

Description	Unit	Provided	Required	Utilization	Result
Out-of-plane axial force	kips/ft	22.1	0.2	0.007	PASS
Out-of-plane axial stress	psi	357.8	2.4	0.007	PASS
Out-of-plane flexural strength	kip_ft/ft	2.097	0.161	0.077	PASS
Out-of-plane shear stress	psi	42.7	1.8	0.042	PASS
In-plane axial force	kips	283.6	1.2	0.004	PASS
In-plane axial stress	psi	615.2	2.4	0.004	PASS
In-plane flexural strength	kip_ft	193.4	33.2	0.172	PASS
In-plane shear stress	psi	75.0	10.0	0.134	PASS
In-plane shear friction	psi	49.7	10.0	0.202	PASS
Biaxial bending				0.188	PASS

Bottom: Max axial

Description	Unit	Provided	Required	Utilization	Result
Out-of-plane axial force	kips/ft	22.1	3.7	0.166	PASS
Out-of-plane axial stress	psi	357.8	54.6	0.152	PASS
Out-of-plane flexural strength	kip_ft/ft	2.667	0.027	0.010	PASS
Out-of-plane shear stress	psi	53.3	1.2	0.022	PASS
In-plane axial force	kips	283.6	27.4	0.097	PASS
In-plane axial stress	psi	615.2	54.6	0.089	PASS
In-plane flexural strength	kip_ft	267.5	59.0	0.221	PASS
In-plane shear stress	psi	75.0	10.3	0.137	PASS
In-plane shear friction	psi	89.4	10.3	0.115	PASS
Biaxial bending				0.221	PASS

Bottom: Min axial

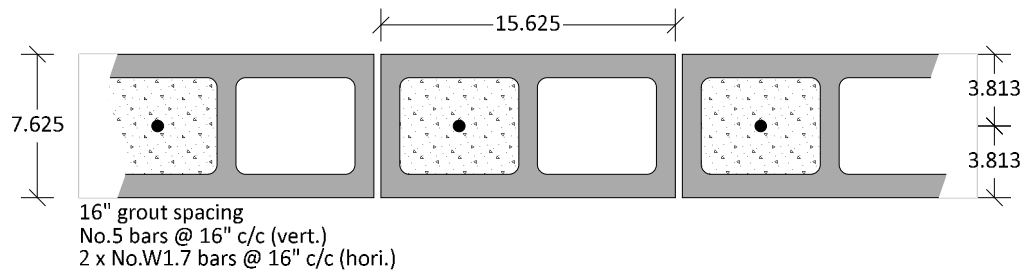
Description	Unit	Provided	Required	Utilization	Result
Out-of-plane axial force	kips/ft	22.1	0.1	0.003	PASS
Out-of-plane axial stress	psi	357.8	1.0	0.003	PASS



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Revision:	Date:	Sheet No. 119
Project Name: JKV-EMS		

Description	Unit	Provided	Required	Utilization	Result
Out-of-plane flexural strength	kip_ft/ft	2.097	0.027	0.013	PASS
Out-of-plane shear stress	psi	42.4	1.2	0.028	PASS
In-plane axial force	kips	283.6	0.5	0.002	PASS
In-plane axial stress	psi	615.2	1.0	0.002	PASS
In-plane flexural strength	kip_ft	193.4	59.0	0.305	PASS
In-plane shear stress	psi	75.0	10.3	0.137	PASS
In-plane shear friction	psi	48.6	10.3	0.211	PASS
Biaxial bending				0.305	PASS



Masonry details

8 in CMU in running bond, face bedded with S class PCL mortar on an unfinished bearing surface

Length of section;	$L = 7.469$ ft
Effective height (minor axis);	$h_{eff,y} = 18.376$ ft
Effective height (major axis);	$h_{eff,x} = 37.854$ ft
Compressive strength of unit;	$f_{cu} = 3250$ psi
Density of masonry units;	$\rho_m = 130$ lb/ft ³
Thickness of masonry units;	$t = 7.625$ in
Height of masonry units;	$h_m = 7.625$ in
Length of masonry units;	$l_m = 15.625$ in
Number of internal webs;	$n_{w,int} = 1$
Internal web thickness;	$t_{w,int} = 1$ in
Number of end webs;	$n_{w,ext} = 2$
End web thickness;	$t_{w,ext} = 1$ in
Face shell thickness;	$t_{fs} = 1.25$ in
Depth of cavity;	$t_c = t - 2 \times t_{fs} = 5.125$ in
Length of cavity;	$l_c = (l_m - n_{w,int} \times t_{w,int} - n_{w,ext} \times t_{w,ext}) / 2 = 6.313$ in
Area of cavity;	$A_c = t_c \times l_c = 32.4$ in ²
Spacing of grout;	$s_{grout} = 16$ in
Density of grout;	$\rho_{grout} = 140$ lb/ft ³

Compressive strength of masonry

Net compr. strength of masonry - TMS602 Table 2;	$f_m = 2500$ psi
Modulus of elasticity for masonry - Table 4.2.2;	$E_m = 900 \times f_m = 2250000$ psi
Shear modulus of masonry;	$G_m = E_m / (2 \times (1 + \nu_m)) = 900000$ psi



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Revision:	Date:	Sheet No. 120
Project Name: JKV-EMS		

Max usable comp. strain in masonry;

$$\epsilon_{mu} = 0.00250$$

Reinforcement details

Allow. tensile stress in vert. reinf. - cl.8.3.3

$$F_s = 32000 \text{ psi}$$

Yield strength of reinforcement

$$f_{y,vert} = 60000 \text{ psi}$$

Modulus of elasticity for reinforcement

$$E_s = 29000000 \text{ psi}$$

Tensile strain in reinforcement at balance point;

$$\epsilon_{ty} = f_{y,vert} / E_s = 0.002069$$

Vertical reinforcement provided

$$\text{No.5 bars @ 16" c/c}$$

Area of vertical reinforcement

$$A_s = \pi \times \phi_{vert}^2 / (4 \times s_{vert}) = 0.23 \text{ in}^2/\text{ft}$$

Max. allow. vertical reinf. diameter - cl.6.1.2.2/5

$$\phi_{vert,max} = \min(\min(t, l_m) / 8, \min(t_c, l_c) / 2) = 0.95 \text{ in}$$

PASS - Maximum allowable vertical bar diameter exceeds provided

Nominal front cover;

$$C_{nom,front} = 3.500 \text{ in}$$

Neither face exposed to earth or weather

Minimum rear cover - cl.6.1.3.5/6.1.4.1;

$$C_{nom,min,rear} = \max(t_{grout,min} + t_{fs}, 1.5 \text{ in}) = 1.750 \text{ in}$$

Minimum front cover;

$$C_{nom,min,front} = \max(t_{grout,min} + t_{fs}, 1.5 \text{ in}) = 1.750 \text{ in}$$

PASS - Cover to reinforcement exceeds minimum cover required

Min allow. clear spacing between grout and unit;

$$t_{grout,min} = 0.500 \text{ in}$$

Clear spacing between grout and unit;

$$s_{vert,clear} = \min(t - (\phi_{vert} + C_{nom,front}), C_{nom,front} - t_{fs}) = 2.250 \text{ in}$$

PASS - Clear spacing between reinf. and masonry unit exceeds minimum allowable

Min req'd vert reinf in shear walls - cl.9.3.6.2;

$$A_{s,h,min} = A_v / 3 = 0.009 \text{ in}^2/\text{ft}$$

PASS - Area of vertical reinforcement exceeds minimum required

Max allow vert spacing in shear walls - cl.9.3.6.2;

$$s_{vert,h,max} = 96.000 \text{ in}$$

PASS - Maximum allowable vertical reinforcement spacing exceeds provided

Allow. tensile stress in hori. reinf. - cl.8.3.3

$$F_{sv} = 30000 \text{ psi}$$

Yield strength of horizontal reinforcement

$$f_{y,hori} = 70000 \text{ psi}$$

Horizontal reinforcement provided

$$2 \times \text{No.3.7592 wires @ 16" c/c}$$

Area of horizontal reinforcement

$$A_v = 2 \times \pi \times \phi_{hori}^2 / (4 \times s_{hori}) = 0.03 \text{ in}^2/\text{ft}$$

Max. allow. horizontal reinf. diameter - cl.6.1.2.2/5

$$\phi_{hori,max} = t_j / 2 = 0.19 \text{ in}$$

PASS - Maximum allowable horizontal bar diameter exceeds provided

Area of cell;

$$A_c = t_c \times l_c = 32.352 \text{ in}^2$$

Area of reinforcement in cell;

$$A_{s,cell} = n_{vert} \times \phi_{vert}^2 \times \pi / 4 = 0.307 \text{ in}^2$$

Max allow. reinforcement in cell - cl.6.1.2.4;

$$A_{s,max,cell} = 0.06 \times A_c = 1.941 \text{ in}^2$$

PASS - Maximum allowable area of reinforcement in cell exceeds provided

Masonry section properties

Properties for walls loaded axially and out of plane

Gross cross-sectional area;

$$A = t - l_c \times t_c / (l_m + t_j) = 67.2 \text{ in}^2/\text{ft}$$

Gross moment of inertia;

$$I = t^3 / 12 - l_c \times t_c^3 / (12 \times (l_m + t_j)) = 390.2 \text{ in}^4/\text{ft}$$

Gross section modulus;

$$S = 2 \times I / t = 102.4 \text{ in}^3/\text{ft}$$

Gross radius of gyration;

$$r = \sqrt{I / A} = 2.4 \text{ in}$$

Net cross-sectional area;

$$A_n = 2 \times t_{fs} + t_c \times (l_c + 2 \times t_{w,ext}) / (l_m + t_j) = 62 \text{ in}^2/\text{ft}$$

Net moment of inertia;

$$I_n = (t^3 - t_c^3) / 12 + ((l_c + 2 \times t_{w,ext}) \times t_c^3 / (12 \times (l_m + t_j))) = 378.6 \text{ in}^4/\text{ft}$$

Net section modulus;

$$S_n = 2 \times I_n / t = 99.3 \text{ in}^3/\text{ft}$$

Net radius of gyration;

$$r_n = \sqrt{I_n / A_n} = 2.5 \text{ in}$$



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Revision:	Date:	Sheet No. 121
Project Name: JKV-EMS		

Properties for walls loaded in-plane

Equivalent thickness;

$$t_{equiv} = A_n = \mathbf{5.163 \text{ in}}$$

Net moment of inertia;

$$I_{x\_net} = t_{equiv} \times L^3 / 12 = \mathbf{309724 \text{ in}^4}$$

Net section modulus;

$$S_{x\_net} = I_{x\_net} / (L / 2) = \mathbf{6912 \text{ in}^3}$$

Net radius of gyration;

$$r_x = \sqrt{(I_{x\_net} / (A_n \times L))} = \mathbf{26 \text{ in}}$$

Properties for horizontally spanning walls

Ratio of grouted bond beams to voids;

$$G_{hori} = 0 = \mathbf{0}$$

Gross cross-sectional area;

$$A_{g\_hori} = t = \mathbf{91.5 \text{ in}^2/\text{ft}}$$

Gross moment of inertia;

$$I_{g\_hori} = t^3 / 12 = \mathbf{443.3 \text{ in}^4/\text{ft}}$$

Net cross-sectional area;

$$A_{n\_hori} = G_{hori} \times t + (1 - G_{hori}) \times (2 \times t_{fs}) = \mathbf{30 \text{ in}^2/\text{ft}}$$

Net moment of inertia;

$$I_{n\_hori} = G_{hori} \times t^3 / 12 + (1 - G_{hori}) \times (t^3 / 12 - (t - 2 \times t_{fs})^3 / 12) = \mathbf{308.7 \text{ in}^4/\text{ft}}$$

Net section modulus;

$$S_{n\_hori} = 2 \times I_{n\_hori} / t = \mathbf{81 \text{ in}^3/\text{ft}}$$

Average out-of-plane section properties

Average cross-sectional area;

$$A_{ave} = (t_j \times A_n + h_m \times A) / (t_j + h_m) = \mathbf{67 \text{ in}^2/\text{ft}}$$

Average moment of inertia;

$$I_{ave} = (t_j \times I_n + h_m \times I) / (t_j + h_m) = \mathbf{389.7 \text{ in}^4/\text{ft}}$$

Average cross-sectional area;

$$A_{ave\_hori} = (n_{w,ext} \times t_{w,ext} + n_{w,int} \times t_{w,int}) / (l_m + t_j) \times A_{g\_hori} + (l_m + t_j - n_{w,ext} \times t_{w,ext} - n_{w,int} \times t_{w,int}) / (l_m + t_j) \times A_{n\_hori} = \mathbf{41.5 \text{ in}^2/\text{ft}}$$

Average moment of inertia;

$$I_{ave\_hori} = (n_{w,ext} \times t_{w,ext} + n_{w,int} \times t_{w,int}) / (l_m + t_j) \times I_{g\_hori} + (l_m + t_j - n_{w,ext} \times t_{w,ext} - n_{w,int} \times t_{w,int}) / (l_m + t_j) \times I_{n\_hori} = \mathbf{334 \text{ in}^4/\text{ft}}$$

Orthotropic properties for section adjustment

Primary bending plane;

Major

Elastic modulus x - horizontal spanning minor axis;

$$E_x = E_m \times I_{ave\_hori} / I_{g\_hori} = \mathbf{1694904 \text{ psi}}$$

Elastic modulus y - vertical spanning major axis;

$$E_y = E_m \times A_{ave} / t = \mathbf{1647260 \text{ psi}}$$

In-plane shear modulus;

$$G_{xy} = G_m \times A_{ave} / t = \mathbf{658904 \text{ psi}}$$

Out-of-plane shear modulus;

$$G_{xz} = G_{xy} = \mathbf{658904 \text{ psi}}$$

Poisson ratio;

$$\nu_{xy} = \mathbf{0.250}$$

Average wall density;

$$\rho_{ave} = (A_m \times \rho_m + A_g \times \rho_{grout}) / t + 2.5 \text{ pcf} = \mathbf{100 \text{ lb/ft}^3}$$

Out-of-plane axial resistance

Slenderness ratio;

$$h_{eff,y} / r = \mathbf{91.532}; < 99$$

Allowable axial force - Eqn.8-18;

$$P_{a\_pos} = (0.25 \times f'_m \times (A_n - A_s)) \times (1 - (h_{eff,y} / (140 \times r))^2) = \mathbf{22.09 \text{ kips/ft}}$$

Out-of-plane axial stress resistance

Slenderness ratio;

$$h_{eff,y} / r = \mathbf{91.532}; < 99$$

Allowable axial stress - cl.8.2.4.1;

$$F_{a\_pos} = (1 / 4) \times f'_m \times (1 - (h_{eff,y} / (140 \times r))^2) = \mathbf{357.8 \text{ psi}}$$

Moment at balance point

Modular ratio;

$$n = E_s / E_m = \mathbf{12.89}$$

Allowable stress in masonry (cl.8.3.4.2.2);

$$F_b = 0.45 \times f'_m = \mathbf{1125 \text{ psi}}$$

Balance point;

$$k_b = n / (n + F_s / F_b) = \mathbf{0.312}$$

Tensile strain in reinforcement;

$$\epsilon_s = F_s / E_s = \mathbf{0.001103}$$

Compressive strain in masonry;

$$\epsilon_m = \epsilon_s \times k_b / (1 - k_b) = \mathbf{0.000500}$$

Effective depth of reinforcement;

$$d = \mathbf{3.813 \text{ in}}$$

Width of T section web;

$$b_w = (l_c + t_{w,ext} + t_{w,int}) / s_{vert} \times 12 \text{ in/ft} = \mathbf{0.520 \text{ ft/ft}}$$



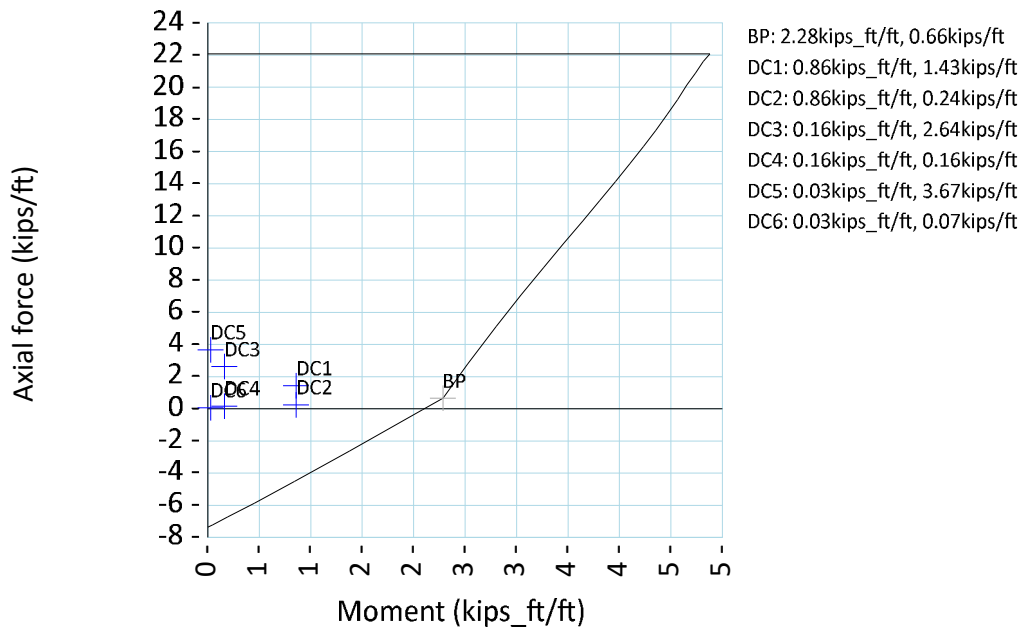
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Revision:	Date:	Sheet No. 122
Project Name: JKV-EMS		

Comp. stress in masonry due to flexure only;
Stress in reinforcement;
Stress at compression side of web;
Compression moment arm to face shell;
Compression in face shell;
Compression at balance point;
Force in tension face reinf. at balance point;
Axial load at balance point;
Moment at balance point;

$$f_b = F_s / n \times k_b \times d / (d - k_b \times d) = \mathbf{1125 \text{ psi}}$$
$$f_s = F_s = \mathbf{32000 \text{ psi}}$$
$$f_{b1} = (k_b \times d - t_{fs}) / (k_b \times d) \times f_b = \mathbf{-58 \text{ psi}}$$
$$x_1 = d - k_b \times d / 3 = \mathbf{3.416 \text{ in}}$$
$$C_1 = f_b / 2 \times k_b \times d = \mathbf{8.025 \text{ kips/ft}}$$
$$C_b = C_1 = \mathbf{8.025 \text{ kips/ft}}$$
$$T_b = f_s \times A_s = \mathbf{7.363 \text{ kips/ft}}$$
$$N_b = \min(C_1 - T_b, P_{a\_pos}) = \mathbf{0.662 \text{ kips/ft}}$$
$$M_b = \max(C_1 \times x_1, 0lb\_ft/ft) = \mathbf{2.285 \text{ kips\_ft/ft}}$$

Out-of-plane interaction diagram



In-plane axial resistance

In-plane axial force;
Depth to reinforcement
Net shear area
Slenderness ratio;
Allowable axial force - Eqn.8-19;

$$P_{in} = P \times L = \mathbf{0.5 \text{ kips}}$$
$$d_{in} = L - l_m / 4 = \mathbf{7.1 \text{ ft}}$$
$$A_{nv,in} = 2 \times d_{in} \times t_{fs} + d_{in} / s_{grout} \times (l_m / (n_{w,int} + 1)) \times (t - 2 \times t_{fs}) = \mathbf{429 \text{ in}^2}$$
$$h_{eff,x} / r_x = \mathbf{17.557}; < 99$$
$$P_{a,in\_pos} = ((0.25 \times f_m \times (A_n - A_s)) \times (1 - (h_{eff,x} / (140 \times r_x))^2) \times L) = \mathbf{283.58 \text{ kips}}$$

In-plane axial stress resistance

Slenderness ratio;
Allowable axial stress - cl.8.2.4.1;

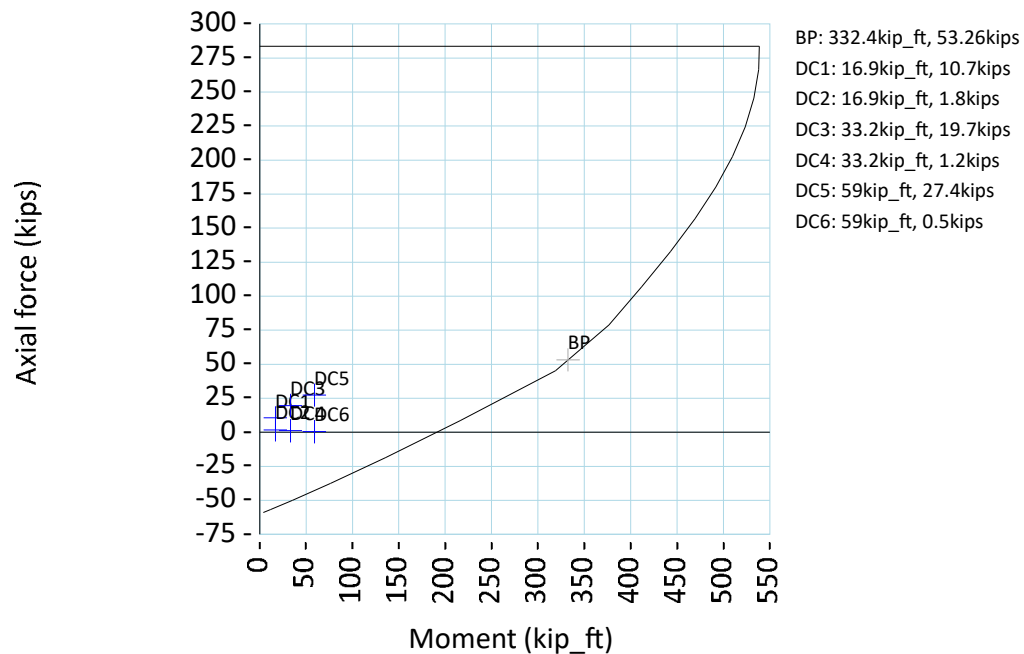
$$h_{eff,x} / r_x = \mathbf{17.557}; < 99$$
$$F_{a,in\_pos} = (1 / 4) \times f_m \times (1 - (h_{eff,x} / (140 \times r_x))^2) = \mathbf{615.2 \text{ psi}}$$



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Revision:	Date:	Sheet No. 123
Project Name: JKV-EMS		

In-plane interaction diagram



From interaction diagram

Axial load at balance point;
Moment at balance point;
Maximum moment;

$N_{b,in} = 53$ kips
 $M_{b,in} = 332$ kip_ft
 $M_{max,in} = 539$ kip_ft

Top: Max axial

Design forces

Axial load;
Axial eccentricity;
Out-of-plane moment;
Out-of-plane shear force;
In-plane moment;
In-plane Shear force;

$P = 1.433$ kips/ft
 $e = 0$ in
 $M_y = -0.857$ kips_ft/ft
 $V_y = 0.174$ kips/ft
 $M_x = 16.9$ kips_ft
 $V_x = 4.5$ kips

Reinforcement effective depth

Effective depth of reinforcement;

$d = 3.813$ in

Out-of-plane axial check

$P / P_{a,pos} = 0.065$

PASS - Pure axial resistance exceeds axial load

Out-of-plane axial stress check

Stress due to axial load;

$f_a = P / A = 21.3$ psi
 $f_a / F_{a,pos} = 0.060$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$M_{limit} = 2.250$ kips_ft/ft



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 124
Project Name: JKV-EMS		

Out-of-plane shear check

Net shear area

Applied shear stress;

Shear resistance of masonry - Eqn 8-26;

Grout factor - cl.8.3.5.1.2;

Overall shear resistance - cl.8.3.5.1.2;

$$\text{Abs}(M_y) / M_{\text{limit}} = \mathbf{0.381}$$

PASS - Design flexural strength exceeds factored bending moment

$$A_{nv} = d \times l_m / ((n_{w,int} + 1) \times s_{grout}) = \mathbf{22.34 \text{ in}^2/\text{ft}}$$

$$f_v = V_y / A_{nv} = \mathbf{7.79 \text{ psi}}$$

$$F_{vm} = 1/2 \times (4 - 1.75 \times \min(\text{Abs}(M_y) / (V_y \times d), 1)) \times \sqrt{(f_m \times 1 \text{ psi})} + 0.25 \times P / A_n = \mathbf{62.03 \text{ psi}}$$

$$\gamma_g = \mathbf{0.75}$$

$$F_v = \min(F_{vm}, 2 \times \sqrt{(f_m \times 1 \text{ psi})}) \times \gamma_g = \mathbf{46.52 \text{ psi}}$$

$$f_v / F_v = \mathbf{0.167}$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane axial check

$$P_{in} / P_{a,in\_pos} = \mathbf{0.038}$$

PASS - Pure axial resistance exceeds axial load

In-plane axial stress check

Stress due to axial load;

$$f_a = P / A = \mathbf{21.3 \text{ psi}}$$

$$f_a / F_{a,in\_pos} = \mathbf{0.035}$$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{\text{limit},in} = \mathbf{223 \text{ kip\_ft}}$$

$$\text{Abs}(M_x) / M_{\text{limit},in} = \mathbf{0.076}$$

PASS - Design flexural strength exceeds factored bending moment

In-plane shear check

Shear stress;

Allowable shear stress of masonry;

$$f_{v,in} = V_x / A_{nv,in} = \mathbf{10.49 \text{ psi}}$$

$$F_{vm,in} = 1/2 \times (4 - 1.75 \times \min(\text{Abs}(M_x) / (V_x \times d_{in}), 1)) \times \sqrt{(f_m \times 1 \text{ psi})} + 0.25 \times P_{in} / (A_n \times L) = \mathbf{82.78 \text{ psi}}$$

$$F_{vs,in} = 0.5 \times (A_v \times F_{sv} \times d_{in} / (A_{nv,in} \times d_{in} \times 1\text{ft})) = \mathbf{46.06 \text{ psi}}$$

$$\gamma_g = \mathbf{0.75}$$

$$F_{v,in} = \min(F_{vm,in} + F_{vs,in}, 2.63 \times \sqrt{(f_m \times 1 \text{ psi})}) \times \gamma_g = \mathbf{96.63 \text{ psi}}$$

$$f_{v,in} / F_{v,in} = \mathbf{0.109}$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane shear friction - 8.3.6

Coefficient of friction

Shear friction stress ($M_x / (V_x \times d_{in})$ lte 0.5)

Shear friction stress ($M_x / (V_x \times d_{in})$ gte 1)

Shear friction stress (interpolate F_{f1} and F_{f2})

$$\mu = \mathbf{0.7}$$

$$F_{f1} = \max(\mu \times (A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = \mathbf{103.3 \text{ psi}}$$

$$F_{f2} = \max(0.65 \times (0.6 \times A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = \mathbf{64.1 \text{ psi}}$$

$$F_f = \mathbf{101.3 \text{ psi}}$$

$$f_{v,in} / F_f = \mathbf{0.104}$$

PASS - Allowable shear friction stress exceeds applied shear stress

Biaxial bending

$$\sqrt{((\text{abs}(M_y) / M_{\text{limit}})^2 + (\text{abs}(M_x) / M_{\text{limit},in})^2)} = \mathbf{0.388}$$

PASS - Biaxial bending utilization is acceptable



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Revision:	Date:	Sheet No. 125
Project Name: JKV-EMS		

Top: Min axial

Design forces

Axial load; $P = 0.241$ kips/ft
Axial eccentricity; $e = 0$ in
Out-of-plane moment; $M_y = -0.857$ kips_ft/ft
Out-of-plane shear force; $V_y = 0.174$ kips/ft
In-plane moment; $M_x = 16.9$ kips_ft
In-plane Shear force; $V_x = 4.5$ kips

Reinforcement effective depth

Effective depth of reinforcement; $d = 3.813$ in

Out-of-plane axial check

$$P / P_{a\_pos} = 0.011$$

PASS - Pure axial resistance exceeds axial load

Out-of-plane axial stress check

Stress due to axial load; $f_a = P / A = 3.6$ psi
 $f_a / F_{a\_pos} = 0.010$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit} = 2.097 \text{ kips\_ft/ft}$$
$$Abs(M_y) / M_{limit} = 0.409$$

PASS - Design flexural strength exceeds factored bending moment

Out-of-plane shear check

Net shear area $A_{nv} = d \times l_m / ((n_{w,int} + 1) \times s_{grout}) = 22.34$ in²/ft
Applied shear stress; $f_v = V_y / A_{nv} = 7.79$ psi
Shear resistance of masonry - Eqn 8-26;
 $F_{vm} = 1/2 \times (4 - 1.75 \times \min(Abs(M_y) / (V_y \times d), 1)) \times \sqrt{(f'_m \times 1 \text{ psi})} + 0.25 \times P / A_n = 57.22$ psi
Grout factor - cl.8.3.5.1.2; $\gamma_g = 0.75$
Overall shear resistance - cl.8.3.5.1.2; $F_v = \min(F_{vm}, 2 \times \sqrt{(f'_m \times 1 \text{ psi})}) \times \gamma_g = 42.92$ psi
 $f_v / F_v = 0.182$

PASS - Allowable shear stress exceeds applied shear stress

In-plane axial check

$$P_{in} / P_{a,in\_pos} = 0.006$$

PASS - Pure axial resistance exceeds axial load

In-plane axial stress check

Stress due to axial load; $f_a = P / A = 3.6$ psi
 $f_a / F_{a,in\_pos} = 0.006$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit,in} = 196 \text{ kip\_ft}$$
$$Abs(M_x) / M_{limit,in} = 0.086$$

PASS - Design flexural strength exceeds factored bending moment



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 126
Project Name: JKV-EMS		

In-plane shear check

Shear stress;

Allowable shear stress of masonry;

Allowable shear stress of steel;

Grout factor;

Allowable shear stress;

$$f_{v,in} = V_x / A_{nv,in} = \mathbf{10.49 \text{ psi}}$$

$$F_{vm,in} = 1/2 \times (4 - 1.75 \times \min(\text{Abs}(M_x) / (V_x \times d_{in}), 1)) \times \sqrt{f'_m \times 1 \text{ psi}} + 0.25 \times P_{in} / (A_n \times L) = \mathbf{77.97 \text{ psi}}$$

$$F_{vs,in} = 0.5 \times (A_v \times F_{sv} \times d_{in} / (A_{nv,in} \times d_{in} \times 1\text{ft})) = \mathbf{46.06 \text{ psi}}$$

$$\gamma_g = \mathbf{0.75}$$

$$F_{v,in} = \min(F_{vm,in} + F_{vs,in}, 2.63 \times \sqrt{f'_m \times 1 \text{ psi}}) \times \gamma_g = \mathbf{93.02 \text{ psi}}$$

$$f_{v,in} / F_{v,in} = \mathbf{0.113}$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane shear friction - 8.3.6

Coefficient of friction

Shear friction stress ($M_x / (V_x / d_{in})$ lte 0.5)

Shear friction stress ($M_x / (V_x / d_{in})$ gte 1)

Shear friction stress (interpolate F_{f1} and F_{f2})

$$\mu = \mathbf{0.7}$$

$$F_{f1} = \max(\mu \times (A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = \mathbf{88.8 \text{ psi}}$$

$$F_{f2} = \max(0.65 \times (0.6 \times A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = \mathbf{50.6 \text{ psi}}$$

$$F_f = \mathbf{86.8 \text{ psi}}$$

$$f_{v,in} / F_f = \mathbf{0.121}$$

PASS - Allowable shear friction stress exceeds applied shear stress

Biaxial bending

$$\sqrt{(\text{abs}(M_y) / M_{limit})^2 + (\text{abs}(M_x) / M_{limit,in})^2} = \mathbf{0.418}$$

PASS - Biaxial bending utilization is acceptable

Mid-fifth: Max axial

Design forces

Axial load;

Axial eccentricity;

Out-of-plane moment;

Out-of-plane shear force;

In-plane moment;

In-plane Shear force;

$$P = \mathbf{2.638 \text{ kips/ft}}$$

$$e = \mathbf{0 \text{ in}}$$

$$M_y = \mathbf{-0.161 \text{ kips\_ft/ft}}$$

$$V_y = \mathbf{0.040 \text{ kips/ft}}$$

$$M_x = \mathbf{33.2 \text{ kips\_ft}}$$

$$V_x = \mathbf{4.3 \text{ kips}}$$

Reinforcement effective depth

Effective depth of reinforcement;

$$d = \mathbf{3.813 \text{ in}}$$

Out-of-plane axial check

$$P / P_{a,pos} = \mathbf{0.119}$$

PASS - Pure axial resistance exceeds axial load

Out-of-plane axial stress check

Stress due to axial load;

$$f_a = P / A = \mathbf{39.2 \text{ psi}}$$

$$f_a / F_{a,pos} = \mathbf{0.110}$$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit} = \mathbf{2.548 \text{ kips\_ft/ft}}$$

$$\text{Abs}(M_y) / M_{limit} = \mathbf{0.063}$$

PASS - Design flexural strength exceeds factored bending moment



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Revision:	Date:	Sheet No. 127
Project Name: JKV-EMS		

Out-of-plane shear check

Net shear area

Applied shear stress;

Shear resistance of masonry - Eqn 8-26;

Grout factor - cl.8.3.5.1.2;

Overall shear resistance - cl.8.3.5.1.2;

$$A_{nv} = d \times l_m / ((n_{w,int} + 1) \times s_{grout}) = \mathbf{22.34 \text{ in}^2/\text{ft}}$$

$$f_v = V_y / A_{nv} = \mathbf{1.8 \text{ psi}}$$

$$F_{vm} = 1/2 \times (4 - 1.75 \times \min(\text{Abs}(M_y) / (V_y \times d), 1)) \times \sqrt{(f'_m \times 1 \text{ psi}) + 0.25 \times P} / A_n = \mathbf{66.89 \text{ psi}}$$

$$\gamma_g = \mathbf{0.75}$$

$$F_v = \min(F_{vm}, 2 \times \sqrt{(f'_m \times 1 \text{ psi})}) \times \gamma_g = \mathbf{50.17 \text{ psi}}$$

$$f_v / F_v = \mathbf{0.036}$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane axial check

$$P_{in} / P_{a,in\_pos} = \mathbf{0.069}$$

PASS - Pure axial resistance exceeds axial load

In-plane axial stress check

Stress due to axial load;

$$f_a = P / A = \mathbf{39.2 \text{ psi}}$$

$$f_a / F_{a,in\_pos} = \mathbf{0.064}$$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit,in} = \mathbf{248 \text{ kip\_ft}}$$

$$\text{Abs}(M_x) / M_{limit,in} = \mathbf{0.134}$$

PASS - Design flexural strength exceeds factored bending moment

In-plane shear check

Shear stress;

Allowable shear stress of masonry;

$$f_{v,in} = V_x / A_{nv,in} = \mathbf{10.03 \text{ psi}}$$

$$F_{vm,in} = 1/2 \times (4 - 1.75 \times \min(\text{Abs}(M_x) / (V_x \times d_{in}), 1)) \times \sqrt{(f'_m \times 1 \text{ psi}) + 0.25 \times P_{in} / (A_n \times L)} = \mathbf{66.89 \text{ psi}}$$

$$F_{vs,in} = 0.5 \times (A_v \times F_{sv} \times d_{in} / (A_{nv,in} / d_{in} \times 1\text{ft})) = \mathbf{46.06 \text{ psi}}$$

$$\gamma_g = \mathbf{0.75}$$

$$F_{v,in} = \min(F_{vm,in} + F_{vs,in}, 2 \times \sqrt{(f'_m \times 1 \text{ psi})}) \times \gamma_g = \mathbf{75 \text{ psi}}$$

$$f_{v,in} / F_{v,in} = \mathbf{0.134}$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane shear friction - 8.3.6

Coefficient of friction

$$\mu = \mathbf{0.7}$$

Shear friction stress ($M_x / (V_x / d_{in})$ lte 0.5)

$$F_{f1} = \max(\mu \times (A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = \mathbf{118 \text{ psi}}$$

Shear friction stress ($M_x / (V_x / d_{in})$ gte 1)

$$F_{f2} = \max(0.65 \times (0.6 \times A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = \mathbf{77.7 \text{ psi}}$$

Shear friction stress as $M_x / (V_x \times d_{in})$ gte 1.0

$$F_f = F_{f2} = \mathbf{77.7 \text{ psi}}$$

$$f_{v,in} / F_f = \mathbf{0.129}$$

PASS - Allowable shear friction stress exceeds applied shear stress

Biaxial bending

$$\sqrt{((\text{abs}(M_y) / M_{limit})^2 + (\text{abs}(M_x) / M_{limit,in})^2)} = \mathbf{0.148}$$

PASS - Biaxial bending utilization is acceptable



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Revision:	Date:	Sheet No. 128
Project Name: JKV-EMS		

Mid-fifth: Min axial

Design forces

Axial load; $P = 0.161$ kips/ft
Axial eccentricity; $e = 0$ in
Out-of-plane moment; $M_y = -0.161$ kips_ft/ft
Out-of-plane shear force; $V_y = 0.040$ kips/ft
In-plane moment; $M_x = 33.2$ kips_ft
In-plane Shear force; $V_x = 4.3$ kips

Reinforcement effective depth

Effective depth of reinforcement; $d = 3.813$ in

Out-of-plane axial check

$$P / P_{a\_pos} = 0.007$$

PASS - Pure axial resistance exceeds axial load

Out-of-plane axial stress check

Stress due to axial load; $f_a = P / A = 2.4$ psi
 $f_a / F_{a\_pos} = 0.007$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit} = 2.097 \text{ kips\_ft/ft}$$

$$Abs(M_y) / M_{limit} = 0.077$$

PASS - Design flexural strength exceeds factored bending moment

Out-of-plane shear check

Net shear area $A_{nv} = d \times l_m / ((n_{w,int} + 1) \times s_{grout}) = 22.34 \text{ in}^2/\text{ft}$
Applied shear stress; $f_v = V_y / A_{nv} = 1.8$ psi
Shear resistance of masonry - Eqn 8-26;
 $F_{vm} = 1/2 \times (4 - 1.75 \times \min(Abs(M_y) / (V_y \times d), 1)) \times \sqrt{f'_m \times 1 \text{ psi}} + 0.25 \times P / A_n = 56.9$ psi
Grout factor - cl.8.3.5.1.2; $\gamma_g = 0.75$
Overall shear resistance - cl.8.3.5.1.2; $F_v = \min(F_{vm}, 2 \times \sqrt{f'_m \times 1 \text{ psi}}) \times \gamma_g = 42.67$ psi
 $f_v / F_v = 0.042$

PASS - Allowable shear stress exceeds applied shear stress

In-plane axial check

$$P_{in} / P_{a,in\_pos} = 0.004$$

PASS - Pure axial resistance exceeds axial load

In-plane axial stress check

Stress due to axial load; $f_a = P / A = 2.4$ psi
 $f_a / F_{a,in\_pos} = 0.004$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit,in} = 193 \text{ kip\_ft}$$

$$Abs(M_x) / M_{limit,in} = 0.172$$

PASS - Design flexural strength exceeds factored bending moment



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 129
Project Name: JKV-EMS		

In-plane shear check

Shear stress;

Allowable shear stress of masonry;

Allowable shear stress of steel;

Grout factor;

Allowable shear stress;

$$f_{v,in} = V_x / A_{nv,in} = \mathbf{10.03 \text{ psi}}$$

$$F_{vm,in} = 1/2 \times (4 - 1.75 \times \min(\text{Abs}(M_x) / (V_x \times d_{in}), 1)) \times \sqrt{(f'_m \times 1 \text{ psi}) + 0.25 \times P_{in} / (A_n \times L)} = \mathbf{56.9 \text{ psi}}$$

$$F_{vs,in} = 0.5 \times (A_v \times F_{sv} \times d_{in} / (A_{nv,in} \times d_{in} \times 1\text{ft})) = \mathbf{46.06 \text{ psi}}$$

$$\gamma_g = \mathbf{0.75}$$

$$F_{v,in} = \min(F_{vm,in} + F_{vs,in}, 2 \times \sqrt{(f'_m \times 1 \text{ psi})}) \times \gamma_g = \mathbf{75 \text{ psi}}$$

$$f_{v,in} / F_{v,in} = \mathbf{0.134}$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane shear friction - 8.3.6

Coefficient of friction

Shear friction stress ($M_x / (V_x \times d_{in})$ lte 0.5)

Shear friction stress ($M_x / (V_x \times d_{in})$ gte 1)

Shear friction stress as $M_x / (V_x \times d_{in})$ gte 1.0

$$\mu = \mathbf{0.7}$$

$$F_{f1} = \max(\mu \times (A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = \mathbf{87.8 \text{ psi}}$$

$$F_{f2} = \max(0.65 \times (0.6 \times A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = \mathbf{49.7 \text{ psi}}$$

$$F_f = F_{f2} = \mathbf{49.7 \text{ psi}}$$

$$f_{v,in} / F_f = \mathbf{0.202}$$

PASS - Allowable shear friction stress exceeds applied shear stress

Biaxial bending

$$\sqrt{((\text{abs}(M_y) / M_{limit})^2 + (\text{abs}(M_x) / M_{limit,in})^2)} = \mathbf{0.188}$$

PASS - Biaxial bending utilization is acceptable

Bottom: Max axial

Design forces

Axial load;

Axial eccentricity;

Out-of-plane moment;

Out-of-plane shear force;

In-plane moment;

In-plane Shear force;

$$P = \mathbf{3.669 \text{ kips/ft}}$$

$$e = \mathbf{0 \text{ in}}$$

$$M_y = \mathbf{0.027 \text{ kips\_ft/ft}}$$

$$V_y = \mathbf{0.027 \text{ kips/ft}}$$

$$M_x = \mathbf{59 \text{ kips\_ft}}$$

$$V_x = \mathbf{4.4 \text{ kips}}$$

Reinforcement effective depth

Effective depth of reinforcement;

$$d = \mathbf{3.813 \text{ in}}$$

Out-of-plane axial check

$$P / P_{a,pos} = \mathbf{0.166}$$

PASS - Pure axial resistance exceeds axial load

Out-of-plane axial stress check

Stress due to axial load;

$$f_a = P / A = \mathbf{54.6 \text{ psi}}$$

$$f_a / F_{a,pos} = \mathbf{0.152}$$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit} = \mathbf{2.667 \text{ kips\_ft/ft}}$$

$$\text{Abs}(M_y) / M_{limit} = \mathbf{0.010}$$

PASS - Design flexural strength exceeds factored bending moment



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 130
Project Name: JKV-EMS		

Out-of-plane shear check

Net shear area

Applied shear stress;

Shear resistance of masonry - Eqn 8-26;

Grout factor - cl.8.3.5.1.2;

Overall shear resistance - cl.8.3.5.1.2;

$$A_{nv} = d \times l_m / ((n_{w,int} + 1) \times s_{grout}) = \mathbf{22.34 \text{ in}^2/\text{ft}}$$

$$f_v = V_y / A_{nv} = \mathbf{1.2 \text{ psi}}$$

$$F_{vm} = 1/2 \times (4 - 1.75 \times \min(\text{Abs}(M_y) / (V_y \times d), 1)) \times \sqrt{(f'_m \times 1 \text{ psi}) + 0.25 \times P} / A_n = \mathbf{71.05 \text{ psi}}$$

$$\gamma_g = \mathbf{0.75}$$

$$F_v = \min(F_{vm}, 2 \times \sqrt{(f'_m \times 1 \text{ psi})}) \times \gamma_g = \mathbf{53.29 \text{ psi}}$$

$$f_v / F_v = \mathbf{0.022}$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane axial check

$$P_{in} / P_{a,in\_pos} = \mathbf{0.097}$$

PASS - Pure axial resistance exceeds axial load

In-plane axial stress check

Stress due to axial load;

$$f_a = P / A = \mathbf{54.6 \text{ psi}}$$

$$f_a / F_{a,in\_pos} = \mathbf{0.089}$$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit,in} = \mathbf{268 \text{ kip\_ft}}$$

$$\text{Abs}(M_x) / M_{limit,in} = \mathbf{0.221}$$

PASS - Design flexural strength exceeds factored bending moment

In-plane shear check

Shear stress;

Allowable shear stress of masonry;

$$f_{v,in} = V_x / A_{nv,in} = \mathbf{10.26 \text{ psi}}$$

$$F_{vm,in} = 1/2 \times (4 - 1.75 \times \min(\text{Abs}(M_x) / (V_x \times d_{in}), 1)) \times \sqrt{(f'_m \times 1 \text{ psi}) + 0.25 \times P_{in} / (A_n \times L)} = \mathbf{71.05 \text{ psi}}$$

$$F_{vs,in} = 0.5 \times (A_v \times F_{sv} \times d_{in} / (A_{nv,in} / d_{in} \times 1\text{ft})) = \mathbf{46.06 \text{ psi}}$$

$$\gamma_g = \mathbf{0.75}$$

$$F_{v,in} = \min(F_{vm,in} + F_{vs,in}, 2 \times \sqrt{(f'_m \times 1 \text{ psi})}) \times \gamma_g = \mathbf{75 \text{ psi}}$$

$$f_{v,in} / F_{v,in} = \mathbf{0.137}$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane shear friction - 8.3.6

Coefficient of friction

$$\mu = \mathbf{0.7}$$

Shear friction stress ($M_x / (V_x / d_{in})$ lte 0.5)

$$F_{f1} = \max(\mu \times (A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = \mathbf{130.6 \text{ psi}}$$

Shear friction stress ($M_x / (V_x / d_{in})$ gte 1)

$$F_{f2} = \max(0.65 \times (0.6 \times A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = \mathbf{89.4 \text{ psi}}$$

Shear friction stress as $M_x / (V_x \times d_{in})$ gte 1.0

$$F_f = F_{f2} = \mathbf{89.4 \text{ psi}}$$

$$f_{v,in} / F_f = \mathbf{0.115}$$

PASS - Allowable shear friction stress exceeds applied shear stress

Biaxial bending

$$\sqrt{((\text{abs}(M_y) / M_{limit})^2 + (\text{abs}(M_x) / M_{limit,in})^2)} = \mathbf{0.221}$$

PASS - Biaxial bending utilization is acceptable



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 131
Project Name: JKV-EMS		

Bottom: Min axial

Design forces

Axial load; $P = 0.067$ kips/ft
Axial eccentricity; $e = 0$ in
Out-of-plane moment; $M_y = 0.027$ kips_ft/ft
Out-of-plane shear force; $V_y = 0.027$ kips/ft
In-plane moment; $M_x = 59$ kips_ft
In-plane Shear force; $V_x = 4.4$ kips

Reinforcement effective depth

Effective depth of reinforcement; $d = 3.813$ in

Out-of-plane axial check

$$P / P_{a\_pos} = 0.003$$

PASS - Pure axial resistance exceeds axial load

Out-of-plane axial stress check

Stress due to axial load; $f_a = P / A = 1.0$ psi
 $f_a / F_{a\_pos} = 0.003$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit} = 2.097 \text{ kips\_ft/ft}$$

$$Abs(M_y) / M_{limit} = 0.013$$

PASS - Design flexural strength exceeds factored bending moment

Out-of-plane shear check

Net shear area $A_{nv} = d \times l_m / ((n_{w,int} + 1) \times s_{grout}) = 22.34 \text{ in}^2/\text{ft}$
Applied shear stress; $f_v = V_y / A_{nv} = 1.2$ psi
Shear resistance of masonry - Eqn 8-26;
 $F_{vm} = 1/2 \times (4 - 1.75 \times \min(Abs(M_y) / (V_y \times d), 1)) \times \sqrt{(f'_m \times 1 \text{ psi})} + 0.25 \times P / A_n = 56.52$ psi
 $\gamma_g = 0.75$
Grout factor - cl.8.3.5.1.2;
Overall shear resistance - cl.8.3.5.1.2;
 $F_v = \min(F_{vm}, 2 \times \sqrt{(f'_m \times 1 \text{ psi})}) \times \gamma_g = 42.39$ psi
 $f_v / F_v = 0.028$

PASS - Allowable shear stress exceeds applied shear stress

In-plane axial check

$$P_{in} / P_{a,in\_pos} = 0.002$$

PASS - Pure axial resistance exceeds axial load

In-plane axial stress check

Stress due to axial load; $f_a = P / A = 1.0$ psi
 $f_a / F_{a,in\_pos} = 0.002$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit,in} = 193 \text{ kip\_ft}$$

$$Abs(M_x) / M_{limit,in} = 0.305$$

PASS - Design flexural strength exceeds factored bending moment



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 132
Project Name: JKV-EMS		

In-plane shear check

Shear stress;

Allowable shear stress of masonry;

Allowable shear stress of steel;

Grout factor;

Allowable shear stress;

$$f_{v,in} = V_x / A_{nv,in} = \mathbf{10.26 \text{ psi}}$$

$$F_{vm,in} = 1/2 \times (4 - 1.75 \times \min(\text{Abs}(M_x) / (V_x \times d_{in}), 1)) \times \sqrt{(f'_m \times 1 \text{ psi}) + 0.25 \times P_{in} / (A_n \times L)} = \mathbf{56.52 \text{ psi}}$$

$$F_{vs,in} = 0.5 \times (A_v \times F_{sv} \times d_{in} / (A_{nv,in} \times d_{in} \times 1\text{ft})) = \mathbf{46.06 \text{ psi}}$$

$$\gamma_g = \mathbf{0.75}$$

$$F_{v,in} = \min(F_{vm,in} + F_{vs,in}, 2 \times \sqrt{(f'_m \times 1 \text{ psi})}) \times \gamma_g = \mathbf{75 \text{ psi}}$$

$$f_{v,in} / F_{v,in} = \mathbf{0.137}$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane shear friction - 8.3.6

Coefficient of friction

Shear friction stress ($M_x / (V_x \times d_{in})$ lte 0.5)

Shear friction stress ($M_x / (V_x \times d_{in})$ gte 1)

Shear friction stress as $M_x / (V_x \times d_{in})$ gte 1.0

$$\mu = \mathbf{0.7}$$

$$F_{f1} = \max(\mu \times (A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = \mathbf{86.7 \text{ psi}}$$

$$F_{f2} = \max(0.65 \times (0.6 \times A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = \mathbf{48.6 \text{ psi}}$$

$$F_f = F_{f2} = \mathbf{48.6 \text{ psi}}$$

$$f_{v,in} / F_f = \mathbf{0.211}$$

PASS - Allowable shear friction stress exceeds applied shear stress

Biaxial bending

$$\sqrt{(\text{abs}(M_y) / M_{limit})^2 + (\text{abs}(M_x) / M_{limit,in})^2} = \mathbf{0.305}$$

PASS - Biaxial bending utilization is acceptable

MASONRY WALL SECTION DESIGN - WP18

In accordance with TMS-16 using the allowable stress design method

Tedds calculation version 1.0.06

Design summary

Overall design status;

PASS

Overall design utilization;

0.073

Top: Max axial

Description	Unit	Provided	Required	Utilization	Result
Out-of-plane axial force	kips/ft	22.1	1.1	0.052	PASS
Out-of-plane axial stress	psi	357.8	17.0	0.048	PASS
Out-of-plane flexural strength	kip_ft/ft	2.250	0.025	0.011	PASS
Out-of-plane shear stress	psi	45.7	0.9	0.020	PASS
In-plane axial force	kips	1861.8	55.3	0.030	PASS
In-plane axial stress	psi	624.8	17.0	0.027	PASS
In-plane flexural strength	kip_ft	8449.7	89.5	0.011	PASS
In-plane shear stress	psi	108.4	1.9	0.018	PASS
In-plane shear friction	psi	99.3	1.9	0.020	PASS
Biaxial bending				0.015	PASS



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 133
Project Name: JKV-EMS		

Top: Min axial

Description	Unit	Provided	Required	Utilization	Result
Out-of-plane axial force	kips/ft	22.1	0.1	0.005	PASS
Out-of-plane axial stress	psi	357.8	1.6	0.004	PASS
Out-of-plane flexural strength	kip_ft/ft	2.097	0.025	0.012	PASS
Out-of-plane shear stress	psi	42.5	0.9	0.022	PASS
In-plane axial force	kips	1861.8	5.2	0.003	PASS
In-plane axial stress	psi	624.8	1.6	0.003	PASS
In-plane flexural strength	kip_ft	7502.4	89.5	0.012	PASS
In-plane shear stress	psi	108.4	1.9	0.018	PASS
In-plane shear friction	psi	87.1	1.9	0.022	PASS
Biaxial bending				0.017	PASS

Mid-fifth: Max axial

Description	Unit	Provided	Required	Utilization	Result
Out-of-plane axial force	kips/ft	22.1	1.6	0.073	PASS
Out-of-plane axial stress	psi	357.8	23.8	0.067	PASS
Out-of-plane flexural strength	kip_ft/ft	2.403	0.031	0.013	PASS
Out-of-plane shear stress	psi	47.0	0.4	0.008	PASS
In-plane axial force	kips	1861.8	77.4	0.042	PASS
In-plane axial stress	psi	624.8	23.8	0.038	PASS
In-plane flexural strength	kip_ft	8866.6	185.1	0.021	PASS
In-plane shear stress	psi	86.3	1.7	0.020	PASS
In-plane shear friction	psi	83.3	1.7	0.021	PASS
Biaxial bending				0.025	PASS

Mid-fifth: Min axial

Description	Unit	Provided	Required	Utilization	Result
Out-of-plane axial force	kips/ft	22.1	0.3	0.014	PASS
Out-of-plane axial stress	psi	357.8	4.6	0.013	PASS
Out-of-plane flexural strength	kip_ft/ft	2.097	0.031	0.015	PASS
Out-of-plane shear stress	psi	43.1	0.4	0.009	PASS
In-plane axial force	kips	1861.8	14.8	0.008	PASS
In-plane axial stress	psi	624.8	4.6	0.007	PASS
In-plane flexural strength	kip_ft	7691.9	185.1	0.024	PASS
In-plane shear stress	psi	86.3	1.7	0.020	PASS
In-plane shear friction	psi	68.6	1.7	0.025	PASS
Biaxial bending				0.028	PASS



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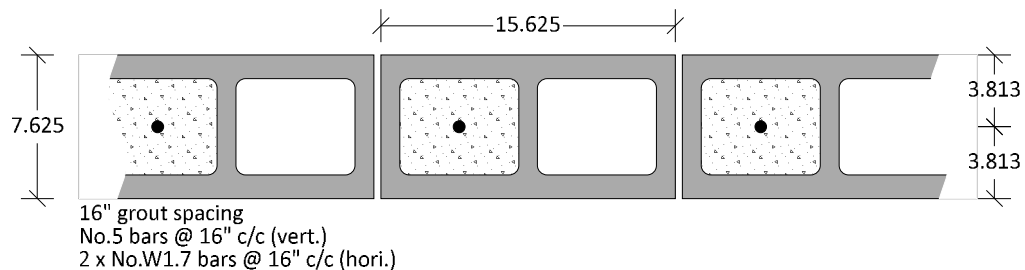
Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 134
Project Name: JKV-EMS		

Bottom: Max axial

Description	Unit	Provided	Required	Utilization	Result
Out-of-plane axial force	kips/ft	22.1	1.6	0.072	PASS
Out-of-plane axial stress	psi	357.8	23.7	0.066	PASS
Out-of-plane flexural strength	kip_ft/ft	2.403	0.006	0.003	PASS
Out-of-plane shear stress	psi	47.0	0.4	0.008	PASS
In-plane axial force	kips	1861.8	77.0	0.041	PASS
In-plane axial stress	psi	624.8	23.7	0.038	PASS
In-plane flexural strength	kip_ft	8866.6	307.7	0.035	PASS
In-plane shear stress	psi	75.0	1.9	0.025	PASS
In-plane shear friction	psi	65.2	1.9	0.029	PASS
Biaxial bending				0.035	PASS

Bottom: Min axial

Description	Unit	Provided	Required	Utilization	Result
Out-of-plane axial force	kips/ft	22.1	0.4	0.016	PASS
Out-of-plane axial stress	psi	357.8	5.4	0.015	PASS
Out-of-plane flexural strength	kip_ft/ft	2.097	0.006	0.003	PASS
Out-of-plane shear stress	psi	43.3	0.4	0.009	PASS
In-plane axial force	kips	1861.8	17.4	0.009	PASS
In-plane axial stress	psi	624.8	5.4	0.009	PASS
In-plane flexural strength	kip_ft	7729.8	307.7	0.040	PASS
In-plane shear stress	psi	75.0	1.9	0.025	PASS
In-plane shear friction	psi	51.8	1.9	0.037	PASS
Biaxial bending				0.040	PASS



Masonry details

8 in CMU in running bond, face bedded with S class PCL mortar on an unfinished bearing surface

Length of section;

$$L = 48.281 \text{ ft}$$

Effective height (minor axis);

$$h_{\text{eff},y} = 18.376 \text{ ft}$$

Effective height (major axis);

$$h_{\text{eff},x} = 37.854 \text{ ft}$$

Compressive strength of unit;

$$f_{cu} = 3250 \text{ psi}$$

Density of masonry units;

$$\rho_m = 130 \text{ lb/ft}^3$$



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 135
Project Name: JKV-EMS		

Thickness of masonry units; $t = 7.625$ in
Height of masonry units; $h_m = 7.625$ in
Length of masonry units; $l_m = 15.625$ in
Number of internal webs; $n_{w,int} = 1$
Internal web thickness; $t_{w,int} = 1$ in
Number of end webs; $n_{w,ext} = 2$
End web thickness; $t_{w,ext} = 1$ in
Face shell thickness; $t_{fs} = 1.25$ in
Depth of cavity; $t_c = t - 2 \times t_{fs} = 5.125$ in
Length of cavity; $l_c = (l_m - n_{w,int} \times t_{w,int} - n_{w,ext} \times t_{w,ext}) / 2 = 6.313$ in
Area of cavity; $A_c = t_c \times l_c = 32.4$ in²
Spacing of grout; $s_{grout} = 16$ in
Density of grout; $\rho_{grout} = 140$ lb/ft³

Compressive strength of masonry

Net compr. strength of masonry - TMS602 Table 2; $f'_m = 2500$ psi
Modulus of elasticity for masonry - Table 4.2.2; $E_m = 900 \times f'_m = 2250000$ psi
Shear modulus of masonry; $G_m = E_m / (2 \times (1 + \nu_m)) = 900000$ psi
Max usable comp. strain in masonry; $\epsilon_{mu} = 0.00250$

Reinforcement details

Allow. tensile stress in vert. reinf. - cl.8.3.3 $F_s = 32000$ psi
Yield strength of reinforcement $f_{y,vert} = 60000$ psi
Modulus of elasticity for reinforcement $E_s = 29000000$ psi
Tensile strain in reinforcement at balance point; $\epsilon_{ty} = f_{y,vert} / E_s = 0.002069$
Vertical reinforcement provided No.5 bars @ 16" c/c
Area of vertical reinforcement $A_s = \pi \times \phi_{vert}^2 / (4 \times s_{vert}) = 0.23$ in²/ft
Max. allow. vertical reinf. diameter - cl.6.1.2.2/5 $\phi_{vert,max} = \min(\min(t, l_m) / 8, \min(t_c, l_c) / 2) = 0.95$ in

PASS - Maximum allowable vertical bar diameter exceeds provided

Nominal front cover; $C_{nom,front} = 3.500$ in

Neither face exposed to earth or weather

Minimum rear cover - cl.6.1.3.5/6.1.4.1; $C_{nom,min,rear} = \max(t_{grout,min} + t_{fs}, 1.5 \text{ in}) = 1.750$ in

Minimum front cover; $C_{nom,min,front} = \max(t_{grout,min} + t_{fs}, 1.5 \text{ in}) = 1.750$ in

PASS - Cover to reinforcement exceeds minimum cover required

Min allow. clear spacing between grout and unit; $t_{grout,min} = 0.500$ in
Clear spacing between grout and unit; $s_{vert,clear} = \min(t - (\phi_{vert} + C_{nom,front}), C_{nom,front} - t_{fs}) = 2.250$ in

PASS - Clear spacing between reinf. and masonry unit exceeds minimum allowable

Min req'd vert reinf in shear walls - cl.9.3.6.2; $A_{s,h,min} = A_v / 3 = 0.009$ in²/ft

PASS - Area of vertical reinforcement exceeds minimum required

Max allow vert spacing in shear walls - cl.9.3.6.2; $s_{vert,h,max} = 96.000$ in

PASS - Maximum allowable vertical reinforcement spacing exceeds provided

Allow. tensile stress in hori. reinf. - cl.8.3.3 $F_{sv} = 30000$ psi
Yield strength of horizontal reinforcement $f_{y,hori} = 70000$ psi
Horizontal reinforcement provided 2 x No.3.7592 wires @ 16" c/c
Area of horizontal reinforcement $A_v = 2 \times \pi \times \phi_{hori}^2 / (4 \times s_{hori}) = 0.03$ in²/ft
Max. allow. horizontal reinf. diameter - cl.6.1.2.2/5 $\phi_{hori,max} = t_j / 2 = 0.19$ in



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 136
Project Name: JKV-EMS		

Area of cell;
Area of reinforcement in cell;
Max allow. reinforcement in cell - cl.6.1.2.4;

PASS - Maximum allowable horizontal bar diameter exceeds provided

$$A_c = t_c \times l_c = \mathbf{32.352 \text{ in}^2}$$

$$A_{s, \text{cell}} = n_{\text{vert}} \times \phi_{\text{vert}}^2 \times \pi / 4 = \mathbf{0.307 \text{ in}^2}$$

$$A_{s, \text{max, cell}} = 0.06 \times A_c = \mathbf{1.941 \text{ in}^2}$$

PASS - Maximum allowable area of reinforcement in cell exceeds provided

Masonry section properties

Properties for walls loaded axially and out of plane

Gross cross-sectional area;	$A = t - l_c \times t_c / (l_m + t_j) = \mathbf{67.2 \text{ in}^2/\text{ft}}$
Gross moment of inertia;	$I = t^3 / 12 - l_c \times t_c^3 / (12 \times (l_m + t_j)) = \mathbf{390.2 \text{ in}^4/\text{ft}}$
Gross section modulus;	$S = 2 \times I / t = \mathbf{102.4 \text{ in}^3/\text{ft}}$
Gross radius of gyration;	$r = \sqrt{I / A} = \mathbf{2.4 \text{ in}}$
Net cross-sectional area;	$A_n = 2 \times t_{fs} + t_c \times (l_c + 2 \times t_{w, \text{ext}}) / (l_m + t_j) = \mathbf{62 \text{ in}^2/\text{ft}}$
Net moment of inertia;	$I_n = (t^3 - t_c^3) / 12 + ((l_c + 2 \times t_{w, \text{ext}}) \times t_c^3 / (12 \times (l_m + t_j))) = \mathbf{378.6 \text{ in}^4/\text{ft}}$
Net section modulus;	$S_n = 2 \times I_n / t = \mathbf{99.3 \text{ in}^3/\text{ft}}$
Net radius of gyration;	$r_n = \sqrt{I_n / A_n} = \mathbf{2.5 \text{ in}}$

Properties for walls loaded in-plane

Equivalent thickness;	$t_{\text{equiv}} = A_n = \mathbf{5.163 \text{ in}}$
Net moment of inertia;	$I_{x, \text{net}} = t_{\text{equiv}} \times L^3 / 12 = \mathbf{83669329 \text{ in}^4}$
Net section modulus;	$S_{x, \text{net}} = I_{x, \text{net}} / (L / 2) = \mathbf{288826 \text{ in}^3}$
Net radius of gyration;	$r_x = \sqrt{I_{x, \text{net}} / (A_n \times L)} = \mathbf{167 \text{ in}}$

Properties for horizontally spanning walls

Ratio of grouted bond beams to voids;	$G_{\text{hori}} = 0 = \mathbf{0}$
Gross cross-sectional area;	$A_{g, \text{hori}} = t = \mathbf{91.5 \text{ in}^2/\text{ft}}$
Gross moment of inertia;	$I_{g, \text{hori}} = t^3 / 12 = \mathbf{443.3 \text{ in}^4/\text{ft}}$
Net cross-sectional area;	$A_{n, \text{hori}} = G_{\text{hori}} \times t + (1 - G_{\text{hori}}) \times (2 \times t_{fs}) = \mathbf{30 \text{ in}^2/\text{ft}}$
Net moment of inertia;	$I_{n, \text{hori}} = G_{\text{hori}} \times t^3 / 12 + (1 - G_{\text{hori}}) \times (t^3 / 12 - (t - 2 \times t_{fs})^3 / 12) = \mathbf{308.7 \text{ in}^4/\text{ft}}$
Net section modulus;	$S_{n, \text{hori}} = 2 \times I_{n, \text{hori}} / t = \mathbf{81 \text{ in}^3/\text{ft}}$

Average out-of-plane section properties

Average cross-sectional area;	$A_{\text{ave}} = (t_j \times A_n + h_m \times A) / (t_j + h_m) = \mathbf{67 \text{ in}^2/\text{ft}}$
Average moment of inertia;	$I_{\text{ave}} = (t_j \times I_n + h_m \times I) / (t_j + h_m) = \mathbf{389.7 \text{ in}^4/\text{ft}}$
Average cross-sectional area;	$A_{\text{ave, hori}} = (n_{w, \text{ext}} \times t_{w, \text{ext}} + n_{w, \text{int}} \times t_{w, \text{int}}) / (l_m + t_j) \times A_{g, \text{hori}} + (l_m + t_j - n_{w, \text{ext}} \times t_{w, \text{ext}} - n_{w, \text{int}} \times t_{w, \text{int}}) / (l_m + t_j) \times A_{n, \text{hori}} = \mathbf{41.5 \text{ in}^2/\text{ft}}$
Average moment of inertia;	$I_{\text{ave, hori}} = (n_{w, \text{ext}} \times t_{w, \text{ext}} + n_{w, \text{int}} \times t_{w, \text{int}}) / (l_m + t_j) \times I_{g, \text{hori}} + (l_m + t_j - n_{w, \text{ext}} \times t_{w, \text{ext}} - n_{w, \text{int}} \times t_{w, \text{int}}) / (l_m + t_j) \times I_{n, \text{hori}} = \mathbf{334 \text{ in}^4/\text{ft}}$

Orthotropic properties for section adjustment

Primary bending plane;	Major
Elastic modulus x - horizontal spanning minor axis;	$E_x = E_m \times I_{\text{ave, hori}} / I_{g, \text{hori}} = \mathbf{1694904 \text{ psi}}$
Elastic modulus y - vertical spanning major axis;	$E_y = E_m \times A_{\text{ave}} / t = \mathbf{1647260 \text{ psi}}$
In-plane shear modulus;	$G_{xy} = G_m \times A_{\text{ave}} / t = \mathbf{658904 \text{ psi}}$
Out-of-plane shear modulus;	$G_{xz} = G_{xy} = \mathbf{658904 \text{ psi}}$
Poison ratio;	$\nu_{xy} = \mathbf{0.250}$



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 137
Project Name: JKV-EMS		

Average wall density;

$$\rho_{ave} = (A_m \times \rho_m + A_g \times \rho_{grout}) / t + 2.5 \text{ pcf} = \mathbf{100 \text{ lb/ft}^3}$$

Out-of-plane axial resistance

Slenderness ratio;

$$h_{eff,y} / r = \mathbf{91.532} < 99$$

Allowable axial force - Eqn.8-18;

$$P_{a\_pos} = (0.25 \times f'_m \times (A_n - A_s)) \times (1 - (h_{eff,y} / (140 \times r))^2) = \mathbf{22.09 \text{ kips/ft}}$$

Out-of-plane axial stress resistance

Slenderness ratio;

$$h_{eff,y} / r = \mathbf{91.532} < 99$$

Allowable axial stress - cl.8.2.4.1;

$$F_{a\_pos} = (1 / 4) \times f'_m \times (1 - (h_{eff,y} / (140 \times r))^2) = \mathbf{357.8 \text{ psi}}$$

Moment at balance point

Modular ratio;

$$n = E_s / E_m = \mathbf{12.89}$$

Allowable stress in masonry (cl.8.3.4.2.2);

$$F_b = 0.45 \times f'_m = \mathbf{1125 \text{ psi}}$$

Balance point;

$$k_b = n / (n + F_s / F_b) = \mathbf{0.312}$$

Tensile strain in reinforcement;

$$\epsilon_s = F_s / E_s = \mathbf{0.001103}$$

Compressive strain in masonry;

$$\epsilon_m = \epsilon_s \times k_b / (1 - k_b) = \mathbf{0.000500}$$

Effective depth of reinforcement;

$$d = \mathbf{3.813 \text{ in}}$$

Width of T section web;

$$b_w = (l_c + t_{w,ext} + t_{w,int}) / s_{vert} \times 12 \text{ in/ft} = \mathbf{0.520 \text{ ft/ft}}$$

Comp. stress in masonry due to flexure only;

$$f_b = F_s / n \times k_b \times d / (d - k_b \times d) = \mathbf{1125 \text{ psi}}$$

Stress in reinforcement;

$$f_s = F_s = \mathbf{32000 \text{ psi}}$$

Stress at compression side of web;

$$f_{b1} = (k_b \times d - t_{fs}) / (k_b \times d) \times f_b = \mathbf{-58 \text{ psi}}$$

Compression moment arm to face shell;

$$x_1 = d - k_b \times d / 3 = \mathbf{3.416 \text{ in}}$$

Compression in face shell;

$$C_1 = f_b / 2 \times k_b \times d = \mathbf{8.025 \text{ kips/ft}}$$

Compression at balance point;

$$C_b = C_1 = \mathbf{8.025 \text{ kips/ft}}$$

Force in tension face reinf. at balance point;

$$T_b = f_s \times A_s = \mathbf{7.363 \text{ kips/ft}}$$

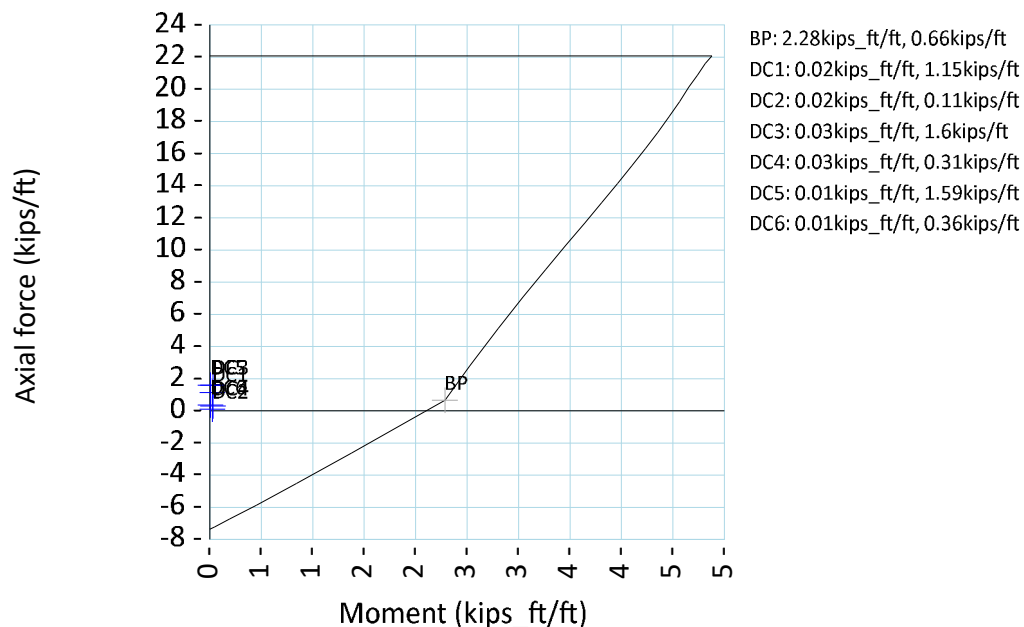
Axial load at balance point;

$$N_b = \min(C_1 - T_b, P_{a\_pos}) = \mathbf{0.662 \text{ kips/ft}}$$

Moment at balance point;

$$M_b = \max(C_1 \times x_1, 0 \text{ lb\_ft/ft}) = \mathbf{2.285 \text{ kips\_ft/ft}}$$

Out-of-plane interaction diagram





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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 138
Project Name: JKV-EMS		

In-plane axial resistance

In-plane axial force;

Depth to reinforcement

Net shear area

Slenderness ratio;

Allowable axial force - Eqn.8-19;

$$P_{in} = P \times L = \mathbf{17.4 \text{ kips}}$$

$$d_{in} = L - l_m / 4 = \mathbf{48.0 \text{ ft}}$$

$$A_{nv,in} = 2 \times d_{in} \times t_{fs} + d_{in} / s_{grout} \times (l_m / (n_{w,int} + 1)) \times (t - 2 \times t_{fs}) = \mathbf{2879 \text{ in}^2}$$

$$h_{eff,x} / r_x = \mathbf{2.716} < 99$$

$$P_{a,in,pos} = ((0.25 \times f'_m \times (A_n - A_s)) \times (1 - (h_{eff,x} / (140 \times r_x))^2) \times L) = \mathbf{1861.78 \text{ kips}}$$

In-plane axial stress resistance

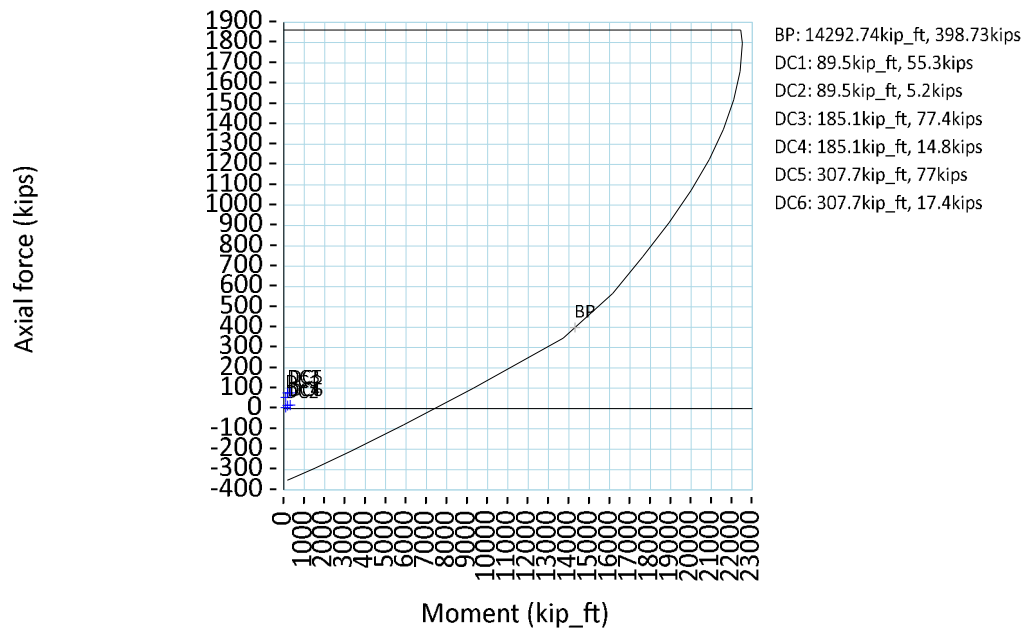
Slenderness ratio;

Allowable axial stress - cl.8.2.4.1;

$$h_{eff,x} / r_x = \mathbf{2.716} < 99$$

$$F_{a,in,pos} = (1 / 4) \times f'_m \times (1 - (h_{eff,x} / (140 \times r_x))^2) = \mathbf{624.8 \text{ psi}}$$

In-plane interaction diagram



From interaction diagram

Axial load at balance point;

Moment at balance point;

Maximum moment;

$$N_{b,in} = \mathbf{399 \text{ kips}}$$

$$M_{b,in} = \mathbf{14293 \text{ kip\_ft}}$$

$$M_{max,in} = \mathbf{22520 \text{ kip\_ft}}$$

Top: Max axial

Design forces

Axial load;

Axial eccentricity;

Out-of-plane moment;

Out-of-plane shear force;

In-plane moment;

In-plane Shear force;

$$P = \mathbf{1.145 \text{ kips/ft}}$$

$$e = \mathbf{0 \text{ in}}$$

$$M_y = \mathbf{-0.025 \text{ kips\_ft/ft}}$$

$$V_y = \mathbf{0.021 \text{ kips/ft}}$$

$$M_x = \mathbf{89.5 \text{ kips\_ft}}$$

$$V_x = \mathbf{5.6 \text{ kips}}$$

Reinforcement effective depth

Effective depth of reinforcement;

$$d = \mathbf{3.813 \text{ in}}$$



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 139
Project Name: JKV-EMS		

Out-of-plane axial check

$$P / P_{a\_pos} = 0.052$$

PASS - Pure axial resistance exceeds axial load

Out-of-plane axial stress check

Stress due to axial load;

$$f_a = P / A = 17.0 \text{ psi}$$

$$f_a / F_{a\_pos} = 0.048$$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit} = 2.250 \text{ kips\_ft/ft}$$

$$Abs(M_y) / M_{limit} = 0.011$$

PASS - Design flexural strength exceeds factored bending moment

Out-of-plane shear check

Net shear area

$$A_{nv} = d \times l_m / ((n_{w,int} + 1) \times s_{grout}) = 22.34 \text{ in}^2/\text{ft}$$

Applied shear stress;

$$f_v = V_y / A_{nv} = 0.93 \text{ psi}$$

Shear resistance of masonry - Eqn 8-26;

$$F_{vm} = 1/2 \times (4 - 1.75 \times \min(Abs(M_y) / (V_y \times d), 1)) \times \sqrt{(f'_m \times 1 \text{ psi}) + 0.25 \times P / A_n} = 60.87 \text{ psi}$$

Grout factor - cl.8.3.5.1.2;

$$\gamma_g = 0.75$$

Overall shear resistance - cl.8.3.5.1.2;

$$F_v = \min(F_{vm}, 2 \times \sqrt{(f'_m \times 1 \text{ psi})}) \times \gamma_g = 45.65 \text{ psi}$$

$$f_v / F_v = 0.02$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane axial check

$$P_{in} / P_{a,in\_pos} = 0.030$$

PASS - Pure axial resistance exceeds axial load

In-plane axial stress check

Stress due to axial load;

$$f_a = P / A = 17.0 \text{ psi}$$

$$f_a / F_{a,in\_pos} = 0.027$$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit,in} = 8450 \text{ kip\_ft}$$

$$Abs(M_x) / M_{limit,in} = 0.011$$

PASS - Design flexural strength exceeds factored bending moment

In-plane shear check

Shear stress;

$$f_{v,in} = V_x / A_{nv,in} = 1.95 \text{ psi}$$

Allowable shear stress of masonry;

$$F_{vm,in} = 1/2 \times (4 - 1.75 \times \min(Abs(M_x) / (V_x \times d_{in}), 1)) \times \sqrt{(f'_m \times 1 \text{ psi}) + 0.25 \times P_{in} / (A_n \times L)} = 90.04 \text{ psi}$$

Allowable shear stress of steel;

$$F_{vs,in} = 0.5 \times (A_v \times F_{sv} \times d_{in} / (A_{nv,in} \times d_{in} \times 1\text{ft})) = 309.22 \text{ psi}$$

Grout factor;

$$\gamma_g = 0.75$$

Allowable shear stress;

$$F_{v,in} = \min(F_{vm,in} + F_{vs,in}, 2.89 \times \sqrt{(f'_m \times 1 \text{ psi})}) \times \gamma_g = 108.38 \text{ psi}$$

$$f_{v,in} / F_{v,in} = 0.018$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane shear friction - 8.3.6

Coefficient of friction

$$\mu = 0.7$$



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 140
Project Name: JKV-EMS		

Shear friction stress ($M_x / (V_x / d_{in}) \leq 0.5$)
Shear friction stress ($M_x / (V_x / d_{in}) \geq 1$)
Shear friction stress as $M_x / (V_x \times d_{in}) \leq 0.5$

$$F_{f1} = \max(\mu \times (A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = \mathbf{99.3 \text{ psi}}$$
$$F_{f2} = \max(0.65 \times (0.6 \times A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = \mathbf{60.3 \text{ psi}}$$
$$F_f = F_{f1} = \mathbf{99.3 \text{ psi}}$$
$$f_{v,in} / F_f = \mathbf{0.020}$$

PASS - Allowable shear friction stress exceeds applied shear stress

Biaxial bending

$$\sqrt{((\text{abs}(M_y) / M_{limit})^2 + (\text{abs}(M_x) / M_{limit,in})^2)} = \mathbf{0.015}$$

PASS - Biaxial bending utilization is acceptable

Top: Min axial

Design forces

Axial load;
Axial eccentricity;
Out-of-plane moment;
Out-of-plane shear force;
In-plane moment;
In-plane Shear force;

$$P = \mathbf{0.108 \text{ kips/ft}}$$
$$e = \mathbf{0 \text{ in}}$$
$$M_y = \mathbf{-0.025 \text{ kips\_ft/ft}}$$
$$V_y = \mathbf{0.021 \text{ kips/ft}}$$
$$M_x = \mathbf{89.5 \text{ kips\_ft}}$$
$$V_x = \mathbf{5.6 \text{ kips}}$$

Reinforcement effective depth

Effective depth of reinforcement;

$$d = \mathbf{3.813 \text{ in}}$$

Out-of-plane axial check

$$P / P_{a\_pos} = \mathbf{0.005}$$

PASS - Pure axial resistance exceeds axial load

Out-of-plane axial stress check

Stress due to axial load;

$$f_a = P / A = \mathbf{1.6 \text{ psi}}$$
$$f_a / F_{a\_pos} = \mathbf{0.004}$$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit} = \mathbf{2.097 \text{ kips\_ft/ft}}$$
$$\text{Abs}(M_y) / M_{limit} = \mathbf{0.012}$$

PASS - Design flexural strength exceeds factored bending moment

Out-of-plane shear check

Net shear area
Applied shear stress;
Shear resistance of masonry - Eqn 8-26;

$$A_{nv} = d \times l_m / ((n_{w,int} + 1) \times s_{grout}) = \mathbf{22.34 \text{ in}^2/\text{ft}}$$
$$f_v = V_y / A_{nv} = \mathbf{0.93 \text{ psi}}$$
$$F_{vm} = 1/2 \times (4 - 1.75 \times \min(\text{Abs}(M_y) / (V_y \times d), 1)) \times \sqrt{(f'_m \times 1 \text{ psi})} + 0.25 \times P / A_n = \mathbf{56.68 \text{ psi}}$$
$$\gamma_g = \mathbf{0.75}$$
$$F_v = \min(F_{vm}, 2 \times \sqrt{(f'_m \times 1 \text{ psi})}) \times \gamma_g = \mathbf{42.51 \text{ psi}}$$
$$f_v / F_v = \mathbf{0.022}$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane axial check

$$P_{in} / P_{a,in\_pos} = \mathbf{0.003}$$

PASS - Pure axial resistance exceeds axial load



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 141
Project Name: JKV-EMS		

In-plane axial stress check

Stress due to axial load;

$$f_a = P / A = 1.6 \text{ psi}$$

$$f_a / F_{a, \text{in\_pos}} = 0.003$$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{\text{limit, in}} = 7502 \text{ kip\_ft}$$

$$\text{Abs}(M_x) / M_{\text{limit, in}} = 0.012$$

PASS - Design flexural strength exceeds factored bending moment

In-plane shear check

Shear stress;

$$f_{v, \text{in}} = V_x / A_{nv, \text{in}} = 1.95 \text{ psi}$$

Allowable shear stress of masonry;

$$F_{vm, \text{in}} = 1/2 \times (4 - 1.75 \times \min(\text{Abs}(M_x) / (V_x \times d_{\text{in}}), 1)) \times \sqrt{(f'_m \times 1 \text{ psi})} + 0.25 \times P_{\text{in}} / (A_n \times L) = 85.85 \text{ psi}$$

Allowable shear stress of steel;

$$F_{vs, \text{in}} = 0.5 \times (A_v \times F_{sv} \times d_{\text{in}} / (A_{nv, \text{in}} \times d_{\text{in}} \times 1 \text{ ft})) = 309.22 \text{ psi}$$

Grout factor;

$$\gamma_g = 0.75$$

Allowable shear stress;

$$F_{v, \text{in}} = \min(F_{vm, \text{in}} + F_{vs, \text{in}}, 2.89 \times \sqrt{(f'_m \times 1 \text{ psi})}) \times \gamma_g = 108.38 \text{ psi}$$

$$f_{v, \text{in}} / F_{v, \text{in}} = 0.018$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane shear friction - 8.3.6

Coefficient of friction

$$\mu = 0.7$$

Shear friction stress ($M_x / (V_x \times d_{\text{in}})$ lte 0.5)

$$F_{f1} = \max(\mu \times (A_s \times d_{\text{in}} \times F_s + P_{\text{in}}) / (A_{nv, \text{in}}), 0 \text{ psi}) = 87.1 \text{ psi}$$

Shear friction stress ($M_x / (V_x \times d_{\text{in}})$ gte 1)

$$F_{f2} = \max(0.65 \times (0.6 \times A_s \times d_{\text{in}} \times F_s + P_{\text{in}}) / (A_{nv, \text{in}}), 0 \text{ psi}) = 49 \text{ psi}$$

Shear friction stress as $M_x / (V_x \times d_{\text{in}})$ lte 0.5

$$F_f = F_{f1} = 87.1 \text{ psi}$$

$$f_{v, \text{in}} / F_f = 0.022$$

PASS - Allowable shear friction stress exceeds applied shear stress

Biaxial bending

$$\sqrt{((\text{abs}(M_y) / M_{\text{limit}})^2 + (\text{abs}(M_x) / M_{\text{limit, in}})^2)} = 0.017$$

PASS - Biaxial bending utilization is acceptable

Mid-fifth: Max axial

Design forces

Axial load;

$$P = 1.603 \text{ kips/ft}$$

Axial eccentricity;

$$e = 0 \text{ in}$$

Out-of-plane moment;

$$M_y = 0.031 \text{ kips\_ft/ft}$$

Out-of-plane shear force;

$$V_y = 0.008 \text{ kips/ft}$$

In-plane moment;

$$M_x = 185.1 \text{ kips\_ft}$$

In-plane Shear force;

$$V_x = 5 \text{ kips}$$

Reinforcement effective depth

Effective depth of reinforcement;

$$d = 3.813 \text{ in}$$

Out-of-plane axial check

$$P / P_{a, \text{pos}} = 0.073$$

PASS - Pure axial resistance exceeds axial load



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 142
Project Name: JKV-EMS		

Out-of-plane axial stress check

Stress due to axial load;

$$f_a = P / A = 23.8 \text{ psi}$$

$$f_a / F_{a\_pos} = 0.067$$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit} = 2.403 \text{ kips\_ft/ft}$$

$$Abs(M_y) / M_{limit} = 0.013$$

PASS - Design flexural strength exceeds factored bending moment

Out-of-plane shear check

Net shear area

$$A_{nv} = d \times l_m / ((n_{w,int} + 1) \times s_{grout}) = 22.34 \text{ in}^2/\text{ft}$$

Applied shear stress;

$$f_v = V_y / A_{nv} = 0.37 \text{ psi}$$

Shear resistance of masonry - Eqn 8-26;

$$F_{vm} = 1/2 \times (4 - 1.75 \times \min(Abs(M_y) / (V_y \times d), 1)) \times \sqrt{(f'_m \times 1 \text{ psi}) + 0.25 \times P / A_n} = 62.72 \text{ psi}$$

Grout factor - cl.8.3.5.1.2;

$$\gamma_g = 0.75$$

Overall shear resistance - cl.8.3.5.1.2;

$$F_v = \min(F_{vm}, 2 \times \sqrt{(f'_m \times 1 \text{ psi})}) \times \gamma_g = 47.04 \text{ psi}$$

$$f_v / F_v = 0.008$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane axial check

$$P_{in} / P_{a,in\_pos} = 0.042$$

PASS - Pure axial resistance exceeds axial load

In-plane axial stress check

Stress due to axial load;

$$f_a = P / A = 23.8 \text{ psi}$$

$$f_a / F_{a,in\_pos} = 0.038$$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit,in} = 8867 \text{ kip\_ft}$$

$$Abs(M_x) / M_{limit,in} = 0.021$$

PASS - Design flexural strength exceeds factored bending moment

In-plane shear check

Shear stress;

$$f_{v,in} = V_x / A_{nv,in} = 1.74 \text{ psi}$$

Allowable shear stress of masonry;

$$F_{vm,in} = 1/2 \times (4 - 1.75 \times \min(Abs(M_x) / (V_x \times d_{in}), 1)) \times \sqrt{(f'_m \times 1 \text{ psi}) + 0.25 \times P_{in} / (A_n \times L)} = 72.7 \text{ psi}$$

Allowable shear stress of steel;

$$F_{vs,in} = 0.5 \times (A_v \times F_{sv} \times d_{in} / (A_{nv,in} / d_{in} \times 1\text{ft})) = 309.22 \text{ psi}$$

Grout factor;

$$\gamma_g = 0.75$$

Allowable shear stress;

$$F_{v,in} = \min(F_{vm,in} + F_{vs,in}, 2.3 \times \sqrt{(f'_m \times 1 \text{ psi})}) \times \gamma_g = 86.25 \text{ psi}$$

$$f_{v,in} / F_{v,in} = 0.02$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane shear friction - 8.3.6

Coefficient of friction

$$\mu = 0.7$$

Shear friction stress ($M_x / (V_x / d_{in})$ lte 0.5)

$$F_{f1} = \max(\mu \times (A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = 104.7 \text{ psi}$$

Shear friction stress ($M_x / (V_x / d_{in})$ gte 1)

$$F_{f2} = \max(0.65 \times (0.6 \times A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = 65.3 \text{ psi}$$

Shear friction stress (interpolate F_{f1} and F_{f2})

$$F_f = 83.3 \text{ psi}$$



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 143
Project Name: JKV-EMS		

Biaxial bending

$$f_{v,in} / F_f = 0.021$$

PASS - Allowable shear friction stress exceeds applied shear stress

$$\sqrt{((abs(M_y) / M_{limit})^2 + (abs(M_x) / M_{limit,in})^2)} = 0.025$$

PASS - Biaxial bending utilization is acceptable

Mid-fifth: Min axial

Design forces

Axial load;

$$P = 0.307 \text{ kips/ft}$$

Axial eccentricity;

$$e = 0 \text{ in}$$

Out-of-plane moment;

$$M_y = 0.031 \text{ kips\_ft/ft}$$

Out-of-plane shear force;

$$V_y = 0.008 \text{ kips/ft}$$

In-plane moment;

$$M_x = 185.1 \text{ kips\_ft}$$

In-plane Shear force;

$$V_x = 5 \text{ kips}$$

Reinforcement effective depth

Effective depth of reinforcement;

$$d = 3.813 \text{ in}$$

Out-of-plane axial check

$$P / P_{a,pos} = 0.014$$

PASS - Pure axial resistance exceeds axial load

Out-of-plane axial stress check

Stress due to axial load;

$$f_a = P / A = 4.6 \text{ psi}$$

$$f_a / F_{a,pos} = 0.013$$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit} = 2.097 \text{ kips\_ft/ft}$$

$$Abs(M_y) / M_{limit} = 0.015$$

PASS - Design flexural strength exceeds factored bending moment

Out-of-plane shear check

Net shear area

$$A_{nv} = d \times l_m / ((n_{w,int} + 1) \times s_{grout}) = 22.34 \text{ in}^2/\text{ft}$$

Applied shear stress;

$$f_v = V_y / A_{nv} = 0.37 \text{ psi}$$

Shear resistance of masonry - Eqn 8-26;

$$F_{vm} = 1/2 \times (4 - 1.75 \times \min(Abs(M_y) / (V_y \times d), 1)) \times \sqrt{(f'_m \times 1 \text{ psi})} + 0.25 \times P / A_n = 57.49 \text{ psi}$$

Grout factor - cl.8.3.5.1.2;

$$\gamma_g = 0.75$$

Overall shear resistance - cl.8.3.5.1.2;

$$F_v = \min(F_{vm}, 2 \times \sqrt{(f'_m \times 1 \text{ psi})}) \times \gamma_g = 43.12 \text{ psi}$$

$$f_v / F_v = 0.009$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane axial check

$$P_{in} / P_{a,in,pos} = 0.008$$

PASS - Pure axial resistance exceeds axial load

In-plane axial stress check

Stress due to axial load;

$$f_a = P / A = 4.6 \text{ psi}$$

$$f_a / F_{a,in,pos} = 0.007$$

PASS - Allowable stress exceeds compressive stress due to axial load



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 144
Project Name: JKV-EMS		

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit,in} = 7692 \text{ kip\_ft}$$

$$Abs(M_x) / M_{limit,in} = 0.024$$

PASS - Design flexural strength exceeds factored bending moment

In-plane shear check

Shear stress;

$$f_{v,in} = V_x / A_{nv,in} = 1.74 \text{ psi}$$

Allowable shear stress of masonry;

$$F_{vm,in} = 1/2 \times (4 - 1.75 \times \min(Abs(M_x) / (V_x \times d_{in}), 1)) \times \sqrt{(f'_m \times 1 \text{ psi}) + 0.25 \times P_{in} / (A_n \times L)} = 67.46 \text{ psi}$$

$$P_{in} / (A_n \times L) = 67.46 \text{ psi}$$

Allowable shear stress of steel;

$$F_{vs,in} = 0.5 \times (A_v \times F_{sv} \times d_{in} / (A_{nv,in} \times d_{in} \times 1 \text{ ft})) = 309.22 \text{ psi}$$

Grout factor;

$$\gamma_g = 0.75$$

Allowable shear stress;

$$F_{v,in} = \min(F_{vm,in} + F_{vs,in}, 2.3 \times \sqrt{(f'_m \times 1 \text{ psi})}) \times \gamma_g = 86.25 \text{ psi}$$

$$f_{v,in} / F_{v,in} = 0.02$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane shear friction - 8.3.6

Coefficient of friction

$$\mu = 0.7$$

Shear friction stress ($M_x / (V_x / d_{in})$ lte 0.5)

$$F_{r1} = \max(\mu \times (A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = 89.5 \text{ psi}$$

Shear friction stress ($M_x / (V_x / d_{in})$ gte 1)

$$F_{r2} = \max(0.65 \times (0.6 \times A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = 51.2 \text{ psi}$$

Shear friction stress (interpolate F_{r1} and F_{r2})

$$F_f = 68.6 \text{ psi}$$

$$f_{v,in} / F_f = 0.025$$

PASS - Allowable shear friction stress exceeds applied shear stress

Biaxial bending

$$\sqrt{((Abs(M_y) / M_{limit})^2 + (Abs(M_x) / M_{limit,in})^2)} = 0.028$$

PASS - Biaxial bending utilization is acceptable

Bottom: Max axial

Design forces

Axial load;

$$P = 1.595 \text{ kips/ft}$$

Axial eccentricity;

$$e = 0 \text{ in}$$

Out-of-plane moment;

$$M_y = 0.006 \text{ kips\_ft/ft}$$

Out-of-plane shear force;

$$V_y = 0.008 \text{ kips/ft}$$

In-plane moment;

$$M_x = 307.7 \text{ kips\_ft}$$

In-plane Shear force;

$$V_x = 5.5 \text{ kips}$$

Reinforcement effective depth

Effective depth of reinforcement;

$$d = 3.813 \text{ in}$$

Out-of-plane axial check

$$P / P_{a\_pos} = 0.072$$

PASS - Pure axial resistance exceeds axial load

Out-of-plane axial stress check

Stress due to axial load;

$$f_a = P / A = 23.7 \text{ psi}$$

$$f_a / F_{a\_pos} = 0.066$$

PASS - Allowable stress exceeds compressive stress due to axial load



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 145
Project Name: JKV-EMS		

From interaction diagram

Limiting moment under applied axial load;

$$M_{\text{limit}} = 2.403 \text{ kips\_ft/ft}$$

$$\text{Abs}(M_y) / M_{\text{limit}} = 0.003$$

PASS - Design flexural strength exceeds factored bending moment

Out-of-plane shear check

Net shear area

$$A_{nv} = d \times l_m / ((n_{w,int} + 1) \times s_{grout}) = 22.34 \text{ in}^2/\text{ft}$$

Applied shear stress;

$$f_v = V_y / A_{nv} = 0.37 \text{ psi}$$

Shear resistance of masonry - Eqn 8-26;

$$F_{vm} = 1/2 \times (4 - 1.75 \times \min(\text{Abs}(M_y) / (V_y \times d), 1)) \times \sqrt{(f'_m \times 1 \text{ psi})} + 0.25 \times P / A_n = 62.69 \text{ psi}$$

Grout factor - cl.8.3.5.1.2;

$$\gamma_g = 0.75$$

Overall shear resistance - cl.8.3.5.1.2;

$$F_v = \min(F_{vm}, 2 \times \sqrt{(f'_m \times 1 \text{ psi})}) \times \gamma_g = 47.01 \text{ psi}$$

$$f_v / F_v = 0.008$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane axial check

$$P_{in} / P_{a,in\_pos} = 0.041$$

PASS - Pure axial resistance exceeds axial load

In-plane axial stress check

Stress due to axial load;

$$f_a = P / A = 23.7 \text{ psi}$$

$$f_a / F_{a,in\_pos} = 0.038$$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{\text{limit},in} = 8867 \text{ kip\_ft}$$

$$\text{Abs}(M_x) / M_{\text{limit},in} = 0.035$$

PASS - Design flexural strength exceeds factored bending moment

In-plane shear check

Shear stress;

$$f_{v,in} = V_x / A_{nv,in} = 1.91 \text{ psi}$$

Allowable shear stress of masonry;

$$F_{vm,in} = 1/2 \times (4 - 1.75 \times \min(\text{Abs}(M_x) / (V_x \times d_{in}), 1)) \times \sqrt{(f'_m \times 1 \text{ psi})} + 0.25 \times P_{in} / (A_n \times L) = 62.69 \text{ psi}$$

Allowable shear stress of steel;

$$F_{vs,in} = 0.5 \times (A_v \times F_{sv} \times d_{in} / (A_{nv,in} \times d_{in} \times 1\text{ft})) = 309.22 \text{ psi}$$

Grout factor;

$$\gamma_g = 0.75$$

Allowable shear stress;

$$F_{v,in} = \min(F_{vm,in} + F_{vs,in}, 2 \times \sqrt{(f'_m \times 1 \text{ psi})}) \times \gamma_g = 75 \text{ psi}$$

$$f_{v,in} / F_{v,in} = 0.025$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane shear friction - 8.3.6

Coefficient of friction

$$\mu = 0.7$$

Shear friction stress ($M_x / (V_x \times d_{in})$ lte 0.5)

$$F_{f1} = \max(\mu \times (A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = 104.6 \text{ psi}$$

Shear friction stress ($M_x / (V_x \times d_{in})$ gte 1)

$$F_{f2} = \max(0.65 \times (0.6 \times A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = 65.2 \text{ psi}$$

Shear friction stress as $M_x / (V_x \times d_{in})$ gte 1.0

$$F_f = F_{f2} = 65.2 \text{ psi}$$

$$f_{v,in} / F_f = 0.029$$

PASS - Allowable shear friction stress exceeds applied shear stress

Biaxial bending

$$\sqrt{((\text{abs}(M_y) / M_{\text{limit}})^2 + (\text{abs}(M_x) / M_{\text{limit},in})^2)} = 0.035$$



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 146
Project Name: JKV-EMS		

PASS - Biaxial bending utilization is acceptable

Bottom: Min axial

Design forces

Axial load; $P = 0.360$ kips/ft
Axial eccentricity; $e = 0$ in
Out-of-plane moment; $M_y = 0.006$ kips_ft/ft
Out-of-plane shear force; $V_y = 0.008$ kips/ft
In-plane moment; $M_x = 307.7$ kips_ft
In-plane Shear force; $V_x = 5.5$ kips

Reinforcement effective depth

Effective depth of reinforcement; $d = 3.813$ in

Out-of-plane axial check

$$P / P_{a\_pos} = 0.016$$

PASS - Pure axial resistance exceeds axial load

Out-of-plane axial stress check

Stress due to axial load; $f_a = P / A = 5.4$ psi
 $f_a / F_{a\_pos} = 0.015$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load; $M_{limit} = 2.097$ kips_ft/ft
 $Abs(M_y) / M_{limit} = 0.003$

PASS - Design flexural strength exceeds factored bending moment

Out-of-plane shear check

Net shear area $A_{nv} = d \times l_m / ((n_{w,int} + 1) \times S_{grout}) = 22.34$ in²/ft
Applied shear stress; $f_v = V_y / A_{nv} = 0.37$ psi
Shear resistance of masonry - Eqn 8-26; $F_{vm} = 1/2 \times (4 - 1.75 \times \min(Abs(M_y) / (V_y \times d), 1)) \times \sqrt{(f'_m \times 1 \text{ psi})} + 0.25 \times P / A_n = 57.7$ psi
Grout factor - cl.8.3.5.1.2; $\gamma_g = 0.75$
Overall shear resistance - cl.8.3.5.1.2; $F_v = \min(F_{vm}, 2 \times \sqrt{(f'_m \times 1 \text{ psi})}) \times \gamma_g = 43.28$ psi
 $f_v / F_v = 0.009$

PASS - Allowable shear stress exceeds applied shear stress

In-plane axial check

$$P_{in} / P_{a\_in\_pos} = 0.009$$

PASS - Pure axial resistance exceeds axial load

In-plane axial stress check

Stress due to axial load; $f_a = P / A = 5.4$ psi
 $f_a / F_{a\_in\_pos} = 0.009$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load; $M_{limit,in} = 7730$ kip_ft
 $Abs(M_x) / M_{limit,in} = 0.040$

PASS - Design flexural strength exceeds factored bending moment



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 147
Project Name: JKV-EMS		

In-plane shear check

Shear stress;

Allowable shear stress of masonry;

Allowable shear stress of steel;

Grout factor;

Allowable shear stress;

$$f_{v,in} = V_x / A_{nv,in} = \mathbf{1.91 \text{ psi}}$$

$$F_{vm,in} = 1/2 \times (4 - 1.75 \times \min(\text{Abs}(M_x) / (V_x \times d_{in}), 1)) \times \sqrt{(f'_m \times 1 \text{ psi}) + 0.25 \times P_{in} / (A_n \times L)} = \mathbf{57.7 \text{ psi}}$$

$$F_{vs,in} = 0.5 \times (A_v \times F_{sv} \times d_{in} / (A_{nv,in} \times d_{in} \times 1\text{ft})) = \mathbf{309.22 \text{ psi}}$$

$$\gamma_g = \mathbf{0.75}$$

$$F_{v,in} = \min(F_{vm,in} + F_{vs,in}, 2 \times \sqrt{(f'_m \times 1 \text{ psi})}) \times \gamma_g = \mathbf{75 \text{ psi}}$$

$$f_{v,in} / F_{v,in} = \mathbf{0.025}$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane shear friction - 8.3.6

Coefficient of friction

Shear friction stress ($M_x / (V_x \times d_{in})$ lte 0.5)

Shear friction stress ($M_x / (V_x \times d_{in})$ gte 1)

Shear friction stress as $M_x / (V_x \times d_{in})$ gte 1.0

$$\mu = \mathbf{0.7}$$

$$F_{f1} = \max(\mu \times (A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = \mathbf{90.1 \text{ psi}}$$

$$F_{f2} = \max(0.65 \times (0.6 \times A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = \mathbf{51.8 \text{ psi}}$$

$$F_f = F_{f2} = \mathbf{51.8 \text{ psi}}$$

$$f_{v,in} / F_f = \mathbf{0.037}$$

PASS - Allowable shear friction stress exceeds applied shear stress

Biaxial bending

$$\sqrt{(\text{abs}(M_y) / M_{limit})^2 + (\text{abs}(M_x) / M_{limit,in})^2} = \mathbf{0.040}$$

PASS - Biaxial bending utilization is acceptable

MASONRY WALL SECTION DESIGN - WP19

In accordance with TMS-16 using the allowable stress design method

Tedds calculation version 1.0.06

Design summary

Overall design status;

PASS

Overall design utilization;

0.313

Top: Max axial

Description	Unit	Provided	Required	Utilization	Result
Out-of-plane axial force	kips/ft	22.1	0.9	0.042	PASS
Out-of-plane axial stress	psi	357.8	13.7	0.038	PASS
Out-of-plane flexural strength	kip_ft/ft	2.250	0.655	0.291	PASS
Out-of-plane shear stress	psi	45.0	2.5	0.055	PASS
In-plane axial force	kips	2290.7	54.8	0.024	PASS
In-plane axial stress	psi	624.8	13.7	0.022	PASS
In-plane flexural strength	kip_ft	12414.1	140.4	0.011	PASS
In-plane shear stress	psi	101.3	1.4	0.014	PASS
In-plane shear friction	psi	96.7	1.4	0.015	PASS
Biaxial bending				0.291	PASS



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 148
Project Name: JKV-EMS		

Top: Min axial

Description	Unit	Provided	Required	Utilization	Result
Out-of-plane axial force	kips/ft	22.1	0.1	0.006	PASS
Out-of-plane axial stress	psi	357.8	2.0	0.005	PASS
Out-of-plane flexural strength	kip_ft/ft	2.097	0.655	0.312	PASS
Out-of-plane shear stress	psi	42.6	2.5	0.058	PASS
In-plane axial force	kips	2290.7	7.8	0.003	PASS
In-plane axial stress	psi	624.8	2.0	0.003	PASS
In-plane flexural strength	kip_ft	11319.1	140.4	0.012	PASS
In-plane shear stress	psi	101.3	1.4	0.014	PASS
In-plane shear friction	psi	87.4	1.4	0.016	PASS
Biaxial bending				0.313	PASS

Mid-fifth: Max axial

Description	Unit	Provided	Required	Utilization	Result
Out-of-plane axial force	kips/ft	22.1	1.7	0.076	PASS
Out-of-plane axial stress	psi	357.8	25.0	0.070	PASS
Out-of-plane flexural strength	kip_ft/ft	2.403	0.310	0.129	PASS
Out-of-plane shear stress	psi	47.3	1.7	0.035	PASS
In-plane axial force	kips	2290.7	99.9	0.044	PASS
In-plane axial stress	psi	624.8	25.0	0.040	PASS
In-plane flexural strength	kip_ft	13462.6	151.3	0.011	PASS
In-plane shear stress	psi	99.8	1.4	0.014	PASS
In-plane shear friction	psi	105.6	1.4	0.014	PASS
Biaxial bending				0.129	PASS

Mid-fifth: Min axial

Description	Unit	Provided	Required	Utilization	Result
Out-of-plane axial force	kips/ft	22.1	0.5	0.022	PASS
Out-of-plane axial stress	psi	357.8	7.2	0.020	PASS
Out-of-plane flexural strength	kip_ft/ft	2.097	0.310	0.148	PASS
Out-of-plane shear stress	psi	43.6	1.7	0.038	PASS
In-plane axial force	kips	2290.7	28.7	0.013	PASS
In-plane axial stress	psi	624.8	7.2	0.012	PASS
In-plane flexural strength	kip_ft	11808.4	151.3	0.013	PASS
In-plane shear stress	psi	99.8	1.4	0.014	PASS
In-plane shear friction	psi	91.5	1.4	0.016	PASS
Biaxial bending				0.148	PASS



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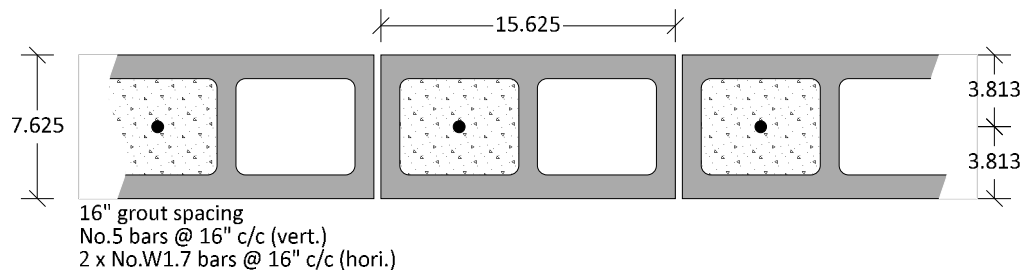
Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 149
Project Name: JKV-EMS		

Bottom: Max axial

Description	Unit	Provided	Required	Utilization	Result
Out-of-plane axial force	kips/ft	22.1	2.4	0.110	PASS
Out-of-plane axial stress	psi	357.8	36.1	0.101	PASS
Out-of-plane flexural strength	kip_ft/ft	2.403	0.007	0.003	PASS
Out-of-plane shear stress	psi	52.8	1.1	0.020	PASS
In-plane axial force	kips	2290.7	144.0	0.063	PASS
In-plane axial stress	psi	624.8	36.1	0.058	PASS
In-plane flexural strength	kip_ft	14452.5	273.0	0.019	PASS
In-plane shear stress	psi	79.9	1.4	0.018	PASS
In-plane shear friction	psi	114.3	1.4	0.013	PASS
Biaxial bending				0.019	PASS

Bottom: Min axial

Description	Unit	Provided	Required	Utilization	Result
Out-of-plane axial force	kips/ft	22.1	0.9	0.040	PASS
Out-of-plane axial stress	psi	357.8	13.0	0.036	PASS
Out-of-plane flexural strength	kip_ft/ft	2.250	0.007	0.003	PASS
Out-of-plane shear stress	psi	48.1	1.1	0.022	PASS
In-plane axial force	kips	2290.7	52.0	0.023	PASS
In-plane axial stress	psi	624.8	13.0	0.021	PASS
In-plane flexural strength	kip_ft	12344.2	273.0	0.022	PASS
In-plane shear stress	psi	79.9	1.4	0.018	PASS
In-plane shear friction	psi	96.1	1.4	0.015	PASS
Biaxial bending				0.022	PASS



Masonry details

8 in CMU in running bond, face bedded with S class PCL mortar on an unfinished bearing surface

Length of section;

$$L = 59.396 \text{ ft}$$

Effective height (minor axis);

$$h_{\text{eff},y} = 18.376 \text{ ft}$$

Effective height (major axis);

$$h_{\text{eff},x} = 37.854 \text{ ft}$$

Compressive strength of unit;

$$f_{cu} = 3250 \text{ psi}$$

Density of masonry units;

$$\rho_m = 130 \text{ lb/ft}^3$$



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 150
Project Name: JKV-EMS		

Thickness of masonry units; $t = 7.625$ in
 Height of masonry units; $h_m = 7.625$ in
 Length of masonry units; $l_m = 15.625$ in
 Number of internal webs; $n_{w,int} = 1$
 Internal web thickness; $t_{w,int} = 1$ in
 Number of end webs; $n_{w,ext} = 2$
 End web thickness; $t_{w,ext} = 1$ in
 Face shell thickness; $t_{fs} = 1.25$ in
 Depth of cavity; $t_c = t - 2 \times t_{fs} = 5.125$ in
 Length of cavity; $l_c = (l_m - n_{w,int} \times t_{w,int} - n_{w,ext} \times t_{w,ext}) / 2 = 6.313$ in
 Area of cavity; $A_c = t_c \times l_c = 32.4$ in²
 Spacing of grout; $s_{grout} = 16$ in
 Density of grout; $\rho_{grout} = 140$ lb/ft³

Compressive strength of masonry

Net compr. strength of masonry - TMS602 Table 2; $f'_m = 2500$ psi
 Modulus of elasticity for masonry - Table 4.2.2; $E_m = 900 \times f'_m = 2250000$ psi
 Shear modulus of masonry; $G_m = E_m / (2 \times (1 + \nu_m)) = 900000$ psi
 Max usable comp. strain in masonry; $\epsilon_{mu} = 0.00250$

Reinforcement details

Allow. tensile stress in vert. reinf. - cl.8.3.3 $F_s = 32000$ psi
 Yield strength of reinforcement $f_{y,vert} = 60000$ psi
 Modulus of elasticity for reinforcement $E_s = 29000000$ psi
 Tensile strain in reinforcement at balance point; $\epsilon_{ty} = f_{y,vert} / E_s = 0.002069$
 Vertical reinforcement provided No.5 bars @ 16" c/c
 Area of vertical reinforcement $A_s = \pi \times \phi_{vert}^2 / (4 \times s_{vert}) = 0.23$ in²/ft
 Max. allow. vertical reinf. diameter - cl.6.1.2.2/5 $\phi_{vert,max} = \min(\min(t, l_m) / 8, \min(t_c, l_c) / 2) = 0.95$ in

PASS - Maximum allowable vertical bar diameter exceeds provided

Nominal front cover; $C_{nom,front} = 3.500$ in

Neither face exposed to earth or weather

Minimum rear cover - cl.6.1.3.5/6.1.4.1; $C_{nom,min,rear} = \max(t_{grout,min} + t_{fs}, 1.5 \text{ in}) = 1.750$ in

Minimum front cover; $C_{nom,min,front} = \max(t_{grout,min} + t_{fs}, 1.5 \text{ in}) = 1.750$ in

PASS - Cover to reinforcement exceeds minimum cover required

Min allow. clear spacing between grout and unit; $t_{grout,min} = 0.500$ in

Clear spacing between grout and unit; $s_{vert,clear} = \min(t - (\phi_{vert} + C_{nom,front}), C_{nom,front} - t_{fs}) = 2.250$ in

PASS - Clear spacing between reinf. and masonry unit exceeds minimum allowable

Min req'd vert reinf in shear walls - cl.9.3.6.2; $A_{s,h,min} = A_v / 3 = 0.009$ in²/ft

PASS - Area of vertical reinforcement exceeds minimum required

Max allow vert spacing in shear walls - cl.9.3.6.2; $s_{vert,h,max} = 96.000$ in

PASS - Maximum allowable vertical reinforcement spacing exceeds provided

Allow. tensile stress in hori. reinf. - cl.8.3.3 $F_{sv} = 30000$ psi

Yield strength of horizontal reinforcement $f_{y,hori} = 70000$ psi

Horizontal reinforcement provided 2 x No.3.7592 wires @ 16" c/c

Area of horizontal reinforcement $A_v = 2 \times \pi \times \phi_{hori}^2 / (4 \times s_{hori}) = 0.03$ in²/ft

Max. allow. horizontal reinf. diameter - cl.6.1.2.2/5 $\phi_{hori,max} = t_j / 2 = 0.19$ in



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 151
Project Name: JKV-EMS		

Area of cell;
Area of reinforcement in cell;
Max allow. reinforcement in cell - cl.6.1.2.4;

PASS - Maximum allowable horizontal bar diameter exceeds provided

$$A_c = t_c \times l_c = \mathbf{32.352 \text{ in}^2}$$
$$A_{s, \text{cell}} = n_{\text{vert}} \times \phi_{\text{vert}}^2 \times \pi / 4 = \mathbf{0.307 \text{ in}^2}$$
$$A_{s, \text{max, cell}} = 0.06 \times A_c = \mathbf{1.941 \text{ in}^2}$$

PASS - Maximum allowable area of reinforcement in cell exceeds provided

Masonry section properties

Properties for walls loaded axially and out of plane

Gross cross-sectional area; $A = t - l_c \times t_c / (l_m + t_j) = \mathbf{67.2 \text{ in}^2/\text{ft}}$

Gross moment of inertia; $I = t^3 / 12 - l_c \times t_c^3 / (12 \times (l_m + t_j)) = \mathbf{390.2 \text{ in}^4/\text{ft}}$

Gross section modulus; $S = 2 \times I / t = \mathbf{102.4 \text{ in}^3/\text{ft}}$

Gross radius of gyration; $r = \sqrt{I / A} = \mathbf{2.4 \text{ in}}$

Net cross-sectional area; $A_n = 2 \times t_{fs} + t_c \times (l_c + 2 \times t_{w, \text{ext}}) / (l_m + t_j) = \mathbf{62 \text{ in}^2/\text{ft}}$

Net moment of inertia; $I_n = (t^3 - t_c^3) / 12 + ((l_c + 2 \times t_{w, \text{ext}}) \times t_c^3 / (12 \times (l_m + t_j))) = \mathbf{378.6 \text{ in}^4/\text{ft}}$

Net section modulus; $S_n = 2 \times I_n / t = \mathbf{99.3 \text{ in}^3/\text{ft}}$

Net radius of gyration; $r_n = \sqrt{I_n / A_n} = \mathbf{2.5 \text{ in}}$

Properties for walls loaded in-plane

Equivalent thickness; $t_{\text{equiv}} = A_n = \mathbf{5.163 \text{ in}}$

Net moment of inertia; $I_{x, \text{net}} = t_{\text{equiv}} \times L^3 / 12 = \mathbf{155775341 \text{ in}^4}$

Net section modulus; $S_{x, \text{net}} = I_{x, \text{net}} / (L / 2) = \mathbf{437111 \text{ in}^3}$

Net radius of gyration; $r_x = \sqrt{I_{x, \text{net}} / (A_n \times L)} = \mathbf{206 \text{ in}}$

Properties for horizontally spanning walls

Ratio of grouted bond beams to voids; $G_{\text{hori}} = 0 = \mathbf{0}$

Gross cross-sectional area; $A_{g, \text{hori}} = t = \mathbf{91.5 \text{ in}^2/\text{ft}}$

Gross moment of inertia; $I_{g, \text{hori}} = t^3 / 12 = \mathbf{443.3 \text{ in}^4/\text{ft}}$

Net cross-sectional area; $A_{n, \text{hori}} = G_{\text{hori}} \times t + (1 - G_{\text{hori}}) \times (2 \times t_{fs}) = \mathbf{30 \text{ in}^2/\text{ft}}$

Net moment of inertia; $I_{n, \text{hori}} = G_{\text{hori}} \times t^3 / 12 + (1 - G_{\text{hori}}) \times (t^3 / 12 - (t - 2 \times t_{fs})^3 / 12) = \mathbf{308.7 \text{ in}^4/\text{ft}}$

Net section modulus; $S_{n, \text{hori}} = 2 \times I_{n, \text{hori}} / t = \mathbf{81 \text{ in}^3/\text{ft}}$

Average out-of-plane section properties

Average cross-sectional area; $A_{\text{ave}} = (t_j \times A_n + h_m \times A) / (t_j + h_m) = \mathbf{67 \text{ in}^2/\text{ft}}$

Average moment of inertia; $I_{\text{ave}} = (t_j \times I_n + h_m \times I) / (t_j + h_m) = \mathbf{389.7 \text{ in}^4/\text{ft}}$

Average cross-sectional area; $A_{\text{ave, hori}} = (n_{w, \text{ext}} \times t_{w, \text{ext}} + n_{w, \text{int}} \times t_{w, \text{int}}) / (l_m + t_j) \times A_{g, \text{hori}} + (l_m + t_j - n_{w, \text{ext}} \times t_{w, \text{ext}} - n_{w, \text{int}} \times t_{w, \text{int}}) / (l_m + t_j) \times A_{n, \text{hori}} = \mathbf{41.5 \text{ in}^2/\text{ft}}$

Average moment of inertia; $I_{\text{ave, hori}} = (n_{w, \text{ext}} \times t_{w, \text{ext}} + n_{w, \text{int}} \times t_{w, \text{int}}) / (l_m + t_j) \times I_{g, \text{hori}} + (l_m + t_j - n_{w, \text{ext}} \times t_{w, \text{ext}} - n_{w, \text{int}} \times t_{w, \text{int}}) / (l_m + t_j) \times I_{n, \text{hori}} = \mathbf{334 \text{ in}^4/\text{ft}}$

Orthotropic properties for section adjustment

Primary bending plane; Major

Elastic modulus x - horizontal spanning minor axis; $E_x = E_m \times I_{\text{ave, hori}} / I_{g, \text{hori}} = \mathbf{1694904 \text{ psi}}$

Elastic modulus y - vertical spanning major axis; $E_y = E_m \times A_{\text{ave}} / t = \mathbf{1647260 \text{ psi}}$

In-plane shear modulus; $G_{xy} = G_m \times A_{\text{ave}} / t = \mathbf{658904 \text{ psi}}$

Out-of-plane shear modulus; $G_{xz} = G_{xy} = \mathbf{658904 \text{ psi}}$

Poison ratio; $\nu_{xy} = \mathbf{0.250}$



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 152
Project Name: JKV-EMS		

Average wall density;

$$\rho_{ave} = (A_m \times \rho_m + A_g \times \rho_{grout}) / t + 2.5 \text{ pcf} = \mathbf{100 \text{ lb/ft}^3}$$

Out-of-plane axial resistance

Slenderness ratio;

$$h_{eff,y} / r = \mathbf{91.532} < 99$$

Allowable axial force - Eqn.8-18;

$$P_{a\_pos} = (0.25 \times f'_m \times (A_n - A_s)) \times (1 - (h_{eff,y} / (140 \times r))^2) = \mathbf{22.09 \text{ kips/ft}}$$

Out-of-plane axial stress resistance

Slenderness ratio;

$$h_{eff,y} / r = \mathbf{91.532} < 99$$

Allowable axial stress - cl.8.2.4.1;

$$F_{a\_pos} = (1 / 4) \times f'_m \times (1 - (h_{eff,y} / (140 \times r))^2) = \mathbf{357.8 \text{ psi}}$$

Moment at balance point

Modular ratio;

$$n = E_s / E_m = \mathbf{12.89}$$

Allowable stress in masonry (cl.8.3.4.2.2);

$$F_b = 0.45 \times f'_m = \mathbf{1125 \text{ psi}}$$

Balance point;

$$k_b = n / (n + F_s / F_b) = \mathbf{0.312}$$

Tensile strain in reinforcement;

$$\epsilon_s = F_s / E_s = \mathbf{0.001103}$$

Compressive strain in masonry;

$$\epsilon_m = \epsilon_s \times k_b / (1 - k_b) = \mathbf{0.000500}$$

Effective depth of reinforcement;

$$d = \mathbf{3.813 \text{ in}}$$

Width of T section web;

$$b_w = (l_c + t_{w,ext} + t_{w,int}) / s_{vert} \times 12 \text{ in/ft} = \mathbf{0.520 \text{ ft/ft}}$$

Comp. stress in masonry due to flexure only;

$$f_b = F_s / n \times k_b \times d / (d - k_b \times d) = \mathbf{1125 \text{ psi}}$$

Stress in reinforcement;

$$f_s = F_s = \mathbf{32000 \text{ psi}}$$

Stress at compression side of web;

$$f_{b1} = (k_b \times d - t_{fs}) / (k_b \times d) \times f_b = \mathbf{-58 \text{ psi}}$$

Compression moment arm to face shell;

$$x_1 = d - k_b \times d / 3 = \mathbf{3.416 \text{ in}}$$

Compression in face shell;

$$C_1 = f_b / 2 \times k_b \times d = \mathbf{8.025 \text{ kips/ft}}$$

Compression at balance point;

$$C_b = C_1 = \mathbf{8.025 \text{ kips/ft}}$$

Force in tension face reinf. at balance point;

$$T_b = f_s \times A_s = \mathbf{7.363 \text{ kips/ft}}$$

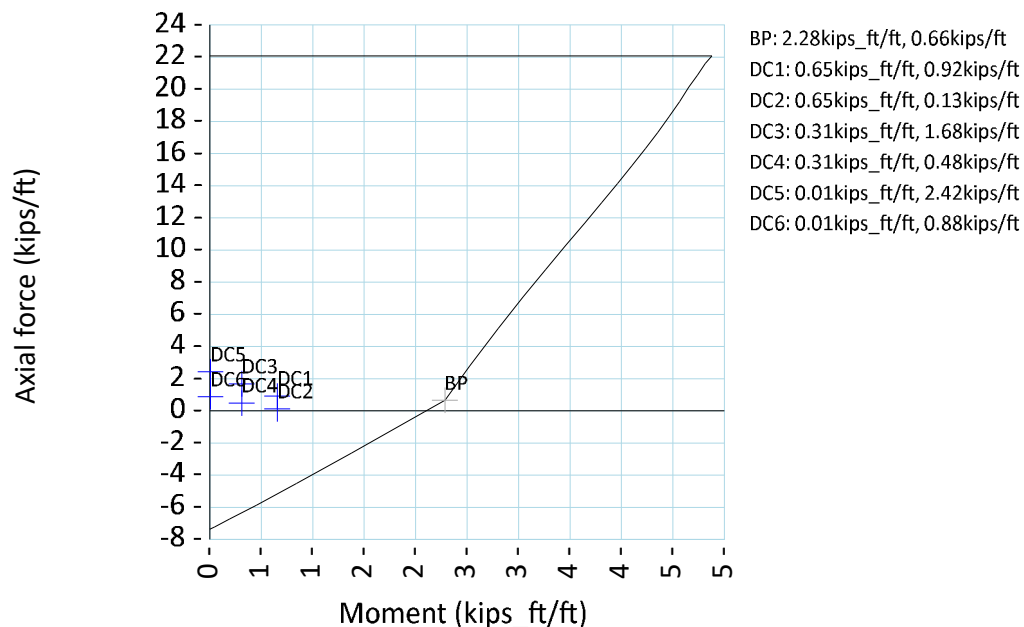
Axial load at balance point;

$$N_b = \min(C_1 - T_b, P_{a\_pos}) = \mathbf{0.662 \text{ kips/ft}}$$

Moment at balance point;

$$M_b = \max(C_1 \times x_1, 0 \text{ lb\_ft/ft}) = \mathbf{2.285 \text{ kips\_ft/ft}}$$

Out-of-plane interaction diagram





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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 153
Project Name: JKV-EMS		

In-plane axial resistance

In-plane axial force;

Depth to reinforcement

Net shear area

Slenderness ratio;

Allowable axial force - Eqn.8-19;

$$P_{in} = P \times L = \mathbf{52 \text{ kips}}$$

$$d_{in} = L - l_m / 4 = \mathbf{59.1 \text{ ft}}$$

$$A_{nv,in} = 2 \times d_{in} \times t_{fs} + d_{in} / s_{grout} \times (l_m / (n_{w,int} + 1)) \times (t - 2 \times t_{fs}) = \mathbf{3546 \text{ in}^2}$$

$$h_{eff,x} / r_x = \mathbf{2.208} < 99$$

$$P_{a,in,pos} = ((0.25 \times f'_m \times (A_n - A_s)) \times (1 - (h_{eff,x} / (140 \times r_x))^2) \times L) = \mathbf{2290.66 \text{ kips}}$$

In-plane axial stress resistance

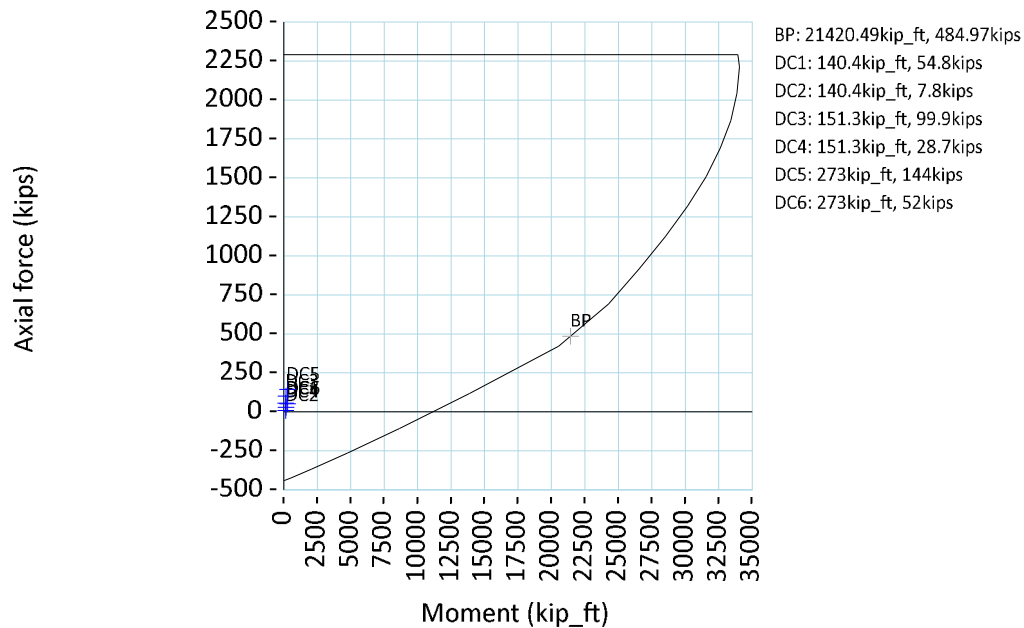
Slenderness ratio;

Allowable axial stress - cl.8.2.4.1;

$$h_{eff,x} / r_x = \mathbf{2.208} < 99$$

$$F_{a,in,pos} = (1 / 4) \times f'_m \times (1 - (h_{eff,x} / (140 \times r_x))^2) = \mathbf{624.8 \text{ psi}}$$

In-plane interaction diagram



From interaction diagram

Axial load at balance point;

Moment at balance point;

Maximum moment;

$$N_{b,in} = \mathbf{485 \text{ kips}}$$

$$M_{b,in} = \mathbf{21420 \text{ kip}_ft}$$

$$M_{max,in} = \mathbf{34046 \text{ kip}_ft}$$

Top: Max axial

Design forces

Axial load;

Axial eccentricity;

Out-of-plane moment;

Out-of-plane shear force;

In-plane moment;

In-plane Shear force;

$$P = \mathbf{0.923 \text{ kips/ft}}$$

$$e = \mathbf{0 \text{ in}}$$

$$M_y = \mathbf{-0.655 \text{ kips}_ft/ft}$$

$$V_y = \mathbf{0.056 \text{ kips/ft}}$$

$$M_x = \mathbf{-140.4 \text{ kips}_ft}$$

$$V_x = \mathbf{5 \text{ kips}}$$

Reinforcement effective depth

Effective depth of reinforcement;

$$d = \mathbf{3.813 \text{ in}}$$



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 154
Project Name: JKV-EMS		

Out-of-plane axial check

$$P / P_{a\_pos} = 0.042$$

PASS - Pure axial resistance exceeds axial load

Out-of-plane axial stress check

Stress due to axial load;

$$f_a = P / A = 13.7 \text{ psi}$$

$$f_a / F_{a\_pos} = 0.038$$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit} = 2.250 \text{ kips\_ft/ft}$$

$$Abs(M_y) / M_{limit} = 0.291$$

PASS - Design flexural strength exceeds factored bending moment

Out-of-plane shear check

Net shear area

$$A_{nv} = d \times l_m / ((n_{w,int} + 1) \times s_{grout}) = 22.34 \text{ in}^2/\text{ft}$$

Applied shear stress;

$$f_v = V_y / A_{nv} = 2.49 \text{ psi}$$

Shear resistance of masonry - Eqn 8-26;

$$F_{vm} = 1/2 \times (4 - 1.75 \times \min(Abs(M_y) / (V_y \times d), 1)) \times \sqrt{(f_m \times 1 \text{ psi})} + 0.25 \times P / A_n = 59.97 \text{ psi}$$

Grout factor - cl.8.3.5.1.2;

$$\gamma_g = 0.75$$

Overall shear resistance - cl.8.3.5.1.2;

$$F_v = \min(F_{vm}, 2 \times \sqrt{(f_m \times 1 \text{ psi})}) \times \gamma_g = 44.98 \text{ psi}$$

$$f_v / F_v = 0.055$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane axial check

$$P_{in} / P_{a,in\_pos} = 0.024$$

PASS - Pure axial resistance exceeds axial load

In-plane axial stress check

Stress due to axial load;

$$f_a = P / A = 13.7 \text{ psi}$$

$$f_a / F_{a,in\_pos} = 0.022$$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit,in} = 12414 \text{ kip\_ft}$$

$$Abs(M_x) / M_{limit,in} = 0.011$$

PASS - Design flexural strength exceeds factored bending moment

In-plane shear check

Shear stress;

$$f_{v,in} = V_x / A_{nv,in} = 1.41 \text{ psi}$$

Allowable shear stress of masonry;

$$F_{vm,in} = 1/2 \times (4 - 1.75 \times \min(Abs(M_x) / (V_x \times d_{in}), 1)) \times \sqrt{(f_m \times 1 \text{ psi})} + 0.25 \times P_{in} / (A_n \times L) = 82.93 \text{ psi}$$

Allowable shear stress of steel;

$$F_{vs,in} = 0.5 \times (A_v \times F_{sv} \times d_{in} / (A_{nv,in} \times d_{in} \times 1\text{ft})) = 380.89 \text{ psi}$$

Grout factor;

$$\gamma_g = 0.75$$

Allowable shear stress;

$$F_{v,in} = \min(F_{vm,in} + F_{vs,in}, 2.7 \times \sqrt{(f_m \times 1 \text{ psi})}) \times \gamma_g = 101.25 \text{ psi}$$

$$f_{v,in} / F_{v,in} = 0.014$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane shear friction - 8.3.6

Coefficient of friction

$$\mu = 0.7$$



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 155
Project Name: JKV-EMS		

Shear friction stress ($M_x / (V_x / d_{in})$ lte 0.5)
Shear friction stress ($M_x / (V_x / d_{in})$ gte 1)
Shear friction stress as $M_x / (V_x \times d_{in})$ lte 0.5

$$F_{f1} = \max(\mu \times (A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = \mathbf{96.7 \text{ psi}}$$
$$F_{f2} = \max(0.65 \times (0.6 \times A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = \mathbf{57.9 \text{ psi}}$$
$$F_f = F_{f1} = \mathbf{96.7 \text{ psi}}$$
$$f_{v,in} / F_f = \mathbf{0.015}$$

PASS - Allowable shear friction stress exceeds applied shear stress

Biaxial bending

$$\sqrt{((\text{abs}(M_y) / M_{limit})^2 + (\text{abs}(M_x) / M_{limit,in})^2)} = \mathbf{0.291}$$

PASS - Biaxial bending utilization is acceptable

Top: Min axial

Design forces

Axial load;
Axial eccentricity;
Out-of-plane moment;
Out-of-plane shear force;
In-plane moment;
In-plane Shear force;

$$P = \mathbf{0.131 \text{ kips/ft}}$$
$$e = \mathbf{0 \text{ in}}$$
$$M_y = \mathbf{-0.655 \text{ kips\_ft/ft}}$$
$$V_y = \mathbf{0.056 \text{ kips/ft}}$$
$$M_x = \mathbf{-140.4 \text{ kips\_ft}}$$
$$V_x = \mathbf{5 \text{ kips}}$$

Reinforcement effective depth

Effective depth of reinforcement;

$$d = \mathbf{3.813 \text{ in}}$$

Out-of-plane axial check

$$P / P_{a\_pos} = \mathbf{0.006}$$

PASS - Pure axial resistance exceeds axial load

Out-of-plane axial stress check

Stress due to axial load;

$$f_a = P / A = \mathbf{2.0 \text{ psi}}$$
$$f_a / F_{a\_pos} = \mathbf{0.005}$$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit} = \mathbf{2.097 \text{ kips\_ft/ft}}$$
$$\text{Abs}(M_y) / M_{limit} = \mathbf{0.312}$$

PASS - Design flexural strength exceeds factored bending moment

Out-of-plane shear check

Net shear area
Applied shear stress;
Shear resistance of masonry - Eqn 8-26;

$$A_{nv} = d \times l_m / ((n_{w,int} + 1) \times s_{grout}) = \mathbf{22.34 \text{ in}^2/\text{ft}}$$
$$f_v = V_y / A_{nv} = \mathbf{2.49 \text{ psi}}$$
$$F_{vm} = 1/2 \times (4 - 1.75 \times \min(\text{Abs}(M_y) / (V_y \times d), 1)) \times \sqrt{(f'_m \times 1 \text{ psi})} + 0.25 \times P / A_n = \mathbf{56.78 \text{ psi}}$$
$$\gamma_g = \mathbf{0.75}$$
$$F_v = \min(F_{vm}, 2 \times \sqrt{(f'_m \times 1 \text{ psi})}) \times \gamma_g = \mathbf{42.58 \text{ psi}}$$
$$f_v / F_v = \mathbf{0.058}$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane axial check

$$P_{in} / P_{a,in\_pos} = \mathbf{0.003}$$

PASS - Pure axial resistance exceeds axial load



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 156
Project Name: JKV-EMS		

In-plane axial stress check

Stress due to axial load;

$$f_a = P / A = 2.0 \text{ psi}$$

$$f_a / F_{a, \text{in\_pos}} = 0.003$$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{\text{limit, in}} = 11319 \text{ kip\_ft}$$

$$\text{Abs}(M_x) / M_{\text{limit, in}} = 0.012$$

PASS - Design flexural strength exceeds factored bending moment

In-plane shear check

Shear stress;

$$f_{v, \text{in}} = V_x / A_{nv, \text{in}} = 1.41 \text{ psi}$$

Allowable shear stress of masonry;

$$F_{vm, \text{in}} = 1/2 \times (4 - 1.75 \times \min(\text{Abs}(M_x) / (V_x \times d_{\text{in}}), 1)) \times \sqrt{(f_m \times 1 \text{ psi}) + 0.25 \times P_{\text{in}} / (A_n \times L)} = 79.73 \text{ psi}$$

Allowable shear stress of steel;

$$F_{vs, \text{in}} = 0.5 \times (A_v \times F_{sv} \times d_{\text{in}} / (A_{nv, \text{in}} \times d_{\text{in}} \times 1 \text{ ft})) = 380.89 \text{ psi}$$

Grout factor;

$$\gamma_g = 0.75$$

Allowable shear stress;

$$F_{v, \text{in}} = \min(F_{vm, \text{in}} + F_{vs, \text{in}}, 2.7 \times \sqrt{(f_m \times 1 \text{ psi})}) \times \gamma_g = 101.25 \text{ psi}$$

$$f_{v, \text{in}} / F_{v, \text{in}} = 0.014$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane shear friction - 8.3.6

Coefficient of friction

$$\mu = 0.7$$

Shear friction stress ($M_x / (V_x \times d_{\text{in}})$ lte 0.5)

$$F_{f1} = \max(\mu \times (A_s \times d_{\text{in}} \times F_s + P_{\text{in}}) / (A_{nv, \text{in}}), 0 \text{ psi}) = 87.4 \text{ psi}$$

Shear friction stress ($M_x / (V_x \times d_{\text{in}})$ gte 1)

$$F_{f2} = \max(0.65 \times (0.6 \times A_s \times d_{\text{in}} \times F_s + P_{\text{in}}) / (A_{nv, \text{in}}), 0 \text{ psi}) = 49.3 \text{ psi}$$

Shear friction stress as $M_x / (V_x \times d_{\text{in}})$ lte 0.5

$$F_f = F_{f1} = 87.4 \text{ psi}$$

$$f_{v, \text{in}} / F_f = 0.016$$

PASS - Allowable shear friction stress exceeds applied shear stress

Biaxial bending

$$\sqrt{((\text{abs}(M_y) / M_{\text{limit}})^2 + (\text{abs}(M_x) / M_{\text{limit, in}})^2)} = 0.313$$

PASS - Biaxial bending utilization is acceptable

Mid-fifth: Max axial

Design forces

Axial load;

$$P = 1.682 \text{ kips/ft}$$

Axial eccentricity;

$$e = 0 \text{ in}$$

Out-of-plane moment;

$$M_y = -0.310 \text{ kips\_ft/ft}$$

Out-of-plane shear force;

$$V_y = 0.037 \text{ kips/ft}$$

In-plane moment;

$$M_x = -151.3 \text{ kips\_ft}$$

In-plane Shear force;

$$V_x = 5.1 \text{ kips}$$

Reinforcement effective depth

Effective depth of reinforcement;

$$d = 3.813 \text{ in}$$

Out-of-plane axial check

$$P / P_{a, \text{pos}} = 0.076$$

PASS - Pure axial resistance exceeds axial load



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 157
Project Name: JKV-EMS		

Out-of-plane axial stress check

Stress due to axial load;

$$f_a = P / A = 25.0 \text{ psi}$$

$$f_a / F_{a\_pos} = 0.070$$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit} = 2.403 \text{ kips\_ft/ft}$$

$$Abs(M_y) / M_{limit} = 0.129$$

PASS - Design flexural strength exceeds factored bending moment

Out-of-plane shear check

Net shear area

$$A_{nv} = d \times l_m / ((n_{w,int} + 1) \times s_{grout}) = 22.34 \text{ in}^2/\text{ft}$$

Applied shear stress;

$$f_v = V_y / A_{nv} = 1.66 \text{ psi}$$

Shear resistance of masonry - Eqn 8-26;

$$F_{vm} = 1/2 \times (4 - 1.75 \times \min(Abs(M_y) / (V_y \times d), 1)) \times \sqrt{(f_m \times 1 \text{ psi}) + 0.25 \times P / A_n} = 63.04 \text{ psi}$$

Grout factor - cl.8.3.5.1.2;

$$\gamma_g = 0.75$$

Overall shear resistance - cl.8.3.5.1.2;

$$F_v = \min(F_{vm}, 2 \times \sqrt{(f_m \times 1 \text{ psi})}) \times \gamma_g = 47.28 \text{ psi}$$

$$f_v / F_v = 0.035$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane axial check

$$P_{in} / P_{a\_in\_pos} = 0.044$$

PASS - Pure axial resistance exceeds axial load

In-plane axial stress check

Stress due to axial load;

$$f_a = P / A = 25.0 \text{ psi}$$

$$f_a / F_{a\_in\_pos} = 0.040$$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit,in} = 13463 \text{ kip\_ft}$$

$$Abs(M_x) / M_{limit,in} = 0.011$$

PASS - Design flexural strength exceeds factored bending moment

In-plane shear check

Shear stress;

$$f_{v,in} = V_x / A_{nv,in} = 1.44 \text{ psi}$$

Allowable shear stress of masonry;

$$F_{vm,in} = 1/2 \times (4 - 1.75 \times \min(Abs(M_x) / (V_x \times d_{in}), 1)) \times \sqrt{(f_m \times 1 \text{ psi}) + 0.25 \times P_{in} / (A_n \times L)} = 84.81 \text{ psi}$$

Allowable shear stress of steel;

$$F_{vs,in} = 0.5 \times (A_v \times F_{sv} \times d_{in} / (A_{nv,in} / d_{in} \times 1\text{ft})) = 380.89 \text{ psi}$$

Grout factor;

$$\gamma_g = 0.75$$

Allowable shear stress;

$$F_{v,in} = \min(F_{vm,in} + F_{vs,in}, 2.66 \times \sqrt{(f_m \times 1 \text{ psi})}) \times \gamma_g = 99.75 \text{ psi}$$

$$f_{v,in} / F_{v,in} = 0.014$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane shear friction - 8.3.6

Coefficient of friction

$$\mu = 0.7$$

Shear friction stress ($M_x / (V_x / d_{in})$ lte 0.5)

$$F_{f1} = \max(\mu \times (A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = 105.6 \text{ psi}$$

Shear friction stress ($M_x / (V_x / d_{in})$ gte 1)

$$F_{f2} = \max(0.65 \times (0.6 \times A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = 66.1 \text{ psi}$$

Shear friction stress as $M_x / (V_x \times d_{in})$ lte 0.5

$$F_f = F_{f1} = 105.6 \text{ psi}$$



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 158
Project Name: JKV-EMS		

Biaxial bending

$$f_{v,in} / F_f = 0.014$$

PASS - Allowable shear friction stress exceeds applied shear stress

$$\sqrt{((abs(M_y) / M_{limit})^2 + (abs(M_x) / M_{limit,in})^2)} = 0.129$$

PASS - Biaxial bending utilization is acceptable

Mid-fifth: Min axial

Design forces

Axial load;

$$P = 0.483 \text{ kips/ft}$$

Axial eccentricity;

$$e = 0 \text{ in}$$

Out-of-plane moment;

$$M_y = -0.310 \text{ kips\_ft/ft}$$

Out-of-plane shear force;

$$V_y = 0.037 \text{ kips/ft}$$

In-plane moment;

$$M_x = -151.3 \text{ kips\_ft}$$

In-plane Shear force;

$$V_x = 5.1 \text{ kips}$$

Reinforcement effective depth

Effective depth of reinforcement;

$$d = 3.813 \text{ in}$$

Out-of-plane axial check

$$P / P_{a\_pos} = 0.022$$

PASS - Pure axial resistance exceeds axial load

Out-of-plane axial stress check

Stress due to axial load;

$$f_a = P / A = 7.2 \text{ psi}$$

$$f_a / F_{a\_pos} = 0.020$$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit} = 2.097 \text{ kips\_ft/ft}$$

$$Abs(M_y) / M_{limit} = 0.148$$

PASS - Design flexural strength exceeds factored bending moment

Out-of-plane shear check

Net shear area

$$A_{nv} = d \times l_m / ((n_{w,int} + 1) \times S_{grout}) = 22.34 \text{ in}^2/\text{ft}$$

Applied shear stress;

$$f_v = V_y / A_{nv} = 1.66 \text{ psi}$$

Shear resistance of masonry - Eqn 8-26;

$$F_{vm} = 1/2 \times (4 - 1.75 \times \min(Abs(M_y) / (V_y \times d), 1)) \times \sqrt{(f'_m \times 1 \text{ psi})} + 0.25 \times P / A_n = 58.2 \text{ psi}$$

Grout factor - cl.8.3.5.1.2;

$$\gamma_g = 0.75$$

Overall shear resistance - cl.8.3.5.1.2;

$$F_v = \min(F_{vm}, 2 \times \sqrt{(f'_m \times 1 \text{ psi})}) \times \gamma_g = 43.65 \text{ psi}$$

$$f_v / F_v = 0.038$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane axial check

$$P_{in} / P_{a,in\_pos} = 0.013$$

PASS - Pure axial resistance exceeds axial load

In-plane axial stress check

Stress due to axial load;

$$f_a = P / A = 7.2 \text{ psi}$$

$$f_a / F_{a,in\_pos} = 0.012$$

PASS - Allowable stress exceeds compressive stress due to axial load



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Revision:	Date:	Sheet No. 159
Project Name: JKV-EMS		

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit,in} = 11808 \text{ kip\_ft}$$

$$Abs(M_x) / M_{limit,in} = 0.013$$

PASS - Design flexural strength exceeds factored bending moment

In-plane shear check

Shear stress;

$$f_{v,in} = V_x / A_{nv,in} = 1.44 \text{ psi}$$

Allowable shear stress of masonry;

$$F_{vm,in} = 1/2 \times (4 - 1.75 \times \min(Abs(M_x) / (V_x \times d_{in}), 1)) \times \sqrt{f'_m \times 1 \text{ psi}} + 0.25 \times$$

$$P_{in} / (A_n \times L) = 79.98 \text{ psi}$$

Allowable shear stress of steel;

$$F_{vs,in} = 0.5 \times (A_v \times F_{sv} \times d_{in} / (A_{nv,in} \times d_{in} \times 1 \text{ ft})) = 380.89 \text{ psi}$$

Grout factor;

$$\gamma_g = 0.75$$

Allowable shear stress;

$$F_{v,in} = \min(F_{vm,in} + F_{vs,in}, 2.66 \times \sqrt{f'_m \times 1 \text{ psi}}) \times \gamma_g = 99.75 \text{ psi}$$

$$f_{v,in} / F_{v,in} = 0.014$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane shear friction - 8.3.6

Coefficient of friction

$$\mu = 0.7$$

Shear friction stress ($M_x / (V_x \times d_{in})$ lte 0.5)

$$F_{r1} = \max(\mu \times (A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = 91.5 \text{ psi}$$

Shear friction stress ($M_x / (V_x \times d_{in})$ gte 1)

$$F_{r2} = \max(0.65 \times (0.6 \times A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = 53.1 \text{ psi}$$

Shear friction stress as $M_x / (V_x \times d_{in})$ lte 0.5

$$F_r = F_{r1} = 91.5 \text{ psi}$$

$$f_{v,in} / F_r = 0.016$$

PASS - Allowable shear friction stress exceeds applied shear stress

Biaxial bending

$$\sqrt{((Abs(M_y) / M_{limit})^2 + (Abs(M_x) / M_{limit,in})^2)} = 0.148$$

PASS - Biaxial bending utilization is acceptable

Bottom: Max axial

Design forces

Axial load;

$$P = 2.424 \text{ kips/ft}$$

Axial eccentricity;

$$e = 0 \text{ in}$$

Out-of-plane moment;

$$M_y = 0.007 \text{ kips\_ft/ft}$$

Out-of-plane shear force;

$$V_y = 0.024 \text{ kips/ft}$$

In-plane moment;

$$M_x = -273 \text{ kips\_ft}$$

In-plane Shear force;

$$V_x = 5.1 \text{ kips}$$

Reinforcement effective depth

Effective depth of reinforcement;

$$d = 3.813 \text{ in}$$

Out-of-plane axial check

$$P / P_{a,pos} = 0.110$$

PASS - Pure axial resistance exceeds axial load

Out-of-plane axial stress check

Stress due to axial load;

$$f_a = P / A = 36.1 \text{ psi}$$

$$f_a / F_{a,pos} = 0.101$$

PASS - Allowable stress exceeds compressive stress due to axial load



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 160
Project Name: JKV-EMS		

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit} = 2.403 \text{ kips\_ft/ft}$$

$$Abs(M_y) / M_{limit} = 0.003$$

PASS - Design flexural strength exceeds factored bending moment

Out-of-plane shear check

Net shear area

$$A_{nv} = d \times l_m / ((n_{w,int} + 1) \times s_{grout}) = 22.34 \text{ in}^2/\text{ft}$$

Applied shear stress;

$$f_v = V_y / A_{nv} = 1.06 \text{ psi}$$

Shear resistance of masonry - Eqn 8-26;

$$F_{vm} = 1/2 \times (4 - 1.75 \times \min(Abs(M_y) / (V_y \times d), 1)) \times \sqrt{(f_m \times 1 \text{ psi})} + 0.25 \times P / A_n = 70.44 \text{ psi}$$

Grout factor - cl.8.3.5.1.2;

$$\gamma_g = 0.75$$

Overall shear resistance - cl.8.3.5.1.2;

$$F_v = \min(F_{vm}, 2.134 \times \sqrt{(f_m \times 1 \text{ psi})}) \times \gamma_g = 52.83 \text{ psi}$$

$$f_v / F_v = 0.02$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane axial check

$$P_{in} / P_{a,in\_pos} = 0.063$$

PASS - Pure axial resistance exceeds axial load

In-plane axial stress check

Stress due to axial load;

$$f_a = P / A = 36.1 \text{ psi}$$

$$f_a / F_{a,in\_pos} = 0.058$$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit,in} = 14452 \text{ kip\_ft}$$

$$Abs(M_x) / M_{limit,in} = 0.019$$

PASS - Design flexural strength exceeds factored bending moment

In-plane shear check

Shear stress;

$$f_{v,in} = V_x / A_{nv,in} = 1.44 \text{ psi}$$

Allowable shear stress of masonry;

$$F_{vm,in} = 1/2 \times (4 - 1.75 \times \min(Abs(M_x) / (V_x \times d_{in}), 1)) \times \sqrt{(f_m \times 1 \text{ psi})} + 0.25 \times P_{in} / (A_n \times L) = 70.14 \text{ psi}$$

Allowable shear stress of steel;

$$F_{vs,in} = 0.5 \times (A_v \times F_{sv} \times d_{in} / (A_{nv,in} \times d_{in} \times 1\text{ft})) = 380.89 \text{ psi}$$

Grout factor;

$$\gamma_g = 0.75$$

Allowable shear stress;

$$F_{v,in} = \min(F_{vm,in} + F_{vs,in}, 2.13 \times \sqrt{(f_m \times 1 \text{ psi})}) \times \gamma_g = 79.87 \text{ psi}$$

$$f_{v,in} / F_{v,in} = 0.018$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane shear friction - 8.3.6

Coefficient of friction

$$\mu = 0.7$$

Shear friction stress ($M_x / (V_x \times d_{in})$ lte 0.5)

$$F_{f1} = \max(\mu \times (A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = 114.3 \text{ psi}$$

Shear friction stress ($M_x / (V_x \times d_{in})$ gte 1)

$$F_{f2} = \max(0.65 \times (0.6 \times A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = 74.2 \text{ psi}$$

Shear friction stress as $M_x / (V_x \times d_{in})$ lte 0.5

$$F_f = F_{f1} = 114.3 \text{ psi}$$

$$f_{v,in} / F_f = 0.013$$

PASS - Allowable shear friction stress exceeds applied shear stress

Biaxial bending

$$\sqrt{((abs(M_y) / M_{limit})^2 + (abs(M_x) / M_{limit,in})^2)} = 0.019$$



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 161
Project Name: JKV-EMS		

PASS - Biaxial bending utilization is acceptable

Bottom: Min axial

Design forces

Axial load; $P = 0.875$ kips/ft
Axial eccentricity; $e = 0$ in
Out-of-plane moment; $M_y = 0.007$ kips_ft/ft
Out-of-plane shear force; $V_y = 0.024$ kips/ft
In-plane moment; $M_x = -273$ kips_ft
In-plane Shear force; $V_x = 5.1$ kips

Reinforcement effective depth

Effective depth of reinforcement; $d = 3.813$ in

Out-of-plane axial check

$$P / P_{a\_pos} = 0.040$$

PASS - Pure axial resistance exceeds axial load

Out-of-plane axial stress check

Stress due to axial load; $f_a = P / A = 13.0$ psi
 $f_a / F_{a\_pos} = 0.036$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load; $M_{limit} = 2.250$ kips_ft/ft
 $Abs(M_y) / M_{limit} = 0.003$

PASS - Design flexural strength exceeds factored bending moment

Out-of-plane shear check

Net shear area $A_{nv} = d \times l_m / ((n_{w,int} + 1) \times S_{grout}) = 22.34$ in²/ft
Applied shear stress; $f_v = V_y / A_{nv} = 1.06$ psi
Shear resistance of masonry - Eqn 8-26; $F_{vm} = 1/2 \times (4 - 1.75 \times \min(Abs(M_y) / (V_y \times d), 1)) \times \sqrt{(f'_m \times 1 \text{ psi})} + 0.25 \times P / A_n = 64.19$ psi
Grout factor - cl.8.3.5.1.2; $\gamma_g = 0.75$
Overall shear resistance - cl.8.3.5.1.2; $F_v = \min(F_{vm}, 2.134 \times \sqrt{(f'_m \times 1 \text{ psi})}) \times \gamma_g = 48.14$ psi
 $f_v / F_v = 0.022$

PASS - Allowable shear stress exceeds applied shear stress

In-plane axial check

$$P_{in} / P_{a\_in\_pos} = 0.023$$

PASS - Pure axial resistance exceeds axial load

In-plane axial stress check

Stress due to axial load; $f_a = P / A = 13.0$ psi
 $f_a / F_{a\_in\_pos} = 0.021$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load; $M_{limit,in} = 12344$ kip_ft
 $Abs(M_x) / M_{limit,in} = 0.022$

PASS - Design flexural strength exceeds factored bending moment



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 162
Project Name: JKV-EMS		

In-plane shear check

Shear stress;

Allowable shear stress of masonry;

Allowable shear stress of steel;

Grout factor;

Allowable shear stress;

$$f_{v,in} = V_x / A_{nv,in} = \mathbf{1.44 \text{ psi}}$$

$$F_{vm,in} = 1/2 \times (4 - 1.75 \times \min(\text{Abs}(M_x) / (V_x \times d_{in}), 1)) \times \sqrt{f_m \times 1 \text{ psi}} + 0.25 \times P_{in} / (A_n \times L) = \mathbf{63.89 \text{ psi}}$$

$$F_{vs,in} = 0.5 \times (A_v \times F_{sv} \times d_{in} / (A_{nv,in} \times d_{in} \times 1\text{ft})) = \mathbf{380.89 \text{ psi}}$$

$$\gamma_g = \mathbf{0.75}$$

$$F_{v,in} = \min(F_{vm,in} + F_{vs,in}, 2.13 \times \sqrt{f_m \times 1 \text{ psi}}) \times \gamma_g = \mathbf{79.87 \text{ psi}}$$

$$f_{v,in} / F_{v,in} = \mathbf{0.018}$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane shear friction - 8.3.6

Coefficient of friction

Shear friction stress ($M_x / (V_x \times d_{in})$ lte 0.5)

Shear friction stress ($M_x / (V_x \times d_{in})$ gte 1)

Shear friction stress as $M_x / (V_x \times d_{in})$ lte 0.5

$$\mu = \mathbf{0.7}$$

$$F_{f1} = \max(\mu \times (A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = \mathbf{96.1 \text{ psi}}$$

$$F_{f2} = \max(0.65 \times (0.6 \times A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = \mathbf{57.4 \text{ psi}}$$

$$F_f = F_{f1} = \mathbf{96.1 \text{ psi}}$$

$$f_{v,in} / F_f = \mathbf{0.015}$$

PASS - Allowable shear friction stress exceeds applied shear stress

Biaxial bending

$$\sqrt{(\text{abs}(M_y) / M_{limit})^2 + (\text{abs}(M_x) / M_{limit,in})^2} = \mathbf{0.022}$$

PASS - Biaxial bending utilization is acceptable

MASONRY WALL SECTION DESIGN - WP20

In accordance with TMS-16 using the allowable stress design method

Tedds calculation version 1.0.06

Design summary

Overall design status;

PASS

Overall design utilization;

0.132

Top: Max axial

Description	Unit	Provided	Required	Utilization	Result
Out-of-plane axial force	kips/ft	22.1	0.9	0.041	PASS
Out-of-plane axial stress	psi	357.8	13.6	0.038	PASS
Out-of-plane flexural strength	kip_ft/ft	1.016	0.009	0.009	PASS
In-plane axial force	kips	1226.8	29.0	0.024	PASS
In-plane axial stress	psi	624.5	13.6	0.022	PASS
In-plane flexural strength	kip_ft	1644.8	105.3	0.064	PASS
In-plane shear stress	psi	94.5	2.9	0.031	PASS
In-plane shear friction	psi	39.4	2.9	0.074	PASS
Biaxial bending				0.065	PASS



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Revision:	Date:	Sheet No. 163
Project Name: JKV-EMS		

Top: Min axial

Description	Unit	Provided	Required	Utilization	Result
Out-of-plane axial force	kips/ft	22.1	0.1	0.006	PASS
Out-of-plane axial stress	psi	357.8	2.0	0.006	PASS
Out-of-plane flexural strength	kip_ft/ft	0.730	0.009	0.013	PASS
In-plane axial force	kips	1226.8	4.3	0.004	PASS
In-plane axial stress	psi	624.5	2.0	0.003	PASS
In-plane flexural strength	kip_ft	1306.3	105.3	0.081	PASS
In-plane shear stress	psi	94.5	2.9	0.031	PASS
In-plane shear friction	psi	30.2	2.9	0.096	PASS
Biaxial bending				0.082	PASS

Mid-fifth: Max axial

Description	Unit	Provided	Required	Utilization	Result
Out-of-plane axial force	kips/ft	22.1	1.4	0.061	PASS
Out-of-plane axial stress	psi	357.8	20.1	0.056	PASS
Out-of-plane flexural strength	kip_ft/ft	1.016	0.028	0.028	PASS
Out-of-plane shear stress	psi	46.3	0.3	0.006	PASS
In-plane axial force	kips	1226.8	42.9	0.035	PASS
In-plane axial stress	psi	624.5	20.1	0.032	PASS
In-plane flexural strength	kip_ft	1829.5	123.7	0.068	PASS
In-plane shear stress	psi	88.5	2.9	0.032	PASS
In-plane shear friction	psi	44.5	2.9	0.064	PASS
Biaxial bending				0.073	PASS

Mid-fifth: Min axial

Description	Unit	Provided	Required	Utilization	Result
Out-of-plane axial force	kips/ft	22.1	0.3	0.015	PASS
Out-of-plane axial stress	psi	357.8	4.9	0.014	PASS
Out-of-plane flexural strength	kip_ft/ft	0.730	0.028	0.039	PASS
Out-of-plane shear stress	psi	43.2	0.3	0.007	PASS
In-plane axial force	kips	1226.8	10.5	0.009	PASS
In-plane axial stress	psi	624.5	4.9	0.008	PASS
In-plane flexural strength	kip_ft	1403.7	123.7	0.088	PASS
In-plane shear stress	psi	88.5	2.9	0.032	PASS
In-plane shear friction	psi	32.5	2.9	0.088	PASS
Biaxial bending				0.096	PASS



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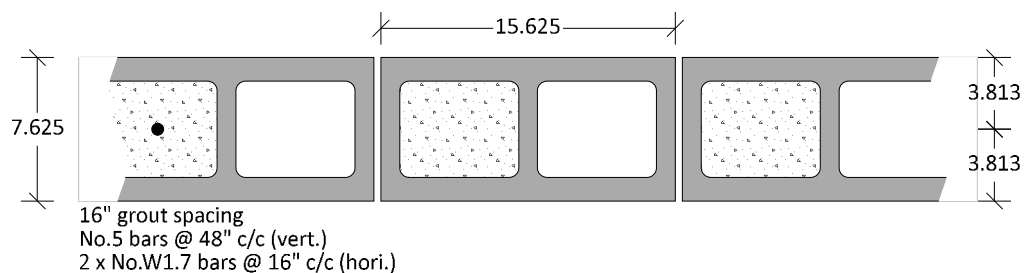
Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 164
Project Name: JKV-EMS		

Bottom: Max axial

Description	Unit	Provided	Required	Utilization	Result
Out-of-plane axial force	kips/ft	22.1	1.5	0.070	PASS
Out-of-plane axial stress	psi	357.8	23.0	0.064	PASS
Out-of-plane flexural strength	kip_ft/ft	1.299	0.006	0.005	PASS
In-plane axial force	kips	1226.8	49.2	0.040	PASS
In-plane axial stress	psi	624.5	23.0	0.037	PASS
In-plane flexural strength	kip_ft	1908.6	177.0	0.093	PASS
In-plane shear stress	psi	78.8	3.2	0.041	PASS
In-plane shear friction	psi	46.9	3.2	0.069	PASS
Biaxial bending				0.093	PASS

Bottom: Min axial

Description	Unit	Provided	Required	Utilization	Result
Out-of-plane axial force	kips/ft	22.1	0.2	0.009	PASS
Out-of-plane axial stress	psi	357.8	3.1	0.009	PASS
Out-of-plane flexural strength	kip_ft/ft	0.730	0.006	0.009	PASS
In-plane axial force	kips	1226.8	6.6	0.005	PASS
In-plane axial stress	psi	624.5	3.1	0.005	PASS
In-plane flexural strength	kip_ft	1348.0	177.0	0.131	PASS
In-plane shear stress	psi	78.8	3.2	0.041	PASS
In-plane shear friction	psi	31.1	3.2	0.104	PASS
Biaxial bending				0.132	PASS



Masonry details

8 in CMU in running bond, face bedded with S class PCL mortar on an unfinished bearing surface

Length of section;	$L = 31.75$ ft
Effective height (minor axis);	$h_{eff,y} = 18.376$ ft
Effective height (major axis);	$h_{eff,x} = 37.854$ ft
Compressive strength of unit;	$f_{cu} = 3250$ psi
Density of masonry units;	$\rho_m = 130$ lb/ft ³
Thickness of masonry units;	$t = 7.625$ in
Height of masonry units;	$h_m = 7.625$ in



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 165
Project Name: JKV-EMS		

Length of masonry units; $l_m = 15.625$ in
Number of internal webs; $n_{w,int} = 1$
Internal web thickness; $t_{w,int} = 1$ in
Number of end webs; $n_{w,ext} = 2$
End web thickness; $t_{w,ext} = 1$ in
Face shell thickness; $t_{fs} = 1.25$ in
Depth of cavity; $t_c = t - 2 \times t_{fs} = 5.125$ in
Length of cavity; $l_c = (l_m - n_{w,int} \times t_{w,int} - n_{w,ext} \times t_{w,ext}) / 2 = 6.313$ in
Area of cavity; $A_c = t_c \times l_c = 32.4$ in²
Spacing of grout; $s_{grout} = 16$ in
Density of grout; $\rho_{grout} = 140$ lb/ft³

Compressive strength of masonry

Net compr. strength of masonry - TMS602 Table 2; $f_m = 2500$ psi
Modulus of elasticity for masonry - Table 4.2.2; $E_m = 900 \times f_m = 2250000$ psi
Shear modulus of masonry; $G_m = E_m / (2 \times (1 + \nu_m)) = 900000$ psi
Max usable comp. strain in masonry; $\epsilon_{mu} = 0.00250$

Reinforcement details

Allow. tensile stress in vert. reinf. - cl.8.3.3 $F_s = 32000$ psi
Yield strength of reinforcement $f_{y,vert} = 60000$ psi
Modulus of elasticity for reinforcement $E_s = 29000000$ psi
Tensile strain in reinforcement at balance point; $\epsilon_{ty} = f_{y,vert} / E_s = 0.002069$
Vertical reinforcement provided No.5 bars @ 48" c/c
Area of vertical reinforcement $A_s = \pi \times \phi_{vert}^2 / (4 \times s_{vert}) = 0.08$ in²/ft
Max. allow. vertical reinf. diameter - cl.6.1.2.2/5 $\phi_{vert,max} = \min(\min(t, l_m) / 8, \min(t_c, l_c) / 2) = 0.95$ in

PASS - Maximum allowable vertical bar diameter exceeds provided

Nominal front cover; $C_{nom,front} = 3.500$ in
Neither face exposed to earth or weather
Minimum rear cover - cl.6.1.3.5/6.1.4.1; $C_{nom,min,rear} = \max(t_{grout,min} + t_{fs}, 1.5 \text{ in}) = 1.750$ in
Minimum front cover; $C_{nom,min,front} = \max(t_{grout,min} + t_{fs}, 1.5 \text{ in}) = 1.750$ in

PASS - Cover to reinforcement exceeds minimum cover required

Min allow. clear spacing between grout and unit; $t_{grout,min} = 0.500$ in
Clear spacing between grout and unit; $s_{vert,clear} = \min(t - (\phi_{vert} + C_{nom,front}), C_{nom,front} - t_{fs}) = 2.250$ in

PASS - Clear spacing between reinf. and masonry unit exceeds minimum allowable

Min req'd vert reinf in shear walls - cl.9.3.6.2; $A_{s,h,min} = A_v / 3 = 0.009$ in²/ft

PASS - Area of vertical reinforcement exceeds minimum required

Max allow vert spacing in shear walls - cl.9.3.6.2; $s_{vert,h,max} = 96.000$ in

PASS - Maximum allowable vertical reinforcement spacing exceeds provided

Allow. tensile stress in hori. reinf. - cl.8.3.3 $F_{sv} = 30000$ psi
Yield strength of horizontal reinforcement $f_{y,hori} = 70000$ psi
Horizontal reinforcement provided 2 x No.3.7592 wires @ 16" c/c
Area of horizontal reinforcement $A_v = 2 \times \pi \times \phi_{hori}^2 / (4 \times s_{hori}) = 0.03$ in²/ft
Max. allow. horizontal reinf. diameter - cl.6.1.2.2/5 $\phi_{hori,max} = t_j / 2 = 0.19$ in

PASS - Maximum allowable horizontal bar diameter exceeds provided



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 166
Project Name: JKV-EMS		

Area of cell;
Area of reinforcement in cell;
Max allow. reinforcement in cell - cl.6.1.2.4;

$$A_c = t_c \times l_c = \mathbf{32.352 \text{ in}^2}$$

$$A_{s,\text{cell}} = n_{\text{vert}} \times \phi_{\text{vert}}^2 \times \pi / 4 = \mathbf{0.307 \text{ in}^2}$$

$$A_{s,\text{max,cell}} = 0.06 \times A_c = \mathbf{1.941 \text{ in}^2}$$

PASS - Maximum allowable area of reinforcement in cell exceeds provided

Masonry section properties

Properties for walls loaded axially and out of plane

Gross cross-sectional area; $A = t - l_c \times t_c / (l_m + t_j) = \mathbf{67.2 \text{ in}^2/\text{ft}}$

Gross moment of inertia; $I = t^3 / 12 - l_c \times t_c^3 / (12 \times (l_m + t_j)) = \mathbf{390.2 \text{ in}^4/\text{ft}}$

Gross section modulus; $S = 2 \times I / t = \mathbf{102.4 \text{ in}^3/\text{ft}}$

Gross radius of gyration; $r = \sqrt{(I / A)} = \mathbf{2.4 \text{ in}}$

Net cross-sectional area; $A_n = 2 \times t_{fs} + t_c \times (l_c + 2 \times t_{w,\text{ext}}) / (l_m + t_j) = \mathbf{62 \text{ in}^2/\text{ft}}$

Net moment of inertia; $I_n = (t^3 - t_c^3) / 12 + ((l_c + 2 \times t_{w,\text{ext}}) \times t_c^3 / (12 \times (l_m + t_j))) = \mathbf{378.6 \text{ in}^4/\text{ft}}$

Net section modulus; $S_n = 2 \times I_n / t = \mathbf{99.3 \text{ in}^3/\text{ft}}$

Net radius of gyration; $r_n = \sqrt{(I_n / A_n)} = \mathbf{2.5 \text{ in}}$

Properties for walls loaded in-plane

Equivalent thickness; $t_{\text{equiv}} = A_n = \mathbf{5.163 \text{ in}}$

Net moment of inertia; $I_{x\_net} = t_{\text{equiv}} \times L^3 / 12 = \mathbf{23793699 \text{ in}^4}$

Net section modulus; $S_{x\_net} = I_{x\_net} / (L / 2) = \mathbf{124901 \text{ in}^3}$

Net radius of gyration; $r_x = \sqrt{(I_{x\_net} / (A_n \times L))} = \mathbf{110 \text{ in}}$

Properties for horizontally spanning walls

Ratio of grouted bond beams to voids; $G_{\text{hori}} = 0 = \mathbf{0}$

Gross cross-sectional area; $A_{g\_hori} = t = \mathbf{91.5 \text{ in}^2/\text{ft}}$

Gross moment of inertia; $I_{g\_hori} = t^3 / 12 = \mathbf{443.3 \text{ in}^4/\text{ft}}$

Net cross-sectional area; $A_{n\_hori} = G_{\text{hori}} \times t + (1 - G_{\text{hori}}) \times (2 \times t_{fs}) = \mathbf{30 \text{ in}^2/\text{ft}}$

Net moment of inertia; $I_{n\_hori} = G_{\text{hori}} \times t^3 / 12 + (1 - G_{\text{hori}}) \times (t^3 / 12 - (t - 2 \times t_{fs})^3 / 12) = \mathbf{308.7 \text{ in}^4/\text{ft}}$

Net section modulus; $S_{n\_hori} = 2 \times I_{n\_hori} / t = \mathbf{81 \text{ in}^3/\text{ft}}$

Average out-of-plane section properties

Average cross-sectional area; $A_{\text{ave}} = (t_j \times A_n + h_m \times A) / (t_j + h_m) = \mathbf{67 \text{ in}^2/\text{ft}}$

Average moment of inertia; $I_{\text{ave}} = (t_j \times I_n + h_m \times I) / (t_j + h_m) = \mathbf{389.7 \text{ in}^4/\text{ft}}$

Average cross-sectional area; $A_{\text{ave\_hori}} = (n_{w,\text{ext}} \times t_{w,\text{ext}} + n_{w,\text{int}} \times t_{w,\text{int}}) / (l_m + t_j) \times A_{g\_hori} + (l_m + t_j - n_{w,\text{ext}} \times t_{w,\text{ext}} - n_{w,\text{int}} \times t_{w,\text{int}}) / (l_m + t_j) \times A_{n\_hori} = \mathbf{41.5 \text{ in}^2/\text{ft}}$

Average moment of inertia; $I_{\text{ave\_hori}} = (n_{w,\text{ext}} \times t_{w,\text{ext}} + n_{w,\text{int}} \times t_{w,\text{int}}) / (l_m + t_j) \times I_{g\_hori} + (l_m + t_j - n_{w,\text{ext}} \times t_{w,\text{ext}} - n_{w,\text{int}} \times t_{w,\text{int}}) / (l_m + t_j) \times I_{n\_hori} = \mathbf{334 \text{ in}^4/\text{ft}}$

Orthotropic properties for section adjustment

Primary bending plane; Major

Elastic modulus x - horizontal spanning minor axis; $E_x = E_m \times I_{\text{ave\_hori}} / I_{g\_hori} = \mathbf{1694904 \text{ psi}}$

Elastic modulus y - vertical spanning major axis; $E_y = E_m \times A_{\text{ave}} / t = \mathbf{1647260 \text{ psi}}$

In-plane shear modulus; $G_{xy} = G_m \times A_{\text{ave}} / t = \mathbf{658904 \text{ psi}}$

Out-of-plane shear modulus; $G_{xz} = G_{xy} = \mathbf{658904 \text{ psi}}$

Poisson ratio; $\nu_{xy} = \mathbf{0.250}$

Average wall density; $\rho_{\text{ave}} = (A_m \times \rho_m + A_g \times \rho_{\text{grout}}) / t + 2.5 \text{ pcf} = \mathbf{100 \text{ lb/ft}^3}$



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 167
Project Name: JKV-EMS		

Out-of-plane axial resistance

Slenderness ratio;

$$h_{eff,y} / r = \mathbf{91.532}; < 99$$

Allowable axial force - Eqn.8-18;

$$P_{a\_pos} = (0.25 \times f_m \times (A_n - A_s)) \times (1 - (h_{eff,y} / (140 \times r))^2) = \mathbf{22.14 \text{ kips/ft}}$$

Out-of-plane axial stress resistance

Slenderness ratio;

$$h_{eff,y} / r = \mathbf{91.532}; < 99$$

Allowable axial stress - cl.8.2.4.1;

$$F_{a\_pos} = (1 / 4) \times f_m \times (1 - (h_{eff,y} / (140 \times r))^2) = \mathbf{357.8 \text{ psi}}$$

Moment at balance point

Modular ratio;

$$n = E_s / E_m = \mathbf{12.89}$$

Allowable stress in masonry (cl.8.3.4.2.2);

$$F_b = 0.45 \times f_m = \mathbf{1125 \text{ psi}}$$

Balance point;

$$k_b = n / (n + F_s / F_b) = \mathbf{0.312}$$

Tensile strain in reinforcement;

$$\epsilon_s = F_s / E_s = \mathbf{0.001103}$$

Compressive strain in masonry;

$$\epsilon_m = \epsilon_s \times k_b / (1 - k_b) = \mathbf{0.000500}$$

Effective depth of reinforcement;

$$d = \mathbf{3.813 \text{ in}}$$

Width of T section web;

$$b_w = (l_c + t_{w,ext} + t_{w,int}) / s_{vert} \times 12 \text{ in/ft} = \mathbf{0.173 \text{ ft/ft}}$$

Comp. stress in masonry due to flexure only;

$$f_b = F_s / n \times k_b \times d / (d - k_b \times d) = \mathbf{1125 \text{ psi}}$$

Stress in reinforcement;

$$f_s = F_s = \mathbf{32000 \text{ psi}}$$

Stress at compression side of web;

$$f_{b1} = (k_b \times d - t_{fs}) / (k_b \times d) \times f_b = \mathbf{-58 \text{ psi}}$$

Compression moment arm to face shell;

$$x_1 = d - k_b \times d / 3 = \mathbf{3.416 \text{ in}}$$

Compression in face shell;

$$C_1 = f_b / 2 \times k_b \times d = \mathbf{8.025 \text{ kips/ft}}$$

Compression at balance point;

$$C_b = C_1 = \mathbf{8.025 \text{ kips/ft}}$$

Force in tension face reinf. at balance point;

$$T_b = f_s \times A_s = \mathbf{2.454 \text{ kips/ft}}$$

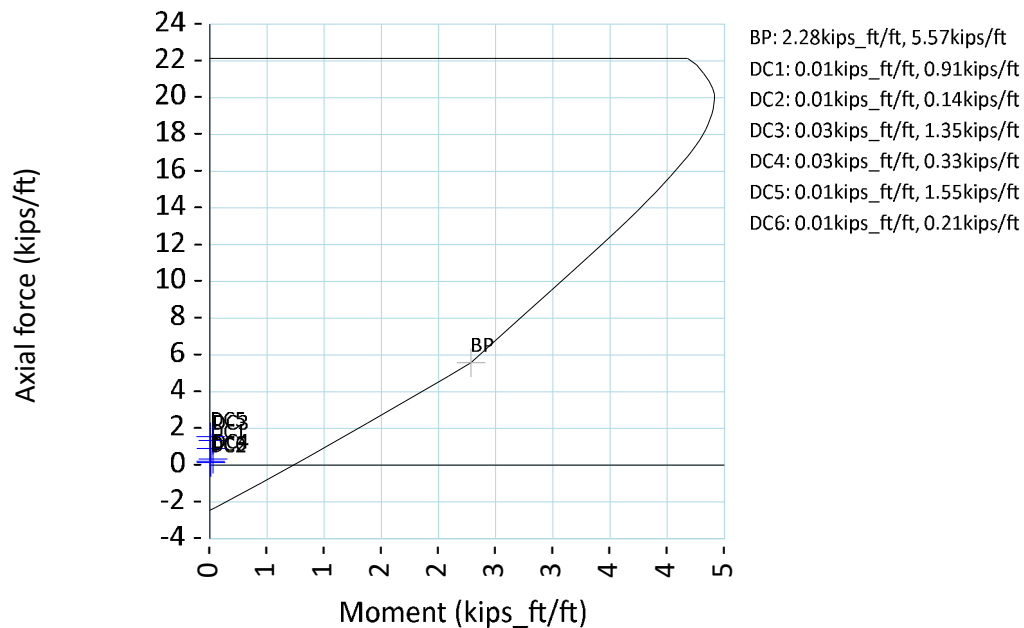
Axial load at balance point;

$$N_b = \min(C_1 - T_b, P_{a\_pos}) = \mathbf{5.570 \text{ kips/ft}}$$

Moment at balance point;

$$M_b = \max(C_1 \times x_1, 0 \text{ lb\_ft/ft}) = \mathbf{2.285 \text{ kips\_ft/ft}}$$

Out-of-plane interaction diagram





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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 168
Project Name: JKV-EMS		

In-plane axial resistance

In-plane axial force;

Depth to reinforcement

Net shear area

Slenderness ratio;

Allowable axial force - Eqn.8-19;

$$P_{in} = P \times L = \mathbf{6.6 \text{ kips}}$$

$$d_{in} = L - l_m / 4 = \mathbf{31.4 \text{ ft}}$$

$$A_{nv,in} = 2 \times d_{in} \times t_{fs} + d_{in} / s_{grout} \times (l_m / (n_{w,int} + 1)) \times (t - 2 \times t_{fs}) = \mathbf{1886 \text{ in}^2}$$

$$h_{eff,x} / r_x = \mathbf{4.130}; < 99$$

$$P_{a,in\_pos} = ((0.25 \times f'_m \times (A_n - A_s)) \times (1 - (h_{eff,x} / (140 \times r_x))^2) \times L) = \mathbf{1226.75 \text{ kips}}$$

In-plane axial stress resistance

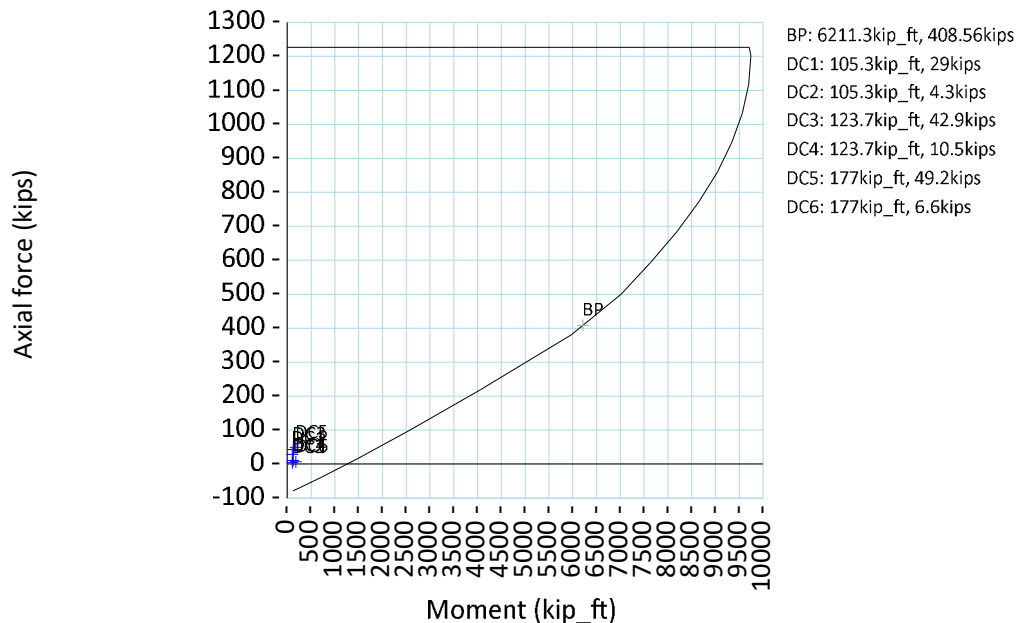
Slenderness ratio;

Allowable axial stress - cl.8.2.4.1;

$$h_{eff,x} / r_x = \mathbf{4.130}; < 99$$

$$F_{a,in\_pos} = (1 / 4) \times f'_m \times (1 - (h_{eff,x} / (140 \times r_x))^2) = \mathbf{624.5 \text{ psi}}$$

In-plane interaction diagram



From interaction diagram

Axial load at balance point;

Moment at balance point;

Maximum moment;

$$N_{b,in} = \mathbf{409 \text{ kips}}$$

$$M_{b,in} = \mathbf{6211 \text{ kip\_ft}}$$

$$M_{max,in} = \mathbf{9745 \text{ kip\_ft}}$$

Top: Max axial

Design forces

Axial load;

Axial eccentricity;

Out-of-plane moment;

Out-of-plane shear force;

In-plane moment;

In-plane Shear force;

$$P = \mathbf{0.913 \text{ kips/ft}}$$

$$e = \mathbf{0 \text{ in}}$$

$$M_y = \mathbf{-0.009 \text{ kips\_ft/ft}}$$

$$V_y = \mathbf{0.000 \text{ kips/ft}}$$

$$M_x = \mathbf{-105.3 \text{ kips\_ft}}$$

$$V_x = \mathbf{5.5 \text{ kips}}$$

Reinforcement effective depth

Effective depth of reinforcement;

$$d = \mathbf{3.813 \text{ in}}$$



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 169
Project Name: JKV-EMS		

Out-of-plane axial check

$$P / P_{a\_pos} = 0.041$$

PASS - Pure axial resistance exceeds axial load

Out-of-plane axial stress check

Stress due to axial load;

$$f_a = P / A = 13.6 \text{ psi}$$

$$f_a / F_{a\_pos} = 0.038$$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit} = 1.016 \text{ kips\_ft/ft}$$

$$Abs(M_y) / M_{limit} = 0.009$$

PASS - Design flexural strength exceeds factored bending moment

In-plane axial check

$$P_{in} / P_{a\_in\_pos} = 0.024$$

PASS - Pure axial resistance exceeds axial load

In-plane axial stress check

Stress due to axial load;

$$f_a = P / A = 13.6 \text{ psi}$$

$$f_a / F_{a\_in\_pos} = 0.022$$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit,in} = 1645 \text{ kip\_ft}$$

$$Abs(M_x) / M_{limit,in} = 0.064$$

PASS - Design flexural strength exceeds factored bending moment

In-plane shear check

Shear stress;

Allowable shear stress of masonry;

$$f_{v,in} = V_x / A_{nv,in} = 2.92 \text{ psi}$$

$$F_{vm,in} = 1/2 \times (4 - 1.75 \times \min(Abs(M_x) / (V_x \times d_{in}), 1)) \times \sqrt{f'_m \times 1 \text{ psi}} + 0.25 \times P_{in} / (A_n \times L) = 77.03 \text{ psi}$$

Allowable shear stress of steel;

$$F_{vs,in} = 0.5 \times (A_v \times F_{sv} \times d_{in} / (A_{nv,in} / d_{in} \times 1 \text{ ft})) = 202.63 \text{ psi}$$

Grout factor;

$$\gamma_g = 0.75$$

Allowable shear stress;

$$F_{v,in} = \min(F_{vm,in} + F_{vs,in}, 2.52 \times \sqrt{f'_m \times 1 \text{ psi}}) \times \gamma_g = 94.5 \text{ psi}$$

$$f_{v,in} / F_{v,in} = 0.031$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane shear friction - 8.3.6

Coefficient of friction

$$\mu = 0.7$$

Shear friction stress ($M_x / (V_x / d_{in})$ lte 0.5)

$$F_{f1} = \max(\mu \times (A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = 39.4 \text{ psi}$$

Shear friction stress ($M_x / (V_x / d_{in})$ gte 1)

$$F_{f2} = \max(0.65 \times (0.6 \times A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = 25.9 \text{ psi}$$

Shear friction stress as $M_x / (V_x \times d_{in})$ lte 0.5

$$F_f = F_{f1} = 39.4 \text{ psi}$$

$$f_{v,in} / F_f = 0.074$$

PASS - Allowable shear friction stress exceeds applied shear stress

Biaxial bending

$$\sqrt{((Abs(M_y) / M_{limit})^2 + (Abs(M_x) / M_{limit,in})^2)} = 0.065$$

PASS - Biaxial bending utilization is acceptable



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 170
Project Name: JKV-EMS		

Top: Min axial

Design forces

Axial load; $P = 0.135$ kips/ft
Axial eccentricity; $e = 0$ in
Out-of-plane moment; $M_y = -0.009$ kips_ft/ft
Out-of-plane shear force; $V_y = 0.000$ kips/ft
In-plane moment; $M_x = -105.3$ kips_ft
In-plane Shear force; $V_x = 5.5$ kips

Reinforcement effective depth

Effective depth of reinforcement; $d = 3.813$ in

Out-of-plane axial check

$$P / P_{a\_pos} = 0.006$$

PASS - Pure axial resistance exceeds axial load

Out-of-plane axial stress check

Stress due to axial load; $f_a = P / A = 2.0$ psi
 $f_a / F_{a\_pos} = 0.006$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load; $M_{limit} = 0.730$ kips_ft/ft
 $Abs(M_y) / M_{limit} = 0.013$

PASS - Design flexural strength exceeds factored bending moment

In-plane axial check

$$P_{in} / P_{a\_in\_pos} = 0.004$$

PASS - Pure axial resistance exceeds axial load

In-plane axial stress check

Stress due to axial load; $f_a = P / A = 2.0$ psi
 $f_a / F_{a\_in\_pos} = 0.003$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load; $M_{limit,in} = 1306$ kip_ft
 $Abs(M_x) / M_{limit,in} = 0.081$

PASS - Design flexural strength exceeds factored bending moment

In-plane shear check

Shear stress; $f_{v,in} = V_x / A_{nv,in} = 2.92$ psi
Allowable shear stress of masonry; $F_{vm,in} = 1/2 \times (4 - 1.75 \times \min(Abs(M_x) / (V_x \times d_{in}), 1)) \times \sqrt{f'_m \times 1 \text{ psi}} + 0.25 \times P_{in} / (A_n \times L) = 73.89$ psi
Allowable shear stress of steel; $F_{vs,in} = 0.5 \times (A_v \times F_{sv} \times d_{in} / (A_{nv,in} / d_{in} \times 1ft)) = 202.63$ psi
Grout factor; $\gamma_g = 0.75$
Allowable shear stress; $F_{v,in} = \min(F_{vm,in} + F_{vs,in}, 2.52 \times \sqrt{f'_m \times 1 \text{ psi}}) \times \gamma_g = 94.5$ psi
 $f_{v,in} / F_{v,in} = 0.031$

PASS - Allowable shear stress exceeds applied shear stress



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 171
Project Name: JKV-EMS		

In-plane shear friction - 8.3.6

Coefficient of friction

$$\mu = 0.7$$

Shear friction stress ($M_x / (V_x / d_{in})$ lte 0.5)

$$F_{f1} = \max(\mu \times (A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = 30.2 \text{ psi}$$

Shear friction stress ($M_x / (V_x / d_{in})$ gte 1)

$$F_{f2} = \max(0.65 \times (0.6 \times A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = 17.4 \text{ psi}$$

Shear friction stress as $M_x / (V_x \times d_{in})$ lte 0.5

$$F_f = F_{f1} = 30.2 \text{ psi}$$

$$f_{v,in} / F_f = 0.096$$

PASS - Allowable shear friction stress exceeds applied shear stress

Biaxial bending

$$\sqrt{((\text{abs}(M_y) / M_{limit})^2 + (\text{abs}(M_x) / M_{limit,in})^2)} = 0.082$$

PASS - Biaxial bending utilization is acceptable

Mid-fifth: Max axial

Design forces

Axial load;

$$P = 1.351 \text{ kips/ft}$$

Axial eccentricity;

$$e = 0 \text{ in}$$

Out-of-plane moment;

$$M_y = -0.028 \text{ kips\_ft/ft}$$

Out-of-plane shear force;

$$V_y = 0.006 \text{ kips/ft}$$

In-plane moment;

$$M_x = -123.7 \text{ kips\_ft}$$

In-plane Shear force;

$$V_x = 5.4 \text{ kips}$$

Reinforcement effective depth

Effective depth of reinforcement;

$$d = 3.813 \text{ in}$$

Out-of-plane axial check

$$P / P_{a\_pos} = 0.061$$

PASS - Pure axial resistance exceeds axial load

Out-of-plane axial stress check

Stress due to axial load;

$$f_a = P / A = 20.1 \text{ psi}$$

$$f_a / F_{a\_pos} = 0.056$$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit} = 1.016 \text{ kips\_ft/ft}$$

$$\text{Abs}(M_y) / M_{limit} = 0.028$$

PASS - Design flexural strength exceeds factored bending moment

Out-of-plane shear check

Net shear area

$$A_{nv} = d \times l_m / ((n_{w,int} + 1) \times s_{grout}) = 22.34 \text{ in}^2/\text{ft}$$

Applied shear stress;

$$f_v = V_y / A_{nv} = 0.28 \text{ psi}$$

Shear resistance of masonry - Eqn 8-26;

$$F_{vm} = 1/2 \times (4 - 1.75 \times \min(\text{Abs}(M_y) / (V_y \times d), 1)) \times \sqrt{(f_m \times 1 \text{ psi})} + 0.25 \times P / A_n = 61.7 \text{ psi}$$

Grout factor - cl.8.3.5.1.2;

$$\gamma_g = 0.75$$

Overall shear resistance - cl.8.3.5.1.2;

$$F_v = \min(F_{vm}, 2 \times \sqrt{(f_m \times 1 \text{ psi})}) \times \gamma_g = 46.28 \text{ psi}$$

$$f_v / F_v = 0.006$$

PASS - Allowable shear stress exceeds applied shear stress



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 172
Project Name: JKV-EMS		

In-plane axial check

$$P_{in} / P_{a,in\_pos} = 0.035$$

PASS - Pure axial resistance exceeds axial load

In-plane axial stress check

Stress due to axial load;

$$f_a = P / A = 20.1 \text{ psi}$$

$$f_a / F_{a,in\_pos} = 0.032$$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit,in} = 1829 \text{ kip\_ft}$$

$$Abs(M_x) / M_{limit,in} = 0.068$$

PASS - Design flexural strength exceeds factored bending moment

In-plane shear check

Shear stress;

$$f_{v,in} = V_x / A_{nv,in} = 2.86 \text{ psi}$$

Allowable shear stress of masonry;

$$F_{vm,in} = 1/2 \times (4 - 1.75 \times \min(Abs(M_x) / (V_x \times d_{in}), 1)) \times \sqrt{f'_m \times 1 \text{ psi}} + 0.25 \times$$

$$P_{in} / (A_n \times L) = 73.56 \text{ psi}$$

$$F_{vs,in} = 0.5 \times (A_v \times F_{sv} \times d_{in} / (A_{nv,in} \times d_{in} \times 1 \text{ ft})) = 202.63 \text{ psi}$$

Allowable shear stress of steel;

$$\gamma_g = 0.75$$

Grout factor;

$$F_{v,in} = \min(F_{vm,in} + F_{vs,in}, 2.36 \times \sqrt{f'_m \times 1 \text{ psi}}) \times \gamma_g = 88.5 \text{ psi}$$

Allowable shear stress;

$$f_{v,in} / F_{v,in} = 0.032$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane shear friction - 8.3.6

Coefficient of friction

$$\mu = 0.7$$

Shear friction stress ($M_x / (V_x \times d_{in})$ lte 0.5)

$$F_{f1} = \max(\mu \times (A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = 44.5 \text{ psi}$$

Shear friction stress ($M_x / (V_x \times d_{in})$ gte 1)

$$F_{f2} = \max(0.65 \times (0.6 \times A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = 30.7 \text{ psi}$$

Shear friction stress as $M_x / (V_x \times d_{in})$ lte 0.5

$$F_f = F_{f1} = 44.5 \text{ psi}$$

$$f_{v,in} / F_f = 0.064$$

PASS - Allowable shear friction stress exceeds applied shear stress

Biaxial bending

$$\sqrt{((Abs(M_y) / M_{limit})^2 + (Abs(M_x) / M_{limit,in})^2)} = 0.073$$

PASS - Biaxial bending utilization is acceptable

Mid-fifth: Min axial

Design forces

Axial load;

$$P = 0.331 \text{ kips/ft}$$

Axial eccentricity;

$$e = 0 \text{ in}$$

Out-of-plane moment;

$$M_y = -0.028 \text{ kips\_ft/ft}$$

Out-of-plane shear force;

$$V_y = 0.006 \text{ kips/ft}$$

In-plane moment;

$$M_x = -123.7 \text{ kips\_ft}$$

In-plane Shear force;

$$V_x = 5.4 \text{ kips}$$

Reinforcement effective depth

Effective depth of reinforcement;

$$d = 3.813 \text{ in}$$



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Revision:	Date:	Sheet No. 173
Project Name: JKV-EMS		

Out-of-plane axial check

$$P / P_{a\_pos} = 0.015$$

PASS - Pure axial resistance exceeds axial load

Out-of-plane axial stress check

Stress due to axial load;

$$f_a = P / A = 4.9 \text{ psi}$$

$$f_a / F_{a\_pos} = 0.014$$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit} = 0.730 \text{ kips\_ft/ft}$$

$$Abs(M_y) / M_{limit} = 0.039$$

PASS - Design flexural strength exceeds factored bending moment

Out-of-plane shear check

Net shear area

$$A_{nv} = d \times l_m / ((n_{w,int} + 1) \times S_{grout}) = 22.34 \text{ in}^2/\text{ft}$$

Applied shear stress;

$$f_v = V_y / A_{nv} = 0.28 \text{ psi}$$

Shear resistance of masonry - Eqn 8-26;

$$F_{vm} = 1/2 \times (4 - 1.75 \times \min(Abs(M_y) / (V_y \times d), 1)) \times \sqrt{(f_m \times 1 \text{ psi})} + 0.25 \times P / A_n = 57.58 \text{ psi}$$

Grout factor - cl.8.3.5.1.2;

$$\gamma_g = 0.75$$

Overall shear resistance - cl.8.3.5.1.2;

$$F_v = \min(F_{vm}, 2 \times \sqrt{(f_m \times 1 \text{ psi})}) \times \gamma_g = 43.19 \text{ psi}$$

$$f_v / F_v = 0.007$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane axial check

$$P_{in} / P_{a,in\_pos} = 0.009$$

PASS - Pure axial resistance exceeds axial load

In-plane axial stress check

Stress due to axial load;

$$f_a = P / A = 4.9 \text{ psi}$$

$$f_a / F_{a,in\_pos} = 0.008$$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit,in} = 1404 \text{ kip\_ft}$$

$$Abs(M_x) / M_{limit,in} = 0.088$$

PASS - Design flexural strength exceeds factored bending moment

In-plane shear check

Shear stress;

$$f_{v,in} = V_x / A_{nv,in} = 2.86 \text{ psi}$$

Allowable shear stress of masonry;

$$F_{vm,in} = 1/2 \times (4 - 1.75 \times \min(Abs(M_x) / (V_x \times d_{in}), 1)) \times \sqrt{(f_m \times 1 \text{ psi})} + 0.25 \times P_{in} / (A_n \times L) = 69.44 \text{ psi}$$

Allowable shear stress of steel;

$$F_{vs,in} = 0.5 \times (A_v \times F_{sv} \times d_{in} / (A_{nv,in} \times d_{in} \times 1\text{ft})) = 202.63 \text{ psi}$$

Grout factor;

$$\gamma_g = 0.75$$

Allowable shear stress;

$$F_{v,in} = \min(F_{vm,in} + F_{vs,in}, 2.36 \times \sqrt{(f_m \times 1 \text{ psi})}) \times \gamma_g = 88.5 \text{ psi}$$

$$f_{v,in} / F_{v,in} = 0.032$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane shear friction - 8.3.6

Coefficient of friction

$$\mu = 0.7$$



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Revision:	Date:	Sheet No. 174
Project Name: JKV-EMS		

Shear friction stress ($M_x / (V_x / d_{in})$ lte 0.5)
Shear friction stress ($M_x / (V_x / d_{in})$ gte 1)
Shear friction stress as $M_x / (V_x \times d_{in})$ lte 0.5

$$F_{f1} = \max(\mu \times (A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = \mathbf{32.5 \text{ psi}}$$
$$F_{f2} = \max(0.65 \times (0.6 \times A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = \mathbf{19.6 \text{ psi}}$$
$$F_f = F_{f1} = \mathbf{32.5 \text{ psi}}$$
$$f_{v,in} / F_f = \mathbf{0.088}$$

PASS - Allowable shear friction stress exceeds applied shear stress

Biaxial bending

$$\sqrt{((\text{abs}(M_y) / M_{limit})^2 + (\text{abs}(M_x) / M_{limit,in})^2)} = \mathbf{0.096}$$

PASS - Biaxial bending utilization is acceptable

Bottom: Max axial

Design forces

Axial load;
Axial eccentricity;
Out-of-plane moment;
Out-of-plane shear force;
In-plane moment;
In-plane Shear force;

$$P = \mathbf{1.550 \text{ kips/ft}}$$
$$e = \mathbf{0 \text{ in}}$$
$$M_y = \mathbf{0.006 \text{ kips\_ft/ft}}$$
$$V_y = \mathbf{0.000 \text{ kips/ft}}$$
$$M_x = \mathbf{-177 \text{ kips\_ft}}$$
$$V_x = \mathbf{6.1 \text{ kips}}$$

Reinforcement effective depth

Effective depth of reinforcement;

$$d = \mathbf{3.813 \text{ in}}$$

Out-of-plane axial check

$$P / P_{a\_pos} = \mathbf{0.070}$$

PASS - Pure axial resistance exceeds axial load

Out-of-plane axial stress check

Stress due to axial load;

$$f_a = P / A = \mathbf{23.0 \text{ psi}}$$
$$f_a / F_{a\_pos} = \mathbf{0.064}$$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit} = \mathbf{1.299 \text{ kips\_ft/ft}}$$
$$\text{Abs}(M_y) / M_{limit} = \mathbf{0.005}$$

PASS - Design flexural strength exceeds factored bending moment

In-plane axial check

$$P_{in} / P_{a,in\_pos} = \mathbf{0.040}$$

PASS - Pure axial resistance exceeds axial load

In-plane axial stress check

Stress due to axial load;

$$f_a = P / A = \mathbf{23.0 \text{ psi}}$$
$$f_a / F_{a,in\_pos} = \mathbf{0.037}$$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit,in} = \mathbf{1909 \text{ kip\_ft}}$$
$$\text{Abs}(M_x) / M_{limit,in} = \mathbf{0.093}$$

PASS - Design flexural strength exceeds factored bending moment



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Revision:	Date:	Sheet No. 175
Project Name: JKV-EMS		

In-plane shear check

Shear stress;

Allowable shear stress of masonry;

Allowable shear stress of steel;

Grout factor;

Allowable shear stress;

$$f_{v,in} = V_x / A_{nv,in} = \mathbf{3.23 \text{ psi}}$$

$$F_{vm,in} = 1/2 \times (4 - 1.75 \times \min(\text{Abs}(M_x) / (V_x \times d_{in}), 1)) \times \sqrt{(f'_m \times 1 \text{ psi}) + 0.25 \times P_{in} / (A_n \times L)} = \mathbf{65.86 \text{ psi}}$$

$$F_{vs,in} = 0.5 \times (A_v \times F_{sv} \times d_{in} / (A_{nv,in} \times d_{in} \times 1\text{ft})) = \mathbf{202.63 \text{ psi}}$$

$$\gamma_g = \mathbf{0.75}$$

$$F_{v,in} = \min(F_{vm,in} + F_{vs,in}, 2.1 \times \sqrt{(f'_m \times 1 \text{ psi})}) \times \gamma_g = \mathbf{78.75 \text{ psi}}$$

$$f_{v,in} / F_{v,in} = \mathbf{0.041}$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane shear friction - 8.3.6

Coefficient of friction

Shear friction stress ($M_x / (V_x \times d_{in})$ lte 0.5)

Shear friction stress ($M_x / (V_x \times d_{in})$ gte 1)

Shear friction stress as $M_x / (V_x \times d_{in})$ lte 0.5

$$\mu = \mathbf{0.7}$$

$$F_{f1} = \max(\mu \times (A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = \mathbf{46.9 \text{ psi}}$$

$$F_{f2} = \max(0.65 \times (0.6 \times A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = \mathbf{32.9 \text{ psi}}$$

$$F_f = F_{f1} = \mathbf{46.9 \text{ psi}}$$

$$f_{v,in} / F_f = \mathbf{0.069}$$

PASS - Allowable shear friction stress exceeds applied shear stress

Biaxial bending

$$\sqrt{((\text{abs}(M_y) / M_{limit})^2 + (\text{abs}(M_x) / M_{limit,in})^2)} = \mathbf{0.093}$$

PASS - Biaxial bending utilization is acceptable

Bottom: Min axial

Design forces

Axial load;

Axial eccentricity;

Out-of-plane moment;

Out-of-plane shear force;

In-plane moment;

In-plane Shear force;

$$P = \mathbf{0.208 \text{ kips/ft}}$$

$$e = \mathbf{0 \text{ in}}$$

$$M_y = \mathbf{0.006 \text{ kips\_ft/ft}}$$

$$V_y = \mathbf{0.000 \text{ kips/ft}}$$

$$M_x = \mathbf{-177 \text{ kips\_ft}}$$

$$V_x = \mathbf{6.1 \text{ kips}}$$

Reinforcement effective depth

Effective depth of reinforcement;

$$d = \mathbf{3.813 \text{ in}}$$

Out-of-plane axial check

$$P / P_{a,pos} = \mathbf{0.009}$$

PASS - Pure axial resistance exceeds axial load

Out-of-plane axial stress check

Stress due to axial load;

$$f_a = P / A = \mathbf{3.1 \text{ psi}}$$

$$f_a / F_{a,pos} = \mathbf{0.009}$$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit} = \mathbf{0.730 \text{ kips\_ft/ft}}$$

$$\text{Abs}(M_y) / M_{limit} = \mathbf{0.009}$$

PASS - Design flexural strength exceeds factored bending moment



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 176
Project Name: JKV-EMS		

In-plane axial check

$$P_{in} / P_{a,in\_pos} = 0.005$$

PASS - Pure axial resistance exceeds axial load

In-plane axial stress check

Stress due to axial load;

$$f_a = P / A = 3.1 \text{ psi}$$

$$f_a / F_{a,in\_pos} = 0.005$$

PASS - Allowable stress exceeds compressive stress due to axial load

From interaction diagram

Limiting moment under applied axial load;

$$M_{limit,in} = 1348 \text{ kip\_ft}$$

$$Abs(M_x) / M_{limit,in} = 0.131$$

PASS - Design flexural strength exceeds factored bending moment

In-plane shear check

Shear stress;

Allowable shear stress of masonry;

$$f_{v,in} = V_x / A_{nv,in} = 3.23 \text{ psi}$$

$$F_{vm,in} = 1/2 \times (4 - 1.75 \times \min(Abs(M_x) / (V_x \times d_{in}), 1)) \times \sqrt{(f'_m \times 1 \text{ psi}) + 0.25 \times}$$

$$P_{in} / (A_n \times L) = 60.44 \text{ psi}$$

$$F_{vs,in} = 0.5 \times (A_v \times F_{sv} \times d_{in} / (A_{nv,in} / d_{in} \times 1 \text{ ft})) = 202.63 \text{ psi}$$

$$\gamma_g = 0.75$$

$$F_{v,in} = \min(F_{vm,in} + F_{vs,in}, 2.1 \times \sqrt{(f'_m \times 1 \text{ psi})}) \times \gamma_g = 78.75 \text{ psi}$$

$$f_{v,in} / F_{v,in} = 0.041$$

PASS - Allowable shear stress exceeds applied shear stress

In-plane shear friction - 8.3.6

Coefficient of friction

$$\mu = 0.7$$

Shear friction stress ($M_x / (V_x \times d_{in})$ lte 0.5)

$$F_{f1} = \max(\mu \times (A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = 31.1 \text{ psi}$$

Shear friction stress ($M_x / (V_x \times d_{in})$ gte 1)

$$F_{f2} = \max(0.65 \times (0.6 \times A_s \times d_{in} \times F_s + P_{in}) / (A_{nv,in}), 0 \text{ psi}) = 18.2 \text{ psi}$$

Shear friction stress as $M_x / (V_x \times d_{in})$ lte 0.5

$$F_f = F_{f1} = 31.1 \text{ psi}$$

$$f_{v,in} / F_f = 0.104$$

PASS - Allowable shear friction stress exceeds applied shear stress

Biaxial bending

$$\sqrt{((Abs(M_y) / M_{limit})^2 + (Abs(M_x) / M_{limit,in})^2)} = 0.132$$

PASS - Biaxial bending utilization is acceptable

STEEL LINTEL AT APP BAY DOOR

In accordance with AISC360-16 (15th Edition) using the ASD method

Tedds calculation version 4.5.05

Safety factors

Shear;

$$\Omega_v = 1.50$$

Flexure;

$$\Omega_b = 1.67$$

Tensile yielding;

$$\Omega_{t,y} = 1.67$$

Tensile rupture;

$$\Omega_{t,r} = 2.00$$

Compression;

$$\Omega_c = 1.67$$



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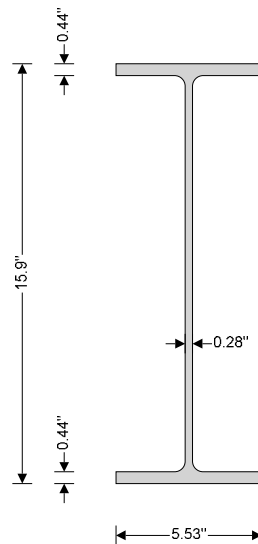
Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 177
Project Name: JKV-EMS		

Design section 1

Section details

Section type;
ASTM steel designation;
Steel yield stress;
Steel tensile stress;
Modulus of elasticity;

W 16x31 (AISC 16th Edn (v16.0))
A992
 $F_y = 50$ ksi
 $F_u = 65$ ksi
 $E = 29000$ ksi



W 16x31 (AISC 16th Edn (v16.0))

Section depth, d , 15.9 in
Section breadth, b_f , 5.53 in
Weight of section, Weight, 31 lbf/ft
Flange thickness, t_f , 0.44 in
Web thickness, t_w , 0.275 in
Root radius, $k - t_r$, 0.402 in
Area of section, A , 9.1 in²
Radius of gyration about x-axis, r_x , 6.41 in
Radius of gyration about y-axis, r_y , 1.17 in
Elastic section modulus about x-axis, S_x , 47.2 in³
Elastic section modulus about y-axis, S_y , 4.49 in³
Plastic section modulus about x-axis, Z_x , 54 in³
Plastic section modulus about y-axis, Z_y , 7.03 in³
Second moment of area about x-axis, I_x , 375 in⁴
Second moment of area about y-axis, I_y , 12.4 in⁴

Analysis results

Required flexural strength - Major axis;
Required flexural strength - Minor axis;
Required shear strength - Major axis;
Required shear strength - Minor axis;

$M_{r,x} = 25.63$ kips_{ft}
 $M_{r,y} = 3.52$ kips_{ft}
 $V_{r,x} = 8.1$ kips
 $V_{r,y} = 1.11$ kips

Restraint spacing

Major axis lateral restraint;
Minor axis lateral restraint;
Torsional restraint;

$L_x = 12.67$ ft
 $L_y = 12.67$ ft
 $L_z = 0$ ft

Classification of sections for local buckling - Section B4

Classification of flanges in flexure - Table B4.1b (case 10)

Width to thickness ratio; $b_f / (2 \times t_f) = 6.28$
Limiting ratio for compact section; $\lambda_{pff} = 0.38 \times \sqrt{E / F_y} = 9.15$
Limiting ratio for non-compact section; $\lambda_{rff} = 1.0 \times \sqrt{E / F_y} = 24.08$; Compact

Classification of web in flexure - Table B4.1b (case 15)

Width to thickness ratio; $(d - 2 \times k) / t_w = 51.69$
Limiting ratio for compact section; $\lambda_{pwf} = 3.76 \times \sqrt{E / F_y} = 90.55$
Limiting ratio for non-compact section; $\lambda_{rwf} = 5.70 \times \sqrt{E / F_y} = 137.27$; Compact

Section is compact in flexure



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 178
Project Name: JKV-EMS		

Design of members for shear - Chapter G

Required shear strength;

$$V_{r,x} = 8.1 \text{ kips}$$

Web area;

$$A_w = d \times t_w = 4.373 \text{ in}^2$$

Web plate buckling coefficient;

$$k_v = 5.34$$

$$(d - 2 \times k) / t_w \leq 2.24 \times \sqrt{(E / F_y)}$$

Web shear coefficient - eq G2-2;

$$C_{v1} = 1.000$$

Nominal shear strength - eq G2-1;

$$V_{n,x} = 0.6 \times F_y \times A_w \times C_{v1} = 131.2 \text{ kips}$$

Safety factor;

$$\Omega_v = 1.50$$

Allowable shear strength;

$$V_{c,x} = V_{n,x} / \Omega_v = 87.5 \text{ kips}$$

$$V_{r,x} / V_{c,x} = 0.093$$

PASS - Allowable shear strength exceeds required shear strength

Required shear strength;

$$V_{r,y} = 1.1 \text{ kips}$$

Web area;

$$A_w = 2 \times b_f \times t_f = 4.866 \text{ in}^2$$

Web plate buckling coefficient;

$$k_v = 1.2$$

$$b_f / (2 \times t_f) \leq 1.10 \times \sqrt{(k_v \times E / F_y)}$$

Web shear coefficient - eq G2-9;

$$C_{v2} = 1.000$$

Nominal shear strength - eq G6-1;

$$V_{n,y} = 0.6 \times F_y \times A_w \times C_{v2} = 146 \text{ kips}$$

Safety factor;

$$\Omega_v = 1.67$$

Allowable shear strength;

$$V_{c,y} = V_{n,y} / \Omega_v = 87.4 \text{ kips}$$

$$V_{r,y} / V_{c,y} = 0.013$$

PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure - Chapter F

Required flexural strength;

$$M_{r,x} = 25.6 \text{ kips\_ft}$$

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1;

$$M_{n,yld,x} = M_{p,x} = F_y \times Z_x = 225 \text{ kips\_ft}$$

Lateral-torsional buckling - Section F2.2

Unbraced length;

$$L_b = 12.67 \text{ ft}$$

Limiting unbraced length for yielding - eq F2-5;

$$L_p = 1.76 \times r_y \times \sqrt{(E / F_y)} = 4.133 \text{ ft}$$

Distance between flange centroids;

$$h_o = 15.5 \text{ in}$$

$$c = 1$$

$$r_{ts} = 1.42 \text{ in}$$

Limiting unbraced length for inelastic LTB - eq F2-6; $L_r = 1.95 \times r_{ts} \times E / (0.7 \times F_y) \times \sqrt{((J \times c / (S_x \times h_o)) + \sqrt{((J \times c / (S_x \times h_o))^2 + 6.76 \times (0.7 \times F_y / E)^2}))} = 11.833 \text{ ft}$

LTB modification factor;

$$C_b = 1.000$$

Critical flexural stress - eq F2-4;

$$F_{cr} = (C_b \times \pi^2 \times E / (L_b / r_{ts})^2) \times \sqrt{(1 + 0.078 \times (J \times c / (S_x \times h_o)) \times (L_b / r_{ts})^2)} = 31.2 \text{ ksi}$$

Nominal flexural strength for lateral-torsional buckling - eq F2-3

$$M_{n,ltb,x} = \min(F_{cr} \times S_x, M_{p,x}) = 122.8 \text{ kips\_ft}$$

Allowable flexural strength - F1

Nominal flexural strength;

$$M_{n,x} = \min(M_{n,yld,x}, M_{n,ltb,x}) = 122.8 \text{ kips\_ft}$$

Allowable flexural strength;

$$M_{c,x} = M_{n,x} / \Omega_b = 73.5 \text{ kips\_ft}$$

$$M_{r,x} / M_{c,x} = 0.349$$

PASS - Allowable flexural strength exceeds required flexural strength



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 179
Project Name: JKV-EMS		

Design of members for flexure - Chapter F

Required flexural strength; $M_{r,y} = 3.5$ kips_ft

Yielding - Section F6.1

Nominal flexural strength for yielding - eq F6-1; $M_{n,yld,y} = M_{p,y} = \min(F_y \times Z_y, 1.6 \times F_y \times S_y) = 29.3$ kips_ft

Compression flange local buckling - Section F6.2

$$\lambda = b_f / (2 \times t_f) = 6.284$$

Nominal flexural strength for compression flange local buckling - eq F6-2;

$$M_{n,flb,y} = M_{p,y} = 29.3 \text{ kips\_ft}$$

Allowable flexural strength - F1

Nominal flexural strength;

$$M_{n,y} = \min(M_{n,yld,y}, M_{n,flb,y}) = 29.3 \text{ kips\_ft}$$

Allowable flexural strength;

$$M_{c,y} = M_{n,y} / \Omega_b = 17.5 \text{ kips\_ft}$$

$$M_{r,y} / M_{c,y} = 0.201$$

PASS - Allowable flexural strength exceeds required flexural strength

Design of members for combined forces - Chapter H

Combined flexure and axial force - eq H1-1b; $M_{r,x} / M_{c,x} + M_{r,y} / M_{c,y} = 0.549$

PASS - Combined flexure and axial force is within acceptable limits

MASONRY LINTEL AT APP BAY WINDOW

Using the allowable stress design method

Tedds calculation version 1.2.04

Design summary

Description	Unit	Capacity	Applied	Utilisation	Result
Bending stress	psi	1125	334.1	0.297	Pass
Shear stress	psi	56.3	26.7	0.475	Pass

Masonry details

Masonry type; Concrete
Density of masonry unit; $\gamma = 115$ lb/ft³
Pattern bond; Running
Mortar type; PCL Type S
Compressive strength of masonry unit; $f'_{cu} = 3250$ psi
Net compressive strength of masonry (Table 2); $f'_m = 2500$ psi
Modulus of elasticity (4.2.2); $E_m = 900 \times f'_m = 2250000$ psi
Allowable flexural tensile stress; (8.2.4.2); $F_t = 106$ psi

Reinforcement details

Allowable tensile stress; $F_s = 32000$ psi
Modulus of elasticity of steel; $E_s = 29000000$ psi

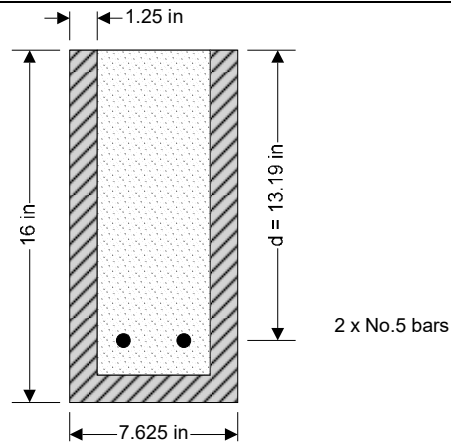
Cover to reinforcement

Bottom cover to reinforcement; $C_{nom\_b} = 1.5$ in
Side cover to reinforcement; $C_{nom\_s} = 1.5$ in



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Revision:	Date:	Sheet No. 180
Project Name: JKV-EMS		



;

Section properties

Modular ratio;
Section width;
Section depth;
Net shear area;
Section modulus;
Depth to tension reinforcement;

$$n = E_s / E_m = 12.89$$

$$b = 7.625 \text{ in}$$

$$h = 16 \text{ in}$$

$$A_{nv} = b \times d = 100.57 \text{ in}^2$$

$$S = b \times h^2 / 6 = 325.33 \text{ in}^3$$

$$d = 13.19 \text{ in}$$

Flexure design (Chapter 8)

Tension reinforcement;
Area of tension reinforcement;
Reinforcement ratio;
Neutral axis factor;
Lever arm factor;
Cracking moment;
Design bending moment;
Tensile stress in reinforcement;
Allowable tensile stress in reinf. (8.3.3.1);

$$2 \times \text{No. 5 bars}$$

$$A_s = N_{bot} \times \text{BarArea}_{bot} = 0.62 \text{ in}^2$$

$$\rho_{ratio} = A_s / (b \times d) = 0.00616$$

$$k = \sqrt{(2 \times \rho_{ratio} \times n + (\rho_{ratio} \times n)^2)} - \rho_{ratio} \times n = 0.327$$

$$j = 1 - k / 3 = 0.891$$

$$M_{cr} = 2.5 \times F_t \times S = 7.2 \text{ kip\_ft}$$

$$M = 5.38 \text{ kip\_ft}$$

$$f_s = M / (A_s \times j \times d) = 8860 \text{ psi}$$

$$F_s = 32000 \text{ psi}$$

$$f_s / F_s = 0.277$$

PASS - Allowable tensile stress exceeds tensile stress due to flexure

Compressive stress in masonry;
Allowable stress in masonry (8.3.4.2.2);

$$f_b = 2 \times M / (j \times k \times b \times d^2) = 334.1 \text{ psi}$$

$$F_b = 0.45 \times f'_m = 1125.0 \text{ psi}$$

$$f_b / F_b = 0.297$$

PASS - Allowable compressive stress exceeds compressive stress due to flexure

Shear design (Chapter 8)

Design shear force;
Depth of shear area;
Moment shear relationship, M/Vd ;
Shear stress (8-21);
Allowable masonry shear stress (8-26);

$$V = 2.69 \text{ kips}$$

$$d_v = 16.00 \text{ in}$$

$$\text{Assume; } M\_Vd_{ratio} = 1$$

$$f_v = V / A_{nv} = 26.7 \text{ psi}$$

$$F_v = 1/2 \times (4.0 - 1.75 \times M\_Vd_{ratio}) \times \sqrt{f'_m \times 1 \text{ psi}}$$

$$F_v = 56.3 \text{ psi}$$



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Revision:	Date:	Sheet No. 181
Project Name: JKV-EMS		

Masonry shear stress ratio;

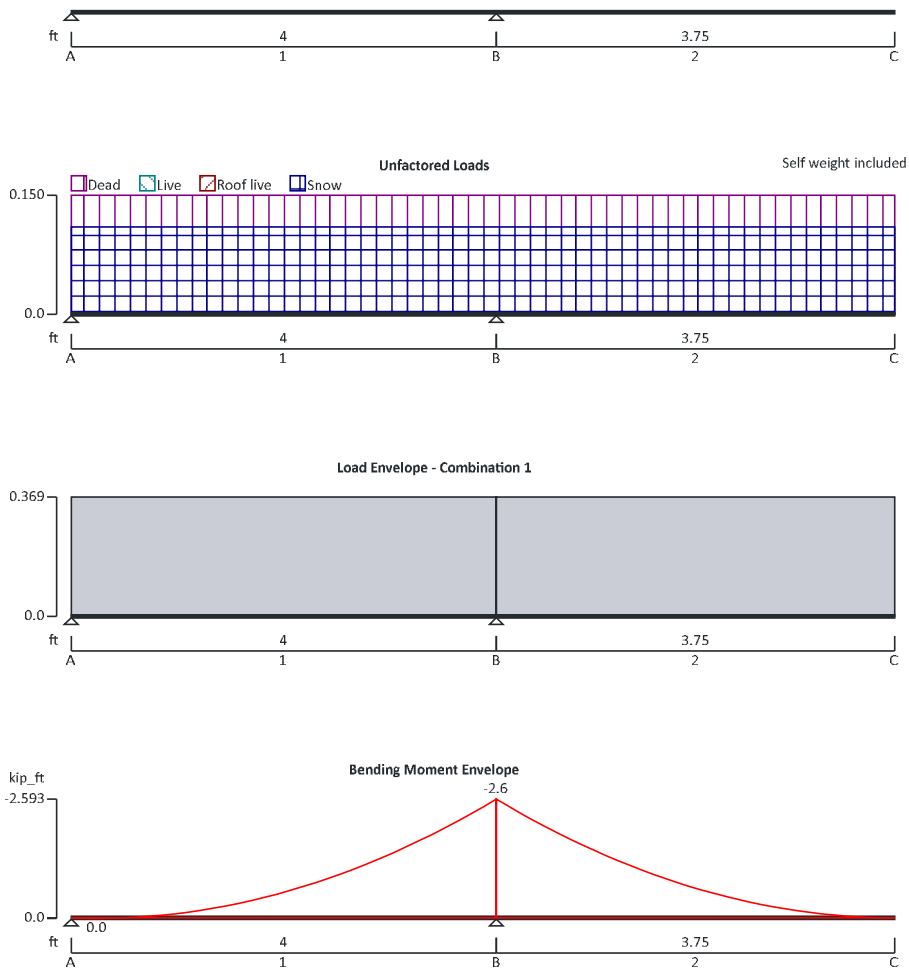
$$f_v / F_v = 0.475$$

PASS - Allowable shear stress exceeds shear stress in masonry

ROOF OUTRIGGER DESIGN AND ANALYSIS - UNIFORM LOAD

In accordance with AISC360-16 using the LRFD method

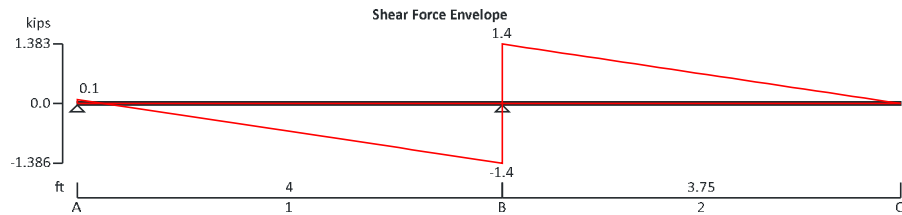
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Revision:	Date:	Sheet No. 182
Project Name: JKV-EMS		



Support conditions

Support A	Vertically restrained Rotationally free
Support B	Vertically restrained Rotationally free
Support C	Vertically free Rotationally free

Applied loading

Beam loads	Dead self weight of beam $\times 1$ Dead full UDL 0.15 kips/ft Snow full UDL 0.11 kips/ft

Load combinations

Load combination 1	Support A	Dead $\times 1.20$ Live $\times 1.60$ Roof live $\times 1.60$ Snow $\times 1.60$ Dead $\times 1.20$ Live $\times 1.60$ Roof live $\times 1.60$ Snow $\times 1.60$
	Support B	Dead $\times 1.20$ Live $\times 1.60$ Roof live $\times 1.60$ Snow $\times 1.60$ Dead $\times 1.20$ Live $\times 1.60$ Roof live $\times 1.60$ Snow $\times 1.60$
	Support C	Dead $\times 1.20$ Live $\times 1.60$ Roof live $\times 1.60$ Snow $\times 1.60$

Analysis results

Maximum moment;	$M_{\max} = 0$ kips_ft;	$M_{\min} = -2.6$ kips_ft



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Revision:	Date:	Sheet No. 183
Project Name: JKV-EMS		

Maximum moment span 1;
Maximum moment span 2;
Maximum shear;
Maximum shear span 1;
Maximum shear span 2;
Deflection;
Deflection span 1;
Deflection span 2;
Maximum reaction at support A;
Unfactored dead load reaction at support A;
Unfactored snow load reaction at support A;
Maximum reaction at support B;
Unfactored dead load reaction at support B;
Unfactored snow load reaction at support B;
Maximum reaction at support C;

$M_{s1\_max} = 0$ kips_ft;
 $M_{s2\_max} = 0$ kips_ft;
 $V_{max} = 1.4$ kips;
 $V_{s1\_max} = 0.1$ kips;
 $V_{s2\_max} = 1.4$ kips;
 $\delta_{max} = 0.3$ in;
 $\delta_{s1\_max} = 0$ in;
 $\delta_{s2\_max} = 0.3$ in;
 $R_{A\_max} = 0.1$ kips;
 $R_{A\_Dead} = 0$ kips
 $R_{A\_Snow} = 0$ kips
 $R_{B\_max} = 2.8$ kips;
 $R_{B\_Dead} = 1.2$ kips
 $R_{B\_Snow} = 0.8$ kips
 $R_{C\_max} = 0$ kips;

$M_{s1\_min} = -2.6$ kips_ft
 $M_{s2\_min} = -2.6$ kips_ft
 $V_{min} = -1.4$ kips
 $V_{s1\_min} = -1.4$ kips
 $V_{s2\_min} = 0$ kips
 $\delta_{min} = 0$ in
 $\delta_{s1\_min} = 0$ in
 $\delta_{s2\_min} = 0$ in
 $R_{A\_min} = 0.1$ kips

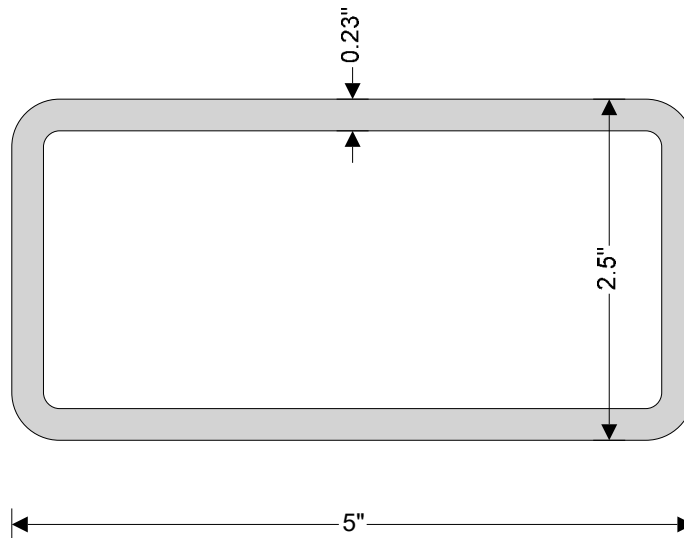
 $R_{B\_min} = 2.8$ kips

 $R_{C\_min} = 0$ kips

Section details

Section type;
ASTM steel designation;
Steel yield stress;
Steel tensile stress;
Modulus of elasticity;

HSS 5x2-1/2x1/4 (AISC 16th Edn (v16.0))
A500 Gr.B 46
 $F_y = 46$ ksi
 $F_u = 58$ ksi
 $E = 29000$ ksi



Resistance factors

Resistance factor for tensile yielding
Resistance factor for tensile rupture
Resistance factor for compression
Resistance factor for flexure

$\phi_{ty} = 0.90$
 $\phi_{tr} = 0.75$
 $\phi_c = 0.90$
 $\phi_b = 0.90$

Lateral bracing

Span 1 has lateral bracing at supports only



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Revision:	Date:	Sheet No. 184
Project Name: JKV-EMS		

Span 2 has lateral bracing at supports only

Cantilever tip is unbraced

Cantilever support is continuous with lateral and torsional restraint

Classification of sections for local buckling - Section B4.1

Classification of flanges in flexure - Table B4.1b (case 17)

Width to thickness ratio; $(d - 3 \times t) / t = \mathbf{18.46}$
Limiting ratio for compact section; $\lambda_{pff} = 1.12 \times \sqrt{E / F_y} = \mathbf{28.12}$
Limiting ratio for non-compact section; $\lambda_{rff} = 1.40 \times \sqrt{E / F_y} = \mathbf{35.15}$; Compact

Classification of web in flexure - Table B4.1b (case 19)

Width to thickness ratio; $(b_f - 3 \times t) / t = \mathbf{7.73}$
Limiting ratio for compact section; $\lambda_{pwf} = 2.42 \times \sqrt{E / F_y} = \mathbf{60.76}$
Limiting ratio for non-compact section; $\lambda_{rwf} = 5.70 \times \sqrt{E / F_y} = \mathbf{143.12}$; Compact

Section is compact in flexure

Design of members for shear - Chapter G

Required shear strength $V_r = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = \mathbf{1.386}$ kips
Web area $A_w = 2 \times (b_f - 3 \times t) \times t = \mathbf{0.839}$ in²
Web plate buckling coefficient $k_v = \mathbf{5}$
Web shear coefficient - eq G2-9 $C_{v2} = \mathbf{1}$
Nominal shear strength – eq G4-1 $V_n = 0.6 \times F_y \times A_w \times C_{v2} = \mathbf{23.164}$ kips
Resistance factor for shear $\phi_v = \mathbf{0.90}$
Design shear strength $V_c = \phi_v \times V_n = \mathbf{20.847}$ kips

PASS - Design shear strength exceeds required shear strength

Design of members for flexure in the minor axis at span 1 - Chapter F

Required flexural strength; $M_r = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = \mathbf{2.593}$ kips_ft

Yielding - Section F7.1

Nominal flexural strength for yielding - eq F7-1; $M_{nyld} = M_p = F_y \times Z_y = \mathbf{11.308}$ kips_ft
Nominal flexural strength; $M_n = M_{nyld} = \mathbf{11.308}$ kips_ft
Design flexural strength; $M_c = \phi_b \times M_n = \mathbf{10.177}$ kips_ft

PASS - Design flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead, live, roof live and snow loads

Limiting deflection; $\delta_{lim} = 2 \times L_{s2} / 250 = \mathbf{0.36}$ in
Maximum deflection span 2; $\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = \mathbf{0.257}$ in

PASS - Maximum deflection does not exceed deflection limit



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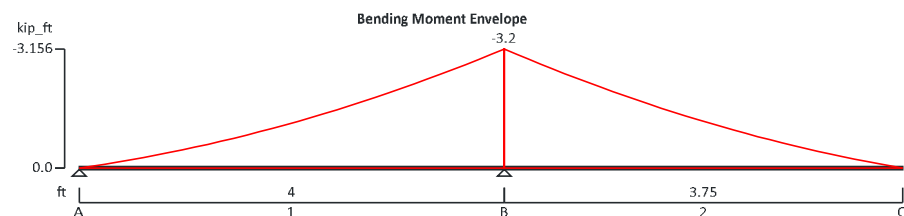
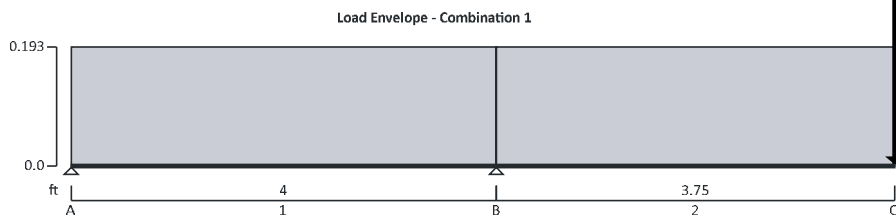
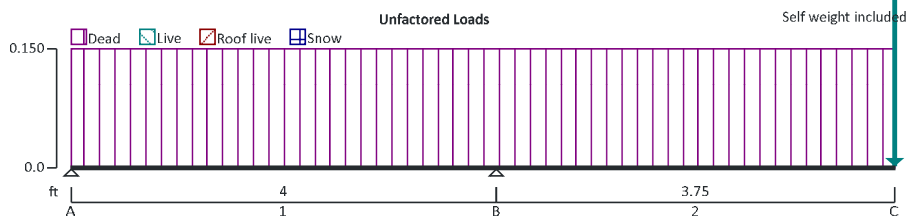
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Project Name: JKV-EMS

ROOF OUTRIGGER DESIGN AND ANALYSIS - POINT LOAD

In accordance with AISC360-16 using the LRFD method

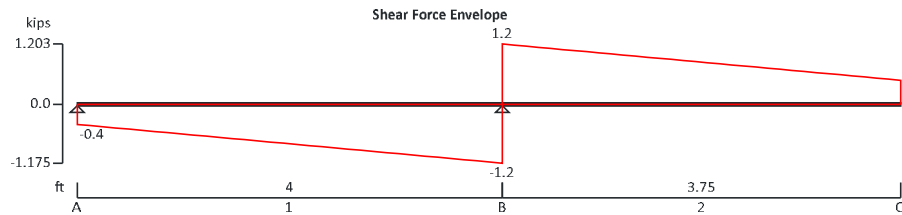
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Revision:	Date:	Sheet No. 186
Project Name: JKV-EMS		



Support conditions

Support A	Vertically restrained Rotationally free
Support B	Vertically restrained Rotationally free
Support C	Vertically free Rotationally free

Applied loading

Beam loads	Dead self weight of beam $\times 1$ Dead full UDL 0.15 kips/ft Live point load 0.3 kips at 93.00 in

Load combinations

Load combination 1	Support A	Dead $\times 1.20$ Live $\times 1.60$ Roof live $\times 1.60$ Snow $\times 1.60$ Dead $\times 1.20$ Live $\times 1.60$ Roof live $\times 1.60$ Snow $\times 1.60$
	Support B	Dead $\times 1.20$ Live $\times 1.60$ Roof live $\times 1.60$ Snow $\times 1.60$ Dead $\times 1.20$ Live $\times 1.60$ Roof live $\times 1.60$ Snow $\times 1.60$
	Support C	Dead $\times 1.20$ Live $\times 1.60$ Roof live $\times 1.60$ Snow $\times 1.60$

Analysis results

Maximum moment;	$M_{\max} = 0$ kips_ft;	$M_{\min} = -3.2$ kips_ft



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Revision:	Date:	Sheet No. 187
Project Name: JKV-EMS		

Maximum moment span 1;
Maximum moment span 2;
Maximum shear;
Maximum shear span 1;
Maximum shear span 2;
Deflection;
Deflection span 1;
Deflection span 2;
Maximum reaction at support A;
Unfactored dead load reaction at support A;
Unfactored live load reaction at support A;
Maximum reaction at support B;
Unfactored dead load reaction at support B;
Unfactored live load reaction at support B;
Maximum reaction at support C;

$M_{s1\_max} = 0$ kips_ft;
 $M_{s2\_max} = 0$ kips_ft;
 $V_{max} = 1.2$ kips;
 $V_{s1\_max} = -0.4$ kips;
 $V_{s2\_max} = 1.2$ kips;
 $\delta_{max} = 0.4$ in;
 $\delta_{s1\_max} = 0$ in;
 $\delta_{s2\_max} = 0.4$ in;
 $R_{A\_max} = -0.4$ kips;
 $R_{A\_Dead} = 0$ kips
 $R_{A\_Live} = -0.3$ kips
 $R_{B\_max} = 2.4$ kips;
 $R_{B\_Dead} = 1.2$ kips
 $R_{B\_Live} = 0.6$ kips
 $R_{C\_max} = 0$ kips;

$M_{s1\_min} = -3.2$ kips_ft
 $M_{s2\_min} = -3.2$ kips_ft
 $V_{min} = -1.2$ kips
 $V_{s1\_min} = -1.2$ kips
 $V_{s2\_min} = 0.5$ kips
 $\delta_{min} = 0$ in
 $\delta_{s1\_min} = 0$ in
 $\delta_{s2\_min} = 0$ in
 $R_{A\_min} = -0.4$ kips

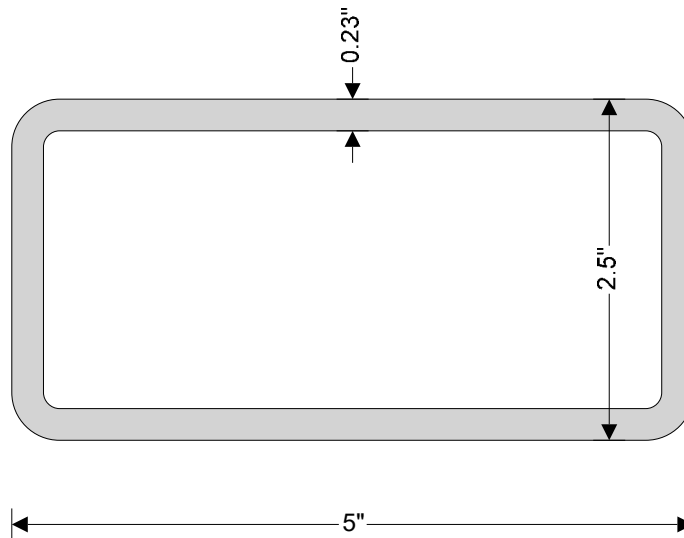
 $R_{B\_min} = 2.4$ kips

 $R_{C\_min} = 0$ kips

Section details

Section type;
ASTM steel designation;
Steel yield stress;
Steel tensile stress;
Modulus of elasticity;

HSS 5x2-1/2x1/4 (AISC 16th Edn (v16.0))
A500 Gr.B 46
 $F_y = 46$ ksi
 $F_u = 58$ ksi
 $E = 29000$ ksi



Resistance factors

Resistance factor for tensile yielding
Resistance factor for tensile rupture
Resistance factor for compression
Resistance factor for flexure

$\phi_{ty} = 0.90$
 $\phi_{tr} = 0.75$
 $\phi_c = 0.90$
 $\phi_b = 0.90$

Lateral bracing

Span 1 has lateral bracing at supports only



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Revision:	Date:	Sheet No. 188
Project Name: JKV-EMS		

Span 2 has lateral bracing at supports only

Cantilever tip is unbraced

Cantilever support is continuous with lateral and torsional restraint

Classification of sections for local buckling - Section B4.1

Classification of flanges in flexure - Table B4.1b (case 17)

Width to thickness ratio; $(d - 3 \times t) / t = \mathbf{18.46}$
Limiting ratio for compact section; $\lambda_{pff} = 1.12 \times \sqrt{E / F_y} = \mathbf{28.12}$
Limiting ratio for non-compact section; $\lambda_{rff} = 1.40 \times \sqrt{E / F_y} = \mathbf{35.15}$; Compact

Classification of web in flexure - Table B4.1b (case 19)

Width to thickness ratio; $(b_f - 3 \times t) / t = \mathbf{7.73}$
Limiting ratio for compact section; $\lambda_{pwf} = 2.42 \times \sqrt{E / F_y} = \mathbf{60.76}$
Limiting ratio for non-compact section; $\lambda_{rwf} = 5.70 \times \sqrt{E / F_y} = \mathbf{143.12}$; Compact

Section is compact in flexure

Design of members for shear - Chapter G

Required shear strength $V_r = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = \mathbf{1.203}$ kips
Web area $A_w = 2 \times (b_f - 3 \times t) \times t = \mathbf{0.839}$ in²
Web plate buckling coefficient $k_v = \mathbf{5}$
Web shear coefficient - eq G2-9 $C_{v2} = \mathbf{1}$
Nominal shear strength – eq G4-1 $V_n = 0.6 \times F_y \times A_w \times C_{v2} = \mathbf{23.164}$ kips
Resistance factor for shear $\phi_v = \mathbf{0.90}$
Design shear strength $V_c = \phi_v \times V_n = \mathbf{20.847}$ kips

PASS - Design shear strength exceeds required shear strength

Design of members for flexure in the minor axis at span 1 - Chapter F

Required flexural strength; $M_r = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = \mathbf{3.156}$ kips_ft

Yielding - Section F7.1

Nominal flexural strength for yielding - eq F7-1; $M_{nyld} = M_p = F_y \times Z_y = \mathbf{11.308}$ kips_ft
Nominal flexural strength; $M_n = M_{nyld} = \mathbf{11.308}$ kips_ft
Design flexural strength; $M_c = \phi_b \times M_n = \mathbf{10.177}$ kips_ft

PASS - Design flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead, live, roof live and snow loads

Limiting deflection; $\delta_{lim} = 2 \times L_{s2} / 250 = \mathbf{0.36}$ in
Maximum deflection span 2; $\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = \mathbf{0.36}$ in



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Job No. 25-003

Revision:

Date:

Sheet No. 189

Project Name: JKV-EMS

ROOF EDGE ANGLE DESIGN

In accordance with AISC360-16 (15th Edition) using the LRFD method

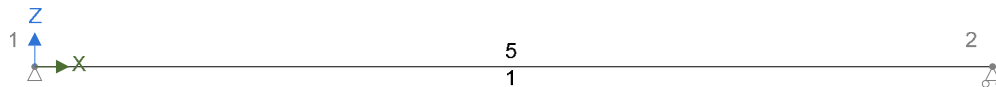
Tedds calculation version 4.5.05

ANALYSIS

Tedds calculation version 1.0.38

Geometry

Geometry (ft) - Steel (AISC) - L 6x6x3/8

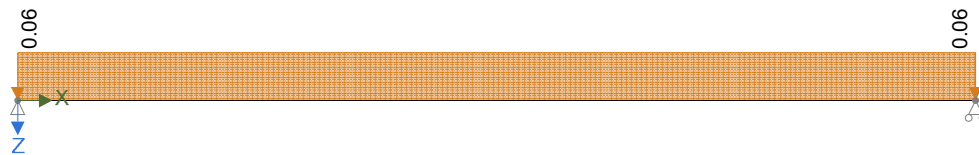


Span	Length (ft)	Section	Start Support	End Support
1	5	L 6x6x3/8	Pinned	Roller Pin X
L 6x6x3/8: Area 4 in ² , Inertia Major 15 in ⁴ , Inertia Minor 15 in ⁴ , Shear area parallel to Minor 0 in ² , Shear area parallel to Major 0 in ²				
Steel (AISC): Density 490 lbm/ft ³ , Youngs 29000 ksi, Shear 11200 ksi, Thermal 0.000012 °C ⁻¹				

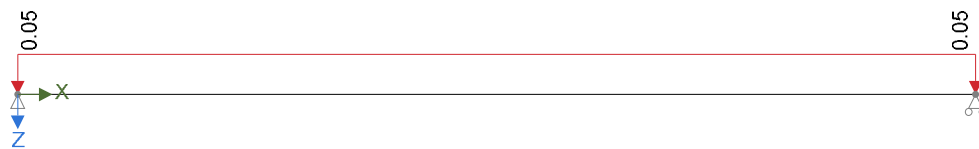
Loading

Self weight included

Dead - Loading (kips/ft)



Live - Loading (kips/ft)



Load combination factors

Load combination	Self Weight	Dead	Live
1.2D + 1.6L (Strength)	1.	1.	1.
	2	2	6
	0	0	0



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Revision:	Date:	Sheet No. 190
Project Name: JKV-EMS		

Load combination	Self Weight	Dead	Live
1.0D + 1.0L (Service)	1.0	1.0	1.0
	0	0	0
	0	0	0

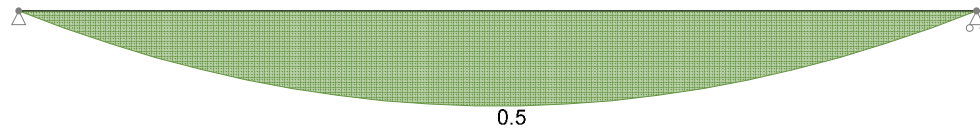
Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Dead	UDL	GlobalZ	0.06 kips/ft
Beam	Live	UDL	GlobalZ	0.05 kips/ft

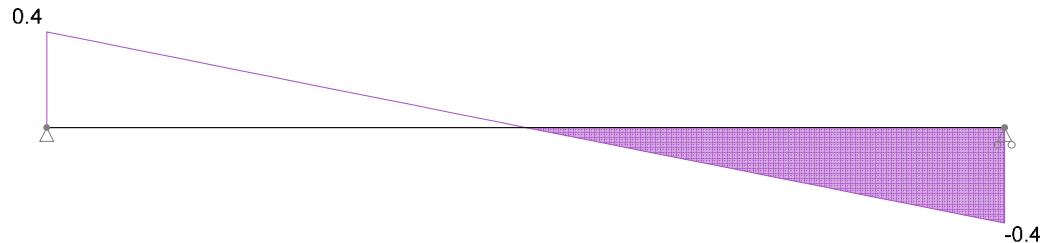
Results

Forces

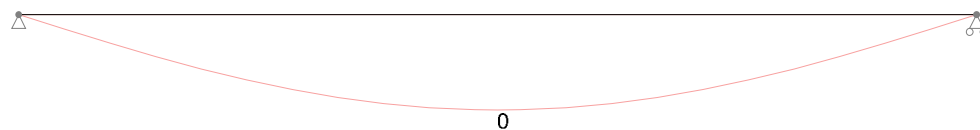
Strength combinations - Moment envelope (kip_ft)



Strength combinations - Shear envelope (kips)



Service combinations - Deflection envelope (in)



Member results

Envelope - Strength combinations

Member	Position (ft)	Shear force (kips)		Moment (kip_ft)	
Beam	0	0.425		0	
	2.5	0		0.531 (max)	



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Revision:	Date:	Sheet No. 191
Project Name: JKV-EMS		

Member	Position (ft)	Shear force (kips)	Moment (kip_ft)
	5	-0.425 (max abs)	0

Envelope - Service combinations

Member	Position (ft)	Deflection (in)
Beam	2.5	0.004 (max)
	5	0

;

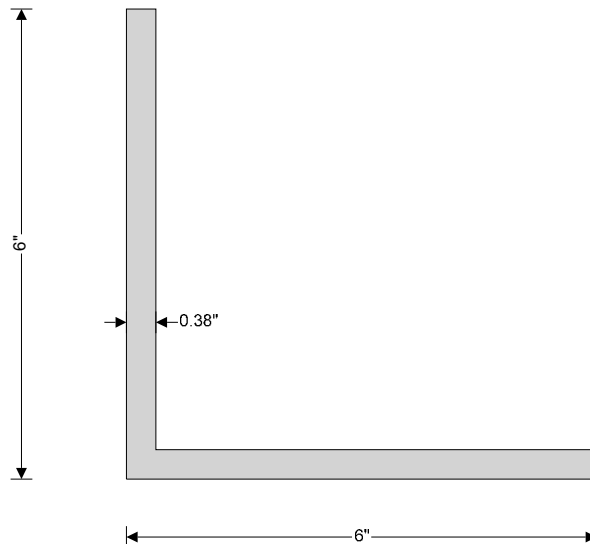
Resistance factors

Shear;	$\phi_v = 0.90$
Flexure;	$\phi_b = 0.90$
Tensile yielding;	$\phi_{t,y} = 0.90$
Tensile rupture;	$\phi_{t,r} = 0.75$
Compression;	$\phi_c = 0.90$

Beam design

Section details

Section type;	L 6x6x3/8 (AISC 16th Edn (v16.0))
ASTM steel designation;	A36
Steel yield stress;	$F_y = 36$ ksi
Steel tensile stress;	$F_u = 58$ ksi
Modulus of elasticity;	$E = 29000$ ksi



L 6x6x3/8 (AISC 16th Edn (v16.0))

Section depth, L_1 , 6 in
Section breadth, L_2 , 6 in
Weight of section, Weight, 14.9 lbf/ft
Section thickness, t , 0.375 in
Area of section, A , 4.4 in²
Radius of gyration about x-axis, r_x , 1.87 in
Radius of gyration about y-axis, r_y , 1.87 in
Elastic section modulus about x-axis, S_x , 3.51 in³
Elastic section modulus about y-axis, S_y , 3.51 in³
Second moment of area about x-axis, I_x , 15.4 in⁴
Second moment of area about y-axis, I_y , 15.4 in⁴



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 192
Project Name: JKV-EMS		

Beam results summary	Unit	Capacity	Maximum	Utilization	Result
Shear resistance (x-x)	kips	43.7	0.4	0.010	PASS
Bending resistance (x-x)	kips_ft	10.2	0.5	0.052	PASS
Deflection (x-x)	in	0.167	0.004	0.024	PASS

Lateral restraint

Both flanges have lateral restraint at supports only

Classification of sections for local buckling - Section B4

Classification of flanges in flexure - Table B4.1b (case 12)

Width to thickness ratio;

$$L_1 / t = 16.00$$

Limiting ratio for compact section;

$$\lambda_{pff} = 0.54 \times \sqrt{E / F_y} = 15.33$$

Limiting ratio for non-compact section;

$$\lambda_{rff} = 0.91 \times \sqrt{E / F_y} = 25.83;$$

Noncompact

Section is noncompact in flexure

Check design at start of span

Design of members for shear - Chapter G

Required shear strength;

$$V_{r,x} = 0.4 \text{ kips}$$

Web area;

$$A_w = L_1 \times t = 2.25 \text{ in}^2$$

Web plate buckling coefficient;

$$k_v = 1.2$$

$$L_1 / t \leq 1.10 \times \sqrt{(k_v \times E / F_y)}$$

Web shear coefficient - eq G2-9;

$$C_{v2} = 1.000$$

Nominal shear strength - eq G3-1;

$$V_{n,x} = 0.6 \times F_y \times L_1 \times t \times C_{v2} = 48.6 \text{ kips}$$

Resistance factor;

$$\phi_v = 0.90$$

Design shear strength;

$$V_{c,x} = \phi_v \times V_{n,x} = 43.7 \text{ kips}$$

$$V_{r,x} / V_{c,x} = 0.01$$

PASS - Design shear strength exceeds required shear strength

Check design 2ft 6in along span

Design of members for flexure - Chapter F

Required flexural strength;

$$M_{r,x} = 0.5 \text{ kips\_ft}$$

Yielding - Section F10.1

Nominal flexural strength for yielding - eq F10-1;

$$M_{n,yld,x} = 1.5 \times F_y \times S_x = 15.8 \text{ kips\_ft}$$

Lateral-torsional buckling - Section F10.2

Unbraced length;

$$L_b = 5 \text{ ft}$$

Width of compression leg;

$$b = L_2 = 6 \text{ in}$$

Yield moment for lateral-torsional buckling;

$$M_y = 0.8 \times F_y \times S_x = 8.424 \text{ kips\_ft}$$

Moment at quarter point of segment;

$$M_A = 0.4 \text{ kips\_ft}$$



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 193
Project Name: JKV-EMS		

Moment at center-line of segment; $M_B = 0.5 \text{ kips\_ft}$
Moment at three quarter point of segment; $M_C = 0.4 \text{ kips\_ft}$
Maximum moment in segment; $M_{\max} = 0.5 \text{ kips\_ft}$
LTB modification factor - eq F1-1; $C_b = 12.5 \times M_{\max} / (2.5 \times M_{\max} + 3 \times M_A + 4 \times M_B + 3 \times M_C) = 1.136$
Yield moment for lateral-torsional buckling; $M_y = 0.8 \times F_y \times S_x = 8.4 \text{ kips\_ft}$
Elastic lateral-torsional buckling moment - eq F10-5a
 $M_{cr} = 0.58 \times E \times b^4 \times t \times C_b \times (\sqrt{(1 + 0.88 \times (L_b \times t / b^2)^2}) - 1) / L_b^2 = 34.2 \text{ kips\_ft}$

Nominal flexural strength for lateral-torsional buckling - eq F10-2
 $M_{n,ltb,x} = \min((1.92 - 1.17 \times \sqrt{(M_y / M_{cr})) \times M_y, 1.5 \times M_y) = 11.3 \text{ kips\_ft}$

Local leg buckling - Section F10.3

Width of leg in compression; $b = L_2 = 6 \text{ in}$
Elastic section modulus for compression; $S_c = 0.8 \times S_x = 2.8 \text{ in}^3$
Nominal flexural strength for leg local buckling - eq F10-6
 $M_{n,llb,x} = F_y \times S_c \times (2.43 - 1.72 \times (b / t) \times \sqrt{(F_y / E)}) = 12.3 \text{ kips\_ft}$

Design flexural strength - F1

Nominal flexural strength; $M_{n,x} = \min(M_{n,yld,x}, M_{n,ltb,x}, M_{n,llb,x}) = 11.3 \text{ kips\_ft}$
Design flexural strength; $M_{c,x} = \phi_b \times M_{n,x} = 10.2 \text{ kips\_ft}$
 $M_{r,x} / M_{c,x} = 0.052$
PASS - Design flexural strength exceeds required flexural strength

Consider Combination 2 - 1.0D + 1.0L (Service)

Check design 2ft 6in along span

Design of members for x-x axis deflection

Maximum deflection; $\delta_x = 0.004 \text{ in}$
Allowable deflection; $\delta_{x,Allowable} = L_{m1\_s1} / 360 = 0.167 \text{ in}$
 $\delta_x / \delta_{x,Allowable} = 0.024$
PASS - Allowable deflection exceeds design deflection

COMBINED FOOTING - INSIDE CORNER COLUMN

In accordance with ACI318-14

Tedds calculation version 3.3.11

Summary results

Overall design status; PASS;
Overall design utilisation; 0.983

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	34.3			Pass
Description	Unit	Applied	Resisting	Utilization	Result
Soil bearing	ksf	1.829	1.86	0.983	Pass



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 194
Project Name: JKV-EMS		

Description	Unit	Applied	Resisting	FoS	Result
Description	Unit	Required	Provided	Utilization	Result
Moment, positive, x-direction	kip_ft	5.6	680.8	0.008	Pass
Moment, positive, y-direction	kip_ft	5.8	680.8	0.009	Pass
Shear, one-way, x-direction	kips	0.4	222.0	0.002	Pass
Shear, one-way, y-direction	kips	0.4	215.2	0.002	Pass
Shear, two-way, Col 1	psi	0.067	189.737	0.000	Pass
Min.area of reinf, bot., x-direction	in ²	4.666	4.740		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	13.0		Pass
Min.area of reinf, bot., y-direction	in ²	4.666	4.740		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	13.0		Pass

Pad footing details

Length of footing;

$$L_x = 6 \text{ ft}$$

Width of footing;

$$L_y = 6 \text{ ft}$$

Footing area;

$$A = L_x \times L_y = 36 \text{ ft}^2$$

Depth of footing;

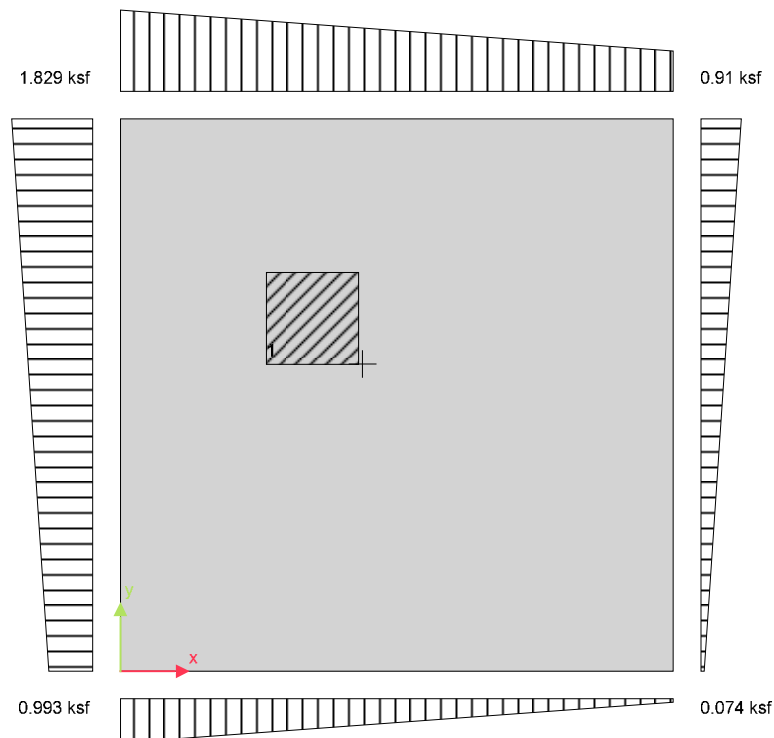
$$h = 36 \text{ in}$$

Depth of soil over footing;

$$h_{\text{soil}} = 0 \text{ in}$$

Density of concrete;

$$\gamma_{\text{conc}} = 150.0 \text{ lb/ft}^3$$





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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 195
Project Name: JKV-EMS		

Column no.1 details

Length of column; $l_{x1} = 12.00$ in
Width of column; $l_{y1} = 12.00$ in
position in x-axis; $x_1 = 25.00$ in
position in y-axis; $y_1 = 46.00$ in

Soil properties

Gross allowable bearing pressure; $q_{allow\_Gross} = 1.86$ ksf;
Density of soil; $\gamma_{soil} = 120.0$ lb/ft³
Angle of internal friction; $\phi_b = 30.0$ deg
Design base friction angle; $\delta_{bb} = 30.0$ deg
Coefficient of base friction; $\tan(\delta_{bb}) = 0.577$
Self weight; $F_{swt} = h \times \gamma_{conc} = 450$ psf

Column no.1 loads

Dead load in z; $F_{Dz1} = 9.2$ kips
Live roof load in z; $F_{Lr1} = 4.5$ kips
Snow load in z; $F_{Sz1} = 4.9$ kips
Wind load in z; $F_{Wz1} = 11.5$ kips

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.620)
1.0D + 1.0L (0.620)
1.0D + 0.6W (0.903)
1.0D + 0.75L + 0.75Lr + 0.45W (0.971)
1.0D + 0.75L + 0.75S + 0.45W (0.983)
1.0D + 0.75L + 0.75R + 0.45W (0.832)

Combination 12 results: 1.0D + 0.75L + 0.75S + 0.45W

Forces on footing

Force in z-axis; $F_{dz} = \gamma_D \times A \times F_{swt} + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_S \times F_{Sz1} + \gamma_W \times F_{Wz1}$
 $= 34.3$ kips

Moments on footing

Moment in x-axis, about x is 0; $M_{dx} = \gamma_D \times A \times F_{swt} \times L_x / 2 + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_S \times (F_{Sz1} \times x_1) + \gamma_W \times (F_{Wz1} \times x_1) = 86.2$ kip_ft
Moment in y-axis, about y is 0; $M_{dy} = \gamma_D \times A \times F_{swt} \times L_y / 2 + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_S \times (F_{Sz1} \times y_1) + \gamma_W \times (F_{Wz1} \times y_1) = 117.8$ kip_ft

Uplift verification

Vertical force; $F_{dz} = 34.25$ kips

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis; $e_{dx} = M_{dx} / F_{dz} - L_x / 2 = -5.797$ in
Eccentricity of base reaction in y-axis; $e_{dy} = M_{dy} / F_{dz} - L_y / 2 = 5.27$ in



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 196
Project Name: JKV-EMS		

Pad base pressures

Minimum base pressure;

Maximum base pressure;

Allowable bearing capacity

Allowable bearing capacity;

$$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{0.993 \text{ ksf}}$$

$$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{1.829 \text{ ksf}}$$

$$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{0.074 \text{ ksf}}$$

$$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{0.91 \text{ ksf}}$$

$$q_{\min} = \min(q_1, q_2, q_3, q_4) = \mathbf{0.074 \text{ ksf}}$$

$$q_{\max} = \max(q_1, q_2, q_3, q_4) = \mathbf{1.829 \text{ ksf}}$$

$$q_{\text{allow}} = q_{\text{allow\_Gross}} = \mathbf{1.86 \text{ ksf}}$$

$$q_{\max} / q_{\text{allow}} = \mathbf{0.983}$$

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN

In accordance with ACI318-14

Tedds calculation version 3.3.11

Material details

Compressive strength of concrete;

Yield strength of reinforcement;

Compression-controlled strain limit (21.2.2);

Cover to top of footing;

Cover to side of footing;

Cover to bottom of footing;

Concrete type;

Concrete modification factor;

Column type;

$$f'_c = \mathbf{4000 \text{ psi}}$$

$$f_y = \mathbf{60000 \text{ psi}}$$

$$\epsilon_{ty} = \mathbf{0.00200}$$

$$c_{nom\_t} = \mathbf{3 \text{ in}}$$

$$c_{nom\_s} = \mathbf{3 \text{ in}}$$

$$c_{nom\_b} = \mathbf{3 \text{ in}}$$

Normal weight

$$\lambda = \mathbf{1.00}$$

Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.008)

1.2D + 1.6L + 0.5Lr (0.009)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on footing

Ultimate force in z-axis;

$$F_{uz} = \gamma_D \times A \times F_{swt} + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_{Lr} \times F_{Lr1} = \mathbf{32.7 \text{ kips}}$$

Moments on footing

Ultimate moment in x-axis, about x is 0;

$$M_{ux} = \gamma_D \times A \times F_{swt} \times L_x / 2 + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_{Lr} \times (F_{Lr1} \times x_1) = \mathbf{86.0 \text{ kip\_ft}}$$

Ultimate moment in y-axis, about y is 0;

$$M_{uy} = \gamma_D \times A \times F_{swt} \times L_y / 2 + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_{Lr} \times (F_{Lr1} \times y_1) = \mathbf{109.3 \text{ kip\_ft}}$$

Eccentricity of base reaction

Eccentricity of base reaction in x-axis;

$$e_{ux} = M_{ux} / F_{uz} - L_x / 2 = \mathbf{-4.467 \text{ in}}$$

Eccentricity of base reaction in y-axis;

$$e_{uy} = M_{uy} / F_{uz} - L_y / 2 = \mathbf{4.06 \text{ in}}$$

Pad base pressures

$$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{0.94 \text{ ksf}}$$



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 197
Project Name: JKV-EMS		

Minimum ultimate base pressure;
Maximum ultimate base pressure;

$$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{1.555 \text{ ksf}}$$

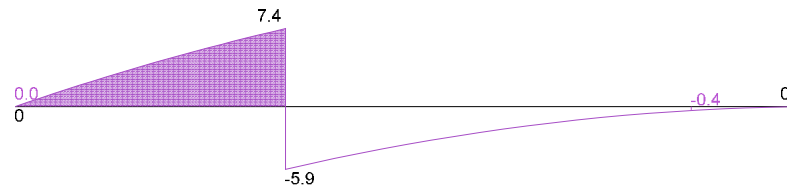
$$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{0.263 \text{ ksf}}$$

$$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{0.878 \text{ ksf}}$$

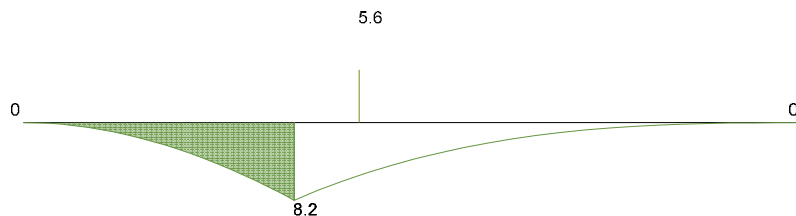
$$q_{u\min} = \min(q_{u1}, q_{u2}, q_{u3}, q_{u4}) = \mathbf{0.263 \text{ ksf}}$$

$$q_{u\max} = \max(q_{u1}, q_{u2}, q_{u3}, q_{u4}) = \mathbf{1.555 \text{ ksf}}$$

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment;

$$M_{u.x.\max} = \mathbf{5.576 \text{ kip\_ft}}$$

Tension reinforcement provided;

6 No.8 bottom bars (13.0 in c/c)

Area of tension reinforcement provided;

$$A_{sx.\text{bot.prov}} = \mathbf{4.74 \text{ in}^2}$$

Minimum area of reinforcement (8.6.1.1);

$$A_{s.\min} = 0.0018 \times L_y \times h = \mathbf{4.666 \text{ in}^2}$$

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2);

$$s_{\max} = \min(2 \times h, 18 \text{ in}) = \mathbf{18 \text{ in}}$$

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement;

$$d = h - c_{\text{nom.b}} - \phi_{y.\text{bot}} - \phi_{x.\text{bot}} / 2 = \mathbf{31.500 \text{ in}}$$

Depth of compression block;

$$a = A_{sx.\text{bot.prov}} \times f_y / (0.85 \times f'_c \times L_y) = \mathbf{1.162 \text{ in}}$$

Neutral axis factor;

$$\beta_1 = \mathbf{0.85}$$

Depth to neutral axis;

$$c = a / \beta_1 = \mathbf{1.367 \text{ in}}$$

Strain in tensile reinforcement;

$$\epsilon_t = 0.003 \times d / c - 0.003 = \mathbf{0.06614}$$

Minimum tensile strain(8.3.3.1);

$$\epsilon_{\min} = 0.004 = \mathbf{0.00400}$$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity;

$$M_n = A_{sx.\text{bot.prov}} \times f_y \times (d - a / 2) = \mathbf{732.783 \text{ kip\_ft}}$$

Flexural strength reduction factor;

$$\phi_f = \min(\max(0.65 + 0.25 \times (\epsilon_t - \epsilon_{ty}) / (0.005 - \epsilon_{ty}), 0.65), 0.9) = \mathbf{0.900}$$

Design moment capacity;

$$\phi M_n = \phi_f \times M_n = \mathbf{659.505 \text{ kip\_ft}}$$

$$M_{u.x.\max} / \phi M_n = \mathbf{0.008}$$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force;

$$V_{u.x} = \mathbf{0.358 \text{ kips}}$$

Depth to reinforcement;

$$d_v = \min(h - c_{\text{nom.b}} - \phi_{x.\text{bot}} / 2, h - c_{\text{nom.t}} - \phi_{x.\text{top}} / 2) = \mathbf{32.5 \text{ in}}$$



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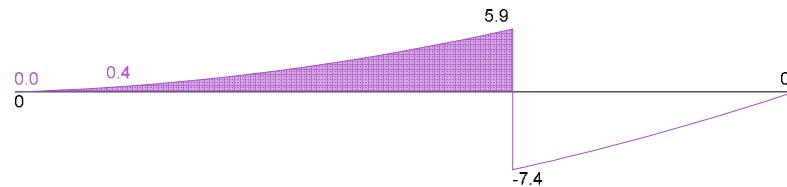
Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 198
Project Name: JKV-EMS		

Shear strength reduction factor;
Nominal shear capacity (Eq. 22.5.5.1);
Design shear capacity;

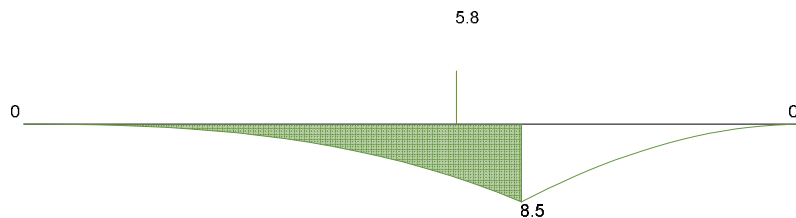
$$\phi_v = 0.75$$
$$V_n = 2 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} \times L_y \times d_v = 295.989 \text{ kips}$$
$$\phi V_n = \phi_v \times V_n = 221.992 \text{ kips}$$
$$V_{u,x} / \phi V_n = 0.002$$

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment;
Tension reinforcement provided;
Area of tension reinforcement provided;
Minimum area of reinforcement (8.6.1.1);

$$M_{u,y,max} = 5.847 \text{ kip\_ft}$$
$$6 \text{ No.8 bottom bars (13.0 in c/c)}$$
$$A_{sy,bot,prov} = 4.74 \text{ in}^2$$
$$A_{s,min} = 0.0018 \times L_x \times h = 4.666 \text{ in}^2$$

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2);

$$s_{max} = \min(2 \times h, 18 \text{ in}) = 18 \text{ in}$$

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement;
Depth of compression block;
Neutral axis factor;
Depth to neutral axis;
Strain in tensile reinforcement;
Minimum tensile strain(8.3.3.1);

$$d = h - c_{nom\_b} - \phi_{y,bot} / 2 = 32.500 \text{ in}$$
$$a = A_{sy,bot,prov} \times f_y / (0.85 \times f'_c \times L_x) = 1.162 \text{ in}$$
$$\beta_1 = 0.85$$
$$c = a / \beta_1 = 1.367 \text{ in}$$
$$\epsilon_t = 0.003 \times d / c - 0.003 = 0.06834$$
$$\epsilon_{min} = 0.004 = 0.00400$$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity;
Flexural strength reduction factor;
Design moment capacity;

$$M_n = A_{sy,bot,prov} \times f_y \times (d - a / 2) = 756.483 \text{ kip\_ft}$$
$$\phi_f = \min(\max(0.65 + 0.25 \times (\epsilon_t - \epsilon_{ty}) / (0.005 - \epsilon_{ty}), 0.65), 0.9) = 0.900$$
$$\phi M_n = \phi_f \times M_n = 680.835 \text{ kip\_ft}$$
$$M_{u,y,max} / \phi M_n = 0.009$$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force;
Depth to reinforcement;

$$V_{u,y} = 0.416 \text{ kips}$$
$$d_v = \min(h - c_{nom\_b} - \phi_{x,bot} - \phi_{y,bot} / 2, h - c_{nom\_t} - \phi_{y,top} / 2) = 31.5 \text{ in}$$



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 199
Project Name: JKV-EMS		

Shear strength reduction factor;
Nominal shear capacity (Eq. 22.5.5.1);
Design shear capacity;

$$\phi_v = \mathbf{0.75}$$
$$V_n = 2 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} \times L_x \times d_v = \mathbf{286.882 \text{ kips}}$$
$$\phi V_n = \phi_v \times V_n = \mathbf{215.161 \text{ kips}}$$
$$V_{u,y} / \phi V_n = \mathbf{0.002}$$

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement;
Shear perimeter length (22.6.4);
Shear perimeter width (22.6.4);
Shear perimeter (22.6.4);
Shear area;
Surcharge loaded area;
Ultimate bearing pressure at center of shear area;
Ultimate shear load;

$$d_{v2} = \mathbf{32 \text{ in}}$$
$$l_{xp} = \mathbf{44.000 \text{ in}}$$
$$l_{yp} = \mathbf{44.000 \text{ in}}$$
$$b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2}) = \mathbf{176.000 \text{ in}}$$
$$A_p = l_{x,perim} \times l_{y,perim} = \mathbf{1936.000 \text{ in}^2}$$
$$A_{sur} = A_p - l_{x1} \times l_{y1} = \mathbf{1792.000 \text{ in}^2}$$
$$q_{up,avg} = \mathbf{1.474 \text{ ksf}}$$
$$F_{up} = \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_{Lr} \times F_{Lrz1} + \gamma_D \times A_p \times F_{swt} - q_{up,avg} \times A_p = \mathbf{0.733 \text{ kips}}$$

Ultimate shear stress from vertical load;
Column geometry factor (Table 22.6.5.2);
Column location factor (22.6.5.3);
Concrete shear strength (22.6.5.2);

$$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0 \text{ psi}) = \mathbf{0.130 \text{ psi}}$$
$$\beta = l_{y1} / l_{x1} = \mathbf{1.00}$$
$$\alpha_s = \mathbf{40}$$
$$v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} = \mathbf{379.473 \text{ psi}}$$
$$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} = \mathbf{586.459 \text{ psi}}$$
$$v_{cpc} = 4 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} = \mathbf{252.982 \text{ psi}}$$
$$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) = \mathbf{252.982 \text{ psi}}$$

Shear strength reduction factor;
Nominal shear stress capacity (Eq. 22.6.1.2);
Design shear stress capacity (8.5.1.1(d));

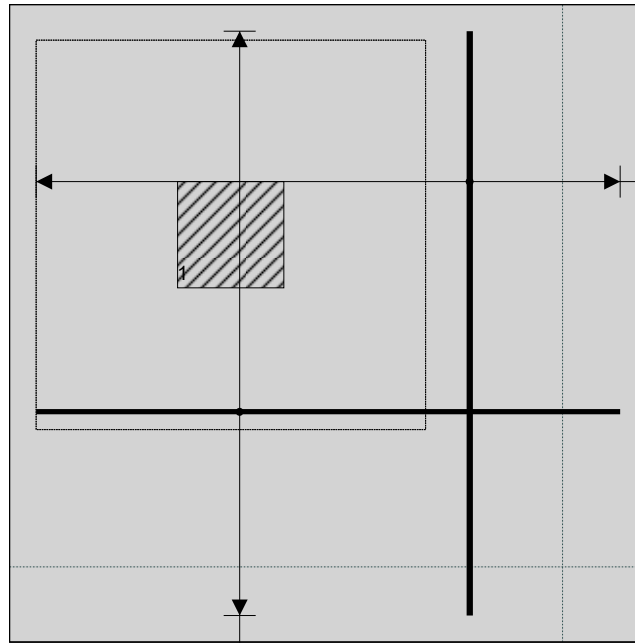
$$\phi_v = \mathbf{0.75}$$
$$V_n = v_{cp} = \mathbf{252.982 \text{ psi}}$$
$$\phi V_n = \phi_v \times V_n = \mathbf{189.737 \text{ psi}}$$
$$V_{ug} / \phi V_n = \mathbf{0.001}$$

PASS - Design shear stress capacity exceeds ultimate shear stress load



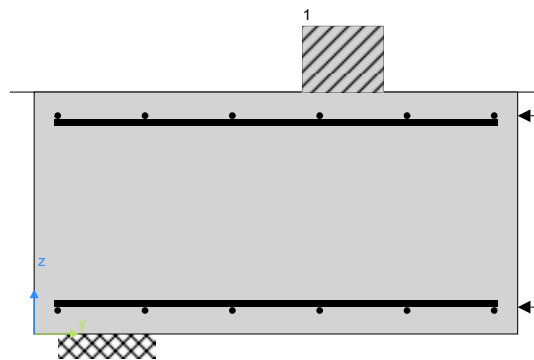
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Revision:	Date:	Sheet No. 200
Project Name: JKV-EMS		



6 No.8 bottom bars (13 in c/c)
6 No.8 top bars (13 in c/c)

6 No.8 bottom bars (13 in c/c)
6 No.8 top bars (13 in c/c)



6 No.8 top bars (13.0 in c/c) (y direction)
6 No.8 top bars (13.0 in c/c) (x direction)

6 No.8 bottom bars (13.0 in c/c) (y direction)
6 No.8 bottom bars (13.0 in c/c) (x direction)



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Revision:	Date:	Sheet No. 201
Project Name: JKV-EMS		

ROOF OUTRIGGER

$$\text{DEAD} = 15 \text{ psf}$$

$$\text{ROOF LINE} = 20 \text{ psf}$$

$$\text{SNOW} = 22 \text{ psf}$$

$$\text{MEP} = 15 \text{ psf}$$

$$\text{DEAD} = 30 \text{ psf}$$

5 ft o.c.

$$W_D = 30(5) = 150 \text{ lb/ft}$$

$$V = 1,017 \text{ k}$$

$$W_{LS} = 22(5) = 110 \text{ lb/ft}$$

$$M = 1,903 \text{ ft k}$$

$$W_T = 150 + 110 = 260 \text{ lb/ft}$$

4 ft o.c.

$$W_D = 30(4) = 120 \text{ lb/ft}$$

$$W_{LS} = 22(4) = 88 \text{ lb/ft}$$

$$W_T = 120 + 88 = 208 \text{ lb/ft}$$

USING HSS 5 x 2 1/2 x 1/4 (SEE TEDDS CALL)

JOIST TOP CHORD EXTENSION

SMALLEST REINFORCED TCX IS CAPABLE
(SEE VULCRAFT LOAD TABLES)

CHECK DEFLECTION

$$\Delta = \frac{W l^4}{8 E I}$$

$$W = 260 \text{ lb/ft} = 0.26 \text{ k/ft} = 0.021667 \text{ k/in}$$

$$l = 3' - 1 1/4" = 37.25 \text{ in}$$

$$\Delta = \frac{0.021667(37.25)^4}{8(29000)(I)}$$

$$0.2 = \frac{41716.1}{8(29000)(I)}$$

$$I_{req} = \frac{41716.1}{232000(0.2)}$$

$$I_{req} = 0.899054 \text{ in}^4$$

USE R10 TCX (REVISE FOR CMU CHANGE)

↑
NOT USED

$$l = 2' - 0" = 24 \text{ in}$$

$$\Delta = \frac{0.021667(24)^4}{8(29000)(I)}$$

$$0.1 = \frac{7188.59}{232000(I)}$$

$$I_{req} = \frac{7188.59}{232000(0.1)}$$

$$I_{req} = 0.309853$$

* USE S9 TCX



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Revision:	Date:	Sheet No. 202
Project Name: JKV-EMS		

ADDITIONAL ROOF LOAD TO CMU FROM OVERHANG

CONSIDER 2'-0"

DEAD = 30 PSF

LIVE = 25 PSF

$TW = 2 \text{ ft}$

$WD = 2(30) = 60 \text{ lb/ft}$

$WL = 2(25) = 50 \text{ lb/ft}$

LOADS TO LINTEL @ APP BAY DOORS

$TW = \frac{34.75}{2} + 2.25 = 15.445 \text{ ft}$

$D = 30 \text{ PSF}$

$L = 25 \text{ PSF}$

5 ft OF BLOCK OVER DOOR

$WD = 15.45(30) = 463.5 \text{ lb/ft}$

$WL = 15.45(25) = 386.25 \text{ lb/ft}$

$W_{SW} = 85(5) = 425 \text{ lb/ft}$

$WT = 463.5 + 386.25 + 425 = 1274.75 \text{ lb/ft}$

$M = \frac{1.27475(12.67)^2}{8} = 25.6244 \text{ ft-k}$

$V = \frac{1.27475(12.67)}{2} = 8.1 \text{ k}$

LOADS TO LINTEL @ APP BAY WINDOWS

$TW = \frac{5}{2} + 2 = 4.5 \text{ ft}$

$WD = 4.5(30) = 135 \text{ lb/ft}$

$WL = 4.5(25) = 112.5 \text{ lb/ft}$

$W_{SW} = 85(5) = 425 \text{ lb/ft}$

$WT = 135 + 112.5 + 425 = 672.5 \text{ lb/ft}$

$M = \frac{0.6725(8)^2}{8} = 5.38 \text{ ft-k}$

$V = \frac{0.6725(8)}{2} = 2.69 \text{ ft-k}$

USE 8" WIDE 16" DEEP MASONRY LINTEL W/(2) #5

MINOR AXIS LOADING

$TW = 5 \text{ ft}$

$WIND = 35 \text{ PSF}$

$W = 35(5) = 175 \text{ lb/ft}$

$M = \frac{0.175(12.67)^2}{8} = 3.52 \text{ ft-k}$

$V = \frac{0.175(12.67)}{2} = 1.11 \text{ k}$

CHECK DEFLECTION W8x23

$\Delta = \frac{5WL^4}{384EI} = \frac{5(0.106417)(152)^4}{384(29000)(98)} = 0.261 \text{ in NG}$

$\frac{L}{600} = \frac{152}{600} = 0.2533$

CHECK W10x31

$\Delta = \frac{5(0.106417)(152)^4}{384(29000)(375)} = 0.068013 \text{ in } \checkmark$



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Revision:	Date:	Sheet No. 203
Project Name: JKV-EMS		

CONNECTIONS



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Revision:	Date:	Sheet No. 204
Project Name: JKV-EMS		

Project:
Project no:
Author:



Project data

Project name
Project number
Author
Description
Date 6/16/2025
Code AISC/ACI

Material

Steel A500B-46, A992-50, A36, A572 Gr.50
Concrete 4000 psi



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Date: 06/16/2025

Job No. 25-003

Revision:

Date:

Sheet No. 205

Project Name: JKV-EMS

Project:
Project no:
Author:



Project item [0.00, 8991.60, -304.80]

Design

Name [0.00, 8991.60, -304.80]
Description
Analysis Stress, strain/ loads in equilibrium
Design code AISC 360-16 (LRFD) / ACI 318-14

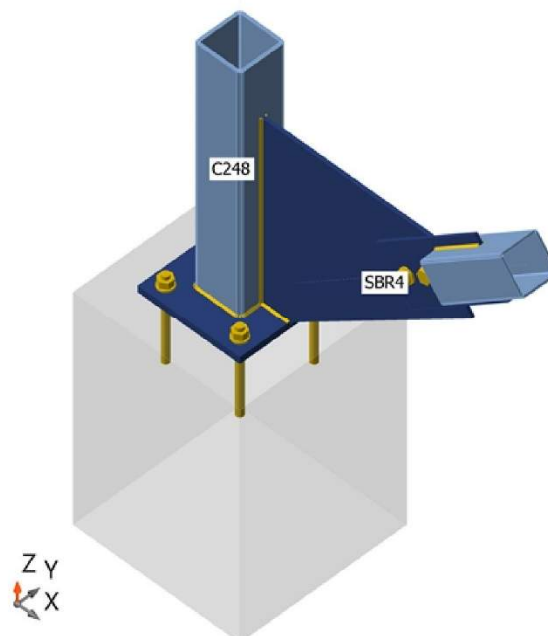
Members

Geometry

Name	Cross-section	β - Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [in]	Offset ey [in]	Offset ez [in]
SBR4	12 - HSS 4x4x0.25	-180.0	-38.3	0.0	0.00	0.00	0.00
C248	6 - HSS 5x5x3/8	0.0	90.0	0.0	0.00	0.00	0.00

Supports and forces

Name	Support	Forces in	X [in]
SBR4 / begin		Position	0.00
C248 / end		Position	0.00





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Revision:	Date:	Sheet No. 206
Project Name: JKV-EMS		

Project:
Project no:
Author:



Cross-sections

Name	Material
12 - HSS 4x4x0.25	A500B-46
6 - HSS 5x5x3/8	A500B-46

Anchors / Bolts

Name	Diameter [in]	f_y [ksi]	f_u [ksi]	Gross area [in ²]
3/4 F1554 Gr.36	0.75	36.0	58.0	0.44
3/4 A325	0.75	92.0	120.0	0.44

Load effects (forces in equilibrium)

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
69 LRFD_{10.8}-1.2D+L+0.5S+W(1)	SBR4 / Begin	3.46	0.00	0.00	0.00	0.00	0.00
	C248 / End	-4.15	0.00	0.00	0.00	0.00	0.00
127 LRFD_{13.2}-0.9D+E(2)	SBR4 / Begin	-2.93	0.00	0.00	0.00	0.00	0.00
	C248 / End	-2.92	0.00	0.00	0.00	0.00	0.00
10 LRFD_{6.1}-1.2D+L+1.6S(3)	SBR4 / Begin	0.41	0.00	0.00	0.00	0.00	0.00
	C248 / End	-6.46	0.00	0.00	0.00	0.00	0.00
108 LRFD_{12.7}-0.9D+W(4)	SBR4 / Begin	3.29	0.00	0.00	0.00	0.00	0.00
	C248 / End	-1.24	0.00	0.00	0.00	0.00	0.00
131 LRFD_{13.6}-0.9D+E(5)	SBR4 / Begin	1.39	0.00	0.00	0.00	0.00	0.00
	C248 / End	-2.92	0.00	0.00	0.00	0.00	0.00
124 LRFD_{11.7}-1.2D+L+0.2S+E(6)	SBR4 / Begin	-1.01	0.00	0.00	0.00	0.00	0.00
	C248 / End	-4.34	0.00	0.00	0.00	0.00	0.00

Unbalanced forces

Name	X [kip]	Y [kip]	Z [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
69 LRFD_{10.8}-1.2D+L+0.5S+W(1)	-2.71	0.00	-6.29	0.07	0.00	-0.08
127 LRFD_{13.2}-0.9D+E(2)	2.30	0.00	-1.10	-0.06	0.00	0.07
10 LRFD_{6.1}-1.2D+L+1.6S(3)	-0.32	0.00	-6.71	0.01	0.00	-0.01
108 LRFD_{12.7}-0.9D+W(4)	-2.58	0.00	-3.28	0.06	0.00	-0.08
131 LRFD_{13.6}-0.9D+E(5)	-1.09	0.00	-3.78	0.03	0.00	-0.03
124 LRFD_{11.7}-1.2D+L+0.2S+E(6)	0.79	0.00	-3.72	-0.02	0.00	0.02

Foundation block

Item	Value	Unit
CB 1		
Dimensions	18.00 x 18.00	in
Depth	25.00	in
Anchor	3/4 F1554 Gr.36	
Anchoring length	7.50	in
Shear force transfer	Anchors	



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Revision:	Date:	Sheet No. 207
Project Name: JKV-EMS		

Project:
Project no:
Author:



Check

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	$0.0 < 5.0\%$	OK
Loc. deformation	$0.0 < 3\%$	OK
Bolts	$10.0 < 100\%$	OK
Anchors	$63.3 < 100\%$	OK
Welds	$30.2 < 100\%$	OK
Concrete block	$2.0 < 100\%$	OK
Buckling	30.15	

Plates

Name	Material	t_p [in]	Loads	σ_{Ed} [ksi]	ϵ_{pl} [%]	$\sigma_{c,Ed}$ [ksi]	Status
SBR4	A500B-46	1/4	69 LRFD_{10.8}-1.2D+L+0.5S+W(1)	2.0	0.0	0.0	OK
C248	A500B-46	3/8	69 LRFD_{10.8}-1.2D+L+0.5S+W(1)	3.3	0.0	0.0	OK
BP1	A500B-46	3/4	69 LRFD_{10.8}-1.2D+L+0.5S+W(1)	1.7	0.0	0.0	OK
SP1	A36	3/8	127 LRFD_{13.2}-0.9D+E(2)	8.9	0.0	0.7	OK
CPL2	A500B-46	3/8	69 LRFD_{10.8}-1.2D+L+0.5S+W(1)	5.9	0.0	0.4	OK

Design data

Material	F_y [ksi]	ϵ_{lim} [%]
A500B-46	46.0	5.0
A36	36.0	5.0

Symbol explanation

t_p	Plate thickness
σ_{Ed}	Equivalent stress
ϵ_{pl}	Plastic strain
$\sigma_{c,Ed}$	Contact stress
F_y	Yield strength
ϵ_{lim}	Limit of plastic strain

Detailed result for SBR4

Design values used in the analysis

$$\phi F_y = 41.4 \text{ ksi}$$

Where:

$$F_y = 46.0 \text{ ksi} \quad \text{-- characteristic yield strength}$$

$$\phi = 0.90 \quad \text{-- resistance factor for steel material AISC 360-16 -- B3.1}$$

Loc. deformation

Name	d_0 [in]	Loads	δ [in]	δ_{lim} [in]	δ/d_0 [%]	Check status
SBR4	4.00	124 LRFD_{11.7}-1.2D+L+0.2S+E(6)	0.00	0.12	0.0	OK
C248	5.00	124 LRFD_{11.7}-1.2D+L+0.2S+E(6)	0.00	0.15	0.0	OK



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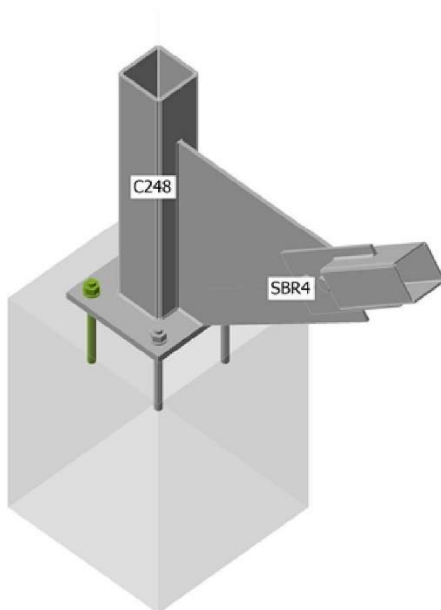
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Revision:	Date:	Sheet No. 208
Project Name: JKV-EMS		

Project:
Project no:
Author:



Symbol explanation

d_0 Cross-section size
 δ Local cross-section deformation
 δ_{lim} Allowed deformation



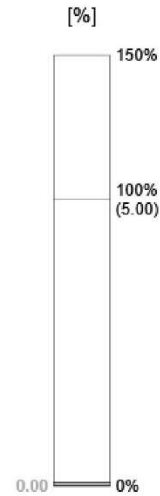
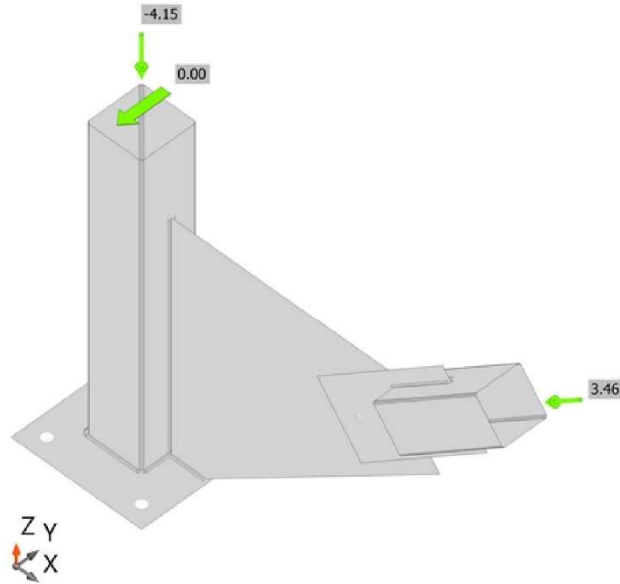
Overall check, 69 LRFD_{10.8}-1.2D+L+0.5S+W(1)



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Revision:	Date:	Sheet No. 209
Project Name: JKV-EMS		

Project:
Project no:
Author:



Strain check, 69 LRFD_{10.8}-1.2D+L+0.5S+W(1)



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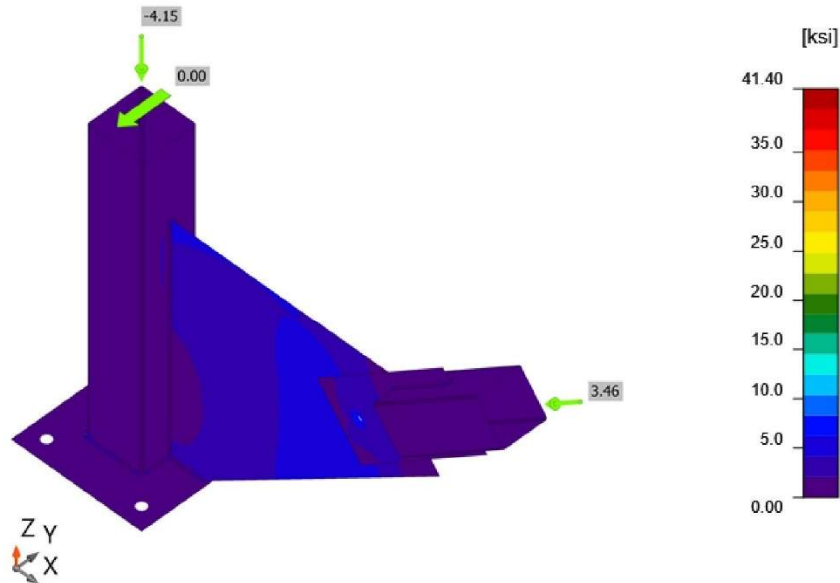
Date:

Sheet No. 210

Project Name: JKV-EMS

Project:
Project no:
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IDEAS StatiCa®
Discrete plate-slab analysis



Equivalent stress, 69 LRFD_{10.8}-1.2D+L+0.5S+W(1)

Bolts

Shape	Item	Grade	Loads	F _t [kip]	V [kip]	φR _{n,bearing} [kip]	U _t [%]	U _s [%]	U _{ts} [%]	Detailing	Status
	B5	3/4 A325 - 1	69 LRFD_{10.8}-1.2D+L+0.5S+W(1)	0.33	1.78	29.38	1.1	10.0	0.0	OK	OK
	B6	3/4 A325 - 1	69 LRFD_{10.8}-1.2D+L+0.5S+W(1)	0.23	1.68	29.38	0.8	9.4	0.0	OK	OK

Design data

Grade	φR _{n,tension} [kip]	φR _{n,shear} [kip]
3/4 A325 - 1	29.79	17.88

Symbol explanation

F _t	Tension force
V	Resultant of bolt shear forces V _y and V _z in shear planes
φR _{n,bearing}	Bolt bearing resistance
U _t	Utilization in tension
U _s	Utilization in shear
U _{ts}	Utilization in tension and shear
φR _{n,tension}	Bolt tension resistance - AISC 360-16 – J3.6
φR _{n,shear}	Bolt shear resistance - AISC 360-16 – J3.6



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Date: 06/16/2025

Job No. 25-003

Revision:

Date:

Sheet No. 211

Project Name: JKV-EMS

Project:
Project no:
Author:



Detailed result for B5

Tension resistance check (AISC 360-16 – J3-1)

$$\phi R_n = \phi \cdot F_{nt} \cdot A_b = 29.79 \text{ kip} \geq F_t = 0.33 \text{ kip}$$

Where:

$F_{nt} = 89.9 \text{ ksi}$ – nominal tensile stress AISC 360-16 – Table J3.2

$A_b = 0.44 \text{ in}^2$ – gross bolt cross-sectional area

$\phi = 0.75$ – resistance factor

Shear resistance check (AISC 360-16 – J3-1)

$$\phi R_n = \phi \cdot F_{nv} \cdot A_b = 17.88 \text{ kip} \geq V = 1.78 \text{ kip}$$

Where:

$F_{nv} = 54.0 \text{ ksi}$ – nominal shear stress AISC 360-16 – Table J3.2

$A_b = 0.44 \text{ in}^2$ – gross bolt cross-sectional area

$\phi = 0.75$ – resistance factor

Bearing resistance check (AISC 360-16 – J3-6)

$$R_n = 1.20 \cdot l_c \cdot t \cdot F_u \leq 2.40 \cdot d \cdot t \cdot F_u$$

$$\phi R_n = 29.38 \text{ kip} \geq V = 1.78 \text{ kip}$$

Where:

$l_c = 2.19 \text{ in}$ – clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material

$t = 3/8 \text{ in}$ – thickness of the plate

$d = 3/4 \text{ in}$ – diameter of a bolt

$F_u = 58.0 \text{ ksi}$ – specified minimum tensile strength of the connected material

$\phi = 0.75$ – resistance factor for bearing at bolt holes

Interaction of tension and shear check (AISC 360-16 – J3-2)

The required stress, in either shear or tension, is less than or equal to 30% of the corresponding available stress and the effects of combined stresses need not to be investigated.

Anchors

Shape	Item	Loads	N_f [kip]	V [kip]	ϕV_{cbg} [kip]	ϕV_{cp} [kip]	U_t [%]	U_s [%]	U_{ts} [%]	Detailing	Status
	A1	127 LRFD_{(13.2)-0.9D+E(2)}	0.00	0.55	4.29	38.71	0.0	53.6	35.4	OK	OK
	A2	69 LRFD_{(10.8)-1.2D+L+0.5S+W(1)}	0.00	0.65	4.29	38.71	0.0	63.3	46.6	OK	OK
	A3	127 LRFD_{(13.2)-0.9D+E(2)}	0.00	0.61	4.29	38.71	0.0	53.6	35.4	OK	OK
	A4	69 LRFD_{(10.8)-1.2D+L+0.5S+W(1)}	0.00	0.71	4.29	38.71	0.0	63.3	46.6	OK	OK

Design data

Grade	ϕN_{sa} [kip]	ϕV_{sa} [kip]
3/4 F1554 Gr.36 - 2	13.56	7.56



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Revision:	Date:	Sheet No. 212
Project Name: JKV-EMS		

Project:
Project no:
Author:



Symbol explanation

N_f	Tension force
V	Resultant of bolt shear forces V_y and V_z in shear planes
ϕV_{cbg}	Concrete breakout strength in shear - ACI 318-14 – 17.5.2
ϕV_{cp}	Concrete pryout strength in shear - ACI 318-14 – 17.5.3
U_t	Utilization in tension
U_s	Utilization in shear
U_{ts}	Utilization in tension and shear
ϕN_{sa}	Steel strength of anchor in tension - ACI 318-14 – 17.4.1
ϕV_{sa}	Steel strength of anchor in shear - ACI 318-14 – 17.5.1

Detailed result for A2

Anchor tensile resistance (ACI 318-14 – 17.4.1)

$$\phi N_{sa} = \phi \cdot A_{se,N} \cdot f_{uta} = 13.56 \text{ kip} \geq N_f = 0.00 \text{ kip}$$

Where:

$$\phi = 0.70 \quad \text{– resistance factor}$$

$$A_{se,N} = 0.33 \text{ in}^2 \quad \text{– tensile stress area}$$

$$f_{uta} = 58.0 \text{ ksi} \quad \text{– specified tensile strength of anchor steel:}$$

- $f_{uta} = \min(125 \text{ ksi}, 1.9 \cdot f_{ya}, f_u)$, where:
 - $f_{ya} = 36.0 \text{ ksi}$ – specified yield strength of anchor steel
 - $f_u = 58.0 \text{ ksi}$ – specified ultimate strength of anchor steel

Shear resistance (ACI 318-14 – 17.5.1)

$$\phi V_{sa} = \phi \cdot 0.6 \cdot A_{se,V} \cdot f_{uta} = 7.56 \text{ kip} \geq V = 0.65 \text{ kip}$$

Where:

$$\phi = 0.65 \quad \text{– resistance factor}$$

$$A_{se,V} = 0.33 \text{ in}^2 \quad \text{– tensile stress area}$$

$$f_{uta} = 58.0 \text{ ksi} \quad \text{– specified tensile strength of anchor steel:}$$

- $f_{uta} = \min(125 \text{ ksi}, 1.9 \cdot f_{ya}, f_u)$, where:
 - $f_{ya} = 36.0 \text{ ksi}$ – specified yield strength of anchor steel
 - $f_u = 58.0 \text{ ksi}$ – specified ultimate strength of anchor steel



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Revision:	Date:	Sheet No. 213
Project Name: JKV-EMS		

Project:
Project no:
Author:



Concrete shear breakout check (ACI 318-14 – 17.5.2)

The check is performed for group of anchors that form common shear breakout cone: A2, A4

$$\phi V_{cbg} = \phi \cdot \frac{A_{Vc}}{A_{Vc0}} \cdot \Psi_{ec,V} \cdot \Psi_{ed,V} \cdot \Psi_{c,V} \cdot \Psi_{h,V} \cdot \Psi_{\alpha,V} \cdot V_{br} = 4.29 \text{ kip} \geq V_g = 2.71 \text{ kip}$$

Where:

$V_g = 2.71 \text{ kip}$ – sum of shear forces of anchors on common base plate

$\phi = 0.65$ – resistance factor

$A_{Vc} = 135.24 \text{ in}^2$ – projected concrete failure area of an anchor or group of anchors

$A_{Vc0} = 112.50 \text{ in}^2$ – projected concrete failure area of one anchor when not limited by corner influences, spacing or member thickness

$\Psi_{ec,V} = 0.98$ – modification factor for anchor groups loaded eccentrically in shear:

- $\Psi_{ec,V} = \frac{1}{1 + \frac{2e'_V}{3c_{a1}}}$, where:
 - $e'_V = 0.19 \text{ in}$ – shear load eccentricity
 - $c_{a1} = 5.00 \text{ in}$ – edge distance in direction of the load
 - $c_{a1} = 5.00 \text{ in}$ – edge distance in direction of the load

$\Psi_{ed,V} = 0.90$ – modification factor for edge effect:

- $\Psi_{ed,V} = 0.7 + 0.3 \cdot \frac{c_{a2}}{1.5 \cdot c_{a1}} \leq 1$, where:
 - $c_{a1} = 5.00 \text{ in}$ – edge distance in direction of the load
 - $c_{a1} = 5.00 \text{ in}$ – edge distance in direction of the load
 - $c_{a2} = 5.00 \text{ in}$ – edge distance in direction perpendicular to the load

$\Psi_{c,V} = 1.00$ – modification factor for concrete conditions

$\Psi_{h,V} = 1.00$ – modification factor for anchors located in a shallow concrete member:

- $\Psi_{h,V} = \sqrt{\frac{1.5 \cdot c_{a1}}{h_a}} \geq 1$, where:
 - $h_a = 25.00 \text{ in}$ – thickness of member in which an anchor is anchored measured parallel to anchor axis

$\Psi_{\alpha,V} = 1.00$ – modification factor for anchors loaded at an angle with the concrete edge

- $\Psi_{\alpha,V} = \sqrt{\frac{1}{(\cos \alpha_V)^2 + (0.5 \sin \alpha_V)^2}}$, where:
 - $\alpha_V = 0.2^\circ$ – angle between direction of shear force and direction perpendicular to concrete edge

$V_b = 6.25 \text{ kip}$ – basic concrete breakout strength of a single anchor in shear:

- $V_b = \min(7.0 \cdot \left(\frac{l_c}{d_a}\right)^{0.2} \cdot \lambda_a \cdot \sqrt{d_a} \cdot \sqrt{f'_c} \cdot c_{a1}^{1.5}, 9.0 \cdot \lambda_a \cdot \sqrt{f'_c} \cdot c_{a1}^{1.5})$, where:
 - $l_c = 6.00 \text{ in}$ – effective length
 - $d_a = 3/4 \text{ in}$ – anchor diameter
 - $\lambda_a = 1.00$ – modification factor for lightweight concrete
 - $f'_c = 4.0 \text{ ksi}$ – concrete compressive strength
 - $c_{a1} = 5.00 \text{ in}$ – edge distance in direction of the load
 - $c_{a1} = 5.00 \text{ in}$ – edge distance in direction of the load



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 214
Project Name: JKV-EMS		

Project:
Project no:
Author:



Concrete pryout resistance (ACI 318-14 – 17.5.3)

The check is performed for group of anchors on common base plate

$$\phi V_{cp} = \phi \cdot k_{cp} \cdot N_{cp} = 38.71 \text{ kip} \geq V_g = 2.72 \text{ kip}$$

Where:

$\phi = 0.65$ – resistance factor

$k_{cp} = 2.00$ – concrete pry-out factor

$N_{cp} = 29.77 \text{ kip}$ – concrete cone tension break-out resistance in case all anchors are in tension

$V_g = 2.72 \text{ kip}$ – sum of shear forces of anchors on common base plate

Interaction of tensile and shear forces (ACI 318-14 – R17.6)

$$U_{tt}^{5/3} + U_{ts}^{5/3} = 0.47 \leq 1.0$$

Where:

$U_{tt} = 0.00$ – maximum ratio of factored tensile force and tensile resistance determined from all appropriate failure modes

$U_{ts} = 0.63$ – maximum ratio of factored shear force and shear resistance determined from all appropriate failure modes

Welds

Item	Edge	Xu	t _w [in]	w [in]	L [in]	L _c [in]	Loads	F _n [kip]	φR _n [kip]	U _t [%]	U _t [%]	Detailing	Status
C248-w 1	SP1	E70xx	1/8	3/16	19.87	0.31	69 LRFD_{10.8}-1.2D+L+0.5S+W(1)	0.12	1.49	8.2	0.0	OK	OK
		E70xx	1/8	3/16	19.87	0.31	69 LRFD_{10.8}-1.2D+L+0.5S+W(1)	0.12	1.50	8.0	0.0	OK	OK
CPL2	SBR4-w 1	E70xx	1/8	3/16	5.96	0.31	69 LRFD_{10.8}-1.2D+L+0.5S+W(1)	0.17	1.31	12.7	0.0	OK	OK
CPL2	SBR4-w 1	E70xx	1/8	3/16	5.96	0.31	69 LRFD_{10.8}-1.2D+L+0.5S+W(1)	0.17	1.31	12.6	0.0	OK	OK
CPL2	SBR4-w 3	E70xx	1/8	3/16	5.96	0.31	69 LRFD_{10.8}-1.2D+L+0.5S+W(1)	0.17	1.31	12.6	0.0	OK	OK
CPL2	SBR4-w 3	E70xx	1/8	3/16	5.96	0.31	69 LRFD_{10.8}-1.2D+L+0.5S+W(1)	0.17	1.31	12.6	0.0	OK	OK
BP1	SP1	E70xx	1/8	3/16	2.99	0.60	69 LRFD_{10.8}-1.2D+L+0.5S+W(1)	0.32	3.29	9.7	0.0	OK	OK
		E70xx	1/8	3/16	2.99	0.60	69 LRFD_{10.8}-1.2D+L+0.5S+W(1)	0.24	3.05	8.0	0.0	OK	OK
BP1	C248	E70xx	1/8	3/16	18.09	0.11	69 LRFD_{10.8}-1.2D+L+0.5S+W(1)	0.14	0.46	30.2	15.8	OK	OK

Design data

Material	F _{exx} [ksi]
E70xx	70.0



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Made by: WNH

Date: 06/16/2025

Job No. 25-003

Revision:

Date:

Sheet No. 215

Project Name: JKV-EMS

Project:
Project no:
Author:



Symbol explanation

t_w	Throat thickness of weld
w	Leg size of weld
L	Length of weld
L_c	Length of weld critical element
F_n	Force in weld critical element
ϕR_n	Weld resistance - AISC 360-16 – J2-4
U_t	Utilization
U_{t_c}	Weld capacity estimation
$\blacktriangle$	Fillet weld
F_{EXX}	Ultimate strength as rated by electrode classification number

Detailed result for BP1 / C248

Weld resistance check (AISC 360-16 – J2-4)

$$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} = 0.46 \text{ kip} \geq F_n = 0.14 \text{ kip}$$

Where:

$$F_{nw} = 42.0 \text{ ksi} \quad \text{— nominal stress of weld material:}$$

- $F_{nw} = 0.6 \cdot F_{EXX}$, where:
 - $F_{EXX} = 70.0 \text{ ksi}$ — electrode classification number, i.e. minimum specified tensile strength
 - Directional strength increase is not used for HSS welds

$$A_{we} = 0.01 \text{ in}^2 \quad \text{— effective area of weld critical element}$$

$$\phi = 0.75 \quad \text{— resistance factor for welded connections}$$

Concrete block

Item	Loads	A_1 [in ²]	A_2 [in ²]	σ [ksi]	$\phi f_{p,max}$ [ksi]	U_t [%]	Status
CB 1	10 LRFD_{(6.1)}-1.2D+L+1.6S(3)	92.24	253.76	0.1	3.7	2.0	OK

Symbol explanation

A_1	Loaded area
A_2	Supporting area
σ	Average stress in concrete
$\phi f_{p,max}$	Concrete bearing resistance
U_t	Utilization



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 216
Project Name: JKV-EMS		

Project:
Project no:
Author:



Detailed result for CB 1

Concrete block compressive resistance check (AISC 360-16 – J8)

$$\phi_c f_{p,max} = 3.7 \text{ ksi} \geq \sigma = 0.1 \text{ ksi}$$

Where:

$f_{p,max} = 5.6 \text{ ksi}$ – concrete block design bearing strength:

- $f_{p,max} = 0.85 \cdot f'_c \cdot \sqrt{\frac{A_2}{A_1}} \leq 1.7 \cdot f'_c$, where:
 - $f'_c = 4.0 \text{ ksi}$ – concrete compressive strength
 - $A_1 = 92.24 \text{ in}^2$ – base plate area in contact with concrete surface
 - $A_2 = 253.76 \text{ in}^2$ – concrete supporting surface

$\phi_c = 0.65$ – resistance factor for concrete



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 217
Project Name: JKV-EMS		

Project:
Project no:
Author:



Buckling

Loads	Shape	Factor [-]
69 LRFD_{10.8}-1.2D+L+0.5S+W(1)	1	30.15
	2	47.43
	3	85.62
	4	123.90
	5	172.35
	6	216.15
127 LRFD_{13.2}-0.9D+E(2)	1	544.96
	2	777.26
	3	1332.65
	4	1349.13
	5	1472.50
	6	1505.44
10 LRFD_{6.1}-1.2D+L+1.6S(3)	1	282.28
	2	468.41
	3	534.99
	4	582.41
	5	596.07
	6	624.85
108 LRFD_{12.7}-0.9D+W(4)	1	31.46
	2	49.13
	3	88.72
	4	128.48
	5	179.46
	6	226.39
131 LRFD_{13.6}-0.9D+E(5)	1	75.66
	2	119.94
	3	216.42
	4	312.75
	5	432.48
	6	532.43
124 LRFD_{11.7}-1.2D+L+0.2S+E(6)	1	877.03
	2	890.18
	3	915.04
	4	961.07
	5	998.70
	6	1056.02



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Date: 06/16/2025

Job No. 25-003

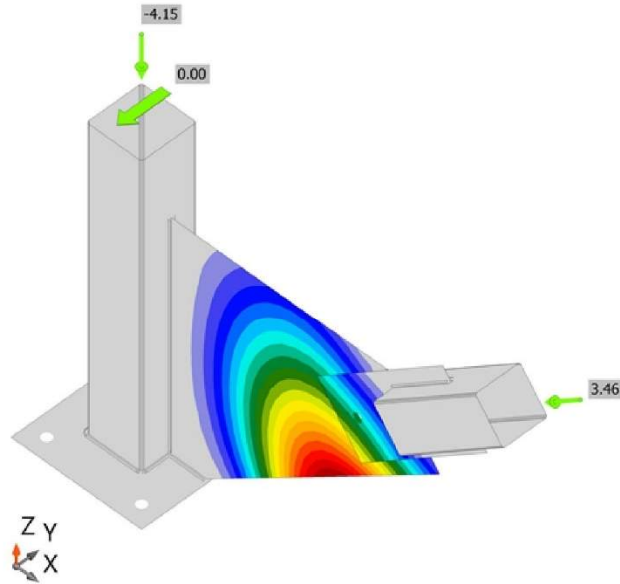
Revision:

Date:

Sheet No. 218

Project Name: JKV-EMS

Project:
Project no:
Author:



First buckling mode shape, 69 LRFD_{10.8}-1.2D+L+0.5S+W(1)

Code settings

Item	Value	Unit	Reference
Friction coefficient - concrete	0.40	-	ACI 349-01 – B.6.1.4
Friction coefficient in slip-resistance	0.30	-	AISC 360-16 – J3.8
Limit plastic strain	0.05	-	
Detailing	Yes		
Distance between bolts [d]	2.66	-	AISC 360-16 – J3.3
Concrete breakout resistance check	Both		
Base metal capacity check at weld fusion face	No		AISC 360-16 – J2-2
Deformation at bolt hole at service load is design consideration	Yes		AISC 360-16 – J3.10
Cracked concrete	Yes		ACI 318-14 – 17
Local deformation check	Yes		
Local deformation limit	0.03	-	CIDECT DG 1, 3 – 1.1
Geometrical nonlinearity (GMNA)	Yes		Analysis with large deformations for hollow section joints



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 219
Project Name: JKV-EMS		

Project:
Project no:
Author:



Project data

Project name
Project number
Author
Description
Date 6/16/2025
Code AISC/ACI

Material

Steel A500B-46, A992-50, A36, A572 Gr.50
Concrete 4000 psi



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 220
Project Name: JKV-EMS		

Project:
Project no:
Author:



Project item CBBC TOS HB/K.1/5.2

Design

Name CBBC TOS HB/K.1/5.2
Description
Analysis Stress, strain/ loads in equilibrium
Design code AISC 360-16 (LRFD) / ACI 318-14

Members

Geometry

Name	Cross-section	β - Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [in]	Offset ey [in]	Offset ez [in]
C245	6 - HSS 5x5x3/8	0.0	90.0	-90.0	0.00	0.00	0.00
TOS HBB55	24 - W 12x14	90.0	0.0	0.0	0.00	0.00	-5.95
TOS HBB54	24 - W 12x14	0.0	0.0	0.0	0.00	0.00	-5.95
TOS HBB60	34 - W 21x44	90.0	0.0	0.0	0.00	0.00	-10.35
TOS HBB119	40 - W 16x26	0.0	0.0	0.0	0.00	0.00	-7.85
SBR1	12 - HSS 4x4x0.25	-90.0	49.6	0.0	0.00	0.00	0.00
SBR4	12 - HSS 4x4x0.25	-180.0	-38.3	0.0	0.00	0.00	0.00

Supports and forces

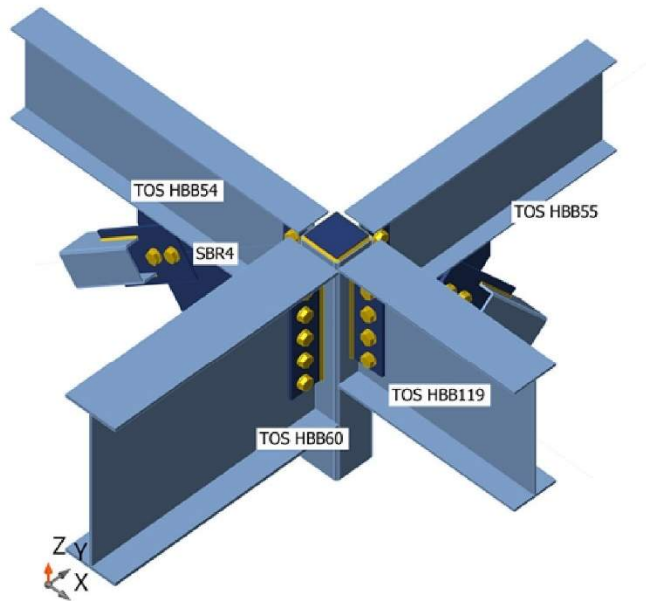
Name	Support	Forces in	X [in]
C245 / begin	N-Vy-Vz-Mx-My-Mz	Position	0.00
TOS HBB55 / end		Position	0.00
TOS HBB54 / begin		Position	0.00
TOS HBB60 / begin		Position	0.00
TOS HBB119 / end		Position	0.00
SBR1 / begin		Position	0.00
SBR4 / end		Position	0.00



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 221
Project Name: JKV-EMS		

Project:
Project no:
Author:



Cross-sections

Name	Material
6 - HSS 5x5x3/8	A500B-46
24 - W 12x14	A992-50
34 - W 21x44	A992-50
40 - W 16x26	A992-50
12 - HSS 4x4x0.25	A500B-46

Bolts

Name	Diameter [in]	f_y [ksi]	f_u [ksi]	Gross area [in ²]
3/4 A325	0.75	92.0	120.0	0.44



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 222
Project Name: JKV-EMS		

Project:
Project no:
Author:



Load effects (forces in equilibrium)

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
10 LRFD_{6.1}-1.2D+L+1.6S(1)	C245 / Begin	47.02	0.00	0.00	0.00	-0.02	0.00
	TOS HBB55 / End	-0.13	0.01	-7.56	0.00	0.00	0.00
	TOS HBB54 / Begin	-0.33	-0.03	-5.47	0.02	0.00	0.00
	TOS HBB60 / Begin	0.17	-0.14	-28.34	0.00	0.00	0.00
	TOS HBB119 / End	-0.12	0.00	-5.54	0.00	0.00	0.00
	SBR1 / Begin	0.00	0.00	0.00	0.00	0.00	0.00
108 LRFD_{12.7}-0.9D+W(2)	SBR4 / End	-0.41	0.00	0.00	0.00	0.00	0.00
	C245 / Begin	5.84	0.00	0.00	0.00	0.00	0.00
	TOS HBB55 / End	0.01	-0.02	-0.96	0.00	0.00	0.00
	TOS HBB54 / Begin	-2.81	-0.04	-0.74	0.00	0.00	0.00
	TOS HBB60 / Begin	0.03	0.05	-4.82	0.00	0.00	0.00
	TOS HBB119 / End	0.25	0.00	-1.09	0.00	0.00	0.00
106 LRFD_{12.5}-0.9D+W(4)	SBR1 / Begin	0.00	0.00	0.00	0.00	0.00	0.00
	SBR4 / End	-3.29	0.00	0.00	0.00	0.00	0.00
	C245 / Begin	7.80	0.00	0.00	0.00	-0.01	0.00
	TOS HBB55 / End	0.05	0.02	-1.55	0.00	0.00	0.00
	TOS HBB54 / Begin	-2.13	-0.03	-1.05	0.01	0.00	0.00
	TOS HBB60 / Begin	-0.02	0.06	-5.79	0.00	0.00	0.00
62 LRFD_{10.1}-1.2D+L+0.5S+W(5)	TOS HBB119 / End	-0.26	0.00	-1.07	0.00	0.00	0.00
	SBR1 / Begin	0.00	0.00	0.00	0.00	0.00	0.00
	SBR4 / End	-3.14	0.00	0.00	0.00	0.00	0.00
	C245 / Begin	23.35	0.00	0.00	0.00	-0.01	0.00
	TOS HBB55 / End	0.11	-0.02	-3.23	0.00	0.00	0.00
	TOS HBB54 / Begin	1.48	-0.01	-2.47	0.01	0.00	0.00
104 LRFD_{12.3}-0.9D+W(3)	TOS HBB60 / Begin	-0.10	-0.19	-13.23	0.00	0.00	0.00
	TOS HBB119 / End	0.17	0.00	-2.60	0.00	0.00	0.00
	SBR1 / Begin	0.00	0.00	0.00	0.00	0.00	0.00
	SBR4 / End	2.37	0.00	0.00	0.00	0.00	0.00
	C245 / Begin	7.47	0.00	0.00	0.00	0.00	0.00
	TOS HBB55 / End	0.17	-0.02	-2.13	0.00	0.00	0.00
9 LRFD_{5}-1.2D+L+1.6Lr(6)	TOS HBB54 / Begin	-2.75	-0.03	-1.30	0.00	0.00	0.00
	TOS HBB60 / Begin	-0.14	0.08	-4.51	0.00	0.00	0.00
	TOS HBB119 / End	0.26	0.00	-1.26	0.00	0.00	0.00
	SBR1 / Begin	0.00	0.00	0.00	0.00	0.00	0.00
	SBR4 / End	-3.25	0.00	0.00	0.00	0.00	0.00
	C245 / Begin	45.71	0.00	0.00	0.00	-0.02	0.00
124 LRFD_{11.7}-1.2D+L+0.2S+E(7)	TOS HBB55 / End	-0.12	0.01	-7.87	0.00	0.00	0.00
	TOS HBB54 / Begin	-0.31	-0.03	-5.25	0.02	0.00	0.00
	TOS HBB60 / Begin	0.16	-0.14	-27.14	0.00	0.00	0.00
	TOS HBB119 / End	-0.11	0.00	-5.32	0.00	0.00	0.00
	SBR1 / Begin	0.00	0.00	0.00	0.00	0.00	0.00
	SBR4 / End	-0.38	0.00	0.00	0.00	0.00	0.00
124 LRFD_{11.7}-1.2D+L+0.2S+E(7)	C245 / Begin	29.40	0.00	0.00	0.00	-0.01	0.00
	TOS HBB55 / End	-0.31	-0.01	-4.72	0.00	0.00	0.00
	TOS HBB54 / Begin	0.65	0.09	-3.46	0.01	0.00	0.00
	TOS HBB60 / Begin	1.18	0.19	-16.81	0.00	0.00	0.00
124 LRFD_{11.7}-1.2D+L+0.2S+E(7)	TOS HBB119 / End	0.03	0.00	-3.41	0.00	0.00	0.00



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 223
Project Name: JKV-EMS		

Project:
Project no:
Author:



Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
	SBR1 / Begin	0.00	0.00	0.00	0.00	0.00	0.00
	SBR4 / End	1.01	0.00	0.00	0.00	0.00	0.00
131 LRFD_{13.6}-0.9D+E(8)	C245 / Begin	18.22	0.00	0.00	0.00	-0.01	0.00
	TOS HBB55 / End	0.20	0.02	-3.12	0.00	0.00	0.00
	TOS HBB54 / Begin	-0.92	-0.11	-2.30	0.01	0.00	0.00
	TOS HBB60 / Begin	-1.04	-0.30	-11.14	0.00	0.00	0.00
	TOS HBB119 / End	-0.16	0.00	-2.25	0.00	0.00	0.00
	SBR1 / Begin	0.00	0.00	0.00	0.00	0.00	0.00
	SBR4 / End	-1.39	0.00	0.00	0.00	0.00	0.00
120 LRFD_{11.3}-1.2D+L+0.2S+E(9)	C245 / Begin	26.84	0.00	0.00	0.00	-0.01	0.00
	TOS HBB55 / End	0.04	0.04	-4.72	0.00	0.00	0.00
	TOS HBB54 / Begin	-1.31	-0.12	-3.46	0.01	0.00	0.00
	TOS HBB60 / Begin	-0.20	-0.20	-16.92	0.00	0.00	0.00
	TOS HBB119 / End	-0.48	0.00	-3.41	0.00	0.00	0.00
	SBR1 / Begin	0.00	0.00	0.00	0.00	0.00	0.00
	SBR4 / End	-3.31	0.00	0.00	0.00	0.00	0.00
127 LRFD_{13.2}-0.9D+E(10)	C245 / Begin	20.79	0.00	0.00	0.00	-0.01	0.00
	TOS HBB55 / End	-0.15	-0.03	-3.13	0.00	0.00	0.00
	TOS HBB54 / Begin	1.04	0.10	-2.29	0.01	0.00	0.00
	TOS HBB60 / Begin	0.34	0.09	-11.04	0.00	0.00	0.00
	TOS HBB119 / End	0.35	0.00	-2.25	0.00	0.00	0.00
	SBR1 / Begin	0.00	0.00	0.00	0.00	0.00	0.00
	SBR4 / End	2.93	0.00	0.00	0.00	0.00	0.00
123 LRFD_{11.6}-1.2D+L+0.2S+E(11)	C245 / Begin	28.04	0.00	0.00	0.00	-0.01	0.00
	TOS HBB55 / End	0.17	0.02	-4.72	0.00	0.00	0.00
	TOS HBB54 / Begin	-0.98	-0.12	-3.46	0.01	0.00	0.00
	TOS HBB60 / Begin	-1.01	-0.33	-16.99	0.00	0.00	0.00
	TOS HBB119 / End	-0.19	0.00	-3.41	0.00	0.00	0.00
	SBR1 / Begin	0.00	0.00	0.00	0.00	0.00	0.00
	SBR4 / End	-1.47	0.00	0.00	0.00	0.00	0.00
105 LRFD_{12.4}-0.9D+W(12)	C245 / Begin	12.18	0.00	0.00	0.00	0.00	0.00
	TOS HBB55 / End	0.15	-0.02	-2.39	0.00	0.00	0.00
	TOS HBB54 / Begin	-2.82	-0.04	-1.78	0.00	0.00	0.00
	TOS HBB60 / Begin	-0.11	0.04	-8.09	0.00	0.00	0.00
	TOS HBB119 / End	0.25	0.00	-1.70	0.00	0.00	0.00
	SBR1 / Begin	0.00	0.00	0.00	0.00	0.00	0.00
	SBR4 / End	-3.31	0.00	0.00	0.00	0.00	0.00
63 LRFD_{10.2}-1.2D+L+0.5S+W(13)	C245 / Begin	28.79	0.00	0.00	0.00	-0.01	0.00
	TOS HBB55 / End	0.10	-0.01	-4.49	0.00	0.00	0.00
	TOS HBB54 / Begin	1.44	-0.01	-3.29	0.01	0.00	0.00
	TOS HBB60 / Begin	-0.08	-0.20	-15.98	0.00	0.00	0.00
	TOS HBB119 / End	0.15	0.00	-3.24	0.00	0.00	0.00
	SBR1 / Begin	0.00	0.00	0.00	0.00	0.00	0.00
	SBR4 / End	2.31	0.00	0.00	0.00	0.00	0.00
133 LRFD_{13.8}-0.9D+E(16)	C245 / Begin	18.47	0.00	0.00	0.00	-0.01	0.00
	TOS HBB55 / End	-0.26	0.01	-3.12	0.00	0.00	0.00
	TOS HBB54 / Begin	0.11	0.04	-2.29	0.01	0.00	0.00
	TOS HBB60 / Begin	1.18	0.18	-10.97	0.00	0.00	0.00
	TOS HBB119 / End	-0.19	0.00	-2.25	0.00	0.00	0.00



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 224
Project Name: JKV-EMS		

Project:
Project no:
Author:



Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
	SBR1 / Begin	0.00	0.00	0.00	0.00	0.00	0.00
	SBR4 / End	-0.70	0.00	0.00	0.00	0.00	0.00
122 LRFD_{11.5}-1.2D+L+0.2S+E(17)	C245 / Begin	29.15	0.00	0.00	0.00	-0.01	0.00
	TOS HBB55 / End	0.15	0.00	-4.72	0.00	0.00	0.00
	TOS HBB54 / Begin	-0.39	-0.07	-3.46	0.01	0.00	0.00
	TOS HBB60 / Begin	-1.04	-0.30	-16.99	0.00	0.00	0.00
	TOS HBB119 / End	0.05	0.00	-3.41	0.00	0.00	0.00
	SBR1 / Begin	0.00	0.00	0.00	0.00	0.00	0.00
	SBR4 / End	0.32	0.00	0.00	0.00	0.00	0.00
102 LRFD_{12.1}-0.9D+W(14)	C245 / Begin	10.27	0.00	0.00	0.00	0.00	0.00
	TOS HBB55 / End	0.15	-0.02	-1.13	0.00	0.00	0.00
	TOS HBB54 / Begin	1.58	0.00	-0.96	0.00	0.00	0.00
	TOS HBB60 / Begin	-0.15	-0.15	-5.32	0.00	0.00	0.00
	TOS HBB119 / End	0.20	0.00	-1.06	0.00	0.00	0.00
	SBR1 / Begin	0.00	0.00	0.00	0.00	0.00	0.00
	SBR4 / End	2.48	0.00	0.00	0.00	0.00	0.00
69 LRFD_{10.8}-1.2D+L+0.5S+W(15)	C245 / Begin	25.22	0.00	0.00	0.00	-0.01	0.00
	TOS HBB55 / End	-0.04	-0.02	-4.49	0.00	0.00	0.00
	TOS HBB54 / Begin	-2.93	-0.05	-3.29	0.01	0.00	0.00
	TOS HBB60 / Begin	0.09	0.00	-15.98	0.00	0.00	0.00
	TOS HBB119 / End	0.20	0.00	-3.24	0.00	0.00	0.00
	SBR1 / Begin	0.00	0.00	0.00	0.00	0.00	0.00
	SBR4 / End	-3.46	0.00	0.00	0.00	0.00	0.00
125 LRFD_{11.8}-1.2D+L+0.2S+E(18)	C245 / Begin	28.29	0.00	0.00	0.00	-0.01	0.00
	TOS HBB55 / End	-0.29	0.01	-4.72	0.00	0.00	0.00
	TOS HBB54 / Begin	0.05	0.04	-3.46	0.01	0.00	0.00
	TOS HBB60 / Begin	1.21	0.15	-16.81	0.00	0.00	0.00
	TOS HBB119 / End	-0.21	0.00	-3.41	0.00	0.00	0.00
	SBR1 / Begin	0.00	0.00	0.00	0.00	0.00	0.00
	SBR4 / End	-0.78	0.00	0.00	0.00	0.00	0.00
130 LRFD_{13.5}-0.9D+E(19)	C245 / Begin	19.33	0.00	0.00	0.00	-0.01	0.00
	TOS HBB55 / End	0.17	0.00	-3.12	0.00	0.00	0.00
	TOS HBB54 / Begin	-0.32	-0.06	-2.29	0.01	0.00	0.00
	TOS HBB60 / Begin	-1.07	-0.27	-11.15	0.00	0.00	0.00
	TOS HBB119 / End	0.08	0.00	-2.25	0.00	0.00	0.00
	SBR1 / Begin	0.00	0.00	0.00	0.00	0.00	0.00
	SBR4 / End	0.40	0.00	0.00	0.00	0.00	0.00
118 LRFD_{11.1}-1.2D+L+0.2S+E(20)	C245 / Begin	30.53	0.00	0.00	0.00	-0.01	0.00
	TOS HBB55 / End	-0.04	-0.03	-4.72	0.00	0.00	0.00
	TOS HBB54 / Begin	0.67	0.05	-3.46	0.01	0.00	0.00
	TOS HBB60 / Begin	-0.29	-0.09	-16.93	0.00	0.00	0.00
	TOS HBB119 / End	0.33	0.00	-3.41	0.00	0.00	0.00
	SBR1 / Begin	0.00	0.00	0.00	0.00	0.00	0.00
	SBR4 / End	2.64	0.00	0.00	0.00	0.00	0.00
129 LRFD_{13.4}-0.9D+E(21)	C245 / Begin	17.09	0.00	0.00	0.00	-0.01	0.00
	TOS HBB55 / End	-0.07	0.04	-3.12	0.00	0.00	0.00
	TOS HBB54 / Begin	-0.94	-0.07	-2.29	0.01	0.00	0.00
	TOS HBB60 / Begin	0.43	-0.03	-11.03	0.00	0.00	0.00
	TOS HBB119 / End	-0.46	0.00	-2.25	0.00	0.00	0.00



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Project Name: JKV-EMS		

Project:
Project no:
Author:



Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
	SBR1 / Begin	0.00	0.00	0.00	0.00	0.00	0.00
	SBR4 / End	-3.02	0.00	0.00	0.00	0.00	0.00
132 LRFD_{13.7}-0.9D+E(22)	C245 / Begin	19.58	0.00	0.00	0.00	-0.01	0.00
	TOS HBB55 / End	-0.29	-0.01	-3.13	0.00	0.00	0.00
	TOS HBB54 / Begin	0.71	0.09	-2.29	0.01	0.00	0.00
	TOS HBB60 / Begin	1.15	0.21	-10.97	0.00	0.00	0.00
	TOS HBB119 / End	0.06	0.00	-2.25	0.00	0.00	0.00
	SBR1 / Begin	0.00	0.00	0.00	0.00	0.00	0.00
	SBR4 / End	1.09	0.00	0.00	0.00	0.00	0.00
121 LRFD_{11.4}-1.2D+L+0.2S+E(23)	C245 / Begin	26.91	0.00	0.00	0.00	-0.01	0.00
	TOS HBB55 / End	-0.10	0.04	-4.72	0.00	0.00	0.00
	TOS HBB54 / Begin	-1.00	-0.08	-3.46	0.01	0.00	0.00
	TOS HBB60 / Begin	0.47	-0.06	-16.87	0.00	0.00	0.00
	TOS HBB119 / End	-0.49	0.00	-3.41	0.00	0.00	0.00
	SBR1 / Begin	0.00	0.00	0.00	0.00	0.00	0.00
	SBR4 / End	-3.10	0.00	0.00	0.00	0.00	0.00
126 LRFD_{13.1}-0.9D+E(24)	C245 / Begin	20.71	0.00	0.00	0.00	-0.01	0.00
	TOS HBB55 / End	-0.02	-0.03	-3.13	0.00	0.00	0.00
	TOS HBB54 / Begin	0.73	0.05	-2.29	0.01	0.00	0.00
	TOS HBB60 / Begin	-0.32	-0.06	-11.09	0.00	0.00	0.00
	TOS HBB119 / End	0.36	0.00	-2.25	0.00	0.00	0.00
	SBR1 / Begin	0.00	0.00	0.00	0.00	0.00	0.00
	SBR4 / End	2.72	0.00	0.00	0.00	0.00	0.00
19 LRFD_{7.6}-1.2D+1.6Lr+0.5W(25)	C245 / Begin	42.15	0.00	0.00	0.00	-0.02	0.00
	TOS HBB55 / End	-0.09	0.02	-7.47	0.00	0.00	0.00
	TOS HBB54 / Begin	-1.33	-0.04	-4.97	0.02	0.00	0.00
	TOS HBB60 / Begin	0.13	-0.09	-25.51	0.00	0.00	0.00
	TOS HBB119 / End	-0.22	0.00	-5.01	0.00	0.00	0.00
	SBR1 / Begin	0.00	0.00	0.00	0.00	0.00	0.00
	SBR4 / End	-1.89	0.00	0.00	0.00	0.00	0.00



209 SE DOUGLAS
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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 226
Project Name: JKV-EMS		

Project:
Project no:
Author:



Unbalanced forces

Name	X [kip]	Y [kip]	Z [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
10 LRFD_{6.1}-1.2D+L+1.6S(1)	0.00	0.00	0.36	0.05	0.13	0.01
108 LRFD_{12.7}-0.9D+W(2)	0.00	0.00	0.27	-0.05	1.26	0.08
106 LRFD_{12.5}-0.9D+W(4)	0.00	0.00	0.27	-0.07	1.29	0.08
62 LRFD_{10.1}-1.2D+L+0.5S+W(5)	0.00	0.00	0.36	0.01	-1.02	-0.06
104 LRFD_{12.3}-0.9D+W(3)	0.00	0.00	0.27	-0.11	1.25	0.08
9 LRFD_{5}-1.2D+L+1.6Lr(6)	0.00	0.00	0.36	0.05	0.11	0.01
124 LRFD_{11.7}-1.2D+L+0.2S+E(7)	-0.29	0.96	0.37	0.93	-0.19	-0.02
131 LRFD_{13.6}-0.9D+E(8)	0.29	-0.96	0.27	-0.89	0.31	0.03
120 LRFD_{11.3}-1.2D+L+0.2S+E(9)	0.96	-0.29	0.37	-0.28	0.82	0.08
127 LRFD_{13.2}-0.9D+E(10)	-0.96	0.29	0.27	0.33	-0.69	-0.07
123 LRFD_{11.6}-1.2D+L+0.2S+E(11)	0.29	-0.96	0.37	-0.88	0.34	0.04
105 LRFD_{12.4}-0.9D+W(12)	0.00	0.00	0.27	-0.11	1.26	0.08
63 LRFD_{10.2}-1.2D+L+0.5S+W(13)	0.00	0.00	0.36	0.01	-1.00	-0.06
133 LRFD_{13.8}-0.9D+E(16)	0.29	0.96	0.27	0.89	0.22	0.02
122 LRFD_{11.5}-1.2D+L+0.2S+E(17)	-0.29	-0.96	0.37	-0.85	-0.10	-0.01
102 LRFD_{12.1}-0.9D+W(14)	0.00	0.00	0.27	-0.01	-1.05	-0.06
69 LRFD_{10.8}-1.2D+L+0.5S+W(15)	0.00	0.00	0.36	-0.03	1.31	0.08
125 LRFD_{11.8}-1.2D+L+0.2S+E(18)	0.29	0.96	0.37	0.90	0.25	0.02
130 LRFD_{13.5}-0.9D+E(19)	-0.29	-0.96	0.27	-0.86	-0.12	-0.01
118 LRFD_{11.1}-1.2D+L+0.2S+E(20)	-0.96	-0.29	0.37	-0.20	-0.64	-0.06
129 LRFD_{13.4}-0.9D+E(21)	0.96	0.29	0.27	0.24	0.76	0.07
132 LRFD_{13.7}-0.9D+E(22)	-0.29	0.96	0.27	0.92	-0.21	-0.03
121 LRFD_{11.4}-1.2D+L+0.2S+E(23)	0.96	0.29	0.37	0.25	0.79	0.08
126 LRFD_{13.1}-0.9D+E(24)	-0.96	-0.29	0.27	-0.21	-0.66	-0.07
19 LRFD_{7.6}-1.2D+1.6Lr+0.5W(25)	0.00	0.00	0.36	0.01	0.74	0.05

Check

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	2.9 < 5.0%	OK
Loc. deformation	0.2 < 3%	OK
Bolts	62.2 < 100%	OK
Welds	81.3 < 100%	OK
Buckling	10.33	
GMNA	Calculated	



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 227
Project Name: JKV-EMS		

Project:
Project no:
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Plates

Name	Material	t_p [in]	Loads	σ_{Ed} [ksi]	ϵ_{pl} [%]	$\sigma_{c,Ed}$ [ksi]	Status
C245	A500B-46	3/8	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	41.4	0.1	0.0	OK
TOS HBB55-bfl 1	A992-50	1/4	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	37.7	0.0	0.0	OK
TOS HBB55-tfl 1	A992-50	1/4	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	45.1	0.2	0.0	OK
TOS HBB55-w 1	A992-50	3/16	9 LRFD_{(5)-1.2D+L+1.6Lr(6)}	45.1	0.4	14.0	OK
TOS HBB54-bfl 1	A992-50	1/4	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	24.6	0.0	0.0	OK
TOS HBB54-tfl 1	A992-50	1/4	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	41.4	0.0	0.0	OK
TOS HBB54-w 1	A992-50	3/16	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	45.0	0.0	8.6	OK
TOS HBB60-bfl 1	A992-50	7/16	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	43.3	0.1	0.0	OK
TOS HBB60-tfl 1	A992-50	7/16	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	45.2	0.6	0.0	OK
TOS HBB60-w 1	A992-50	3/8	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	45.2	0.7	19.2	OK
TOS HBB119-bfl 1	A992-50	3/8	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	3.9	0.0	0.0	OK
TOS HBB119-tfl 1	A992-50	3/8	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	3.9	0.0	0.0	OK
TOS HBB119-w 1	A992-50	1/4	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	17.2	0.0	1.1	OK
SBR1	A500B-46	1/4	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	10.6	0.0	0.0	OK
SBR4	A500B-46	1/4	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	5.0	0.0	0.0	OK
FP1	A36	3/8	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	32.4	0.1	13.1	OK
STIFF1	A36	3/8	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	10.9	0.0	0.0	OK
SP1	A36	3/8	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	30.8	0.0	6.0	OK
SP2	A36	3/8	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	32.4	0.1	6.7	OK
CPL1	A36	3/8	69 LRFD_{(10.8)-1.2D+L+0.5S+W(15)}	9.5	0.0	1.5	OK
SHP2	A36	3/8	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	32.6	0.6	19.7	OK
SP3	A36	3/8	9 LRFD_{(5)-1.2D+L+1.6Lr(6)}	32.4	0.2	5.7	OK
SP4	A36	3/8	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	32.5	0.4	10.3	OK
CPL2	A500B-46	3/8	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	12.0	0.0	0.6	OK
SHP3	A36	3/8	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	12.1	0.0	1.1	OK
SHP4	A36	3/8	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	33.2	2.9	19.2	OK

Design data

Material	F_y [ksi]	ϵ_{lim} [%]
A500B-46	46.0	5.0
A992-50	50.0	5.0
A36	36.0	5.0

Symbol explanation

t_p	Plate thickness
σ_{Ed}	Equivalent stress
ϵ_{pl}	Plastic strain
$\sigma_{c,Ed}$	Contact stress
F_y	Yield strength
ϵ_{lim}	Limit of plastic strain



Project:
Project no:
Author:



Detailed result for SHP4

Design values used in the analysis

$$\phi F_y = 32.4 \text{ ksi}$$

Where:

$$F_y = 36.0 \text{ ksi} \quad \text{-- characteristic yield strength}$$

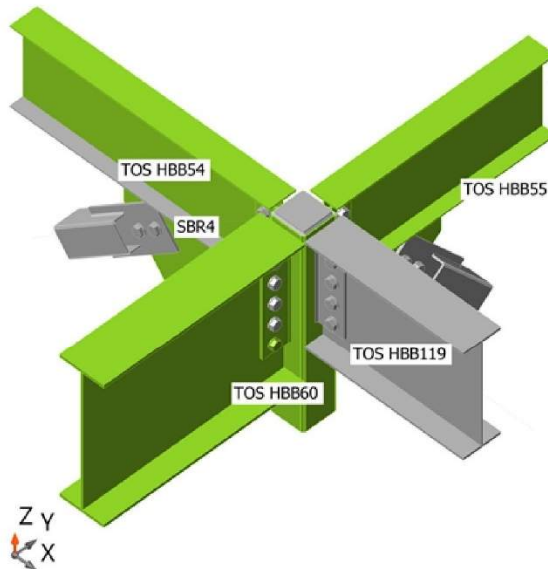
$$\phi = 0.90 \quad \text{-- resistance factor for steel material AISC 360-16 -- B3.1}$$

Loc. deformation

Name	d_0 [in]	Loads	δ [in]	δ_{lim} [in]	δ/d_0 [%]	Check status
C245	5.00	9 LRFD_{(5)}-1.2D+L+1.6Lr(6)	0.01	0.15	0.2	OK
SBR1	4.00	19 LRFD_{(7.6)}-1.2D+1.6Lr+0.5W(25)	0.00	0.12	0.0	OK
SBR4	4.00	19 LRFD_{(7.6)}-1.2D+1.6Lr+0.5W(25)	0.00	0.12	0.0	OK

Symbol explanation

d_0	Cross-section size
δ	Local cross-section deformation
δ_{lim}	Allowed deformation



Overall check, 10 LRFD_{(6.1)}-1.2D+L+1.6S(1)

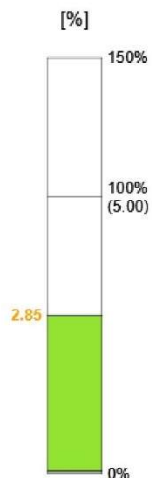
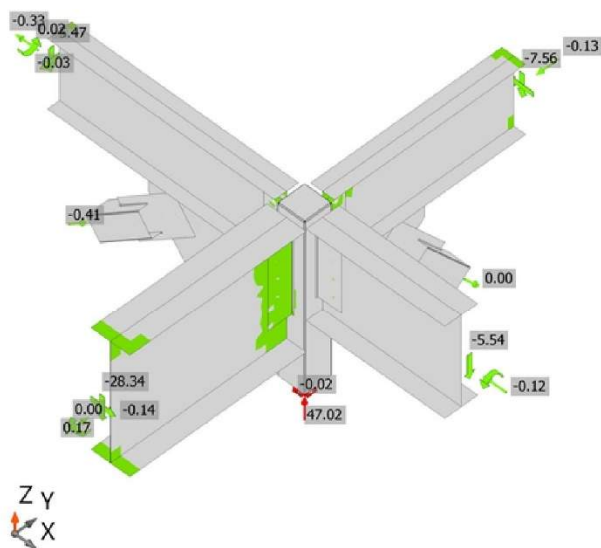


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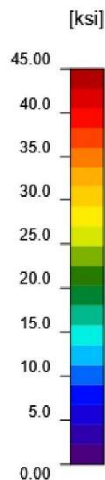
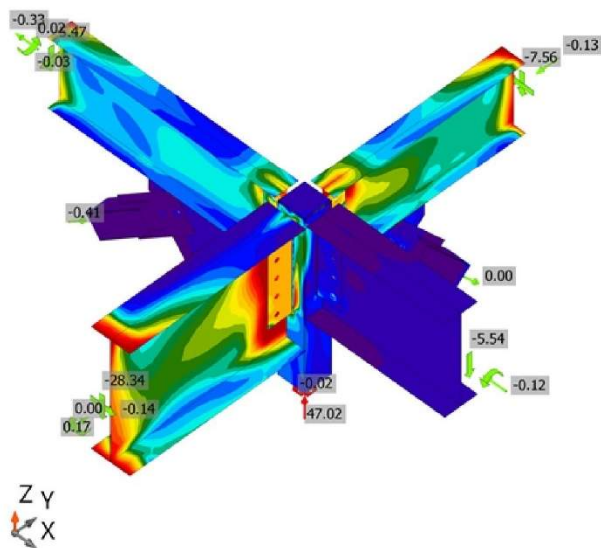
Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 229
Project Name: JKV-EMS		

Project:
Project no:
Author:

StatiCa®
Efficient structural analysis



Strain check, 10 LRFD_{(6.1)-1.2D+L+1.6S(1)}



Equivalent stress, 10 LRFD_{(6.1)-1.2D+L+1.6S(1)}



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 230
Project Name: JKV-EMS		

Project:
Project no:
Author:



Bolts

Shape	Item	Grade	Loads	F_t [kip]	V [kip]	$\phi R_{n,bearing}$ [kip]	U_t [%]	U_s [%]	U_{ts} [%]	Detailing	Status
	B1	3/4 A325 - 1	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	6.65	1.29	17.55	22.3	7.3	0.0	OK	OK
	B2	3/4 A325 - 1	120 LRFD_{(11.3)-1.2D+L+0.2S+E(9)}	0.15	0.91	17.55	0.5	5.2	0.0	OK	OK
	B3	3/4 A325 - 1	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	3.25	0.48	17.55	10.9	2.7	0.0	OK	OK
	B4	3/4 A325 - 1	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	4.92	1.03	29.37	16.5	5.7	0.0	OK	OK
	B5	3/4 A325 - 1	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	1.33	1.42	29.37	4.5	7.9	0.0	OK	OK
	B6	3/4 A325 - 1	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	1.71	1.73	27.96	5.8	9.7	0.0	OK	OK
	B7	3/4 A325 - 1	69 LRFD_{(10.8)-1.2D+L+0.5S+W(15)}	0.46	1.86	29.37	1.5	10.4	0.0	OK	OK
	B8	3/4 A325 - 1	127 LRFD_{(13.2)-0.9D+E(10)}	0.24	1.43	12.58	0.8	11.4	0.0	OK	OK
	B9	3/4 A325 - 1	9 LRFD_{(5)-1.2D+L+1.6Lr(6)}	11.23	1.74	17.55	37.7	9.9	0.0	OK	OK
	B10	3/4 A325 - 1	9 LRFD_{(5)-1.2D+L+1.6Lr(6)}	2.70	1.45	17.55	9.1	8.3	0.0	OK	OK
	B11	3/4 A325 - 1	9 LRFD_{(5)-1.2D+L+1.6Lr(6)}	6.34	1.19	17.55	21.3	6.8	0.0	OK	OK
	B12	3/4 A325 - 1	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	7.43	1.20	29.37	24.9	6.7	0.0	OK	OK
	B13	3/4 A325 - 1	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	1.99	1.75	29.37	6.7	9.8	0.0	OK	OK
	B14	3/4 A325 - 1	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	2.59	2.31	29.37	8.7	12.9	0.0	OK	OK
	B15	3/4 A325 - 1	9 LRFD_{(5)-1.2D+L+1.6Lr(6)}	0.64	0.06	29.36	2.1	0.3	0.0	OK	OK
	B16	3/4 A325 - 1	9 LRFD_{(5)-1.2D+L+1.6Lr(6)}	0.70	0.06	12.36	2.4	0.5	0.0	OK	OK
	B17	3/4 A325 - 1	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	0.64	3.11	21.94	2.2	17.4	0.0	OK	OK
	B18	3/4 A325 - 1	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	0.21	1.68	21.94	0.7	9.4	0.0	OK	OK
	B19	3/4 A325 - 1	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	0.27	1.61	21.94	0.9	9.0	0.0	OK	OK
	B20	3/4 A325 - 1	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	0.53	3.09	21.94	1.8	17.3	0.0	OK	OK



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 231
Project Name: JKV-EMS		

Project:
Project no:
Author:



Shape	Item	Grade	Loads	F_t [kip]	V [kip]	$\phi R_{n,bearing}$ [kip]	U_t [%]	U_s [%]	U_{ts} [%]	Detailing	Status
	B21	3/4 A325 - 1	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	13.33	9.53	30.71	44.7	53.3	58.3	OK	OK
	B22	3/4 A325 - 1	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	4.05	7.56	30.71	13.6	42.3	0.0	OK	OK
	B23	3/4 A325 - 1	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	4.52	6.38	30.71	15.2	35.7	0.0	OK	OK
	B24	3/4 A325 - 1	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	4.96	7.63	30.71	16.6	42.7	0.0	OK	OK
	B25	3/4 A325 - 1	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	7.18	11.12	26.68	24.1	62.2	0.0	OK	OK

Design data

Grade	$\phi R_{n,tension}$ [kip]	$\phi R_{n,shear}$ [kip]
3/4 A325 - 1	29.79	17.88

Symbol explanation

F_t	Tension force
V	Resultant of bolt shear forces V_y and V_z in shear planes
$\phi R_{n,bearing}$	Bolt bearing resistance
U_t	Utilization in tension
U_s	Utilization in shear
U_{ts}	Utilization in tension and shear
$\phi R_{n,tension}$	Bolt tension resistance - AISC 360-16 – J3.6
$\phi R_{n,shear}$	Bolt shear resistance - AISC 360-16 – J3.6

Detailed result for B25

Tension resistance check (AISC 360-16 – J3-1)

$$\phi R_n = \phi \cdot F_{nt} \cdot A_b = 29.79 \text{ kip} \geq F_t = 7.18 \text{ kip}$$

Where:

$$F_{nt} = 89.9 \text{ ksi} \quad \text{— nominal tensile stress AISC 360-16 – Table J3.2}$$

$$A_b = 0.44 \text{ in}^2 \quad \text{— gross bolt cross-sectional area}$$

$$\phi = 0.75 \quad \text{— resistance factor}$$

Shear resistance check (AISC 360-16 – J3-1)

$$\phi R_n = \phi \cdot F_{nv} \cdot A_b = 17.88 \text{ kip} \geq V = 11.12 \text{ kip}$$

Where:

$$F_{nv} = 54.0 \text{ ksi} \quad \text{— nominal shear stress AISC 360-16 – Table J3.2}$$

$$A_b = 0.44 \text{ in}^2 \quad \text{— gross bolt cross-sectional area}$$

$$\phi = 0.75 \quad \text{— resistance factor}$$



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 232
Project Name: JKV-EMS		

Project:
Project no:
Author:



Bearing resistance check (AISC 360-16 – J3-6)

$$R_n = 1.20 \cdot l_c \cdot t \cdot F_u \leq 2.40 \cdot d \cdot t \cdot F_u$$

$$\phi R_n = 26.68 \text{ kip} \geq V = 11.12 \text{ kip}$$

Where:

$l_c = 1.30 \text{ in}$ – clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material

$t = 3/8 \text{ in}$ – thickness of the plate

$d = 3/4 \text{ in}$ – diameter of a bolt

$F_u = 58.0 \text{ ksi}$ – specified minimum tensile strength of the connected material

$\phi = 0.75$ – resistance factor for bearing at bolt holes

Interaction of tension and shear check (AISC 360-16 – J3-2)

The required stress, in either shear or tension, is less than or equal to 30% of the corresponding available stress and the effects of combined stresses need not be investigated.



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 233
Project Name: JKV-EMS		

Project:
Project no:
Author:



Welds

Item	Edge	Xu	t _w [in]	w [in]	L [in]	L _c [in]	Loads	F _n [kip]	φR _n [kip]	U _t [%]	U _t [%]	Detailing	Status
C245-w 4	FP1	E70xx	3/16	1/4	8.89	0.31	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	0.63	2.47	25.6	13.6	OK	OK
		E70xx	3/16	1/4	8.94	0.31	9 LRFD_{(5)-1.2D+L+1.6Lr(6)}	0.87	2.38	36.5	16.9	OK	OK
TOS HBB54- bfl 1	SP1	E70xx	1/8	3/16	22.95	1.00	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	4.01	5.33	75.2	52.3	OK	OK
C245-w 4	SP2	E70xx	3/16	1/4	8.94	0.31	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	1.09	2.09	52.0	27.1	OK	OK
		E70xx	3/16	1/4	8.94	0.31	118 LRFD_{(11.1)-1.2D+L+0.2S+E(20)}	0.50	1.92	26.0	10.2	OK	OK
CPL1	SBR4- w 1	E70xx	1/8	3/16	5.96	0.31	69 LRFD_{(10.8)-1.2D+L+0.5S+W(15)}	0.29	1.31	22.2	9.5	OK	OK
CPL1	SBR4- w 1	E70xx	1/8	3/16	5.96	0.31	69 LRFD_{(10.8)-1.2D+L+0.5S+W(15)}	0.16	1.31	12.3	0.0	OK	OK
CPL1	SBR4- w 3	E70xx	1/8	3/16	5.96	0.31	69 LRFD_{(10.8)-1.2D+L+0.5S+W(15)}	0.16	1.31	12.3	0.0	OK	OK
CPL1	SBR4- w 3	E70xx	1/8	3/16	5.96	0.31	69 LRFD_{(10.8)-1.2D+L+0.5S+W(15)}	0.29	1.31	22.3	8.4	OK	OK
C245-w 3	SHP2	E70xx	3/16	1/4	8.94	0.31	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	1.04	2.25	46.4	25.5	OK	OK
		E70xx	3/16	1/4	8.94	0.31	9 LRFD_{(5)-1.2D+L+1.6Lr(6)}	1.27	2.05	61.9	35.4	OK	OK
TOS HBB55- bfl 1	SP3	E70xx	1/8	3/16	16.96	0.85	9 LRFD_{(5)-1.2D+L+1.6Lr(6)}	3.49	4.62	75.6	51.5	OK	OK
C245-w 3	SP4	E70xx	1/8	3/16	8.94	0.31	9 LRFD_{(5)-1.2D+L+1.6Lr(6)}	1.14	1.66	68.7	37.3	OK	OK
		E70xx	1/8	3/16	8.94	0.31	9 LRFD_{(5)-1.2D+L+1.6Lr(6)}	0.77	1.42	54.4	34.6	OK	OK
CPL2	SBR1- w 1	E70xx	1/8	3/16	5.96	0.31	9 LRFD_{(5)-1.2D+L+1.6Lr(6)}	0.08	1.31	5.9	0.0	OK	OK
CPL2	SBR1- w 1	E70xx	1/8	3/16	5.96	0.31	9 LRFD_{(5)-1.2D+L+1.6Lr(6)}	0.08	1.31	6.3	0.0	OK	OK
CPL2	SBR1- w 3	E70xx	1/8	3/16	5.96	0.31	9 LRFD_{(5)-1.2D+L+1.6Lr(6)}	0.06	1.31	4.3	0.0	OK	OK
CPL2	SBR1- w 3	E70xx	1/8	3/16	5.96	0.31	9 LRFD_{(5)-1.2D+L+1.6Lr(6)}	0.06	1.31	4.8	0.0	OK	OK
C245-w 2	SHP3	E70xx	3/16	1/4	11.93	0.31	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	0.30	2.38	12.7	0.0	OK	OK
		E70xx	3/16	1/4	11.93	0.31	9 LRFD_{(5)-1.2D+L+1.6Lr(6)}	0.52	2.60	20.0	8.1	OK	OK
C245-w 1	SHP4	E70xx	3/16	1/4	14.91	0.31	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	1.99	2.53	78.8	62.4	OK	OK
		E70xx	3/16	1/4	14.91	0.31	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	2.08	2.56	81.3	73.9	OK	OK



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 234
Project Name: JKV-EMS		

Project:
Project no:
Author:



Item	Edge	Xu	t _w [in]	w [in]	L [in]	L _c [in]	Loads	F _n [kip]	φR _n [kip]	Ut [%]	Ut _c [%]	Detailing	Status
STIFF1	C245	E70xx	3/16	1/4	16.95	0.02	10 LRFD_{(6.1)-1.2D+L+1.6S(1)}	0.04	0.09	43.3	23.9	OK	OK

Design data

Material	F _{EXX} [ksi]
E70xx	70.0

Symbol explanation

t _w	Throat thickness of weld
w	Leg size of weld
L	Length of weld
L _c	Length of weld critical element
F _n	Force in weld critical element
φR _n	Weld resistance - AISC 360-16 – J2-4
Ut	Utilization
Ut _c	Weld capacity estimation
▲	Fillet weld
F _{EXX}	Ultimate strength as rated by electrode classification number

Detailed result for C245-w 1 / SHP4

Weld resistance check (AISC 360-16 – J2-4)

$$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} = 2.56 \text{ kip} \geq F_n = 2.08 \text{ kip}$$

Where:

$$F_{nw} = 62.1 \text{ ksi} \text{ – nominal stress of weld material:}$$

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot \sin^{1.5} \theta)$, where:
 - $F_{EXX} = 70.0 \text{ ksi}$ – electrode classification number, i.e. minimum specified tensile strength
 - $\theta = 76.5^\circ$ – angle of loading measured from the weld longitudinal axis

$$A_{we} = 0.05 \text{ in}^2 \text{ – effective area of weld critical element}$$

$$\phi = 0.75 \text{ – resistance factor for welded connections}$$



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 235
Project Name: JKV-EMS		

Project:
Project no:
Author:



Buckling

Loads	Shape	Factor [-]
10 LRFD_{(6.1)}-1.2D+L+1.6S(1)	1	10.75
	2	11.07
	3	11.73
	4	12.20
	5	12.22
	6	13.58
108 LRFD_{(12.7)}-0.9D+W(2)	1	68.45
	2	82.01
	3	84.37
	4	87.89
	5	94.19
	6	96.90
106 LRFD_{(12.5)}-0.9D+W(4)	1	52.78
	2	54.43
	3	57.33
	4	57.77
	5	59.96
	6	68.38
62 LRFD_{(10.1)}-1.2D+L+0.5S+W(5)	1	24.89
	2	25.65
	3	26.31
	4	26.57
	5	26.89
	6	28.10
104 LRFD_{(12.3)}-0.9D+W(3)	1	39.14
	2	40.21
	3	43.31
	4	45.75
	5	52.27
	6	55.07
9 LRFD_{(5)}-1.2D+L+1.6Lr(6)	1	10.33
	2	10.63
	3	11.27
	4	11.72
	5	12.74
	6	14.07
124 LRFD_{(11.7)}-1.2D+L+0.2S+E(7)	1	17.10
	2	17.54
	3	18.76
	4	18.83
	5	19.29
	6	20.44
131 LRFD_{(13.6)}-0.9D+E(8)	1	26.55
	2	27.32
	3	28.97
	4	30.11



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 236
Project Name: JKV-EMS		

Project:
Project no:
Author:



Loads	Shape	Factor [-]
	5	30.39
	6	33.76
120 LRFD_{(11.3)}-1.2D+L+0.2S+E(9)	1	17.33
	2	17.83
	3	18.91
	4	19.61
	5	19.64
	6	22.57
127 LRFD_{(13.2)}-0.9D+E(10)	1	25.95
	2	26.60
	3	27.38
	4	28.47
	5	28.94
	6	29.41
123 LRFD_{(11.6)}-1.2D+L+0.2S+E(11)	1	17.45
	2	17.96
	3	19.04
	4	19.80
	5	20.03
	6	22.21
105 LRFD_{(12.4)}-0.9D+W(12)	1	34.76
	2	35.65
	3	38.10
	4	39.57
	5	40.35
	6	47.71
63 LRFD_{(10.2)}-1.2D+L+0.5S+W(13)	1	18.38
	2	18.85
	3	19.14
	4	20.17
	5	20.56
	6	20.79
133 LRFD_{(13.8)}-0.9D+E(16)	1	25.59
	2	26.35
	3	27.92
	4	28.43
	5	29.00
	6	31.55
122 LRFD_{(11.5)}-1.2D+L+0.2S+E(17)	1	17.42
	2	17.93
	3	19.00
	4	19.65
	5	19.78
	6	21.67
102 LRFD_{(12.1)}-0.9D+W(14)	1	54.48
	2	56.71



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 237
Project Name: JKV-EMS		

Project:
Project no:
Author:



Loads	Shape	Factor [-]
	3	57.52
	4	61.71
	5	72.82
	6	73.46
69 LRFD_{10.8}-1.2D+L+0.5S+W(15)	1	18.25
	2	18.71
	3	20.01
	4	20.61
	5	21.92
	6	24.71
125 LRFD_{11.8}-1.2D+L+0.2S+E(18)	1	17.03
	2	17.53
	3	18.58
	4	19.03
	5	19.31
	6	21.13
130 LRFD_{13.5}-0.9D+E(19)	1	26.66
	2	27.33
	3	29.24
	4	29.81
	5	30.11
	6	32.76
118 LRFD_{11.1}-1.2D+L+0.2S+E(20)	1	17.35
	2	17.78
	3	18.95
	4	19.06
	5	19.68
	6	20.26
129 LRFD_{13.4}-0.9D+E(21)	1	26.00
	2	26.74
	3	28.37
	4	28.63
	5	29.43
	6	34.27
132 LRFD_{13.7}-0.9D+E(22)	1	25.69
	2	26.34
	3	27.92
	4	28.25
	5	29.11
	6	29.94
121 LRFD_{11.4}-1.2D+L+0.2S+E(23)	1	17.20
	2	17.70
	3	18.78
	4	19.26
	5	19.49
	6	22.31



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Made by: WNH

Date: 06/16/2025

Job No. 25-003

Revision:

Date:

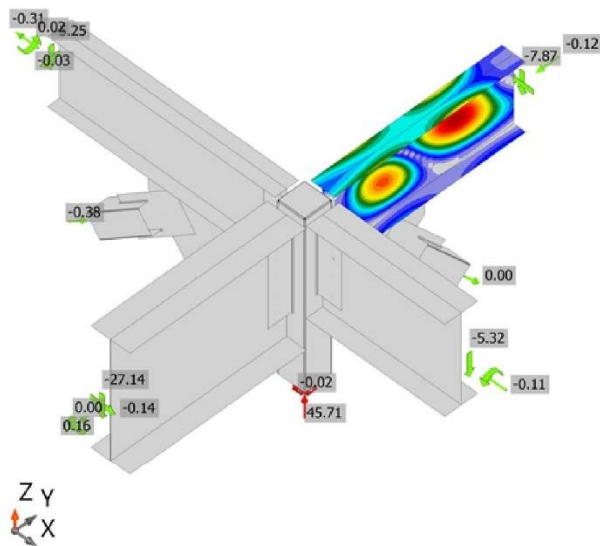
Sheet No. 238

Project Name: JKV-EMS

Project:
Project no:
Author:



Loads	Shape	Factor [-]
126 LRFD_{13.1}-0.9D+E(24)	1	26.25
	2	26.90
	3	27.99
	4	28.79
	5	29.67
	6	29.90
19 LRFD_{7.6}-1.2D+1.6Lr+0.5W(25)	1	10.91
	2	11.22
	3	11.90
	4	12.37
	5	13.75
	6	14.83



First buckling mode shape, 9 LRFD_{5}-1.2D+L+1.6Lr(6)



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 239
Project Name: JKV-EMS		

Project:
Project no:
Author:



Code settings

Item	Value	Unit	Reference
Friction coefficient - concrete	0.40	-	ACI 349-01 – B.6.1.4
Friction coefficient in slip-resistance	0.30	-	AISC 360-16 – J3.8
Limit plastic strain	0.05	-	
Detailing	Yes		
Distance between bolts [d]	2.66	-	AISC 360-16 – J3.3
Concrete breakout resistance check	Both		
Base metal capacity check at weld fusion face	No		AISC 360-16 – J2-2
Deformation at bolt hole at service load is design consideration	Yes		AISC 360-16 – J3.10
Cracked concrete	Yes		ACI 318-14 – 17
Local deformation check	Yes		
Local deformation limit	0.03	-	CIDECT DG 1, 3 – 1.1
Geometrical nonlinearity (GMNA)	Yes		Analysis with large deformations for hollow section joints



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 240
Project Name: JKV-EMS		

EXISTING BUILDING



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Revision:	Date:	Sheet No. 241
Project Name: JKV-EMS		

EXISTING BUILDING - NEW SUPPORT ON GL 6 FROM GL B TO E

ROOF LOADING

$$\text{DEAD} = 33 - 25 = 8 \text{ PSF (PER EXISTING DRAWINGS TOP CHORD)}$$

$$\text{LIVE} = 25 \text{ PSF (PER EXISTING DRAWINGS)}$$

$$\text{BOTTOM CHORD} = 10 \text{ PSF (PER EXISTING DRAWINGS)}$$

$$\text{COMBINED DEAD} = 8 + 10 = 18 \text{ PSF ADD 12 PSF FOR TRUSS WEIGHT}$$

$$\text{DEAD} = 30 \text{ PSF}$$

$$\text{LIVE} = 25 \text{ PSF}$$

LOADS TO GL 6

$$T_W \approx 31 \text{ ft}$$

$$W_D = (30)(31) = 930 \text{ lb/ft}$$

$$W_L = (25)(31) = 775 \text{ lb/ft}$$

TRY SPANNING FULL LENGTH

$$L = 33.67 \text{ ft} \rightarrow W21 \times 68 \text{ w/1.194 in DEFL. NG}$$

TRY WITH COLUMN IN KITCHEN BEHIND REFRIGERATORS

$$\text{SPAN 1} = 28.0833 \text{ ft}$$

$$\text{SPAN 2} = 5.5833 \text{ ft}$$

$$\rightarrow W18 \times 35 \Delta = 0.832 \text{ in (USING IN MODEL)}$$

$$W16 \times 36 \Delta = 0.948 \text{ in.}$$

(SEE TEDDS CALC)

DEAD REACTIONS

$$R_{AD} = 10.7 \text{ k}$$

$$R_{BD} = 33.4 \text{ k}$$

$$R_{CD} = -11.6 \text{ k}$$

LIVE REACTIONS

$$R_{AL} = 8.6 \text{ k}$$

$$R_{BL} = 26.8 \text{ k}$$

$$R_{CL} = -9.3 \text{ k}$$

WORST CASE LOAD TO COLUMN

$$P = 1.2(33.4) + (1.6(26.8)) = 82.96 \text{ k}$$

$$\rightarrow \text{USE HSS } 4 \times 4 \times \frac{3}{8}$$

(SEE TEDDS CALC)

LOAD TO END COLUMN

$$P = 1.2(10.7) + (1.6(8.6)) = 26.6 \text{ k}$$

$$\rightarrow \text{COULD USE } 3" \text{ STD PIPE}$$

(SEE TEDDS CALC)



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Revision:	Date:	Sheet No. 242
Project Name: JKV-EMS		

EXISTING BUILDING - NEW SUPPORT ON GLB FROM GL 5 TO 7.5

$$TW1 = \frac{10.5}{2} = 5.25 \text{ ft}$$

$$TW2 = \frac{10.5}{2} + \frac{33.5}{2} = 22 \text{ ft}$$

$$L_{OF TW1} = 10.25 \text{ ft}$$

$$L_{OF TW2} = 10.25 \text{ ft}$$

$$L = 20.5 \text{ ft}$$

$$W_{D1} = 5.25 (30) = 157.5 \text{ lb/ft}$$

$$W_{L1} = 5.25 (25) = 131.25 \text{ lb/ft}$$

$$W_{D2} = 22 (30) = 660 \text{ lb/ft}$$

$$W_{L2} = 22 (25) = 550 \text{ lb/ft}$$

POINT LOADS @ 10.25 ft

$$P_D = -11.6 \text{ k}$$

$$P_L = -9.3 \text{ k}$$

USE W18 x 35 (SEE TEDDS CALC)

$$R_{AD} = 5.1 \text{ k}$$

$$R_{AL} = 3.8 \text{ k}$$

$$R_{BD} = 8.2 \text{ k}$$

$$R_{BL} = 6.5 \text{ k}$$

$$R_A = 12.2 \text{ k FACTORED}$$

$$R_B = 20.2 \text{ k FACTORED}$$

COLUMN @ B/5

→ 3" STD. PIPE (SEE TEDDS CALC)

COLUMN @ B/7.5

→ 3" STD. PIPE (SEE TEDDS CALC)

ADDITIONAL LOAD TO B/5

$$P_D = 0.1575 \left(\frac{20.5}{2} \right) = 1.62 \text{ k}$$

$$P_L = 0.13125 \left(\frac{20.5}{2} \right) = 1.35 \text{ k}$$

$$P = (1.2(1.62)) + (1.6(1.35)) = 4.11 \text{ k FACTORED}$$



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Date: 06/16/2025

Job No. 25-003

Revision:

Date:

Sheet No. 243

Project Name: JKV-EMS

NEW ROOF ON EXISTING BUILDING

DEAD = 30 PSF (20 TOP CHORD, 10 BOT. CHORD)

LIVE = 25 PSF

OUTLOOKER AT NEW ROOF TRUSS

OVERHANG = 1'-4"

SPACING = 2'-0"

$W_D = 2(30) = 60 \frac{\text{lb}}{\text{ft}}$

$W_L = 2(25) = 50 \frac{\text{lb}}{\text{ft}}$

USE 2x4 MIN. (SEE TEDDS CALC.)

CONCRETE COLUMN @ CANOPY SUPPORT

LENGTH OF COLUMN = 4.25 ft

SIZE = 8" x 8"

$P = 7000 \text{ lb}$

$V = \frac{11(35)}{2} = 192.5 \text{ lb}$

$M = 192.5(4.25) = 818.2 \text{ ft-lb}$

USE 8x8 W/ (4) #4 & #3 TIES @ 8" o.c. (SEE TEDDS CALC.)

FOOTING

3' x 3' x 2' W/ (4) #6 TOP & BOT. EA. WAY (SEE TEDDS CALC.)



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Revision:	Date:	Sheet No. 244
Project Name: JKV-EMS		

FOOTINGS FOR COLUMNS IN EXISTING

B/5 - ADD TO EXISTING

B/7.5 - NEW

B4/6 - NEW

E/6 - NEW

SERVICE LOADING @ COLUMN LOCATIONS

B/5

$$P_D = 5.1 + 1.62 = 6.72 \text{ K}$$

$$P_L = 3.8 + 1.35 = 5.15 \text{ K}$$

$$P_T = 6.72 + 5.15 = 11.87 \text{ K}$$

$$\text{MIN AREA} = \frac{11.87}{1.5} = 7.913 \text{ ft}^2$$

USE 3x3 MIN. (SEE TEDDS CALC)

→ 3x3 x 12" W/(4) #4 EACH WAY BOT.

B/7.5

$$P_D = 8.2 \text{ K}$$

$$P_L = 6.5 \text{ K}$$

$$P_T = 8.2 + 6.5 = 14.7 \text{ K}$$

$$\text{MIN. AREA} = \frac{14.7}{1.5} = 9.8 \text{ ft}^2$$

USE 4x4 MIN. (SEE TEDDS CALC)

→ 4x4 x 12" W/(5) #5 EACH WAY BOT.

B4/6

$$P_D = 33.4 \text{ K}$$

$$P_L = 26.8 \text{ K}$$

$$P_T = 60.2 \text{ K}$$

$$\text{MIN AREA} = \frac{60.2}{1.5} = 40.133 \text{ ft}^2$$

USE 7x7 MIN. (SEE TEDDS CALC)

→ 7x7 x 12" W/(8) #5 EACH WAY BOT.

E/6

$$P_D = 10.7 \text{ K}$$

$$P_L = 8.6 \text{ K}$$

$$P_T = 10.7 + 8.6 = 19.3 \text{ K}$$

$$\text{MIN AREA} = \frac{19.3}{1.5} = 12.87 \text{ ft}^2$$

USE 4x4 MIN. (SEE TEDDS CALC)

→ 4x4 x 12" W/(5) #5 EACH WAY BOT.

BRICK SUPPORT ON EXIST. BUILD. FND.

BRICK HEIGHT = 4.5 FT

$$SW = 4.5(40) = 180 \text{ lb/ft}$$

USE 12" TALL CORBEL W/#5 TENSION BARS
@ 8" O.C. W/(1) #3 TIE (SEE TEDDS CALC)



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Revision:	Date:	Sheet No. 245
Project Name: JKV-EMS		

SNOW LOAD ON CANOPY

In accordance with ASCE7-16

Tedds calculation version 1.0.12

Building details

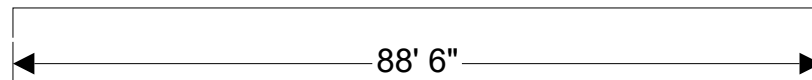
Roof type; Flat
Width of roof; b = 88.50 ft

Ground snow load

Ground snow load (Figure 7.2-1); $p_g = 20.00 \text{ lb/ft}^2$
Density of snow; $\gamma = \min(0.13 \times p_g / 1 \text{ ft} + 14 \text{ lb/ft}^3, 30 \text{ lb/ft}^3) = 16.60 \text{ lb/ft}^3$
Surface roughness category (Sect. 26.7); C
Exposure condition (Table 7.3-1); Fully exposed
Exposure factor (Table 7.3-1); $C_e = 0.90$
Thermal condition (Table 7.3-2); Unheated structures
Thermal factor (Table 7.3-2); $C_t = 1.20$
Importance category (Table 1.5-1); II
Importance factor (Table 1.5-2); $I_s = 1.00$
Min snow load for low slope roofs (Sect 7.3.4); $p_{f\_min} = I_s \times p_g = 20.00 \text{ lb/ft}^2$
Flat roof snow load (Sect 7.3); $p_f = 0.7 \times C_e \times C_t \times I_s \times p_g = 15.12 \text{ lb/ft}^2$

Balanced load

20.0 psf



Roof elevation

Drift calculations

Balanced snow load height; $h_b = p_f / \gamma = 0.91 \text{ ft}$
Length of upper roof; $l_u = 88.50 \text{ ft}$
Length of lower roof; $l_l = 8.50 \text{ ft}$
Height diff between upper and lower roofs; $h_{diff} = 4.00 \text{ ft}$
Height from balance load to top of upper roof; $h_c = h_{diff} - h_b = 3.09 \text{ ft}$
Drift height leeward drift; $h_{d\_l} = \min(\sqrt{l_s} \times (0.43 \times (\max(20 \text{ ft}, l_u) \times 1 \text{ ft}^2)^{1/3} \times (p_g / 1 \text{ lb/ft}^2 + 10)^{1/4} - 1.5 \text{ ft}), 0.6 \times l_l) = 2.98 \text{ ft}$
Drift height windward drift; $h_{d\_w} = \min(0.75 \times \sqrt{l_s} \times (0.43 \times (\max(20 \text{ ft}, l_l) \times 1 \text{ ft}^2)^{1/3} \times (p_g / 1 \text{ lb/ft}^2 + 10)^{1/4} - 1.5 \text{ ft}), \sqrt{l_s \times p_g \times l_l / (4 \times \gamma)}) = 0.92 \text{ ft}$
Maximum lw/ww drift height; $h_{d\_max} = \max(h_{d\_w}, h_{d\_l}) = 2.98 \text{ ft}$
Drift height; $h_d = \min(h_{d\_max}, h_c) = 2.98 \text{ ft}$
Drift width; $W_d = \min(4 \times h_{d\_max}, 8 \times h_c) = 11.94 \text{ ft}$
Drift surcharge load; $p_d = h_d \times \gamma = 49.55 \text{ lb/ft}^2$



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 246
Project Name: JKV-EMS		

64.7 psf

15.1 psf

8' 6"

Elevation on snow drift

CANOPY ROOF JOIST

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

Tedds calculation version 2.2.24

Member1 span 1 results summary	Unit	Capacity	Maximum	Utilization	Result
Bending stress	lb/in ²	1208	742	0.615	PASS
Shear stress	lb/in ²	135	75	0.552	PASS
Bearing stress	lb/in ²	425	111	0.261	PASS
Deflection	in	0.34	0.112	0.330	PASS
Member1 span 2 results summary	Unit	Capacity	Maximum	Utilization	Result
Bending stress	lb/in ²	1208	131	0.108	PASS
Shear stress	lb/in ²	135	24	0.178	PASS
Bearing stress	lb/in ²	425	111	0.261	PASS
Deflection	in	0.17	0.067	0.391	PASS

ANALYSIS

Tedds calculation version 1.0.38



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Date: 06/16/2025

Job No. 25-003

Revision:

Date:

Sheet No. 247

Project Name: JKV-EMS

Geometry

Geometry (ft) - Spruce-Pine-Fir(2" & wider(No.2))

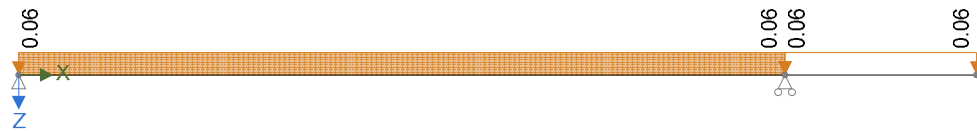


Span	Length (ft)	Section	Start Support	End Support
1	6.8	2x8	Pinned	Roller Pin X
2	1.7	2"x8"	Roller Pin X	Free
2x8: Area 11 in ² , Inertia Major 48 in ⁴ , Inertia Minor 2 in ⁴ , Shear area parallel to Minor 9 in ² , Shear area parallel to Major 9 in ²				
2x8(1): Area 11 in ² , Inertia Major 48 in ⁴ , Inertia Minor 2 in ⁴ , Shear area parallel to Minor 9 in ² , Shear area parallel to Major 9 in ²				
2"x8": Area 11 in ² , Inertia Major 48 in ⁴ , Inertia Minor 2 in ⁴ , Shear area parallel to Minor 9 in ² , Shear area parallel to Major 9 in ²				
Spruce-Pine-Fir(2" & wider(No.2)): Density 29.1 lbm/ft ³ , Youngs 1400 ksi, Shear 87 ksi, Thermal 0 °C ⁻¹				

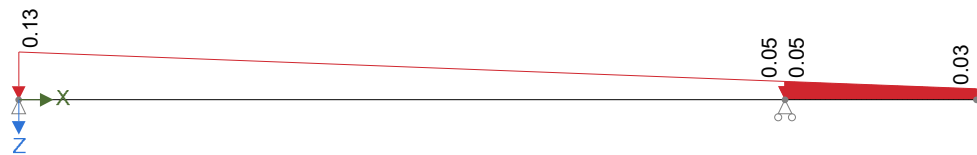
Loading

Self weight included

Dead - Loading (kips/ft)



Roof Live - Loading (kips/ft)



Wind - Loading (kips/ft)





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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 248
Project Name: JKV-EMS		

Load combination factors

Load combination	Self Weight	Dead	Roof Live	Wind
1.0D + 1.0Lr (Strength)	1.0 0 0	1.0 0 0	1.0 0 0	
1.0D + 0.6W (Strength)	1.0 0 0	1.0 0 0		0.6 0
1.0D + 0.75L + 0.75Lr + 0.45W (Strength)	1.0 0 0	1.0 0 0	0.7 5	0.4 5
0.6D + 0.6W (Strength)	0.6 0	0.6 0		0.6 0
1.0D + 1.0Lr (Service)	1.0 0 0	1.0 0 0	1.0 0 0	
1.0Lr (Service)			1.0 0 0	
1.0W (Service)				1.0 0 0

Member Loads

Member	Load case	Load Type	Orientation	Description
Member1	Dead	UDL	GlobalZ	0.06 kips/ft
Member1	Roof Live	VDL	GlobalZ	0.13 kips/ft to 0.03 kips/ft

Element Loads

Element	Load case	Load Type	Orientation	Description
1	Wind	UDL	GlobalZ	-0.05 kips/ft
2	Wind	UDL	GlobalZ	-0.1 kips/ft



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Revision:	Date:	Sheet No. 249
Project Name: JKV-EMS		

Results

Reactions

Load case: Self Weight

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.007	0
2	0	0.012	0

Load case: Dead

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.191	0
2	0	0.319	0

Load case: Roof Live

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.342	0
2	0	0.336	0

Load case: Wind

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	-0.149	0
2	0	-0.361	0

Load combination: 1.0D + 1.0Lr (Strength)

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.54	0
2	0	0.667	0



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Made by: WNH

Date: 06/16/2025

Job No. 25-003

Revision:

Date:

Sheet No. 250

Project Name: JKV-EMS

Load combination: 1.0D + 0.6W (Strength)

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.109	0
2	0	0.114	0

Load combination: 1.0D + 0.75L + 0.75Lr + 0.45W (Strength)

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.388	0
2	0	0.42	0

Load combination: 0.6D + 0.6W (Strength)

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.03	0
2	0	-0.018	0

Load combination: 1.0D + 1.0Lr (Service)

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.54	0
2	0	0.667	0

Load combination: 1.0Lr (Service)

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.342	0
2	0	0.336	0

Load combination: 1.0W (Service)

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	-0.149	0
2	0	-0.361	0



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Job No. 25-003

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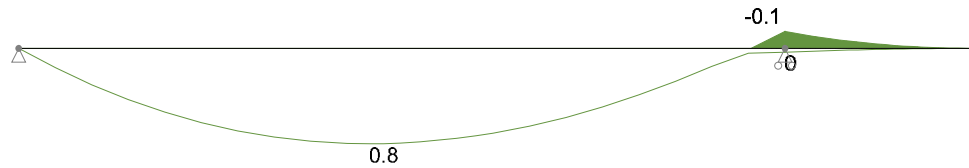
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Sheet No. 251

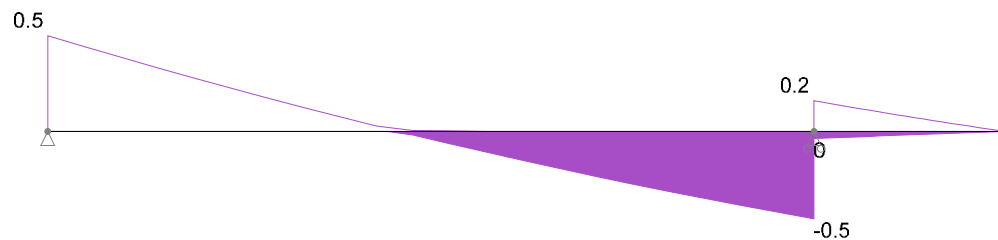
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Forces

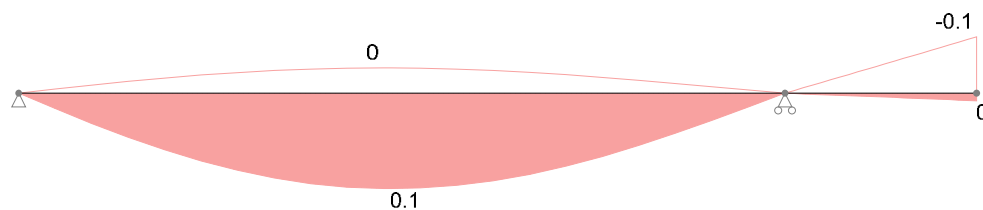
Strength combinations - Moment envelope (kip_ft)



Strength combinations - Shear envelope (kips)



Service combinations - Deflection envelope (in)



Member results

Envelope - Strength combinations

Member	Position (ft)	Shear force (kips)		Moment (kip_ft)	
Member1	0	0.54 (max abs)	0.03	0	
	3.12	0.009	0	0.813 (max)	0.057
	6.8	0.174	-0.493	0.033	-0.143 (min)

Envelope - Service combinations

Member	Position (ft)	Deflection (in)	
Member1	3.31	0.112 (max)	-0.03
	8.5	0.009	-0.067 (min)

;

Member1 - Span 1

Member details

Service condition;

Dry



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 252
Project Name: JKV-EMS		

Sawn lumber section details

Number of sections in member;
Nominal breadth of sections;
Breadth of sections;
Nominal depth of sections;
Depth of sections;
Material;

$$N = 1$$

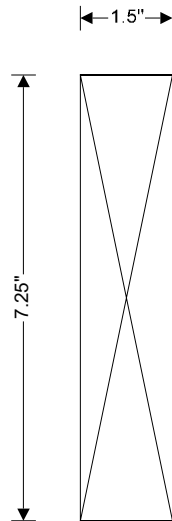
$$b_{nom} = 2 \text{ in}$$

$$b = 1.5 \text{ in}$$

$$d_{nom} = 8 \text{ in}$$

$$d = 7.25 \text{ in}$$

Spruce-Pine-Fir, 2" & wider, No.2 grade



2"x8" sawn lumber section

Cross-sectional area, A , 10.875 in²

Section modulus, S_x , 13.1 in³

Section modulus, S_y , 2.7 in³

Second moment of area, I_x , 47.6 in⁴

Second moment of area, I_y , 2 in⁴

Radius of gyration, r_x , 2.093 in

Radius of gyration, r_y , 0.433 in

Spruce-Pine-Fir, 2" & wider, No.2 grade

Bending, F_b , 875 psi

Shear parallel to grain, F_v , 135 psi

Compression parallel to grain, F_c , 1150 psi

Compression perpendicular to grain, F_{c_perp} , 425 psi

Tension parallel to grain, F_t , 450 psi

Modulus of elasticity, E , 1400000 psi

Minimum modulus of elasticity, E_{min} , 510000 psi

Density, ρ , 29.098 lbm/ft³

Specific gravity, G , 0.42

Span details

Bearing length;

$$L_b = 4 \text{ in}$$

Consider Combination 1 - 1.0D + 1.0Lr (Strength)

Adjustment factors - Table 4.3.1

Ten years load duration factor - Table 2.3.2;

$$C_D = 1$$

Size factor for bending - Table 4A;

$$C_{Fb} = 1.2$$

Repetitive member factor - Table 4.3.9;

$$C_r = 1.15$$

Check design at start of span

Bending members - Shear - cl.3.4

Design shear force;

$$V_x = 540 \text{ lb}$$

Design shear stress - Table 4.3.1;

$$F_{v,x}' = F_v \times C_D = 135 \text{ lb/in}^2$$

Actual shear stress - eq.3.4-2;

$$f_{v,x} = 3 \times V_x / (2 \times b \times d) = 75 \text{ lb/in}^2$$

$$f_{v,x} / F_{v,x}' = 0.552$$

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression;

$$R_x = 540 \text{ lb}$$

Design bearing stress - Table 4.3.1;

$$F_{c\_perp,x}' = F_{c\_perp} = 425 \text{ lb/in}^2$$

Actual bearing stress;

$$f_{c\_perp,x} = R_x / (b \times L_b) = 90 \text{ lb/in}^2$$

$$f_{c\_perp,x} / F_{c\_perp,x}' = 0.212$$

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 253
Project Name: JKV-EMS		

Check design 3ft 1.399in along span

Bending members - Flexure - cl.3.3

Design bending moment;

$$M_x = 813 \text{ lb\_ft}$$

Design bending stress - Table 4.3.1;

$$F_{b,x}' = F_b \times C_D \times C_{Fb} \times C_r = 1208 \text{ lb/in}^2$$

Actual bending stress - eq.3.3-2;

$$f_{b,x} = M_x / S_x = 742 \text{ lb/in}^2$$

$$f_{b,x} / F_{b,x}' = 0.615$$

PASS - Design bending stress exceeds actual bending stress

Check design at end of span

Bending members - Flexure - cl.3.3

Design bending moment;

$$M_x = 143 \text{ lb\_ft}$$

Design bending stress - Table 4.3.1;

$$F_{b,x}' = F_b \times C_D \times C_{Fb} \times C_r = 1208 \text{ lb/in}^2$$

Actual bending stress - eq.3.3-2;

$$f_{b,x} = M_x / S_x = 131 \text{ lb/in}^2$$

$$f_{b,x} / F_{b,x}' = 0.108$$

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force;

$$V_x = 493 \text{ lb}$$

Design shear stress - Table 4.3.1;

$$F_{v,x}' = F_v \times C_D = 135 \text{ lb/in}^2$$

Actual shear stress - eq.3.4-2;

$$f_{v,x} = 3 \times V_x / (2 \times b \times d) = 68 \text{ lb/in}^2$$

$$f_{v,x} / F_{v,x}' = 0.503$$

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression;

$$R_x = 667 \text{ lb}$$

Design bearing stress - Table 4.3.1;

$$F_{c\_perp,x}' = F_{c\_perp} = 425 \text{ lb/in}^2$$

Actual bearing stress;

$$f_{c\_perp,x} = R_x / (b \times L_b) = 111 \text{ lb/in}^2$$

$$f_{c\_perp,x} / F_{c\_perp,x}' = 0.261$$

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Consider Combination 5 - 1.0D + 1.0Lr (Service)

Check design 3ft 3.677in along span

Bending members - Deflection - cl.3.5

Instantaneous deflection;

$$\delta_x = 0.112 \text{ in}$$

Allowable deflection;

$$\delta_{x,Allowable} = L_{m1\_s1} / 240 = 0.34 \text{ in}$$

$$\delta_x / \delta_{x,Allowable} = 0.33$$

PASS - Allowable deflection exceeds instantaneous deflection

Member1 - Span 2

Member details

Service condition;

Dry

Sawn lumber section details

Number of sections in member;

$$N = 1$$

Nominal breadth of sections;

$$b_{nom} = 2 \text{ in}$$

Breadth of sections;

$$b = 1.5 \text{ in}$$

Nominal depth of sections;

$$d_{nom} = 8 \text{ in}$$

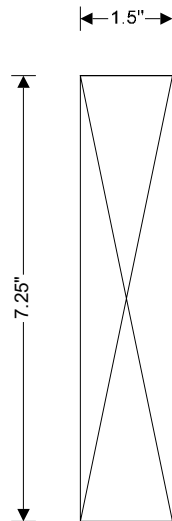


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Revision:	Date:	Sheet No. 254
Project Name: JKV-EMS		

Depth of sections;

Material;



d = 7.25 in

Spruce-Pine-Fir, 2" & wider, No.2 grade

2"x8" sawn lumber section

Cross-sectional area, A, 10.875 in²

Section modulus, S_x, 13.1 in³

Section modulus, S_y, 2.7 in³

Second moment of area, I_x, 47.6 in⁴

Second moment of area, I_y, 2 in⁴

Radius of gyration, r_x, 2.093 in

Radius of gyration, r_y, 0.433 in

Spruce-Pine-Fir, 2" & wider, No.2 grade

Bending, F_b, 875 psi

Shear parallel to grain, F_v, 135 psi

Compression parallel to grain, F_c, 1150 psi

Compression perpendicular to grain, F_{c_perp}, 425 psi

Tension parallel to grain, F_t, 450 psi

Modulus of elasticity, E, 1400000 psi

Minimum modulus of elasticity, E_{min}, 510000 psi

Density, ρ, 29.098 lbm/ft³

Specific gravity, G, 0.42

Span details

Bearing length;

L_b = 4 in

Consider Combination 1 - 1.0D + 1.0Lr (Strength)

Adjustment factors - Table 4.3.1

Ten years load duration factor - Table 2.3.2;

C_D = 1

Size factor for bending - Table 4A;

C_{Fb} = 1.2

Repetitive member factor - Table 4.3.9;

C_r = 1.15

Check design at start of span

Bending members - Flexure - cl.3.3

Design bending moment;

M_x = 143 lb_{ft}

Design bending stress - Table 4.3.1;

F_{b,x'} = F_b × C_D × C_{Fb} × C_r = 1208 lb/in²

Actual bending stress - eq.3.3-2;

f_{b,x} = M_x / S_x = 131 lb/in²

f_{b,x} / F_{b,x'} = 0.108

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force;

V_x = 174 lb

Design shear stress - Table 4.3.1;

F_{v,x'} = F_v × C_D = 135 lb/in²

Actual shear stress - eq.3.4-2;

f_{v,x} = 3 × V_x / (2 × b × d) = 24 lb/in²

f_{v,x} / F_{v,x'} = 0.178

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression;

R_x = 667 lb

Design bearing stress - Table 4.3.1;

F_{c_perp,x'} = F_{c_perp} = 425 lb/in²

Actual bearing stress;

f_{c_perp,x} = R_x / (b × L_b) = 111 lb/in²

f_{c_perp,x} / F_{c_perp,x'} = 0.261



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 255
Project Name: JKV-EMS		

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Consider Combination 5 - 1.0D + 1.0Lr (Service)

Check design at end of span

Bending members - Deflection - cl.3.5

Instantaneous deflection;

$$\delta_x = 0.067 \text{ in}$$

Allowable deflection;

$$\delta_{x, \text{Allowable}} = L_{m1\_s2} / 120 = 0.17 \text{ in}$$

$$\delta_x / \delta_{x, \text{Allowable}} = 0.391$$

PASS - Allowable deflection exceeds instantaneous deflection

CANOPY ROOF GIRDER

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

Tedds calculation version 2.2.24

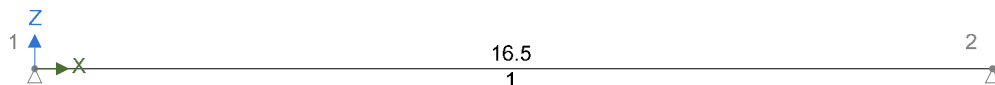
Girder results summary	Unit	Capacity	Maximum	Utilization	Result
Bending stress	lb/in ²	2600	1406	0.541	PASS
Shear stress	lb/in ²	265	78	0.295	PASS
Bearing stress	lb/in ²	650	143	0.220	PASS
Deflection	in	0.792	0.547	0.690	PASS

ANALYSIS

Tedds calculation version 1.0.38

Geometry

Geometry (ft) - 26F-1.9E - 5"x11"



Span	Length (ft)	Section	Start Support	End Support
1	16.5	5"x11"	Pinned	Pinned
5"x11": Area 55 in ² , Inertia Major 555 in ⁴ , Inertia Minor 115 in ⁴ , Shear area parallel to Minor 46 in ² , Shear area parallel to Major 46 in ²				
26F-1.9E: Density 34.2 lbm/ft ³ , Youngs 2000 ksi, Shear 125 ksi, Thermal 0 °C ⁻¹				

Loading

Self weight included



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Date: 06/16/2025

Job No. 25-003

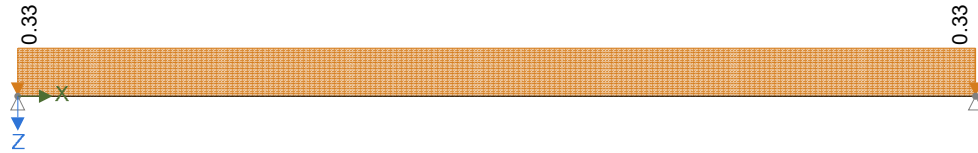
Revision:

Date:

Sheet No. 256

Project Name: JKV-EMS

Design Load - Loading (kips/ft)



Load combination factors

Load combination	Self Weight	Design Load
Design Loading (Strength)	1.00	1.00

Member Loads

Member	Load case	Load Type	Orientation	Description
Girder	Design Load	UDL	GlobalZ	0.33 kips/ft

Results

Reactions

Load case: Self Weight

Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
1	0	0.108	0
2	0	0.108	0

Load case: Design Load

Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
1	0	2.756	0
2	0	2.756	0

Load combination: Design Loading (Strength)

Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
1	0	2.863	0
2	0	2.863	0



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Job No. 25-003

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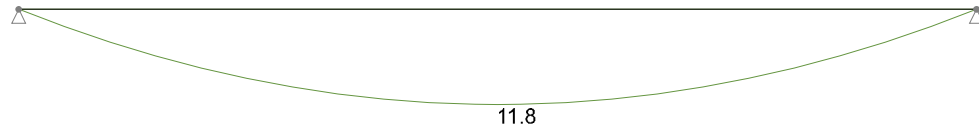
Date:

Sheet No. 257

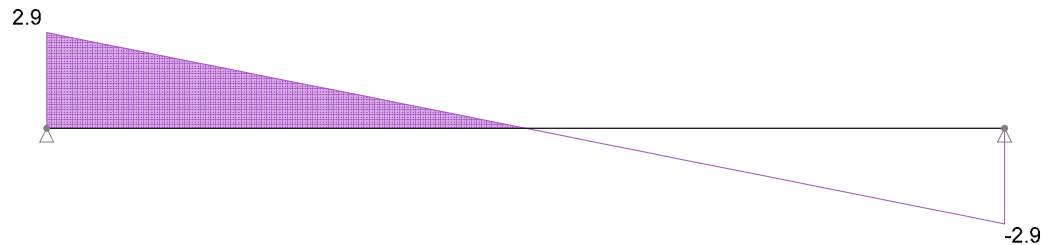
Project Name: JKV-EMS

Forces

Strength combinations - Moment envelope (kip_ft)



Strength combinations - Shear envelope (kips)



Member results

Envelope - Strength combinations

Member	Position (ft)	Shear force (kips)	Moment (kip_ft)
Girder	0	2.863 (max abs)	0 (min)
	8.25	0	11.811 (max)
	16.5	-2.863	0 (min)

Girder - Span 1

Member details

Service condition;

Dry

Glulam section details

Number of sawn lumber sections in member;

N = 1

Breadth of sections;

b = 5 in

Depth of sections;

d = 11 in

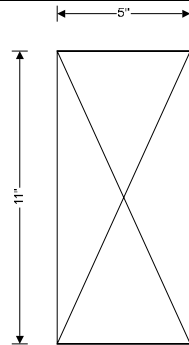
Material;

Glulam, 26F-1.9E grade



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Revision:	Date:	Sheet No. 258
Project Name: JKV-EMS		



5"x11" glulam section

Cross-sectional area, A , 55 in²
Section modulus, S_x , 100.8 in³
Section modulus, S_y , 45.8 in³
Second moment of area, I_x , 554.6 in⁴
Second moment of area, I_y , 114.6 in⁴
Radius of gyration, r_x , 3.175 in
Radius of gyration, r_y , 1.443 in
Glulam grade 26F-1.9E
Bending about x-x axis, bottom of beam in tension, F_{bx_pos} , 2600 psi
Bending about x-x axis, top of beam in tension, F_{bx_neg} , 1950 psi
Bending about y-y axis, F_{by} , 1600 psi
Shear parallel to grain, bending about x-x axis, F_{vx} , 265 psi
Shear parallel to grain, bending about y-y axis, F_{vy} , 230 psi
Compression parallel to grain, F_c , 1600 psi
Compression perpendicular to grain, compression face, $F_{c\_perp\_comp}$, 650 psi
Compression perpendicular to grain, tension face, $F_{c\_perp\_tens}$, 650 psi
Compression perpendicular to grain, F_{c_perp} , 560 psi
Tension parallel to grain, F_t , 1150 psi
Modulus of elasticity, E_{truss} , 2000000 psi
Modulus of elasticity, E_{truss} , 1700000 psi
Modulus of elasticity, E_{truss} , 1900000 psi
Modulus of elasticity, E_{truss} , 1600000 psi
Minimum modulus of elasticity, E_{ymin} , 1000000 psi
Minimum modulus of elasticity, E_{ymin} , 850000 psi
Density, ρ , 34.204 lbm/ft³
Specific gravity, G , 0.5

Span details

Bearing length;

$$L_b = 4 \text{ in}$$

Consider Combination 1 - Design Loading (Strength)

Adjustment factors - Table 5.3.1

Ten years load duration factor - Table 2.3.2;

$$C_D = 1$$

Check design at start of span

Bending members - Shear - cl.3.4

Design shear force;

$$V_x = 2863 \text{ lb}$$

Design shear stress - Table 5.3.1;

$$F_{v,x}' = F_{vx} \times C_D = 265 \text{ lb/in}^2$$

Actual shear stress - eq.3.4-2;

$$f_{v,x} = 3 \times V_x / (2 \times b \times d) = 78 \text{ lb/in}^2$$

$$f_{v,x} / F_{v,x}' = 0.295$$

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression;

$$R_x = 2863 \text{ lb}$$

Des.bear.stress (tens.face) - Table 5.3.1;

$$F_{c\_perp,x}' = F_{c\_perp\_tens} = 650 \text{ lb/in}^2$$

Actual bearing stress;

$$f_{c\_perp,x} = R_x / (b \times L_b) = 143 \text{ lb/in}^2$$

$$f_{c\_perp,x} / F_{c\_perp,x}' = 0.220$$

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Check design 8ft 3in along span

Bending members - Flexure - cl.3.3

Design bending moment;

$$M_x = 11811 \text{ lb\_ft}$$

Des.bend.stress (+ve mom) - Table 5.3.1;

$$F_{b,x}' = F_{bx\_pos} \times C_D = 2600 \text{ lb/in}^2$$

Actual bending stress - eq.3.3-2;

$$f_{b,x} = M_x / S_x = 1406 \text{ lb/in}^2$$

$$f_{b,x} / F_{b,x}' = 0.541$$

PASS - Design bending stress exceeds actual bending stress



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 259
Project Name: JKV-EMS		

Check design at end of span

Bending members - Shear - cl.3.4

Design shear force;

$$V_x = 2863 \text{ lb}$$

Design shear stress - Table 5.3.1;

$$F_{v,x}' = F_{vx} \times C_D = 265 \text{ lb/in}^2$$

Actual shear stress - eq.3.4-2;

$$f_{v,x} = 3 \times V_x / (2 \times b \times d) = 78 \text{ lb/in}^2$$

$$f_{v,x} / F_{v,x}' = 0.295$$

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression;

$$R_x = 2863 \text{ lb}$$

Des.bear.stress (tens.face) - Table 5.3.1;

$$F_{c\_perp,x}' = F_{c\_perp\_tens} = 650 \text{ lb/in}^2$$

Actual bearing stress;

$$f_{c\_perp,x} = R_x / (b \times L_b) = 143 \text{ lb/in}^2$$

$$f_{c\_perp,x} / F_{c\_perp,x}' = 0.220$$

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Check design 8ft 3in along span

Bending members - Deflection - cl.3.5

Instantaneous deflection;

$$\delta_x = 0.547 \text{ in}$$

Allowable deflection;

$$\delta_{x,Allowable} = L_{m1\_s1} / 250 = 0.792 \text{ in}$$

$$\delta_x / \delta_{x,Allowable} = 0.69$$

PASS - Allowable deflection exceeds instantaneous deflection

CANOPY ROOF LEDGER

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

Tedds calculation version 2.2.24

Ledger span 1 results summary	Unit	Capacity	Maximum	Utilization	Result
Bending stress	lb/in ²	1208	183	0.151	PASS
Shear stress	lb/in ²	135	47	0.350	PASS
Bearing stress	lb/in ²	425	114	0.269	PASS
Deflection	in	0.1	0.006	0.055	PASS
Ledger span 2 results summary	Unit	Capacity	Maximum	Utilization	Result
Bending stress	lb/in ²	1050	183	0.174	PASS
Shear stress	lb/in ²	131	47	0.361	PASS



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Date: 06/16/2025

Job No. 25-003

Revision:

Date:

Sheet No. 260

Project Name: JKV-EMS

Bearing stress	lb/in ²	285	114	0.402	PASS
Deflection	in	0.1	0.006	0.061	PASS

ANALYSIS

Tedds calculation version 1.0.38

Geometry

Geometry (ft) - Spruce-Pine-Fir(2" & wider(No.2))

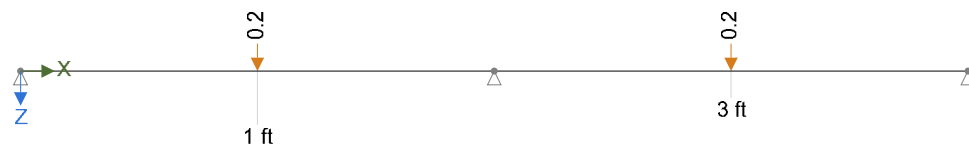


Span	Length (ft)	Section	Start Support	End Support
1	2	2x8	Pinned	Pinned
2	2	2"x8"	Pinned	Pinned
2x8: Area 11 in ² , Inertia Major 48 in ⁴ , Inertia Minor 2 in ⁴ , Shear area parallel to Minor 9 in ² , Shear area parallel to Major 9 in ²				
2x8(1): Area 11 in ² , Inertia Major 48 in ⁴ , Inertia Minor 2 in ⁴ , Shear area parallel to Minor 9 in ² , Shear area parallel to Major 9 in ²				
2"x8": Area 11 in ² , Inertia Major 48 in ⁴ , Inertia Minor 2 in ⁴ , Shear area parallel to Minor 9 in ² , Shear area parallel to Major 9 in ²				
Spruce-Pine-Fir(2" & wider(No.2)): Density 29.1 lbf/ft ³ , Youngs 1400 ksi, Shear 87 ksi, Thermal 0 °C ⁻¹				

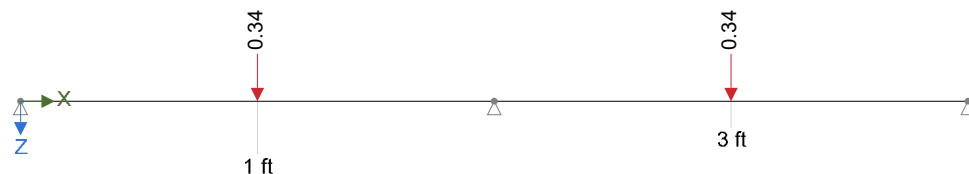
Loading

Self weight included

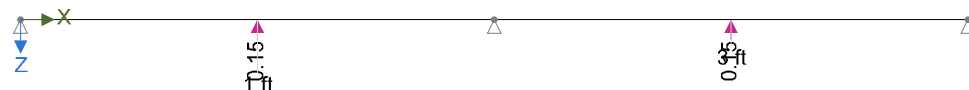
Dead - Loading (kips)



Roof Live - Loading (kips)



Wind - Loading (kips)





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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 261
Project Name: JKV-EMS		

Load combination factors

Load combination	Self Weight	Dead	Roof Live	Wind
1.0D + 1.0Lr (Strength)	1.0 0 0	1.0 0 0	1.0 0 0	
1.0D + 0.6W (Strength)	1.0 0 0	1.0 0 0		0.6 0
1.0D + 0.75L + 0.75Lr + 0.45W (Strength)	1.0 0 0	1.0 0 0	0.7 5	0.4 5
0.6D + 0.6W (Strength)	0.6 0	0.6 0		0.6 0
1.0D + 1.0Lr (Service)	1.0 0 0	1.0 0 0	1.0 0 0	
1.0Lr (Service)			1.0 0 0	
1.0W (Service)				1.0 0 0

Member Loads

Member	Load case	Load Type	Orientation	Description
Ledger	Dead	Point load	GlobalZ	0.2 kips at 1 ft
Ledger	Dead	Point load	GlobalZ	0.2 kips at 3 ft
Ledger	Roof Live	Point load	GlobalZ	0.34 kips at 1 ft
Ledger	Roof Live	Point load	GlobalZ	0.34 kips at 3 ft
Ledger	Wind	Point load	GlobalZ	-0.15 kips at 1 ft
Ledger	Wind	Point load	GlobalZ	-0.15 kips at 3 ft



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Made by: WNH

Date: 06/16/2025

Job No. 25-003

Revision:

Date:

Sheet No. 262

Project Name: JKV-EMS

Results

Reactions

Load case: Self Weight

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.002	0
2	0	0.005	0
3	0	0.002	0

Load case: Dead

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.073	0
2	0	0.25	0
3	0	0.073	0

Load case: Roof Live

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.126	0
2	0	0.431	0
3	0	0.126	0

Load case: Wind

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	-0.055	0
2	0	-0.188	0
3	0	-0.055	0

Load combination: 1.0D + 1.0Lr (Strength)

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.201	0
2	0	0.686	0



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Made by: WNH

Date: 06/16/2025

Job No. 25-003

Revision:

Date:

Sheet No. 263

Project Name: JKV-EMS

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
3	0	0.201	0

Load combination: 1.0D + 0.6W (Strength)

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.042	0
2	0	0.142	0
3	0	0.042	0

Load combination: 1.0D + 0.75L + 0.75Lr + 0.45W (Strength)

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.145	0
2	0	0.494	0
3	0	0.145	0

Load combination: 0.6D + 0.6W (Strength)

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.012	0
2	0	0.04	0
3	0	0.012	0

Load combination: 1.0D + 1.0Lr (Service)

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.201	0
2	0	0.686	0
3	0	0.201	0



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Job No. 25-003

Revision:

Date:

Sheet No. 264

Project Name: JKV-EMS

Load combination: 1.0Lr (Service)

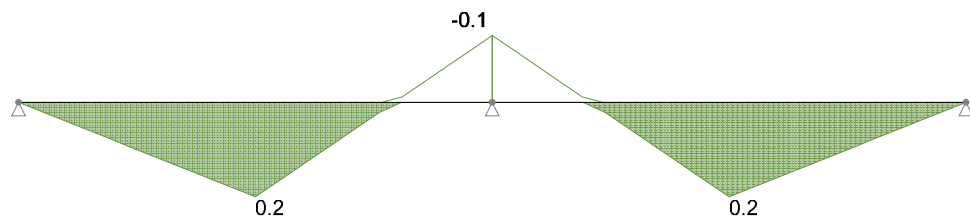
Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.126	0
2	0	0.431	0
3	0	0.126	0

Load combination: 1.0W (Service)

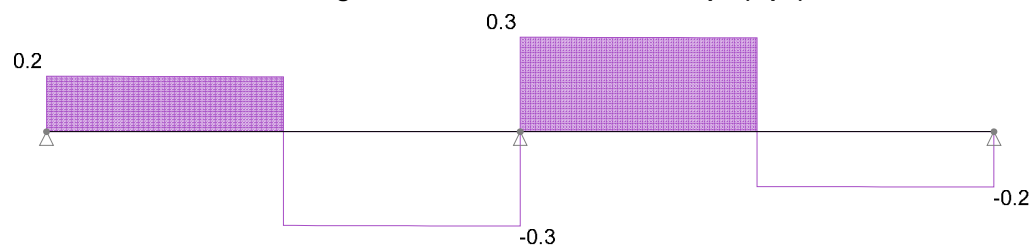
Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	-0.055	0
2	0	-0.188	0
3	0	-0.055	0

Forces

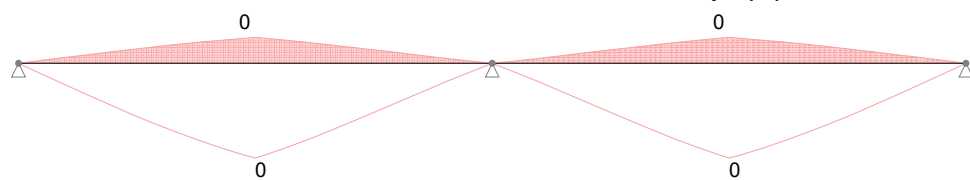
Strength combinations - Moment envelope (kip_ft)



Strength combinations - Shear envelope (kips)



Service combinations - Deflection envelope (in)





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Date: 06/16/2025

Job No. 25-003

Revision:

Date:

Sheet No. 265

Project Name: JKV-EMS

Member results

Envelope - Strength combinations

Member	Position (ft)	Shear force (kips)		Moment (kip_ft)	
Ledger	1	0.199	-0.341	0.2 (max)	0.011
	2	0.343 (max abs)	-0.343	-0.008	-0.142 (min)

Envelope - Service combinations

Member	Position (ft)	Deflection (in)	
Ledger	3	0.006 (max)	-0.002 (min)

;

Ledger - Span 1

Member details

Service condition;

Wet

Sawn lumber section details

Number of sections in member;

N = 1

Nominal breadth of sections;

b_{nom} = 2 in

Breadth of sections;

b = 1.5 in

Nominal depth of sections;

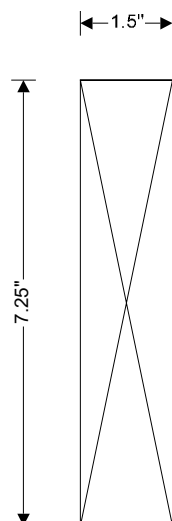
d_{nom} = 8 in

Depth of sections;

d = 7.25 in

Material;

Spruce-Pine-Fir, 2" & wider, No.2 grade



2"x8" sawn lumber section

Cross-sectional area, A, 10.875 in²

Section modulus, S_x, 13.1 in³

Section modulus, S_y, 2.7 in³

Second moment of area, I_x, 47.6 in⁴

Second moment of area, I_y, 2 in⁴

Radius of gyration, r_x, 2.093 in

Radius of gyration, r_y, 0.433 in

Spruce-Pine-Fir, 2" & wider, No.2 grade

Bending, F_b, 875 psi

Shear parallel to grain, F_v, 135 psi

Compression parallel to grain, F_c, 1150 psi

Compression perpendicular to grain, F_{c_perp}, 425 psi

Tension parallel to grain, F_t, 450 psi

Modulus of elasticity, E, 1400000 psi

Minimum modulus of elasticity, E_{min}, 510000 psi

Density, ρ, 29.098 lbm/ft³

Specific gravity, G, 0.42

Span details

Bearing length;

L_b = 4 in



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Revision:	Date:	Sheet No. 266
Project Name: JKV-EMS		

Consider Combination 1 - 1.0D + 1.0Lr (Strength)

Adjustment factors - Table 4.3.1

Ten years load duration factor - Table 2.3.2; $C_D = 1$
Size factor for bending - Table 4A; $C_{Fb} = 1.2$
Repetitive member factor - Table 4.3.9; $C_r = 1.15$

Check design at start of span

Bending members - Shear - cl.3.4

Design shear force; $V_x = 201 \text{ lb}$
Design shear stress - Table 4.3.1; $F_{v,x}' = F_v \times C_D = 135 \text{ lb/in}^2$
Actual shear stress - eq.3.4-2; $f_{v,x} = 3 \times V_x / (2 \times b \times d) = 28 \text{ lb/in}^2$
 $f_{v,x} / F_{v,x}' = 0.206$

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression; $R_x = 201 \text{ lb}$
Design bearing stress - Table 4.3.1; $F_{c\_perp,x}' = F_{c\_perp} = 425 \text{ lb/in}^2$
Actual bearing stress; $f_{c\_perp,x} = R_x / (b \times L_b) = 34 \text{ lb/in}^2$
 $f_{c\_perp,x} / F_{c\_perp,x}' = 0.079$

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Check design 1ft along span

Bending members - Flexure - cl.3.3

Design bending moment; $M_x = 200 \text{ lb\_ft}$
Design bending stress - Table 4.3.1; $F_{b,x}' = F_b \times C_D \times C_{Fb} \times C_r = 1208 \text{ lb/in}^2$
Actual bending stress - eq.3.3-2; $f_{b,x} = M_x / S_x = 183 \text{ lb/in}^2$
 $f_{b,x} / F_{b,x}' = 0.151$

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force; $V_x = 341 \text{ lb}$
Design shear stress - Table 4.3.1; $F_{v,x}' = F_v \times C_D = 135 \text{ lb/in}^2$
Actual shear stress - eq.3.4-2; $f_{v,x} = 3 \times V_x / (2 \times b \times d) = 47 \text{ lb/in}^2$
 $f_{v,x} / F_{v,x}' = 0.348$

PASS - Design shear stress exceeds actual shear stress

Check design at end of span

Bending members - Flexure - cl.3.3

Design bending moment; $M_x = 142 \text{ lb\_ft}$
Design bending stress - Table 4.3.1; $F_{b,x}' = F_b \times C_D \times C_{Fb} \times C_r = 1208 \text{ lb/in}^2$
Actual bending stress - eq.3.3-2; $f_{b,x} = M_x / S_x = 129 \text{ lb/in}^2$
 $f_{b,x} / F_{b,x}' = 0.107$

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force; $V_x = 343 \text{ lb}$
Design shear stress - Table 4.3.1; $F_{v,x}' = F_v \times C_D = 135 \text{ lb/in}^2$



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 267
Project Name: JKV-EMS		

Actual shear stress - eq.3.4-2;

$$f_{v,x} = 3 \times V_x / (2 \times b \times d) = 47 \text{ lb/in}^2$$

$$f_{v,x} / F_{v,x}' = 0.350$$

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression;

$$R_x = 686 \text{ lb}$$

Design bearing stress - Table 4.3.1;

$$F_{c\_perp,x}' = F_{c\_perp} = 425 \text{ lb/in}^2$$

Actual bearing stress;

$$f_{c\_perp,x} = R_x / (b \times L_b) = 114 \text{ lb/in}^2$$

$$f_{c\_perp,x} / F_{c\_perp,x}' = 0.269$$

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Consider Combination 5 - 1.0D + 1.0Lr (Service)

Check design 1ft along span

Bending members - Deflection - cl.3.5

Instantaneous deflection;

$$\delta_x = 0.006 \text{ in}$$

Allowable deflection;

$$\delta_{x,Allowable} = L_{m1\_s1} / 240 = 0.1 \text{ in}$$

$$\delta_x / \delta_{x,Allowable} = 0.055$$

PASS - Allowable deflection exceeds instantaneous deflection

Ledger - Span 2

Member details

Service condition;

Dry

Sawn lumber section details

Number of sections in member;

$$N = 1$$

Nominal breadth of sections;

$$b_{nom} = 2 \text{ in}$$

Breadth of sections;

$$b = 1.5 \text{ in}$$

Nominal depth of sections;

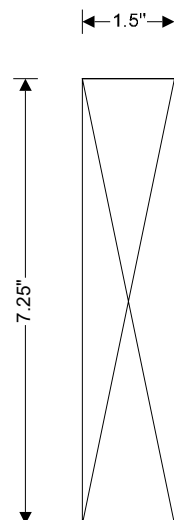
$$d_{nom} = 8 \text{ in}$$

Depth of sections;

$$d = 7.25 \text{ in}$$

Material;

Spruce-Pine-Fir, 2" & wider, No.2 grade



2"x8" sawn lumber section

Cross-sectional area, A , 10.875 in²

Section modulus, S_x , 13.1 in³

Section modulus, S_y , 2.7 in³

Second moment of area, I_x , 47.6 in⁴

Second moment of area, I_y , 2 in⁴

Radius of gyration, r_x , 2.093 in

Radius of gyration, r_y , 0.433 in

Spruce-Pine-Fir, 2" & wider, No.2 grade

Bending, F_b , 875 psi

Shear parallel to grain, F_v , 135 psi

Compression parallel to grain, F_c , 1150 psi

Compression perpendicular to grain, F_{c_perp} , 425 psi

Tension parallel to grain, F_t , 450 psi

Modulus of elasticity, E , 1400000 psi

Minimum modulus of elasticity, E_{min} , 510000 psi

Density, ρ , 29.098 lbm/ft³

Specific gravity, G , 0.42



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 268
Project Name: JKV-EMS		

Span details

Bearing length;

$$L_b = 4 \text{ in}$$

Consider Combination 1 - 1.0D + 1.0Lr (Strength)

Adjustment factors - Table 4.3.1

Ten years load duration factor - Table 2.3.2;

$$C_D = 1$$

Wet service factor for bending - Table 4A;

$$C_{Mb} = 1$$

Wet service factor for shear - Table 4A;

$$C_{Mv} = 0.97$$

Wet service factor for perp.comp. - Table 4A;

$$C_{Mc\_perp} = 0.67$$

Wet service factor for mod.of elasticity - Table 4A;

$$C_{ME} = 0.9$$

Size factor for bending - Table 4A;

$$C_{Fb} = 1.2$$

Check design at start of span

Bending members - Flexure - cl.3.3

Design bending moment;

$$M_x = 142 \text{ lb\_ft}$$

Design bending stress - Table 4.3.1;

$$F_{b,x'} = F_b \times C_D \times C_{Fb} = 1050 \text{ lb/in}^2$$

Actual bending stress - eq.3.3-2;

$$f_{b,x} = M_x / S_x = 129 \text{ lb/in}^2$$

$$f_{b,x} / F_{b,x'} = 0.123$$

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force;

$$V_x = 343 \text{ lb}$$

Design shear stress - Table 4.3.1;

$$F_{v,x'} = F_v \times C_D \times C_{Mv} = 131 \text{ lb/in}^2$$

Actual shear stress - eq.3.4-2;

$$f_{v,x} = 3 \times V_x / (2 \times b \times d) = 47 \text{ lb/in}^2$$

$$f_{v,x} / F_{v,x'} = 0.361$$

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression;

$$R_x = 686 \text{ lb}$$

Design bearing stress - Table 4.3.1;

$$F_{c\_perp,x'} = F_{c\_perp} \times C_{Mc\_perp} = 285 \text{ lb/in}^2$$

Actual bearing stress;

$$f_{c\_perp,x} = R_x / (b \times L_b) = 114 \text{ lb/in}^2$$

$$f_{c\_perp,x} / F_{c\_perp,x'} = 0.402$$

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Check design 1ft along span

Bending members - Flexure - cl.3.3

Design bending moment;

$$M_x = 200 \text{ lb\_ft}$$

Design bending stress - Table 4.3.1;

$$F_{b,x'} = F_b \times C_D \times C_{Fb} = 1050 \text{ lb/in}^2$$

Actual bending stress - eq.3.3-2;

$$f_{b,x} = M_x / S_x = 183 \text{ lb/in}^2$$

$$f_{b,x} / F_{b,x'} = 0.174$$

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force;

$$V_x = 341 \text{ lb}$$

Design shear stress - Table 4.3.1;

$$F_{v,x'} = F_v \times C_D \times C_{Mv} = 131 \text{ lb/in}^2$$

Actual shear stress - eq.3.4-2;

$$f_{v,x} = 3 \times V_x / (2 \times b \times d) = 47 \text{ lb/in}^2$$

$$f_{v,x} / F_{v,x'} = 0.359$$



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Revision:	Date:	Sheet No. 269
Project Name: JKV-EMS		

PASS - Design shear stress exceeds actual shear stress

Consider Combination 5 - 1.0D + 1.0Lr (Service)

Check design 1ft along span

Bending members - Deflection - cl.3.5

Instantaneous deflection;

$$\delta_x' = \delta_x / C_{ME} = \mathbf{0.006 \text{ in}}$$

Allowable deflection;

$$\delta_{x, \text{Allowable}} = L_{m1\_s2} / 240 = \mathbf{0.1 \text{ in}}$$

$$\delta_x' / \delta_{x, \text{Allowable}} = \mathbf{0.061}$$

PASS - Allowable deflection exceeds instantaneous deflection

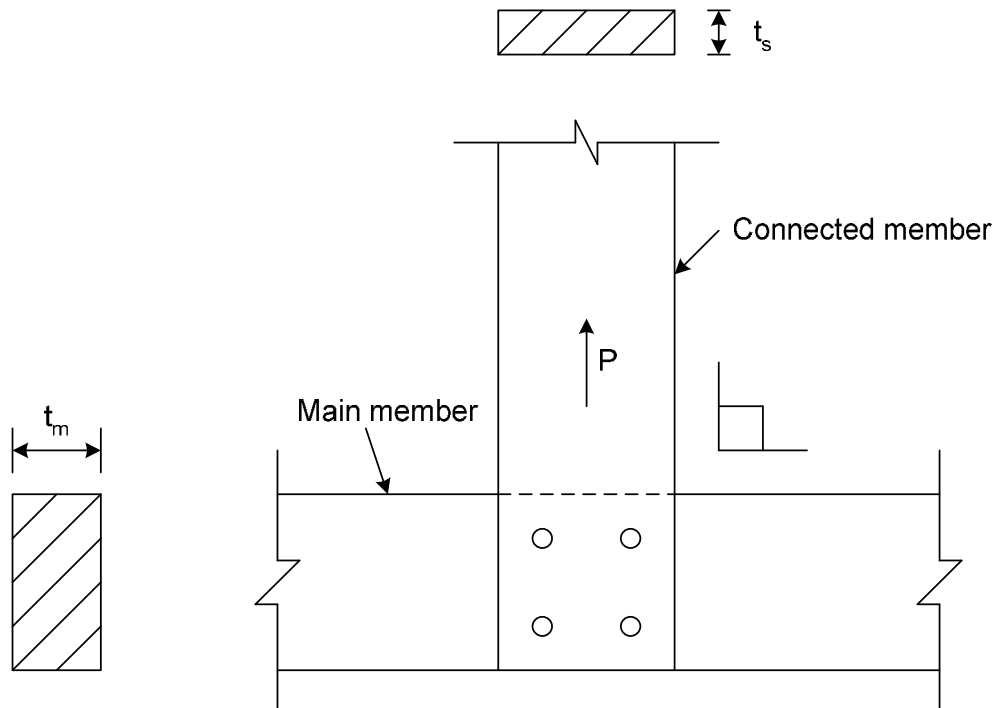
CANOPY LEDGER CONNECTION

In accordance with NDS 2018

Tedds calculation version 1.2.04

Design results summary

	Unit	Required	Provided	Utilization	Result
Connection capacity	lbs	270	369	0.731	PASS



Main timber member details

Species of main member;

Spruce-Pine-Fir

Size of main member (Table 1B);

2 x 10 ;



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Revision:	Date:	Sheet No. 270
Project Name: JKV-EMS		

Number of main member;
Thickness of main member;
Angle of load to grain of main member;

$N_m = 1$
 $t_m = 1.500$ in
 $\theta_m = 90^\circ$

Connected timber member details

Species of connected member;
Size of connected member (Table 1B);
Number of connected member;
Thickness of connected member;
Number of interfaces;

Spruce-Pine-Fir
2 x 8
 $N_s = 1$
 $t_s = 1.500$ in
 $N_{int} = (N_m + N_s) - 1 = 1$

Wood screw details

Wood screw number (Table L3);
Wood screw diameter (Table L3);
Wood screw root diameter of threaded portion;
Length of wood screw;
Number of rows of wood screw;
Number of columns of wood screw;
Total number of wood screw;

#14
 $D = 0.242$ in
 $D_r = 0.196$ in
 $L = 3.000$ in
 $R = 3$
 $C = 1$
 $N_{total} = R \times C = 3$

Applied load

Applied load to the connection;

$P = 270$ lb

Dowel bearing length (main) (12.3.5)

Actual penetration in main member;
Size of tapered tip;
Dowel bearing length in main member;

$P_{act} = L - t_s = 1.500$ in
 $E = 2 \times D = 0.484$ in
 $l_m = 1.016$ in

Dowel bearing length (connected) (12.3.5)

Dowel bearing length in connected member;

$l_s = t_s = 1.500$ in

Bending yield strength (wood screw) (Table 12L to 12M footnote no. 2)

Bending yield strength of wood screw;

$F_{yb} = 70000$ psi

Dowel bearing strength (main member) (Table 12.3.3 footnote no. 2)

Dowel bearing strength of main member;

$F_{em} = 16600 \times G_m^{1.84} \times 1$ psi = **3364** psi

Dowel bearing strength (connected timber member) (Table 12.3.3 footnote no. 2)

Dowel bearing strength of connected member;

$F_{es} = 16600 \times G_s^{1.84} \times 1$ psi = **3364** psi

Preliminary yield limit equation coefficients (Table 12.3.1A notes)

Dowel bearing strength ratio;

$R_e = F_{em} / F_{es} = 1.000$

Dowel bearing length ratio;

$R_t = l_m / l_s = 0.677$

Preliminary yield limit equation coefficient k_1 ;

$k_1 = ((\sqrt{(R_e + (2 \times R_e^2 \times (1 + R_t + R_t^2)) + (R_t^2 \times R_e^3))}) - (R_e \times (1 + R_t))) / (1 + R_e)$
 $k_1 = 0.358$

Preliminary yield limit equation coefficient k_2 ;

$k_2 = -1 + \sqrt{((2 \times (1 + R_e)) + ((2 \times F_{yb} \times (1 + (2 \times R_e)) \times D_r^2)) / (3 \times F_{em} \times l_m^2))}$
 $k_2 = 1.356$

Preliminary yield limit equation coefficient k_3 ;

$k_3 = -1 + \sqrt{(((2 \times (1 + R_e)) / R_e) + ((2 \times F_{yb} \times (2 + R_e) \times D_r^2)) / (3 \times F_{em} \times l_s^2))}$
 $k_3 = 1.170$

Angle of load to grain coefficient k_θ ;

$k_\theta = 1 + (0.25 \times \max(\theta_m, \theta_s) / 90) = 1.250$

Reduction term (Table 12.3.1B);

$K_D = 2.46$



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Revision:	Date:	Sheet No. 271
Project Name: JKV-EMS		

Yield limit equations (single shear)

Mode I_m (eq. 12.3-1);

$$Z_{Im} = (D_r \times l_m \times F_{em}) / K_D = \mathbf{272 \text{ lb}}$$

Mode I_s (eq. 12.3-2);

$$Z_{Is} = (D_r \times l_s \times F_{es}) / K_D = \mathbf{402 \text{ lb}}$$

Mode II (eq. 12.3-3);

$$Z_{II} = (k_1 \times D_r \times l_s \times F_{es}) / K_D = \mathbf{144 \text{ lb}}$$

Mode III_m (eq. 12.3-4);

$$Z_{III_m} = (k_2 \times D_r \times l_m \times F_{em}) / ((1 + (2 \times R_e)) \times K_D) = \mathbf{123 \text{ lb}}$$

Mode III_s (eq. 12.3-5);

$$Z_{III_s} = (k_3 \times D_r \times l_s \times F_{em}) / ((2 + R_e) \times K_D) = \mathbf{157 \text{ lb}}$$

Mode IV (eq. 12.3-6);

$$Z_{IV} = D_r^2 \times (\sqrt{((2 \times F_{em} \times F_{yb}) / (3 \times (1 + R_e))))} / K_D = \mathbf{138 \text{ lb}}$$

$$Z = \min (Z_{Im}, Z_{Is}, Z_{II}, Z_{III_m}, Z_{III_s}, Z_{IV}) = \mathbf{123 \text{ lb}}$$

Nominal capacity of single fastener;

$$Z = \mathbf{123 \text{ lb}}$$

Spacing requirements: wood connected member (Table C12.1.5.7)

(not prebored holes)

Edge distance

Edge distance;

$$e = 2.5 \times D = \mathbf{0.605 \text{ in}}$$

End distance

Members in tension;

$$a_{\text{tension}} = 15 \times D = \mathbf{3.630 \text{ in}}$$

Member in compression;

$$a_{\text{comp}} = 10 \times D = \mathbf{2.420 \text{ in}}$$

Center to center spacing

Center to center spacing (parallel to grain);

$$s_p = 15 \times D = \mathbf{3.630 \text{ in}}$$

Center to center spacing (perpendicular to grain);

$$s_q = 10 \times D = \mathbf{2.420 \text{ in}}$$

Row spacing

Row spacing (in line);

$$s_{\text{row\_in line}} = 5 \times D = \mathbf{1.210 \text{ in}}$$

Row spacing (staggered);

$$s_{\text{row\_staggered}} = 2.5 \times D = \mathbf{0.605 \text{ in}}$$

Geometry factor C_Δ (12.5.1)

Geometry factor (for D < ¼ in);

$$C_{\Delta} = \mathbf{1.00}$$

Adjustment factor

Load duration factor (Table 2.3.2);

$$C_D = \mathbf{1.00}$$

Wet service factor (Table 11.3.3);

$$C_M = \mathbf{1.0}$$

Temperature factor (Table 11.3.4);

$$C_t = \mathbf{1.0}$$

Group action factor (eq. 11.3-1);

$$C_g = \mathbf{1.0}$$

Geometry factor (12.5.1);

$$C_{\Delta} = \mathbf{1.00}$$

End grain factor (12.5.2);

$$C_{eg} = \mathbf{1.0}$$

Diaphragm factor (12.5.3);

$$C_{di} = \mathbf{1.0}$$

Toe nail factor (12.5.4);

$$C_{tn} = \mathbf{1.0}$$

Total capacity of connection

Capacity of connection;

$$Z' = Z \times N_{\text{total}} \times N_{\text{int}} \times C_D \times C_M \times C_{\Delta} = \mathbf{369 \text{ lb}}$$

$$P / Z' = \mathbf{0.731}$$

Design result

PASS - Connection capacity exceeds applied load



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Revision:	Date:	Sheet No. 272
Project Name: JKV-EMS		

CANOPY IN WALL GIRDER COLUMN

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

Tedds calculation version 2.2.24

Design summary

Overall design utilisation; 0.262
Overall design status; PASS

Design section s1 results summary	Unit	Capacity	Maximum	Utilization	Result
Bending stress	lb/in ²	1138	298	0.262	PASS
Compressive stress	lb/in ²	704	0	0.000	PASS
Bending and axial force				0.262	PASS

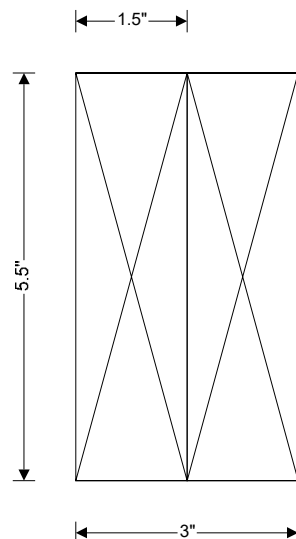
Design section 1

Member details

Service condition; Dry

Sawn lumber section details

Number of sections in member; N = 2
Nominal breadth of sections; b_{nom} = 2 in
Breadth of sections; b = 1.5 in
Nominal depth of sections; d_{nom} = 6 in
Depth of sections; d = 5.5 in
Material; Spruce-Pine-Fir, 2" & wider, No.2 grade



2/2"x6" sawn lumber sections

Cross-sectional area, A, 16.5 in²

Section modulus, S_x, 15.1 in³

Section modulus, S_y, 8.3 in³

Second moment of area, I_x, 41.6 in⁴

Second moment of area, I_y, 12.4 in⁴

Radius of gyration, r_x, 1.588 in

Radius of gyration, r_y, 0.866 in

Spruce-Pine-Fir, 2" & wider, No.2 grade

Bending, F_b, 875 psi

Shear parallel to grain, F_v, 135 psi

Compression parallel to grain, F_c, 1150 psi

Compression perpendicular to grain, F_{c,perp}, 425 psi

Tension parallel to grain, F_t, 450 psi

Modulus of elasticity, E, 1400000 psi

Minimum modulus of elasticity, E_{min}, 510000 psi

Density, ρ, 29.098 lbm/ft³

Specific gravity, G, 0.42



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 273
Project Name: JKV-EMS		

Span details

Unbraced length - Major axis; $L_x = 10$ ft
Effective bending length - Major axis; $L_{e,x} = 1.63 \times L_x + 3 \times N \times b = 17.05$ ft
Column buckling length - Major axis; $L_{b,x} = L_x = 10$ ft
Unbraced length - Minor axis; $L_y = 0$ ft

Analysis results

Design bending moment - Major axis; $M_x = 375$ lb_ft
Design axial compression force; $P = 3$ lb

Adjustment factors - Table 4.3.1

Ten years load duration factor - Table 2.3.2; $C_D = 1$
Size factor for bending - Table 4A; $C_{Fb} = 1.3$
Size factor for compression - Table 4A; $C_{Fc} = 1.1$
Reference compression design value; $F_c^* = F_c \times C_D \times C_{Fc} = 1265$ lb/in²
Adjusted modulus of elasticity; $E_{min}' = E_{min} = 510000$ lb/in²
Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (L_{b,x} / d)^2 = 881$ lb/in²
Column stability factor - eq.3.7-1

$$C_P = (1 + (F_{cE} / F_c^*)) / 1.6 - \sqrt{((1 + (F_{cE} / F_c^*)) / 1.6)^2 - (F_{cE} / F_c^*) / 0.8} = 0.557$$

Compression members - General - cl.3.6

Design axial compression force; $P = 3$ lb
Design compression parallel to grain - Table 4.3.1; $F_c' = F_c \times C_D \times C_{Fc} \times C_P = 704$ lb/in²
Actual compression parallel to grain; $f_c = P / (N \times b \times d) = 0$ lb/in²
 $f_c / F_c' = 0.000$

PASS - Design compression stress exceeds actual compression stress

Bending members - Flexure - cl.3.3

Design bending moment; $M_x = 375$ lb_ft
Design bending stress - Table 4.3.1; $F_{b,x}' = F_b \times C_D \times C_{Fb} = 1138$ lb/in²
Actual bending stress - eq.3.3-2; $f_{b,x} = M_x / S_x = 298$ lb/in²
 $f_{b,x} / F_{b,x}' = 0.262$

PASS - Design bending stress exceeds actual bending stress

Combined bending and axial loading - cl.3.9

Critical buckling design value in x-axis; $F_{cE1} = 0.822 \times E_{min}' / (L_{b,x} / d)^2 = 881$ lb/in²
Critical buckling design value in y-axis; $F_{cE2} = 0.822 \times E_{min}' = 419220$ lb/in²
Bending and compression check - eqs.3.9-3 and 3.9-4

$$\max((f_c / F_c')^2 + f_{b,x} / (F_{b,x}' \times (1 - (f_c / F_{cE1}))), (f_c / F_{cE2})) = 0.262; < 1.0$$

PASS - Combined bending and compressive stresses are within permissible limits



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Job No. 25-003

Revision:

Date:

Sheet No. 274

Project Name: JKV-EMS

CANOPY COLUMN

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

Tedds calculation version 2.2.24

Design summary

Overall design utilisation; 0.516
Overall design status; PASS

Design section s1 results summary	Unit	Capacity	Maximum	Utilization	Result
Bending stress	lb/in ²	499	81	0.163	PASS
Shear stress	lb/in ²	125	4	0.030	PASS
Compressive stress	lb/in ²	384	198	0.516	PASS
Bending and axial force				0.503	PASS

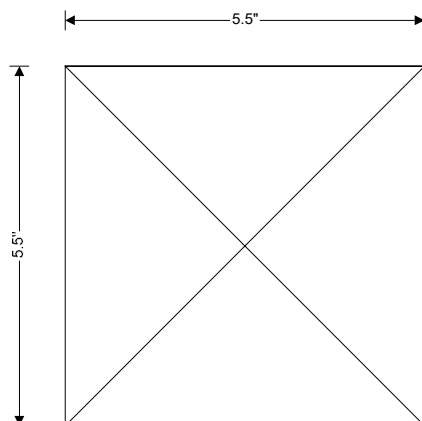
Design section 1

Member details

Service condition; Dry

Sawn lumber section details

Number of sections in member; N = 1
Nominal breadth of sections; b_{nom} = 6 in
Breadth of sections; b = 5.5 in
Nominal depth of sections; d_{nom} = 6 in
Depth of sections; d = 5.5 in
Material; Spruce-Pine-Fir, Posts and timbers, No.2 grade



6"x6" sawn lumber section

Cross-sectional area, A, 30.25 in²

Section modulus, S_x, 27.7 in³

Section modulus, S_y, 27.7 in³

Second moment of area, I_x, 76.3 in⁴

Second moment of area, I_y, 76.3 in⁴

Radius of gyration, r_x, 1.588 in

Radius of gyration, r_y, 1.588 in

Spruce-Pine-Fir, Posts and timbers, No.2 grade

Bending, F_b, 500 psi

Shear parallel to grain, F_v, 125 psi

Compression parallel to grain, F_c, 500 psi

Compression perpendicular to grain, F_{c_perp}, 425 psi

Tension parallel to grain, F_t, 325 psi

Modulus of elasticity, E, 1000000 psi

Minimum modulus of elasticity, E_{min}, 370000 psi

Density, ρ, 29.098 lbm/ft³

Specific gravity, G, 0.42



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 275
Project Name: JKV-EMS		

Span details

Unbraced length - Major axis;

$$L_x = 10 \text{ ft}$$

Effective bending length - Major axis;

$$L_{e,x} = 1.63 \times L_x + 3 \times b = 17.675 \text{ ft}$$

Column buckling length - Major axis;

$$L_{b,x} = L_x = 10 \text{ ft}$$

Unbraced length - Minor axis;

$$L_y = 10 \text{ ft}$$

Effective bending length - Minor axis;

$$L_{e,y} = 1.63 \times L_y + 3 \times d = 17.675 \text{ ft}$$

Column buckling length - Minor axis;

$$L_{b,y} = L_y = 10 \text{ ft}$$

Bearing length;

$$L_b = 4 \text{ in}$$

Analysis results

Design bending moment - Major axis;

$$M_x = 188 \text{ lb\_ft}$$

Design shear force - Major axis;

$$V_x = 75 \text{ lb}$$

Design axial compression force;

$$P = 6000 \text{ lb}$$

Adjustment factors - Table 4.3.1

Ten years load duration factor - Table 2.3.2;

$$C_D = 1$$

Depth-to-breadth ratio;

$$d_{nom} / b_{nom} = 1.00$$

Slenderness ratio - eq.3.3-5;

$$R_B = \sqrt{(L_{e,y} \times d / b^2)} = 6.210$$

Reference bending design value;

$$F_b^* = F_b \times C_D = 500 \text{ lb/in}^2$$

Adjusted modulus of elasticity;

$$E_{min}' = E_{min} = 370000 \text{ lb/in}^2$$

Critical buckling design value - cl.3.3.3.8;

$$F_{bE} = 1.2 \times E_{min}' / R_B^2 = 11513 \text{ lb/in}^2$$

Beam stability factor - eq.3.3-6

$$C_L = [1 + (F_{bE} / F_b^*)] / 1.9 - \sqrt{([1 + (F_{bE} / F_b^*)] / 1.9)^2 - (F_{bE} / F_b^*) / 0.95} = 0.998$$

Reference compression design value;

$$F_c^* = F_c \times C_D = 500 \text{ lb/in}^2$$

Adjusted modulus of elasticity;

$$E_{min}' = E_{min} = 370000 \text{ lb/in}^2$$

Critical buckling design value;

$$F_{cE} = 0.822 \times E_{min}' / (L_{b,y} / b)^2 = 639 \text{ lb/in}^2$$

Column stability factor - eq.3.7-1

$$C_P = (1 + (F_{cE} / F_c^*)) / 1.6 - \sqrt{((1 + (F_{cE} / F_c^*)) / 1.6)^2 - (F_{cE} / F_c^*) / 0.8} = 0.768$$

Compression members - General - cl.3.6

Design axial compression force;

$$P = 6000 \text{ lb}$$

Design compression parallel to grain - Table 4.3.1;

$$F_c' = F_c \times C_D \times C_P = 384 \text{ lb/in}^2$$

Actual compression parallel to grain;

$$f_c = P / (b \times d) = 198 \text{ lb/in}^2$$

$$f_c / F_c' = 0.516$$

PASS - Design compression stress exceeds actual compression stress

Bending members - Flexure - cl.3.3

Design bending moment;

$$M_x = 188 \text{ lb\_ft}$$

Design bending stress - Table 4.3.1;

$$F_{b,x}' = F_b \times C_D \times C_L = 499 \text{ lb/in}^2$$

Actual bending stress - eq.3.3-2;

$$f_{b,x} = M_x / S_x = 81 \text{ lb/in}^2$$

$$f_{b,x} / F_{b,x}' = 0.163$$

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force;

$$V_x = 75 \text{ lb}$$

Design shear stress - Table 4.3.1;

$$F_{v,x}' = F_v \times C_D = 125 \text{ lb/in}^2$$

Actual shear stress - eq.3.4-2;

$$f_{v,x} = 3 \times V_x / (2 \times b \times d) = 4 \text{ lb/in}^2$$



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 276
Project Name: JKV-EMS		

$$f_{v,x} / F_{v,x}' = 0.030$$

PASS - Design shear stress exceeds actual shear stress

Combined bending and axial loading - cl.3.9

Critical buckling design value in x-axis;

$$F_{cE1} = 0.822 \times E_{min}' / (L_{b,x} / d)^2 = 639 \text{ lb/in}^2$$

Critical buckling design value in y-axis;

$$F_{cE2} = 0.822 \times E_{min}' / (L_{b,y} / b)^2 = 639 \text{ lb/in}^2$$

Bending and compression check - eqs.3.9-3 and 3.9-4

$$\max((f_c / F_c')^2 + f_{b,x} / (F_{b,x}' \times (1 - (f_c / F_{cE1}))), (f_c / F_{cE2})) = 0.503; < 1.0$$

PASS - Combined bending and compressive stresses are within permissible limits

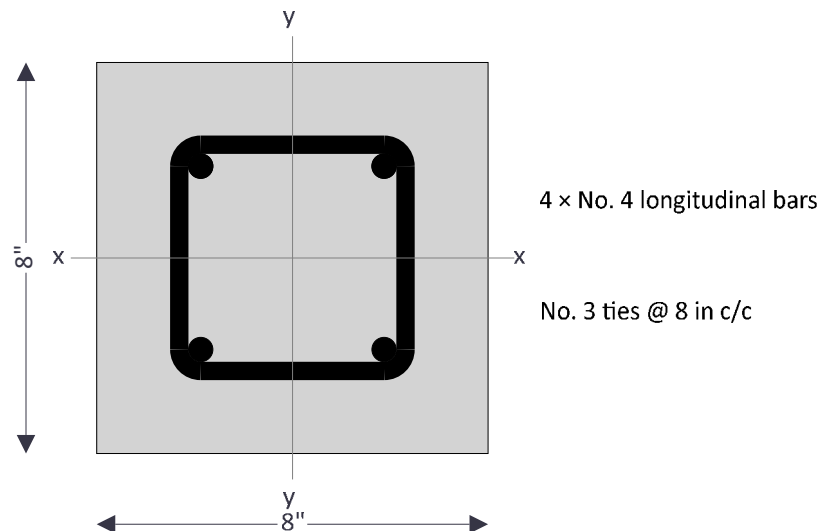
CANOPY CONCRETE COLUMN

In accordance with ACI318-14

Tedds calculation version 2.2.07

Design summary

Description	Unit	Capacity	Applied	Utilization	Result
For ultimate axial load 11.2 kips:					PASS
Bending x axis	kip_ft	12.6	1.3	0.103	PASS



Applied loads

Ultimate axial force acting on column;

$$P_{u,act} = 11.2 \text{ kips}$$

Ultimate moment about major (X) axis;

$$M_{ux,act} = 1.3 \text{ kips\_ft}$$

Geometry of column

Depth of column (larger dimension of column);

$$h = 8.0 \text{ in}$$

Width of column (smaller dimension of column);

$$b = 8.0 \text{ in}$$

Clear cover to reinforcement (both sides);

$$c_c = 1.5 \text{ in}$$

Unsupported height of column about x axis;

$$l_{ux} = 4.3 \text{ ft}$$



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 277
Project Name: JKV-EMS		

Effective height factor about x axis;
Column state about the x axis;
Unsupported height of column about y axis;
Effective height factor about y axis;
Column state about the y axis;

$k_x = 1.00$
Braced
 $l_{uy} = 4.3$ ft
 $k_y = 1.00$
Braced

Check on overall column dimensions

Column dimensions are OK - $h < 4b$

Reinforcement of column

Numbers of bars of longitudinal steel;
Longitudinal steel bar diameter number;
Diameter of longitudinal bar;
Stirrup bar diameter number;
Diameter of stirrup bar;
Specified yield strength of reinforcement;
Specified compressive strength of concrete;
Modulus of elasticity of bar reinforcement;
Modulus of elasticity of concrete;
Yield strain;
Ultimate design strain;

$N = 4$
 $D_{bar\_num} = 4$
 $D_{long} = 0.500$ in
 $D_{stir\_num} = 3$
 $D_{stir} = 0.375$ in
 $f_y = 60000$ psi
 $f'_c = 4000$ psi
 $E_s = 29 \times 10^6$ psi
 $E_c = 57000 \times f'_c{}^{1/2} \times (1\text{psi})^{1/2} = 3604997$ psi
 $\epsilon_y = f_y / E_s = 0.00207$
 $\epsilon_c = 0.003$ in/in

Check for minimum area of steel - 10.6.1.1

Gross area of column;
Area of steel;
Minimum area of steel required;

$A_g = h \times b = 64.000$ in²
 $A_{st} = N \times (\pi \times D_{long}^2) / 4 = 0.785$ in²
 $A_{st\_min} = 0.01 \times A_g = 0.640$ in²

$A_{st} > A_{st\_min}$, PASS - Minimum steel check

Check for maximum area of steel - 10.6.1.1

Permissible maximum area of steel;

$A_{st\_max} = 0.08 \times A_g = 5.120$ in²

$A_{st} < A_{st\_max}$, PASS - Maximum steel check

Design of column ties - 25.7.2

Spacing of lateral ties;
16 times longitudinal bar diameter;
48 times tie bar diameter;
Least column dimension;
Required tie spacing ;

$s_{v\_ties} = 8.000$ in
 $s_{v1} = 16 \times D_{long} = 8.000$ in
 $s_{v2} = 48 \times D_{stir} = 18.000$ in
 $s_{v3} = \min(h, b) = 8.000$ in
 $s = \min(s_{v1}, s_{v2}, s_{v3}) = 8.000$ in

$s_{v\_ties} < s$ PASS

Slenderness check about x axis

Radius of gyration;
Actual slenderness ratio;

$r_x = 0.3 \times h = 2.4$ in
 $s_{rx\_act} = k_x \times l_{ux} / r_x = 21.25$

Slenderness ratio is less than 22, slenderness effects may be neglected

Slenderness check about y axis

Radius of gyration;
Actual slenderness ratio;

$r_y = 0.3 \times b = 2.4$ in
 $s_{ry\_act} = k_y \times l_{uy} / r_y = 21.25$

Slenderness ratio is less than 22, slenderness effects may be neglected



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 278
Project Name: JKV-EMS		

Axial load capacity of axially loaded column

Strength reduction factor; $\phi = 0.65$
Area of steel on compression face; $A'_s = A_{st} / 2 = \mathbf{0.393 \text{ in}^2}$
Area of steel on tension face; $A_s = A_{st} / 2 = \mathbf{0.393 \text{ in}^2}$
Net axial load capacity of column; $P_n = 0.8 \times (0.85 \times f'_c \times (A_g - A_{st}) + f_y \times A_{st}) = \mathbf{209.643 \text{ kips}}$
Ultimate axial load capacity of column; $P_u = \phi \times P_n = \mathbf{136.268 \text{ kips}}$

PASS : Column is safe in axial loading

Uniaxially loaded column about major axis

Details of column cross-section

c/d_t ratio; $r_{xb} = \mathbf{0.309}$
Effective cover to reinforcement; $d' = c_c + D_{stir} + (D_{long}/2) = \mathbf{2.125 \text{ in}}$
Spacing between bars; $s = ((h - (2 \times d')) / ((N_y) - 1)) = \mathbf{3.750 \text{ in}}$
Depth of tension steel; $d_t = h - d' = \mathbf{5.875 \text{ in}}$
Depth of NA from extreme compression face; $c_x = r_{xb} \times d_t = \mathbf{1.812 \text{ in}}$
Factor of depth of compressive stress block; $\beta_1 = \mathbf{0.850}$
Depth of equivalent rectangular stress block; $a_x = \min((\beta_1 \times c_x), h) = \mathbf{1.541 \text{ in}}$
Yield strain in steel; $\epsilon_{sx} = f_y / E_s = \mathbf{0.002}$
Strength reduction factor; $\phi_x = \mathbf{0.900}$

Details of concrete block

Force carried by concrete

Forces carried by concrete; $P_{xcon} = 0.85 \times f'_c \times b \times a_x = \mathbf{41.905 \text{ kips}}$

Moment carried by concrete

Moment carried by concrete; $M_{xcon} = P_{xcon} \times ((h/2) - (a_x/2)) = \mathbf{11.278 \text{ kip\_ft}}$

Details of steel layer 1

Depth of layer; $x_{x1} = \mathbf{2.125 \text{ in}}$
Strain of layer; $\epsilon_{x1} = \epsilon_c \times (1 - x_{x1} / c_x) = \mathbf{-0.00052}$
Stress in layer; $\sigma_{x1} = \max(-1 \times f_y, E_s \times \epsilon_{x1}) = \mathbf{-15000.98 \text{ psi}}$
Force carried by layer; $P_{x1} = N_x \times A_{bar} \times \sigma_{x1} = \mathbf{-5.891 \text{ kips}}$
Moment carried by steel layer; $M_{x1} = P_{x1} \times ((h/2) - x_{x1}) = \mathbf{-0.920 \text{ kip\_ft}}$

Details of steel layer 2

Depth of layer; $x_{x2} = \mathbf{5.875 \text{ in}}$
Strain of layer; $\epsilon_{x2} = \epsilon_c \times (1 - x_{x2} / c_x) = \mathbf{-0.00672}$
Stress in layer; $\sigma_{x2} = \max(-1 \times f_y, E_s \times \epsilon_{x2}) = \mathbf{-60000.00 \text{ psi}}$
Force carried by layer; $P_{x2} = N_x \times A_{bar} \times \sigma_{x2} = \mathbf{-23.562 \text{ kips}}$
Moment carried by steel layer; $M_{x2} = P_{x2} \times ((h/2) - x_{x2}) = \mathbf{3.682 \text{ kip\_ft}}$

Force carried by steel

Sum of forces by steel; $P_{xs} = \mathbf{-29.5 \text{ kips}}$

Total force carried by column

Nominal axial load strength; $P_{nx} = \mathbf{12.452 \text{ kips}}$
Strength reduction factor; $\phi_x = \mathbf{0.900}$
Ultimate axial load carrying capacity of column; $P_{ux} = \phi_x \times P_{nx} = \mathbf{11.207 \text{ kips}}$



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 279
Project Name: JKV-EMS		

Total moment carried by column

Total moment carried by column; $M_{ox} = 14.039$ kip_ft;
Ultimate moment strength capacity of column; $M_{ux} = \phi_x \times M_{ox} = 12.635$ kip_ft

Check load capacity for uniaxial loads about the x axis

Factored axial load; $P_{u\_act} = 11.2$ kips
Ultimate axial capacity; $P_{ux} = 11.2$ kips

PASS - Ultimate axial capacity exceeds factored axial load

Factored moment about x axis; $M_{ux\_act} = 1.3$ kip_ft
Ultimate moment capacity about the x axis; $M_{ux} = 12.6$ kip_ft

PASS - Ultimate moment capacity exceeds factored moment about x axis

;

CANOPY FOOTING

In accordance with ACI318-14

Tedds calculation version 3.3.11

Summary results

Overall design status; PASS;
Overall design utilisation; 0.928

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	10.7			Pass
Overturning stability, x	kip_ft	0.90	-16.09	17.88	Pass
Description	Unit	Applied	Resisting	Utilization	Result
Soil bearing	ksf	1.392	1.5	0.928	Pass
Description	Unit	Required	Provided	Utilization	Result
Moment, positive, x-direction	kip_ft	2.9	159.9	0.018	Pass
Moment, positive, y-direction	kip_ft	2.4	154.0	0.016	Pass
Shear, two-way, Col 1	psi	1.775	189.737	0.009	Pass
Min.area of reinf, bot., x-direction	in ²	1.555	1.760		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	9.7		Pass
Min.area of reinf, bot., y-direction	in ²	1.555	1.760		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	9.7		Pass

Pad footing details

Length of footing; $L_x = 3$ ft
Width of footing; $L_y = 3$ ft
Footing area; $A = L_x \times L_y = 9$ ft²
Depth of footing; $h = 24$ in
Depth of soil over footing; $h_{soil} = 12$ in
Density of concrete; $\gamma_{conc} = 150.0$ lb/ft³



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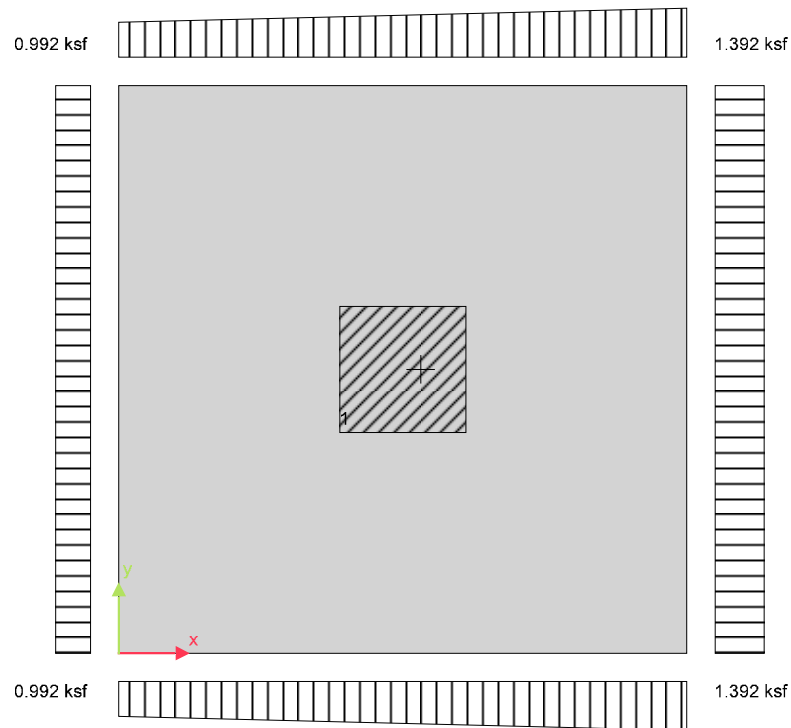
Job No. 25-003

Revision:

Date:

Sheet No. 280

Project Name: JKV-EMS



Column no.1 details

Length of column;

$$l_{x1} = 8.00 \text{ in}$$

Width of column;

$$l_{y1} = 8.00 \text{ in}$$

position in x-axis;

$$x_1 = 18.00 \text{ in}$$

position in y-axis;

$$y_1 = 18.00 \text{ in}$$

Soil properties

Gross allowable bearing pressure;

$$Q_{\text{allow\_Gross}} = 1.5 \text{ ksf};$$

Density of soil;

$$\gamma_{\text{soil}} = 120.0 \text{ lb/ft}^3$$

Angle of internal friction;

$$\phi_b = 30.0 \text{ deg}$$

Design base friction angle;

$$\delta_{bb} = 30.0 \text{ deg}$$

Coefficient of base friction;

$$\tan(\delta_{bb}) = 0.577$$

Footing loads

Self weight;

$$F_{\text{swt}} = h \times \gamma_{\text{conc}} = 300 \text{ psf}$$

Soil weight;

$$F_{\text{soil}} = h_{\text{soil}} \times \gamma_{\text{soil}} = 120 \text{ psf}$$

Column no.1 loads

Dead load in z;

$$F_{Dz1} = 1.0 \text{ kips}$$

Live load in z;

$$F_{Lz1} = 6.0 \text{ kips}$$

Live load moment in x;

$$M_{Lx1} = 0.9 \text{ kip\_ft}$$

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.350)



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 281
Project Name: JKV-EMS		

1.0D + 1.0L (0.928)

Combination 2 results: 1.0D + 1.0L

Forces on footing

Force in z-axis;

$$F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} = 10.7 \text{ kips}$$

Moments on footing

Moment in x-axis, about x is 0;

$$M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1 + M_{Lx1}) = 17.0 \text{ kip\_ft}$$

Moment in y-axis, about y is 0;

$$M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) = 16.1 \text{ kip\_ft}$$

Uplift verification

Vertical force;

$$F_{dz} = 10.727 \text{ kips}$$

PASS - Footing is not subject to uplift

Stability against overturning in x direction, moment about x is L_x

Overturning moment;

$$M_{OTxL} = \gamma_L \times (M_{Lx1}) = 0.9 \text{ kip\_ft}$$

Resisting moment;

$$M_{RXL} = -1 \times (\gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2)) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times (x_1 - L_x)) + \gamma_L \times (F_{Lz1} \times (x_1 - L_x)) = -16.09 \text{ kip\_ft}$$

Factor of safety;

$$\text{abs}(M_{RXL} / M_{OTxL}) = 17.878$$

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis;

$$e_{dx} = M_{dx} / F_{dz} - L_x / 2 = 1.007 \text{ in}$$

Eccentricity of base reaction in y-axis;

$$e_{dy} = M_{dy} / F_{dz} - L_y / 2 = 0 \text{ in}$$

Pad base pressures

$$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = 0.992 \text{ ksf}$$

$$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = 0.992 \text{ ksf}$$

$$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = 1.392 \text{ ksf}$$

$$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = 1.392 \text{ ksf}$$

Minimum base pressure;

$$q_{\min} = \min(q_1, q_2, q_3, q_4) = 0.992 \text{ ksf}$$

Maximum base pressure;

$$q_{\max} = \max(q_1, q_2, q_3, q_4) = 1.392 \text{ ksf}$$

Allowable bearing capacity

Allowable bearing capacity;

$$q_{\text{allow}} = q_{\text{allow\_Gross}} = 1.5 \text{ ksf}$$

$$q_{\max} / q_{\text{allow}} = 0.928$$

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN

In accordance with ACI318-14

Tedds calculation version 3.3.11

Material details

Compressive strength of concrete;

$$f'_c = 4000 \text{ psi}$$

Yield strength of reinforcement;

$$f_y = 60000 \text{ psi}$$

Compression-controlled strain limit (21.2.2);

$$\epsilon_{ty} = 0.00200$$



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 282
Project Name: JKV-EMS		

Cover to top of footing;
Cover to side of footing;
Cover to bottom of footing;
Concrete type;
Concrete modification factor;
Column type;

$C_{nom\_t} = 3$ in
 $C_{nom\_s} = 3$ in
 $C_{nom\_b} = 3$ in
Normal weight
 $\lambda = 1.00$
Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.002)
1.2D + 1.6L + 0.5Lr (0.018)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on footing

Ultimate force in z-axis;

$$F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} = 15.3 \text{ kips}$$

Moments on footing

Ultimate moment in x-axis, about x is 0;

$$M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1 + M_{Lx1}) = 24.3 \text{ kip\_ft}$$

Ultimate moment in y-axis, about y is 0;

$$M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) = 22.9 \text{ kip\_ft}$$

Eccentricity of base reaction

Eccentricity of base reaction in x-axis;

$$e_{ux} = M_{ux} / F_{uz} - L_x / 2 = 1.131 \text{ in}$$

Eccentricity of base reaction in y-axis;

$$e_{uy} = M_{uy} / F_{uz} - L_y / 2 = 0 \text{ in}$$

Pad base pressures

$$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = 1.377 \text{ ksf}$$

$$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = 1.377 \text{ ksf}$$

$$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = 2.017 \text{ ksf}$$

$$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = 2.017 \text{ ksf}$$

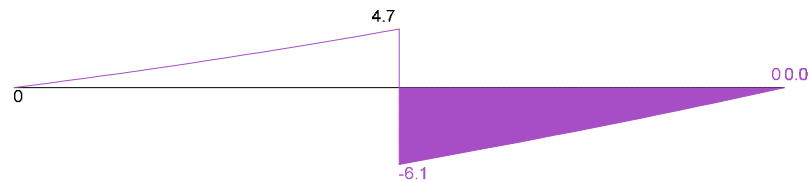
Minimum ultimate base pressure;

$$q_{umin} = \min(q_{u1}, q_{u2}, q_{u3}, q_{u4}) = 1.377 \text{ ksf}$$

Maximum ultimate base pressure;

$$q_{umax} = \max(q_{u1}, q_{u2}, q_{u3}, q_{u4}) = 2.017 \text{ ksf}$$

Shear diagram, x axis (kips)

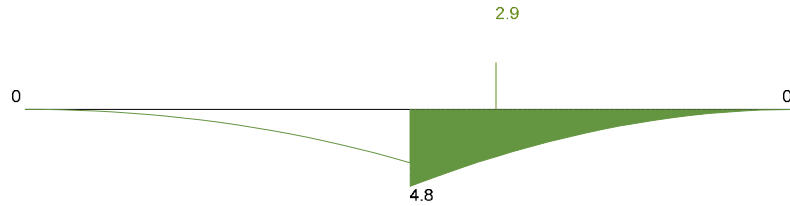




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Revision:	Date:	Sheet No. 283
Project Name: JKV-EMS		

Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment;

$$M_{u,x,max} = 2.919 \text{ kip\_ft}$$

Tension reinforcement provided;

$$4 \text{ No.6 bottom bars (9.7 in c/c)}$$

Area of tension reinforcement provided;

$$A_{sx,bot,prov} = 1.76 \text{ in}^2$$

Minimum area of reinforcement (8.6.1.1);

$$A_{s,min} = 0.0018 \times L_y \times h = 1.555 \text{ in}^2$$

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2);

$$s_{max} = \min(2 \times h, 18 \text{ in}) = 18 \text{ in}$$

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement;

$$d = h - C_{nom,b} - \phi_{x,bot} / 2 = 20.625 \text{ in}$$

Depth of compression block;

$$a = A_{sx,bot,prov} \times f_y / (0.85 \times f'_c \times L_y) = 0.863 \text{ in}$$

Neutral axis factor;

$$\beta_1 = 0.85$$

Depth to neutral axis;

$$c = a / \beta_1 = 1.015 \text{ in}$$

Strain in tensile reinforcement;

$$\epsilon_t = 0.003 \times d / c - 0.003 = 0.05796$$

Minimum tensile strain(8.3.3.1);

$$\epsilon_{min} = 0.004 = 0.00400$$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity;

$$M_n = A_{sx,bot,prov} \times f_y \times (d - a / 2) = 177.704 \text{ kip\_ft}$$

Flexural strength reduction factor;

$$\phi_f = \min(\max(0.65 + 0.25 \times (\epsilon_t - \epsilon_{ty}) / (0.005 - \epsilon_{ty}), 0.65), 0.9) = 0.900$$

Design moment capacity;

$$\phi M_n = \phi_f \times M_n = 159.934 \text{ kip\_ft}$$

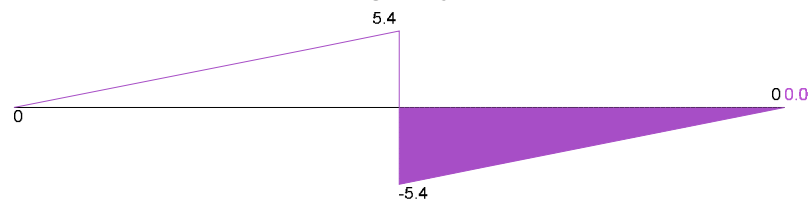
$$M_{u,x,max} / \phi M_n = 0.018$$

PASS - Design moment capacity exceeds ultimate moment load

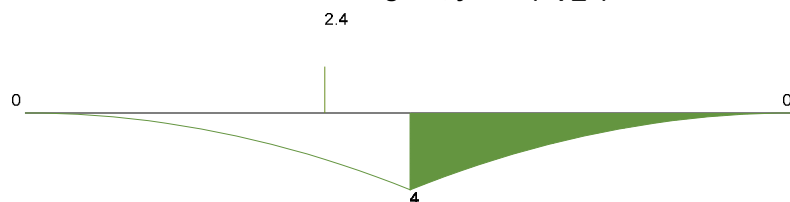
One-way shear design, x direction

One-way shear design does not apply. Shear failure plane fall outside extents of foundation.

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)





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Revision:	Date:	Sheet No. 284
Project Name: JKV-EMS		

Moment design, y direction, positive moment

Ultimate bending moment; $M_{u,y,max} = 2.437 \text{ kip\_ft}$
Tension reinforcement provided; 4 No.6 bottom bars (9.7 in c/c)
Area of tension reinforcement provided; $A_{sy,bot,prov} = 1.76 \text{ in}^2$
Minimum area of reinforcement (8.6.1.1); $A_{s,min} = 0.0018 \times L_x \times h = 1.555 \text{ in}^2$

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2); $s_{max} = \min(2 \times h, 18 \text{ in}) = 18 \text{ in}$

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_{x,bot} - \phi_{y,bot} / 2 = 19.875 \text{ in}$
Depth of compression block; $a = A_{sy,bot,prov} \times f_y / (0.85 \times f'_c \times L_x) = 0.863 \text{ in}$
Neutral axis factor; $\beta_1 = 0.85$
Depth to neutral axis; $c = a / \beta_1 = 1.015 \text{ in}$
Strain in tensile reinforcement; $\epsilon_t = 0.003 \times d / c - 0.003 = 0.05574$
Minimum tensile strain(8.3.3.1); $\epsilon_{min} = 0.004 = 0.00400$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity; $M_n = A_{sy,bot,prov} \times f_y \times (d - a / 2) = 171.104 \text{ kip\_ft}$
Flexural strength reduction factor; $\phi_f = \min(\max(0.65 + 0.25 \times (\epsilon_t - \epsilon_{ty}) / (0.005 - \epsilon_{ty}), 0.65), 0.9) = 0.900$
Design moment capacity; $\phi M_n = \phi_f \times M_n = 153.994 \text{ kip\_ft}$
 $M_{u,y,max} / \phi M_n = 0.016$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

One-way shear design does not apply. Shear failure plane fall outside extents of foundation.

Two-way shear design at column 1

Depth to reinforcement; $d_{v2} = 20.25 \text{ in}$
Shear perimeter length (22.6.4); $l_{xp} = 28.250 \text{ in}$
Shear perimeter width (22.6.4); $l_{yp} = 28.250 \text{ in}$
Shear perimeter (22.6.4); $b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2}) = 113.000 \text{ in}$
Shear area; $A_p = l_{x,perim} \times l_{y,perim} = 798.062 \text{ in}^2$
Surcharge loaded area; $A_{sur} = A_p - l_{x1} \times l_{y1} = 734.062 \text{ in}^2$
Ultimate bearing pressure at center of shear area; $q_{up,avg} = 1.697 \text{ ksf}$
Ultimate shear load; $F_{up} = \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up,avg} \times A_p = 4.061 \text{ kips}$
Ultimate shear stress from vertical load; $v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0 \text{ psi}) = 1.775 \text{ psi}$
Column geometry factor (Table 22.6.5.2); $\beta = l_{y1} / l_{x1} = 1.00$
Column location factor (22.6.5.3); $\alpha_s = 40$
Concrete shear strength (22.6.5.2); $v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} = 379.473 \text{ psi}$
 $v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} = 579.844 \text{ psi}$
 $v_{cpc} = 4 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} = 252.982 \text{ psi}$
 $v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) = 252.982 \text{ psi}$
Shear strength reduction factor; $\phi_v = 0.75$
Nominal shear stress capacity (Eq. 22.6.1.2); $v_n = v_{cp} = 252.982 \text{ psi}$
Design shear stress capacity (8.5.1.1(d)); $\phi v_n = \phi_v \times v_n = 189.737 \text{ psi}$



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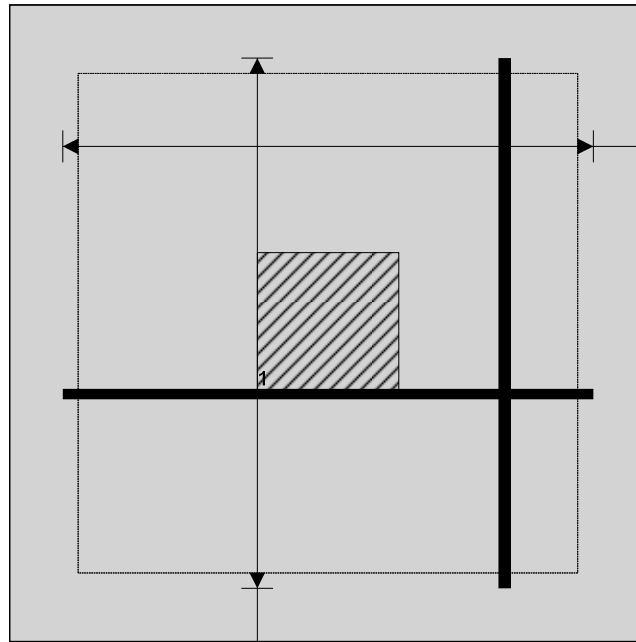
Date:

Sheet No. 285

Project Name: JKV-EMS

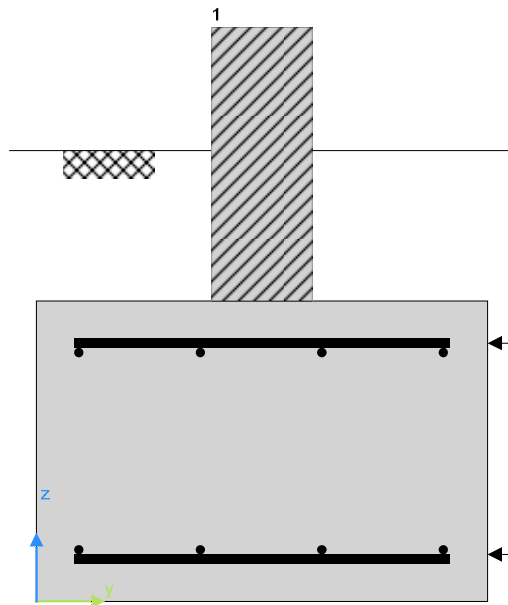
$$V_{ug} / \phi V_n = 0.009$$

PASS - Design shear stress capacity exceeds ultimate shear stress load



4 No.6 bottom bars (9.7 in c/c)
4 No.6 top bars (9.7 in c/c)

4 No.6 bottom bars (9.7 in c/c)
4 No.6 top bars (9.7 in c/c)



4 No.6 top bars (9.7 in c/c) (x direction)
4 No.6 top bars (9.7 in c/c) (y direction)

4 No.6 bottom bars (9.7 in c/c) (x direction)
4 No.6 bottom bars (9.7 in c/c) (y direction)



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Revision:	Date:	Sheet No. 286
Project Name: JKV-EMS		

NEW ROOF TRUSS TOP CHORD EXTENSION AND OUTLOOKER

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

Tedds calculation version 2.2.24

Member1 span 1 results summary	Unit	Capacity	Maximum	Utilization	Result
Bending stress	lb/in ²	1509	490	0.324	PASS
Shear stress	lb/in ²	135	48	0.353	PASS
Bearing stress	lb/in ²	425	56	0.131	PASS
Deflection	in	0.05	0.003	0.052	PASS
Member1 span 2 results summary	Unit	Capacity	Maximum	Utilization	Result
Bending stress	lb/in ²	1509	490	0.324	PASS
Shear stress	lb/in ²	135	48	0.353	PASS
Bearing stress	lb/in ²	425	56	0.131	PASS
Deflection	in	0.1	0.039	0.386	PASS

ANALYSIS

Tedds calculation version 1.0.38

Geometry

Geometry (ft) - Spruce-Pine-Fir(2" && wider(No.2))



Span	Length (ft)	Section	Start Support	End Support
1	1.5	2x4	Pinned	Roller Pin X
2	1.5	2"x4"	Roller Pin X	Free
2x4: Area 5 in ² , Inertia Major 5 in ⁴ , Inertia Minor 1 in ⁴ , Shear area parallel to Minor 4 in ² , Shear area parallel to Major 4 in ²				
2"x4": Area 5 in ² , Inertia Major 5 in ⁴ , Inertia Minor 1 in ⁴ , Shear area parallel to Minor 4 in ² , Shear area parallel to Major 4 in ²				
Spruce-Pine-Fir(2" && wider(No.2)): Density 29.1 lbm/ft ³ , Youngs 1400 ksi, Shear 87 ksi, Thermal 0 °C ⁻¹				



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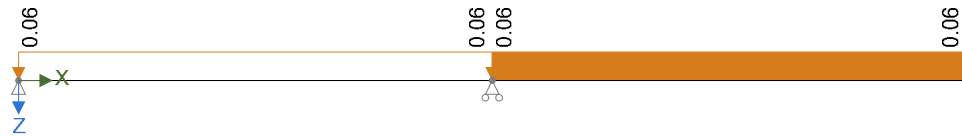
Sheet No. 287

Project Name: JKV-EMS

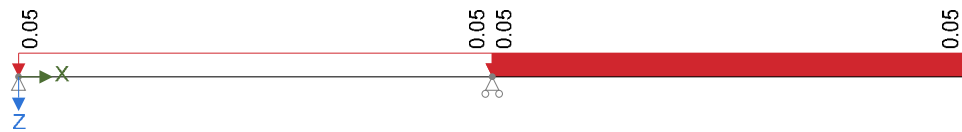
Loading

Self weight included

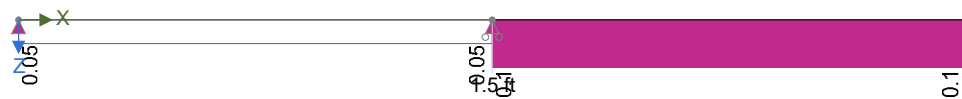
Dead - Loading (kips/ft)



Roof Live - Loading (kips/ft)



Wind - Loading (kips/ft)



Load combination factors

Load combination	Self Weight	Dead	Roof Live	Wind
1.0D + 1.0Lr (Strength)	1.0 0.0 0.0	1.0 0.0 0.0	1.0 0.0 0.0	
1.0D + 0.6W (Strength)	1.0 0.0 0.0	1.0 0.0 0.0		0.6 0.0
1.0D + 0.75L + 0.75Lr + 0.45W (Strength)	1.0 0.0 0.0	1.0 0.0 0.0	0.7 0.5	0.4 0.5
0.6D + 0.6W (Strength)	0.6 0.0 0.0	0.6 0.0 0.0		0.6 0.0
1.0D + 1.0Lr (Service)	1.0 0.0 0.0	1.0 0.0 0.0	1.0 0.0 0.0	
1.0Lr (Service)			1.0 0.0 0.0	



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Revision:	Date:	Sheet No. 288
Project Name: JKV-EMS		

Load combination	Self Weight	Dead	Roof Live	Wind
1.0W (Service)				1.000

Member Loads

Member	Load case	Load Type	Orientation	Description
Member1	Dead	UDL	GlobalZ	0.06 kips/ft
Member1	Roof Live	UDL	GlobalZ	0.05 kips/ft

Element Loads

Element	Load case	Load Type	Orientation	Description
1	Wind	UDL	GlobalZ	-0.05 kips/ft
2	Wind	UDL	GlobalZ	-0.1 kips/ft

Results

Reactions

Load case: Self Weight

Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
1	0	0	0
2	0	0.003	0

Load case: Dead

Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
1	0	0	0
2	0	0.18	0

Load case: Roof Live

Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
1	0	0	0
2	0	0.15	0



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Revision:	Date:	Sheet No. 289
Project Name: JKV-EMS		

Load case: Wind

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.038	0
2	0	-0.262	0

Load combination: 1.0D + 1.0Lr (Strength)

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0	0
2	0	0.333	0

Load combination: 1.0D + 0.6W (Strength)

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.023	0
2	0	0.026	0

Load combination: 1.0D + 0.75L + 0.75Lr + 0.45W (Strength)

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.017	0
2	0	0.178	0

Load combination: 0.6D + 0.6W (Strength)

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.023	0
2	0	-0.048	0

Load combination: 1.0D + 1.0Lr (Service)

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0	0
2	0	0.333	0



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Job No. 25-003

Revision:

Date:

Sheet No. 290

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Load combination: 1.0Lr (Service)

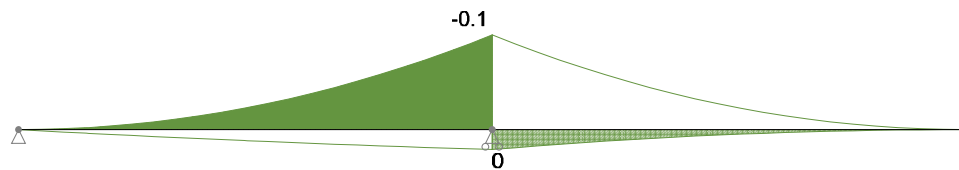
Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0	0
2	0	0.15	0

Load combination: 1.0W (Service)

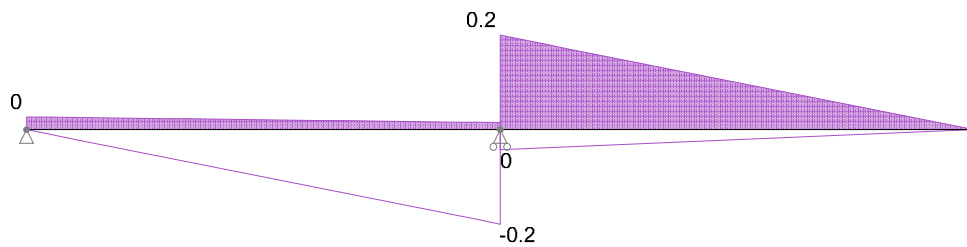
Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.038	0
2	0	-0.262	0

Forces

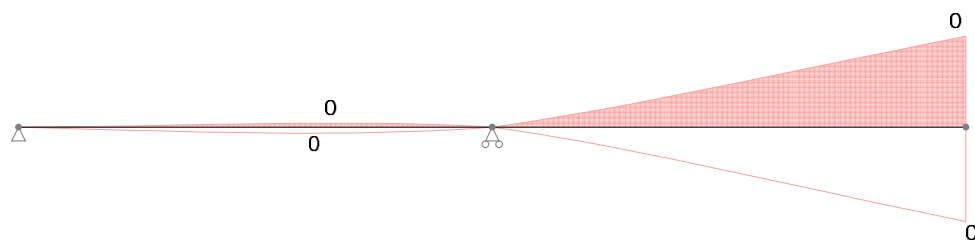
Strength combinations - Moment envelope (kip_ft)



Strength combinations - Shear envelope (kips)



Service combinations - Deflection envelope (in)





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Date: 06/16/2025

Job No. 25-003

Revision:

Date:

Sheet No. 291

Project Name: JKV-EMS

Member results

Envelope - Strength combinations

Member	Position (ft)	Shear force (kips)		Moment (kip_ft)	
Member1	1.5	0.167 (max abs)	-0.167	0.026 (max)	-0.125 (min)

Envelope - Service combinations

Member	Position (ft)	Deflection (in)	
Member1	3	0.04 (max)	-0.039 (min)

;

Member1 - Span 1

Member details

Service condition;

Dry

Sawn lumber section details

Number of sections in member;

N = 1

Nominal breadth of sections;

b_{nom} = 2 in

Breadth of sections;

b = 1.5 in

Nominal depth of sections;

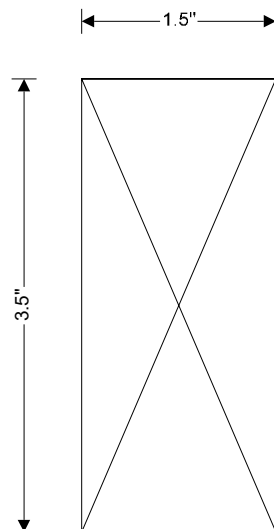
d_{nom} = 4 in

Depth of sections;

d = 3.5 in

Material;

Spruce-Pine-Fir, 2" & wider, No.2 grade



2"x4" sawn lumber section

Cross-sectional area, A, 5.25 in²

Section modulus, S_x, 3.1 in³

Section modulus, S_y, 1.3 in³

Second moment of area, I_x, 5.4 in⁴

Second moment of area, I_y, 1 in⁴

Radius of gyration, r_x, 1.01 in

Radius of gyration, r_y, 0.433 in

Spruce-Pine-Fir, 2" & wider, No.2 grade

Bending, F_b, 875 psi

Shear parallel to grain, F_v, 135 psi

Compression parallel to grain, F_c, 1150 psi

Compression perpendicular to grain, F_{c_perp}, 425 psi

Tension parallel to grain, F_t, 450 psi

Modulus of elasticity, E, 1400000 psi

Minimum modulus of elasticity, E_{min}, 510000 psi

Density, ρ, 29.098 lbm/ft³

Specific gravity, G, 0.42

Span details

Bearing length;

L_b = 4 in



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Revision:	Date:	Sheet No. 292
Project Name: JKV-EMS		

Consider Combination 1 - 1.0D + 1.0Lr (Strength)

Adjustment factors - Table 4.3.1

Ten years load duration factor - Table 2.3.2; $C_D = 1$
Size factor for bending - Table 4A; $C_{Fb} = 1.5$
Repetitive member factor - Table 4.3.9; $C_r = 1.15$

Check design at start of span

Design for bearing - cl.3.10

Design perpendicular compression; $R_x = 0 \text{ lb}$
Design bearing stress - Table 4.3.1; $F_{c\_perp,x'} = F_{c\_perp} = 425 \text{ lb/in}^2$
Actual bearing stress; $f_{c\_perp,x} = R_x / (b \times L_b) = 0 \text{ lb/in}^2$
 $f_{c\_perp,x} / F_{c\_perp,x'} = 0.000$

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Check design at end of span

Bending members - Flexure - cl.3.3

Design bending moment; $M_x = 125 \text{ lb\_ft}$
Design bending stress - Table 4.3.1; $F_{b,x'} = F_b \times C_D \times C_{Fb} \times C_r = 1509 \text{ lb/in}^2$
Actual bending stress - eq.3.3-2; $f_{b,x} = M_x / S_x = 490 \text{ lb/in}^2$
 $f_{b,x} / F_{b,x'} = 0.324$

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force; $V_x = 167 \text{ lb}$
Design shear stress - Table 4.3.1; $F_{v,x'} = F_v \times C_D = 135 \text{ lb/in}^2$
Actual shear stress - eq.3.4-2; $f_{v,x} = 3 \times V_x / (2 \times b \times d) = 48 \text{ lb/in}^2$
 $f_{v,x} / F_{v,x'} = 0.353$

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression; $R_x = 333 \text{ lb}$
Design bearing stress - Table 4.3.1; $F_{c\_perp,x'} = F_{c\_perp} = 425 \text{ lb/in}^2$
Actual bearing stress; $f_{c\_perp,x} = R_x / (b \times L_b) = 56 \text{ lb/in}^2$
 $f_{c\_perp,x} / F_{c\_perp,x'} = 0.131$

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Consider Combination 7 - 1.0W (Service)

Check design 11.013in along span

Bending members - Deflection - cl.3.5

Instantaneous deflection; $\delta_x = 0.003 \text{ in}$
Allowable deflection; $\delta_{x,Allowable} = L_{m1\_s1} / 360 = 0.05 \text{ in}$
 $\delta_x / \delta_{x,Allowable} = 0.052$

PASS - Allowable deflection exceeds instantaneous deflection



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Revision:	Date:	Sheet No. 293
Project Name: JKV-EMS		

Member1 - Span 2

Member details

Service condition;

Dry

Sawn lumber section details

Number of sections in member;

$N = 1$

Nominal breadth of sections;

$b_{nom} = 2$ in

Breadth of sections;

$b = 1.5$ in

Nominal depth of sections;

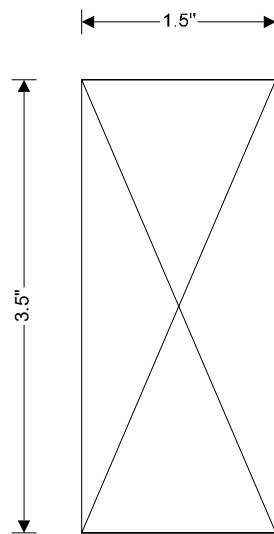
$d_{nom} = 4$ in

Depth of sections;

$d = 3.5$ in

Material;

Spruce-Pine-Fir, 2" & wider, No.2 grade



2"x4" sawn lumber section

Cross-sectional area, A , 5.25 in²

Section modulus, S_x , 3.1 in³

Section modulus, S_y , 1.3 in³

Second moment of area, I_x , 5.4 in⁴

Second moment of area, I_y , 1 in⁴

Radius of gyration, r_x , 1.01 in

Radius of gyration, r_y , 0.433 in

Spruce-Pine-Fir, 2" & wider, No.2 grade

Bending, F_b , 875 psi

Shear parallel to grain, F_v , 135 psi

Compression parallel to grain, F_c , 1150 psi

Compression perpendicular to grain, $F_{c_{perp}}$, 425 psi

Tension parallel to grain, F_t , 450 psi

Modulus of elasticity, E , 1400000 psi

Minimum modulus of elasticity, E_{min} , 510000 psi

Density, ρ , 29.098 lbm/ft³

Specific gravity, G , 0.42

Span details

Bearing length;

$L_b = 4$ in

Consider Combination 1 - 1.0D + 1.0Lr (Strength)

Adjustment factors - Table 4.3.1

Ten years load duration factor - Table 2.3.2;

$C_D = 1$

Size factor for bending - Table 4A;

$C_{Fb} = 1.5$

Repetitive member factor - Table 4.3.9;

$C_r = 1.15$

Check design at start of span

Bending members - Flexure - cl.3.3

Design bending moment;

$M_x = 125$ lb_{ft}

Design bending stress - Table 4.3.1;

$F_{b,x}' = F_b \times C_D \times C_{Fb} \times C_r = 1509$ lb/in²

Actual bending stress - eq.3.3-2;

$f_{b,x} = M_x / S_x = 490$ lb/in²

$f_{b,x} / F_{b,x}' = 0.324$

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force;

$V_x = 167$ lb



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Revision:	Date:	Sheet No. 294
Project Name: JKV-EMS		

Design shear stress - Table 4.3.1;

Actual shear stress - eq.3.4-2;

$$F_{v,x'} = F_v \times C_D = 135 \text{ lb/in}^2$$

$$f_{v,x} = 3 \times V_x / (2 \times b \times d) = 48 \text{ lb/in}^2$$

$$f_{v,x} / F_{v,x'} = 0.353$$

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression;

Design bearing stress - Table 4.3.1;

Actual bearing stress;

$$R_x = 333 \text{ lb}$$

$$F_{c\_perp,x'} = F_{c\_perp} = 425 \text{ lb/in}^2$$

$$f_{c\_perp,x} = R_x / (b \times L_b) = 56 \text{ lb/in}^2$$

$$f_{c\_perp,x} / F_{c\_perp,x'} = 0.131$$

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Consider Combination 7 - 1.0W (Service)

Check design at end of span

Bending members - Deflection - cl.3.5

Instantaneous deflection;

Allowable deflection;

$$\delta_x = 0.039 \text{ in}$$

$$\delta_{x,Allowable} = L_{m1\_s2} / 180 = 0.1 \text{ in}$$

$$\delta_x / \delta_{x,Allowable} = 0.386$$

PASS - Allowable deflection exceeds instantaneous deflection

NEW TOWER OUTRIGGER

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

Tedds calculation version 2.2.24

Member1 span 1 results summary	Unit	Capacity	Maximum	Utilization	Result
Bending stress	lb/in ²	1208	95	0.078	PASS
Shear stress	lb/in ²	135	23	0.168	PASS
Bearing stress	lb/in ²	425	50	0.119	PASS
Deflection	in	0.138	0.002	0.013	PASS
Member1 span 2 results summary	Unit	Capacity	Maximum	Utilization	Result
Bending stress	lb/in ²	1208	95	0.078	PASS
Shear stress	lb/in ²	135	19	0.141	PASS



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Date: 06/16/2025

Job No. 25-003

Revision:

Date:

Sheet No. 295

Project Name: JKV-EMS

Bearing stress	lb/in ²	425	50	0.119	PASS
Deflection	in	0.1	0.007	0.066	PASS

ANALYSIS

Tedds calculation version 1.0.38

Geometry

Geometry (ft) - Spruce-Pine-Fir(2" & wider(No.2)) - 2"x8"

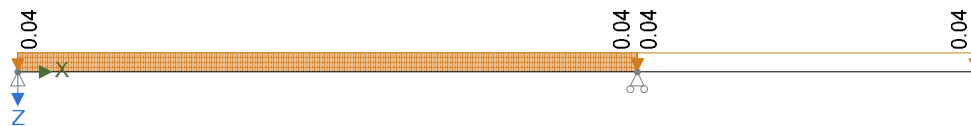


Span	Length (ft)	Section	Start Support	End Support
1	2.75	2"x8"	Pinned	Roller Pin X
2	1.5	2"x8"	Roller Pin X	Free
2"x8": Area 11 in ² , Inertia Major 48 in ⁴ , Inertia Minor 2 in ⁴ , Shear area parallel to Minor 9 in ² , Shear area parallel to Major 9 in ²				
Spruce-Pine-Fir(2" & wider(No.2)): Density 29.1 lbm/ft ³ , Youngs 1400 ksi, Shear 87 ksi, Thermal 0 °C ⁻¹				

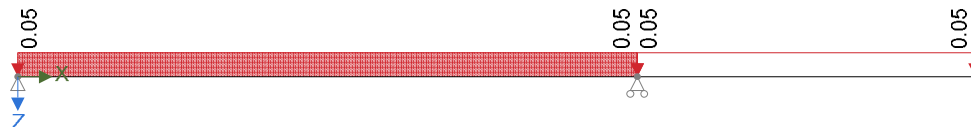
Loading

Self weight included

Dead - Loading (kips/ft)



Roof Live - Loading (kips/ft)



Wind - Loading (kips/ft)





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Revision:	Date:	Sheet No. 296
Project Name: JKV-EMS		

Load combination factors

Load combination	Self Weight	Dead	Roof Live	Wind
1.0D + 1.0Lr (Strength)	1.0 0 0	1.0 0 0	1.0 0 0	
1.0D + 0.6W (Strength)	1.0 0 0	1.0 0 0		0.6 0
1.0D + 0.75L + 0.75Lr + 0.45W (Strength)	1.0 0 0	1.0 0 0	0.7 5	0.4 5
0.6D + 0.6W (Strength)	0.6 0	0.6 0		0.6 0
1.0D + 1.0Lr (Service)	1.0 0 0	1.0 0 0	1.0 0 0	
1.0Lr (Service)			1.0 0 0	
1.0W (Service)				1.0 0 0

Member Loads

Member	Load case	Load Type	Orientation	Description
Member1	Dead	UDL	GlobalZ	0.04 kips/ft
Member1	Roof Live	UDL	GlobalZ	0.05 kips/ft

Element Loads

Element	Load case	Load Type	Orientation	Description
1	Wind	UDL	GlobalZ	-0.05 kips/ft
2	Wind	UDL	GlobalZ	-0.1 kips/ft



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Revision:	Date:	Sheet No. 297
Project Name: JKV-EMS		

Results

Reactions

Load case: Self Weight

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.002	0
2	0	0.007	0

Load case: Dead

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.039	0
2	0	0.131	0

Load case: Roof Live

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.048	0
2	0	0.164	0

Load case: Wind

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	-0.028	0
2	0	-0.26	0

Load combination: 1.0D + 1.0Lr (Strength)

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.089	0
2	0	0.303	0



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Date: 06/16/2025

Job No. 25-003

Revision:

Date:

Sheet No. 298

Project Name: JKV-EMS

Load combination: 1.0D + 0.6W (Strength)

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.024	0
2	0	-0.017	0

Load combination: 1.0D + 0.75L + 0.75Lr + 0.45W (Strength)

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.064	0
2	0	0.145	0

Load combination: 0.6D + 0.6W (Strength)

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.008	0
2	0	-0.073	0

Load combination: 1.0D + 1.0Lr (Service)

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.089	0
2	0	0.303	0

Load combination: 1.0Lr (Service)

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	0.048	0
2	0	0.164	0

Load combination: 1.0W (Service)

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	-0.028	0
2	0	-0.26	0



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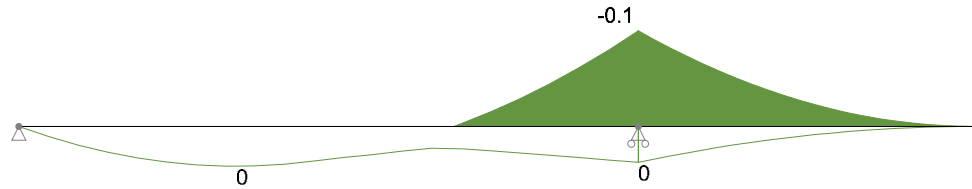
Date:

Sheet No. 299

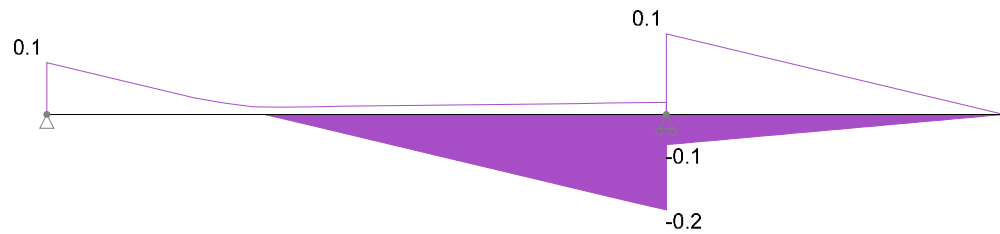
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Forces

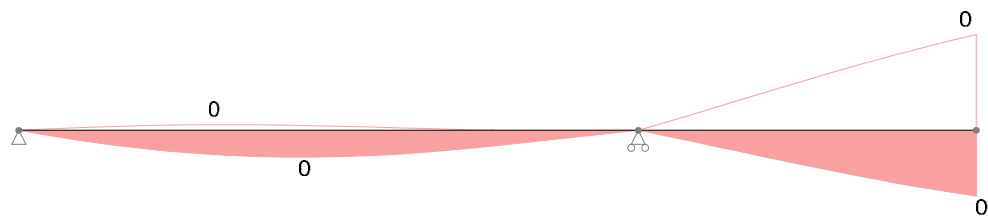
Strength combinations - Moment envelope (kip_ft)



Strength combinations - Shear envelope (kips)



Service combinations - Deflection envelope (in)



Member results

Envelope - Strength combinations

Member	Position (ft)	Shear force (kips)		Moment (kip_ft)	
Member1	0.97	0.012	0	0.043 (max)	0.01
	2.75	0.138	-0.164 (max abs)	0.039	-0.104 (min)

Envelope - Service combinations

Member	Position (ft)	Deflection (in)	
Member1	4.25	0.005 (max)	-0.007 (min)

;

Member1 - Span 1

Member details

Service condition;

Dry



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Revision:	Date:	Sheet No. 300
Project Name: JKV-EMS		

Sawn lumber section details

Number of sections in member;
Nominal breadth of sections;
Breadth of sections;
Nominal depth of sections;
Depth of sections;
Material;

$$N = 1$$

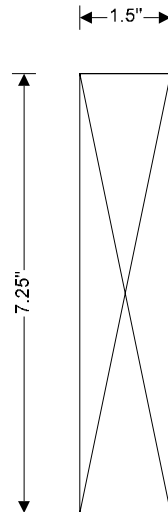
$$b_{nom} = 2 \text{ in}$$

$$b = 1.5 \text{ in}$$

$$d_{nom} = 8 \text{ in}$$

$$d = 7.25 \text{ in}$$

Spruce-Pine-Fir, 2" & wider, No.2 grade



2"x8" sawn lumber section

Cross-sectional area, A , 10.875 in²

Section modulus, S_x , 13.1 in³

Section modulus, S_y , 2.7 in³

Second moment of area, I_x , 47.6 in⁴

Second moment of area, I_y , 2 in⁴

Radius of gyration, r_x , 2.093 in

Radius of gyration, r_y , 0.433 in

Spruce-Pine-Fir, 2" & wider, No.2 grade

Bending, F_b , 875 psi

Shear parallel to grain, F_v , 135 psi

Compression parallel to grain, F_c , 1150 psi

Compression perpendicular to grain, F_{c_perp} , 425 psi

Tension parallel to grain, F_t , 450 psi

Modulus of elasticity, E , 1400000 psi

Minimum modulus of elasticity, E_{min} , 510000 psi

Density, ρ , 29.098 lbm/ft³

Specific gravity, G , 0.42

Span details

Bearing length;

$$L_b = 4 \text{ in}$$

Consider Combination 1 - 1.0D + 1.0Lr (Strength)

Adjustment factors - Table 4.3.1

Ten years load duration factor - Table 2.3.2;

$$C_D = 1$$

Size factor for bending - Table 4A;

$$C_{Fb} = 1.2$$

Repetitive member factor - Table 4.3.9;

$$C_r = 1.15$$

Check design at start of span

Bending members - Shear - cl.3.4

Design shear force;

$$V_x = 89 \text{ lb}$$

Design shear stress - Table 4.3.1;

$$F_{v,x}' = F_v \times C_D = 135 \text{ lb/in}^2$$

Actual shear stress - eq.3.4-2;

$$f_{v,x} = 3 \times V_x / (2 \times b \times d) = 12 \text{ lb/in}^2$$

$$f_{v,x} / F_{v,x}' = 0.091$$

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression;

$$R_x = 89 \text{ lb}$$

Design bearing stress - Table 4.3.1;

$$F_{c\_perp,x}' = F_{c\_perp} = 425 \text{ lb/in}^2$$

Actual bearing stress;

$$f_{c\_perp,x} = R_x / (b \times L_b) = 15 \text{ lb/in}^2$$

$$f_{c\_perp,x} / F_{c\_perp,x}' = 0.035$$

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain



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Revision:	Date:	Sheet No. 301
Project Name: JKV-EMS		

Check design at end of span

Bending members - Flexure - cl.3.3

Design bending moment;

Design bending stress - Table 4.3.1;

Actual bending stress - eq.3.3-2;

$$M_x = 104 \text{ lb\_ft}$$

$$F_{b,x}' = F_b \times C_D \times C_{Fb} \times C_r = 1208 \text{ lb/in}^2$$

$$f_{b,x} = M_x / S_x = 95 \text{ lb/in}^2$$

$$f_{b,x} / F_{b,x}' = 0.078$$

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force;

Design shear stress - Table 4.3.1;

Actual shear stress - eq.3.4-2;

$$V_x = 164 \text{ lb}$$

$$F_{v,x}' = F_v \times C_D = 135 \text{ lb/in}^2$$

$$f_{v,x} = 3 \times V_x / (2 \times b \times d) = 23 \text{ lb/in}^2$$

$$f_{v,x} / F_{v,x}' = 0.168$$

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression;

Design bearing stress - Table 4.3.1;

Actual bearing stress;

$$R_x = 303 \text{ lb}$$

$$F_{c\_perp,x}' = F_{c\_perp} = 425 \text{ lb/in}^2$$

$$f_{c\_perp,x} = R_x / (b \times L_b) = 50 \text{ lb/in}^2$$

$$f_{c\_perp,x} / F_{c\_perp,x}' = 0.119$$

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Consider Combination 5 - 1.0D + 1.0Lr (Service)

Check design 1ft 2.939in along span

Bending members - Deflection - cl.3.5

Instantaneous deflection;

Allowable deflection;

$$\delta_x = 0.002 \text{ in}$$

$$\delta_{x,Allowable} = L_{m1\_s1} / 240 = 0.138 \text{ in}$$

$$\delta_x / \delta_{x,Allowable} = 0.013$$

PASS - Allowable deflection exceeds instantaneous deflection

Member1 - Span 2

Member details

Service condition;

Dry

Sawn lumber section details

Number of sections in member;

Nominal breadth of sections;

Breadth of sections;

Nominal depth of sections;

Depth of sections;

Material;

$$N = 1$$

$$b_{nom} = 2 \text{ in}$$

$$b = 1.5 \text{ in}$$

$$d_{nom} = 8 \text{ in}$$

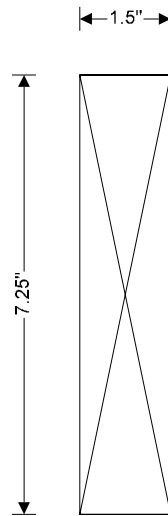
$$d = 7.25 \text{ in}$$

Spruce-Pine-Fir, 2" && wider, No.2 grade



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Revision:	Date:	Sheet No. 302
Project Name: JKV-EMS		



2"x8" sawn lumber section

Cross-sectional area, A , 10.875 in²

Section modulus, S_x , 13.1 in³

Section modulus, S_y , 2.7 in³

Second moment of area, I_x , 47.6 in⁴

Second moment of area, I_y , 2 in⁴

Radius of gyration, r_x , 2.093 in

Radius of gyration, r_y , 0.433 in

Spruce-Pine-Fir, 2" & wider, No.2 grade

Bending, F_b , 875 psi

Shear parallel to grain, F_v , 135 psi

Compression parallel to grain, F_c , 1150 psi

Compression perpendicular to grain, F_{c_perp} , 425 psi

Tension parallel to grain, F_t , 450 psi

Modulus of elasticity, E , 1400000 psi

Minimum modulus of elasticity, E_{min} , 510000 psi

Density, ρ , 29.098 lbm/ft³

Specific gravity, G , 0.42

Span details

Bearing length;

$L_b = 4$ in

Consider Combination 1 - 1.0D + 1.0Lr (Strength)

Adjustment factors - Table 4.3.1

Ten years load duration factor - Table 2.3.2;

$C_D = 1$

Size factor for bending - Table 4A;

$C_{Fb} = 1.2$

Repetitive member factor - Table 4.3.9;

$C_r = 1.15$

Check design at start of span

Bending members - Flexure - cl.3.3

Design bending moment;

$M_x = 104$ lb_{ft}

Design bending stress - Table 4.3.1;

$F_{b,x}' = F_b \times C_D \times C_{Fb} \times C_r = 1208$ lb/in²

Actual bending stress - eq.3.3-2;

$f_{b,x} = M_x / S_x = 95$ lb/in²

$f_{b,x} / F_{b,x}' = 0.078$

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force;

$V_x = 138$ lb

Design shear stress - Table 4.3.1;

$F_{v,x}' = F_v \times C_D = 135$ lb/in²

Actual shear stress - eq.3.4-2;

$f_{v,x} = 3 \times V_x / (2 \times b \times d) = 19$ lb/in²

$f_{v,x} / F_{v,x}' = 0.141$

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression;

$R_x = 303$ lb

Design bearing stress - Table 4.3.1;

$F_{c\_perp,x}' = F_{c\_perp} = 425$ lb/in²

Actual bearing stress;

$f_{c\_perp,x} = R_x / (b \times L_b) = 50$ lb/in²

$f_{c\_perp,x} / F_{c\_perp,x}' = 0.119$

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain



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Revision:	Date:	Sheet No. 303
Project Name: JKV-EMS		

Consider Combination 7 - 1.0W (Service)

Check design at end of span

Bending members - Deflection - cl.3.5

Instantaneous deflection;

$$\delta_x = 0.007 \text{ in}$$

Allowable deflection;

$$\delta_{x, \text{Allowable}} = L_{m1\_s2} / 180 = 0.1 \text{ in}$$

$$\delta_x / \delta_{x, \text{Allowable}} = 0.066$$

PASS - Allowable deflection exceeds instantaneous deflection

NEW TOWER JOIST

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

Tedds calculation version 2.2.24

Member1 span 1 results summary	Unit	Capacity	Maximum	Utilization	Result
Bending stress	lb/in ²	1208	95	0.078	PASS
Shear stress	lb/in ²	135	19	0.141	PASS
Bearing stress	lb/in ²	425	88	0.207	PASS
Deflection	in	0.15	0.072	0.482	PASS
Member1 span 2 results summary	Unit	Capacity	Maximum	Utilization	Result
Bending stress	lb/in ²	1208	653	0.541	PASS
Shear stress	lb/in ²	135	58	0.427	PASS
Bearing stress	lb/in ²	425	104	0.245	PASS
Deflection	in	0.438	0.154	0.352	PASS
Member1 span 3 results summary	Unit	Capacity	Maximum	Utilization	Result
Bending stress	lb/in ²	1050	213	0.203	PASS
Shear stress	lb/in ²	135	29	0.212	PASS
Bearing stress	lb/in ²	425	104	0.245	PASS



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Date: 06/16/2025

Job No. 25-003

Revision:

Date:

Sheet No. 304

Project Name: JKV-EMS

Deflection

in

0.225

0.09

0.399

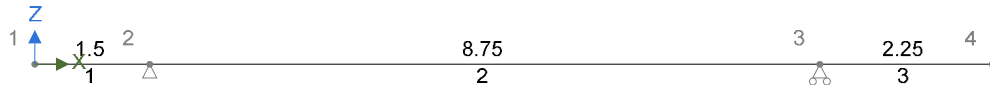
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ANALYSIS

Tedds calculation version 1.0.38

Geometry

Geometry (ft) - Spruce-Pine-Fir(2" && wider(No.2)) - 2"x8"



Span	Length (ft)	Section	Start Support	End Support
1	1.5	2"x8"	Free	Pinned
2	8.75	2"x8"	Pinned	Roller Pin X
3	2.25	2"x8"	Roller Pin X	Free
2"x8": Area 11 in ² , Inertia Major 48 in ⁴ , Inertia Minor 2 in ⁴ , Shear area parallel to Minor 9 in ² , Shear area parallel to Major 9 in ²				
Spruce-Pine-Fir(2" && wider(No.2)): Density 29.1 lbm/ft ³ , Youngs 1400 ksi, Shear 87 ksi, Thermal 0 °C ⁻¹				

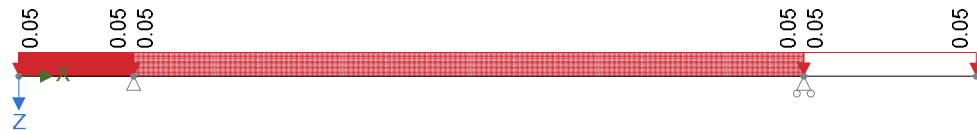
Loading

Self weight included

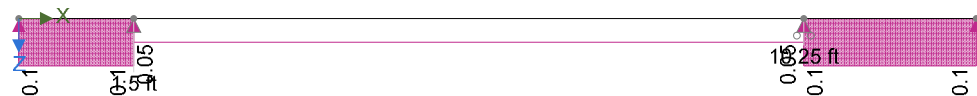
Dead - Loading (kips/ft)



Roof Live - Loading (kips/ft)



Wind - Loading (kips/ft)





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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 305
Project Name: JKV-EMS		

Load combination factors

Load combination	Self Weight	Dead	Roof Live	Wind
1.0D + 1.0Lr (Strength)	1.0 0 0	1.0 0 0	1.0 0 0	
1.0D + 0.6W (Strength)	1.0 0 0	1.0 0 0		0.6 0
1.0D + 0.75L + 0.75Lr + 0.45W (Strength)	1.0 0 0	1.0 0 0	0.7 5	0.4 5
0.6D + 0.6W (Strength)	0.6 0	0.6 0		0.6 0
1.0D + 1.0Lr (Service)	1.0 0 0	1.0 0 0	1.0 0 0	
1.0Lr (Service)			1.0 0 0	
1.0W (Service)				1.0 0 0

Member Loads

Member	Load case	Load Type	Orientation	Description
Member1	Dead	UDL	GlobalZ	0.04 kips/ft
Member1	Roof Live	UDL	GlobalZ	0.05 kips/ft

Element Loads

Element	Load case	Load Type	Orientation	Description
1	Wind	UDL	GlobalZ	-0.1 kips/ft
2	Wind	UDL	GlobalZ	-0.05 kips/ft
3	Wind	UDL	GlobalZ	-0.1 kips/ft



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 306
Project Name: JKV-EMS		

Results

Reactions

Load case: Self Weight

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
2	0	0.013	0
3	0	0.015	0

Load case: Dead

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
2	0	0.229	0
3	0	0.271	0

Load case: Roof Live

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
2	0	0.286	0
3	0	0.339	0

Load case: Wind

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
2	0	-0.353	0
3	0	-0.46	0

Load combination: 1.0D + 1.0Lr (Strength)

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
2	0	0.527	0
3	0	0.626	0



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Revision:	Date:	Sheet No. 307
Project Name: JKV-EMS		

Load combination: 1.0D + 0.6W (Strength)

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
2	0	0.03	0
3	0	0.01	0

Load combination: 1.0D + 0.75L + 0.75Lr + 0.45W (Strength)

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
2	0	0.297	0
3	0	0.334	0

Load combination: 0.6D + 0.6W (Strength)

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
2	0	-0.067	0
3	0	-0.104	0

Load combination: 1.0D + 1.0Lr (Service)

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
2	0	0.527	0
3	0	0.626	0

Load combination: 1.0Lr (Service)

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
2	0	0.286	0
3	0	0.339	0

Load combination: 1.0W (Service)

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
2	0	-0.353	0
3	0	-0.46	0



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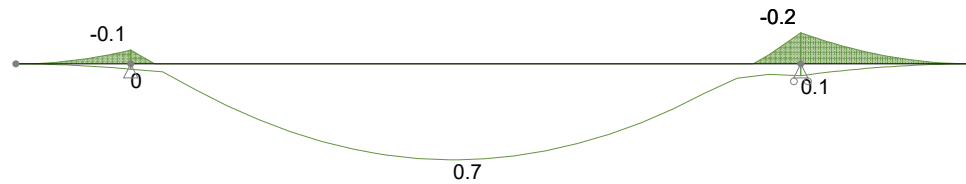
Date:

Sheet No. 308

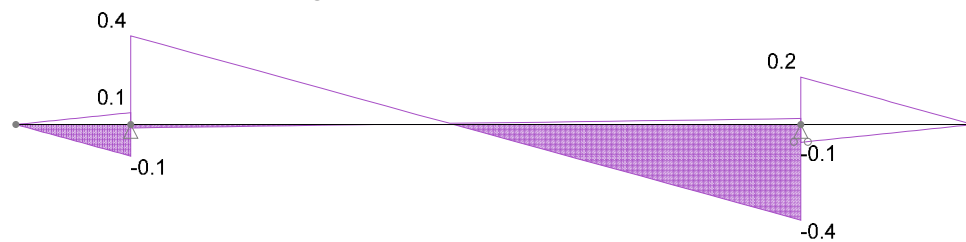
Project Name: JKV-EMS

Forces

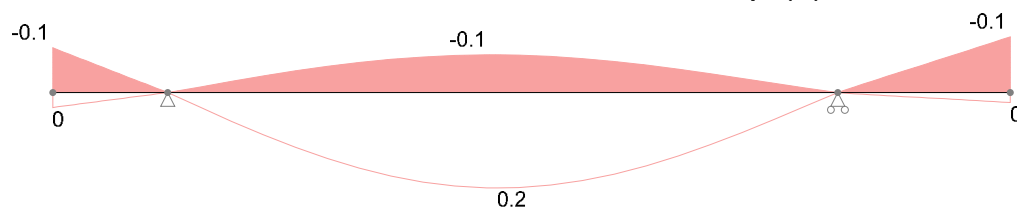
Strength combinations - Moment envelope (kip_ft)



Strength combinations - Shear envelope (kips)



Service combinations - Deflection envelope (in)



Member results

Envelope - Strength combinations

Member	Position (ft)	Shear force (kips)		Moment (kip_ft)	
Member1	1.5	0.389	-0.138	0.039	-0.104
	5.71	0.005	0	0.715 (max)	0.018
	10.25	0.207	-0.418 (max abs)	0.088	-0.233 (min)

Envelope - Service combinations

Member	Position (ft)	Deflection (in)	
Member1	5.81	0.154 (max)	-0.061
	12.5	0.016	-0.09 (min)

;

Member1 - Span 1

Member details

Service condition;

Dry



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Revision:	Date:	Sheet No. 309
Project Name: JKV-EMS		

Sawn lumber section details

Number of sections in member;
Nominal breadth of sections;
Breadth of sections;
Nominal depth of sections;
Depth of sections;
Material;

$$N = 1$$

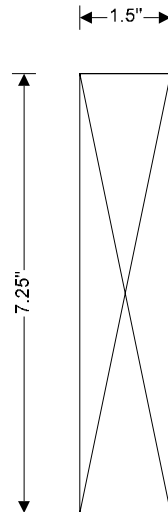
$$b_{nom} = 2 \text{ in}$$

$$b = 1.5 \text{ in}$$

$$d_{nom} = 8 \text{ in}$$

$$d = 7.25 \text{ in}$$

Spruce-Pine-Fir, 2" & wider, No.2 grade



2"x8" sawn lumber section

Cross-sectional area, A , 10.875 in²

Section modulus, S_x , 13.1 in³

Section modulus, S_y , 2.7 in³

Second moment of area, I_x , 47.6 in⁴

Second moment of area, I_y , 2 in⁴

Radius of gyration, r_x , 2.093 in

Radius of gyration, r_y , 0.433 in

Spruce-Pine-Fir, 2" & wider, No.2 grade

Bending, F_b , 875 psi

Shear parallel to grain, F_v , 135 psi

Compression parallel to grain, F_c , 1150 psi

Compression perpendicular to grain, F_{c_perp} , 425 psi

Tension parallel to grain, F_t , 450 psi

Modulus of elasticity, E , 1400000 psi

Minimum modulus of elasticity, E_{min} , 510000 psi

Density, ρ , 29.098 lbm/ft³

Specific gravity, G , 0.42

Span details

Bearing length;

$$L_b = 4 \text{ in}$$

Consider Combination 1 - 1.0D + 1.0Lr (Strength)

Adjustment factors - Table 4.3.1

Ten years load duration factor - Table 2.3.2;

$$C_D = 1$$

Size factor for bending - Table 4A;

$$C_{Fb} = 1.2$$

Repetitive member factor - Table 4.3.9;

$$C_r = 1.15$$

Check design at end of span

Bending members - Flexure - cl.3.3

Design bending moment;

$$M_x = 104 \text{ lb\_ft}$$

Design bending stress - Table 4.3.1;

$$F_{b,x}' = F_b \times C_D \times C_{Fb} \times C_r = 1208 \text{ lb/in}^2$$

Actual bending stress - eq.3.3-2;

$$f_{b,x} = M_x / S_x = 95 \text{ lb/in}^2$$

$$f_{b,x} / F_{b,x}' = 0.078$$

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force;

$$V_x = 138 \text{ lb}$$

Design shear stress - Table 4.3.1;

$$F_{v,x}' = F_v \times C_D = 135 \text{ lb/in}^2$$

Actual shear stress - eq.3.4-2;

$$f_{v,x} = 3 \times V_x / (2 \times b \times d) = 19 \text{ lb/in}^2$$

$$f_{v,x} / F_{v,x}' = 0.141$$

PASS - Design shear stress exceeds actual shear stress



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Revision:	Date:	Sheet No. 310
Project Name: JKV-EMS		

Design for bearing - cl.3.10

Design perpendicular compression;

Design bearing stress - Table 4.3.1;

Actual bearing stress;

$$R_x = 527 \text{ lb}$$

$$F_{c\_perp,x'} = F_{c\_perp} = 425 \text{ lb/in}^2$$

$$f_{c\_perp,x} = R_x / (b \times L_b) = 88 \text{ lb/in}^2$$

$$f_{c\_perp,x} / F_{c\_perp,x'} = 0.207$$

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Consider Combination 5 - 1.0D + 1.0Lr (Service)

Check design at start of span

Bending members - Deflection - cl.3.5

Instantaneous deflection;

Allowable deflection;

$$\delta_x = 0.072 \text{ in}$$

$$\delta_{x,Allowable} = L_{m1\_s1} / 120 = 0.15 \text{ in}$$

$$\delta_x / \delta_{x,Allowable} = 0.482$$

PASS - Allowable deflection exceeds instantaneous deflection

Member1 - Span 2

Member details

Service condition;

Dry

Sawn lumber section details

Number of sections in member;

$$N = 1$$

Nominal breadth of sections;

$$b_{nom} = 2 \text{ in}$$

Breadth of sections;

$$b = 1.5 \text{ in}$$

Nominal depth of sections;

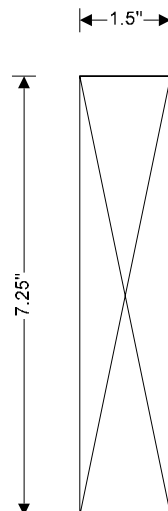
$$d_{nom} = 8 \text{ in}$$

Depth of sections;

$$d = 7.25 \text{ in}$$

Material;

Spruce-Pine-Fir, 2" & wider, No.2 grade



2"x8" sawn lumber section

Cross-sectional area, A , 10.875 in²

Section modulus, S_x , 13.1 in³

Section modulus, S_y , 2.7 in³

Second moment of area, I_x , 47.6 in⁴

Second moment of area, I_y , 2 in⁴

Radius of gyration, r_x , 2.093 in

Radius of gyration, r_y , 0.433 in

Spruce-Pine-Fir, 2" & wider, No.2 grade

Bending, F_b , 875 psi

Shear parallel to grain, F_v , 135 psi

Compression parallel to grain, F_c , 1150 psi

Compression perpendicular to grain, F_{c_perp} , 425 psi

Tension parallel to grain, F_t , 450 psi

Modulus of elasticity, E , 1400000 psi

Minimum modulus of elasticity, E_{min} , 510000 psi

Density, ρ , 29.098 lbm/ft³

Specific gravity, G , 0.42

Span details

Bearing length;

$$L_b = 4 \text{ in}$$



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Revision:	Date:	Sheet No. 311
Project Name: JKV-EMS		

Consider Combination 1 - 1.0D + 1.0Lr (Strength)

Adjustment factors - Table 4.3.1

Ten years load duration factor - Table 2.3.2; $C_D = 1$
Size factor for bending - Table 4A; $C_{Fb} = 1.2$
Repetitive member factor - Table 4.3.9; $C_r = 1.15$

Check design at start of span

Bending members - Flexure - cl.3.3

Design bending moment; $M_x = 104 \text{ lb\_ft}$
Design bending stress - Table 4.3.1; $F_{b,x'} = F_b \times C_D \times C_{Fb} \times C_r = 1208 \text{ lb/in}^2$
Actual bending stress - eq.3.3-2; $f_{b,x} = M_x / S_x = 95 \text{ lb/in}^2$
 $f_{b,x} / F_{b,x'} = 0.078$

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force; $V_x = 389 \text{ lb}$
Design shear stress - Table 4.3.1; $F_{v,x'} = F_v \times C_D = 135 \text{ lb/in}^2$
Actual shear stress - eq.3.4-2; $f_{v,x} = 3 \times V_x / (2 \times b \times d) = 54 \text{ lb/in}^2$
 $f_{v,x} / F_{v,x'} = 0.397$

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression; $R_x = 527 \text{ lb}$
Design bearing stress - Table 4.3.1; $F_{c\_perp,x'} = F_{c\_perp} = 425 \text{ lb/in}^2$
Actual bearing stress; $f_{c\_perp,x} = R_x / (b \times L_b) = 88 \text{ lb/in}^2$
 $f_{c\_perp,x} / F_{c\_perp,x'} = 0.207$

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Check design 4ft 2.571in along span

Bending members - Flexure - cl.3.3

Design bending moment; $M_x = 715 \text{ lb\_ft}$
Design bending stress - Table 4.3.1; $F_{b,x'} = F_b \times C_D \times C_{Fb} \times C_r = 1208 \text{ lb/in}^2$
Actual bending stress - eq.3.3-2; $f_{b,x} = M_x / S_x = 653 \text{ lb/in}^2$
 $f_{b,x} / F_{b,x'} = 0.541$

PASS - Design bending stress exceeds actual bending stress

Check design at end of span

Bending members - Flexure - cl.3.3

Design bending moment; $M_x = 233 \text{ lb\_ft}$
Design bending stress - Table 4.3.1; $F_{b,x'} = F_b \times C_D \times C_{Fb} \times C_r = 1208 \text{ lb/in}^2$
Actual bending stress - eq.3.3-2; $f_{b,x} = M_x / S_x = 213 \text{ lb/in}^2$
 $f_{b,x} / F_{b,x'} = 0.176$

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force; $V_x = 418 \text{ lb}$
Design shear stress - Table 4.3.1; $F_{v,x'} = F_v \times C_D = 135 \text{ lb/in}^2$



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Revision:	Date:	Sheet No. 312
Project Name: JKV-EMS		

Actual shear stress - eq.3.4-2;

$$f_{v,x} = 3 \times V_x / (2 \times b \times d) = 58 \text{ lb/in}^2$$

$$f_{v,x} / F_{v,x}' = 0.427$$

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression;

$$R_x = 626 \text{ lb}$$

Design bearing stress - Table 4.3.1;

$$F_{c\_perp,x}' = F_{c\_perp} = 425 \text{ lb/in}^2$$

Actual bearing stress;

$$f_{c\_perp,x} = R_x / (b \times L_b) = 104 \text{ lb/in}^2$$

$$f_{c\_perp,x} / F_{c\_perp,x}' = 0.245$$

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Consider Combination 5 - 1.0D + 1.0Lr (Service)

Check design 4ft 3.761in along span

Bending members - Deflection - cl.3.5

Instantaneous deflection;

$$\delta_x = 0.154 \text{ in}$$

Allowable deflection;

$$\delta_{x,Allowable} = L_{m1\_s2} / 240 = 0.438 \text{ in}$$

$$\delta_x / \delta_{x,Allowable} = 0.352$$

PASS - Allowable deflection exceeds instantaneous deflection

Member1 - Span 3

Member details

Service condition;

Dry

Sawn lumber section details

Number of sections in member;

$$N = 1$$

Nominal breadth of sections;

$$b_{nom} = 2 \text{ in}$$

Breadth of sections;

$$b = 1.5 \text{ in}$$

Nominal depth of sections;

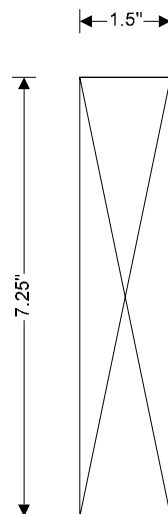
$$d_{nom} = 8 \text{ in}$$

Depth of sections;

$$d = 7.25 \text{ in}$$

Material;

Spruce-Pine-Fir, 2" & wider, No.2 grade



2"x8" sawn lumber section

Cross-sectional area, A , 10.875 in²

Section modulus, S_x , 13.1 in³

Section modulus, S_y , 2.7 in³

Second moment of area, I_x , 47.6 in⁴

Second moment of area, I_y , 2 in⁴

Radius of gyration, r_x , 2.093 in

Radius of gyration, r_y , 0.433 in

Spruce-Pine-Fir, 2" & wider, No.2 grade

Bending, F_b , 875 psi

Shear parallel to grain, F_v , 135 psi

Compression parallel to grain, F_c , 1150 psi

Compression perpendicular to grain, F_{c_perp} , 425 psi

Tension parallel to grain, F_t , 450 psi

Modulus of elasticity, E , 1400000 psi

Minimum modulus of elasticity, E_{min} , 510000 psi

Density, ρ , 29.098 lbm/ft³

Specific gravity, G , 0.42



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Revision:	Date:	Sheet No. 313
Project Name: JKV-EMS		

Span details

Bearing length;

$$L_b = 4 \text{ in}$$

Consider Combination 1 - 1.0D + 1.0Lr (Strength)

Adjustment factors - Table 4.3.1

Ten years load duration factor - Table 2.3.2;

$$C_D = 1$$

Size factor for bending - Table 4A;

$$C_{Fb} = 1.2$$

Check design at start of span

Bending members - Flexure - cl.3.3

Design bending moment;

$$M_x = 233 \text{ lb\_ft}$$

Design bending stress - Table 4.3.1;

$$F_{b,x}' = F_b \times C_D \times C_{Fb} = 1050 \text{ lb/in}^2$$

Actual bending stress - eq.3.3-2;

$$f_{b,x} = M_x / S_x = 213 \text{ lb/in}^2$$

$$f_{b,x} / F_{b,x}' = 0.203$$

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force;

$$V_x = 207 \text{ lb}$$

Design shear stress - Table 4.3.1;

$$F_{v,x}' = F_v \times C_D = 135 \text{ lb/in}^2$$

Actual shear stress - eq.3.4-2;

$$f_{v,x} = 3 \times V_x / (2 \times b \times d) = 29 \text{ lb/in}^2$$

$$f_{v,x} / F_{v,x}' = 0.212$$

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression;

$$R_x = 626 \text{ lb}$$

Design bearing stress - Table 4.3.1;

$$F_{c\_perp,x}' = F_{c\_perp} = 425 \text{ lb/in}^2$$

Actual bearing stress;

$$f_{c\_perp,x} = R_x / (b \times L_b) = 104 \text{ lb/in}^2$$

$$f_{c\_perp,x} / F_{c\_perp,x}' = 0.245$$

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Consider Combination 5 - 1.0D + 1.0Lr (Service)

Check design at end of span

Bending members - Deflection - cl.3.5

Instantaneous deflection;

$$\delta_x = 0.09 \text{ in}$$

Allowable deflection;

$$\delta_{x,Allowable} = L_{m1\_s3} / 120 = 0.225 \text{ in}$$

$$\delta_x / \delta_{x,Allowable} = 0.399$$

PASS - Allowable deflection exceeds instantaneous deflection



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Revision:	Date:	Sheet No. 314
Project Name: JKV-EMS		

NEW TOWER JOIST GIRDER

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

Tedds calculation version 2.2.24

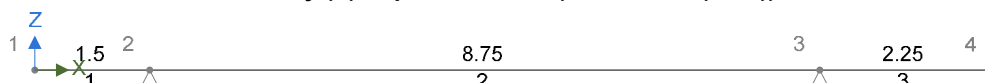
Member1 span 1 results summary	Unit	Capacity	Maximum	Utilization	Result
Bending stress	lb/in ²	1208	109	0.091	PASS
Shear stress	lb/in ²	135	16	0.118	PASS
Bearing stress	lb/in ²	425	86	0.201	PASS
Deflection	in	0.15	0.07	0.465	PASS
Member1 span 2 results summary	Unit	Capacity	Maximum	Utilization	Result
Bending stress	lb/in ²	1208	645	0.535	PASS
Shear stress	lb/in ²	135	55	0.407	PASS
Bearing stress	lb/in ²	425	102	0.239	PASS
Deflection	in	0.438	0.151	0.346	PASS
Member1 span 3 results summary	Unit	Capacity	Maximum	Utilization	Result
Bending stress	lb/in ²	1050	201	0.191	PASS
Shear stress	lb/in ²	135	21	0.154	PASS
Bearing stress	lb/in ²	425	102	0.239	PASS
Deflection	in	0.225	0.088	0.390	PASS

ANALYSIS

Tedds calculation version 1.0.38

Geometry

Geometry (ft) - Spruce-Pine-Fir(2" & wider(No.2)) - 2"x8"





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Job No. 25-003

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Sheet No. 315

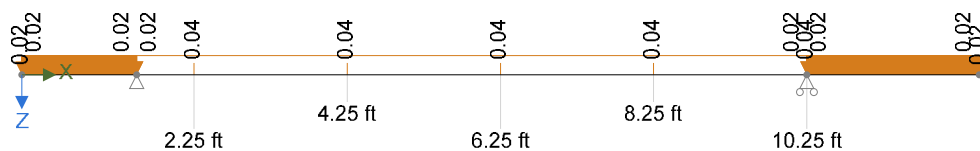
Project Name: JKV-EMS

Span	Length (ft)	Section	Start Support	End Support
1	1.5	2"x8"	Free	Pinned
2	8.75	2"x8"	Pinned	Roller Pin X
3	2.25	2"x8"	Roller Pin X	Free
2"x8": Area 11 in ² , Inertia Major 48 in ⁴ , Inertia Minor 2 in ⁴ , Shear area parallel to Minor 9 in ² , Shear area parallel to Major 9 in ²				
Spruce-Pine-Fir(2" & wider(No.2)): Density 29.1 lbm/ft ³ , Youngs 1400 ksi, Shear 87 ksi, Thermal 0 °C ⁻¹				

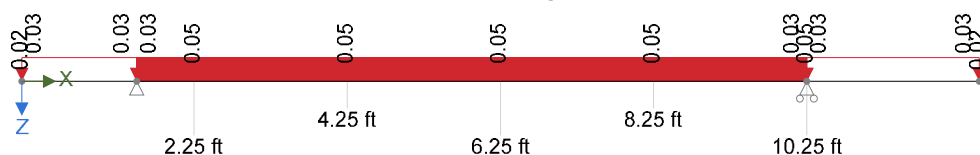
Loading

Self weight included

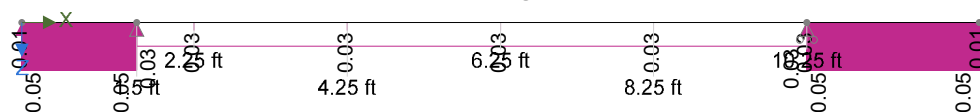
Dead - Loading (kips/ft,kips)



Roof Live - Loading (kips/ft,kips)



Wind - Loading (kips/ft,kips)



Load combination factors

Load combination	Self Weight	Dead	Roof Live	Wind
1.0D + 1.0Lr (Strength)	1.0	1.0	1.0	
	0	0	0	
	0	0	0	
1.0D + 0.6W (Strength)	1.0	1.0		0.6
	0	0		0
	0	0		0



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 316
Project Name: JKV-EMS		

Load combination	Self Weight	Dead	Roof Live	Wind
1.0D + 0.75L + 0.75Lr + 0.45W (Strength)	1.000	1.000	0.750	0.450
0.6D + 0.6W (Strength)	0.600	0.600		0.600
1.0D + 1.0Lr (Service)	1.000	1.000	1.000	
1.0Lr (Service)			1.000	
1.0W (Service)				1.000

Member Loads

Member	Load case	Load Type	Orientation	Description
Member1	Dead	UDL	GlobalZ	0.02 kips/ft
Member1	Dead	Point load	GlobalZ	0.02 kips at 0 ft
Member1	Dead	Point load	GlobalZ	0.04 kips at 2.25 ft
Member1	Dead	Point load	GlobalZ	0.04 kips at 4.25 ft
Member1	Dead	Point load	GlobalZ	0.04 kips at 6.25 ft
Member1	Dead	Point load	GlobalZ	0.04 kips at 8.25 ft
Member1	Dead	Point load	GlobalZ	0.04 kips at 10.25 ft
Member1	Dead	Point load	GlobalZ	0.02 kips at 12.5 ft
Member1	Roof Live	UDL	GlobalZ	0.03 kips/ft
Member1	Roof Live	Point load	GlobalZ	0.02 kips at 0 ft
Member1	Roof Live	Point load	GlobalZ	0.05 kips at 2.25 ft
Member1	Roof Live	Point load	GlobalZ	0.05 kips at 4.25 ft
Member1	Roof Live	Point load	GlobalZ	0.05 kips at 6.25 ft
Member1	Roof Live	Point load	GlobalZ	0.05 kips at 8.25 ft
Member1	Roof Live	Point load	GlobalZ	0.05 kips at 10.25 ft
Member1	Roof Live	Point load	GlobalZ	0.02 kips at 12.5 ft



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 317
Project Name: JKV-EMS		

Member	Load case	Load Type	Orientation	Description
Member1	Wind	Point load	GlobalZ	-0.01 kips at 0 ft
Member1	Wind	Point load	GlobalZ	-0.03 kips at 2.25 ft
Member1	Wind	Point load	GlobalZ	-0.03 kips at 4.25 ft
Member1	Wind	Point load	GlobalZ	-0.03 kips at 6.25 ft
Member1	Wind	Point load	GlobalZ	-0.03 kips at 8.25 ft
Member1	Wind	Point load	GlobalZ	-0.03 kips at 10.25 ft
Member1	Wind	Point load	GlobalZ	-0.01 kips at 12.5 ft

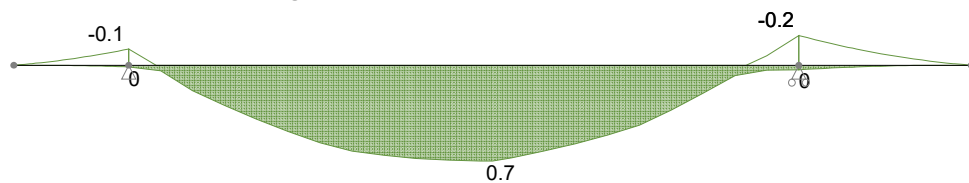
Element Loads

Element	Load case	Load Type	Orientation	Description
1	Wind	UDL	GlobalZ	-0.05 kips/ft
2	Wind	UDL	GlobalZ	-0.02 kips/ft
3	Wind	UDL	GlobalZ	-0.05 kips/ft

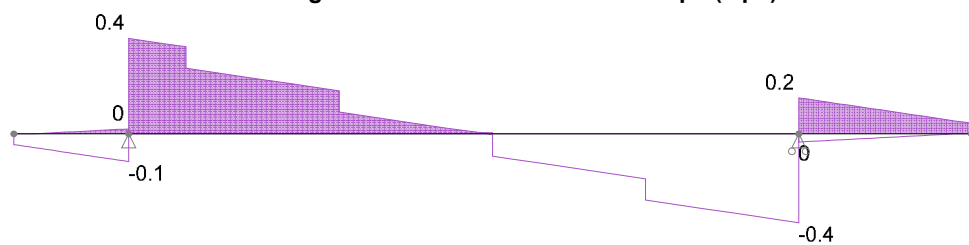
Results

Forces

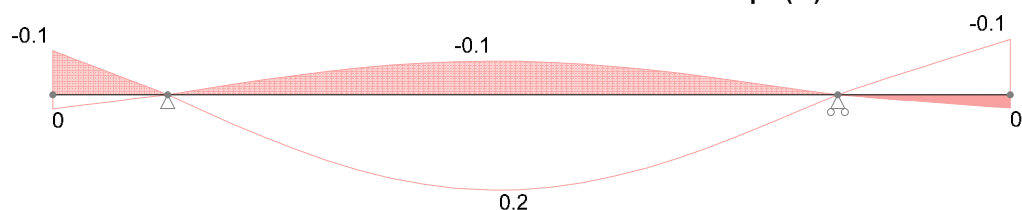
Strength combinations - Moment envelope (kip_ft)



Strength combinations - Shear envelope (kips)



Service combinations - Deflection envelope (in)





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Revision:	Date:	Sheet No. 318
Project Name: JKV-EMS		

Member results

Envelope - Strength combinations

Member	Position (ft)	Shear force (kips)		Moment (kip_ft)	
Member1	1.5	0.399 (max abs)	-0.115	0.013	-0.12
	6.17	0.005	0	0.707 (max)	0.046
	10.25	0.151	-0.37	0.033	-0.22 (min)

Envelope - Service combinations

Member	Position (ft)	Deflection (in)	
Member1	5.84	0.151 (max)	-0.053
	12.5	0.021	-0.088 (min)

;

Member1 - Span 1

Member details

Service condition;

Dry

Sawn lumber section details

Number of sections in member;

N = 1

Nominal breadth of sections;

b_{nom} = 2 in

Breadth of sections;

b = 1.5 in

Nominal depth of sections;

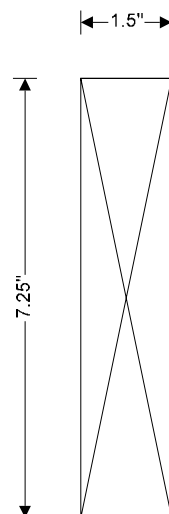
d_{nom} = 8 in

Depth of sections;

d = 7.25 in

Material;

Spruce-Pine-Fir, 2" & wider, No.2 grade



2"x8" sawn lumber section

Cross-sectional area, A, 10.875 in²

Section modulus, S_x, 13.1 in³

Section modulus, S_y, 2.7 in³

Second moment of area, I_x, 47.6 in⁴

Second moment of area, I_y, 2 in⁴

Radius of gyration, r_x, 2.093 in

Radius of gyration, r_y, 0.433 in

Spruce-Pine-Fir, 2" & wider, No.2 grade

Bending, F_b, 875 psi

Shear parallel to grain, F_v, 135 psi

Compression parallel to grain, F_c, 1150 psi

Compression perpendicular to grain, F_{c_perp}, 425 psi

Tension parallel to grain, F_t, 450 psi

Modulus of elasticity, E, 1400000 psi

Minimum modulus of elasticity, E_{min}, 510000 psi

Density, ρ, 29.098 lbm/ft³

Specific gravity, G, 0.42

Span details

Bearing length;

L_b = 4 in



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Revision:	Date:	Sheet No. 319
Project Name: JKV-EMS		

Consider Combination 1 - 1.0D + 1.0Lr (Strength)

Adjustment factors - Table 4.3.1

Ten years load duration factor - Table 2.3.2; $C_D = 1$
Size factor for bending - Table 4A; $C_{Fb} = 1.2$
Repetitive member factor - Table 4.3.9; $C_r = 1.15$

Check design at start of span

Bending members - Shear - cl.3.4

Design shear force; $V_x = 45 \text{ lb}$
Design shear stress - Table 4.3.1; $F_{v,x'} = F_v \times C_D = 135 \text{ lb/in}^2$
Actual shear stress - eq.3.4-2; $f_{v,x} = 3 \times V_x / (2 \times b \times d) = 6 \text{ lb/in}^2$
 $f_{v,x} / F_{v,x'} = 0.045$

PASS - Design shear stress exceeds actual shear stress

Check design at end of span

Bending members - Flexure - cl.3.3

Design bending moment; $M_x = 120 \text{ lb\_ft}$
Design bending stress - Table 4.3.1; $F_{b,x'} = F_b \times C_D \times C_{Fb} \times C_r = 1208 \text{ lb/in}^2$
Actual bending stress - eq.3.3-2; $f_{b,x} = M_x / S_x = 109 \text{ lb/in}^2$
 $f_{b,x} / F_{b,x'} = 0.091$

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force; $V_x = 115 \text{ lb}$
Design shear stress - Table 4.3.1; $F_{v,x'} = F_v \times C_D = 135 \text{ lb/in}^2$
Actual shear stress - eq.3.4-2; $f_{v,x} = 3 \times V_x / (2 \times b \times d) = 16 \text{ lb/in}^2$
 $f_{v,x} / F_{v,x'} = 0.118$

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression; $R_x = 514 \text{ lb}$
Design bearing stress - Table 4.3.1; $F_{c\_perp,x'} = F_{c\_perp} = 425 \text{ lb/in}^2$
Actual bearing stress; $f_{c\_perp,x} = R_x / (b \times L_b) = 86 \text{ lb/in}^2$
 $f_{c\_perp,x} / F_{c\_perp,x'} = 0.201$

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Consider Combination 5 - 1.0D + 1.0Lr (Service)

Check design at start of span

Bending members - Deflection - cl.3.5

Instantaneous deflection; $\delta_x = 0.07 \text{ in}$
Allowable deflection; $\delta_{x,Allowable} = L_{m1\_s1} / 120 = 0.15 \text{ in}$
 $\delta_x / \delta_{x,Allowable} = 0.465$

PASS - Allowable deflection exceeds instantaneous deflection



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Revision:	Date:	Sheet No. 320
Project Name: JKV-EMS		

Member1 - Span 2

Member details

Service condition;

Dry

Sawn lumber section details

Number of sections in member;

$N = 1$

Nominal breadth of sections;

$b_{nom} = 2$ in

Breadth of sections;

$b = 1.5$ in

Nominal depth of sections;

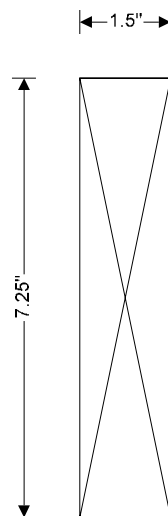
$d_{nom} = 8$ in

Depth of sections;

$d = 7.25$ in

Material;

Spruce-Pine-Fir, 2" & wider, No.2 grade



2"x8" sawn lumber section

Cross-sectional area, A , 10.875 in²

Section modulus, S_x , 13.1 in³

Section modulus, S_y , 2.7 in³

Second moment of area, I_x , 47.6 in⁴

Second moment of area, I_y , 2 in⁴

Radius of gyration, r_x , 2.093 in

Radius of gyration, r_y , 0.433 in

Spruce-Pine-Fir, 2" & wider, No.2 grade

Bending, F_b , 875 psi

Shear parallel to grain, F_v , 135 psi

Compression parallel to grain, F_c , 1150 psi

Compression perpendicular to grain, $F_{c_{perp}}$, 425 psi

Tension parallel to grain, F_t , 450 psi

Modulus of elasticity, E , 1400000 psi

Minimum modulus of elasticity, E_{min} , 510000 psi

Density, ρ , 29.098 lbm/ft³

Specific gravity, G , 0.42

Span details

Bearing length;

$L_b = 4$ in

Consider Combination 1 - 1.0D + 1.0Lr (Strength)

Adjustment factors - Table 4.3.1

Ten years load duration factor - Table 2.3.2;

$C_D = 1$

Size factor for bending - Table 4A;

$C_{Fb} = 1.2$

Repetitive member factor - Table 4.3.9;

$C_r = 1.15$

Check design at start of span

Bending members - Flexure - cl.3.3

Design bending moment;

$M_x = 120$ lb_{ft}

Design bending stress - Table 4.3.1;

$F_{b,x}' = F_b \times C_D \times C_{Fb} \times C_r = 1208$ lb/in²

Actual bending stress - eq.3.3-2;

$f_{b,x} = M_x / S_x = 109$ lb/in²

$f_{b,x} / F_{b,x}' = 0.091$

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force;

$V_x = 399$ lb



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Revision:	Date:	Sheet No. 321
Project Name: JKV-EMS		

Design shear stress - Table 4.3.1;

Actual shear stress - eq.3.4-2;

$$F_{v,x}' = F_v \times C_D = 135 \text{ lb/in}^2$$

$$f_{v,x} = 3 \times V_x / (2 \times b \times d) = 55 \text{ lb/in}^2$$

$$f_{v,x} / F_{v,x}' = 0.407$$

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression;

Design bearing stress - Table 4.3.1;

Actual bearing stress;

$$R_x = 514 \text{ lb}$$

$$F_{c\_perp,x}' = F_{c\_perp} = 425 \text{ lb/in}^2$$

$$f_{c\_perp,x} = R_x / (b \times L_b) = 86 \text{ lb/in}^2$$

$$f_{c\_perp,x} / F_{c\_perp,x}' = 0.201$$

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Check design 4ft 8.067in along span

Bending members - Flexure - cl.3.3

Design bending moment;

Design bending stress - Table 4.3.1;

Actual bending stress - eq.3.3-2;

$$M_x = 707 \text{ lb\_ft}$$

$$F_{b,x}' = F_b \times C_D \times C_{Fb} \times C_r = 1208 \text{ lb/in}^2$$

$$f_{b,x} = M_x / S_x = 645 \text{ lb/in}^2$$

$$f_{b,x} / F_{b,x}' = 0.535$$

PASS - Design bending stress exceeds actual bending stress

Check design at end of span

Bending members - Flexure - cl.3.3

Design bending moment;

Design bending stress - Table 4.3.1;

Actual bending stress - eq.3.3-2;

$$M_x = 220 \text{ lb\_ft}$$

$$F_{b,x}' = F_b \times C_D \times C_{Fb} \times C_r = 1208 \text{ lb/in}^2$$

$$f_{b,x} = M_x / S_x = 201 \text{ lb/in}^2$$

$$f_{b,x} / F_{b,x}' = 0.166$$

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force;

Design shear stress - Table 4.3.1;

Actual shear stress - eq.3.4-2;

$$V_x = 370 \text{ lb}$$

$$F_{v,x}' = F_v \times C_D = 135 \text{ lb/in}^2$$

$$f_{v,x} = 3 \times V_x / (2 \times b \times d) = 51 \text{ lb/in}^2$$

$$f_{v,x} / F_{v,x}' = 0.379$$

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression;

Design bearing stress - Table 4.3.1;

Actual bearing stress;

$$R_x = 610 \text{ lb}$$

$$F_{c\_perp,x}' = F_{c\_perp} = 425 \text{ lb/in}^2$$

$$f_{c\_perp,x} = R_x / (b \times L_b) = 102 \text{ lb/in}^2$$

$$f_{c\_perp,x} / F_{c\_perp,x}' = 0.239$$

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Consider Combination 5 - 1.0D + 1.0Lr (Service)

Check design 4ft 4.066in along span

Bending members - Deflection - cl.3.5

Instantaneous deflection;

Allowable deflection;

$$\delta_x = 0.151 \text{ in}$$

$$\delta_{x,Allowable} = L_{m1\_s2} / 240 = 0.438 \text{ in}$$



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Revision:	Date:	Sheet No. 322
Project Name: JKV-EMS		

$$\delta_x / \delta_{x, \text{Allowable}} = 0.346$$

PASS - Allowable deflection exceeds instantaneous deflection

Member1 - Span 3

Member details

Service condition;

Dry

Sawn lumber section details

Number of sections in member;

$$N = 1$$

Nominal breadth of sections;

$$b_{\text{nom}} = 2 \text{ in}$$

Breadth of sections;

$$b = 1.5 \text{ in}$$

Nominal depth of sections;

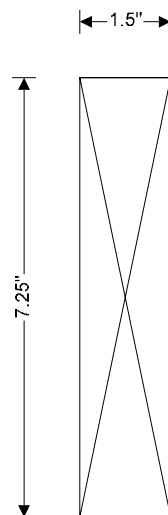
$$d_{\text{nom}} = 8 \text{ in}$$

Depth of sections;

$$d = 7.25 \text{ in}$$

Material;

Spruce-Pine-Fir, 2" & wider, No.2 grade



2"x8" sawn lumber section

Cross-sectional area, A , 10.875 in²

Section modulus, S_x , 13.1 in³

Section modulus, S_y , 2.7 in³

Second moment of area, I_x , 47.6 in⁴

Second moment of area, I_y , 2 in⁴

Radius of gyration, r_x , 2.093 in

Radius of gyration, r_y , 0.433 in

Spruce-Pine-Fir, 2" & wider, No.2 grade

Bending, F_b , 875 psi

Shear parallel to grain, F_v , 135 psi

Compression parallel to grain, F_c , 1150 psi

Compression perpendicular to grain, $F_{c, \text{perp}}$, 425 psi

Tension parallel to grain, F_t , 450 psi

Modulus of elasticity, E , 1400000 psi

Minimum modulus of elasticity, E_{min} , 510000 psi

Density, ρ , 29.098 lbm/ft³

Specific gravity, G , 0.42

Span details

Bearing length;

$$L_b = 4 \text{ in}$$

Consider Combination 1 - 1.0D + 1.0Lr (Strength)

Adjustment factors - Table 4.3.1

Ten years load duration factor - Table 2.3.2;

$$C_D = 1$$

Size factor for bending - Table 4A;

$$C_{Fb} = 1.2$$

Check design at start of span

Bending members - Flexure - cl.3.3

Design bending moment;

$$M_x = 220 \text{ lb\_ft}$$

Design bending stress - Table 4.3.1;

$$F_{b, x'} = F_b \times C_D \times C_{Fb} = 1050 \text{ lb/in}^2$$

Actual bending stress - eq.3.3-2;

$$f_{b, x} = M_x / S_x = 201 \text{ lb/in}^2$$

$$f_{b, x} / F_{b, x'} = 0.191$$

PASS - Design bending stress exceeds actual bending stress



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Revision:	Date:	Sheet No. 323
Project Name: JKV-EMS		

Bending members - Shear - cl.3.4

Design shear force;

Design shear stress - Table 4.3.1;

Actual shear stress - eq.3.4-2;

$$V_x = 151 \text{ lb}$$

$$F_{v,x}' = F_v \times C_D = 135 \text{ lb/in}^2$$

$$f_{v,x} = 3 \times V_x / (2 \times b \times d) = 21 \text{ lb/in}^2$$

$$f_{v,x} / F_{v,x}' = 0.154$$

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression;

Design bearing stress - Table 4.3.1;

Actual bearing stress;

$$R_x = 610 \text{ lb}$$

$$F_{c\_perp,x}' = F_{c\_perp} = 425 \text{ lb/in}^2$$

$$f_{c\_perp,x} = R_x / (b \times L_b) = 102 \text{ lb/in}^2$$

$$f_{c\_perp,x} / F_{c\_perp,x}' = 0.239$$

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Consider Combination 5 - 1.0D + 1.0Lr (Service)

Check design at end of span

Bending members - Deflection - cl.3.5

Instantaneous deflection;

Allowable deflection;

$$\delta_x = 0.088 \text{ in}$$

$$\delta_{x,Allowable} = L_{m1\_s3} / 120 = 0.225 \text{ in}$$

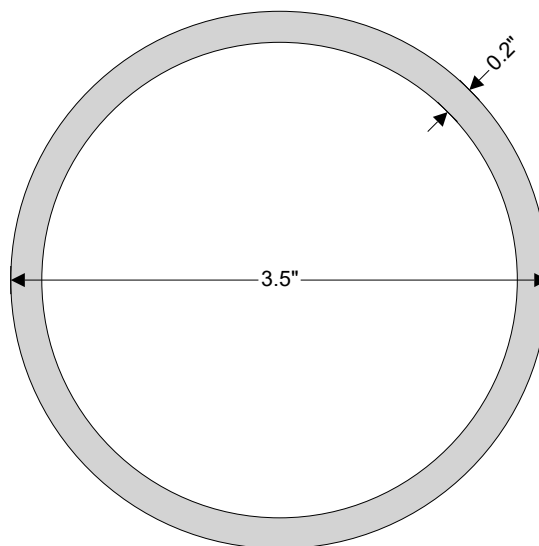
$$\delta_x / \delta_{x,Allowable} = 0.39$$

PASS - Allowable deflection exceeds instantaneous deflection

EXISTING STEEL COLUMN - END SUPPORT - GRID LINE B-5

In accordance with AISC360-16 and the LRFD method

Tedds calculation version 1.0.11





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Revision:	Date:	Sheet No. 324
Project Name: JKV-EMS		

Column and loading details

Column details

Column section;

Pipe STD x3

Design loading

Required axial strength;

$P_r = 16$ kips; (Compression)

Moment about x axis at end 1;

$M_{x1} = 0.0$ kips_ft

Moment about x axis at end 2;

$M_{x2} = 0.0$ kips_ft

Maximum moment about x axis;

$M_x = \max(\text{abs}(M_{x1}), \text{abs}(M_{x2})) = 0.0$ kips_ft

Moment about y axis at end 1;

$M_{y1} = 0.0$ kips_ft

Moment about y axis at end 2;

$M_{y2} = 0.0$ kips_ft

Maximum moment about y axis;

$M_y = \max(\text{abs}(M_{y1}), \text{abs}(M_{y2})) = 0.0$ kips_ft

Maximum shear force parallel to y axis;

$V_{ry} = 0.0$ kips

Maximum shear force parallel to x axis;

$V_{rx} = 0.0$ kips

Material details

Steel grade;

A53 Gr. B

Yield strength;

$F_y = 35$ ksi

Ultimate strength;

$F_u = 60$ ksi

Modulus of elasticity;

$E = 29000$ ksi

Shear modulus of elasticity;

$G = 11200$ ksi

Unbraced lengths

For buckling about x axis;

$L_x = 120$ in

For buckling about y axis;

$L_y = 120$ in

For torsional buckling;

$L_z = 120$ in

Effective length factors

For buckling about x axis;

$K_x = 1.00$

For buckling about y axis;

$K_y = 1.00$

For torsional buckling;

$K_z = 1.00$

Effective unbraced lengths

For buckling about x axis;

$L_{cx} = L_x \times K_x = 120$ in

For buckling about y axis;

$L_{cy} = L_y \times K_y = 120$ in

For torsional buckling;

$L_{cz} = L_z \times K_z = 120$ in

Section classification

Section classification for local buckling (cl. B4)

Width to thickness ratio;

$\lambda = D_o / t = 17.413$

Compression

Limit for nonslender section;

$\lambda_{r_c} = 0.11 \times E / F_y = 91.143$

The section is nonslender in compression



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 325
Project Name: JKV-EMS		

Slenderness

Member slenderness

Slenderness ratio about x axis;

$$SR_x = L_{cx} / r_x = 102.6$$

Slenderness ratio about y axis;

$$SR_y = L_{cy} / r_y = 102.6$$

Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress;

$$F_{ex} = \pi^2 \times E / (SR_x)^2 = 27.2 \text{ ksi}$$

Flexural buckling stress;

$$F_{crx} = (0.658^{F_y / F_{ex}}) \times F_y = 20.4 \text{ ksi}$$

Nominal compressive strength for flexural buckling; $P_{nx} = F_{crx} \times A = 42.3 \text{ kips}$

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress;

$$F_{ey} = \pi^2 \times E / (SR_y)^2 = 27.2 \text{ ksi}$$

Flexural buckling stress;

$$F_{cry} = (0.658^{F_y / F_{ey}}) \times F_y = 20.4 \text{ ksi}$$

Nominal compressive strength for flexural buckling; $P_{ny} = F_{cry} \times A = 42.3 \text{ kips}$

Design compressive strength (cl.E1)

Resistance factor for compression;

$$\phi_c = 0.90$$

Design compressive strength;

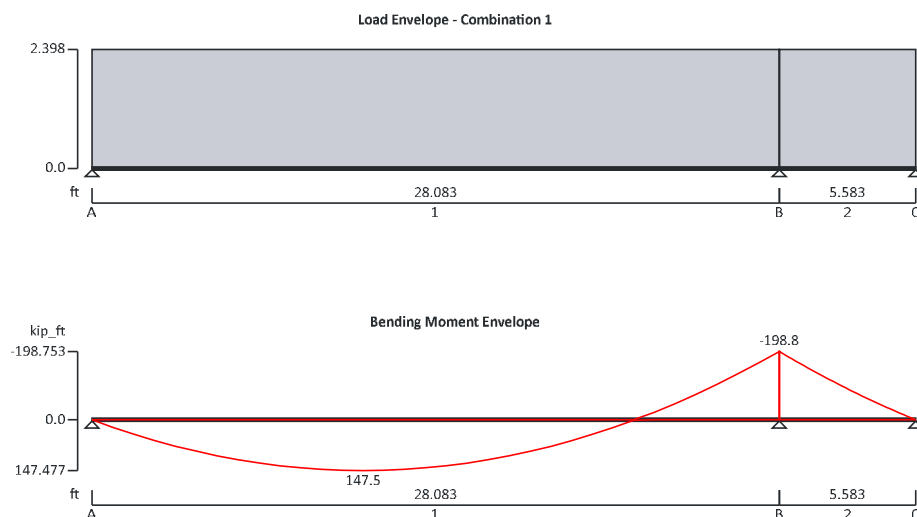
$$P_c = \phi_c \times \min(P_{nx}, P_{ny}) = 38.1 \text{ kips}$$

PASS - The design compressive strength exceeds the required compressive strength

STEEL BEAM ANALYSIS & DESIGN (AISC360-16)

In accordance with AISC360-16 using the LRFD method

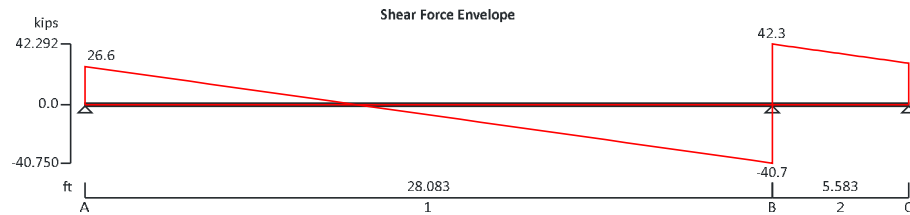
Tedds calculation version 3.0.16





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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 326
Project Name: JKV-EMS		



Support conditions

Support A	Vertically restrained Rotationally free
Support B	Vertically restrained Rotationally free
Support C	Vertically restrained Rotationally free

Applied loading

Beam loads	Dead self weight of beam $\times 1$ Dead full UDL 0.93 kips/ft Live full UDL 0.775 kips/ft

Load combinations

Load combination 1	Support A	Dead $\times 1.20$ Live $\times 1.60$ Dead $\times 1.20$ Live $\times 1.60$
		Dead $\times 1.20$ Live $\times 1.60$ Dead $\times 1.20$ Live $\times 1.60$
		Dead $\times 1.20$ Live $\times 1.60$
	Support B	Dead $\times 1.20$ Live $\times 1.60$ Dead $\times 1.20$ Live $\times 1.60$
		Dead $\times 1.20$ Live $\times 1.60$
	Support C	Dead $\times 1.20$ Live $\times 1.60$

Analysis results

Maximum moment;	$M_{\max} = 147.5$ kips_ft;	$M_{\min} = -198.8$ kips_ft
Maximum moment span 1;	$M_{s1\_max} = 147.5$ kips_ft;	$M_{s1\_min} = -198.8$ kips_ft
Maximum moment span 1 segment 1;	$M_{s1\_seg1\_max} = 127.6$ kips_ft;	$M_{s1\_seg1\_min} = 0$ kips_ft
Maximum moment span 1 segment 2;	$M_{s1\_seg2\_max} = 147.5$ kips_ft;	$M_{s1\_seg2\_min} = 0$ kips_ft
Maximum moment span 1 segment 3;	$M_{s1\_seg3\_max} = 137$ kips_ft;	$M_{s1\_seg3\_min} = 0$ kips_ft
Maximum moment span 1 segment 4;	$M_{s1\_seg4\_max} = 28.2$ kips_ft;	$M_{s1\_seg4\_min} = -198.8$ kips_ft
Maximum moment span 2;	$M_{s2\_max} = 0$ kips_ft;	$M_{s2\_min} = -198.8$ kips_ft
Maximum shear;	$V_{\max} = 42.3$ kips;	$V_{\min} = -40.7$ kips
Maximum shear span 1;	$V_{s1\_max} = 26.6$ kips;	$V_{s1\_min} = -40.7$ kips
Maximum shear span 1 segment 1;	$V_{s1\_seg1\_max} = 26.6$ kips;	$V_{s1\_seg1\_min} = 0$ kips
Maximum shear span 1 segment 2;	$V_{s1\_seg2\_max} = 9.8$ kips;	$V_{s1\_seg2\_min} = -7.1$ kips
Maximum shear span 1 segment 3;	$V_{s1\_seg3\_max} = 0$ kips;	$V_{s1\_seg3\_min} = -23.9$ kips



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 327
Project Name: JKV-EMS		

Maximum shear span 1 segment 4;

Maximum shear span 2;

Deflection;

Deflection span 1;

Deflection span 2;

Maximum reaction at support A;

Unfactored dead load reaction at support A;

Unfactored live load reaction at support A;

Maximum reaction at support B;

Unfactored dead load reaction at support B;

Unfactored live load reaction at support B;

Maximum reaction at support C;

Unfactored dead load reaction at support C;

Unfactored live load reaction at support C;

$V_{s1\_seg4\_max} = 0$ kips;

$V_{s2\_max} = 42.3$ kips;

$\delta_{max} = 0.8$ in;

$\delta_{s1\_max} = 0.8$ in;

$\delta_{s2\_max} = 0$ in;

$R_{A\_max} = 26.6$ kips;

$R_{A\_Dead} = 10.7$ kips

$R_{A\_Live} = 8.6$ kips

$R_{B\_max} = 83$ kips;

$R_{B\_Dead} = 33.4$ kips

$R_{B\_Live} = 26.8$ kips

$R_{C\_max} = -28.9$ kips;

$R_{C\_Dead} = -11.6$ kips

$R_{C\_Live} = -9.3$ kips

$V_{s1\_seg4\_min} = -40.7$ kips

$V_{s2\_min} = 28.9$ kips

$\delta_{min} = 0$ in

$\delta_{s1\_min} = 0$ in

$\delta_{s2\_min} = 0$ in

$R_{A\_min} = 26.6$ kips

$R_{B\_min} = 83$ kips

$R_{C\_min} = -28.9$ kips

Section details

Section type;

ASTM steel designation;

Steel yield stress;

Steel tensile stress;

Modulus of elasticity;

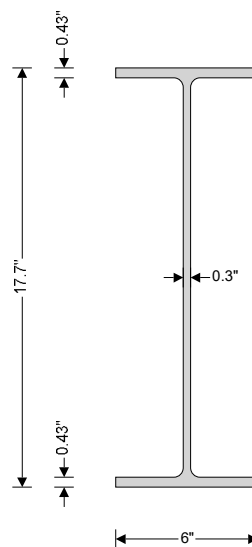
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$F_y = 50$ ksi

$F_u = 65$ ksi

$E = 29000$ ksi



Resistance factors

Resistance factor for tensile yielding

$\phi_{ty} = 0.90$

Resistance factor for tensile rupture

$\phi_{tr} = 0.75$

Resistance factor for compression

$\phi_c = 0.90$

Resistance factor for flexure

$\phi_b = 0.90$

Lateral bracing

Span 1 has lateral bracing at supports plus quarter points

Span 2 has continuous lateral bracing



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Revision:	Date:	Sheet No. 328
Project Name: JKV-EMS		

Classification of sections for local buckling - Section B4.1

Classification of flanges in flexure - Table B4.1b (case 10)

Width to thickness ratio; $b_f / (2 \times t_f) = 7.06$
Limiting ratio for compact section; $\lambda_{pff} = 0.38 \times \sqrt{[E / F_y]} = 9.15$
Limiting ratio for non-compact section; $\lambda_{rff} = 1.0 \times \sqrt{[E / F_y]} = 24.08$; Compact

Classification of web in flexure - Table B4.1b (case 15)

Width to thickness ratio; $(d - 2 \times k) / t_w = 53.49$
Limiting ratio for compact section; $\lambda_{pwf} = 3.76 \times \sqrt{[E / F_y]} = 90.55$
Limiting ratio for non-compact section; $\lambda_{rwf} = 5.70 \times \sqrt{[E / F_y]} = 137.27$; Compact

Section is compact in flexure

Design of members for shear - Chapter G

Required shear strength $V_r = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 42.292$ kips
Web area $A_w = d \times t_w = 5.31$ in²
Web plate buckling coefficient $k_v = 5.34$
Web shear coefficient - eq G2-3 $C_{v1} = 1$
Nominal shear strength – eq G6-1 $V_n = 0.6 \times F_y \times A_w \times C_{v1} = 159.300$ kips
Resistance factor for shear $\phi_v = 1.00$
Design shear strength $V_c = \phi_v \times V_n = 159.300$ kips

PASS - Design shear strength exceeds required shear strength

Design of members for flexure in the major axis at span 1 segment 4 - Chapter F

Required flexural strength; $M_r = \max(\text{abs}(M_{s1\_seg4\_max}), \text{abs}(M_{s1\_seg4\_min})) = 198.753$ kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1; $M_{nyld} = M_p = F_y \times Z_x = 277.083$ kips_ft

Lateral-torsional buckling - Section F2.2

Unbraced length; $L_b = L_{s1\_seg4} = 84.25$ in
Limiting unbraced length for yielding - eq F2-5; $L_p = 1.76 \times r_y \times \sqrt{[E / F_y]} = 51.711$ in
Distance between flange centroids; $h_o = d - t_f = 17.275$ in
 $c = 1$
 $r_{ts} = \sqrt{[(I_y \times C_w) / S_x]} = 1.514$ in

Limiting unbraced length for inelastic LTB - eq F2-6

$$L_r = 1.95 \times r_{ts} \times E / (0.7 \times F_y) \times \sqrt{[(J \times c / (S_x \times h_o)) + \sqrt{((J \times c / (S_x \times h_o))^2 + 6.76 \times (0.7 \times F_y / E)^2)}]} = 148.563 \text{ in}$$

Cross-section mono-symmetry parameter; $R_m = 1.000$
Moment at quarter point of segment; $M_A = 17.425$ kips_ft
Moment at center-line of segment; $M_B = 70.480$ kips_ft
Moment at three quarter point of segment; $M_C = 130.922$ kips_ft
Maximum moment in segment; $M_{abs} = 198.753$ kips_ft
Lateral torsional buckling modification factor - eq F1-1; $C_b = 12.5 \times M_{abs} / [2.5 \times M_{abs} + 3 \times M_A + 4 \times M_B + 3 \times M_C] = 2.030$
Nominal flexural strength for lateral torsional buckling - eq F2-2; $M_{nlb} = C_b \times [M_p - (M_p - 0.7 \times F_y \times S_x) \times (L_b - L_p) / (L_r - L_p)] = 488.087$ kips_ft
Nominal flexural strength; $M_n = \min(M_{nyld}, M_{nlb}) = 277.083$ kips_ft
Design flexural strength; $M_c = \phi_b \times M_n = 249.375$ kips_ft



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Revision:	Date:	Sheet No. 329
Project Name: JKV-EMS		

PASS - Design flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead and live loads

Limiting deflection;

$$\delta_{lim} = L_{s1} / 325 = 1.037 \text{ in}$$

Maximum deflection span 1;

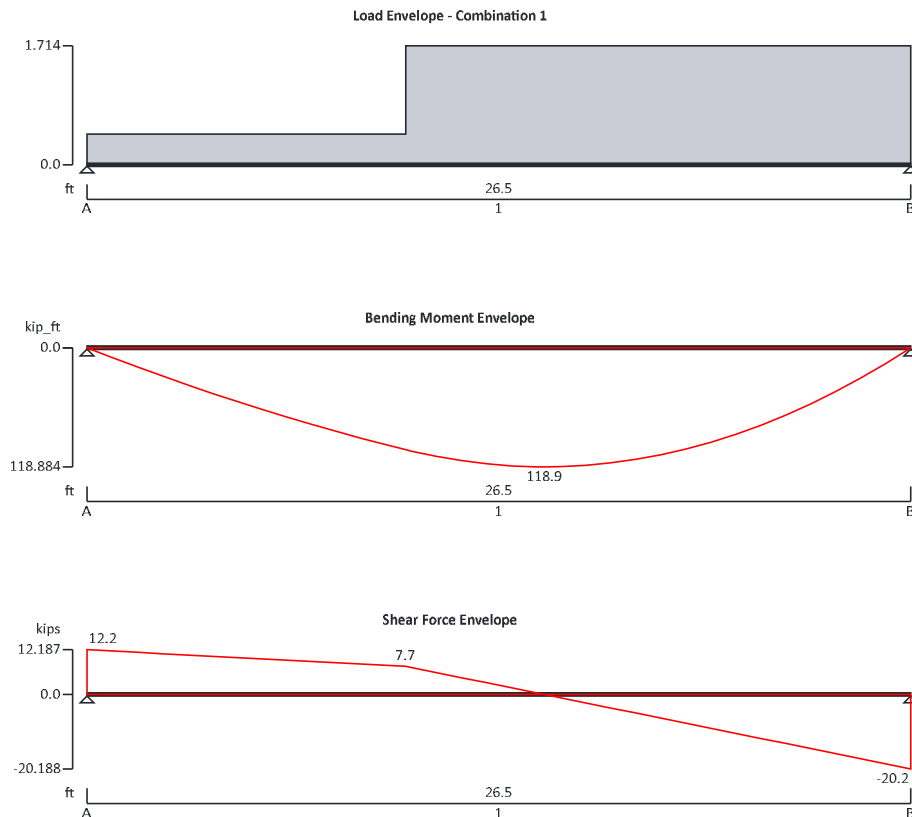
$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 0.832 \text{ in}$$

PASS - Maximum deflection does not exceed deflection limit

NEW STEEL BEAM SUPPORT IN EXISTING - GRID LINE B - NO POINT LOAD

In accordance with AISC360-16 using the LRFD method

Tedds calculation version 3.0.16



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\times$ 1



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Revision:	Date:	Sheet No. 330
Project Name: JKV-EMS		

Load combinations

Load combination 1

Support A

Dead $\times$ 1.20

Live $\times$ 1.60

Dead $\times$ 1.20

Live $\times$ 1.60

Support B

Dead $\times$ 1.20

Live $\times$ 1.60

Analysis results

Maximum moment;

Maximum moment span 1 segment 1;

Maximum moment span 1 segment 2;

Maximum moment span 1 segment 3;

Maximum moment span 1 segment 4;

Maximum shear;

Maximum shear span 1 segment 1;

Maximum shear span 1 segment 2;

Maximum shear span 1 segment 3;

Maximum shear span 1 segment 4;

Deflection segment 5;

Maximum reaction at support A;

Unfactored dead load reaction at support A;

Unfactored live load reaction at support A;

Maximum reaction at support B;

Unfactored dead load reaction at support B;

Unfactored live load reaction at support B;

$M_{max} = 118.9$ kips_ft;

$M_{s1\_seg1\_max} = 71.1$ kips_ft;

$M_{s1\_seg2\_max} = 117$ kips_ft;

$M_{s1\_seg3\_max} = 118.9$ kips_ft;

$M_{s1\_seg4\_max} = 96.1$ kips_ft;

$V_{max} = 12.2$ kips;

$V_{s1\_seg1\_max} = 12.2$ kips;

$V_{s1\_seg2\_max} = 9.3$ kips;

$V_{s1\_seg3\_max} = 2.5$ kips;

$V_{s1\_seg4\_max} = 0$ kips;

$\delta_{max} = 0.7$ in;

$R_{A\_max} = 12.2$ kips;

$R_{A\_Dead} = 5.1$ kips

$R_{A\_Live} = 3.8$ kips

$R_{B\_max} = 20.2$ kips;

$R_{B\_Dead} = 8.2$ kips

$R_{B\_Live} = 6.5$ kips

$M_{min} = 0$ kips_ft

$M_{s1\_seg1\_min} = 0$ kips_ft

$M_{s1\_seg2\_min} = 0$ kips_ft

$M_{s1\_seg3\_min} = 0$ kips_ft

$M_{s1\_seg4\_min} = 0$ kips_ft

$V_{min} = -20.2$ kips

$V_{s1\_seg1\_min} = 0$ kips

$V_{s1\_seg2\_min} = 0$ kips

$V_{s1\_seg3\_min} = -8.8$ kips

$V_{s1\_seg4\_min} = -20.2$ kips

$\delta_{min} = 0$ in

$R_{A\_min} = 12.2$ kips

$R_{B\_min} = 20.2$ kips

Section details

Section type;

ASTM steel designation;

Steel yield stress;

Steel tensile stress;

Modulus of elasticity;

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$F_y = 50$ ksi

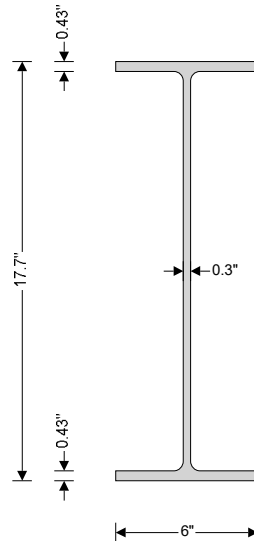
$F_u = 65$ ksi

$E = 29000$ ksi



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Revision:	Date:	Sheet No. 331
Project Name: JKV-EMS		



Resistance factors

Resistance factor for tensile yielding	$\phi_{ty} = 0.90$
Resistance factor for tensile rupture	$\phi_{tr} = 0.75$
Resistance factor for compression	$\phi_c = 0.90$
Resistance factor for flexure	$\phi_b = 0.90$

Lateral bracing

Span 1 has lateral bracing at supports plus quarter points

Classification of sections for local buckling - Section B4.1

Classification of flanges in flexure - Table B4.1b (case 10)

Width to thickness ratio;	$b_f / (2 \times t_f) = 7.06$	
Limiting ratio for compact section;	$\lambda_{pff} = 0.38 \times \sqrt{E / F_y} = 9.15$	
Limiting ratio for non-compact section;	$\lambda_{rff} = 1.0 \times \sqrt{E / F_y} = 24.08$;	Compact

Classification of web in flexure - Table B4.1b (case 15)

Width to thickness ratio;	$(d - 2 \times k) / t_w = 53.49$	
Limiting ratio for compact section;	$\lambda_{pwf} = 3.76 \times \sqrt{E / F_y} = 90.55$	
Limiting ratio for non-compact section;	$\lambda_{rwf} = 5.70 \times \sqrt{E / F_y} = 137.27$;	Compact

Section is compact in flexure

Design of members for shear - Chapter G

Required shear strength	$V_r = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 20.188$ kips
Web area	$A_w = d \times t_w = 5.31$ in ²
Web plate buckling coefficient	$k_v = 5.34$
Web shear coefficient - eq G2-3	$C_{v1} = 1$
Nominal shear strength – eq G6-1	$V_n = 0.6 \times F_y \times A_w \times C_{v1} = 159.300$ kips
Resistance factor for shear	$\phi_v = 1.00$
Design shear strength	$V_c = \phi_v \times V_n = 159.300$ kips

PASS - Design shear strength exceeds required shear strength



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Revision:	Date:	Sheet No. 332
Project Name: JKV-EMS		

Design of members for flexure in the major axis at span 1 segment 3 - Chapter F

Required flexural strength; $M_r = \max(\text{abs}(M_{s1\_seg3\_max}), \text{abs}(M_{s1\_seg3\_min})) = 118.884 \text{ kips\_ft}$

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1; $M_{nyld} = M_p = F_y \times Z_x = 277.083 \text{ kips\_ft}$

Lateral-torsional buckling - Section F2.2

Unbraced length; $L_b = L_{s1\_seg3} = 79.5 \text{ in}$

Limiting unbraced length for yielding - eq F2-5; $L_p = 1.76 \times r_y \times \sqrt{E / F_y} = 51.711 \text{ in}$

Distance between flange centroids; $h_o = d - t_f = 17.275 \text{ in}$

$c = 1$

$r_{ts} = \sqrt{[(I_y \times C_w) / S_x]} = 1.514 \text{ in}$

Limiting unbraced length for inelastic LTB - eq F2-6

$L_r = 1.95 \times r_{ts} \times E / (0.7 \times F_y) \times \sqrt{[(J \times c / (S_x \times h_o)) + ((J \times c / (S_x \times h_o))^2 + 6.76 \times (0.7 \times F_y / E)^2)]} = 148.563 \text{ in}$

Cross-section mono-symmetry parameter; $R_m = 1.000$

Moment at quarter point of segment; $M_A = 118.855 \text{ kips\_ft}$

Moment at center-line of segment; $M_B = 115.981 \text{ kips\_ft}$

Moment at three quarter point of segment; $M_C = 108.406 \text{ kips\_ft}$

Maximum moment in segment; $M_{abs} = 118.884 \text{ kips\_ft}$

Lateral torsional buckling modification factor - eq F1-1; $C_b = 12.5 \times M_{abs} / [2.5 \times M_{abs} + 3 \times M_A + 4 \times M_B + 3 \times M_C] = 1.030$

Nominal flexural strength for lateral torsional buckling - eq F2-2; $M_{nlb} = C_b \times [M_p - (M_p - 0.7 \times F_y \times S_x) \times (L_b - L_p) / (L_r - L_p)] = 253.132 \text{ kips\_ft}$

Nominal flexural strength; $M_n = \min(M_{nyld}, M_{nlb}) = 253.132 \text{ kips\_ft}$

Design flexural strength; $M_c = \phi_b \times M_n = 227.819 \text{ kips\_ft}$

PASS - Design flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead and live loads

Limiting deflection; $\delta_{lim} = L_{s1} / 325 = 0.978 \text{ in}$

Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 0.712 \text{ in}$

PASS - Maximum deflection does not exceed deflection limit



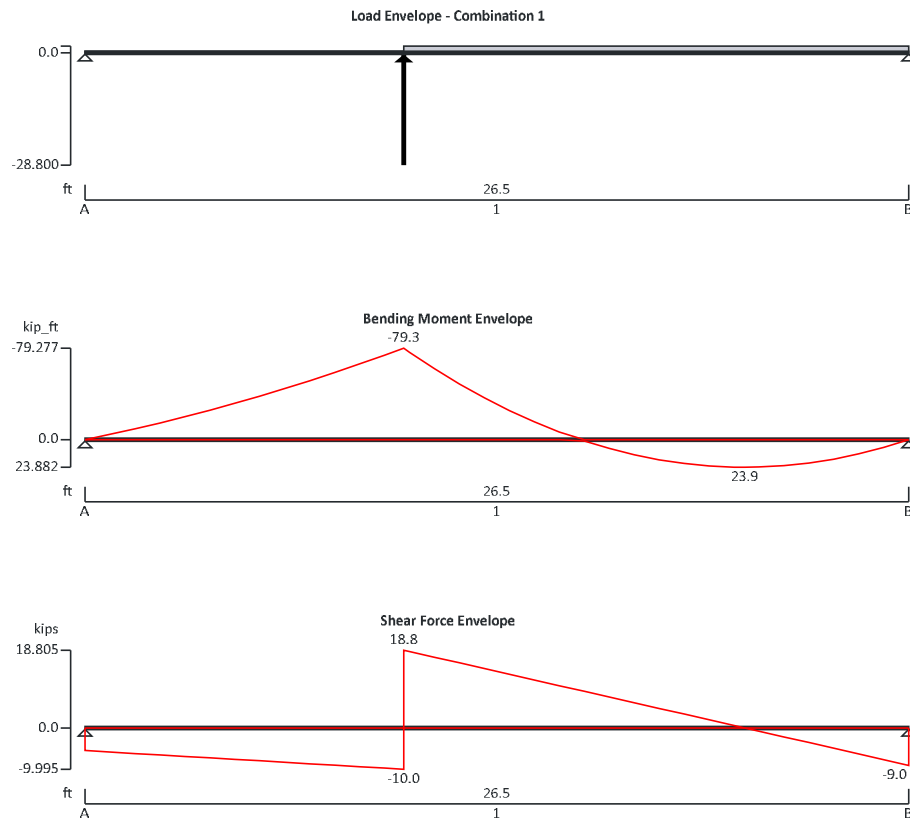
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Revision:	Date:	Sheet No. 333
Project Name: JKV-EMS		

NEW STEEL BEAM SUPPORT IN EXISTING - GRID LINE B - WITH POINT LOAD

In accordance with AISC360-16 using the LRFD method

Tedds calculation version 3.0.16



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\times 1$

WD1 - Dead partial UDL 0.158 kips/ft from 0.00 in to 123.00 in

WD2 - Dead partial UDL 0.66 kips/ft from 123.00 in to 318.00 in

WL1 - Live partial UDL 0.131 kips/ft from 0.00 in to 123.00 in

WL2 - Live partial UDL 0.55 kips/ft from 123.00 in to 318.00 in

Dead point load -11.6 kips at 123.00 in

Live point load -9.3 kips at 123.00 in

Load combinations

Load combination 1

Support A

Dead $\times 1.20$

Live $\times 1.60$



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 334
Project Name: JKV-EMS		

Analysis results

Maximum moment;
Maximum moment span 1 segment 1;
Maximum moment span 1 segment 2;
Maximum moment span 1 segment 3;
Maximum moment span 1 segment 4;
Maximum shear;
Maximum shear span 1 segment 1;
Maximum shear span 1 segment 2;
Maximum shear span 1 segment 3;
Maximum shear span 1 segment 4;
Deflection segment 5;
Maximum reaction at support A;
Unfactored dead load reaction at support A;
Unfactored live load reaction at support A;
Maximum reaction at support B;
Unfactored dead load reaction at support B;
Unfactored live load reaction at support B;

Section details

Section type;
ASTM steel designation;
Steel yield stress;
Steel tensile stress;
Modulus of elasticity;

Support B

$M_{max} = 23.9$ kips_ft;
 $M_{s1\_seg1\_max} = 0$ kips_ft;
 $M_{s1\_seg2\_max} = 0$ kips_ft;
 $M_{s1\_seg3\_max} = 22.3$ kips_ft;
 $M_{s1\_seg4\_max} = 23.9$ kips_ft;
 $V_{max} = 18.8$ kips;
 $V_{s1\_seg1\_max} = 0$ kips;
 $V_{s1\_seg2\_max} = 18.8$ kips;
 $V_{s1\_seg3\_max} = 13.7$ kips;
 $V_{s1\_seg4\_max} = 2.3$ kips;
 $\delta_{max} = 0$ in;
 $R_{A\_max} = -5.5$ kips;
 $R_{A\_Dead} = -2.1$ kips
 $R_{A\_Live} = -1.9$ kips
 $R_{B\_max} = 9$ kips;
 $R_{B\_Dead} = 3.7$ kips
 $R_{B\_Live} = 2.9$ kips

Dead $\times 1.20$

Live $\times 1.60$

Dead $\times 1.20$

Live $\times 1.60$

$M_{min} = -79.3$ kips_ft
 $M_{s1\_seg1\_min} = -45.9$ kips_ft
 $M_{s1\_seg2\_min} = -79.3$ kips_ft
 $M_{s1\_seg3\_min} = -30.6$ kips_ft
 $M_{s1\_seg4\_min} = 0$ kips_ft
 $V_{min} = -10$ kips
 $V_{s1\_seg1\_min} = -8.4$ kips
 $V_{s1\_seg2\_min} = -8.4$ kips
 $V_{s1\_seg3\_min} = 0$ kips
 $V_{s1\_seg4\_min} = -9$ kips
 $\delta_{min} = 0.2$ in
 $R_{A\_min} = -5.5$ kips

$R_{B\_min} = 9$ kips

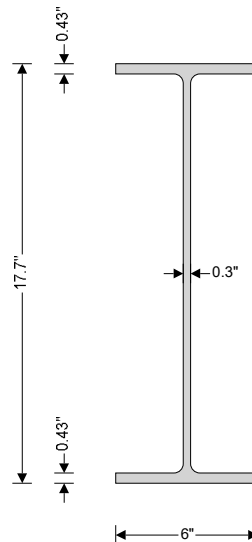
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$F_y = 50$ ksi

$F_u = 65$ ksi

$E = 29000$ ksi





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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 335
Project Name: JKV-EMS		

Resistance factors

Resistance factor for tensile yielding	$\phi_{ty} = 0.90$
Resistance factor for tensile rupture	$\phi_{tr} = 0.75$
Resistance factor for compression	$\phi_c = 0.90$
Resistance factor for flexure	$\phi_b = 0.90$

Lateral bracing

Span 1 has lateral bracing at supports plus quarter points

Classification of sections for local buckling - Section B4.1

Classification of flanges in flexure - Table B4.1b (case 10)

Width to thickness ratio;	$b_f / (2 \times t_f) = 7.06$	
Limiting ratio for compact section;	$\lambda_{pff} = 0.38 \times \sqrt{E / F_y} = 9.15$	
Limiting ratio for non-compact section;	$\lambda_{rff} = 1.0 \times \sqrt{E / F_y} = 24.08$;	Compact

Classification of web in flexure - Table B4.1b (case 15)

Width to thickness ratio;	$(d - 2 \times k) / t_w = 53.49$	
Limiting ratio for compact section;	$\lambda_{pwf} = 3.76 \times \sqrt{E / F_y} = 90.55$	
Limiting ratio for non-compact section;	$\lambda_{rwf} = 5.70 \times \sqrt{E / F_y} = 137.27$;	Compact

Section is compact in flexure

Design of members for shear - Chapter G

Required shear strength	$V_r = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 18.805$ kips
Web area	$A_w = d \times t_w = 5.31$ in ²
Web plate buckling coefficient	$k_v = 5.34$
Web shear coefficient - eq G2-3	$C_{v1} = 1$
Nominal shear strength – eq G6-1	$V_n = 0.6 \times F_y \times A_w \times C_{v1} = 159.300$ kips
Resistance factor for shear	$\phi_v = 1.00$
Design shear strength	$V_c = \phi_v \times V_n = 159.300$ kips

PASS - Design shear strength exceeds required shear strength

Design of members for flexure in the major axis at span 1 segment 2 - Chapter F

Required flexural strength;	$M_r = \max(\text{abs}(M_{s1\_seg2\_max}), \text{abs}(M_{s1\_seg2\_min})) = 79.277$ kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1;	$M_{nyld} = M_p = F_y \times Z_x = 277.083$ kips_ft

Lateral-torsional buckling - Section F2.2

Unbraced length;	$L_b = L_{s1\_seg2} = 79.5$ in
Limiting unbraced length for yielding - eq F2-5;	$L_p = 1.76 \times r_y \times \sqrt{E / F_y} = 51.711$ in
Distance between flange centroids;	$h_o = d - t_f = 17.275$ in
	$c = 1$
	$r_{ts} = \sqrt{[(I_y \times C_w) / S_x]} = 1.514$ in

Limiting unbraced length for inelastic LTB - eq F2-6

$$L_r = 1.95 \times r_{ts} \times E / (0.7 \times F_y) \times \sqrt{[(J \times c / (S_x \times h_o)) + ((J \times c / (S_x \times h_o))^2 + 6.76 \times (0.7 \times F_y / E)^2)]} = 148.563 \text{ in}$$

Cross-section mono-symmetry parameter;	$R_m = 1.000$

Moment at quarter point of segment;	$M_A = 60.454$ kips_ft



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 336
Project Name: JKV-EMS		

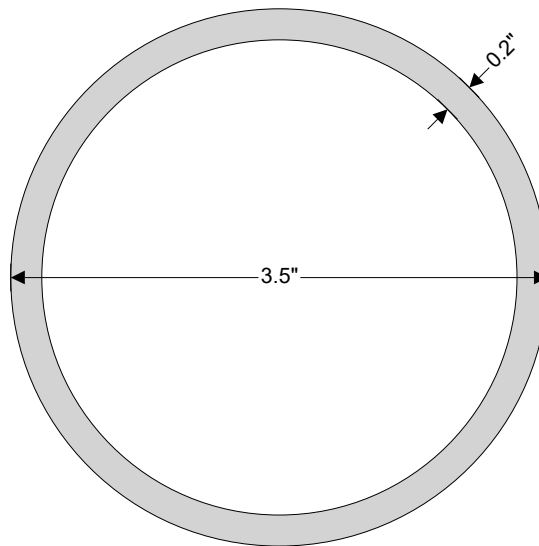
Moment at center-line of segment; $M_B = 76.175$ kips_ft
 Moment at three quarter point of segment; $M_C = 55.554$ kips_ft
 Maximum moment in segment; $M_{abs} = 79.277$ kips_ft
 Lateral torsional buckling modification factor - eq F1-1; $C_b = 12.5 \times M_{abs} / [2.5 \times M_{abs} + 3 \times M_A + 4 \times M_B + 3 \times M_C] = 1.165$
 Nominal flexural strength for lateral torsional buckling - eq F2-2; $M_{nltb} = C_b \times [M_p - (M_p - 0.7 \times F_y \times S_x) \times (L_b - L_p) / (L_r - L_p)] = 286.236$ kips_ft
 Nominal flexural strength; $M_n = \min(M_{nyld}, M_{nltb}) = 277.083$ kips_ft
 Design flexural strength; $M_c = \phi_b \times M_n = 249.375$ kips_ft
PASS - Design flexural strength exceeds required flexural strength

Design of members for vertical deflection
 Consider deflection due to dead and live loads
 Limiting deflection; $\delta_{lim} = L_{s1} / 325 = 0.978$ in
 Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 0.198$ in
PASS - Maximum deflection does not exceed deflection limit

NEW STEEL COLUMN - END SUPPORT - GRID LINE E-6

In accordance with AISC360-16 and the LRFD method

Tedds calculation version 1.0.11



Column and loading details

Column details

Column section;

Pipe STD x3

Design loading

Required axial strength;

$P_r = 27$ kips; (Compression)

Moment about x axis at end 1;

$M_{x1} = 0.0$ kips_ft

Moment about x axis at end 2;

$M_{x2} = 0.0$ kips_ft



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 337
Project Name: JKV-EMS		

Maximum moment about x axis;
Moment about y axis at end 1;
Moment about y axis at end 2;

$$M_x = \max(\text{abs}(M_{x1}), \text{abs}(M_{x2})) = 0.0 \text{ kips\_ft}$$
$$M_{y1} = 0.0 \text{ kips\_ft}$$
$$M_{y2} = 0.0 \text{ kips\_ft}$$

Maximum moment about y axis;
Maximum shear force parallel to y axis;
Maximum shear force parallel to x axis;

$$M_y = \max(\text{abs}(M_{y1}), \text{abs}(M_{y2})) = 0.0 \text{ kips\_ft}$$
$$V_{ry} = 0.0 \text{ kips}$$
$$V_{rx} = 0.0 \text{ kips}$$

Material details

Steel grade;
Yield strength;
Ultimate strength;
Modulus of elasticity;
Shear modulus of elasticity;

A53 Gr. B

$$F_y = 35 \text{ ksi}$$
$$F_u = 60 \text{ ksi}$$
$$E = 29000 \text{ ksi}$$
$$G = 11200 \text{ ksi}$$

Unbraced lengths

For buckling about x axis;
For buckling about y axis;
For torsional buckling;

$$L_x = 120 \text{ in}$$
$$L_y = 120 \text{ in}$$
$$L_z = 120 \text{ in}$$

Effective length factors

For buckling about x axis;
For buckling about y axis;
For torsional buckling;

$$K_x = 1.00$$
$$K_y = 1.00$$
$$K_z = 1.00$$

Effective unbraced lengths

For buckling about x axis;
For buckling about y axis;
For torsional buckling;

$$L_{cx} = L_x \times K_x = 120 \text{ in}$$
$$L_{cy} = L_y \times K_y = 120 \text{ in}$$
$$L_{cz} = L_z \times K_z = 120 \text{ in}$$

Section classification

Section classification for local buckling (cl. B4)

Width to thickness ratio;

$$\lambda = D_o / t = 17.413$$

Compression

Limit for nonslender section;

$$\lambda_{r\_c} = 0.11 \times E / F_y = 91.143$$

The section is nonslender in compression

Slenderness

Member slenderness

Slenderness ratio about x axis;
Slenderness ratio about y axis;

$$SR_x = L_{cx} / r_x = 102.6$$
$$SR_y = L_{cy} / r_y = 102.6$$

Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress;
Flexural buckling stress;

$$F_{ex} = \pi^2 \times E / (SR_x)^2 = 27.2 \text{ ksi}$$
$$F_{crx} = (0.658^{F_y / F_{ex}}) \times F_y = 20.4 \text{ ksi}$$



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 338
Project Name: JKV-EMS		

Nominal compressive strength for flexural buckling; $P_{nx} = F_{crx} \times A = 42.3$ kips

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress;

$$F_{ey} = \pi^2 \times E / (SR_y)^2 = 27.2 \text{ ksi}$$

Flexural buckling stress;

$$F_{cry} = (0.658^{F_y / F_{ey}}) \times F_y = 20.4 \text{ ksi}$$

Nominal compressive strength for flexural buckling; $P_{ny} = F_{cry} \times A = 42.3$ kips

Design compressive strength (cl.E1)

Resistance factor for compression;

$$\phi_c = 0.90$$

Design compressive strength;

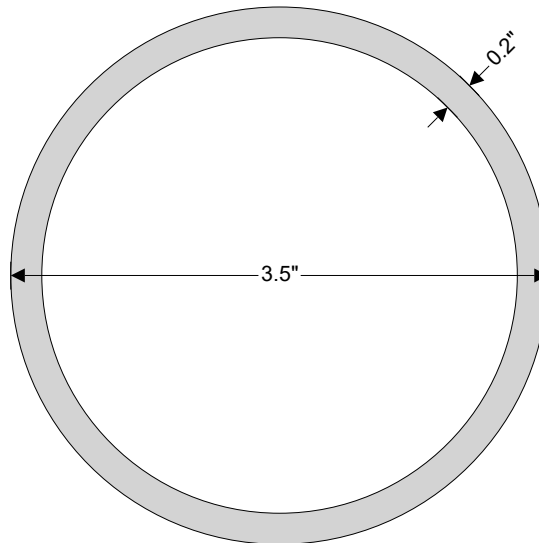
$$P_c = \phi_c \times \min(P_{nx}, P_{ny}) = 38.1 \text{ kips}$$

PASS - The design compressive strength exceeds the required compressive strength

NEW STEEL COLUMN - END SUPPORT - GRID LINE B-7.5

In accordance with AISC360-16 and the LRFD method

Tedds calculation version 1.0.11



Column and loading details

Column details

Column section;

Pipe STD x3

Design loading

Required axial strength;

$P_r = 20$ kips; (Compression)

Moment about x axis at end 1;

$M_{x1} = 0.0$ kips_ft

Moment about x axis at end 2;

$M_{x2} = 0.0$ kips_ft

Maximum moment about x axis;

$M_x = \max(\text{abs}(M_{x1}), \text{abs}(M_{x2})) = 0.0$ kips_ft

Moment about y axis at end 1;

$M_{y1} = 0.0$ kips_ft

Moment about y axis at end 2;

$M_{y2} = 0.0$ kips_ft



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 339
Project Name: JKV-EMS		

Maximum moment about y axis;
Maximum shear force parallel to y axis;
Maximum shear force parallel to x axis;

$$M_y = \max(\text{abs}(M_{y1}), \text{abs}(M_{y2})) = \mathbf{0.0 \text{ kips\_ft}}$$
$$V_{ry} = \mathbf{0.0 \text{ kips}}$$
$$V_{rx} = \mathbf{0.0 \text{ kips}}$$

Material details

Steel grade;
Yield strength;
Ultimate strength;
Modulus of elasticity;
Shear modulus of elasticity;

$$\mathbf{A53 \text{ Gr. B}}$$
$$F_y = \mathbf{35 \text{ ksi}}$$
$$F_u = \mathbf{60 \text{ ksi}}$$
$$E = \mathbf{29000 \text{ ksi}}$$
$$G = \mathbf{11200 \text{ ksi}}$$

Unbraced lengths

For buckling about x axis;
For buckling about y axis;
For torsional buckling;

$$L_x = \mathbf{120 \text{ in}}$$
$$L_y = \mathbf{120 \text{ in}}$$
$$L_z = \mathbf{120 \text{ in}}$$

Effective length factors

For buckling about x axis;
For buckling about y axis;
For torsional buckling;

$$K_x = \mathbf{1.00}$$
$$K_y = \mathbf{1.00}$$
$$K_z = \mathbf{1.00}$$

Effective unbraced lengths

For buckling about x axis;
For buckling about y axis;
For torsional buckling;

$$L_{cx} = L_x \times K_x = \mathbf{120 \text{ in}}$$
$$L_{cy} = L_y \times K_y = \mathbf{120 \text{ in}}$$
$$L_{cz} = L_z \times K_z = \mathbf{120 \text{ in}}$$

Section classification

Section classification for local buckling (cl. B4)

Width to thickness ratio;

$$\lambda = D_o / t = \mathbf{17.413}$$

Compression

Limit for nonslender section;

$$\lambda_{r-c} = 0.11 \times E / F_y = \mathbf{91.143}$$

The section is nonslender in compression

Slenderness

Member slenderness

Slenderness ratio about x axis;
Slenderness ratio about y axis;

$$SR_x = L_{cx} / r_x = \mathbf{102.6}$$
$$SR_y = L_{cy} / r_y = \mathbf{102.6}$$

Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress;

$$F_{ex} = \pi^2 \times E / (SR_x)^2 = \mathbf{27.2 \text{ ksi}}$$

Flexural buckling stress;

$$F_{crx} = (0.658^{F_y / F_{ex}}) \times F_y = \mathbf{20.4 \text{ ksi}}$$

Nominal compressive strength for flexural buckling; $P_{nx} = F_{crx} \times A = \mathbf{42.3 \text{ kips}}$

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress;

$$F_{ey} = \pi^2 \times E / (SR_y)^2 = \mathbf{27.2 \text{ ksi}}$$

Flexural buckling stress;

$$F_{cry} = (0.658^{F_y / F_{ey}}) \times F_y = \mathbf{20.4 \text{ ksi}}$$



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 340
Project Name: JKV-EMS		

Nominal compressive strength for flexural buckling; $P_{ny} = F_{cry} \times A = 42.3$ kips

Design compressive strength (cl.E1)

Resistance factor for compression;

$$\phi_c = 0.90$$

Design compressive strength;

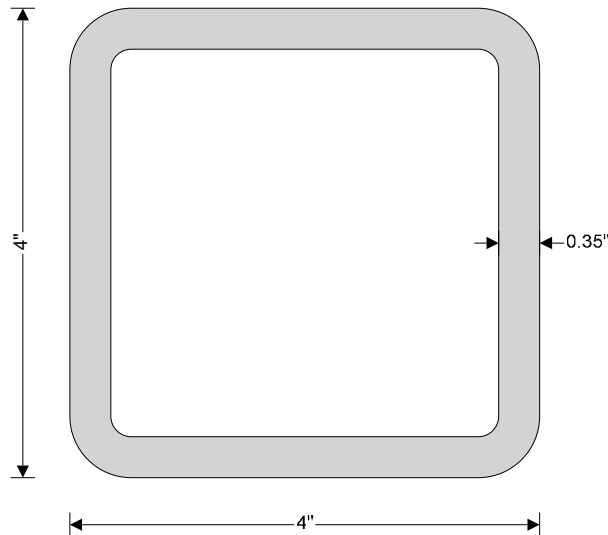
$$P_c = \phi_c \times \min(P_{nx}, P_{ny}) = 38.1 \text{ kips}$$

PASS - The design compressive strength exceeds the required compressive strength

NEW STEEL COLUMN - MID SUPPORT - GRID LINE B.4-6

In accordance with AISC360-16 and the LRFD method

Tedds calculation version 1.0.11



Column and loading details

Column details

Column section;

HSS 4x4x3/8

Design loading

Required axial strength;

$P_r = 83$ kips; (Compression)

Moment about x axis at end 1;

$M_{x1} = 0.0$ kips_ft

Moment about x axis at end 2;

$M_{x2} = 0.0$ kips_ft

Maximum moment about x axis;

$M_x = \max(\text{abs}(M_{x1}), \text{abs}(M_{x2})) = 0.0$ kips_ft

Moment about y axis at end 1;

$M_{y1} = 0.0$ kips_ft

Moment about y axis at end 2;

$M_{y2} = 0.0$ kips_ft

Maximum moment about y axis;

$M_y = \max(\text{abs}(M_{y1}), \text{abs}(M_{y2})) = 0.0$ kips_ft

Maximum shear force parallel to y axis;

$V_{ry} = 0.0$ kips

Maximum shear force parallel to x axis;

$V_{rx} = 0.0$ kips

Material details

Steel grade;

A500 Gr. B



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 341
Project Name: JKV-EMS		

Yield strength; $F_y = 46$ ksi
Ultimate strength; $F_u = 58$ ksi
Modulus of elasticity; $E = 29000$ ksi
Shear modulus of elasticity; $G = 11200$ ksi

Unbraced lengths

For buckling about x axis; $L_x = 120$ in
For buckling about y axis; $L_y = 120$ in
For torsional buckling; $L_z = 120$ in

Effective length factors

For buckling about x axis; $K_x = 1.00$
For buckling about y axis; $K_y = 1.00$
For torsional buckling; $K_z = 1.00$

Effective unbraced lengths

For buckling about x axis; $L_{cx} = L_x \times K_x = 120$ in
For buckling about y axis; $L_{cy} = L_y \times K_y = 120$ in
For torsional buckling; $L_{cz} = L_z \times K_z = 120$ in

Section classification

Section classification for local buckling (cl. B4)

Critical flange width; $b = b_f - 3 \times t = 2.953$ in
Critical web width; $h = d - 3 \times t = 2.953$ in
Width to thickness ratio of flange (compression); $\lambda_{f_c} = b / t = 8.461$
Width to thickness ratio of web (compression); $\lambda_{w_c} = h / t = 8.461$
Width to thickness ratio of flange (major flexure); $\lambda_{f_{fx}} = b / t = 8.461$
Width to thickness ratio of web (major flexure); $\lambda_{w_{fx}} = h / t = 8.461$
Width to thickness ratio of flange (minor flexure); $\lambda_{f_{fy}} = h / t = 8.461$
Width to thickness ratio of web (minor flexure); $\lambda_{w_{fy}} = b / t = 8.461$

Compression

Limit for nonslender section; $\lambda_{r_c} = 1.40 \times \sqrt{(E / F_y)} = 35.152$

The section is nonslender in compression

Slenderness

Member slenderness

Slenderness ratio about x axis; $SR_x = L_{cx} / r_x = 81.6$
Slenderness ratio about y axis; $SR_y = L_{cy} / r_y = 81.6$

Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress; $F_{ex} = \pi^2 \times E / (SR_x)^2 = 43$ ksi
Flexural buckling stress; $F_{crx} = (0.658^{F_y / F_{ex}}) \times F_y = 29.4$ ksi
Nominal compressive strength for flexural buckling; $P_{nx} = F_{crx} \times A = 140.4$ kips



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 342
Project Name: JKV-EMS		

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress;

$$F_{ey} = \pi^2 \times E / (SR_y)^2 = 43 \text{ ksi}$$

Flexural buckling stress;

$$F_{cry} = (0.658^{F_y / F_{ey}}) \times F_y = 29.4 \text{ ksi}$$

Nominal compressive strength for flexural buckling; $P_{ny} = F_{cry} \times A = 140.4$ kips

Design compressive strength (cl.E1)

Resistance factor for compression;

$$\phi_c = 0.90$$

Design compressive strength;

$$P_c = \phi_c \times \min(P_{nx}, P_{ny}) = 126.4; \text{ kips}$$

PASS - The design compressive strength exceeds the required compressive strength

FOOTING AT EXISTING COLUMN - GRID LINE B-5

In accordance with ACI318-14

Tedds calculation version 3.3.11

Summary results

Overall design status;

FAIL; Min. area of reinf., bot, x-direction

Overall design utilisation;

1.296 (Doesn't meet current ACI minimum areas of reinforcement. Strength is adequate)

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	13.2			Pass
Description	Unit	Applied	Resisting	Utilization	Result
Soil bearing	ksf	1.469	1.5	0.979	Pass
Description	Unit	Required	Provided	Utilization	Result
Moment, positive, x-direction	kip_ft	3.4	23.2	0.148	Pass
Moment, positive, y-direction	kip_ft	3.4	21.9	0.157	Pass
Shear, one-way, x-direction	kips	2.4	29.9	0.080	Pass
Shear, one-way, y-direction	kips	2.4	28.2	0.084	Pass
Shear, two-way, Col 1	psi	20.927	189.737	0.110	Pass
Min.area of reinf, bot., x-direction	in ²	0.778	0.600		Fail
Max.reinf.spacing, bot, x-direction	in	18.0	14.7		Pass
Min.area of reinf, bot., y-direction	in ²	0.778	0.600		Fail
Max.reinf.spacing, bot, y-direction	in	18.0	14.7		Pass

Pad footing details

Length of footing;

$$L_x = 3 \text{ ft}$$

Width of footing;

$$L_y = 3 \text{ ft}$$

Footing area;

$$A = L_x \times L_y = 9 \text{ ft}^2$$

Depth of footing;

$$h = 12 \text{ in}$$

Depth of soil over footing;

$$h_{soil} = 0 \text{ in}$$

Density of concrete;

$$\gamma_{conc} = 150.0 \text{ lb/ft}^3$$



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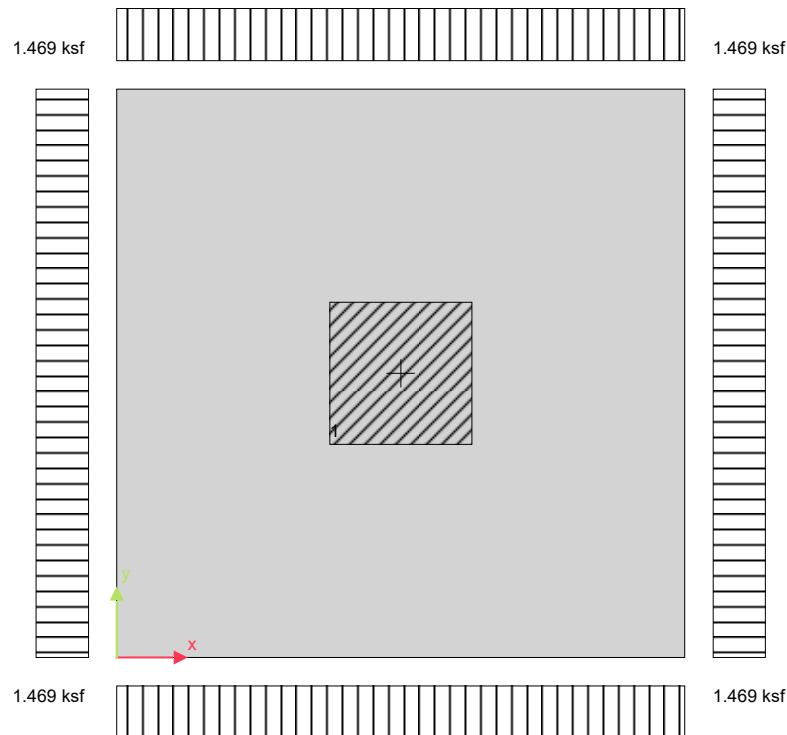
Job No. 25-003

Revision:

Date:

Sheet No. 343

Project Name: JKV-EMS



Column no.1 details

Length of column;

$$l_{x1} = 9.00 \text{ in}$$

Width of column;

$$l_{y1} = 9.00 \text{ in}$$

position in x-axis;

$$x_1 = 18.00 \text{ in}$$

position in y-axis;

$$y_1 = 18.00 \text{ in}$$

Soil properties

Gross allowable bearing pressure;

$$q_{\text{allow\_Gross}} = 1.5 \text{ ksf};$$

Density of soil;

$$\gamma_{\text{soil}} = 120.0 \text{ lb/ft}^3$$

Angle of internal friction;

$$\phi_b = 30.0 \text{ deg}$$

Design base friction angle;

$$\delta_{bb} = 30.0 \text{ deg}$$

Coefficient of base friction;

$$\tan(\delta_{bb}) = 0.577$$

Footing loads

Self weight;

$$F_{\text{swt}} = h \times \gamma_{\text{conc}} = 150 \text{ psf}$$

Column no.1 loads

Dead load in z;

$$F_{Dz1} = 6.7 \text{ kips}$$

Live load in z;

$$F_{Lz1} = 5.2 \text{ kips}$$

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.598)

1.0D + 1.0L (0.979)



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 344
Project Name: JKV-EMS		

Combination 2 results: 1.0D + 1.0L

Forces on footing

Force in z-axis;

$$F_{dz} = \gamma_D \times A \times F_{swt} + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} = \mathbf{13.2 \text{ kips}}$$

Moments on footing

Moment in x-axis, about x is 0;

$$M_{dx} = \gamma_D \times A \times F_{swt} \times L_x / 2 + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) = \mathbf{19.8 \text{ kip\_ft}}$$

Moment in y-axis, about y is 0;

$$M_{dy} = \gamma_D \times A \times F_{swt} \times L_y / 2 + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) = \mathbf{19.8 \text{ kip\_ft}}$$

Uplift verification

Vertical force;

$$F_{dz} = \mathbf{13.22 \text{ kips}}$$

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis;

$$e_{dx} = M_{dx} / F_{dz} - L_x / 2 = \mathbf{0 \text{ in}}$$

Eccentricity of base reaction in y-axis;

$$e_{dy} = M_{dy} / F_{dz} - L_y / 2 = \mathbf{0 \text{ in}}$$

Pad base pressures

$$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{1.469 \text{ ksf}}$$

$$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{1.469 \text{ ksf}}$$

$$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{1.469 \text{ ksf}}$$

$$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{1.469 \text{ ksf}}$$

$$q_{\min} = \min(q_1, q_2, q_3, q_4) = \mathbf{1.469 \text{ ksf}}$$

$$q_{\max} = \max(q_1, q_2, q_3, q_4) = \mathbf{1.469 \text{ ksf}}$$

Minimum base pressure;

Maximum base pressure;

Allowable bearing capacity

Allowable bearing capacity;

$$q_{\text{allow}} = q_{\text{allow\_Gross}} = \mathbf{1.5 \text{ ksf}}$$

$$q_{\max} / q_{\text{allow}} = \mathbf{0.979}$$

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN

In accordance with ACI318-14

Tedds calculation version 3.3.11

Material details

Compressive strength of concrete;

$$f'_c = \mathbf{4000 \text{ psi}}$$

Yield strength of reinforcement;

$$f_y = \mathbf{60000 \text{ psi}}$$

Compression-controlled strain limit (21.2.2);

$$\epsilon_{ty} = \mathbf{0.00200}$$

Cover to top of footing;

$$c_{\text{nom\_t}} = \mathbf{3 \text{ in}}$$

Cover to side of footing;

$$c_{\text{nom\_s}} = \mathbf{3 \text{ in}}$$

Cover to bottom of footing;

$$c_{\text{nom\_b}} = \mathbf{3 \text{ in}}$$

Concrete type;

Normal weight

Concrete modification factor;

$$\lambda = \mathbf{1.00}$$

Column type;

Concrete



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 345
Project Name: JKV-EMS		

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.091)

1.2D + 1.6L + 0.5Lr (0.157)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on footing

Ultimate force in z-axis;

$$F_{uz} = \gamma_D \times A \times F_{swt} + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} = \mathbf{17.9 \text{ kips}}$$

Moments on footing

Ultimate moment in x-axis, about x is 0;

$$M_{ux} = \gamma_D \times A \times F_{swt} \times L_x / 2 + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) = \mathbf{26.9 \text{ kip\_ft}}$$

Ultimate moment in y-axis, about y is 0;

$$M_{uy} = \gamma_D \times A \times F_{swt} \times L_y / 2 + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) = \mathbf{26.9 \text{ kip\_ft}}$$

Eccentricity of base reaction

Eccentricity of base reaction in x-axis;

$$e_{ux} = M_{ux} / F_{uz} - L_x / 2 = \mathbf{0 \text{ in}}$$

Eccentricity of base reaction in y-axis;

$$e_{uy} = M_{uy} / F_{uz} - L_y / 2 = \mathbf{0 \text{ in}}$$

Pad base pressures

$$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{1.992 \text{ ksf}}$$

$$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{1.992 \text{ ksf}}$$

$$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{1.992 \text{ ksf}}$$

$$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{1.992 \text{ ksf}}$$

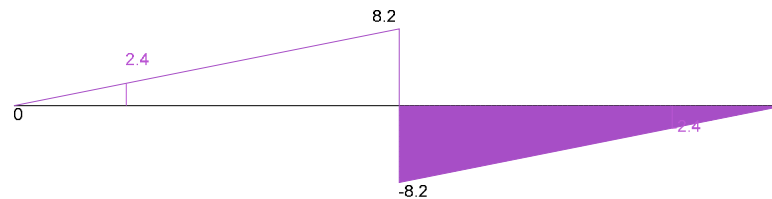
Minimum ultimate base pressure;

$$q_{umin} = \min(q_{u1}, q_{u2}, q_{u3}, q_{u4}) = \mathbf{1.992 \text{ ksf}}$$

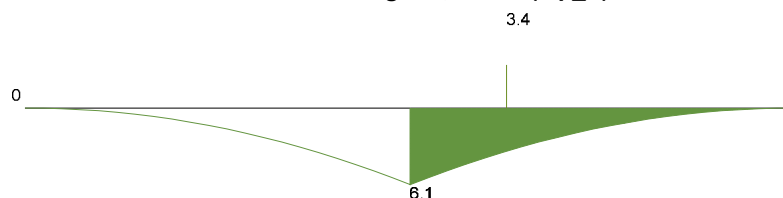
Maximum ultimate base pressure;

$$q_{umax} = \max(q_{u1}, q_{u2}, q_{u3}, q_{u4}) = \mathbf{1.992 \text{ ksf}}$$

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment;

$$M_{u.x,max} = \mathbf{3.439 \text{ kip\_ft}}$$

Tension reinforcement provided;

$$3 \text{ No.4 bottom bars (14.7 in c/c)}$$

Area of tension reinforcement provided;

$$A_{sx,bot,prov} = \mathbf{0.6 \text{ in}^2}$$

Minimum area of reinforcement (8.6.1.1);

$$A_{s,min} = 0.0018 \times L_y \times h = \mathbf{0.778 \text{ in}^2}$$



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Revision:	Date:	Sheet No. 346
Project Name: JKV-EMS		

FAIL - Minimum area of reinforcement required exceeds area of reinforcement provided

Maximum spacing of reinforcement (8.7.2.2); $s_{max} = \min(2 \times h, 18 \text{ in}) = 18 \text{ in}$

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement;

$$d = h - c_{nom\_b} - \phi_{x\_bot} / 2 = 8.750 \text{ in}$$

Depth of compression block;

$$a = A_{sx\_bot\_prov} \times f_y / (0.85 \times f'_c \times L_y) = 0.294 \text{ in}$$

Neutral axis factor;

$$\beta_1 = 0.85$$

Depth to neutral axis;

$$c = a / \beta_1 = 0.346 \text{ in}$$

Strain in tensile reinforcement;

$$\epsilon_t = 0.003 \times d / c - 0.003 = 0.07286$$

Minimum tensile strain(8.3.3.1);

$$\epsilon_{min} = 0.004 = 0.00400$$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity;

$$M_n = A_{sx\_bot\_prov} \times f_y \times (d - a / 2) = 25.809 \text{ kip\_ft}$$

Flexural strength reduction factor;

$$\phi_f = \min(\max(0.65 + 0.25 \times (\epsilon_t - \epsilon_{ty}) / (0.005 - \epsilon_{ty}), 0.65), 0.9) = 0.900$$

Design moment capacity;

$$\phi M_n = \phi_f \times M_n = 23.228 \text{ kip\_ft}$$

$$M_{u.x,max} / \phi M_n = 0.148$$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force;

$$V_{u.x} = 2.378 \text{ kips}$$

Depth to reinforcement;

$$d_v = h - c_{nom\_b} - \phi_{x\_bot} / 2 = 8.75 \text{ in}$$

Shear strength reduction factor;

$$\phi_v = 0.75$$

Nominal shear capacity (Eq. 22.5.5.1);

$$V_n = 2 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} \times L_y \times d_v = 39.845 \text{ kips}$$

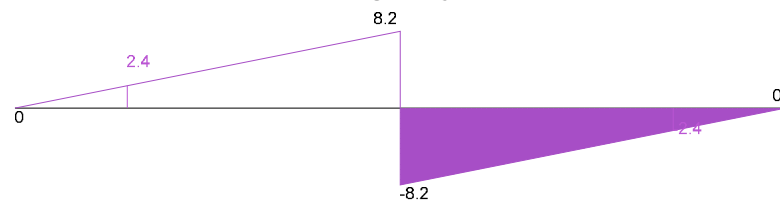
Design shear capacity;

$$\phi V_n = \phi_v \times V_n = 29.884 \text{ kips}$$

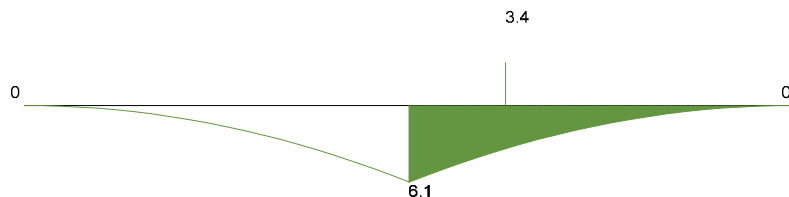
$$V_{u.x} / \phi V_n = 0.080$$

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment;

$$M_{u.y,max} = 3.439 \text{ kip\_ft}$$

Tension reinforcement provided;

$$3 \text{ No.4 bottom bars (14.7 in c/c)}$$

Area of tension reinforcement provided;

$$A_{sy\_bot\_prov} = 0.6 \text{ in}^2$$

Minimum area of reinforcement (8.6.1.1);

$$A_{s,min} = 0.0018 \times L_x \times h = 0.778 \text{ in}^2$$

FAIL - Minimum area of reinforcement required exceeds area of reinforcement provided



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 347
Project Name: JKV-EMS		

Maximum spacing of reinforcement (8.7.2.2);

$$s_{max} = \min(2 \times h, 18 \text{ in}) = 18 \text{ in}$$

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement;

$$d = h - C_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2 = 8.250 \text{ in}$$

Depth of compression block;

$$a = A_{sy.bot,prov} \times f_y / (0.85 \times f'_c \times L_x) = 0.294 \text{ in}$$

Neutral axis factor;

$$\beta_1 = 0.85$$

Depth to neutral axis;

$$c = a / \beta_1 = 0.346 \text{ in}$$

Strain in tensile reinforcement;

$$\epsilon_t = 0.003 \times d / c - 0.003 = 0.06853$$

Minimum tensile strain(8.3.3.1);

$$\epsilon_{min} = 0.004 = 0.00400$$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity;

$$M_n = A_{sy.bot,prov} \times f_y \times (d - a / 2) = 24.309 \text{ kip\_ft}$$

Flexural strength reduction factor;

$$\phi_f = \min(\max(0.65 + 0.25 \times (\epsilon_t - \epsilon_{ty}) / (0.005 - \epsilon_{ty}), 0.65), 0.9) = 0.900$$

Design moment capacity;

$$\phi M_n = \phi_f \times M_n = 21.878 \text{ kip\_ft}$$

$$M_{u,y,max} / \phi M_n = 0.157$$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force;

$$V_{u,y} = 2.378 \text{ kips}$$

Depth to reinforcement;

$$d_v = h - C_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2 = 8.25 \text{ in}$$

Shear strength reduction factor;

$$\phi_v = 0.75$$

Nominal shear capacity (Eq. 22.5.5.1);

$$V_n = 2 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} \times L_x \times d_v = 37.568 \text{ kips}$$

Design shear capacity;

$$\phi V_n = \phi_v \times V_n = 28.176 \text{ kips}$$

$$V_{u,y} / \phi V_n = 0.084$$

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement;

$$d_{v2} = 8.5 \text{ in}$$

Shear perimeter length (22.6.4);

$$l_{xp} = 17.500 \text{ in}$$

Shear perimeter width (22.6.4);

$$l_{yp} = 17.500 \text{ in}$$

Shear perimeter (22.6.4);

$$b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2}) = 70.000 \text{ in}$$

Shear area;

$$A_p = l_{x,perim} \times l_{y,perim} = 306.250 \text{ in}^2$$

Surcharge loaded area;

$$A_{sur} = A_p - l_{x1} \times l_{y1} = 225.250 \text{ in}^2$$

Ultimate bearing pressure at center of shear area;

$$q_{up,avg} = 1.992 \text{ ksf}$$

Ultimate shear load;

$$F_{up} = \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_D \times A_p \times F_{swt} - q_{up,avg} \times A_p = 12.451 \text{ kips}$$

Ultimate shear stress from vertical load;

$$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0 \text{ psi}) = 20.927 \text{ psi}$$

Column geometry factor (Table 22.6.5.2);

$$\beta = l_{y1} / l_{x1} = 1.00$$

Column location factor (22.6.5.3);

$$\alpha_s = 40$$

Concrete shear strength (22.6.5.2);

$$v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} = 379.473 \text{ psi}$$

$$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} = 433.684 \text{ psi}$$

$$v_{cpc} = 4 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} = 252.982 \text{ psi}$$

$$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) = 252.982 \text{ psi}$$

Shear strength reduction factor;

$$\phi_v = 0.75$$

Nominal shear stress capacity (Eq. 22.6.1.2);

$$v_n = v_{cp} = 252.982 \text{ psi}$$

Design shear stress capacity (8.5.1.1(d));

$$\phi v_n = \phi_v \times v_n = 189.737 \text{ psi}$$

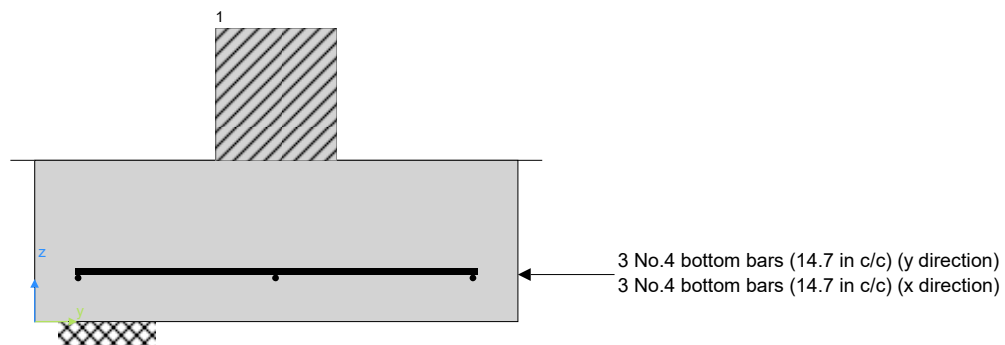
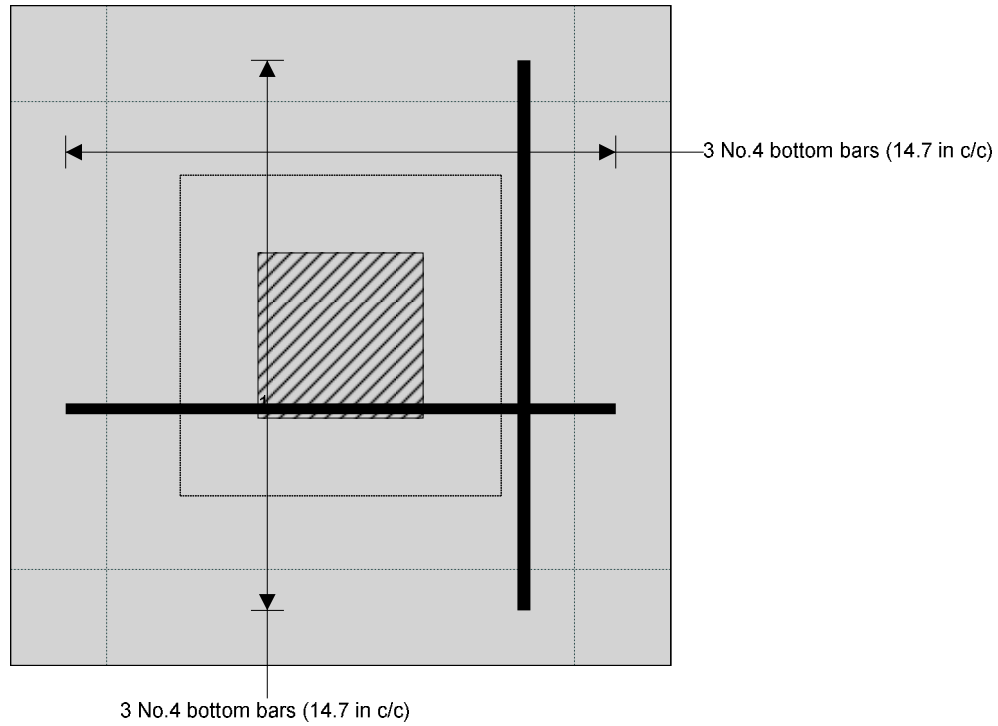


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Revision:	Date:	Sheet No. 348
Project Name: JKV-EMS		

$$V_{ug} / \phi V_n = 0.110$$

PASS - Design shear stress capacity exceeds ultimate shear stress load





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Revision:	Date:	Sheet No. 349
Project Name: JKV-EMS		

FOOTING AT NEW COLUMN - GRID LINE B.4-6

In accordance with ACI318-14

Tedds calculation version 3.3.11

Summary results

Overall design status; PASS;
Overall design utilisation; 0.919

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	67.6			Pass
Description	Unit	Applied	Resisting	Utilization	Result
Soil bearing	ksf	1.379	1.5	0.919	Pass
Description	Unit	Required	Provided	Utilization	Result
Moment, positive, x-direction	kip_ft	56.3	94.0	0.599	Pass
Moment, positive, y-direction	kip_ft	56.3	87.1	0.647	Pass
Shear, one-way, x-direction	kips	28.6	69.2	0.413	Pass
Shear, one-way, y-direction	kips	28.6	64.2	0.445	Pass
Shear, two-way, Col 1	psi	128.322	189.737	0.676	Pass
Min.area of reinf, bot., x-direction	in ²	1.814	2.480		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	11.0		Pass
Min.area of reinf, bot., y-direction	in ²	1.814	2.480		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	11.0		Pass

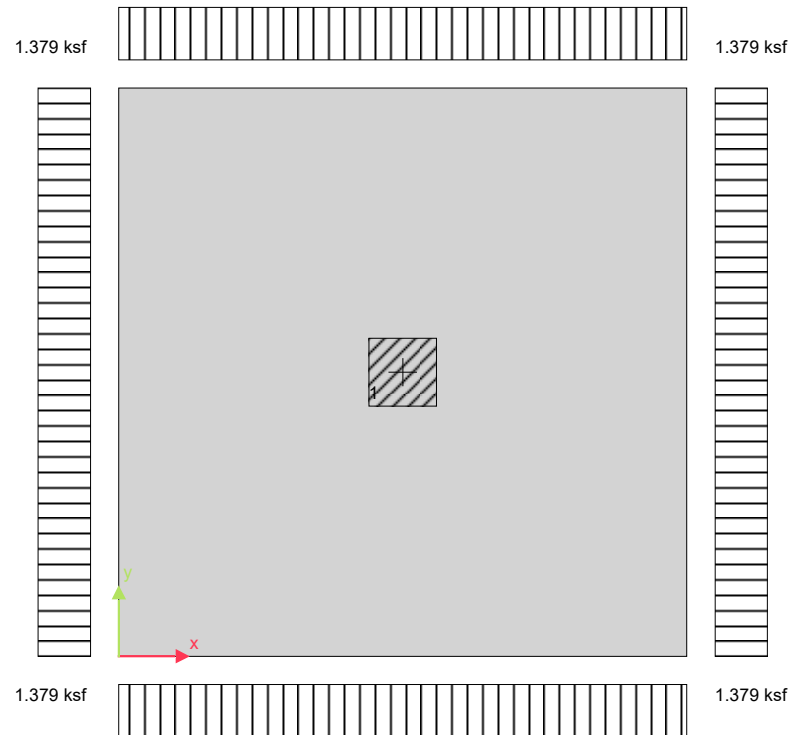
Pad footing details

Length of footing; $L_x = 7$ ft
Width of footing; $L_y = 7$ ft
Footing area; $A = L_x \times L_y = 49$ ft²
Depth of footing; $h = 12$ in
Depth of soil over footing; $h_{\text{soil}} = 0$ in
Density of concrete; $\gamma_{\text{conc}} = 150.0$ lb/ft³



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Revision:	Date:	Sheet No. 350
Project Name: JKV-EMS		



Column no.1 details

Length of column; $l_{x1} = 10.00$ in
Width of column; $l_{y1} = 10.00$ in
position in x-axis; $x_1 = 42.00$ in
position in y-axis; $y_1 = 42.00$ in

Soil properties

Gross allowable bearing pressure; $Q_{allow\_Gross} = 1.5$ ksf;
Density of soil; $\gamma_{soil} = 120.0$ lb/ft³
Angle of internal friction; $\phi_b = 30.0$ deg
Design base friction angle; $\delta_{bb} = 30.0$ deg
Coefficient of base friction; $\tan(\delta_{bb}) = 0.577$

Footing loads

Self weight; $F_{swt} = h \times \gamma_{conc} = 150$ psf

Column no.1 loads

Dead load in z; $F_{Dz1} = 33.4$ kips
Live load in z; $F_{Lz1} = 26.8$ kips

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.554)
1.0D + 1.0L (0.919)



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 351
Project Name: JKV-EMS		

Combination 2 results: 1.0D + 1.0L

Forces on footing

Force in z-axis;

$$F_{dz} = \gamma_D \times A \times F_{swt} + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} = \mathbf{67.6 \text{ kips}}$$

Moments on footing

Moment in x-axis, about x is 0;

$$M_{dx} = \gamma_D \times A \times F_{swt} \times L_x / 2 + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) = \mathbf{236.4 \text{ kip\_ft}}$$

Moment in y-axis, about y is 0;

$$M_{dy} = \gamma_D \times A \times F_{swt} \times L_y / 2 + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) = \mathbf{236.4 \text{ kip\_ft}}$$

Uplift verification

Vertical force;

$$F_{dz} = \mathbf{67.55 \text{ kips}}$$

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis;

$$e_{dx} = M_{dx} / F_{dz} - L_x / 2 = \mathbf{0 \text{ in}}$$

Eccentricity of base reaction in y-axis;

$$e_{dy} = M_{dy} / F_{dz} - L_y / 2 = \mathbf{0 \text{ in}}$$

Pad base pressures

$$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{1.379 \text{ ksf}}$$

$$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{1.379 \text{ ksf}}$$

$$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{1.379 \text{ ksf}}$$

$$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{1.379 \text{ ksf}}$$

$$q_{\min} = \min(q_1, q_2, q_3, q_4) = \mathbf{1.379 \text{ ksf}}$$

$$q_{\max} = \max(q_1, q_2, q_3, q_4) = \mathbf{1.379 \text{ ksf}}$$

Minimum base pressure;

Maximum base pressure;

Allowable bearing capacity

Allowable bearing capacity;

$$q_{\text{allow}} = q_{\text{allow\_Gross}} = \mathbf{1.5 \text{ ksf}}$$

$$q_{\max} / q_{\text{allow}} = \mathbf{0.919}$$

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN

In accordance with ACI318-14

Tedds calculation version 3.3.11

Material details

Compressive strength of concrete;

$$f'_c = \mathbf{4000 \text{ psi}}$$

Yield strength of reinforcement;

$$f_y = \mathbf{60000 \text{ psi}}$$

Compression-controlled strain limit (21.2.2);

$$\epsilon_{ty} = \mathbf{0.00200}$$

Cover to top of footing;

$$c_{\text{nom\_t}} = \mathbf{3 \text{ in}}$$

Cover to side of footing;

$$c_{\text{nom\_s}} = \mathbf{3 \text{ in}}$$

Cover to bottom of footing;

$$c_{\text{nom\_b}} = \mathbf{3 \text{ in}}$$

Concrete type;

Normal weight

Concrete modification factor;

$$\lambda = \mathbf{1.00}$$

Column type;

Concrete



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 352
Project Name: JKV-EMS		

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.381)

1.2D + 1.6L + 0.5Lr (0.676)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on footing

Ultimate force in z-axis;

$$F_{uz} = \gamma_D \times A \times F_{swt} + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} = \mathbf{91.8 \text{ kips}}$$

Moments on footing

Ultimate moment in x-axis, about x is 0;

$$M_{ux} = \gamma_D \times A \times F_{swt} \times L_x / 2 + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) = \mathbf{321.2 \text{ kip\_ft}}$$

Ultimate moment in y-axis, about y is 0;

$$M_{uy} = \gamma_D \times A \times F_{swt} \times L_y / 2 + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) = \mathbf{321.2 \text{ kip\_ft}}$$

Eccentricity of base reaction

Eccentricity of base reaction in x-axis;

$$e_{ux} = M_{ux} / F_{uz} - L_x / 2 = \mathbf{0 \text{ in}}$$

Eccentricity of base reaction in y-axis;

$$e_{uy} = M_{uy} / F_{uz} - L_y / 2 = \mathbf{0 \text{ in}}$$

Pad base pressures

$$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{1.873 \text{ ksf}}$$

$$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{1.873 \text{ ksf}}$$

$$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{1.873 \text{ ksf}}$$

$$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{1.873 \text{ ksf}}$$

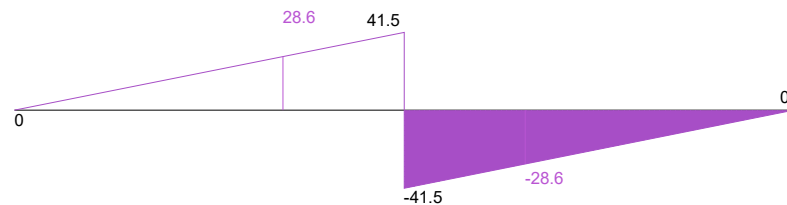
Minimum ultimate base pressure;

$$q_{umin} = \min(q_{u1}, q_{u2}, q_{u3}, q_{u4}) = \mathbf{1.873 \text{ ksf}}$$

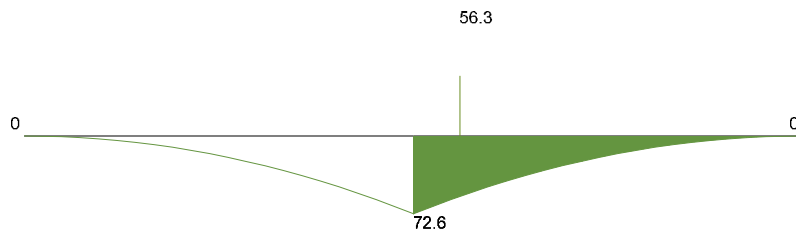
Maximum ultimate base pressure;

$$q_{umax} = \max(q_{u1}, q_{u2}, q_{u3}, q_{u4}) = \mathbf{1.873 \text{ ksf}}$$

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment;

$$M_{u.x.max} = \mathbf{56.335 \text{ kip\_ft}}$$

Tension reinforcement provided;

$$\mathbf{8 \text{ No.5 bottom bars (11.0 in c/c)}}$$

Area of tension reinforcement provided;

$$A_{sx.bot.prov} = \mathbf{2.48 \text{ in}^2}$$



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 353
Project Name: JKV-EMS		

Minimum area of reinforcement (8.6.1.1);

$$A_{s,min} = 0.0018 \times L_y \times h = \mathbf{1.814 \text{ in}^2}$$

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2);

$$s_{max} = \min(2 \times h, 18 \text{ in}) = \mathbf{18 \text{ in}}$$

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement;

$$d = h - c_{nom,b} - \phi_{x,bot} / 2 = \mathbf{8.688 \text{ in}}$$

Depth of compression block;

$$a = A_{sx,bot,prov} \times f_y / (0.85 \times f'_c \times L_y) = \mathbf{0.521 \text{ in}}$$

Neutral axis factor;

$$\beta_1 = \mathbf{0.85}$$

Depth to neutral axis;

$$c = a / \beta_1 = \mathbf{0.613 \text{ in}}$$

Strain in tensile reinforcement;

$$\epsilon_t = 0.003 \times d / c - 0.003 = \mathbf{0.03952}$$

Minimum tensile strain(8.3.3.1);

$$\epsilon_{min} = 0.004 = \mathbf{0.00400}$$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity;

$$M_n = A_{sx,bot,prov} \times f_y \times (d - a / 2) = \mathbf{104.495 \text{ kip\_ft}}$$

Flexural strength reduction factor;

$$\phi_f = \min(\max(0.65 + 0.25 \times (\epsilon_t - \epsilon_{ty}) / (0.005 - \epsilon_{ty}), 0.65), 0.9) = \mathbf{0.900}$$

Design moment capacity;

$$\phi M_n = \phi_f \times M_n = \mathbf{94.045 \text{ kip\_ft}}$$

$$M_{u,x,max} / \phi M_n = \mathbf{0.599}$$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force;

$$V_{u,x} = \mathbf{28.579 \text{ kips}}$$

Depth to reinforcement;

$$d_v = h - c_{nom,b} - \phi_{x,bot} / 2 = \mathbf{8.688 \text{ in}}$$

Shear strength reduction factor;

$$\phi_v = \mathbf{0.75}$$

Nominal shear capacity (Eq. 22.5.5.1);

$$V_n = 2 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} \times L_y \times d_v = \mathbf{92.307 \text{ kips}}$$

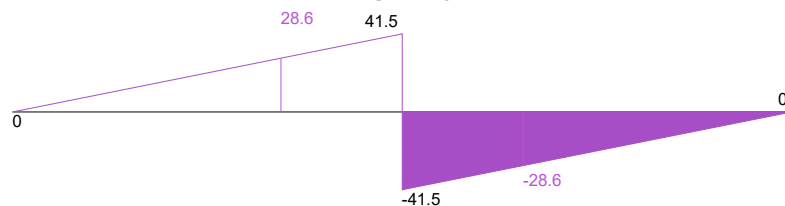
Design shear capacity;

$$\phi V_n = \phi_v \times V_n = \mathbf{69.23 \text{ kips}}$$

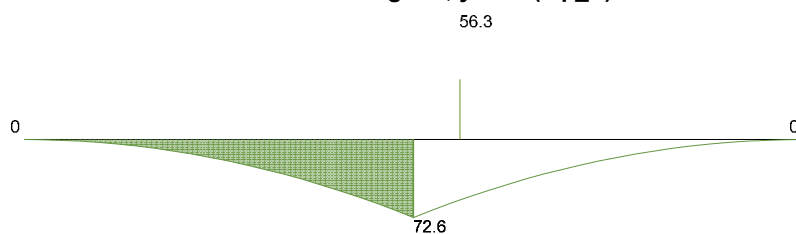
$$V_{u,x} / \phi V_n = \mathbf{0.413}$$

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment;

$$M_{u,y,max} = \mathbf{56.335 \text{ kip\_ft}}$$

Tension reinforcement provided;

$$\mathbf{8 \text{ No.5 bottom bars (11.0 in c/c)}}$$

Area of tension reinforcement provided;

$$A_{sy,bot,prov} = \mathbf{2.48 \text{ in}^2}$$



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 354
Project Name: JKV-EMS		

Minimum area of reinforcement (8.6.1.1);

$$A_{s,min} = 0.0018 \times L_x \times h = \mathbf{1.814 \text{ in}^2}$$

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2);

$$s_{max} = \min(2 \times h, 18 \text{ in}) = \mathbf{18 \text{ in}}$$

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement;

$$d = h - c_{nom\_b} - \phi_{x,bot} - \phi_{y,bot} / 2 = \mathbf{8.062 \text{ in}}$$

Depth of compression block;

$$a = A_{sy,bot,prov} \times f_y / (0.85 \times f'_c \times L_x) = \mathbf{0.521 \text{ in}}$$

Neutral axis factor;

$$\beta_1 = \mathbf{0.85}$$

Depth to neutral axis;

$$c = a / \beta_1 = \mathbf{0.613 \text{ in}}$$

Strain in tensile reinforcement;

$$\epsilon_t = 0.003 \times d / c - 0.003 = \mathbf{0.03646}$$

Minimum tensile strain(8.3.3.1);

$$\epsilon_{min} = 0.004 = \mathbf{0.00400}$$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity;

$$M_n = A_{sy,bot,prov} \times f_y \times (d - a / 2) = \mathbf{96.745 \text{ kip\_ft}}$$

Flexural strength reduction factor;

$$\phi_f = \min(\max(0.65 + 0.25 \times (\epsilon_t - \epsilon_{ty}) / (0.005 - \epsilon_{ty}), 0.65), 0.9) = \mathbf{0.900}$$

Design moment capacity;

$$\phi M_n = \phi_f \times M_n = \mathbf{87.07 \text{ kip\_ft}}$$

$$M_{u,y,max} / \phi M_n = \mathbf{0.647}$$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force;

$$V_{u,y} = \mathbf{28.579 \text{ kips}}$$

Depth to reinforcement;

$$d_v = h - c_{nom\_b} - \phi_{x,bot} - \phi_{y,bot} / 2 = \mathbf{8.062 \text{ in}}$$

Shear strength reduction factor;

$$\phi_v = \mathbf{0.75}$$

Nominal shear capacity (Eq. 22.5.5.1);

$$V_n = 2 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} \times L_x \times d_v = \mathbf{85.666 \text{ kips}}$$

Design shear capacity;

$$\phi V_n = \phi_v \times V_n = \mathbf{64.25 \text{ kips}}$$

$$V_{u,y} / \phi V_n = \mathbf{0.445}$$

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement;

$$d_{v2} = \mathbf{8.375 \text{ in}}$$

Shear perimeter length (22.6.4);

$$l_{xp} = \mathbf{18.375 \text{ in}}$$

Shear perimeter width (22.6.4);

$$l_{yp} = \mathbf{18.375 \text{ in}}$$

Shear perimeter (22.6.4);

$$b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2}) = \mathbf{73.500 \text{ in}}$$

Shear area;

$$A_p = l_{x,perim} \times l_{y,perim} = \mathbf{337.641 \text{ in}^2}$$

Surcharge loaded area;

$$A_{sur} = A_p - l_{x1} \times l_{y1} = \mathbf{237.641 \text{ in}^2}$$

Ultimate bearing pressure at center of shear area;

$$q_{up,avg} = \mathbf{1.873 \text{ ksf}}$$

Ultimate shear load;

$$F_{up} = \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_D \times A_p \times F_{swt} - q_{up,avg} \times A_p = \mathbf{78.990 \text{ kips}}$$

Ultimate shear stress from vertical load;

$$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0 \text{ psi}) = \mathbf{128.322 \text{ psi}}$$

Column geometry factor (Table 22.6.5.2);

$$\beta = l_{y1} / l_{x1} = \mathbf{1.00}$$

Column location factor (22.6.5.3);

$$\alpha_s = \mathbf{40}$$

Concrete shear strength (22.6.5.2);

$$v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} = \mathbf{379.473 \text{ psi}}$$

$$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} = \mathbf{414.753 \text{ psi}}$$

$$v_{cpc} = 4 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} = \mathbf{252.982 \text{ psi}}$$

$$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) = \mathbf{252.982 \text{ psi}}$$

Shear strength reduction factor;

$$\phi_v = \mathbf{0.75}$$

Nominal shear stress capacity (Eq. 22.6.1.2);

$$V_n = v_{cp} = \mathbf{252.982 \text{ psi}}$$



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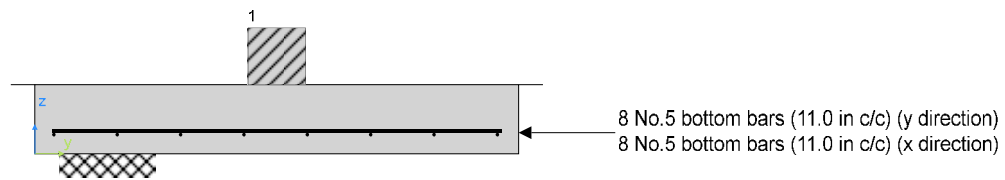
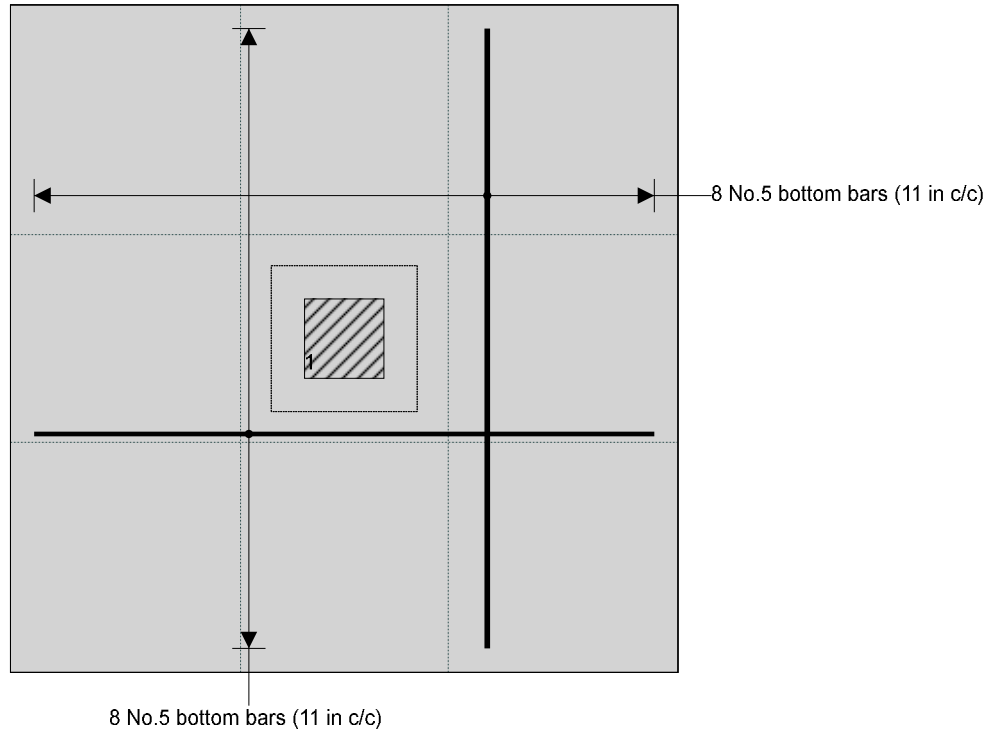
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Revision:	Date:	Sheet No. 355
Project Name: JKV-EMS		

Design shear stress capacity (8.5.1.1(d));

$$\phi V_n = \phi_v \times V_n = 189.737 \text{ psi}$$

$$V_{ug} / \phi V_n = 0.676$$

PASS - Design shear stress capacity exceeds ultimate shear stress load





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Revision:	Date:	Sheet No. 356
Project Name: JKV-EMS		

FOOTING AT NEW COLUMN - GRID LINE B-7.5

In accordance with ACI318-14

Tedds calculation version 3.3.11

Summary results

Overall design status; PASS;
Overall design utilisation; 0.712

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	17.1			Pass
Description	Unit	Applied	Resisting	Utilization	Result
Soil bearing	ksf	1.069	1.5	0.712	Pass
Description	Unit	Required	Provided	Utilization	Result
Moment, positive, x-direction	kip_ft	6.7	58.6	0.114	Pass
Moment, positive, y-direction	kip_ft	6.7	54.2	0.123	Pass
Shear, one-way, x-direction	kips	4.8	39.6	0.122	Pass
Shear, one-way, y-direction	kips	4.8	36.7	0.131	Pass
Shear, two-way, Col 1	psi	30.217	189.737	0.159	Pass
Min.area of reinf, bot., x-direction	in ²	1.037	1.550		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	10.3		Pass
Min.area of reinf, bot., y-direction	in ²	1.037	1.550		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	10.3		Pass

Pad footing details

Length of footing; $L_x = 4$ ft
Width of footing; $L_y = 4$ ft
Footing area; $A = L_x \times L_y = 16$ ft²
Depth of footing; $h = 12$ in
Depth of soil over footing; $h_{soil} = 0$ in
Density of concrete; $\gamma_{conc} = 150.0$ lb/ft³



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Revision:	Date:	Sheet No. 358
Project Name: JKV-EMS		

Combination 2 results: 1.0D + 1.0L

Forces on footing

Force in z-axis;

$$F_{dz} = \gamma_D \times A \times F_{swt} + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} = \mathbf{17.1 \text{ kips}}$$

Moments on footing

Moment in x-axis, about x is 0;

$$M_{dx} = \gamma_D \times A \times F_{swt} \times L_x / 2 + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) = \mathbf{34.2 \text{ kip\_ft}}$$

Moment in y-axis, about y is 0;

$$M_{dy} = \gamma_D \times A \times F_{swt} \times L_y / 2 + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) = \mathbf{34.2 \text{ kip\_ft}}$$

Uplift verification

Vertical force;

$$F_{dz} = \mathbf{17.1 \text{ kips}}$$

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis;

$$e_{dx} = M_{dx} / F_{dz} - L_x / 2 = \mathbf{0 \text{ in}}$$

Eccentricity of base reaction in y-axis;

$$e_{dy} = M_{dy} / F_{dz} - L_y / 2 = \mathbf{0 \text{ in}}$$

Pad base pressures

$$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{1.069 \text{ ksf}}$$

$$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{1.069 \text{ ksf}}$$

$$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{1.069 \text{ ksf}}$$

$$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{1.069 \text{ ksf}}$$

$$q_{\min} = \min(q_1, q_2, q_3, q_4) = \mathbf{1.069 \text{ ksf}}$$

$$q_{\max} = \max(q_1, q_2, q_3, q_4) = \mathbf{1.069 \text{ ksf}}$$

Minimum base pressure;

Maximum base pressure;

Allowable bearing capacity

Allowable bearing capacity;

$$q_{\text{allow}} = q_{\text{allow\_Gross}} = \mathbf{1.5 \text{ ksf}}$$

$$q_{\max} / q_{\text{allow}} = \mathbf{0.712}$$

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN

In accordance with ACI318-14

Tedds calculation version 3.3.11

Material details

Compressive strength of concrete;

$$f'_c = \mathbf{4000 \text{ psi}}$$

Yield strength of reinforcement;

$$f_y = \mathbf{60000 \text{ psi}}$$

Compression-controlled strain limit (21.2.2);

$$\epsilon_{ty} = \mathbf{0.00200}$$

Cover to top of footing;

$$c_{\text{nom\_t}} = \mathbf{3 \text{ in}}$$

Cover to side of footing;

$$c_{\text{nom\_s}} = \mathbf{3 \text{ in}}$$

Cover to bottom of footing;

$$c_{\text{nom\_b}} = \mathbf{3 \text{ in}}$$

Concrete type;

Normal weight

Concrete modification factor;

$$\lambda = \mathbf{1.00}$$

Column type;

Concrete



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Revision:	Date:	Sheet No. 359
Project Name: JKV-EMS		

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.090)

1.2D + 1.6L + 0.5Lr (0.159)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on footing

Ultimate force in z-axis;

$$F_{uz} = \gamma_D \times A \times F_{swt} + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} = \mathbf{23.1 \text{ kips}}$$

Moments on footing

Ultimate moment in x-axis, about x is 0;

$$M_{ux} = \gamma_D \times A \times F_{swt} \times L_x / 2 + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) = \mathbf{46.2 \text{ kip\_ft}}$$

Ultimate moment in y-axis, about y is 0;

$$M_{uy} = \gamma_D \times A \times F_{swt} \times L_y / 2 + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) = \mathbf{46.2 \text{ kip\_ft}}$$

Eccentricity of base reaction

Eccentricity of base reaction in x-axis;

$$e_{ux} = M_{ux} / F_{uz} - L_x / 2 = \mathbf{0 \text{ in}}$$

Eccentricity of base reaction in y-axis;

$$e_{uy} = M_{uy} / F_{uz} - L_y / 2 = \mathbf{0 \text{ in}}$$

Pad base pressures

$$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{1.445 \text{ ksf}}$$

$$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{1.445 \text{ ksf}}$$

$$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{1.445 \text{ ksf}}$$

$$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{1.445 \text{ ksf}}$$

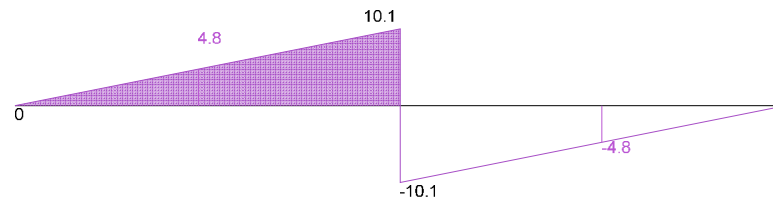
$$q_{umin} = \min(q_{u1}, q_{u2}, q_{u3}, q_{u4}) = \mathbf{1.445 \text{ ksf}}$$

$$q_{umax} = \max(q_{u1}, q_{u2}, q_{u3}, q_{u4}) = \mathbf{1.445 \text{ ksf}}$$

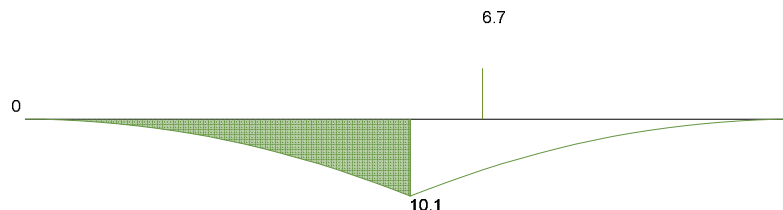
Minimum ultimate base pressure;

Maximum ultimate base pressure;

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment;

$$M_{u.x.max} = \mathbf{6.681 \text{ kip\_ft}}$$

Tension reinforcement provided;

$$5 \text{ No.5 bottom bars (10.3 in c/c)}$$

Area of tension reinforcement provided;

$$A_{sx.bot.prov} = \mathbf{1.55 \text{ in}^2}$$

Minimum area of reinforcement (8.6.1.1);

$$A_{s.min} = 0.0018 \times L_y \times h = \mathbf{1.037 \text{ in}^2}$$



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Revision:	Date:	Sheet No. 360
Project Name: JKV-EMS		

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2);

$$s_{max} = \min(2 \times h, 18 \text{ in}) = 18 \text{ in}$$

Depth to tension reinforcement;

$$d = h - c_{nom\_b} - \phi_{x\_bot} / 2 = 8.688 \text{ in}$$

Depth of compression block;

$$a = A_{sx\_bot\_prov} \times f_y / (0.85 \times f'_c \times L_y) = 0.570 \text{ in}$$

Neutral axis factor;

$$\beta_1 = 0.85$$

Depth to neutral axis;

$$c = a / \beta_1 = 0.670 \text{ in}$$

Strain in tensile reinforcement;

$$\epsilon_t = 0.003 \times d / c - 0.003 = 0.03588$$

Minimum tensile strain(8.3.3.1);

$$\epsilon_{min} = 0.004 = 0.00400$$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity;

$$M_n = A_{sx\_bot\_prov} \times f_y \times (d - a / 2) = 65.12 \text{ kip\_ft}$$

Flexural strength reduction factor;

$$\phi_f = \min(\max(0.65 + 0.25 \times (\epsilon_t - \epsilon_{ty}) / (0.005 - \epsilon_{ty}), 0.65), 0.9) = 0.900$$

Design moment capacity;

$$\phi M_n = \phi_f \times M_n = 58.608 \text{ kip\_ft}$$

$$M_{u.x\_max} / \phi M_n = 0.114$$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force;

$$V_{u.x} = 4.823 \text{ kips}$$

Depth to reinforcement;

$$d_v = h - c_{nom\_b} - \phi_{x\_bot} / 2 = 8.688 \text{ in}$$

Shear strength reduction factor;

$$\phi_v = 0.75$$

Nominal shear capacity (Eq. 22.5.5.1);

$$V_n = 2 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} \times L_y \times d_v = 52.747 \text{ kips}$$

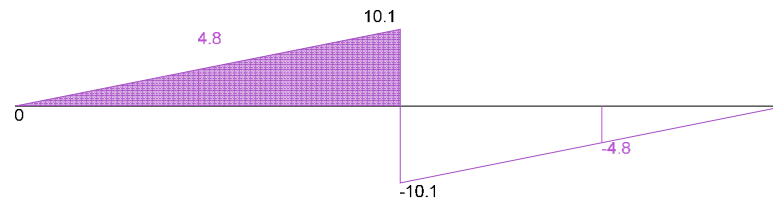
Design shear capacity;

$$\phi V_n = \phi_v \times V_n = 39.56 \text{ kips}$$

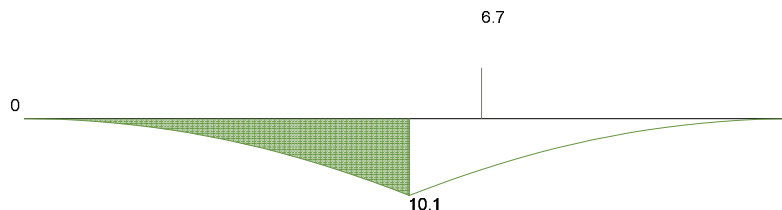
$$V_{u.x} / \phi V_n = 0.122$$

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment;

$$M_{u.y\_max} = 6.681 \text{ kip\_ft}$$

Tension reinforcement provided;

$$5 \text{ No.5 bottom bars (10.3 in c/c)}$$

Area of tension reinforcement provided;

$$A_{sy\_bot\_prov} = 1.55 \text{ in}^2$$

Minimum area of reinforcement (8.6.1.1);

$$A_{s,min} = 0.0018 \times L_x \times h = 1.037 \text{ in}^2$$



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 361
Project Name: JKV-EMS		

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2);

$$s_{max} = \min(2 \times h, 18 \text{ in}) = 18 \text{ in}$$

Depth to tension reinforcement;

$$d = h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2 = 8.062 \text{ in}$$

Depth of compression block;

$$a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x) = 0.570 \text{ in}$$

Neutral axis factor;

$$\beta_1 = 0.85$$

Depth to neutral axis;

$$c = a / \beta_1 = 0.670 \text{ in}$$

Strain in tensile reinforcement;

$$\epsilon_t = 0.003 \times d / c - 0.003 = 0.03308$$

Minimum tensile strain(8.3.3.1);

$$\epsilon_{min} = 0.004 = 0.00400$$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity;

$$M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2) = 60.276 \text{ kip\_ft}$$

Flexural strength reduction factor;

$$\phi_f = \min(\max(0.65 + 0.25 \times (\epsilon_t - \epsilon_{ty}) / (0.005 - \epsilon_{ty}), 0.65), 0.9) = 0.900$$

Design moment capacity;

$$\phi M_n = \phi_f \times M_n = 54.249 \text{ kip\_ft}$$

$$M_{u.y.max} / \phi M_n = 0.123$$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force;

$$V_{u.y} = 4.823 \text{ kips}$$

Depth to reinforcement;

$$d_v = h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2 = 8.062 \text{ in}$$

Shear strength reduction factor;

$$\phi_v = 0.75$$

Nominal shear capacity (Eq. 22.5.5.1);

$$V_n = 2 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} \times L_x \times d_v = 48.952 \text{ kips}$$

Design shear capacity;

$$\phi V_n = \phi_v \times V_n = 36.714 \text{ kips}$$

$$V_{u.y} / \phi V_n = 0.131$$

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement;

$$d_{v2} = 8.375 \text{ in}$$

Shear perimeter length (22.6.4);

$$l_{xp} = 17.375 \text{ in}$$

Shear perimeter width (22.6.4);

$$l_{yp} = 17.375 \text{ in}$$

Shear perimeter (22.6.4);

$$b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2}) = 69.500 \text{ in}$$

Shear area;

$$A_p = l_{x,perim} \times l_{y,perim} = 301.891 \text{ in}^2$$

Surcharge loaded area;

$$A_{sur} = A_p - l_{x1} \times l_{y1} = 220.891 \text{ in}^2$$

Ultimate bearing pressure at center of shear area;

$$q_{up.avg} = 1.445 \text{ ksf}$$

Ultimate shear load;

$$F_{up} = \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_D \times A_p \times F_{swt} - q_{up.avg} \times A_p = 17.588 \text{ kips}$$

Ultimate shear stress from vertical load;

$$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0 \text{ psi}) = 30.217 \text{ psi}$$

Column geometry factor (Table 22.6.5.2);

$$\beta = l_{y1} / l_{x1} = 1.00$$

Column location factor (22.6.5.3);

$$\alpha_s = 40$$

Concrete shear strength (22.6.5.2);

$$v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} = 379.473 \text{ psi}$$

$$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} = 431.344 \text{ psi}$$

$$v_{cpc} = 4 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} = 252.982 \text{ psi}$$

$$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) = 252.982 \text{ psi}$$

Shear strength reduction factor;

$$\phi_v = 0.75$$

Nominal shear stress capacity (Eq. 22.6.1.2);

$$v_n = v_{cp} = 252.982 \text{ psi}$$

Design shear stress capacity (8.5.1.1(d));

$$\phi v_n = \phi_v \times v_n = 189.737 \text{ psi}$$



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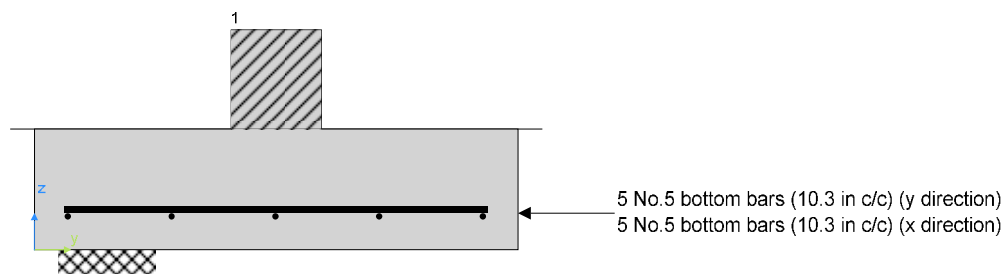
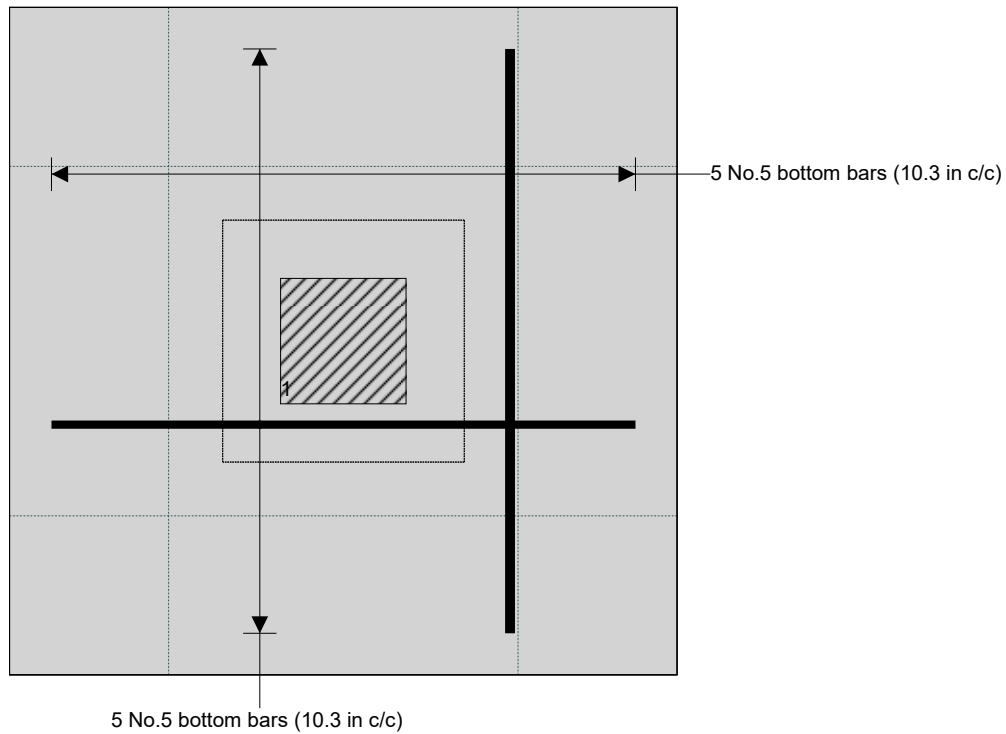
Date:

Sheet No. 362

Project Name: JKV-EMS

$$V_{ug} / \phi V_n = 0.159$$

PASS - Design shear stress capacity exceeds ultimate shear stress load





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Revision:	Date:	Sheet No. 363
Project Name: JKV-EMS		

FOOTING AT NEW COLUMN - GRID LINE E-6

In accordance with ACI318-14

Tedds calculation version 3.3.11

Summary results

Overall design status; PASS;
Overall design utilisation; 0.904

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	21.7			Pass
Description	Unit	Applied	Resisting	Utilization	Result
Soil bearing	ksf	1.356	1.5	0.904	Pass
Description	Unit	Required	Provided	Utilization	Result
Moment, positive, x-direction	kip_ft	8.8	58.6	0.150	Pass
Moment, positive, y-direction	kip_ft	8.8	54.2	0.162	Pass
Shear, one-way, x-direction	kips	6.3	39.6	0.160	Pass
Shear, one-way, y-direction	kips	6.3	36.7	0.173	Pass
Shear, two-way, Col 1	psi	39.712	189.737	0.209	Pass
Min.area of reinf, bot., x-direction	in ²	1.037	1.550		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	10.3		Pass
Min.area of reinf, bot., y-direction	in ²	1.037	1.550		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	10.3		Pass

Pad footing details

Length of footing; $L_x = 4$ ft
Width of footing; $L_y = 4$ ft
Footing area; $A = L_x \times L_y = 16$ ft²
Depth of footing; $h = 12$ in
Depth of soil over footing; $h_{soil} = 0$ in
Density of concrete; $\gamma_{conc} = 150.0$ lb/ft³



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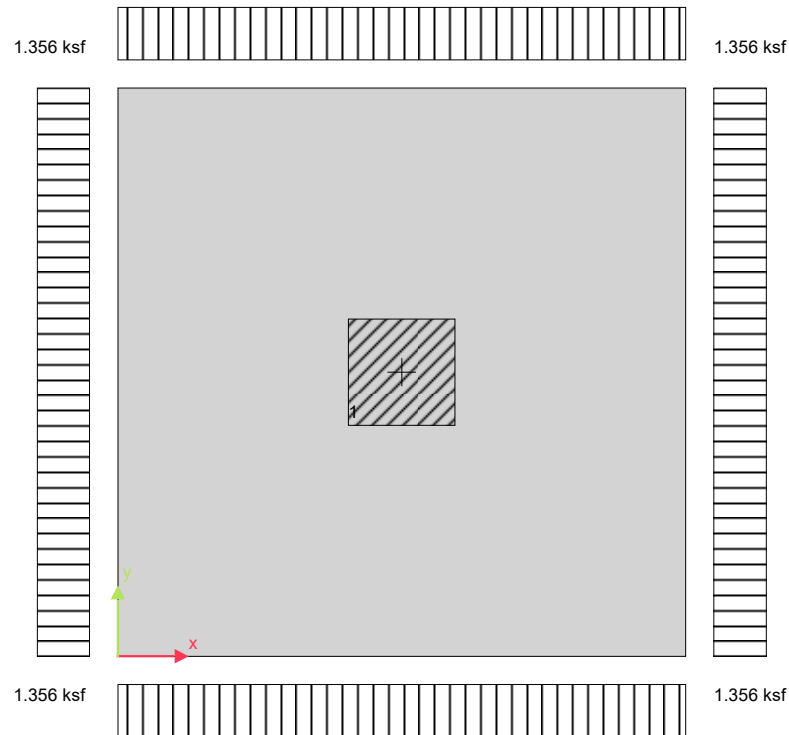
Job No. 25-003

Revision:

Date:

Sheet No. 364

Project Name: JKV-EMS



Column no.1 details

Length of column;

$$l_{x1} = 9.00 \text{ in}$$

Width of column;

$$l_{y1} = 9.00 \text{ in}$$

position in x-axis;

$$x_1 = 24.00 \text{ in}$$

position in y-axis;

$$y_1 = 24.00 \text{ in}$$

Soil properties

Gross allowable bearing pressure;

$$Q_{\text{allow\_Gross}} = 1.5 \text{ ksf};$$

Density of soil;

$$\gamma_{\text{soil}} = 120.0 \text{ lb/ft}^3$$

Angle of internal friction;

$$\phi_b = 30.0 \text{ deg}$$

Design base friction angle;

$$\delta_{bb} = 30.0 \text{ deg}$$

Coefficient of base friction;

$$\tan(\delta_{bb}) = 0.577$$

Footing loads

Self weight;

$$F_{\text{swt}} = h \times \gamma_{\text{conc}} = 150 \text{ psf}$$

Column no.1 loads

Dead load in z;

$$F_{Dz1} = 10.7 \text{ kips}$$

Live load in z;

$$F_{Lz1} = 8.6 \text{ kips}$$

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.546)

1.0D + 1.0L (0.904)



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Revision:	Date:	Sheet No. 365
Project Name: JKV-EMS		

Combination 2 results: 1.0D + 1.0L

Forces on footing

Force in z-axis;

$$F_{dz} = \gamma_D \times A \times F_{swt} + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} = \mathbf{21.7 \text{ kips}}$$

Moments on footing

Moment in x-axis, about x is 0;

$$M_{dx} = \gamma_D \times A \times F_{swt} \times L_x / 2 + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) = \mathbf{43.4 \text{ kip\_ft}}$$

Moment in y-axis, about y is 0;

$$M_{dy} = \gamma_D \times A \times F_{swt} \times L_y / 2 + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) = \mathbf{43.4 \text{ kip\_ft}}$$

Uplift verification

Vertical force;

$$F_{dz} = \mathbf{21.7 \text{ kips}}$$

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis;

$$e_{dx} = M_{dx} / F_{dz} - L_x / 2 = \mathbf{0 \text{ in}}$$

Eccentricity of base reaction in y-axis;

$$e_{dy} = M_{dy} / F_{dz} - L_y / 2 = \mathbf{0 \text{ in}}$$

Pad base pressures

$$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{1.356 \text{ ksf}}$$

$$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{1.356 \text{ ksf}}$$

$$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{1.356 \text{ ksf}}$$

$$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{1.356 \text{ ksf}}$$

$$q_{min} = \min(q_1, q_2, q_3, q_4) = \mathbf{1.356 \text{ ksf}}$$

$$q_{max} = \max(q_1, q_2, q_3, q_4) = \mathbf{1.356 \text{ ksf}}$$

Minimum base pressure;

Maximum base pressure;

Allowable bearing capacity

Allowable bearing capacity;

$$q_{allow} = q_{allow\_Gross} = \mathbf{1.5 \text{ ksf}}$$

$$q_{max} / q_{allow} = \mathbf{0.904}$$

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN

In accordance with ACI318-14

Tedds calculation version 3.3.11

Material details

Compressive strength of concrete;

$$f'_c = \mathbf{4000 \text{ psi}}$$

Yield strength of reinforcement;

$$f_y = \mathbf{60000 \text{ psi}}$$

Compression-controlled strain limit (21.2.2);

$$\epsilon_{ty} = \mathbf{0.00200}$$

Cover to top of footing;

$$c_{nom\_t} = \mathbf{3 \text{ in}}$$

Cover to side of footing;

$$c_{nom\_s} = \mathbf{3 \text{ in}}$$

Cover to bottom of footing;

$$c_{nom\_b} = \mathbf{3 \text{ in}}$$

Concrete type;

Normal weight

Concrete modification factor;

$$\lambda = \mathbf{1.00}$$

Column type;

Concrete



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Revision:	Date:	Sheet No. 366
Project Name: JKV-EMS		

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.118)

1.2D + 1.6L + 0.5Lr (0.209)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on footing

Ultimate force in z-axis;

$$F_{uz} = \gamma_D \times A \times F_{swt} + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} = \mathbf{29.5 \text{ kips}}$$

Moments on footing

Ultimate moment in x-axis, about x is 0;

$$M_{ux} = \gamma_D \times A \times F_{swt} \times L_x / 2 + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) = \mathbf{59.0 \text{ kip\_ft}}$$

Ultimate moment in y-axis, about y is 0;

$$M_{uy} = \gamma_D \times A \times F_{swt} \times L_y / 2 + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) = \mathbf{59.0 \text{ kip\_ft}}$$

Eccentricity of base reaction

Eccentricity of base reaction in x-axis;

$$e_{ux} = M_{ux} / F_{uz} - L_x / 2 = \mathbf{0 \text{ in}}$$

Eccentricity of base reaction in y-axis;

$$e_{uy} = M_{uy} / F_{uz} - L_y / 2 = \mathbf{0 \text{ in}}$$

Pad base pressures

$$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{1.842 \text{ ksf}}$$

$$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{1.842 \text{ ksf}}$$

$$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{1.842 \text{ ksf}}$$

$$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{1.842 \text{ ksf}}$$

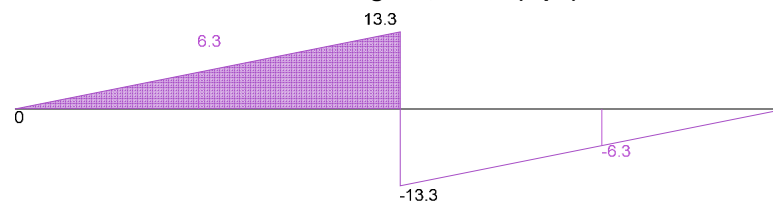
$$q_{umin} = \min(q_{u1}, q_{u2}, q_{u3}, q_{u4}) = \mathbf{1.842 \text{ ksf}}$$

$$q_{umax} = \max(q_{u1}, q_{u2}, q_{u3}, q_{u4}) = \mathbf{1.842 \text{ ksf}}$$

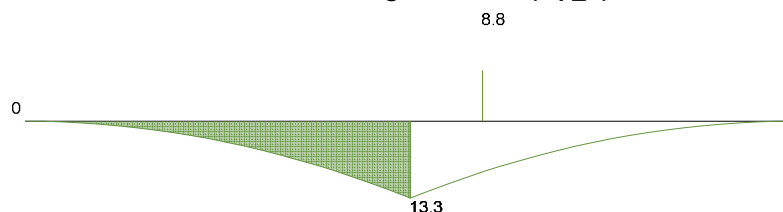
Minimum ultimate base pressure;

Maximum ultimate base pressure;

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment;

$$M_{u.x.max} = \mathbf{8.78 \text{ kip\_ft}}$$

Tension reinforcement provided;

$$5 \text{ No.5 bottom bars (10.3 in c/c)}$$

Area of tension reinforcement provided;

$$A_{sx.bot.prov} = \mathbf{1.55 \text{ in}^2}$$

Minimum area of reinforcement (8.6.1.1);

$$A_{s.min} = 0.0018 \times L_y \times h = \mathbf{1.037 \text{ in}^2}$$



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Revision:	Date:	Sheet No. 367
Project Name: JKV-EMS		

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2);

$$s_{max} = \min(2 \times h, 18 \text{ in}) = 18 \text{ in}$$

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement;

$$d = h - c_{nom\_b} - \phi_{x\_bot} / 2 = 8.688 \text{ in}$$

Depth of compression block;

$$a = A_{sx\_bot\_prov} \times f_y / (0.85 \times f'_c \times L_y) = 0.570 \text{ in}$$

Neutral axis factor;

$$\beta_1 = 0.85$$

Depth to neutral axis;

$$c = a / \beta_1 = 0.670 \text{ in}$$

Strain in tensile reinforcement;

$$\epsilon_t = 0.003 \times d / c - 0.003 = 0.03588$$

Minimum tensile strain(8.3.3.1);

$$\epsilon_{min} = 0.004 = 0.00400$$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity;

$$M_n = A_{sx\_bot\_prov} \times f_y \times (d - a / 2) = 65.12 \text{ kip\_ft}$$

Flexural strength reduction factor;

$$\phi_f = \min(\max(0.65 + 0.25 \times (\epsilon_t - \epsilon_{ty}) / (0.005 - \epsilon_{ty}), 0.65), 0.9) = 0.900$$

Design moment capacity;

$$\phi M_n = \phi_f \times M_n = 58.608 \text{ kip\_ft}$$

$$M_{u.x\_max} / \phi M_n = 0.150$$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force;

$$V_{u.x} = 6.338 \text{ kips}$$

Depth to reinforcement;

$$d_v = h - c_{nom\_b} - \phi_{x\_bot} / 2 = 8.688 \text{ in}$$

Shear strength reduction factor;

$$\phi_v = 0.75$$

Nominal shear capacity (Eq. 22.5.5.1);

$$V_n = 2 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} \times L_y \times d_v = 52.747 \text{ kips}$$

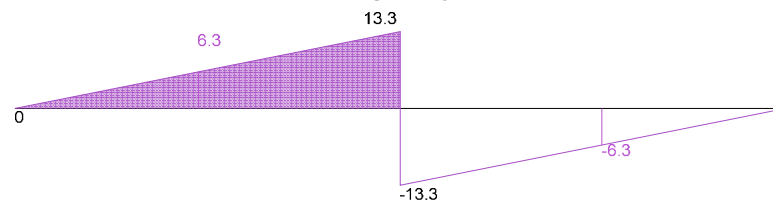
Design shear capacity;

$$\phi V_n = \phi_v \times V_n = 39.56 \text{ kips}$$

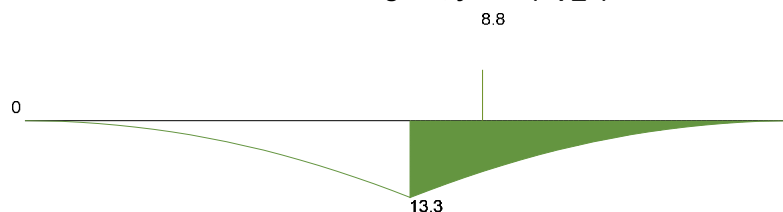
$$V_{u.x} / \phi V_n = 0.160$$

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment;

$$M_{u.y\_max} = 8.78 \text{ kip\_ft}$$

Tension reinforcement provided;

$$5 \text{ No.5 bottom bars (10.3 in c/c)}$$

Area of tension reinforcement provided;

$$A_{sy\_bot\_prov} = 1.55 \text{ in}^2$$

Minimum area of reinforcement (8.6.1.1);

$$A_{s,min} = 0.0018 \times L_x \times h = 1.037 \text{ in}^2$$



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Revision:	Date:	Sheet No. 368
Project Name: JKV-EMS		

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2);

$$s_{max} = \min(2 \times h, 18 \text{ in}) = 18 \text{ in}$$

Depth to tension reinforcement;

$$d = h - C_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2 = 8.062 \text{ in}$$

Depth of compression block;

$$a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x) = 0.570 \text{ in}$$

Neutral axis factor;

$$\beta_1 = 0.85$$

Depth to neutral axis;

$$c = a / \beta_1 = 0.670 \text{ in}$$

Strain in tensile reinforcement;

$$\epsilon_t = 0.003 \times d / c - 0.003 = 0.03308$$

Minimum tensile strain(8.3.3.1);

$$\epsilon_{min} = 0.004 = 0.00400$$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity;

$$M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2) = 60.276 \text{ kip\_ft}$$

Flexural strength reduction factor;

$$\phi_f = \min(\max(0.65 + 0.25 \times (\epsilon_t - \epsilon_{ty}) / (0.005 - \epsilon_{ty}), 0.65), 0.9) = 0.900$$

Design moment capacity;

$$\phi M_n = \phi_f \times M_n = 54.249 \text{ kip\_ft}$$

$$M_{u.y.max} / \phi M_n = 0.162$$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force;

$$V_{u.y} = 6.338 \text{ kips}$$

Depth to reinforcement;

$$d_v = h - C_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2 = 8.062 \text{ in}$$

Shear strength reduction factor;

$$\phi_v = 0.75$$

Nominal shear capacity (Eq. 22.5.5.1);

$$V_n = 2 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} \times L_x \times d_v = 48.952 \text{ kips}$$

Design shear capacity;

$$\phi V_n = \phi_v \times V_n = 36.714 \text{ kips}$$

$$V_{u.y} / \phi V_n = 0.173$$

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement;

$$d_{v2} = 8.375 \text{ in}$$

Shear perimeter length (22.6.4);

$$l_{xp} = 17.375 \text{ in}$$

Shear perimeter width (22.6.4);

$$l_{yp} = 17.375 \text{ in}$$

Shear perimeter (22.6.4);

$$b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2}) = 69.500 \text{ in}$$

Shear area;

$$A_p = l_{x,perim} \times l_{y,perim} = 301.891 \text{ in}^2$$

Surcharge loaded area;

$$A_{sur} = A_p - l_{x1} \times l_{y1} = 220.891 \text{ in}^2$$

Ultimate bearing pressure at center of shear area;

$$q_{up.avg} = 1.842 \text{ ksf}$$

Ultimate shear load;

$$F_{up} = \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_D \times A_p \times F_{swt} - q_{up.avg} \times A_p = 23.115 \text{ kips}$$

Ultimate shear stress from vertical load;

$$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0 \text{ psi}) = 39.712 \text{ psi}$$

Column geometry factor (Table 22.6.5.2);

$$\beta = l_{y1} / l_{x1} = 1.00$$

Column location factor (22.6.5.3);

$$\alpha_s = 40$$

Concrete shear strength (22.6.5.2);

$$v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} = 379.473 \text{ psi}$$

$$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} = 431.344 \text{ psi}$$

$$v_{cpc} = 4 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} = 252.982 \text{ psi}$$

$$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) = 252.982 \text{ psi}$$

Shear strength reduction factor;

$$\phi_v = 0.75$$

Nominal shear stress capacity (Eq. 22.6.1.2);

$$v_n = v_{cp} = 252.982 \text{ psi}$$

Design shear stress capacity (8.5.1.1(d));

$$\phi v_n = \phi_v \times v_n = 189.737 \text{ psi}$$



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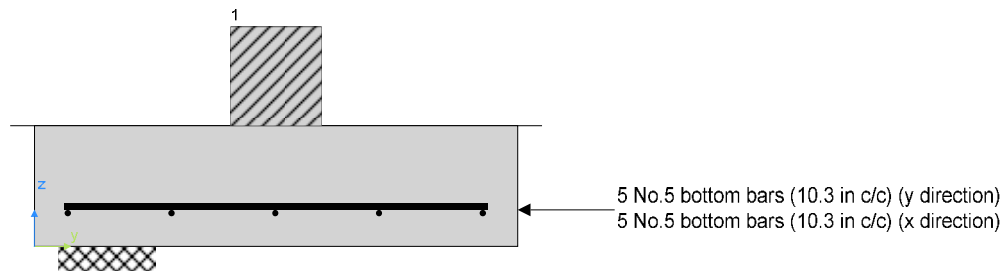
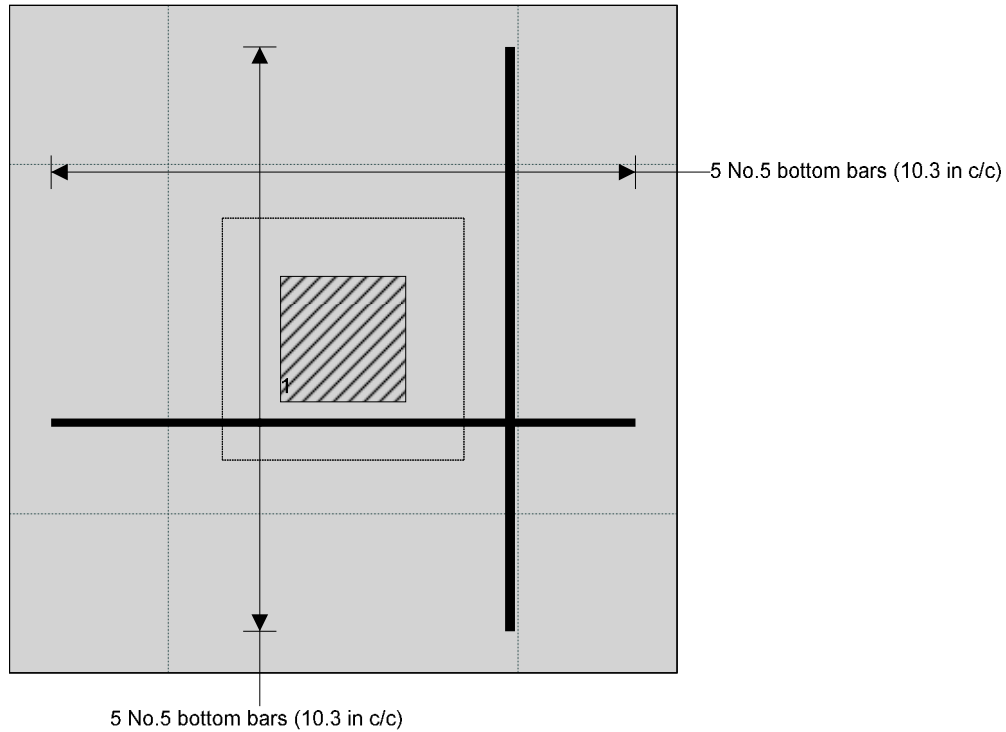
Date:

Sheet No. 369

Project Name: JKV-EMS

$$V_{ug} / \phi V_n = 0.209$$

PASS - Design shear stress capacity exceeds ultimate shear stress load





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Revision:	Date:	Sheet No. 370
Project Name: JKV-EMS		

MECH SCREEN FRAMING

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

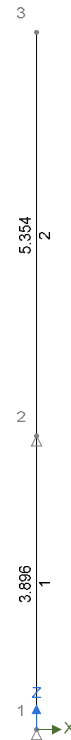
Tedds calculation version 2.2.24

ANALYSIS

Tedds calculation version 1.0.38

Geometry

Geometry (ft)



Span	Length (ft)	Section	Start Support	End Support
1	3.9	2"x6"	Pinned	Pinned
2	5.35	2x6	Pinned	Free
2x6: Area 12 in ² , Inertia Major 36 in ⁴ , Inertia Minor 4 in ⁴ , Shear area parallel to Minor 10 in ² , Shear area parallel to Major 10 in ²				
2"x6": Area 12 in ² , Inertia Major 36 in ⁴ , Inertia Minor 4 in ⁴ , Shear area parallel to Minor 10 in ² , Shear area parallel to Major 10 in ²				
Spruce-Pine-Fir(2" && wider(No.2)) (2018): Density 29.1 lbm/ft ³ , Youngs 1400 ksi, Shear 87 ksi, Thermal 0 °C ⁻¹				
Spruce-Pine-Fir(2" && wider(No.2)): Density 30.6 lbm/ft ³ , Youngs 1400 ksi, Shear 87 ksi, Thermal 0 °C ⁻¹				

Loading

Self weight included



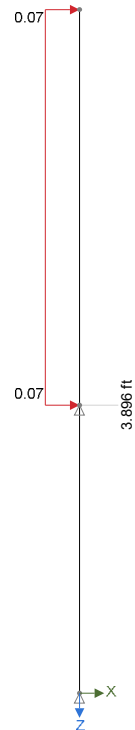
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Revision:	Date:	Sheet No. 371
Project Name: JKV-EMS		

Dead - Loading (kips)



Wind - Loading (kips/ft)





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Revision:	Date:	Sheet No. 372
Project Name: JKV-EMS		

Load combination factors

Load combination	Self Weight	Dead	Wind
1.0D (Strength)	1.00	1.00	
0.6D + 0.6W (Strength)	0.60	0.60	0.60

Member Loads

Member	Load case	Load Type	Orientation	Description
Mech Screen	Dead	Point load	GlobalZ	0.11 kips at 9.25 ft
Mech Screen	Wind	UDL	GlobalX	0.07 kips/ft at 3.896 ft to 9.25 ft

Results

Forces

Strength combinations - Moment envelope (kip_ft)

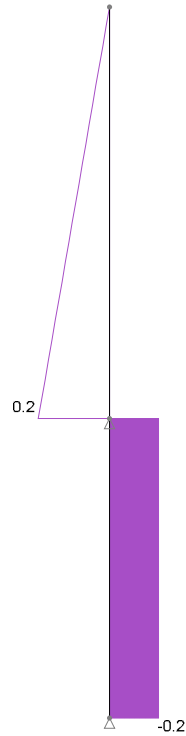




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Revision:	Date:	Sheet No. 373
Project Name: JKV-EMS		

Strength combinations - Shear envelope (kips)



;

Mech Screen - Span 1

Member details

Service condition;

Wet

Sawn lumber section details

Number of sections in member;

N = 1

Nominal breadth of sections;

b_{nom} = 2 in

Breadth of sections;

b = 2 in

Nominal depth of sections;

d_{nom} = 6 in

Depth of sections;

d = 6 in

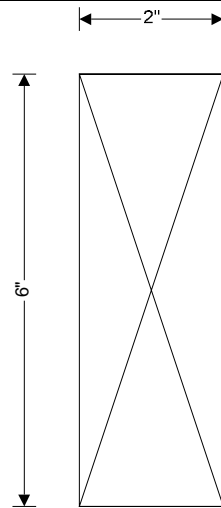
Material;

Spruce-Pine-Fir, 2" && wider, No.2 grade



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Revision:	Date:	Sheet No. 374
Project Name: JKV-EMS		



2"x6" sawn lumber section

Cross-sectional area, A , 12 in²

Section modulus, S_x , 12 in³

Section modulus, S_y , 4 in³

Second moment of area, I_x , 36 in⁴

Second moment of area, I_y , 4 in⁴

Radius of gyration, r_x , 1.732 in

Radius of gyration, r_y , 0.577 in

Spruce-Pine-Fir, 2" & wider, No.2 grade

Bending, F_b , 875 psi

Shear parallel to grain, F_v , 135 psi

Compression parallel to grain, F_c , 1150 psi

Compression perpendicular to grain, F_{c_perp} , 425 psi

Tension parallel to grain, F_t , 450 psi

Modulus of elasticity, E , 1400000 psi

Minimum modulus of elasticity, E_{min} , 510000 psi

Density, ρ , 30.6 lbm/ft³

Specific gravity, G , 0.42

Span details

Bearing length;

$L_b = 4$ in

Consider Combination 2 - 0.6D + 0.6W (Strength)

Adjustment factors - Table 4.3.1

Ten minutes load duration factor - Table 2.3.2;

$C_D = 1.6$

Wet service factor for bending - Table 4A;

$C_{Mb} = 1$

Wet service factor for shear - Table 4A;

$C_{Mv} = 0.97$

Wet service factor for compression - Table 4A;

$C_{Mc} = 0.8$

Wet service factor for perp.comp. - Table 4A;

$C_{Mc\_perp} = 0.67$

Wet service factor for mod.of elasticity - Table 4A;

$C_{ME} = 0.9$

Size factor for bending - Table 4A;

$C_{Fb} = 1.3$

Size factor for compression - Table 4A;

$C_{Fc} = 1.1$

Repetitive member factor - Table 4.3.9;

$C_r = 1.15$

Reference compression design value;

$F_c^* = F_c \times C_D \times C_{Mc} \times C_{Fc} = 1619$ lb/in²

Adjusted modulus of elasticity;

$E_{min}' = E_{min} \times C_{ME} = 459000$ lb/in²

Critical buckling design value;

$F_{cE} = 0.822 \times E_{min}' = 377298$ lb/in²

Column stability factor - eq.3.7-1

$$C_P = (1 + (F_{cE} / F_c^*)) / 1.6 - \sqrt{((1 + (F_{cE} / F_c^*)) / 1.6)^2 - (F_{cE} / F_c^*) / 0.8} = 0.999$$

Check design at start of span

Compression members - General - cl.3.6

Design axial compression force;

$P = 3$ lb

Design compression parallel to grain - Table 4.3.1;

$F_c' = F_c \times C_D \times C_{Mc} \times C_{Fc} \times C_P = 1618$ lb/in²

Actual compression parallel to grain;

$f_c = P / (b \times d) = 0$ lb/in²

$f_c / F_c' = 0.000$

PASS - Design compression stress exceeds actual compression stress

Bending members - Shear - cl.3.4

Design shear force;

$V_x = 155$ lb



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Revision:	Date:	Sheet No. 375
Project Name: JKV-EMS		

Design shear stress - Table 4.3.1;

Actual shear stress - eq.3.4-2;

$$F_{v,x'} = F_v \times C_D \times C_{Mv} = \mathbf{210 \text{ lb/in}^2}$$

$$f_{v,x} = 3 \times V_x / (2 \times b \times d) = \mathbf{19 \text{ lb/in}^2}$$

$$f_{v,x} / F_{v,x'} = \mathbf{0.092}$$

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression;

Design bearing stress - Table 4.3.1;

Actual bearing stress;

$$R_x = \mathbf{155 \text{ lb}}$$

$$F_{c\_perp,x'} = F_{c\_perp} \times C_{Mc\_perp} = \mathbf{285 \text{ lb/in}^2}$$

$$f_{c\_perp,x} = R_x / (b \times L_b) = \mathbf{19 \text{ lb/in}^2}$$

$$f_{c\_perp,x} / F_{c\_perp,x'} = \mathbf{0.068}$$

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Check design at end of span

Tension members - cl.3.8

Design axial tension force;

Design tension parallel to grain - Table 4.3.1;

Actual tension parallel to grain;

$$P = \mathbf{3 \text{ lb}}$$

$$F_t' = F_t \times C_D \times C_{Ft} = \mathbf{936 \text{ lb/in}^2}$$

$$f_t = P / (b \times d) = \mathbf{0 \text{ lb/in}^2}$$

$$f_t / F_t' = \mathbf{0.000}$$

PASS - Design tension stress exceeds actual tension stress

Bending members - Flexure - cl.3.3

Design bending moment;

Design bending stress - Table 4.3.1;

Actual bending stress - eq.3.3-2;

$$M_x = \mathbf{602 \text{ lb\_ft}}$$

$$F_{b,x'} = F_b \times C_D \times C_{Fb} \times C_r = \mathbf{2093 \text{ lb/in}^2}$$

$$f_{b,x} = M_x / S_x = \mathbf{602 \text{ lb/in}^2}$$

$$f_{b,x} / F_{b,x'} = \mathbf{0.288}$$

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force;

Design shear stress - Table 4.3.1;

Actual shear stress - eq.3.4-2;

$$V_x = \mathbf{155 \text{ lb}}$$

$$F_{v,x'} = F_v \times C_D \times C_{Mv} = \mathbf{210 \text{ lb/in}^2}$$

$$f_{v,x} = 3 \times V_x / (2 \times b \times d) = \mathbf{19 \text{ lb/in}^2}$$

$$f_{v,x} / F_{v,x'} = \mathbf{0.092}$$

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression;

Design bearing stress - Table 4.3.1;

Actual bearing stress;

$$R_x = \mathbf{379 \text{ lb}}$$

$$F_{c\_perp,x'} = F_{c\_perp} \times C_{Mc\_perp} = \mathbf{285 \text{ lb/in}^2}$$

$$f_{c\_perp,x} = R_x / (b \times L_b) = \mathbf{47 \text{ lb/in}^2}$$

$$f_{c\_perp,x} / F_{c\_perp,x'} = \mathbf{0.167}$$

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Combined bending and axial loading - cl.3.9

Reference bending design value without C_L ;

Reference bending design value without C_v ;

Bending and axial tension - eq.3.9-1 and 3.9-2;

$$F_{bt1}^* = F_b \times C_D \times C_{Fb} \times C_r = \mathbf{2093 \text{ lb/in}^2}$$

$$F_{bt1}^{**} = F_b \times C_D \times C_{Fb} \times C_r = \mathbf{2093 \text{ lb/in}^2}$$

$$\max(f_t / F_t' + f_{b,x} / F_{bt1}^*, (f_{b,x} - f_t) / F_{bt1}^{**}) = \mathbf{0.288} < 1.0$$

PASS - Combined bending and tensile stresses are within permissible limits



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 376
Project Name: JKV-EMS		

Check design 2ft 2.991in along span

Bending members - Deflection - cl.3.5

Instantaneous deflection;

$$\delta_x' = \delta_x / C_{ME} = \mathbf{0.022 \text{ in}}$$

Allowable deflection;

$$\delta_{x, \text{Allowable}} = L_{m1\_s1} / 250 = \mathbf{0.187 \text{ in}}$$

$$\delta_x' / \delta_{x, \text{Allowable}} = \mathbf{0.119}$$

PASS - Allowable deflection exceeds instantaneous deflection

BRICK SUPPORT CORBEL ON EXISTING

In accordance with ACI318-14

Tedds calculation version 1.2.04

Design summary

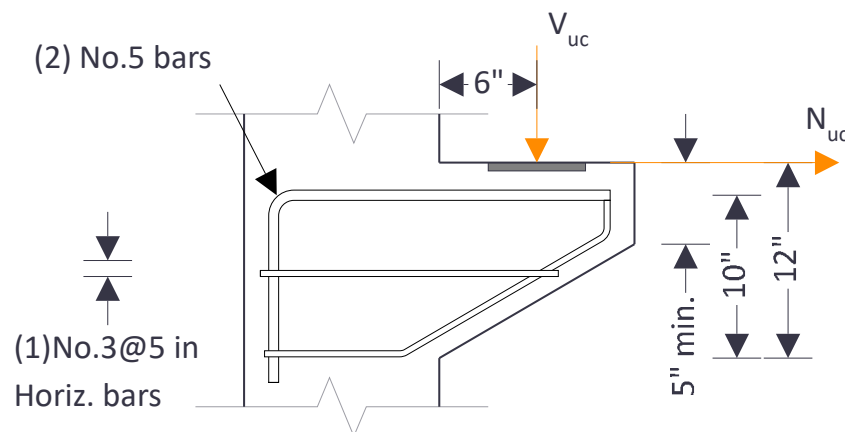
Overall design status;

PASS

Overall design utilization;

0.963

Description	Unit	Required	Provided	Utilization	Result
Nominal vertical load	kips	0.4	128	0.003	PASS
Tension reinforcement	in ²	0.427	0.614	0.695	PASS
Horizontal reinforcement	in ²	0.213	0.221	0.963	PASS
Horiz reinforcement spacing	in	6.667	5	0.75	PASS



;

Corbel geometry

Width of corbel;

b = **16.000 in**

Total thickness of corbel;

h = **12.000 in**

Depth to main reinforcement;

d = **10.000 in**

Material properties

Yield strength of reinforcement;

f_y = **60000 psi**

Compressive strength of concrete;

f_c = **4000 psi**



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 377
Project Name: JKV-EMS		

Type of concrete;

Unit weight of concrete;

Corbel construction;

Coefficient of friction;

Design load data

Factored vertical load ;

Distance of vertical load from face of column;

Horizontal force acting on corbel;

Strength reduction factor;

Vertical load capacity check

Nominal vertical load;

Nominal vertical load capacity;

Normal weight

$$\lambda = 1.00$$

Hardened & unroughened concrete

$$\mu = 0.6 \times \lambda = 0.60$$

$$V_u = 0.300 \text{ kips}$$

$$a = 6.000 \text{ in}$$

$$N_{uc} = 0.000 \text{ kips}$$

$$\phi = 0.750$$

$$V_n = V_u / \phi = 0.400 \text{ kips}$$

$$V_{n\_cap} = \min(0.2 \times f'_c \times b \times d, (480 \text{ psi} + 0.08 \times f'_c) \times b \times d, 1600 \text{ psi} \times b \times d) \\ = 128.000 \text{ kips}$$

PASS - Nominal vertical load is less than the nominal vertical load capacity

Find shear friction reinforcement

Minimum horizontal force acting on corbel;

Horizontal force acting on corbel;

For balanced condition;

Area for shear friction reinforcement;

$$N_{uc\_min} = 0.2 \times V_u = 0.060 \text{ kips}$$

$$N_{uc\_act} = \max(N_{uc}, N_{uc\_min}) = 0.060 \text{ kips}$$

$$j = 0.85$$

$$A_{vf} = V_n / (f_y \times \mu) = 0.011 \text{ in}^2$$

$$A_f = [(V_u \times a) + (N_{uc\_act} \times (h - d))] / (\phi \times f_y \times j \times d) = 0.005 \text{ in}^2$$

$$A_n = N_{uc\_act} / (\phi \times f_y) = 0.001 \text{ in}^2$$

Nominal area required;

Main tension reinforcement check

Minimum area of tension reinforcement;

Area of tension reinforcement required;

$$A_{sc\_min} = 0.04 \times f'_c \times b \times d / f_y = 0.427 \text{ in}^2$$

$$A_{sc} = \max([(2 \times A_{vf}) / 3] + A_n, A_f + A_n) = 0.009 \text{ in}^2$$

$$A_{sc\_req} = \max(A_{sc}, A_{sc\_min}) = 0.427 \text{ in}^2$$

2 No.5 bars

$$D_{tension} = 0.625 \text{ in}$$

$$N_{tension} = 2$$

$$A_{sc\_prov} = N_{tension} \times \pi \times D_{tension}^2 / 4 = 0.614 \text{ in}^2$$

PASS - Tension reinforcement is adequate

Horizontal reinforcement check

Minimum area of horizontal reinforcement;

Area of horizontal reinforcement required;

$$A_{h\_min} = 0.5 \times (A_{sc\_req} - A_n) = 0.213 \text{ in}^2$$

$$A_h = \max(A_{vf} / 3, A_f / 2) = 0.004 \text{ in}^2$$

$$A_{h\_req} = \max(A_h, A_{h\_min}) = 0.213 \text{ in}^2$$

1 No.3 bars

$$D_{horizontal} = 0.375 \text{ in}$$

$$N_{horizontal} = 1$$

$$A_{h\_prov} = 2 \times N_{horizontal} \times \pi \times D_{horizontal}^2 / 4 = 0.221 \text{ in}^2$$

PASS - Horizontal reinforcement is adequate

Horizontal reinforcement;

Diameter of horizontal bars;

Number of horizontal bars;

Area of horizontal reinforcement provided;

Maximum spacing of horizontal bars;

Actual spacing of horizontal bars;

$$S_{h\_max} = 2 \times d / (3 \times N_{horizontal}) = 6.667 \text{ in}$$

$$S_h = 5.000 \text{ in}$$

PASS - Horizontal bar spacing is adequate



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Made by: WNH	Date: 06/16/2025	Job No. 25-003
Revision:	Date:	Sheet No. 378
Project Name: JKV-EMS		

Design Summary

Single corbel 16.0" wide by 12.0" deep with 4000 psi concrete and 60000 psi steel

Provide main tension reinforcement as 2 No. 5 bars

Provide horizontal reinforcement as 1 No. 3 link bars at maximum 5.000" on center

;