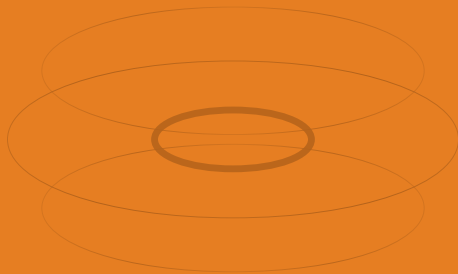


OWNER

Meter Skid Canopy Steel Design

Calculation



Document No.: [REDACTED] STR-CAL-003

Rev. A, Issued for Review

Date: September 29, 2026

Project: [REDACTED] [REDACTED]



TORUS

[REDACTED] [REDACTED], [REDACTED] County, [REDACTED]

ISSUE SUMMARY AND APPROVAL

This page records each issue of STR-CAL-003. The preparer, reviewer and approver sign for each revision.

Rev.	Date	Issue purpose	Description
A	2026-09-29	Issued for Review	First issue

Contributors

Rev.	Discipline	Prepared by	Reviewed by	Approved by
A	Structural			

CONTENTS

1.	OBJECTIVE.....	1
2.	METHODOLOGY	1
3.	DESIGN INPUTS.....	1
4.	ASSUMPTIONS.....	2
5.	REFERENCES	2
6.	CALCULATION.....	3
	6.1. LOADS	3
	6.2. FRAME ANALYSIS.....	3
	6.3. PURLINS	5
	6.4. RAFTERS	5
	6.5. COLUMNS.....	5
	6.6. BRACING	6
	6.7. BASE PLATES AND ANCHOR RODS	6
	6.8. SERVICEABILITY	6
	6.9. COLUMN REACTIONS FOR THE FOUNDATIONS	7
7.	RESULTS AND CONCLUSIONS	7

FIGURES AND TABLES

Figure 6-1. Rafter Bending Moment, Interior Frame 4

Table 3-1: Design Inputs.....1

Table 6-1: Free-Roof Net Pressure Coefficients 3

Table 6-2: Frame Analysis, Largest Factored Forces.....4

Table 6-3: Deflection Checks 7

Table 6-4: Governing Column Base Reactions, ASD.....7

Table 7-1: Member Design Summary 7

Table 7-2: Canopy Steel Take-off 8

APPENDICES

APPENDIX A. CANOPY FRAMING PLAN AND ELEVATIONS 9

1. OBJECTIVE

This calculation designs the steel sunshade canopy over meter skid SK-110: roof deck, purlins, rafters, columns, bracing, base plates and anchor rods. It checks every member for strength to AISC 360-22 and for deflection, and it gives the column base reactions used to design the canopy piers in STR-CAL-002, Foundation Design.

2. METHODOLOGY

The canopy keeps direct sun off the ultrasonic meters, their transmitters and the meter tubes, which limits the temperature difference between the gas and the tube wall and so protects the measurement. It spans the skid on 4 transverse frames at 17.33 ft, each a pair of columns rigidly connected to a rafter with 2 ft overhangs, under a 0.5 in 12 monoslope roof that falls to the south. The clear height is 14 ft at the low side, which clears the motor-operated valve actuators, and one bolted roof panel over each meter can be lifted off so a crane can remove the meter.

Each frame is fixed at its base. In the transverse direction the frames resist wind as portals; in the longitudinal direction the columns act as cantilevers and the purlins tie them together so all eight share the load. Vertical X-bracing in the long walls was the alternative. It was rejected because it would close off the long sides of the skid, which operators use to reach the valves and transmitters, and the cantilever columns carry the longitudinal load with a strength ratio below 0.2.

TORUS analyzed an interior frame and an end frame by the direct stiffness method, with the rafter divided into 0.5 ft elements and each column into four, for dead, roof live, snow, eight wind cases and seismic. Members were checked to AISC 360-22 by load and resistance factor design; base plates by AISC Design Guide 1; and anchor rods to ACI 318-19 Chapter 17. Snow enters the combinations at 1.0 S as the principal load and at 0.15 S with seismic, the ASCE 7-22 factors for its strength-level snow loads.

3. DESIGN INPUTS

The inputs are listed in Table 3-1. Wind, snow and seismic values come from STR-CAL-001, Structural Design Loads.

Table 3-1: Design Inputs

Input	Value	Source
Column grid	52 ft E-W x 20 ft N-S, 4 frames at 17.33 ft	Plot plan, STR-DWG-007
Roof	56 x 24 ft, slope 0.5 in 12 (2.4°), high side north	STR-DWG-007
Column height	14.0 ft south, 14.83 ft north, top of pier to rafter centerline	Actuator clearance

Input	Value	Source
Steel	W shapes A992, HSS A500 Grade C, $F_y = 50$ ksi; plates and angles A36	Project steel specification
Dead load	Deck 1.2 psf, lights and trays 1.8 psf, purlins 2.92 psf, plus member weight	Calculated
Roof live load L_r	20 psf	ASCE 7-22 Sec. 4.8
Snow S	10 psf	STR-CAL-001
Wind	108 mph, Exposure C; $q_h = 25.4$ psf; $K_d = 0.85$, $G = 0.85$	STR-CAL-001 (HOLD H-06)
Seismic	$S_{DS} = 0.128$; $R = 1.25$; $C_s = 0.1024$	STR-CAL-001 (HOLD H-06)
Concrete pier	30 in diameter, $f'_c = 4,000$ psi	STR-CAL-002

4. ASSUMPTIONS

1. The canopy clear height of 14 ft suits typical actuator heights for NPS 10 Class 600 ball valves. *Verification required against vendor data (HOLD H-15).*
2. The roof deck, 26 gauge R-panel through-fastened at every purlin, braces the purlin top flanges but is not relied on as a diaphragm.
3. Column effective length factor $K = 2.1$ in both directions, the cantilever value. It is conservative for the transverse frames.
4. The equipment under the canopy may block wind flow, so both clear and obstructed free-roof coefficients are enveloped.
5. Lights and cable trays hung from the purlins weigh no more than 1.8 psf over the roof area.

5. REFERENCES

1. ANSI/AISC 360-22, Specification for Structural Steel Buildings (2022).
2. AISC Steel Construction Manual, 16th Edition (2023), section properties.
3. AISC Design Guide 1, Base Connection Design for Steel Structures, 3rd Edition (2024).
4. ASCE/SEI 7-22, Minimum Design Loads and Associated Criteria for Buildings and Other Structures (2022).
5. ACI 318-19, Building Code Requirements for Structural Concrete, Chapter 17.
6. STR-CAL-001, Structural Design Loads; STR-CAL-002, Foundation Design.
7. STR-DWG-007, Canopy Framing Plan and Elevations (Appendix A).

6. CALCULATION

6.1. LOADS

The canopy is an open building with a free monoslope roof. The net wind pressure on each half of the roof is:

ASCE 7-22 Eq. 27.3-2

$$p = q_h K_d G C_N = 25.42(0.85)(0.85) C_N = 18.36 C_N \text{ psf}$$

with the net pressure coefficients C_N for a roof angle near zero given in Table 6-1. Positive values act downward. Each case is applied with the wind from the south and from the north. Wind along the canopy uses $C_N = -1.2$ over the end frame, the obstructed value within one roof height of the windward edge.

Table 6-1: Free-Roof Net Pressure Coefficients

Case	C_{NW} , windward half	C_{NL} , leeward half	Windward, psf	Leeward, psf
A, clear flow	+1.2	+0.3	+22.0	+5.5
B, clear flow	-1.1	-0.1	-20.2	-1.8
A, obstructed flow	-0.5	-1.2	-9.2	-22.0
B, obstructed flow	-1.1	-0.6	-20.2	-11.0

Note 1: ASCE 7-22 Figure 27.3-4, monoslope free roof, roof angle 0° to 7.5° .

Wind on the rafters and columns themselves uses $C_f = 2.0$ and is not less than the 16 psf minimum of ASCE 7-22 Sec. 29.7. The roof dead load is 5.9 psf plus member weight. Roof live load, 20 psf, governs over snow, 10 psf.

6.2. FRAME ANALYSIS

Figure 6-1 shows the rafter moment in the interior frame for the governing gravity case and for the largest uplift case. Under gravity the rafter sags between the columns and hogs at the knees, where the moment steps by the column moment. Under uplift the pattern reverses, and the rafter bottom flange is in compression across most of the span, which is why the bottom flange is braced.

Figure 6-1. Rafter Bending Moment, Interior Frame

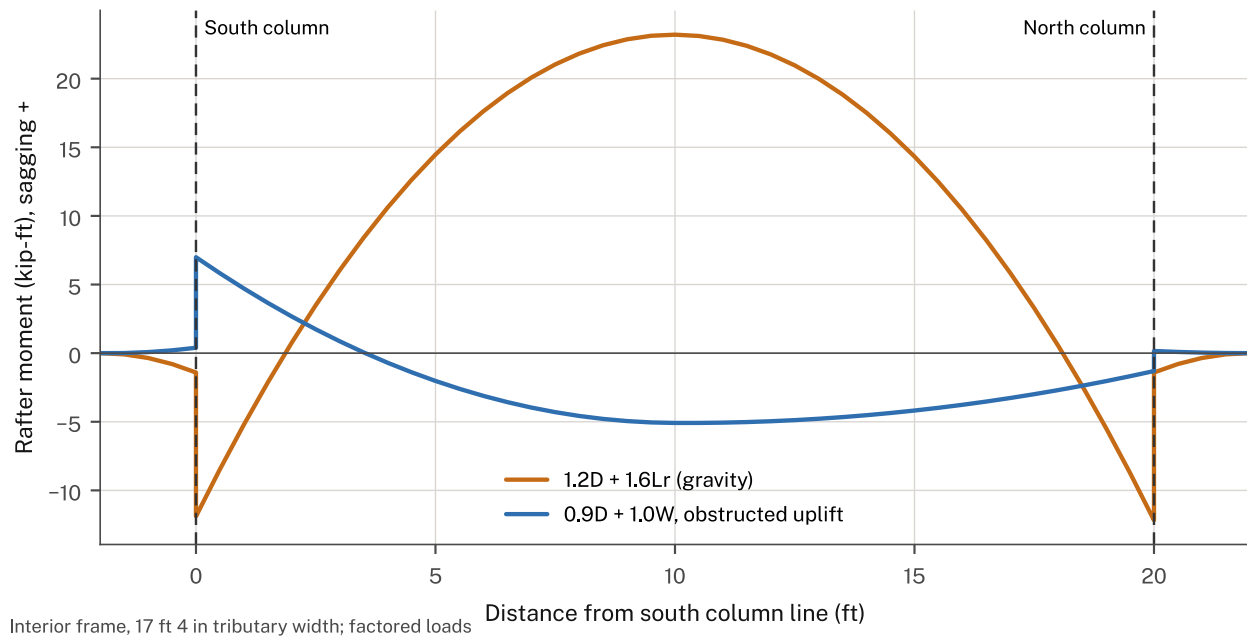


Table 6-2 gives the combinations that produce the largest member forces.

Table 6-2: Frame Analysis, Largest Factored Forces

Frame	Combination	Rafter +M, kip-ft	Rafter -M, kip-ft	South column M, kip-ft	North column M, kip-ft
Interior	1.2D + 1.6Lr + 0.5W (case A clear, from south)	27.4	-16.0	10.2	14.4
Interior	1.2D + 1.6Lr	23.2	-12.2	10.5	10.8
Interior	1.2D + 0.5Lr + 1.0W (case A clear, from south)	19.5	-13.1	4.3	12.3
Interior	1.2D + 0.5Lr + 1.0W (case A clear, from north)	19.2	-13.3	12.4	4.9
End	1.2D + 1.6Lr + 0.5W (case A clear, from south)	17.2	-10.1	6.3	9.1
End	1.2D + 1.6Lr	14.6	-7.7	6.6	6.8
Interior	0.9D + 1.0W (case A clear, from south)	12.9	-9.5	3.1	9.1
Interior	0.9D + 1.0W (case A clear, from north)	12.6	-9.7	9.3	2.3

6.3. PURLINS

Purlins are W8X10 at 4 ft, simple spans of 17.33 ft between frames. Hot-rolled sections were chosen over cold-formed Z-purlins so that every member of the canopy is hot-dip galvanized after fabrication and bolted with the same clip details. Under gravity the deck braces the top flange:

AISC 360-22 Sec. F2.1

$$w_u = 1.2(23.7) + 1.6(20)(4) = 156 \text{ lb/ft}, \quad M_u = \frac{w_u L^2}{8} = 5.87 \text{ kip}\cdot\text{ft}$$

$$\phi M_n = 0.9 F_y Z_x = 0.9(50)(8.87)/12 = 33.3 \text{ kip}\cdot\text{ft}$$

Under uplift, with $C_N = -1.2$, the net load is 67 lb/ft upward and the bottom flange is in compression. One line of bridging at midspan gives $L_b = 8.67$ ft, which lies between $L_p = 2.97$ ft and $L_r = 8.69$ ft; with $C_b = 1.14$ the design strength is 23.4 kip-ft against a demand of 2.51 kip-ft.

6.4. RAFTERS

The rafters are W10X22. The purlins brace the top flange at 4 ft. The bottom flange is braced by L2x2x1/4 fly braces at every second purlin, so its unbraced length is 8 ft. The largest sagging moment is 27.4 kip-ft, under 1.2D + 1.6Lr + 0.5W (case A clear, from south), and the largest hogging moment 16.0 kip-ft, under 1.2D + 1.6Lr + 0.5W (case A clear, from south):

AISC 360-22 Eq. F2-2

$$\phi M_n = 0.9 \left[M_p - (M_p - 0.7 F_y S_x) \frac{L_b - L_p}{L_r - L_p} \right] = 84.2 \text{ kip}\cdot\text{ft} \quad (L_b = 8 \text{ ft})$$

With the top flange braced at 4 ft, less than $L_p = 4.70$ ft, the full plastic moment 97.5 kip-ft is available for sagging. Shear is 8.4 kip against 73.4 kip. The W10X22 is larger than strength requires. It was chosen so that the bolted moment connection at the knee has enough depth, and to keep roof deflection small.

6.5. COLUMNS

The columns are HSS6X6X3/8, compact for flexure ($b/t = 14.2$). With $KL = 2.1 \times 14.83 \text{ ft} = 31.2 \text{ ft}$, $KL/r = 164$, below the 200 limit:

AISC 360-22 Eq. E3-2

$$F_{cr} = 0.658^{F_y/F_e} F_y = 9.3 \text{ ksi}, \quad \phi P_n = 0.9 F_{cr} A_g = 63.7 \text{ kip}$$

The governing combination for the column is 1.2D + 1.6Lr + 0.5W (case A clear, from south), with $P_u = 10.0 \text{ kip}$ and $M_u = 14.4 \text{ kip}\cdot\text{ft}$ against $\phi M_n = 59.2 \text{ kip}\cdot\text{ft}$:

AISC 360-22 Eq. H1-1b

$$\frac{P_u}{2\phi P_n} + \frac{M_u}{\phi M_n} = 0.32 \leq 1.0$$

Along the canopy, wind of 5.18 kip on the rafters seen edge-on and on the columns is shared by the eight cantilever columns, 0.65 kip each, with a base moment of 7.6 kip-ft and an interaction ratio of 0.13. The HSS6X6X3/8 was chosen over a lighter HSS6x6x1/4 to keep longitudinal drift low and to give the columns nearest the road some resistance to incidental impact; bollards protect the south columns.

6.6. BRACING

Each fly brace is a point brace for the rafter bottom flange. For the largest hogging moment $M_r = 191$ kip-in and $h_o = 9.84$ in:

AISC 360-22 Eq. A-6-7, A-6-8

$$P_{rb} = \frac{0.02 M_r C_d}{h_o} = 0.39 \text{ kip}, \quad \beta_{br} = \frac{1}{\phi} \frac{10 M_r C_d}{L_{br} h_o} = 2.70 \text{ kip/in}$$

The brace force along the L2x2x1/4 is 0.49 kip against a compression strength of 28.3 kip, and its axial stiffness, 1,140 kip/in, far exceeds the requirement. One line of L1-1/2x1-1/2x3/16 bridging at purlin midspan, anchored to the eave purlins, braces the purlin bottom flanges.

6.7. BASE PLATES AND ANCHOR RODS

The columns sit on PL 14 x 14 x 1 in base plates on 30 in piers. The concrete bearing strength is:

AISC 360-22 Eq. J8-2

$$f_{p, max} = \phi 0.85 f'_c \sqrt{A_2/A_1} = 0.65(0.85)(4.0)(1.90) = 4.20 \text{ ksi}$$

The governing base moment is 9.6 kip-ft with 6.5 kip axial, under 1.2D + 0.5Lr + 1.0W (case A clear, from south). Its eccentricity, 17.6 in, exceeds the critical value of 6.9 in, so the large-moment method of Design Guide 1 applies. The bearing length is 0.21 in and the rods on the tension side carry 5.9 kip:

AISC Design Guide 1

$$Y = \left(f + \frac{N}{2} \right) - \sqrt{\left(f + \frac{N}{2} \right)^2 - \frac{2P_u(e+f)}{q_{max}}}, \quad T_u = q_{max}Y - P_u$$

The plate needs 0.67 in; 1 in is provided. Four 3/4 in F1554 Grade 36 headed rods on a 10 in square, embedded 18 in, carry 4.4 kip tension and 0.42 kip shear per rod. Their ACI 318-19 Chapter 17 interaction is 0.48. The rods sit 7.9 in from the pier face, so concrete breakout is not relied on; the pier vertical bars are lapped with the rods as anchor reinforcement, which needs 0.20 in² and is provided by the pier cage.

6.8. SERVICEABILITY

Deflections under service loads are compared with their limits in Table 6-3.

Table 6-3: Deflection Checks

Check	Deflection	Ratio	Limit
Purlin, roof live load	0.18 in	L/1,143	L/240
Rafter midspan, roof live load	0.22 in	L/1,113	L/240
Rafter 2 ft overhang tip, roof live load	0.06 in		0.20 in
Frame drift, 0.6W transverse	0.19 in	h/940	h/100
Column drift, 0.6W longitudinal	0.37 in	h/483	h/100

Note 1: h/100 applies because the canopy has no cladding or glazing to damage.

6.9. COLUMN REACTIONS FOR THE FOUNDATIONS

Table 6-4 gives the service-level (ASD) column base reactions that govern the canopy piers in STR-CAL-002. Positive axial load is compression into the pier.

Table 6-4: Governing Column Base Reactions, ASD

Governs	Axial P, kip	Shear H, kip	Moment M, kip-ft	Column line	Combination
Maximum compression	6.62	0.40	0.95	South	D + 0.45W (case A clear, from south) + 0.75Lr
Maximum uplift	-1.41	0.58	3.70	North	0.6D + 0.6W (case B obstructed, from north)
Maximum moment	5.99	1.10	6.23	North	D + 0.45W (case A clear, from south) + 0.75Lr
Maximum shear	6.04	1.14	5.99	South	D + 0.45W (case A clear, from north) + 0.75Lr

Note 1: Longitudinal wind, not concurrent with the above: H = 0.39 kip and M = 4.55 kip-ft per column.

7. RESULTS AND CONCLUSIONS

Table 7-1 summarizes the member checks. Every ratio is at or below 1.0.

Table 7-1: Member Design Summary

Member	Section	Demand	Design strength	Ratio
Purlin, gravity	W8X10	5.87 kip-ft	33.3 kip-ft	0.18
Purlin, uplift	W8X10	2.51 kip-ft	23.4 kip-ft	0.11
Rafter, sagging	W10X22	27.4 kip-ft	97.5 kip-ft	0.28
Rafter, hogging	W10X22	16.0 kip-ft	84.2 kip-ft	0.19
Rafter, shear	W10X22	8.4 kip	73.4 kip	0.11

Member	Section	Demand	Design strength	Ratio
Column, transverse frame	HSS6X6X3/8	P 10.0 kip, M 14.4 kip-ft	Eq. H1-1b	0.32
Column, longitudinal	HSS6X6X3/8	M 7.6 kip-ft	Eq. H1-1b	0.13
Fly brace	L2x2x1/4	0.49 kip	28.3 kip	0.02
Base plate	PL 14x14x1	0.67 in required	1.00 in	0.67
Anchor rods	4 x 3/4 in F1554-36	N 4.4, V 0.42 kip per rod	ACI 318-19 Ch. 17	0.48

The steel quantities are listed in Table 7-2.

Table 7-2: Canopy Steel Take-off

Item	Quantity	Weight, lb
Purlins W8X10	7 lines x 56 ft	3,920
Rafters W10X22	4 x 24.0 ft	2,110
Columns HSS6X6X3/8	8	3,170
Base plates PL 14x14x1	8	440
Fly braces L2x2x1/4	32	130
Bridging L1-1/2x1-1/2x3/16	3 lines x 24 ft	130
Connections and bolts, 10 %		920
Total	5.4 tons	10,830
Roof deck, 26 gauge R-panel	1,411 sf with laps	

Note 1: Rounded items may not add exactly to the total, which is computed from unrounded weights.

TORUS draws the following conclusions:

1. The canopy is framed with W8X10 purlins at 4 ft, W10X22 rafters and HSS6X6X3/8 columns on 4 fixed-base frames, a total of 5.4 tons of steel.
2. The rafter bottom flanges shall be braced by fly braces at every second purlin, because uplift and knee moments put them in compression.
3. Column base plates are PL 14 x 14 x 1 in with four 3/4 in F1554 Grade 36 headed rods, 18 in embedment, lapped with the pier reinforcement.
4. The roof shall have one removable bolted panel over each meter so the meters can be lifted out by crane; the meter change-out procedure shall be coordinated with Measurement.
5. All canopy steel shall be hot-dip galvanized after fabrication, with galvanized A325 bolts.

The following items remain open: site wind, snow and seismic data (HOLD H-06) and actuator heights from the valve vendor (HOLD H-15). The canopy piers depend on the geotechnical report (HOLD H-07).

APPENDIX A. CANOPY FRAMING PLAN AND ELEVATIONS



24'-0" ROOF

B

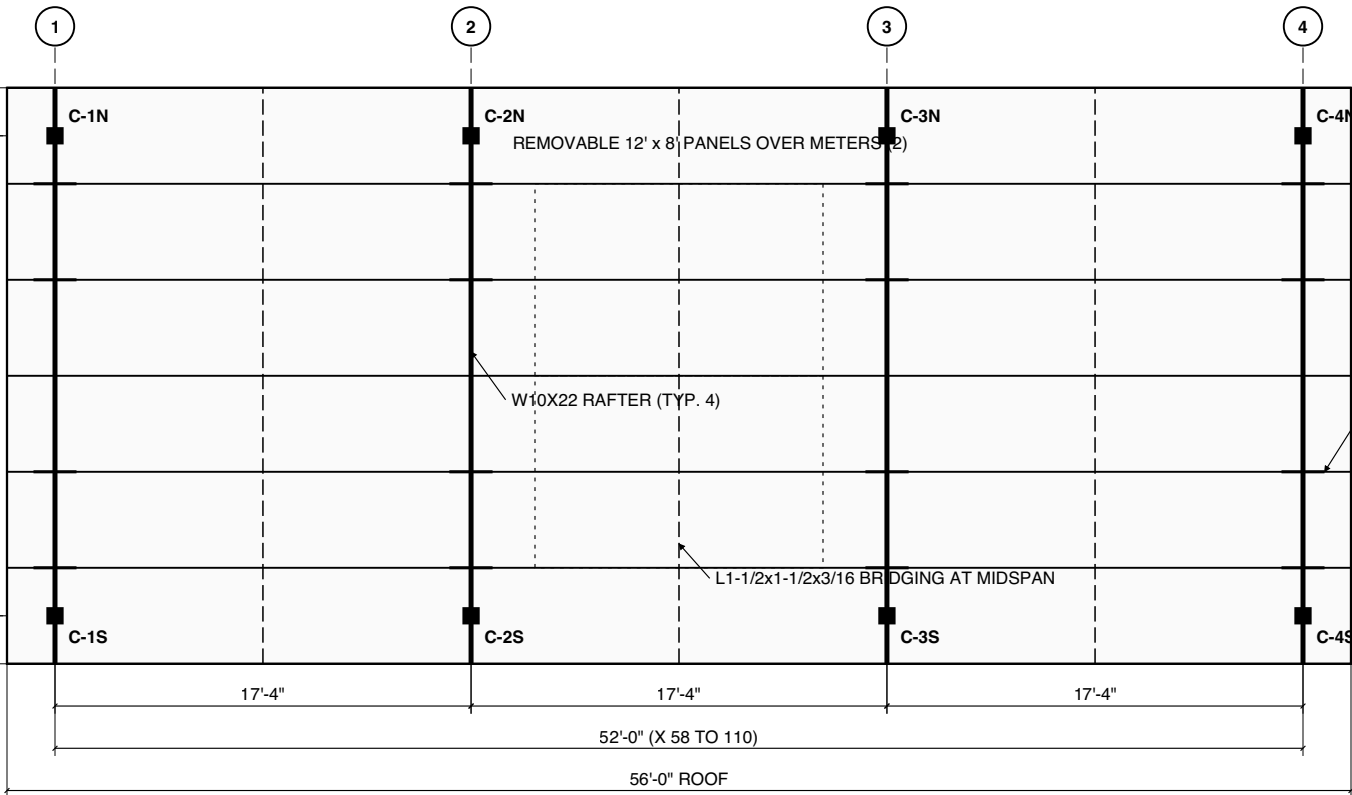
C

B

A

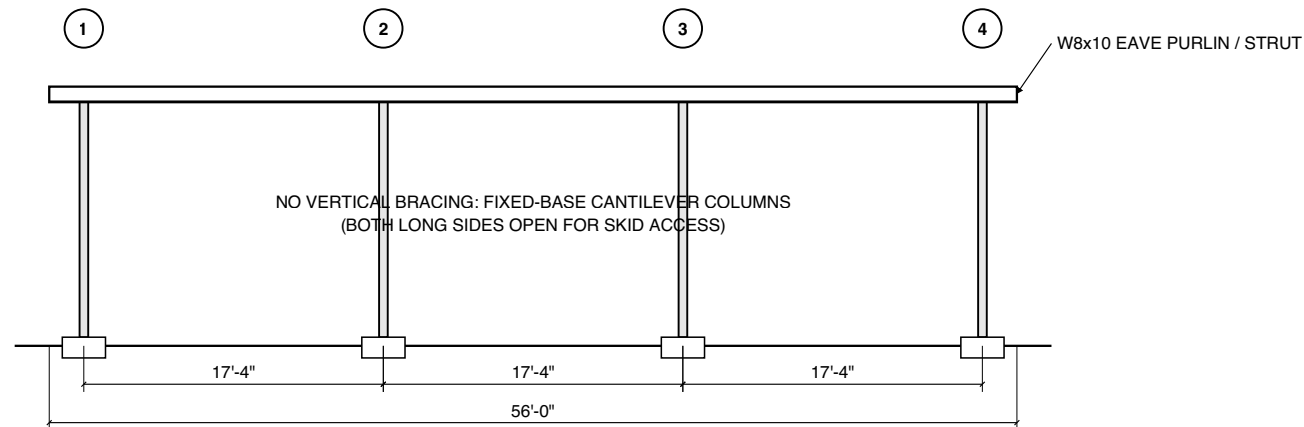
ROOF FRAMING PLAN

SCALE 1/8" = 1'-0" (APPROX.) GRID: 1 AT X = 58.0, S AT Y = 87.0



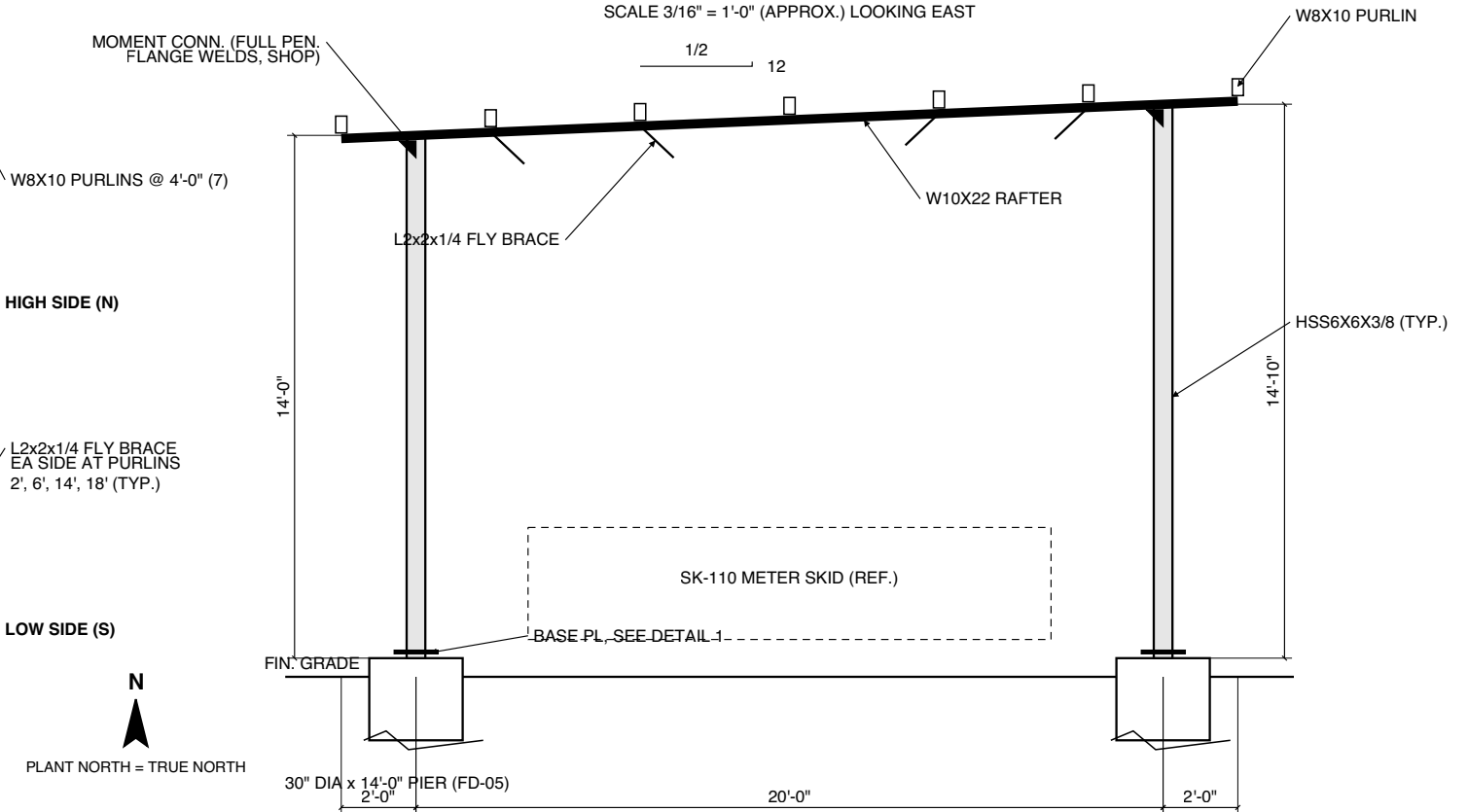
LONGITUDINAL ELEVATION (SOUTH LINE)

SCALE 3/32" = 1'-0" (APPROX.) LOOKING NORTH



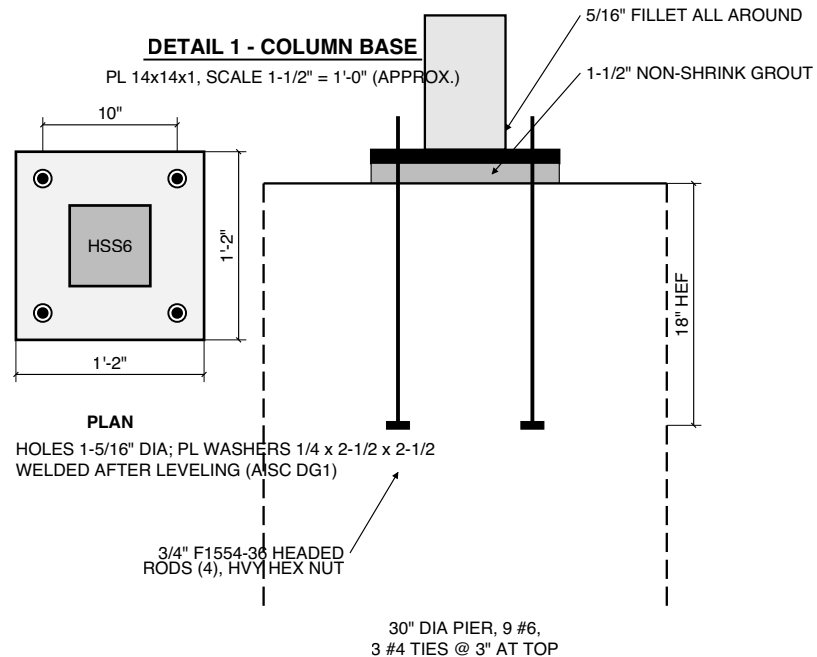
TRANSVERSE FRAME (GRIDS 1 TO 4)

SCALE 3/16" = 1'-0" (APPROX.) LOOKING EAST



DETAIL 1 - COLUMN BASE

PL 14x14x1, SCALE 1-1/2" = 1'-0" (APPROX.)



MEMBER SCHEDULE (STR-CAL-003)

MEMBER	SIZE	QTY	WT (LB)
PURLIN	W8X10	7 x 56'	3,920
RAFTER	W10X22	4 x 24'	2,114
COLUMN	HSS6X6X3/8	8	3,172
BASE PLATE	PL 14x14x1	8	444
FLY BRACE	L2x2x1/4	32	129
BRIDGING	L1-1/2x1-1/2x3/16	3 x 24'	130
CONN. / MISC	10 %		921
TOTAL			10,829 (5.41 T)

GENERAL NOTES:

- W SHAPES ASTM A992; HSS A500 GR C (FY 50); PL, L A36.
- HOT-DIP GALVANIZE ALL STEEL AFTER FAB (ASTM A123).
- BOLTS A325 GALV. 3/4" MIN; PURLIN CLIPS 2 BOLTS.
- ROOF: 26 GA GALVALUME R-PANEL, THROUGH-FASTENED, 1,411 SF INCL. LAPS; PURLIN TOP FLANGE BRACING.
- REMOVABLE BOLTED PANELS OVER EACH METER (METER REMOVAL BY CRANE), COORDINATE WITH MEA.
- COLUMN LOCATIONS AND PIERS: STR-DWG-001/-004.
- PROTECT SOUTH COLUMNS WITH BOLLARDS (STR-DWG-001).

NOTES / REFERENCES

- DESIGN PER STR-CAL-003 (AISC 360-22 LRFD); ASCE 7-22 V 108 MPH EXP C, Lr 20, S 10 PSF (H-06).
- TRANSVERSE: FIXED-BASE PORTAL FRAMES; LONGITUDINAL: FIXED-BASE CANTILEVER COLUMNS.
- COLUMN GRID: 1 TO 4 AT X = 58.0, 75.3, 92.7, 110.0; S AT Y = 87.0, N AT Y = 107.0 (LOCAL GRID).
- HEIGHTS ARE FROM TOP OF PIER (FG + 6") TO RAFTER CENTERLINE.
- ANCHOR RODS: ASTM F1554 GR 36 GALV., 4 x 3/4" PER COLUMN, 10" SQUARE, HEF 18".
- STEEL TAKE-OFF 5.41 TONS (10,829 LB) INCL. 10 % CONNECTIONS.

REVISIONS

REV	DATE	DESCRIPTION	BY	CHK	APP
A	2026-09-29	ISSUED FOR REVIEW (IFR)			

PRELIMINARY - ISSUED FOR REVIEW - NOT FOR CONSTRUCTION

ENGINEER OF RECORD SEAL

NOT SEALED
PRELIMINARY

PE NO. _____

PROJECT

120 MMSCFD RECEIPT INTO _____
COUNTY, _____
DISCIPLINE: STRUCTURAL
SCALE: AS NOTED

SK-110 CANOPY FRAMING

PLAN, ELEVATIONS AND DETAILS

DRAWN: _____ CHECKED: _____ APPROVED: _____

DWG NO.

STR-DWG-007

SHEET 1 OF 1 REV A