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STRUCTURAL DOCUMENTATION PACKET

DATE: 8/28/2024
PROJECT NUMBER: 23-CHK004

PROJECT:
Chiko Premier Carport Generic Design
Comprehensive Structural Analysis
State of New Jersey

CLIENT:
Chiko Solar Mounting Solutions
No. 878 Cheng Lui Rd.
Jaiding District, Shanghai 201808, China

PREPARED BY:
Rachel Keith
Engineered Power Solutions, Inc.

ENGINEER OF RECORD:
Paul J. Przybylinski
LSEA Corporation



PACKET EXPIRES ON 12/31/26 AND IS SUBJECT TO
ANNUAL REVIEW & RENEWAL

This packet is valid for this project at 185 Thomas Street, Bloomfield, NJ 07003



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1.0 – RESULTS & SCOPE OF WORK

1.1 – Overview of Analysis & Results

- **Governing Building Code:**

2021 New Jersey UCC Building Sub-code 2021

Based on the 2021 *International Building Code (IBC)* and referencing the 2016 *Minimum Design Loads for Buildings and Other Structures* by the American Society of Civil Engineers (ASCE 7-16)

- **Project Description:**

The project consists of the generic structural analysis of a ground mounted photovoltaic (PV) carport structure based on a number of set design conditions as listed in this packet. The carport structure consists of steel racking components which support between (9) and (108) PV modules in portrait (3x3 to 3x12) (rail parallel to long side of module). The Premier Carport structure resists wind uplift loads using a combination of self-weight and either concrete pier foundations, concrete spread footings, or concrete grade beams embedded into the ground.

Chiko Solar Mounting Solutions (Chiko) has hired Engineered Power Solutions, Inc. (EPS) to address the structural design of the Premier Carport components, connections, and concrete foundations. This packet provides a summary of the acceptable conditions and parameters based on the structural analysis for each of the design parameters listed in this packet.



- Specific Design Requirements and Features of the Chiko Premier Carport

- Each specific project shall be reviewed by the Engineer of Record (EOR) to ensure the site-specific design conditions (wind speed, exposure, snow load, seismic accelerations, etc.) are within the scope of this packet. An approval memo for each specific project shall be submitted by the EOR to validate the use of this packet. This packet is not valid without a site-specific approval memo and EPS (and the project EOR) take no responsibility for any projects installed without a valid (stamped and signed) site-specific approval memo.
- Risk Category: Although the structure itself presents a low risk to human life in the event of failure, EPS considers the Chiko Premier Carport a Risk Category II structure due to the regular interaction people will conduct with the structure (ruling out Risk Category I).
- Module Size: The analysis in this packet is based on the following module size:
 - Large-Format Modules (LFM): size is approximately 90" x 45"

The Chiko Premier Carport can also be used for smaller modules (assuming the carport geometry remains the same), as they will have less area and therefore would be conservative. EPS shall be notified in writing of any projects that use modules larger than 90" x 45" in either dimension.



- **Geotechnical Report:** As this is a generic packet, EPS has not been provided with a geotechnical report and therefore has used the code minimum soil values presented in IBC Table 1806.2 – Presumptive Load-Bearing Values. It is assumed that the soil type is clay, silt, sand, or gravel and concrete piers, spread footings, or grade beams are a viable foundation type. EPS shall be notified if bedrock, cobbles/boulders, or if any other soil conditions are encountered that do not allow the concrete foundations to be installed to their required depth, or if the site has the potential for unique geotechnical conditions (shallow bedrock, deep frost depth, frost jacking forces, expansive soils, high differential settlement, etc.). EPS has conservatively defaulted to using Class 5 material values for the design of the foundations. However, EPS recommends the owner obtain a geotechnical report prior to project design. EPS is **NOT** responsible for the classification of the soil.

- **Special Inspection:** "Special Inspections" as defined in Chapter 17 of the IBC are **not** required for the Chiko Premier Carport per the Exemptions listed in Section 1704.2 unless required otherwise (in writing) by the building official and/or AHJ.

- **Lateral Loads:** The Chiko Premier Carport resists lateral loads imposed by wind and seismic through the concrete pier foundations, concrete spread footings, or concrete grade beams. See Section 4.0 for additional anchorage information.



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- Ground Slope:

It is assumed that the ground slope is negligible and consistent and that there are no significant ground undulations. A site-specific analysis is required for sites with a significant slope for the Chiko Premier Carport as well as for sites with significant ground undulations.

- Rail Cantilevers: Rails may only cantilever the prescribed length given in this packet and on any approved (stamped and signed) Chiko drawings.



- **Scope of Work:**

The solar designer (Chiko) has hired EPS to address the following structural items:

- Determination of the structural requirements for the Chiko Premier Carport according to the governing building code (*International Building Code* (IBC) and ASCE 7) and based on the following parameters:
 - 5° Racking Tilt Angle
 - Risk (Occupancy) Category II Structure
 - Maximum Design Wind Speed (ASCE 7-16 (3) second gust wind speed) as described in Section 3.2.5 per carport size and loading scenario.
 - Wind Exposure Category "C"
 - No Topographic Features ($K_{zt} = 1.00$)
 - Maximum Ground Snow Load (p_g) as described in Section 3.2.5 per carport size and loading scenario.
 - Seismic Response Coefficient S_{DS} equal to 1.500g or less* for all Premier Carport sizes except the 4 Car Carport which is 1.300g or less* as described in Section 3.2.4.
*Note: Foundation sizes are dependent on S_{DS} value.
- Structural evaluation of the racking components.
- Racking connection design.
- Concrete foundation design.
- General Notes and Structural Specifications.



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- **Not within EPS's Scope of work:**
 - Design and Structural Justification of the module capacity and module to rail connection.
 - Design parameters outside of those listed in this packet.
 - Classification of soil type or site-specific soil conditions/requirements.
 - Corrosion protection (including dissimilar metal issues), water intrusion, electrical, mechanical, and other non-structural issues.
 - Installation means and methods, site safety, corrosion protection, electrical/mechanical issues, and all other non-structural issues.

- **Referenced As-Built Documents:**
 - Various Part Details and Drawings by Chiko Solar Mounting Solutions (See Appendix)

- **Results:** EPS has determined that if the Chiko Premier Carport, if built in accordance with the requirements given in Chiko's Plans/Drawings, Chiko's installation instructions, and the requirements listed in this packet, will be structurally adequate to support the imposed design loads listed in this packet.

EPS (or the EOR) shall supply a site-specific approval memo, stamped and signed by a licensed Professional Engineer (P.E.), for each individual project confirming the application of this packet to the project under consideration. This packet is not valid without the site-specific approval memo.

Please note that this Structural Documentation Packet is part of the construction documents and the project shall be constructed in accordance with the approved plan set and the requirements in this packet.



- **Limitations and Assumptions:**

Construction documents, details, plans, and product information regarding the proposed carport racking has been provided to EPS by Chiko. Any changes to the installation requirements given in Chiko's plans must be approved in writing by EPS prior to implementation. All non-structural issues including but not limited to waterproofing, corrosion protection, electrical, drainage, and mechanical issues are not the responsibility of EPS and must be addressed by the solar designer, installer, and/or owner before PV installation begins. EPS is not responsible for water intrusion issues, corrosion, other external sources of damage, or any failures due to water damage, material defects, other external sources of damage, incorrect construction issues, or improper installation/inspection/maintenance.

Although this packet may be used to confirm design load limitations, it shall be noted that a site-specific analysis to determine the site-specific design parameters and local code requirements is required by a qualified entity for individual projects. Sites with design scenarios which differ and/or are outside the scenarios specified in this packet must be addressed by a licensed design professional on a site-specific basis. Any changes to the required design results given in this packet such as changes to framing member sizes, module sizes and quantity, differing tilts, installation requirements, etc. must be approved in writing by EPS (or the Engineer of Record (EOR)) prior to implementation.



1.2 – General Notes and Structural Specifications

- General Notes:
 - The purpose of this structural documentation packet is to provide the items listed in the scope of work with a minimum level of resistance to the code prescribed forces (gravity and lateral) based on the requirements given in the governing building code.
 - The contractor is responsible for construction means and methods and the safety of personnel on site. EPS does not have control of site procedures and construction means and methods and therefore is not responsible for those means and methods and/or site safety.
 - Any changes to the approved plans and details for any reason must have written approval of the Engineer of Record before being implemented.
 - All inspection reports filed by a building official or special inspector shall be provided to the Engineer of Record.
 - See the "Limitations and Assumptions" in section 1.1 for additional requirements.
 - All drawings and details are not to be scaled.



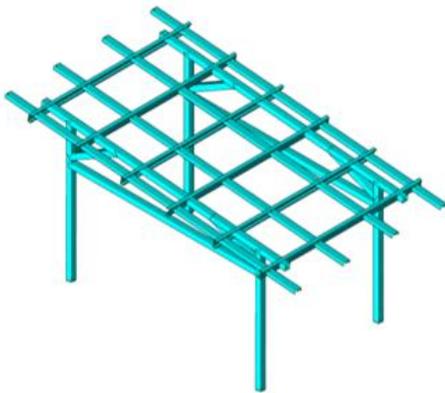
- Material Specifications:
 - Concrete (if required):
 - All concrete work shall conform to ACI standards and specifications.
 - 4,000 psi Compressive Strength min. (f'c) at 28 days
 - Type II Cement
 - Maximum slump of 5"
 - Concrete shall be air entrained in accordance with IBC Section 1904.2 and ACI 318
 - Special Inspection is not required unless required by the Building Department/Official.
 - Aggregates shall be per ASTM C33 (Max. size 1-1/2")
 - The mix design is to be adjusted for local soil and exposure conditions and shall be approved by the Engineer of Record.
 - Reinforcing Steel (if Required):
 - Shall be per ASTM A615 Grade 60 (deformed)
 - Min Fy = 60 ksi for #4 bars and up; Fy = 40 ksi for #3 bars
 - Shall be free of rust and dirt
 - Rebar splices shall have a lap no less than 48 bar diameters unless noted otherwise.
 - All clear cover distances shall be 3" unless noted otherwise.
 - All steel members shall have a minimum yield strength (Fy) as follows:
 - Connection Hardware: 34 ksi
 - M-Rails: 50 ksi
 - C-Rails (purlins): 50 ksi
 - Beams, Kickers, Braces, Columns: 50 ksi
 - All bolts shall be A307 (or approved equivalent) unless noted otherwise.

This Structural Documentation Packet is the property of EPS and shall not be used on any project other than the one referenced on the title sheet without written approval from the Engineer of Record.

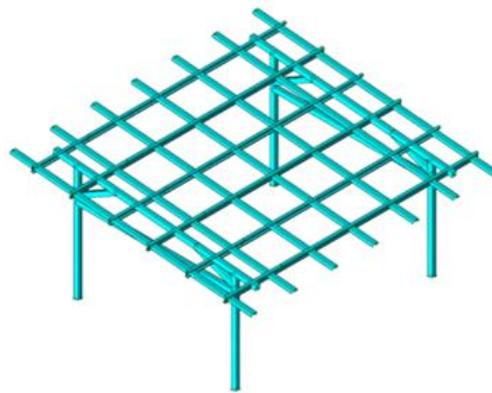


2.0 – PREMIER CARPORT DIMENSIONS

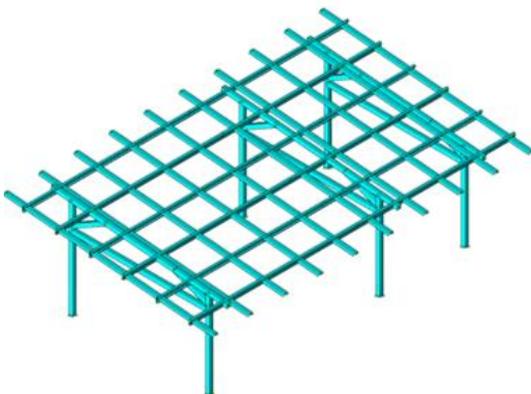
See Appendix for Carport Dimensions.



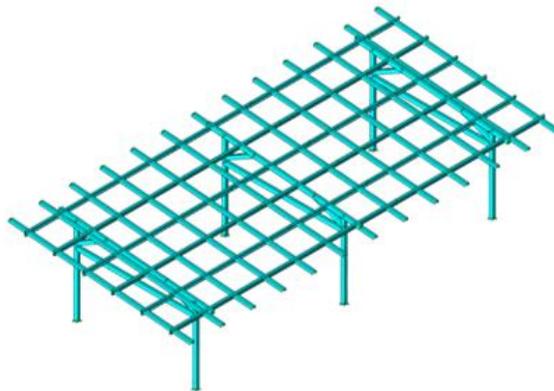
1 Car Carport



2 Car Carport



3 Car Carport



4 Car Carport

The Chiko Premier Carport has (4) different configurations:

- 1 Car Carport which supports 9 modules (3x3 layout) on 4 columns
- 2 Car Carport which supports 18 modules (3x6 layout) on 4 columns
- 3 Car Carport which supports 27 modules (3x9 layout) on 6 columns
- 4 Car Carport which supports 36 modules (3x12 layout) on 6 columns



3.0 – GRAVITY AND LATERAL CALCULATIONS

3.1 – Site Design Parameters

- Ground Mount Geometry:**

Array Size:	
1 Car Carport	3 mod. (tall), 3 mod. (wide)
2 Car Carport	3 mod. (tall), 6 mod. (wide)
3 Car Carport	3 mod. (tall), 9 mod. (wide)
4 Car Carport	3 mod. (tall), 12 mod. (wide)
Max. Module Size/Weight:	LFM: 90" x 45" Modules to have a max. weight of 3.0 psf
Max. Array Height:	≤ 15 ft.
Array Tilt:	5°

- Wind Design Parameters:**

Wind Speed (3 second gust) (V):	Varies
Exposure Category	C
Wind Directionality Factor (Kd):	0.85
Velocity Pressure Exposure (Kh):	0.85
Topographic Factor (Kzt):	1.00
Risk Category:	II
Guest Factor (G)	0.85

- Snow Loads:**

Ground Snow Load (p _g):	Varies
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- Seismic Design Parameters:**

Site Class (Assumed per IBC 1613.2.2):	D
S _{DS} :	≤ 1.5g (1-3 Car Carports) < 1.3g (4 Car Carport)
Seismic Design Category:	D



- **Soil Design Parameters (per Table 1806.2):**

It is assumed that the soil type is clay, silt, sand, or gravel and concrete piers, spread footings, or grade beams are a viable foundation type. EPS shall be notified if bedrock, cobbles/boulders, or if any other soil conditions are encountered that do not allow the concrete foundations to be installed to their required depth. If the site has the potential for unique geotechnical conditions (shallow bedrock, deep frost depth, frost jacking forces, expansive soils, high differential settlement, etc.) EPS recommends the owner obtain a geotechnical report prior to project design. EPS has conservatively defaulted to using Class 5 material values for the design of the foundations. However, EPS recommends the owner obtain a geotechnical report prior to project design. EPS is NOT responsible for the classification of the soil.



3.2 – Premier Carport Design Loads

The following pages document the determination of the imposed design loads based on the parameters listed in Section 3.1, the proposed materials, and the requirements given in the governing building code.

3.2.1 – Dead Loads

The modules for the residential carport are assumed to be large-format modules mounted with the rail parallel to long side of module. The maximum listed weight is 3.0 psf with a size of 90" x 45". Modules are attached to the rails along their length (90") transferring a maximum load of 11.25 plf per module to interior rails and 5.7 plf per module to exterior rails.

3.2.2 – Snow Loads

The ground snow load used in analysis varies per carport and per wind speed (described in Section 3.2.5) between 0 psf and 45 psf. For the purposes of this explanation, EPS will demonstrate the calculation of loads for 30 psf ground snow load. All other snow loads have been calculated in a similar manner.

Per ASCE 7 section 7.3, the flat roof snow load is determined by equation 7.3-1 where C_e has conservatively been taken as 1.0 (partially exposed), C_t has been taken as 1.2 (open air structure), and I_s has been taken as 1.0. Per section 7.4, the sloped roof snow load (p_s) is $C_s * p_f$ where p_f is the flat roof snow load and C_s would be determined per Figure 7.4-1 7-2c for an unobstructed slippery surface. The snow loads have been analyzed for a ground snow load of 30 psf which equates to a sloped roof snow load of 25.2 psf*.

*Note: due to the 5° slope per ASCE 7-16 Section 7.3.4, EPS has considered use of the minimum snow load when necessary, using the following equation: $p_s = I_s * p_g$



3.2.3 – Wind Loads

The maximum wind loads (normal to the module surface) were calculated using the wind factors listed in 3.1 of this packet and per ASCE 7-16 Section 27.3.2 with Figure 27.3-4 for a 5° tilt. The linear forces to the rails represent the distributed loads at each rail with the Back (tall) column representing the line load on the rails on the upper half of the array and the Front (short) representing the line load on the rails on the lower half of the array. All (4) wind cases have been analyzed in the structural model as shown later in this packet (example case provided on the following page).



Wind Pressures and Loads for Ground Mounted PV Systems (ASCE 7-16)

Governing Building Code:	2021 IBC
Wind Speed (3 Second Gust Speed) (V):	110 MPH
Exposure Category:	C
Wind Directionality Factor (K_d):	0.85
Velocity Pressure Exposure Coefficient (K_z):	0.85
Topographic Factor (K_{zt}):	1.00

Velocity Pressure (q): **22.38 psf**

Gust Effect Factor (G): 0.85

Module Width: 45.00 in.

Module Length: 90.00 in.

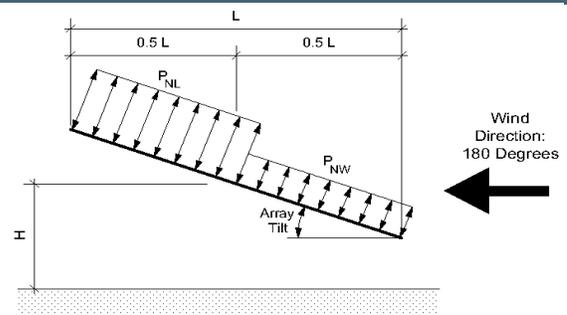
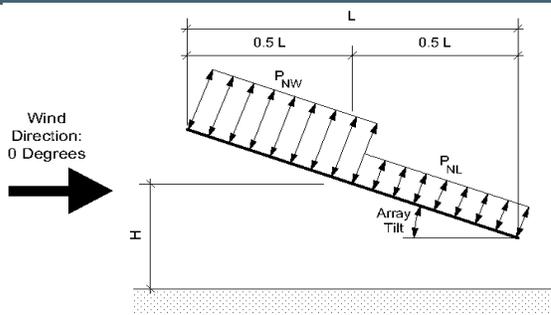
Module Area: 28.13 ft²

Tributary Width of Rails: 3.75 ft.

Module Weight 85.0 lbs.

GC_N Factors & Design Wind Pressures Normal to Module Face

Array Tilt	Wind Direction	Module Zone	C _{NW}	C _{NL}	Wind Pressures (psf)		Forces on Rails (plf)	
					P _{NW}	P _{NL}	Back (Tall)	Front (Short)
5°	0°	A	1.2	0.3	22.8	5.7	85.6	21.4
		B	-1.1	-0.1	-20.9	-1.9	-78.5	-7.1
	180°	A	1.2	0.3	22.8	5.7	21.4	85.6
		B	-1.1	-0.1	-20.9	-1.9	-7.1	-78.5





3.2.4 – Seismic Loads

The maximum assumed spectral accelerations are shown below:

- o Site Class = D (Assumed per IBC 1613.2.2)
- o S_{DS} =

1 Car Carport	1.50g
2 Car Carport	1.50g
3 Car Carport	1.50g
4 Car Carport	1.30g

Based on these accelerations the worst-case Seismic Design Category is “D”. Using a seismic importance factor of 1.00, and an R of 1.25 based on ASCE 7-16 Table 12.2-1 for a Steel Ordinary Cantilever Column System, the base shear can be calculated:

- o Base Shear (V) = $C_s * W$ (per Equation 12.8-1)
 - W = weight (dead load)
 - Governing C_s (1, 2, and 3 Car Carports) = $S_{DS} / (R/I) = 1.2$
 - Governing C_s (4Car Carport) = $S_{DS} / (R/I) = 1.04$

$$V \text{ (1, 2, and 3 Car Carports)} = 1.2 * W \text{ (Strength Level)}$$

$$V \text{ (4 Car Carport)} = 1.04 * W \text{ (Strength Level)}$$

Using the weight of the modules and tributary area to each module to purlin connection point, a seismic force has been calculated for each direction:

- o Solar module weight = 3.0 psf
- o Tributary width to each interior connection point = 3.75 ft.
- o Dead load per interior connection point (line load) = 11.25 plf
- o Seismic force in each direction at each interior point:
 - 1, 2, and 3 Car Carports
 - $V_{max.} = 1.2 * 11.25 \text{ lbs.} = 13.5 \text{ plf}$
 - 4 Car Carport
 - $V_{max.} = 1.04 * 11.25 \text{ lbs.} = 11.7 \text{ plf}$

These forces are listed in the following racking analysis.
(Note that the self-weight of the racking and framing has been taken into account in models)



3.2.5 – Acceptable Loading Scenarios

Each size of the Chiko Premier carport has been analyzed for different ASCE 7-16 wind speeds in combination with ground snow loads. The following tables show boxes filled in green that are the allowable wind speed and snow load that each carport size of the Chiko Premier carport can accommodate.

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Allowable Wind and Snow Load Combinations per Carport Size

1 Car Carport		Ground Snow Load (pg)					
		10 psf	20 psf	30 psf	40 psf	50 psf	60 psf
ASCE 7-16 Wind Speed (3 sec. gust) (V)	100 mph					NP	NP
	110 mph					NP	NP
	120 mph					NP	NP
	130 mph					NP	NP
	140 mph				NP	NP	NP
	150 mph				NP	NP	NP

2 Car Carport		Ground Snow Load (pg)					
		10 psf	20 psf	30 psf	40 psf	50 psf	60 psf
ASCE 7-16 Wind Speed (3 sec. gust) (V)	100 mph		NP	NP	NP	NP	NP
	110 mph		NP	NP	NP	NP	NP
	120 mph		NP	NP	NP	NP	NP
	130 mph	NP	NP	NP	NP	NP	NP
	140 mph	NP	NP	NP	NP	NP	NP
	150 mph	NP	NP	NP	NP	NP	NP

3 Car Carport		Ground Snow Load (pg)					
		10 psf	20 psf	30 psf	40 psf	50 psf	60 psf
ASCE 7-16 Wind Speed (3 sec. gust) (V)	100 mph		NP	NP	NP	NP	NP
	110 mph		NP	NP	NP	NP	NP
	120 mph		NP	NP	NP	NP	NP
	130 mph	NP	NP	NP	NP	NP	NP
	140 mph	NP	NP	NP	NP	NP	NP
	150 mph	NP	NP	NP	NP	NP	NP

4 Car Carport		Ground Snow Load (pg)					
		10 psf	20 psf	30 psf	40 psf	50 psf	60 psf
ASCE 7-16 Wind Speed (3 sec. gust) (V)	100 mph		NP	NP	NP	NP	NP
	110 mph		NP	NP	NP	NP	NP
	120 mph	NP	NP	NP	NP	NP	NP
	130 mph	NP	NP	NP	NP	NP	NP
	140 mph	NP	NP	NP	NP	NP	NP
	150 mph	NP	NP	NP	NP	NP	NP

NP – Not Permitted



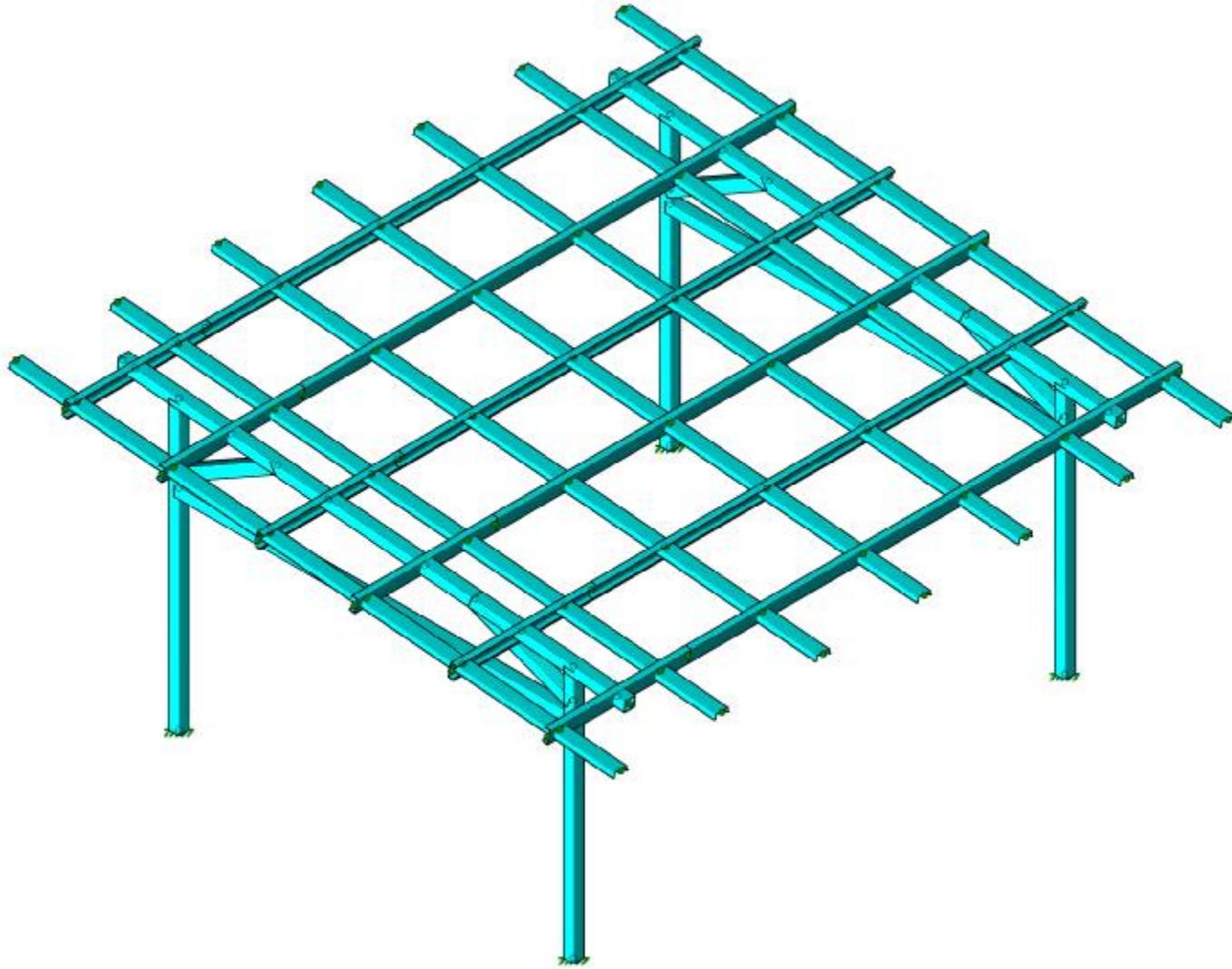
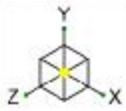
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3.3 – Carport Framing Analysis

The following pages document an example of the structural design of the Chiko Premier Carport under an envelope of all applicable load combinations given in ASCE 7-16.

The following analysis and results are for the 2 Car Chiko Premier Carport with a 5° tilt, an 18" diameter concrete pier, Class 5 Soils, an S_{DS} Value = 1.5g, and 110 MPH wind speed (ASCE 7-16 wind speed) with 10 psf ground snow load.

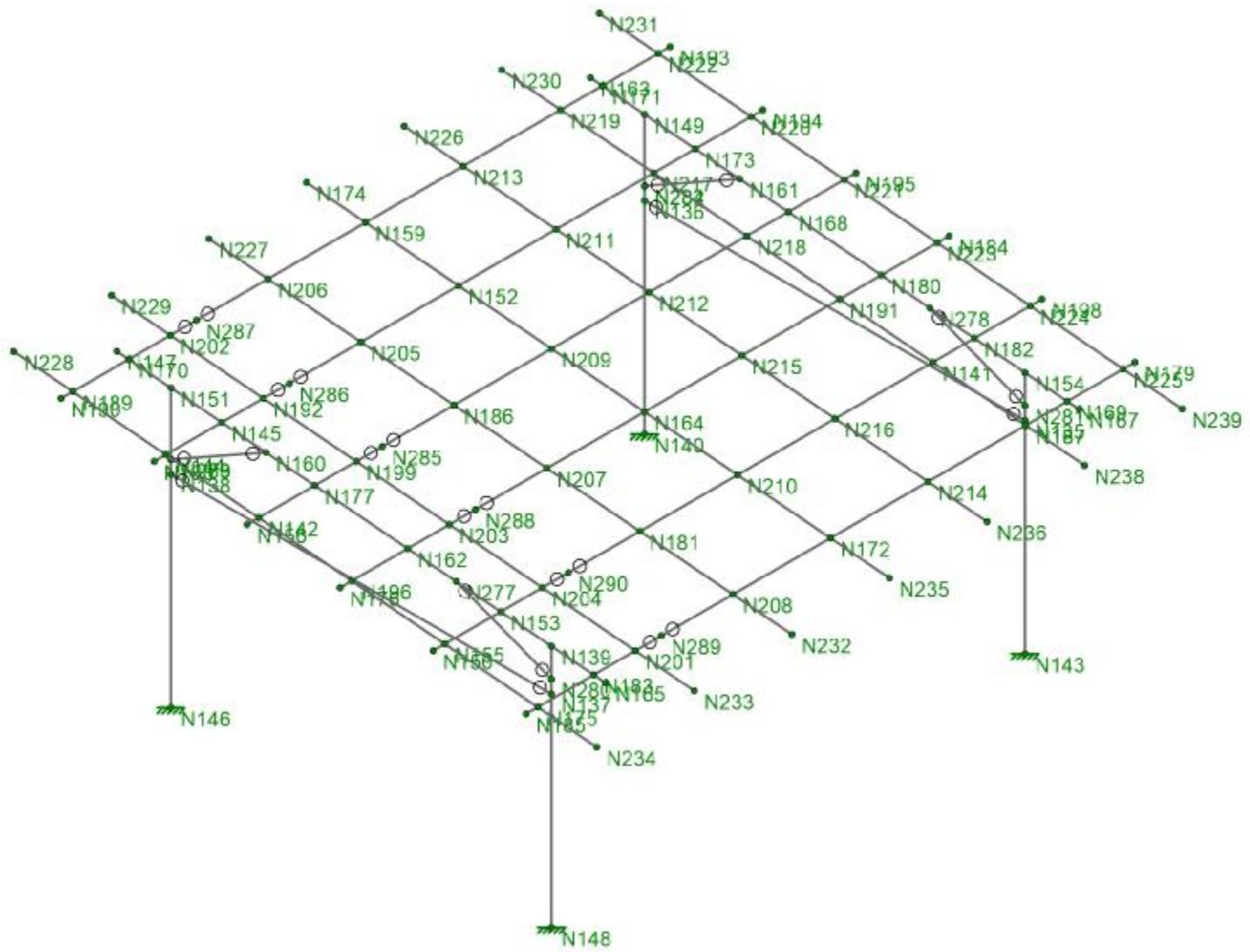
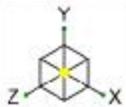
The following example may or may not reflect the worst-case connection forces from all carport sizes and loading scenarios.



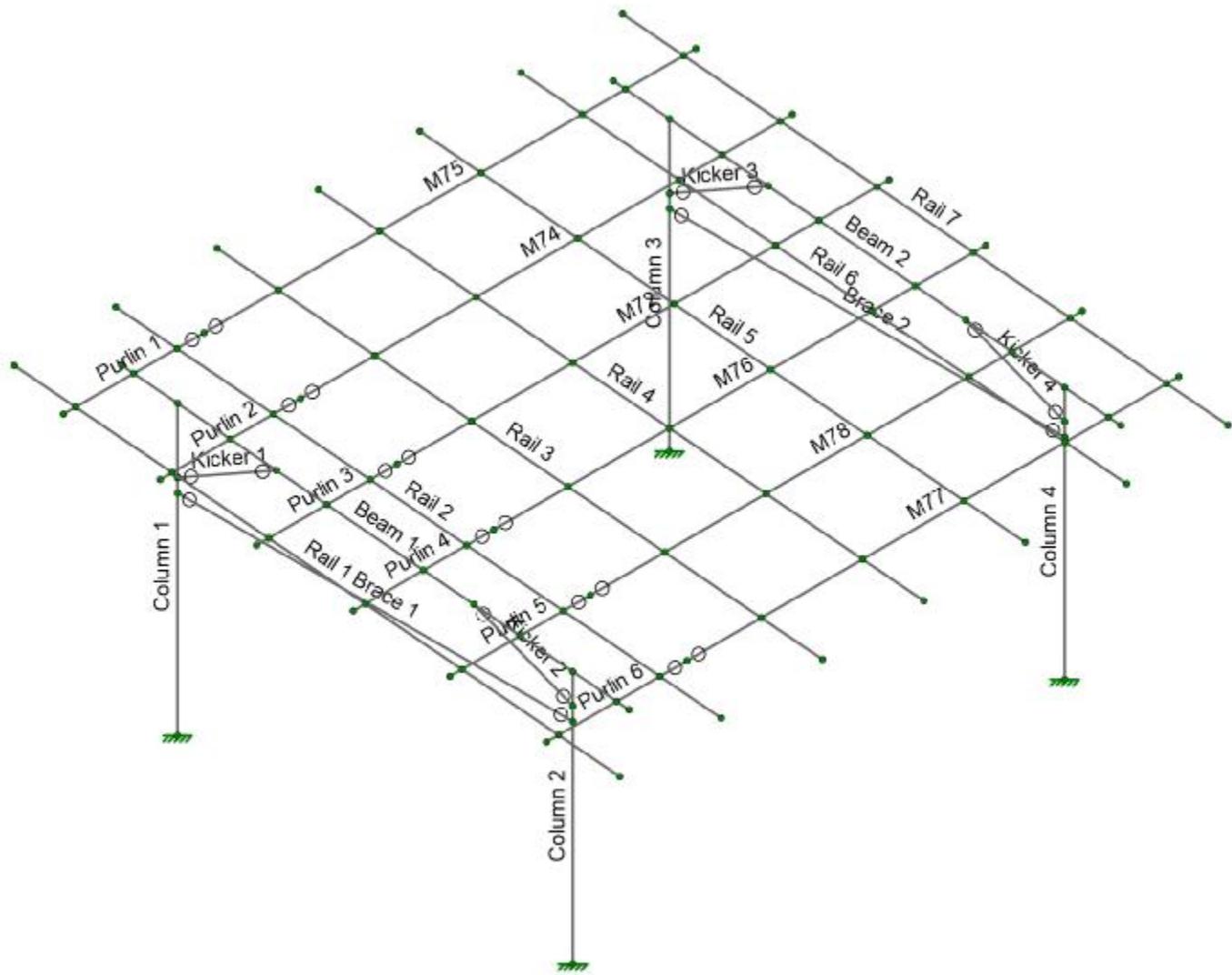
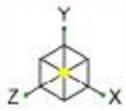
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Chiko Premier Carport

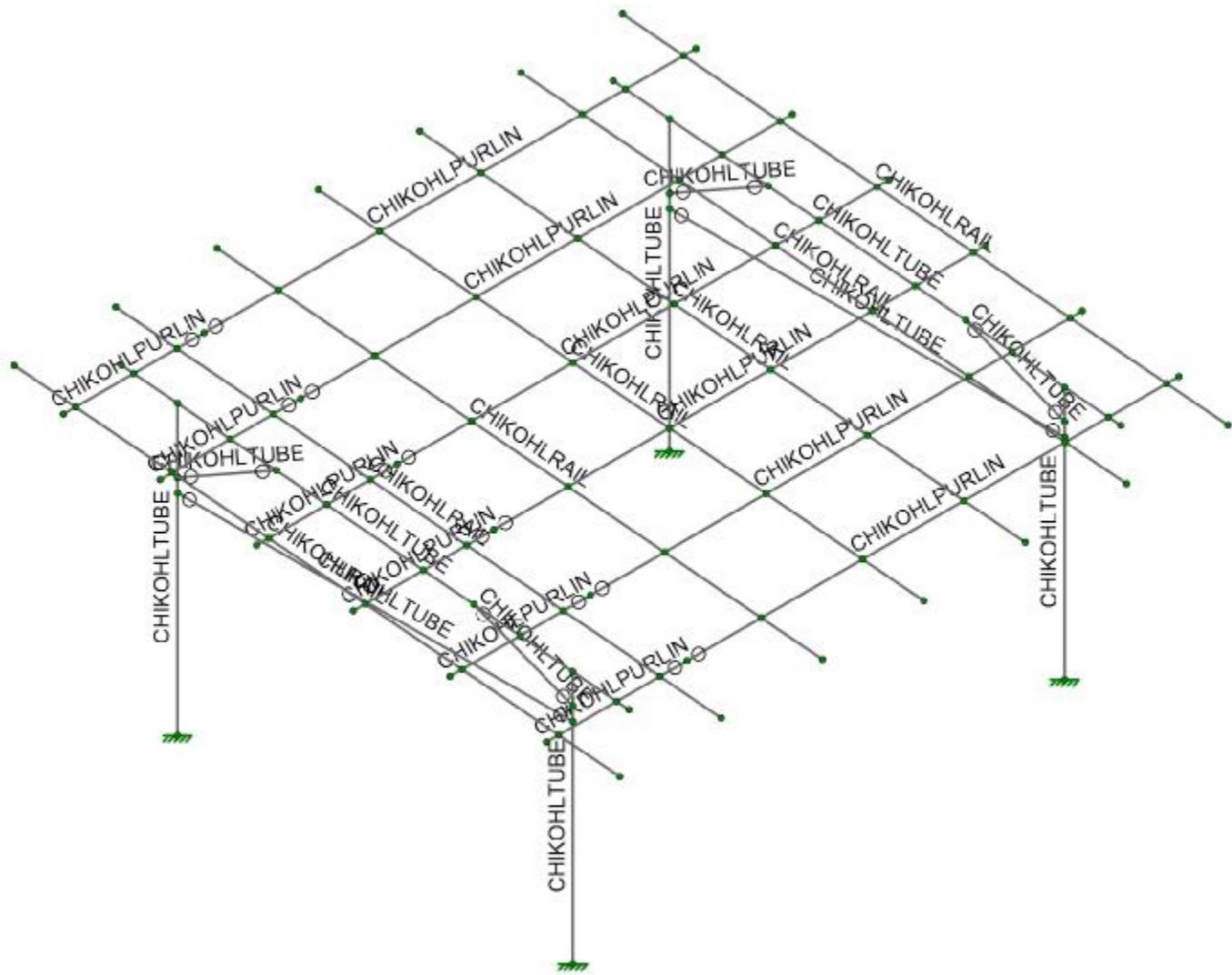
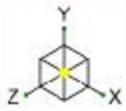
SK-1
Dec 15, 2023
Chiko High Load Carport - 2 Cars - 110 mph, 1...



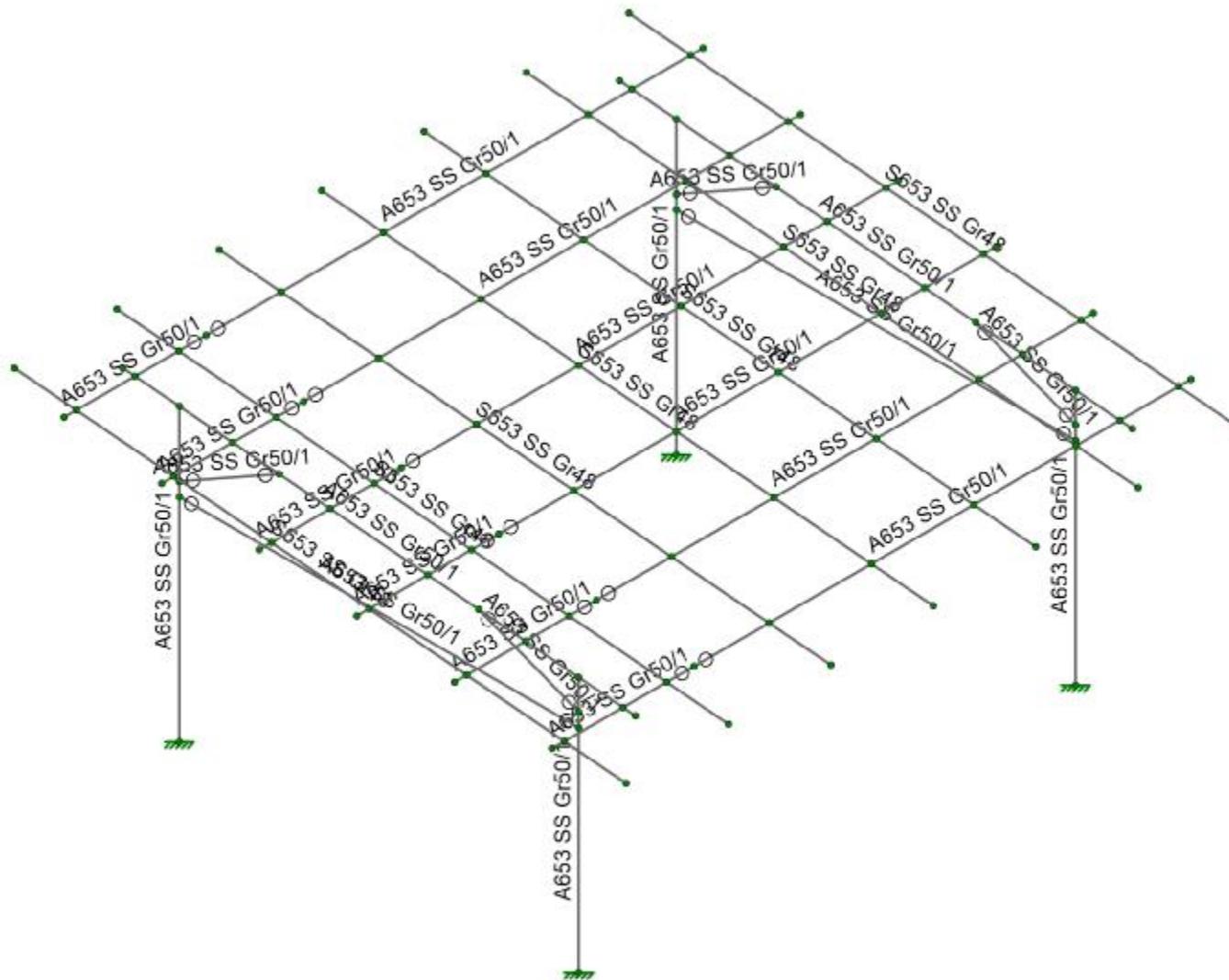
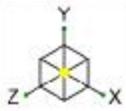
EPS	Chiko Premier Carport	SK-2
RK		Dec 15, 2023
23-CHK004		Chiko High Load Carport - 2 Cars - 110 mph, 1...



EPS	Chiko Premier Carport	SK-3
RK		Dec 15, 2023
23-CHK004		Chiko High Load Carport - 2 Cars - 110 mph, 1...



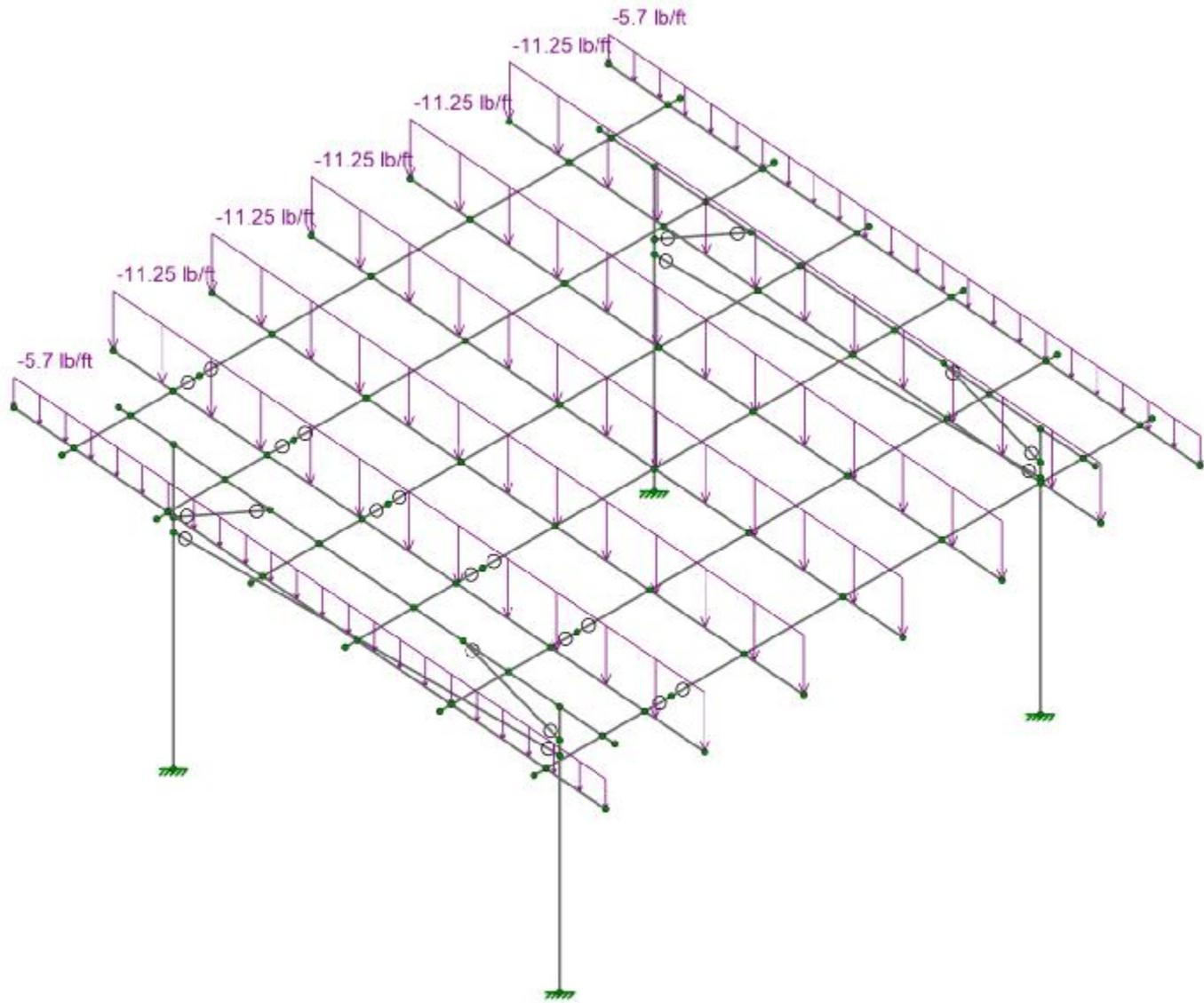
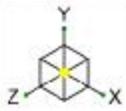
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23-CHK004		Chiko High Load Carport - 2 Cars - 110 mph, 1...



EPS
 RK
 23-CHK004

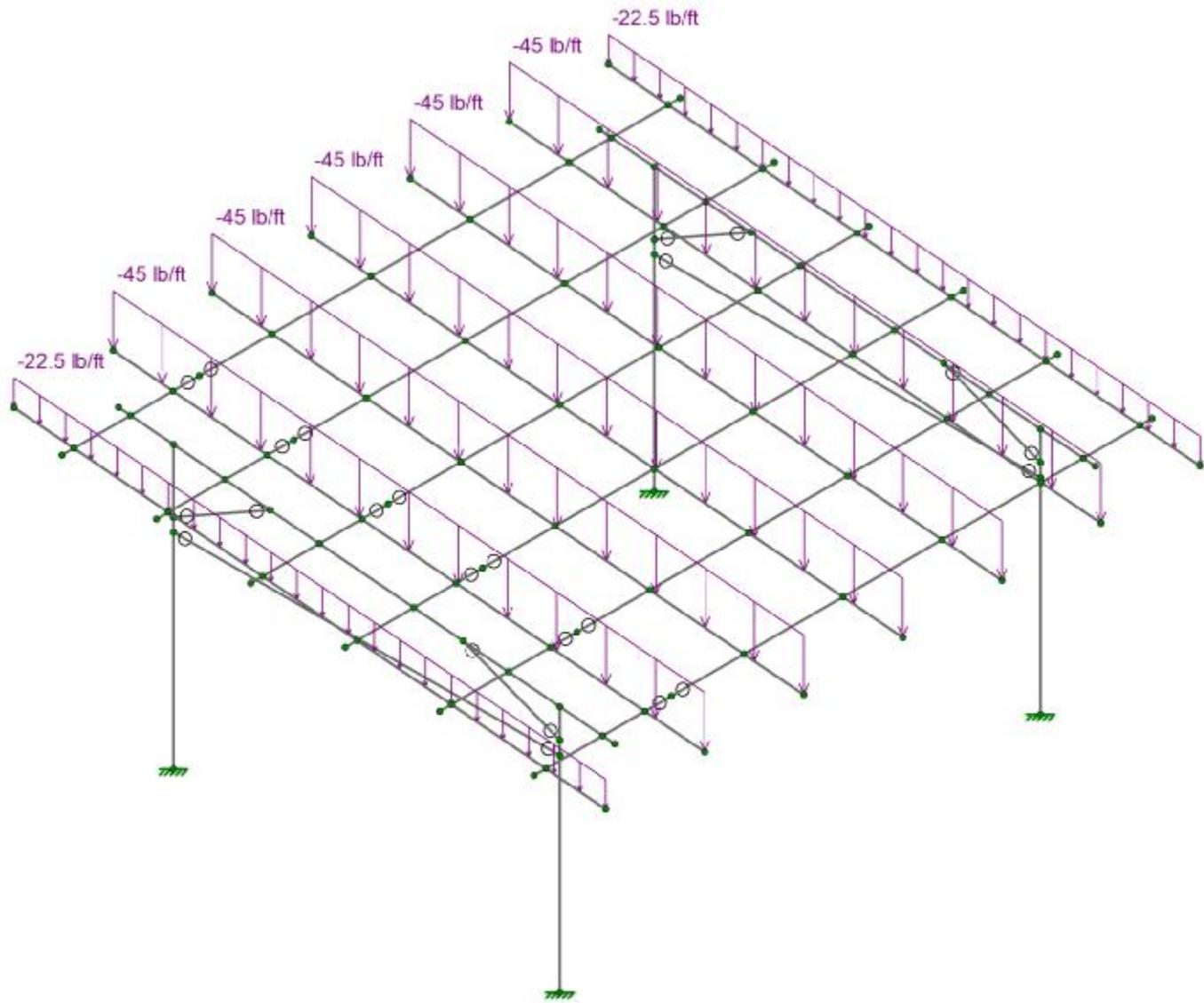
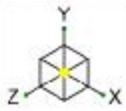
Chiko Premier Carport

SK-5
 Dec 15, 2023
 Chiko High Load Carport - 2 Cars - 110 mph, 1...



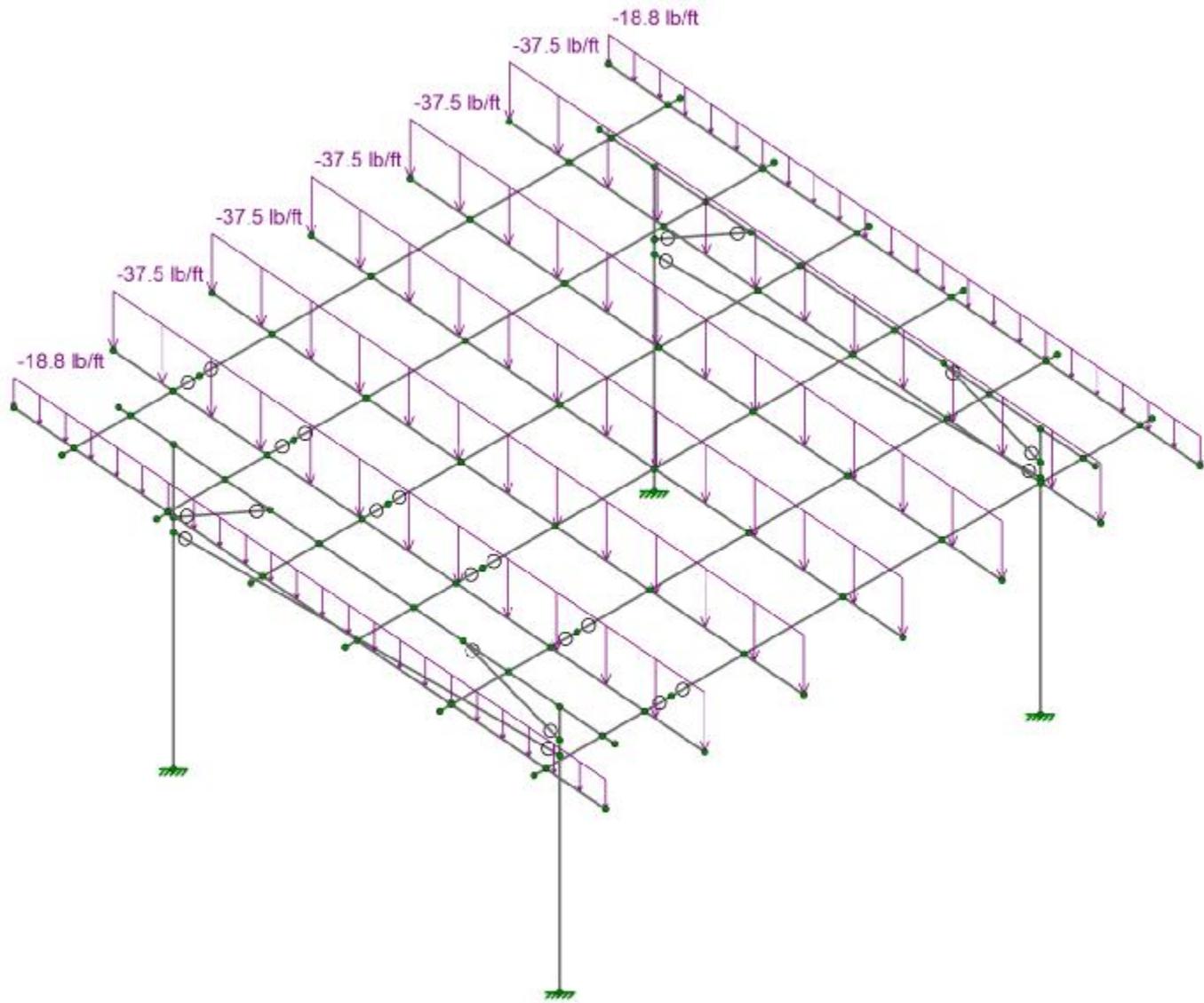
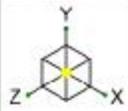
Loads: BLC 1, Dead

EPS	Chiko Premier Carport	SK-6
RK		Dec 15, 2023
23-CHK004		Chiko High Load Carport - 2 Cars - 110 mph, 1...



Loads: BLC 2, Roof Live

EPS	Chiko Premier Carport	SK-7
RK		Dec 15, 2023
23-CHK004		Chiko High Load Carport - 2 Cars - 110 mph, 1...

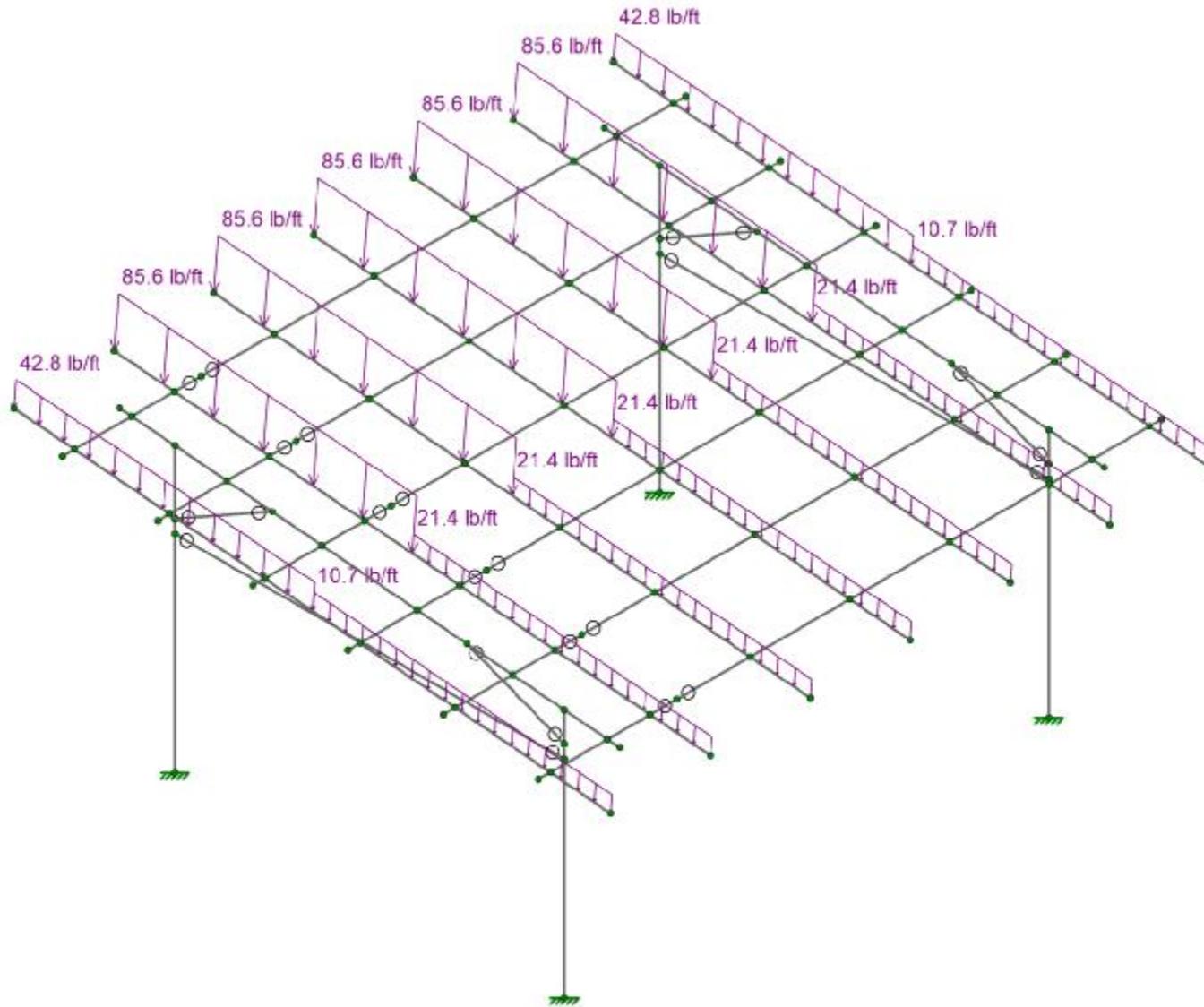
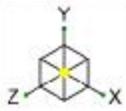


Loads: BLC 3, Snow

EPS
RK
23-CHK004

Chiko Premier Carport

SK-8
Dec 15, 2023
Chiko High Load Carport - 2 Cars - 110 mph, 1...

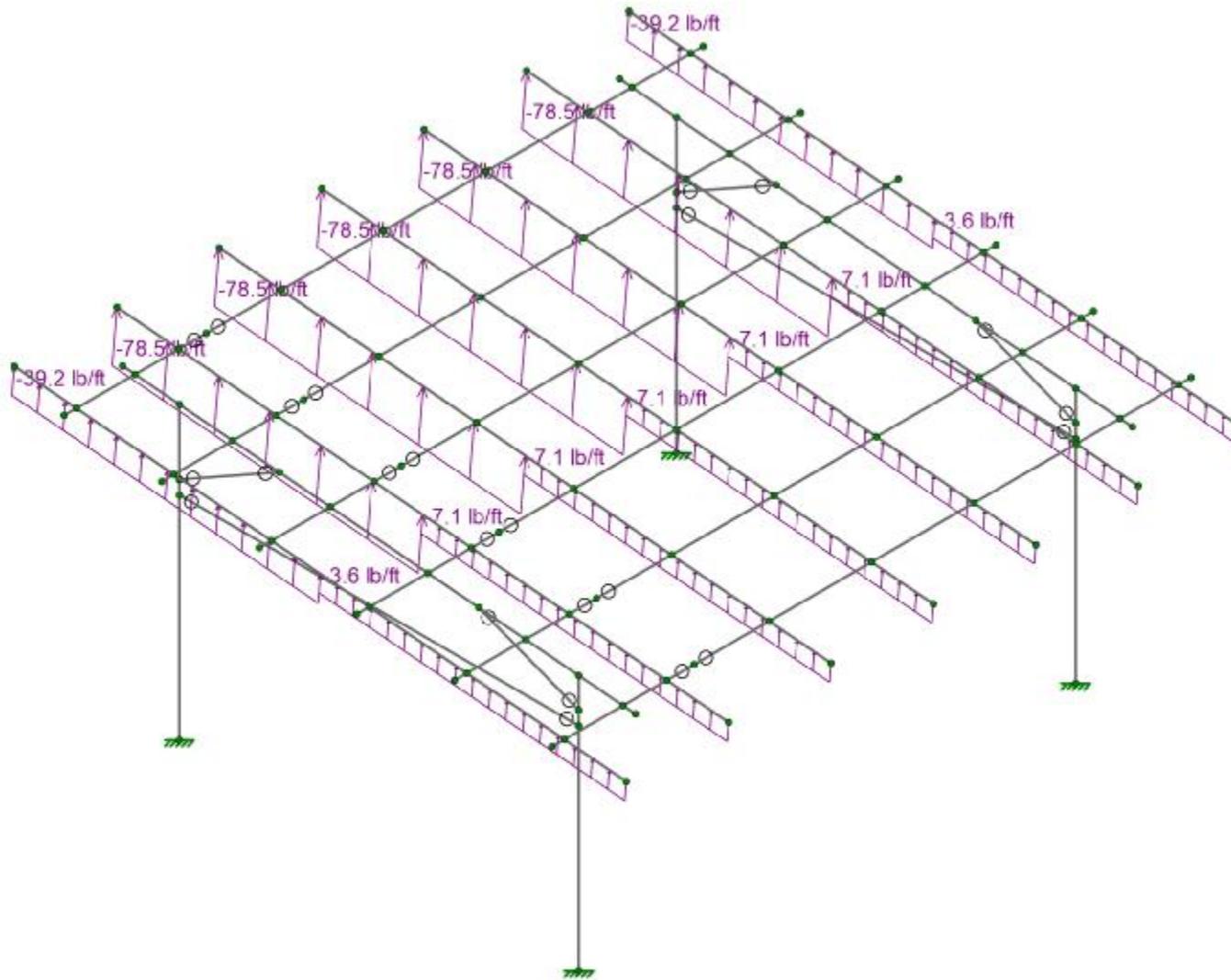
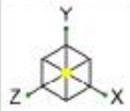


Loads: BLC 4, Wind - 0 - A

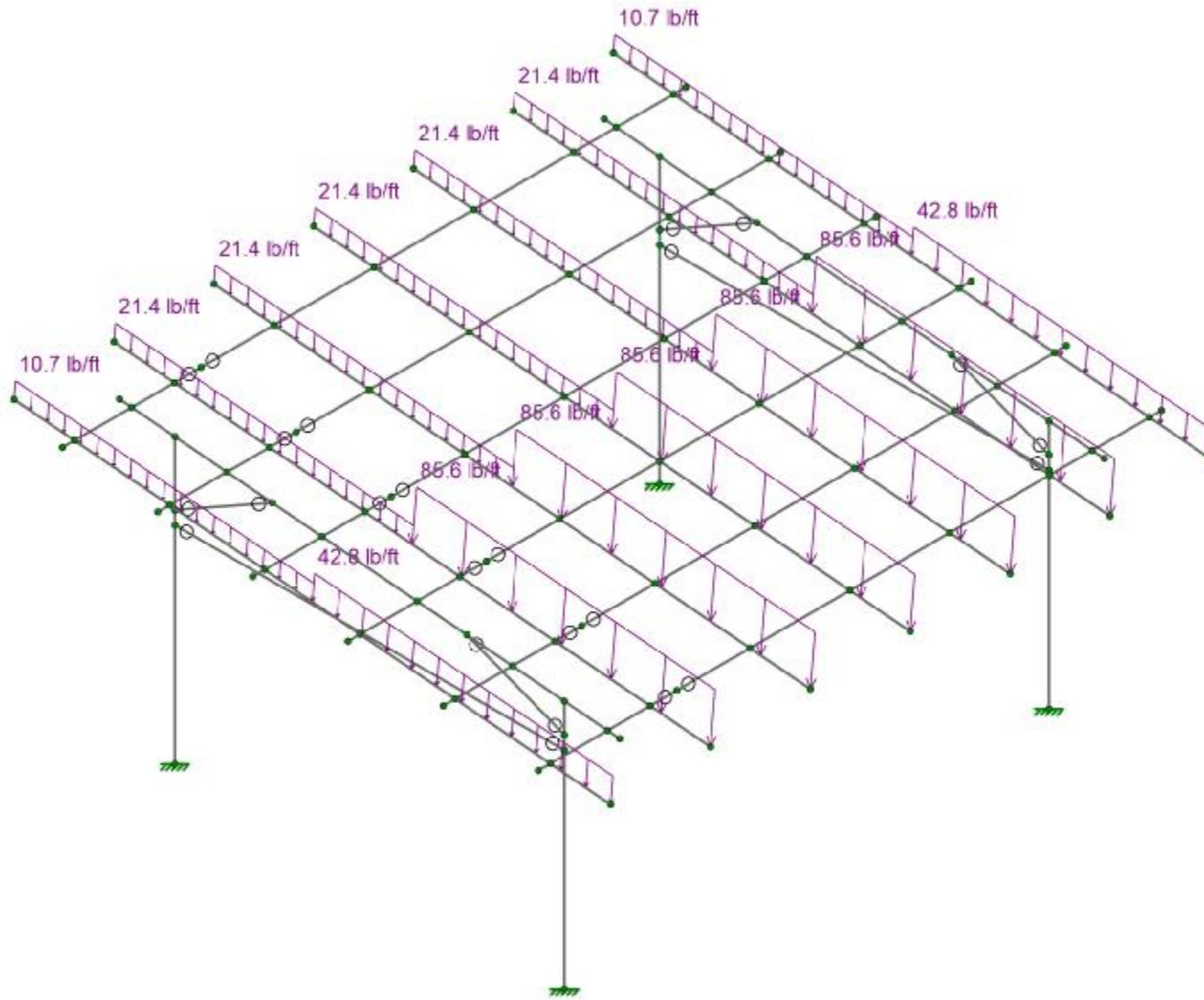
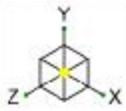
EPS
 RK
 23-CHK004

Chiko Premier Carport

SK-9
 Dec 15, 2023
 Chiko High Load Carport - 2 Cars - 110 mph, 1...

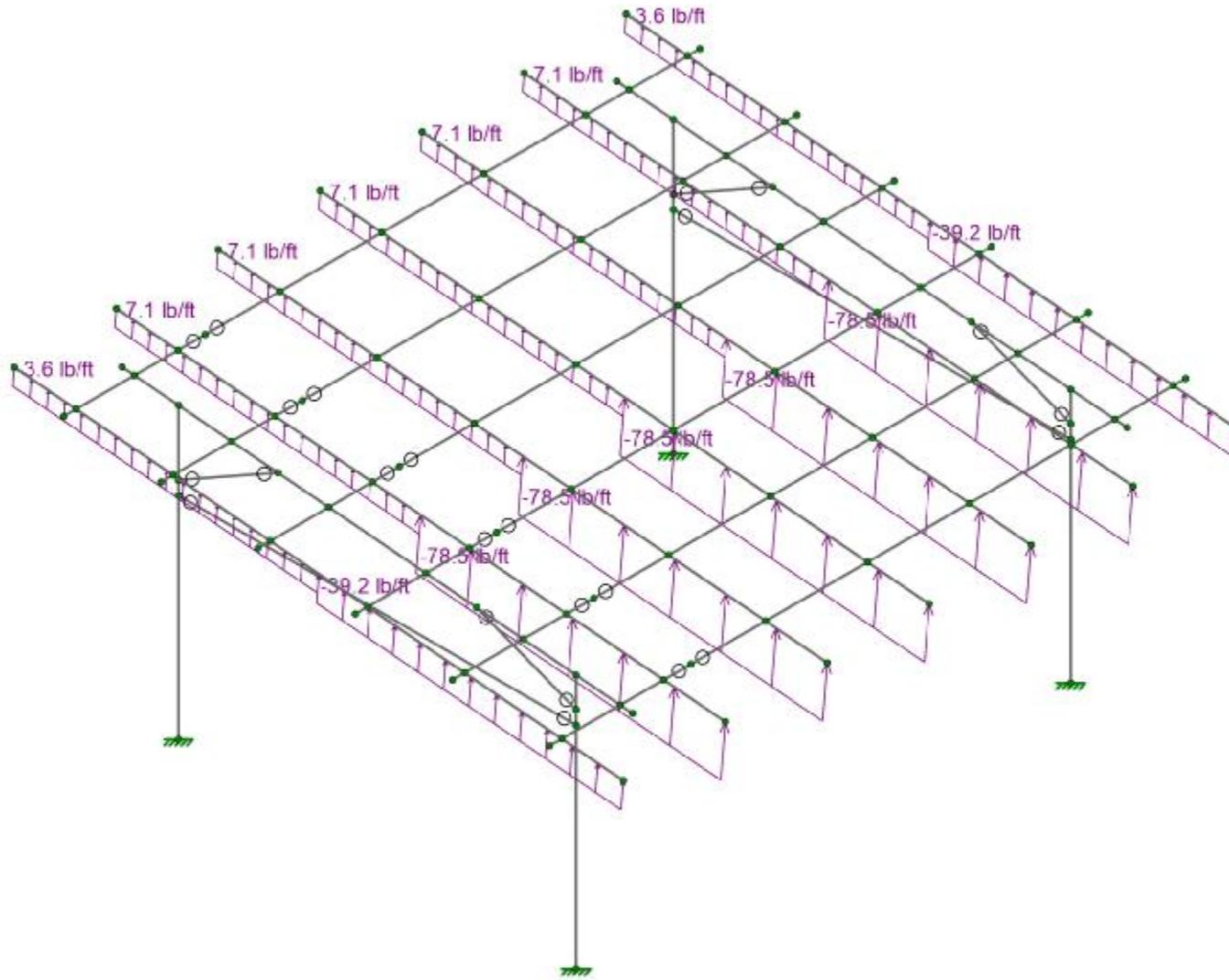
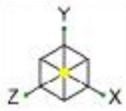


Loads: BLC 5, Wind - 0 - B		
EPS	Chiko Premier Carport	SK-10
RK		Dec 15, 2023
23-CHK004		Chiko High Load Carport - 2 Cars - 110 mph, 1...



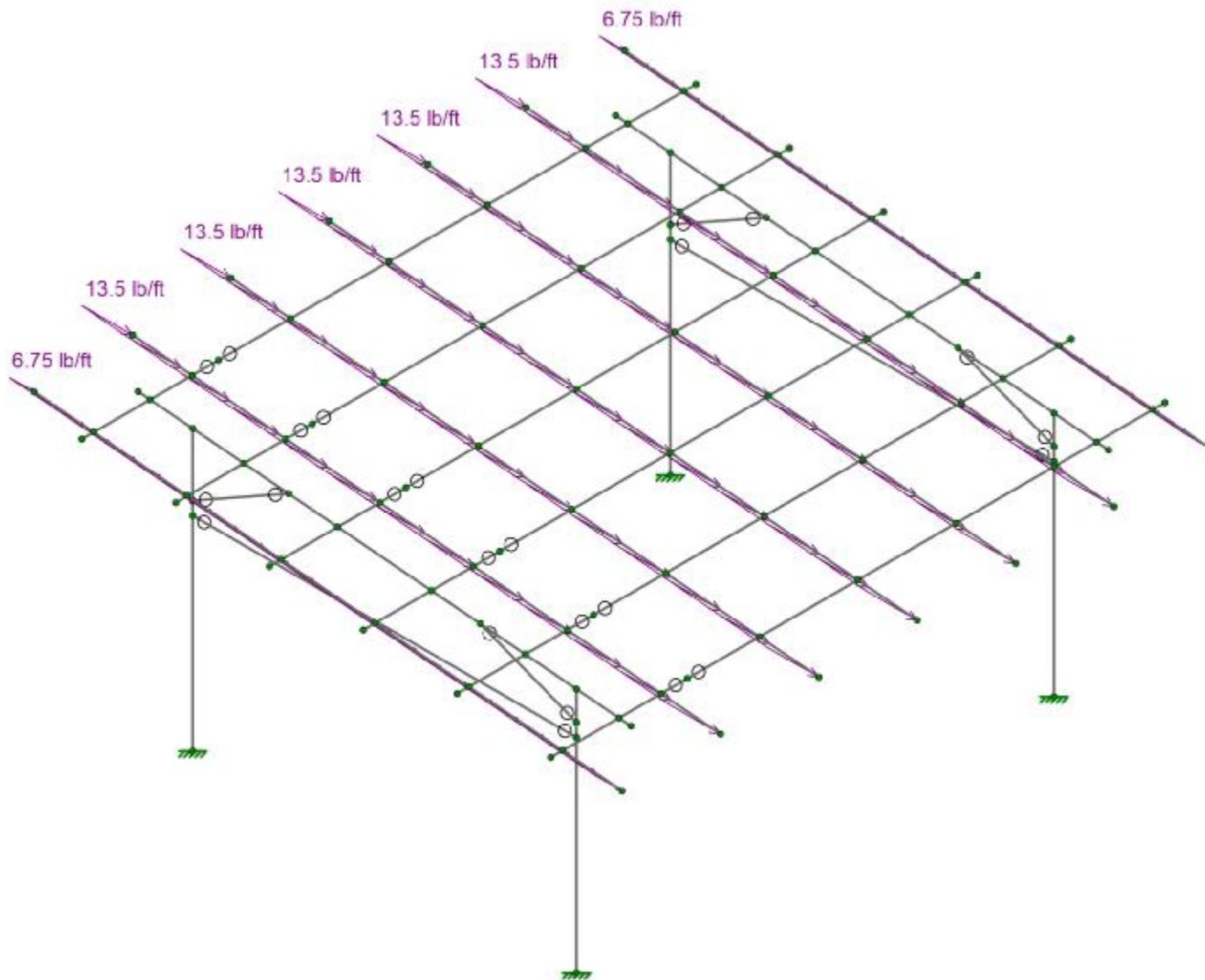
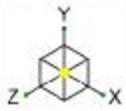
Loads: BLC 6, Wind - 180 - A

EPS	Chiko Premier Carport	SK-11
RK		Dec 15, 2023
23-CHK004		Chiko High Load Carport - 2 Cars - 110 mph, 1...



Loads: BLC 7, Wind - 180 - B

EPS	Chiko Premier Carport	SK-12
RK		Dec 15, 2023
23-CHK004		Chiko High Load Carport - 2 Cars - 110 mph, 1...

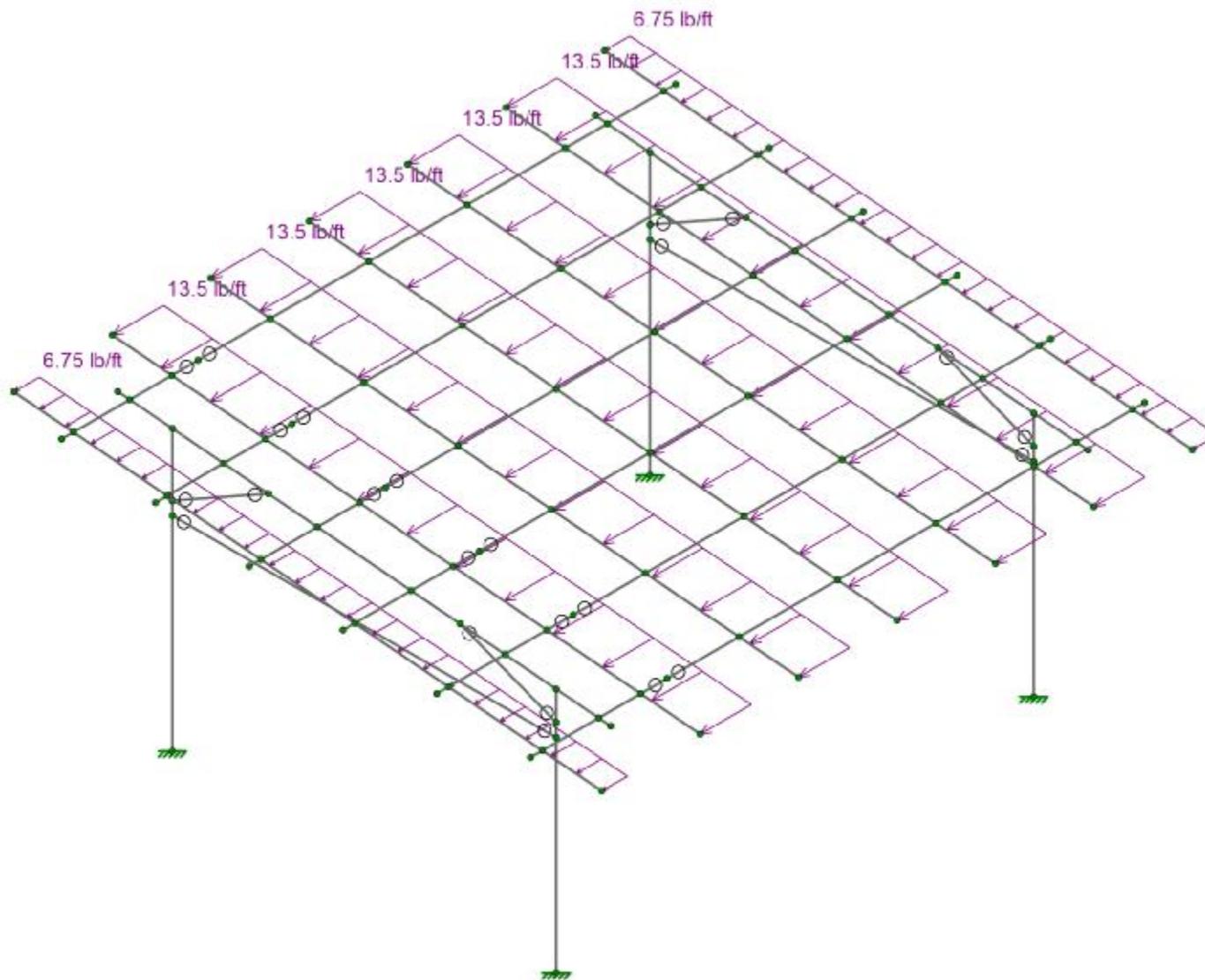
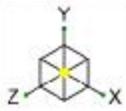


Loads: BLC 8, Seismic X

EPS
RK
23-CHK004

Chiko Premier Carport

SK-13
Dec 15, 2023
Chiko High Load Carport - 2 Cars - 110 mph, 1...

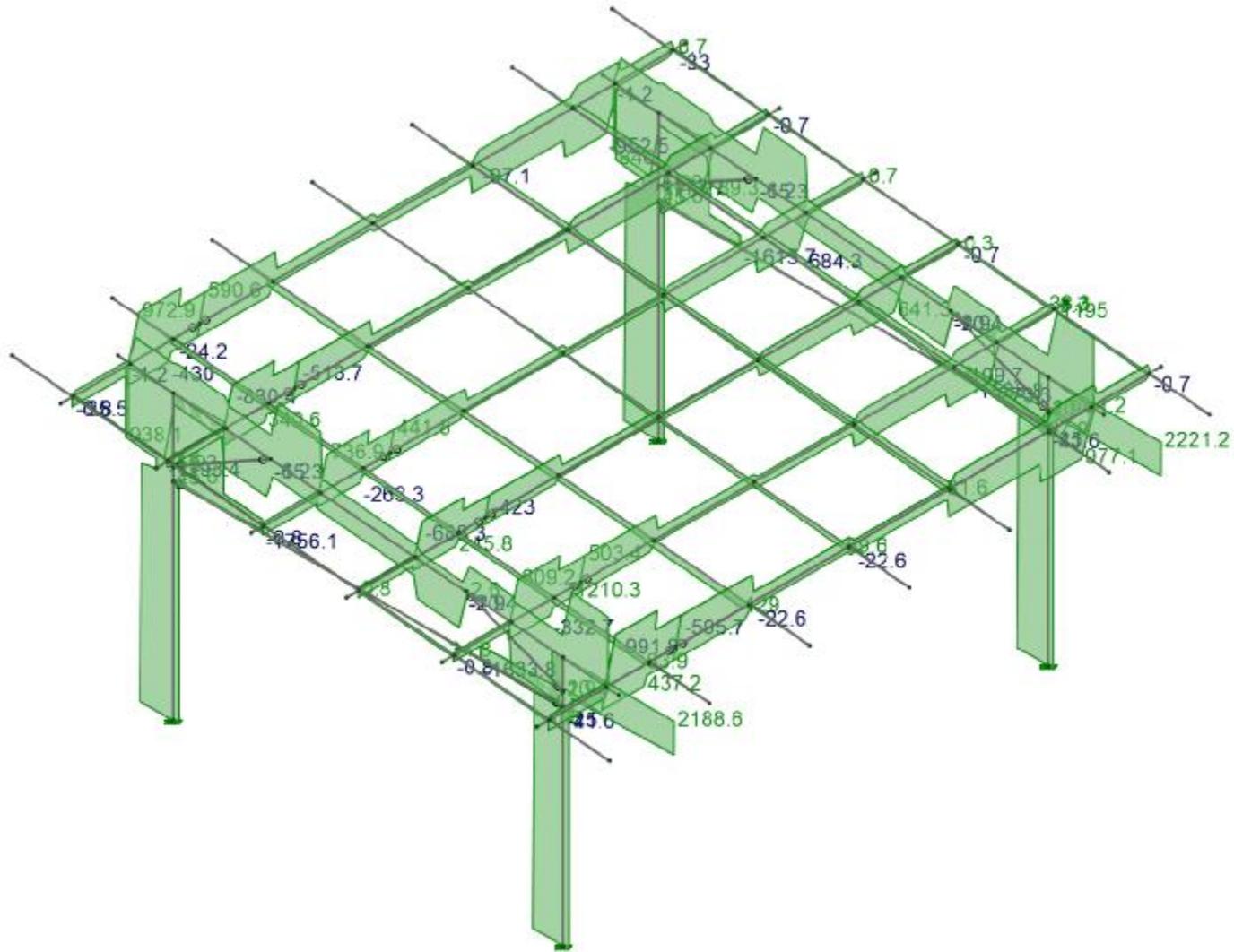
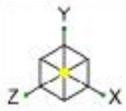


Loads: BLC 9, Seismic Z

EPS
 RK
 23-CHK004

Chiko Premier Carport

SK-14
 Dec 15, 2023
 Chiko High Load Carport - 2 Cars - 110 mph, 1...

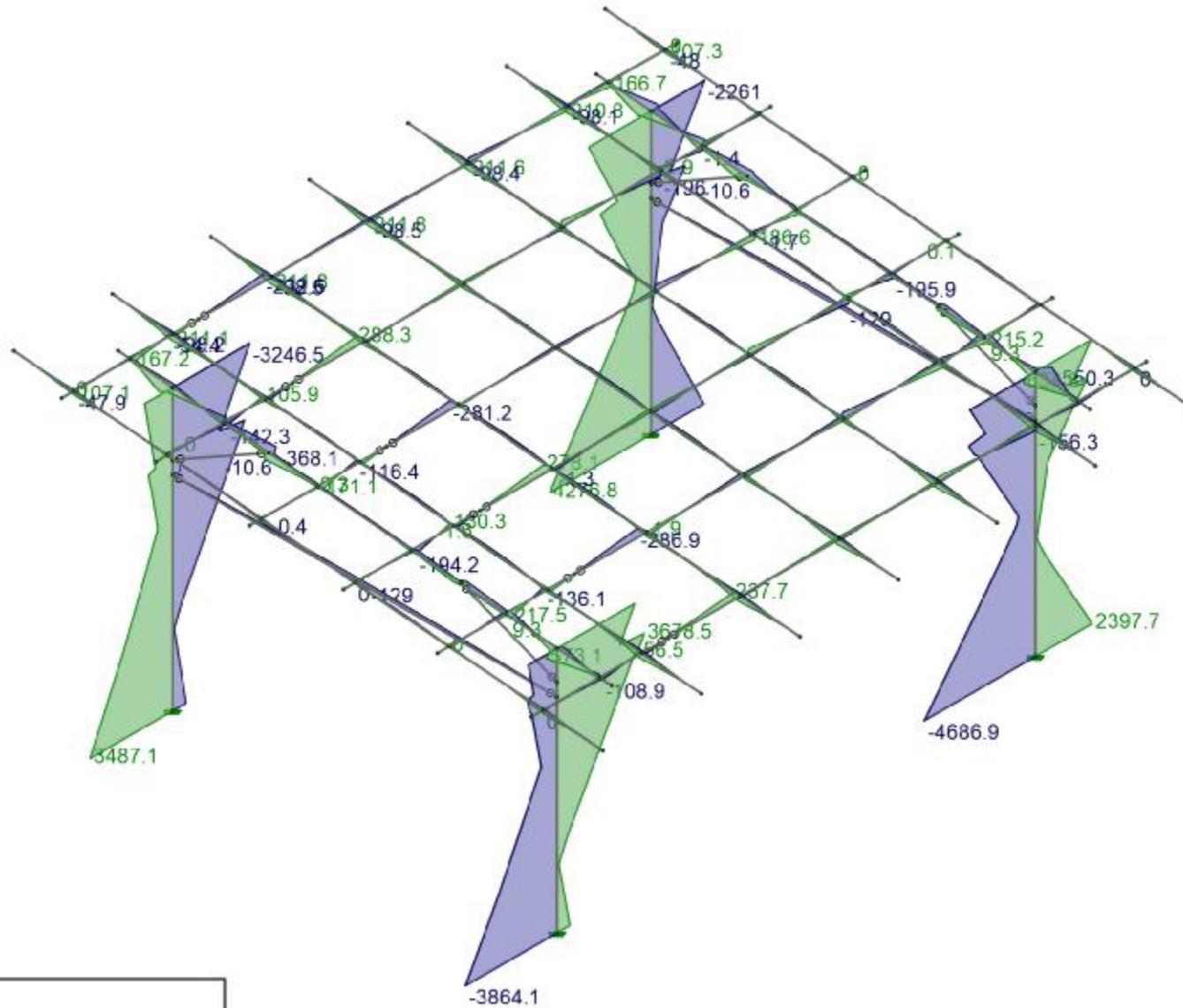
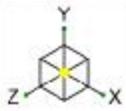


Envelope Only Solution
Member y Shear Forces (lbs) (Enveloped)

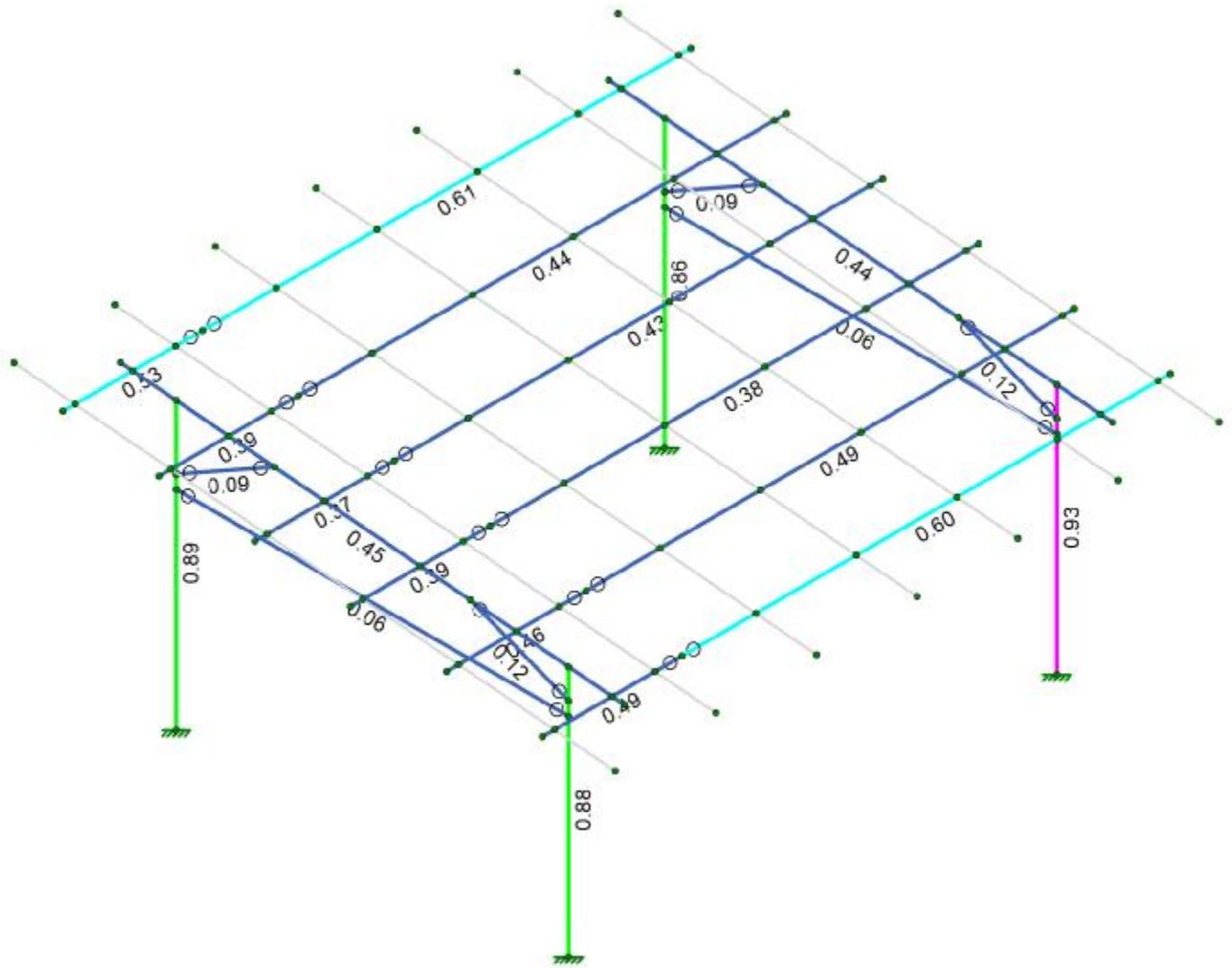
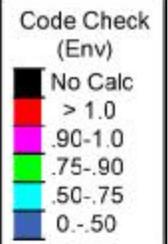
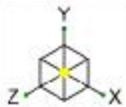
EPS
RK
23-CHK004

Chiko Premier Carport

SK-17
Dec 15, 2023
Chiko High Load Carport - 2 Cars - 110 mph, 1...

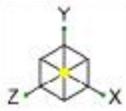


Envelope Only Solution Member y Bending Moments (lb-ft) (Enveloped)		
EPS	Chiko Premier Carport	SK-19
RK		Dec 15, 2023
23-CHK004		Chiko High Load Carport - 2 Cars - 110 mph, 1...



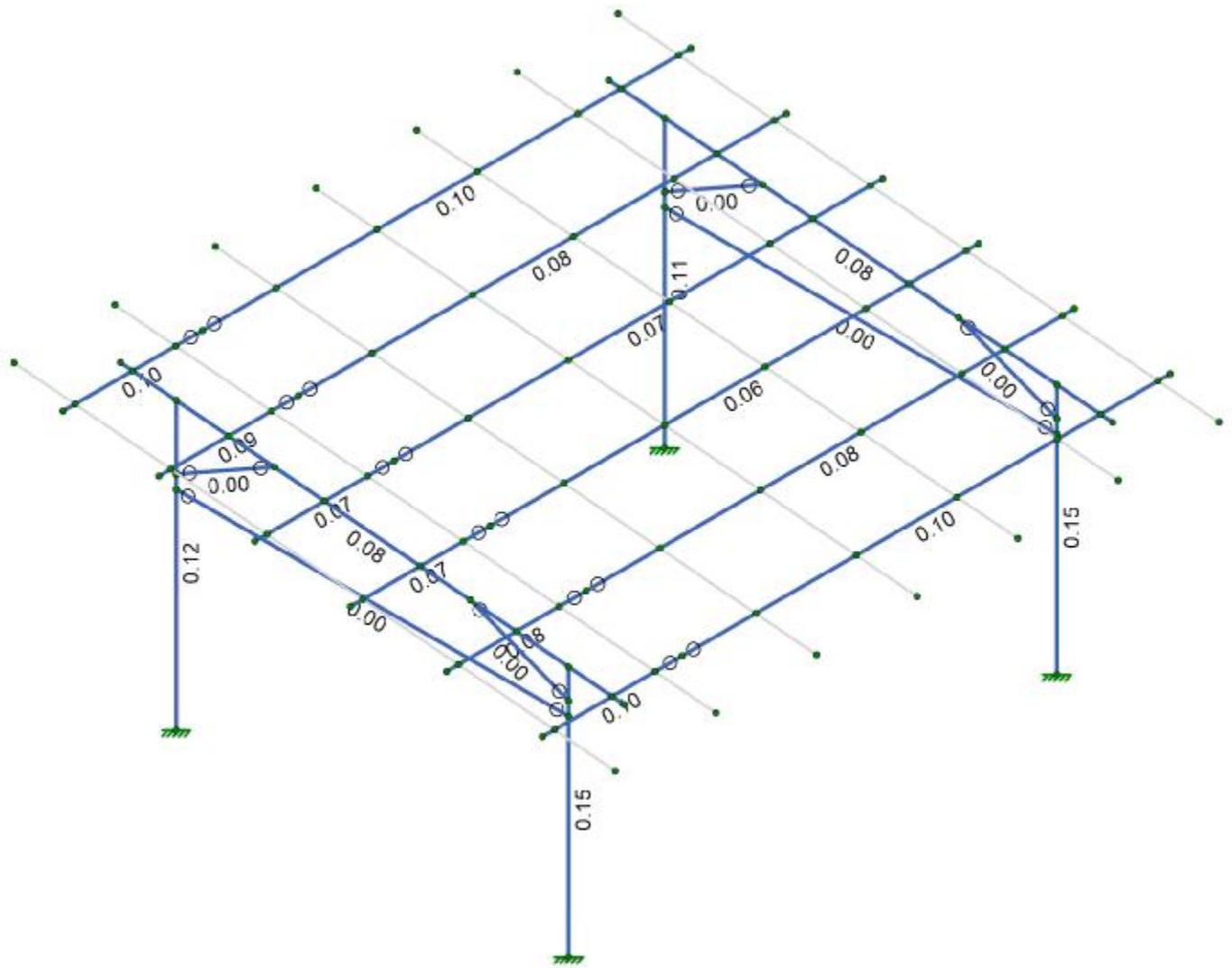
Member Code Checks Displayed (Enveloped)
Envelope Only Solution

EPS	Chiko Premier Carport	SK-21
RK		Dec 15, 2023
23-CHK004		Chiko High Load Carport - 2 Cars - 110 mph, 1...



Shear Check (Env)

- No Calc
- > 1.0
- .90-1.0
- .75-.90
- .50-.75
- 0-.50



Member Shear Checks Displayed (Enveloped)
Envelope Only Solution

EPS	Chiko Premier Carport	SK-22
RK		Dec 15, 2023
23-CHK004		Chiko High Load Carport - 2 Cars - 110 mph, 1...

Cold Formed Steel Properties

	Label	E [ksj]	G [ksj]	Nu	Therm. Coeff. [1e ⁵ F ⁻¹]	Density [k/ft ³]	Yield [ksj]	Fu [ksj]
1	A653 SS Gr34	29500	11346	0.3	0.65	0.49	34	45
2	A653 SS Gr33	29500	11346	0.3	0.65	0.49	33	45
3	S653 SS Gr48	29500	11346	0.3	0.65	0.49	48	65
4	A653 SS Gr50/1	29500	11346	0.3	0.65	0.49	50	65

Cold Formed Steel Design Parameters

	Label	Shape	Length [in]	Lb y-y [in]	Lb z-z [in]	Lcomp top [in]	Lcomp bot [in]	Function
1	Rail 2	CHIKOHLRAIL	272.442	Segment	Segment	Segment	Segment	Lateral
2	Column 4	CHIKOHLTUBE	113.386			Segment	Segment	Lateral
3	Column 3	CHIKOHLTUBE	128.886			Segment	Segment	Lateral
4	Column 2	CHIKOHLTUBE	113.386			Segment	Segment	Lateral
5	Purlin 3	CHIKOHLPURLIN	62.992	32	32	45.4	32	Lateral
6	Beam 2	CHIKOHLTUBE	228.346	178	178	44	90	Lateral
7	Purlin 2	CHIKOHLPURLIN	62.992	32	32	32	45.4	Lateral
8	Beam 1	CHIKOHLTUBE	228.346	178	178	44	90	Lateral
9	Purlin 1	CHIKOHLPURLIN	62.992	32	32	45.4	32	Lateral
10	Rail 1	CHIKOHLRAIL	272.442	Segment	Segment	Segment	Segment	Lateral
11	Purlin 4	CHIKOHLPURLIN	62.992	32	32	32	45.4	Lateral
12	Purlin 6	CHIKOHLPURLIN	62.992	32	32	32	45.4	Lateral
13	Rail 4	CHIKOHLRAIL	272.442	Segment	Segment	Segment	Segment	Lateral
14	Rail 5	CHIKOHLRAIL	272.442	Segment	Segment	Segment	Segment	Lateral
15	Rail 6	CHIKOHLRAIL	272.442	Segment	Segment	Segment	Segment	Lateral
16	Rail 7	CHIKOHLRAIL	272.442	Segment	Segment	Segment	Segment	Lateral
17	Column 1	CHIKOHLTUBE	128.886			Segment	Segment	Lateral
18	Purlin 5	CHIKOHLPURLIN	62.992	32	32	45.4	32	Lateral
19	Rail 3	CHIKOHLRAIL	272.442	Segment	Segment	Segment	Segment	Lateral
20	M62	CHIKOHLTUBE	177.165			Lbyy		Lateral
21	M63	CHIKOHLTUBE	177.165			Lbyy		Lateral
22	M65	CHIKOHLTUBE	50.82			Lbyy		Lateral
23	M66	CHIKOHLTUBE	47.503			Lbyy		Lateral
24	M68	CHIKOHLTUBE	50.82			Lbyy		Lateral
25	M70	CHIKOHLTUBE	47.503			Lbyy		Lateral
26	M72	CHIKOHLPURLIN	220.5	190	190	45.4	75	Lateral
27	M74	CHIKOHLPURLIN	220.5	190	190	75	45.4	Lateral
28	M75	CHIKOHLPURLIN	220.5	190	190	45.4	75	Lateral
29	M76	CHIKOHLPURLIN	220.5	190	190	75	45.4	Lateral
30	M77	CHIKOHLPURLIN	220.5	190	190	75	45.4	Lateral
31	M78	CHIKOHLPURLIN	220.5	190	190	45.4	75	Lateral

Basic Load Cases

	BLC Description	Category	X Gravity	Y Gravity	Z Gravity	Distributed
1	Dead	DL		-1		7
2	Roof Live	RLL				7
3	Snow	SL				7
4	Wind - 0 - A	WL				14
5	Wind - 0 - B	WL				14
6	Wind - 180 - A	WL				14
7	Wind - 180 - B	WL				14
8	Seismic X	ELX	1.2			7
9	Seismic Z	ELZ			1.2	7

Load Combinations

	Description	Solve	P-Delta	BLC	Factor										
1	ASCE ASD 1	Yes	Y	DL	1										
2	ASCE ASD 2	Yes	Y	DL	1	LL	1	LLS	1						
3	ASCE ASD 3 (a)	Yes	Y	DL	1	RLL	1								
4	ASCE ASD 3 (b)	Yes	Y	DL	1	SL	1	SLN	1						
5	ASCE ASD 4 (a)	Yes	Y	DL	1	LL	0.75	LLS	0.75	RLL	0.75				
6	ASCE ASD 4 (b)	Yes	Y	DL	1	LL	0.75	LLS	0.75	SL	0.75	SLN	0.75		
7	ASCE ASD 5 (a)	Yes	Y	DL	1	4	0.6								
8	ASCE ASD 5 (a)	Yes	Y	DL	1	5	0.6								
9	ASCE ASD 5 (a)	Yes	Y	DL	1	6	0.6								
10	ASCE ASD 5 (a)	Yes	Y	DL	1	7	0.6								
11	ASCE ASD 6 (a)	Yes	Y	DL	1	4	0.45	LL	0.75	LLS	0.75	RLL	0.75		
12	ASCE ASD 6 (a)	Yes	Y	DL	1	5	0.45	LL	0.75	LLS	0.75	RLL	0.75		
13	ASCE ASD 6 (a)	Yes	Y	DL	1	6	0.45	LL	0.75	LLS	0.75	RLL	0.75		
14	ASCE ASD 6 (a)	Yes	Y	DL	1	7	0.45	LL	0.75	LLS	0.75	RLL	0.75		
15	ASCE ASD 6 (b)	Yes	Y	DL	1	4	0.45	LL	0.75	LLS	0.75	SL	0.75	SLN	0.75
16	ASCE ASD 6 (b)	Yes	Y	DL	1	5	0.45	LL	0.75	LLS	0.75	SL	0.75	SLN	0.75
17	ASCE ASD 6 (b)	Yes	Y	DL	1	6	0.45	LL	0.75	LLS	0.75	SL	0.75	SLN	0.75
18	ASCE ASD 6 (b)	Yes	Y	DL	1	7	0.45	LL	0.75	LLS	0.75	SL	0.75	SLN	0.75
19	ASCE ASD 6 (c)	Yes	Y	DL	1	4	0.45	LL	0.75	LLS	0.75				
20	ASCE ASD 6 (c)	Yes	Y	DL	1	5	0.45	LL	0.75	LLS	0.75				
21	ASCE ASD 6 (c)	Yes	Y	DL	1	6	0.45	LL	0.75	LLS	0.75				
22	ASCE ASD 6 (c)	Yes	Y	DL	1	7	0.45	LL	0.75	LLS	0.75				
23	ASCE ASD 7	Yes	Y	DL	0.6	4	0.6								
24	ASCE ASD 7	Yes	Y	DL	0.6	5	0.6								
25	ASCE ASD 7	Yes	Y	DL	0.6	6	0.6								
26	ASCE ASD 7	Yes	Y	DL	0.6	7	0.6								
27	ASCE ASD 8 (a)	Yes	Y	DL	1	ELX	0.7								
28	ASCE ASD 8 (b)	Yes	Y	DL	1	ELZ	0.7								
29	ASCE ASD 9 (a)	Yes	Y	DL	1	ELX	0.525	LL	0.75	LLS	0.75	SL	0.75	SLN	0.75
30	ASCE ASD 9 (b)	Yes	Y	DL	1	ELZ	0.525	LL	0.75	LLS	0.75	SL	0.75	SLN	0.75
31	ASCE ASD 10 (a)	Yes	Y	DL	0.6	ELX	0.7								
32	ASCE ASD 10 (b)	Yes	Y	DL	0.6	ELZ	0.7								

Member Distributed Loads (BLC 1 : Dead)

	Member Label	Direction	Start Magnitude [lb/ft, F, psf, lb-ft/in]	End Magnitude [lb/ft, F, psf, lb-ft/in]	Start Location [(in, %)]	End Location [(in, %)]
1	Rail 2	Y	-11.25	-11.25	0	%100
2	Rail 1	Y	-5.7	-5.7	0	%100
3	Rail 4	Y	-11.25	-11.25	0	%100
4	Rail 5	Y	-11.25	-11.25	0	%100
5	Rail 6	Y	-11.25	-11.25	0	%100
6	Rail 7	Y	-5.7	-5.7	0	%100
7	Rail 3	Y	-11.25	-11.25	0	%100

Member Distributed Loads (BLC 2 : Roof Live)

	Member Label	Direction	Start Magnitude [lb/ft, F, psf, lb-ft/in]	End Magnitude [lb/ft, F, psf, lb-ft/in]	Start Location [(in, %)]	End Location [(in, %)]
1	Rail 2	Y	-45	-45	0	%100
2	Rail 1	Y	-22.5	-22.5	0	%100
3	Rail 4	Y	-45	-45	0	%100
4	Rail 5	Y	-45	-45	0	%100
5	Rail 6	Y	-45	-45	0	%100
6	Rail 7	Y	-22.5	-22.5	0	%100
7	Rail 3	Y	-45	-45	0	%100

Member Distributed Loads (BLC 3 : Snow)

Member Label	Direction	Start Magnitude [lb/ft, F, psf, lb-ft/in]	End Magnitude [lb/ft, F, psf, lb-ft/in]	Start Location [(in, %)]	End Location [(in, %)]
1	Rail 2	Y	-37.5	-37.5	0 %100
2	Rail 3	Y	-37.5	-37.5	0 %100
3	Rail 4	Y	-37.5	-37.5	0 %100
4	Rail 5	Y	-37.5	-37.5	0 %100
5	Rail 6	Y	-37.5	-37.5	0 %100
6	Rail 1	Y	-18.8	-18.8	0 %100
7	Rail 7	Y	-18.8	-18.8	0 %100

Member Distributed Loads (BLC 4 : Wind - 0 - A)

Member Label	Direction	Start Magnitude [lb/ft, F, psf, lb-ft/in]	End Magnitude [lb/ft, F, psf, lb-ft/in]	Start Location [(in, %)]	End Location [(in, %)]
1	Rail 2	z	85.6	85.6	0 %50
2	Rail 2	z	21.4	21.4	%50 %100
3	Rail 1	z	42.8	42.8	0 %50
4	Rail 1	z	10.7	10.7	%50 %100
5	Rail 4	z	21.4	21.4	%50 %100
6	Rail 4	z	85.6	85.6	0 %50
7	Rail 5	z	85.6	85.6	0 %50
8	Rail 5	z	21.4	21.4	%50 %100
9	Rail 6	z	85.6	85.6	0 %50
10	Rail 6	z	21.4	21.4	%50 %100
11	Rail 7	z	10.7	10.7	%50 %100
12	Rail 7	z	42.8	42.8	0 %50
13	Rail 3	z	21.4	21.4	%50 %100
14	Rail 3	z	85.6	85.6	0 %50

Member Distributed Loads (BLC 5 : Wind - 0 - B)

Member Label	Direction	Start Magnitude [lb/ft, F, psf, lb-ft/in]	End Magnitude [lb/ft, F, psf, lb-ft/in]	Start Location [(in, %)]	End Location [(in, %)]
1	Rail 2	z	-78.5	-78.5	0 %50
2	Rail 2	z	-7.1	-7.1	%50 %100
3	Rail 1	z	-39.2	-39.2	0 %50
4	Rail 1	z	-3.6	-3.6	%50 %100
5	Rail 4	z	-7.1	-7.1	%50 %100
6	Rail 4	z	-78.5	-78.5	0 %50
7	Rail 5	z	-78.5	-78.5	0 %50
8	Rail 5	z	-7.1	-7.1	%50 %100
9	Rail 6	z	-7.1	-7.1	%50 %100
10	Rail 6	z	-78.5	-78.5	0 %50
11	Rail 7	z	-39.2	-39.2	0 %50
12	Rail 7	z	-3.6	-3.6	%50 %100
13	Rail 3	z	-7.1	-7.1	%50 %100
14	Rail 3	z	-78.5	-78.5	0 %50

Member Distributed Loads (BLC 6 : Wind - 180 - A)

Member Label	Direction	Start Magnitude [lb/ft, F, psf, lb-ft/in]	End Magnitude [lb/ft, F, psf, lb-ft/in]	Start Location [(in, %)]	End Location [(in, %)]
1	Rail 2	z	21.4	21.4	0 %50
2	Rail 2	z	85.6	85.6	%50 %100
3	Rail 1	z	42.8	42.8	%50 %100
4	Rail 1	z	10.7	10.7	0 %50
5	Rail 4	z	85.6	85.6	%50 %100
6	Rail 4	z	21.4	21.4	0 %50
7	Rail 5	z	85.6	85.6	%50 %100



Member Distributed Loads (BLC 6 : Wind - 180 - A) (Continued)

Member	Label	Direction	Start Magnitude [lb/ft, F, psf, lb-ft/in]	End Magnitude [lb/ft, F, psf, lb-ft/in]	Start Location [(in, %)]	End Location [(in, %)]
8	Rail 5	z	21.4	21.4	0	%50
9	Rail 6	z	21.4	21.4	0	%50
10	Rail 6	z	85.6	85.6	%50	%100
11	Rail 7	z	10.7	10.7	0	%50
12	Rail 7	z	42.8	42.8	%50	%100
13	Rail 3	z	21.4	21.4	0	%50
14	Rail 3	z	85.6	85.6	%50	%100

Member Distributed Loads (BLC 7 : Wind - 180 - B)

Member	Label	Direction	Start Magnitude [lb/ft, F, psf, lb-ft/in]	End Magnitude [lb/ft, F, psf, lb-ft/in]	Start Location [(in, %)]	End Location [(in, %)]
1	Rail 2	z	-78.5	-78.5	%50	%100
2	Rail 2	z	-7.1	-7.1	0	%50
3	Rail 1	z	-39.2	-39.2	%50	%100
4	Rail 1	z	-3.6	-3.6	0	%50
5	Rail 4	z	-78.5	-78.5	%50	%100
6	Rail 4	z	-7.1	-7.1	0	%50
7	Rail 5	z	-7.1	-7.1	0	%50
8	Rail 5	z	-78.5	-78.5	%50	%100
9	Rail 6	z	-78.5	-78.5	%50	%100
10	Rail 6	z	-7.1	-7.1	0	%50
11	Rail 7	z	-39.2	-39.2	%50	%100
12	Rail 7	z	-3.6	-3.6	0	%50
13	Rail 3	z	-78.5	-78.5	%50	%100
14	Rail 3	z	-7.1	-7.1	0	%50

Member Distributed Loads (BLC 8 : Seismic X)

Member	Label	Direction	Start Magnitude [lb/ft, F, psf, lb-ft/in]	End Magnitude [lb/ft, F, psf, lb-ft/in]	Start Location [(in, %)]	End Location [(in, %)]
1	Rail 2	X	13.5	13.5	0	%100
2	Rail 1	X	6.75	6.75	0	%100
3	Rail 4	X	13.5	13.5	0	%100
4	Rail 5	X	13.5	13.5	0	%100
5	Rail 6	X	13.5	13.5	0	%100
6	Rail 7	X	6.75	6.75	0	%100
7	Rail 3	X	13.5	13.5	0	%100

Member Distributed Loads (BLC 9 : Seismic Z)

Member	Label	Direction	Start Magnitude [lb/ft, F, psf, lb-ft/in]	End Magnitude [lb/ft, F, psf, lb-ft/in]	Start Location [(in, %)]	End Location [(in, %)]
1	Rail 2	Z	13.5	13.5	0	%100
2	Rail 1	Z	6.75	6.75	0	%100
3	Rail 4	Z	13.5	13.5	0	%100
4	Rail 5	Z	13.5	13.5	0	%100
5	Rail 6	Z	13.5	13.5	0	%100
6	Rail 7	Z	6.75	6.75	0	%100
7	Rail 3	Z	13.5	13.5	0	%100

Envelope Maximum Member Section Forces

Member	Axial [lb]	Loc [in]	LCy Shear [lb]	Loc [in]	LCz Shear [lb]	Loc [in]	LC Torque [lb-ft]	Loc [in]	LCy-y Moment [lb-ft]	Loc [in]	LCz-z Moment [lb-ft]	Loc [in]
1	max 158.899	156.087	28 83.944	244.063	30 180.396	25.541	11 0.018	113.518	13 211.068	28.379	11 147.28	28.379
2	min -77.193	201.494	29 -24.246	28.379	32 -180.328	246.901	13 -0.018	158.925	11 -98.202	28.379	24 -149.898	244.063
3	max 3012.53	0	13 2221.228	113.386	29 848.173	0	32 222.286	113.386	32 2397.686	0	13 2254.276	100.394



Company : EPS
 Designer : RK
 Job Number : 23-CHK004
 Model Name : Chiko Premier Carport

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Envelope Maximum Member Section Forces (Continued)

Member	Axial[lb]	Loc[in]	LCy Shear[lb]	Loc[in]	LCz Shear[lb]	Loc[in]	LC Torque[lb-ft]	Loc[in]	LCy-y Moment[lb-ft]	Loc[in]	LCz-z Moment[lb-ft]	Loc[in]	LC							
4	min	-963.764	99.213	26	-1588.842	94.488	29	-622.681	0	13	-225.284	100.394	13	-4686.907	0	32	-2979.745	0	27	
5	Column 3	max	3090.772	0	11	846.013	128.886	11	531.372	93.979	11	366.698	128.886	11	4276.792	0	32	3116.006	0	27
6		min	-1060.833	99.35	24	-1615.669	95.322	11	-714.759	0	32	-295.811	100.692	32	-2261.006	128.886	32	-2032.004	99.35	31
7	Column 2	max	3092.267	0	13	2188.621	113.386	29	676.374	0	30	277.42	113.386	13	3678.479	99.213	13	2229.056	100.394	27
8		min	-1003.248	99.213	26	-1633.806	94.488	29	-209.8	100.394	26	-137.702	94.488	13	-3864.111	0	28	-2930.377	0	31
9	Purlin 3	max	167.115	50.525	11	736.874	31.496	11	40.184	30.84	27	0.184	50.525	26	171.086	31.496	29	1645.988	31.496	11
10		min	-47.478	31.496	24	-263.31	50.525	24	-183.661	31.496	29	-0.319	5.905	13	-116.442	50.525	27	-565.67	31.496	24
11	Beam 2	max	1604.985	156.988	13	1194.984	204.56	13	467.161	221.21	13	1551.939	23.786	11	634.261	204.56	32	1814.764	204.56	13
12		min	-2581.078	159.366	27	-1173.67	23.786	11	-496.583	204.56	32	-1621.528	204.56	13	-550.347	204.56	13	-1142.985	135.58	13
13	Purlin 2	max	106.859	31.496	28	340.562	50.525	24	159.663	31.496	27	0.132	50.525	26	105.861	50.525	27	754.346	31.496	24
14		min	-42.382	31.496	24	-830.357	31.496	11	-77.253	30.84	28	-0.396	31.496	13	-142.288	31.496	27	-1866.848	31.496	11
15	Beam 1	max	1708.584	156.988	13	1210.258	204.56	13	318.423	23.786	11	1757.477	221.21	13	573.133	204.56	13	1838.028	204.56	13
16		min	-2553.819	159.366	27	-1193.38	23.786	11	-480.772	204.56	13	-1741.782	7.136	11	-368.07	68.98	11	-1234.856	135.58	13
17	Purlin 1	max	350.776	62.992	11	972.857	31.496	11	33.374	50.525	24	0.558	50.525	11	167.153	31.496	11	2176.851	31.496	11
18		min	-94.016	51.181	24	-430.035	50.525	24	-140.144	31.496	29	-0.297	31.496	24	-54.352	50.525	29	-950.308	31.496	24
19	Rail 1	max	44.249	156.087	11	34.801	198.656	32	101.096	244.063	13	0.017	113.518	13	107.097	28.379	11	45.762	158.925	32
20		min	-119.681	116.355	32	-35.508	28.379	28	-94.819	28.379	11	-0.018	158.925	11	-47.948	28.379	24	-50.408	28.379	28
21	Purlin 4	max	66.075	62.992	11	245.817	50.525	26	207.144	31.496	29	0.316	31.496	11	130.261	50.525	29	531.241	31.496	26
22		min	-16.058	31.496	24	-688.282	31.496	13	-53.702	30.84	29	-0.2	31.496	24	-194.22	31.496	29	-1548.721	31.496	13
23	Purlin 6	max	489.266	50.525	13	437.245	50.525	26	106.846	31.496	29	0.265	50.525	26	56.547	50.525	29	963.946	31.496	26
24		min	-171.214	31.496	26	-991.809	31.496	13	-46.206	30.84	13	-0.488	31.496	13	-108.888	31.496	29	-2214.268	31.496	13
25	Rail 4	max	46.588	70.948	29	89.621	244.063	28	180.404	25.541	11	0.003	198.656	30	211.795	28.379	11	106.045	158.925	32
26		min	-45.33	201.494	29	-22.605	246.901	28	-180.319	246.901	13	-0.003	73.786	30	-98.474	28.379	24	-150.632	244.063	28
27	Rail 5	max	35.067	70.948	29	81.584	244.063	32	180.402	25.541	11	0.006	156.087	7	211.635	28.379	11	190.787	244.063	29
28		min	-31.293	201.494	29	-97.126	28.379	29	-180.321	246.901	13	-0.006	116.355	24	-98.443	28.379	24	-189.975	28.379	29
29	Rail 6	max	59.914	113.518	29	67.86	244.063	32	182.034	244.063	13	0.008	156.087	7	210.764	28.379	11	205.325	244.063	29
30		min	-111.489	116.355	32	-109.737	201.494	29	-180.327	246.901	13	-0.008	116.355	24	-98.148	28.379	24	-208.716	28.379	29
31	Rail 7	max	130.921	156.087	28	38.306	198.656	28	97.453	244.063	13	0.009	198.656	11	107.331	28.379	11	51.786	158.925	28
32		min	-18.652	201.494	31	-32.974	28.379	32	-92.019	28.379	11	-0.008	73.786	13	-47.976	28.379	24	-48.161	244.063	30
33	Column 1	max	3214.281	0	11	938.1	128.886	11	191.196	128.886	24	149.048	128.886	24	3487.114	0	28	3063.12	0	31
34		min	-1107.416	99.35	24	-1756.091	95.322	11	-587.765	0	30	-528.766	100.692	11	-3246.455	128.886	11	-1995.967	99.35	31
35	Purlin 5	max	92.579	31.496	28	809.169	31.496	13	38.449	30.84	27	0.298	50.525	11	217.496	31.496	30	1827.598	31.496	13
36		min	-37.196	30.84	32	-332.744	50.525	26	-223.521	31.496	30	-0.095	5.905	24	-136.076	50.525	30	-739.152	31.496	26
37	Rail 3	max	28.399	70.948	29	128.961	244.063	30	180.401	25.541	11	0.005	156.087	24	211.844	28.379	11	241.171	28.379	29
38		min	-26.834	201.494	29	-22.605	246.901	28	-180.322	246.901	13	-0.005	116.355	7	-98.485	28.379	24	-248.724	244.063	29
39	M62	max	253.809	177.165	24	41.614	0	27	34.956	177.165	32	129.241	177.165	13	0	177.165	32	0	177.165	32
40		min	-1669.664	0	11	-41.614	177.165	9	-34.956	0	28	-63.776	0	26	-129.019	88.582	28	-153.594	88.582	27
41	M63	max	233.939	177.165	24	41.614	0	27	34.956	177.165	32	39.789	177.165	26	0	177.165	32	0	177.165	32
42		min	-1561.558	0	11	-41.614	177.165	9	-34.956	0	28	-73.744	0	13	-129.019	88.582	28	-153.594	88.582	27
43	M65	max	3092.293	0	11	15.32	0	27	10.027	50.82	28	1242.642	50.82	11	0	50.82	32	0	50.82	32
44		min	-1205.716	0	31	-15.32	50.82	27	-10.027	0	32	-348.531	0	24	-10.616	25.41	32	-16.22	25.41	27
45	M66	max	4091.426	0	29	10.404	0	9	9.373	0	32	296.353	47.503	26	9.275	23.751	28	0	47.503	32
46		min	-407.873	47.503	26	-10.404	47.503	13	-9.373	47.503	28	-1148.568	0	13	0	0	1	-10.296	23.751	9
47	M68	max	2826.412	0	11	15.32	0	27	10.027	50.82	32	803.889	50.82	32	0	50.82	32	0	50.82	32
48		min	-1250.692	0	31	-15.32	50.82	27	-10.027	0	28	-886.483	0	11	-10.616	25.41	28	-16.22	25.41	27
49	M70	max	4078.213	0	29	10.404	0	7	9.373	0	28	826.066	47.503	13	9.275	23.751	32	0	47.503	32
50		min	-338.5	47.503	26	-10.404	47.503	18	-9.373	47.503	32	-802.942	0	32	0	0	1	-10.296	23.751	7
51	M72	max	166.372	32.156	11	441.848	0	11	184.184	188.344	29	0.196	213.609	13	186.601	167.672	29	1067.247	188.344	11
52		min	-109.972	188.344	32	-684.314	188.344	11	-109.18	0	29	-0.151	169.969	7	-281.162	32.156	29	-1846.953	78.094	11
53	M74	max	99.403	188.344	11	789.347	188.344	11	111.854	0	29	0.23	188.344	13	288.328	32.156	29	2086.181	78.094	11
54		min	-143.558	188.344	32	-513.694	0	11	-165.267	188.344	27	-0.111	190.641	32	-196.037	167.672	29	-1580.896	188.344	11
55	M75	max	415	78.094	11	590.61	0	11	141.101	188.344	29	0.287	188.344	24	166.675	188.344	11	1926.943	188.344	11
56		min	-202.197	188.344	32	-952.496	188.344	11	-91.056	0	29	-0.587	169.969	11	-232.595	32.156	29	-2398.935	78.094	11
57	M76	max	66.075	32.156	11	641.276	188.344	13	108.038	0	29	0.15	188.344	24	278.103	32.156	29	1776.443	78.094	13
58		min	-60.195	188.344	32	-422.979	0	13	-218.382	188.344										



Company : EPS
 Designer : RK
 Job Number : 23-CHK004
 Model Name : Chiko Premier Carport

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Envelope Maximum Member Section Forces (Continued)

Member	Axial[lb]	Loc[in]	LCy Shear[lb]	Loc[in]	LCz Shear[lb]	Loc[in]	LC Torque[lb-ft]	Loc[in]	LCy-y Moment[lb-ft]	Loc[in]	LCz-z Moment[lb-ft]	Loc[in]	LC
59 M77	max 479.041	188.344	13 977.084	188.344	13 92.947	0	29 0.521	188.344	13 237.664	32.156	29 2411.438	78.094	13
60	min -431.882	188.344	32 -595.721	0	13 -95.972	167.672	29 -0.26	169.969	26 -156.306	167.672	29 -2016.835	188.344	13
61 M78	max 106.727	124.031	13 503.355	0	13 233.156	188.344	29 0.101	213.609	32 215.25	188.344	29 1593.412	188.344	13
62	min -129.018	188.344	32 -773.257	188.344	13 -147.781	169.969	32 -0.15	169.969	11 -286.901	32.156	29 -2036.684	78.094	13

Envelope AISI S100-16: ASD Member Cold Formed Steel Code Checks

Member	Shape	Code Check	Loc[in]	LC Shear Check	Loc[in]	Dir	LC Pn/Om[lb]	Tn/Om[lb]	Mnyy/Om[lb-ft]	Mnzz/Om[lb-ft]	Vny/Om[lb]	Vnz/Om[lb]	Cb	Eqn
1 Rail 2	CHIKOHLRAIL	- Can't do code check, flange w/t > limit (Table B4.1-1)												
2 Column 4	CHIKOHLTUBE	0.933	0	32 0.15	113.386	y	29 28419.535	49601.13	5323.328	5323.328	14785.867	14785.867	1.351	H1.2-1
3 Column 3	CHIKOHLTUBE	0.862	0	32 0.109	99.35	y	11 27157.259	49601.13	5323.328	5323.328	14785.867	14785.867	2.154	H1.2-1
4 Column 2	CHIKOHLTUBE	0.885	99.213	13 0.148	113.386	y	29 28419.535	49601.13	5323.328	5323.328	14785.867	14785.867	2.261	H1.2-1
5 Purlin 3	CHIKOHLPURLIN	0.37	31.496	11 0.075	31.496	y	11 19064.487	35198.791	1360.961	5607.876	10198.376	7236.577	1	H1.2-1
6 Beam 2	CHIKOHLTUBE	0.444	204.56	13 0.081	204.56	y	13 22813.831	49601.13	5323.328	5323.328	14785.867	14785.867	1	H1.1-2
7 Purlin 2	CHIKOHLPURLIN	0.389	31.496	11 0.086	31.496	y	11 19064.487	35198.791	1297.45	5458.185	10198.376	7236.577	1	H1.2-1
8 Beam 1	CHIKOHLTUBE	0.453	204.56	13 0.082	204.56	y	13 22813.831	49601.13	5323.328	5323.328	14785.867	14785.867	1	H1.1-2
9 Purlin 1	CHIKOHLPURLIN	0.527	31.496	11 0.104	31.496	y	11 19064.487	35198.791	1360.961	5607.876	10198.376	7236.577	1	H1.2-1
10 Rail 1	CHIKOHLRAIL	- Can't do code check, flange w/t > limit (Table B4.1-1)												
11 Purlin 4	CHIKOHLPURLIN	0.387	31.496	13 0.07	31.496	y	13 19064.487	35198.791	1297.45	5378.657	10198.376	7236.577	1	H1.2-1
12 Purlin 6	CHIKOHLPURLIN	0.49	31.496	13 0.104	31.496	y	13 19064.487	35198.791	1297.45	5439.647	10198.376	7236.577	1	H1.2-1
13 Rail 4	CHIKOHLRAIL	- Can't do code check, flange w/t > limit (Table B4.1-1)												
14 Rail 5	CHIKOHLRAIL	- Can't do code check, flange w/t > limit (Table B4.1-1)												
15 Rail 6	CHIKOHLRAIL	- Can't do code check, flange w/t > limit (Table B4.1-1)												
16 Rail 7	CHIKOHLRAIL	- Can't do code check, flange w/t > limit (Table B4.1-1)												
17 Column 1	CHIKOHLTUBE	0.888	128.886	11 0.119	99.35	y	11 27157.259	49601.13	5323.328	5323.328	14785.867	14785.867	2.383	H1.2-1
18 Purlin 5	CHIKOHLPURLIN	0.46	31.496	13 0.083	31.496	y	13 19064.487	35198.791	1360.961	5607.876	10198.376	7236.577	1	H1.2-1
19 Rail 3	CHIKOHLRAIL	- Can't do code check, flange w/t > limit (Table B4.1-1)												
20 M62	CHIKOHLTUBE	0.064	88.583	30 0.003	177.165	y	17 22912.569	49601.13	6218.203	6219.19	14785.867	14785.867	1.136	H1.1-1
21 M63	CHIKOHLTUBE	0.058	88.583	30 0.003	177.165	y	9 22912.569	49601.13	6218.203	6219.19	14785.867	14785.867	1.136	H1.1-1
22 M65	CHIKOHLTUBE	0.095	24.351	11 0.001	50.82	y	27 33332.728	49601.13	5323.328	5323.328	14785.867	14785.867	1.136	H1.2-1
23 M66	CHIKOHLTUBE	0.123	21.277	29 0.001	47.503	y	13 33554.184	49601.13	5323.328	5323.328	14785.867	14785.867	1.136	H1.2-1
24 M68	CHIKOHLTUBE	0.087	24.351	11 0.001	50.82	y	27 33332.728	49601.13	5323.328	5323.328	14785.867	14785.867	1.136	H1.2-1
25 M70	CHIKOHLTUBE	0.123	21.277	29 0.001	47.503	y	18 33554.184	49601.13	5323.328	5323.328	14785.867	14785.867	1.136	H1.2-1
26 M72	CHIKOHLPURLIN	0.434	78.094	11 0.069	188.344	y	11 3491.361	35198.791	1297.45	5446.685	10198.376	7236.577	1	H1.2-1
27 M74	CHIKOHLPURLIN	0.438	80.391	11 0.08	188.344	y	11 3491.361	35198.791	1360.961	5607.876	10198.376	7236.577	1	H1.2-1
28 M75	CHIKOHLPURLIN	0.606	188.344	11 0.102	188.344	y	11 3491.361	35198.791	1307.835	4864.995	10198.376	7236.577	1	H1.2-1
29 M76	CHIKOHLPURLIN	0.38	78.094	13 0.064	188.344	y	13 3491.361	35198.791	1360.961	5607.876	10198.376	7236.577	1	H1.2-1
30 M77	CHIKOHLPURLIN	0.596	188.344	13 0.104	188.344	y	13 3491.361	35198.791	1297.45	4864.995	10198.376	7236.577	1	H1.2-1
31 M78	CHIKOHLPURLIN	0.491	188.344	13 0.078	188.344	y	13 3491.361	35198.791	1307.835	4864.995	10198.376	7236.577	1	H1.2-1

Envelope Node Reactions

Node Label	X [lb]	LC	Y [lb]	LC	Z [lb]	LC	MX [lb-ft]	LC	MY [lb-ft]	LC	MZ [lb-ft]	LC
1 N140	max 96.645	25	3090.772	11	521.886	11	2224.999	11	98.339	32	3116.006	27
2	min -670.72	27	-1007.865	24	-712.513	32	-4276.792	32	-69.049	13	-517.396	9
3 N143	max 89.979	25	3012.53	13	614.113	13	2397.686	13	73.277	13	2979.745	27
4	min -609.544	27	-910.834	26	-846.6	32	-4686.907	32	-67.918	32	-494.146	9
5 N146	max 104.859	7	3214.281	11	188.254	24	566.406	24	84.683	30	3063.12	31
6	min -659.597	31	-1054.449	24	-576.123	30	-3487.114	28	-21.839	24	-550.844	9
7 N148	max 109.068	7	3092.267	13	206.892	26	538.749	26	42.888	26	2930.377	31
8	min -600.728	31	-950.318	26	-664.521	30	-3864.111	28	-137.702	13	-540.491	7
9 N287	max NC		NC		NC		LOCKED		LOCKED		NC	
10	min NC		NC		NC		LOCKED		LOCKED		NC	
11 N286	max NC		NC		NC		LOCKED		LOCKED		NC	
12	min NC		NC		NC		LOCKED		LOCKED		NC	



Envelope Node Reactions (Continued)

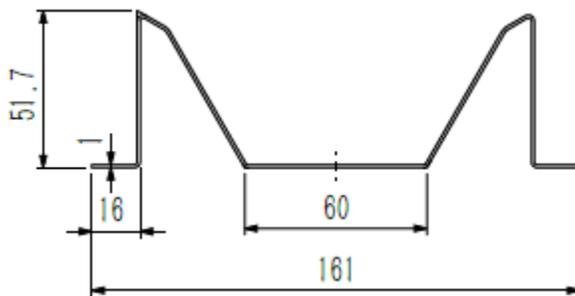
	Node Label		X [lb]	LC	Y [lb]	LC	Z [lb]	LC	MX [lb-ft]	LC	MY [lb-ft]	LC	MZ [lb-ft]	LC
13	N285	max	NC		NC		NC		LOCKED		LOCKED		NC	
14		min	NC		NC		NC		LOCKED		LOCKED		NC	
15	N288	max	NC		NC		NC		LOCKED		LOCKED		NC	
16		min	NC		NC		NC		LOCKED		LOCKED		NC	
17	N289	max	NC		NC		NC		LOCKED		LOCKED		NC	
18		min	NC		NC		NC		LOCKED		LOCKED		NC	
19	N290	max	NC		NC		NC		LOCKED		LOCKED		NC	
20		min	NC		NC		NC		LOCKED		LOCKED		NC	
21	Totals:	max	381.105	7	10889.824	13	0	11						
22		min	-2538.412	31	-1669.649	26	-2538.412	32						



3.3.1 – Rail Calculations

The following pages contain the calculations for the worst-case uplift and downward forces on the rails. This includes dead, wind, and snow applied using the combinations in the governing building code.

Chiko M-Rail



Plane Properties:
Area: 0.50091 sq in
Perimeter: 25.525 in
Centroid:
xC: 11.48 in
yC: -5.235 in
Section Properties:
About The Centroidal X-Axis:
Moment of Inertia: 0.23968 in ⁴
Section Modulus: 0.17171 in ³
Radius of Gyration: 0.69172 in
About The Centroidal Y-Axis:
Moment of Inertia: 2.2267 in ⁴
Section Modulus: 0.70259 in ³
Radius of Gyration: 2.1084 in

Chiko M-Rail Profile and Section Properties (Profile per Chiko)

Per the AISC Steel Construction Manual, the allowable stress for the proposed Chiko M-Rail (50 ksi Steel) is $(2/3) * 50 \text{ ksi} = 33 \text{ ksi}$

The maximum moment of any Premium Carport size under the loads given in this packet on the M-Rails (example analysis shown in the Risa Printout in Section 3.4) is 214 lbs.-ft which, based on the section modulus shown above of 0.17171 in³, produces a stress of 15.0 ksi – less than the allowable stress of 33 ksi.

Therefore, the Chiko M-Rail is adequate for the proposed design forces.



3.4 – Connections

The previous RISA printout shows an example of the maximum loads (axial, shear, moment, etc.) to each member at the connection points under one of the prescribed loading scenarios. While only one RISA analysis is shown previously, all carport sizes under all governing loading scenarios have been analyzed similarly. The following analysis provides the capacity of each connection checked against the maximum envelope loads from all carport sizes under the prescribed loading scenarios. All bolt allowable capacities are per the AISC Steel Construction Manual for A307 1/4", 3/8", 1/2", and 5/8" bolts as noted in each calculation (excluding the connection from the M-Rail Clip to the M-Rail and the connection into the concrete pier).



ENGINEERED POWER SOLUTIONS

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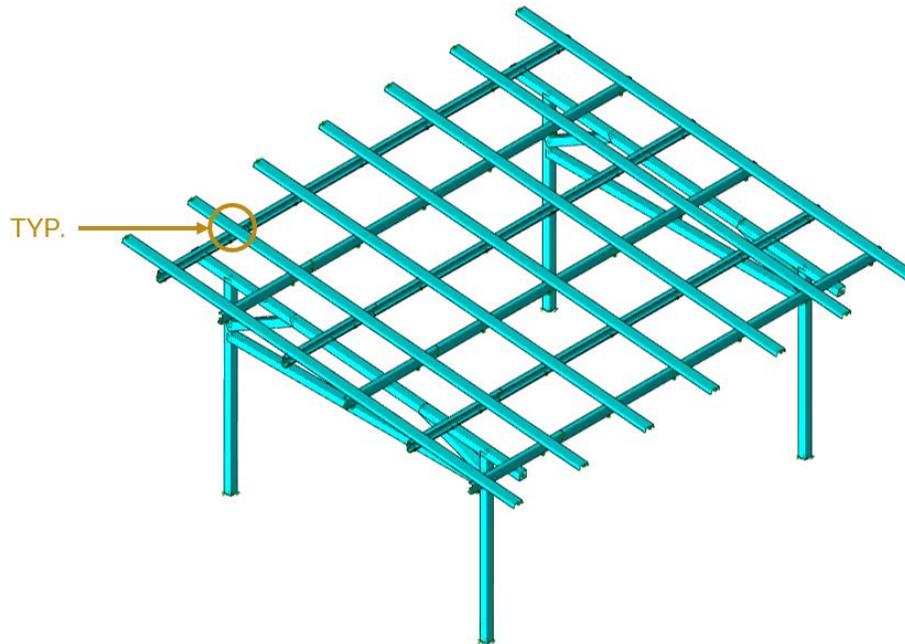
- M-Rail Clip to M-Rail:

EPS has analyzed the worst-case shear loads from the loading given in this packet. EPS has found the shear loads for the M-Rail Clip to M-Rail connection to be no greater than 1,000 lbs. EPS has found the tension loads for the M-Rail Clip to M-Rail connection to be no greater than 225 lbs. to (1) bolt.

- Connection Hardware: (2) ITW Buildex ¼" Screws (or approved equal)
- Minimum member thickness: 0.039" (1 mm) (M-Rail)
- Minimum member ultimate strength (F_u): 45ksi (Bracket)
- Screw Shear Allowable Capacity (per ICC Report ESR-1976, See Appendix):
647 lbs. * 2 bolts = 1,294 lbs. \geq 1,000 lbs. \rightarrow OK
- Screw Tension Allowable Capacity (per ICC Report ESR-1976, See Appendix):
239 lbs. \geq 225 lbs. \rightarrow OK



- M-Rail to Purlin:

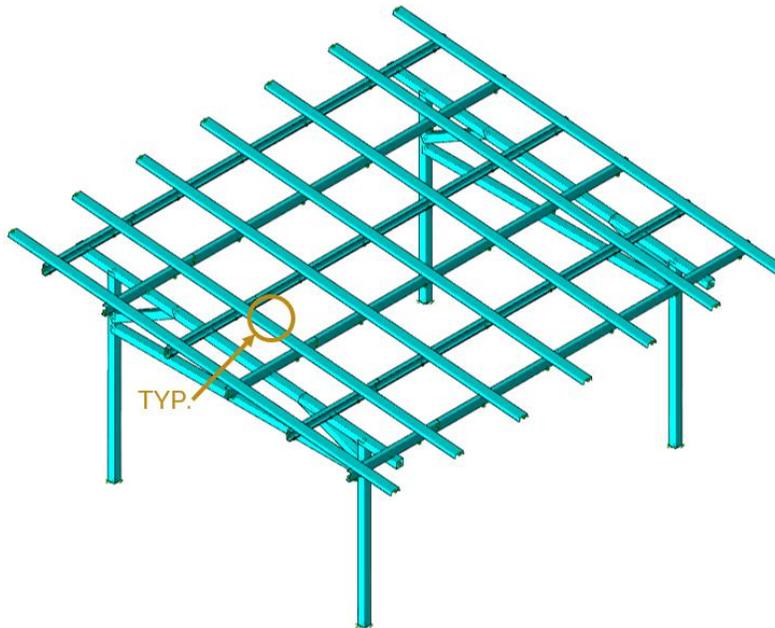


EPS has analyzed the worst-case shear loads from the loading given in this packet. EPS has found the shear loads for the M-Rail to Purlin connection to be no greater than 1,000 lbs. EPS has found the tension loads for the M-Rail to Purlin connection to be no greater than 1,500 lbs.

- Connection Hardware: (2) A307 1/4" Dia. Bolt
- Bolt Shear Allowable Capacity (Single Shear):
661 lbs. * 2 bolts = 1,322 lbs. ≥ 1,000 lbs. → OK
- Bolt Tension Allowable Capacity:
1,103 lbs. * 2 bolts = 2,206 lbs. ≥ 1,500 lbs. → OK
- Bearing Strength at Screw Holes (Eq. J3-6a) Based on Racking Framing
- Minimum member thickness: 0.039" (1 mm) (M-Rail)
- Minimum member ultimate strength (F_u): 45ksi (Bracket)
 - $(2.4 * 0.25 * 0.039 * 2 \text{ bolts} * 45\text{ksi}) = 2,106 \text{ lbs.} / \Omega =$
1,053 lbs. ≥ 300 lbs. → OK
 - Where $\Omega = 2.00$ (ASD level)



- M-Rail Splice:

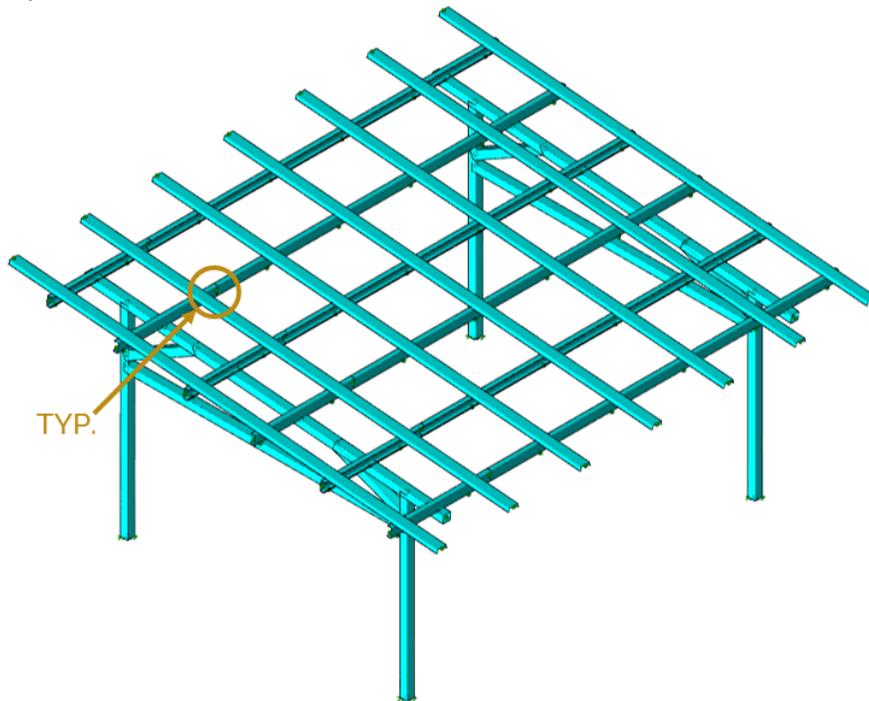


EPS has analyzed the worst-case shear loads from the loading given in this packet. EPS has found the shear loads for the M-Rail Splice connection to be no greater than 4,500 lbs. EPS has found the tension loads for the M-Rail Splice connection to be negligible.

- Connection Hardware: (6) A307 ¼" Dia. Bolt
- Bolt Shear Allowable Capacity (Single Shear):
661 lbs. * 8 bolts = 5,288 lbs. ≥ 4,500 lbs. → OK
- Bearing Strength at Screw Holes (Eq. J3-6a) Based on Racking Framing
- Minimum member thickness: 0.039" (1 mm) (M-Rail)
- Minimum member ultimate strength (F_u): 45ksi (Bracket)
 - $(2.4 * 0.25" * 0.039" * 2 \text{ bolts} * 45\text{ksi}) = 2,106 \text{ lbs.} / \Omega =$
1,053 lbs. ≥ 300 lbs. → OK
 - Where $\Omega = 2.00$ (ASD level)



- Purlin Splice:

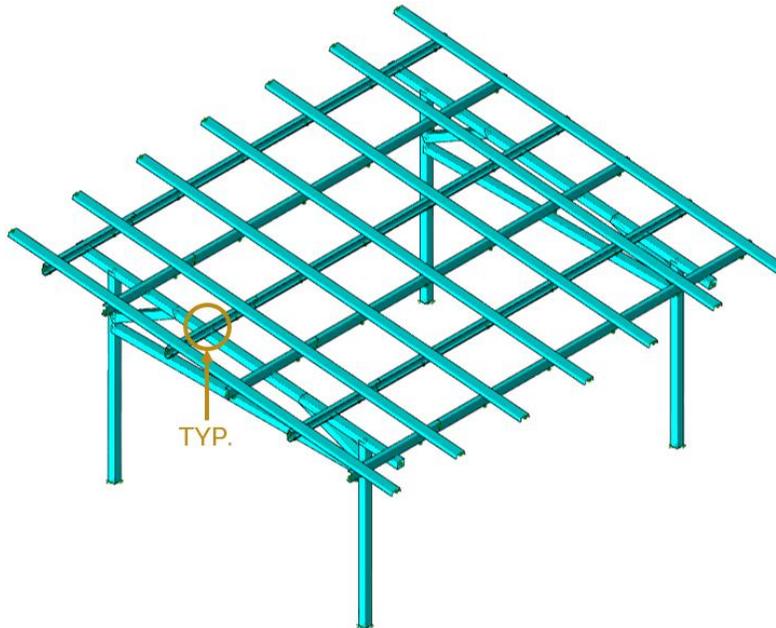


EPS has analyzed the worst-case shear loads from the loading given in this packet. EPS has found the shear loads for the Purlin Splice connection to be no greater than 1,100 lbs. to a single bolt. EPS has found the tension loads for the Purlin Splice connection to be no greater than 2,100 lbs. to a single bolt.

- Connection Hardware: (16) A307 3/8" Dia. Bolt*
*Conservative as actual bolt is M10.
- Bolt Shear Allowable Capacity (Single Shear):
1,489 lbs. \geq 1,100 lbs. \rightarrow OK
- Bolt Tension Allowable Capacity:
2,482 lbs. \geq 2,100 lbs. \rightarrow OK
- Bearing Strength at Screw Holes (Eq. J3-6a) Based on Racking Framing
- Minimum member thickness: 0.098" (2.5 mm) (Purlin)
- Minimum member ultimate strength (F_u): 65ksi* (Bracket and Purlin)
*(conservatively assumed)
 - $(2.4 * 0.375 * 0.098 * 65\text{ksi}) = 5.733 \text{ lbs.} / \Omega =$
2,866 lbs. \geq 1,100 lbs. \rightarrow OK
 - Where $\Omega = 2.00$ (ASD level)



- Purlin to Beam:

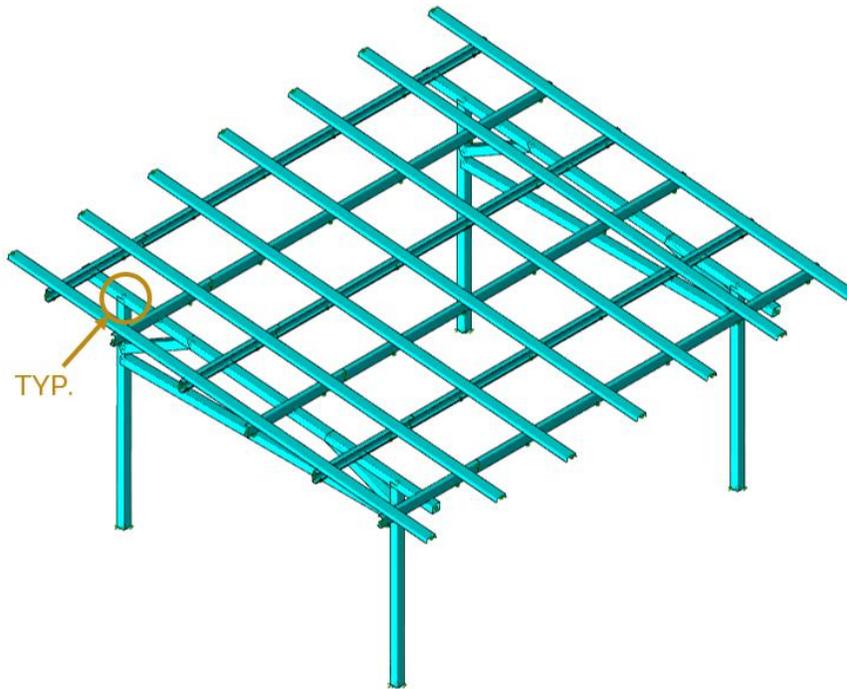


EPS has analyzed the worst-case shear loads from the loading given in this packet. EPS has found the shear loads for the Purlin to Beam connection to be no greater than 600 lbs. and 400 lbs. to a single 3/8" dia. and 1/2" dia. bolt respectively. EPS has found the tension loads for the Purlin to Beam connection to be no greater than 300 lbs. and 500 lbs. to a single 3/8" dia. and 1/2" dia. bolt respectively.

- Connection Hardware: (4) A307 3/8" Dia. Bolt* and (6) A307 1/2" Dia. Bolt*
*Conservative as actual bolt is M10 and M14
- Bolt Shear Allowable Capacity (Single Shear):
 - 3/8" Dia.: 1,489 lbs. \geq 600 lbs. \rightarrow OK
 - 1/2" Dia.: 2,647 lbs. \geq 400 lbs. \rightarrow OK
- Bolt Tension Allowable Capacity:
 - 3/8" Dia.: 2,482 lbs. \geq 300 lbs. \rightarrow OK
 - 1/2" Dia.: 4,413 lbs. \geq 500 lbs. \rightarrow OK
- Bearing Strength at Screw Holes (Eq. J3-6a) Based on Racking Framing
- Minimum member thickness: 0.098" (2.5 mm) (Purlin)
- Minimum member ultimate strength (F_u): 45ksi* (Bracket)
*(conservatively assumed)
 - $(2.4 * 0.375 * 0.098 * 45\text{ksi}) = 3,969 \text{ lbs.} / \Omega =$
1,984 lbs. \geq 600 lbs. \rightarrow OK
 - Where $\Omega = 2.00$ (ASD level)



- Beam to Column:

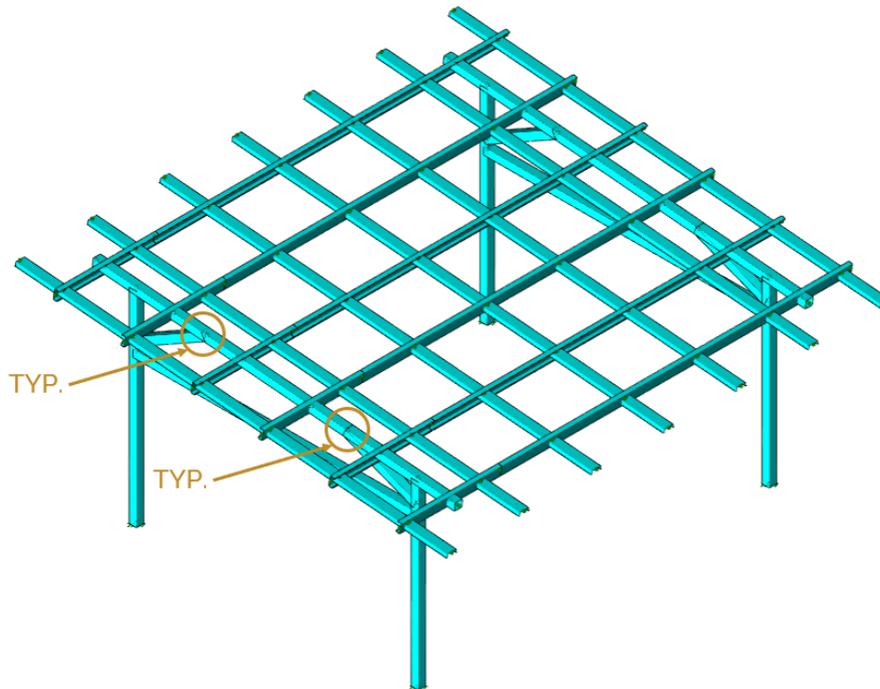


EPS has analyzed the worst-case shear loads from the loading given in this packet. EPS has found the shear loads for the Beam to Column connection to be no greater than 3,900 lbs. to a single bolt. EPS has found the tension loads for the Beam to Column connection to be no greater than 3,900 lbs. to a single bolt.

- Connection Hardware: (2) A307 1/2" Dia. Bolt*
*Conservative as actual bolt is M14.
- Bolt Shear Allowable Capacity (Double Shear):
5,302 lbs. \geq 3,900 lbs. \rightarrow OK
- Bolt Tension Allowable Capacity:
4,413 lbs. \geq 3,900 lbs. \rightarrow OK
- Bearing Strength at Screw Holes (Eq. J3-6a) Based on Racking Framing (Double Shear)
- Minimum member thickness: 0.09" (2.3 mm) (Beam)
- Minimum member ultimate strength (F_u): 45ksi* (Bracket)
*(Conservatively assumed)
 - $(2.4 \times 0.5 \times 0.09 \times 45 \text{ksi} \times 2 \text{ holes}) = 9,720 \text{ lbs.} / \Omega =$
4,860 lbs. \geq 3,900 lbs. \rightarrow OK
 - Where $\Omega = 2.00$ (ASD level)



- Diagonal Brace to Beam:

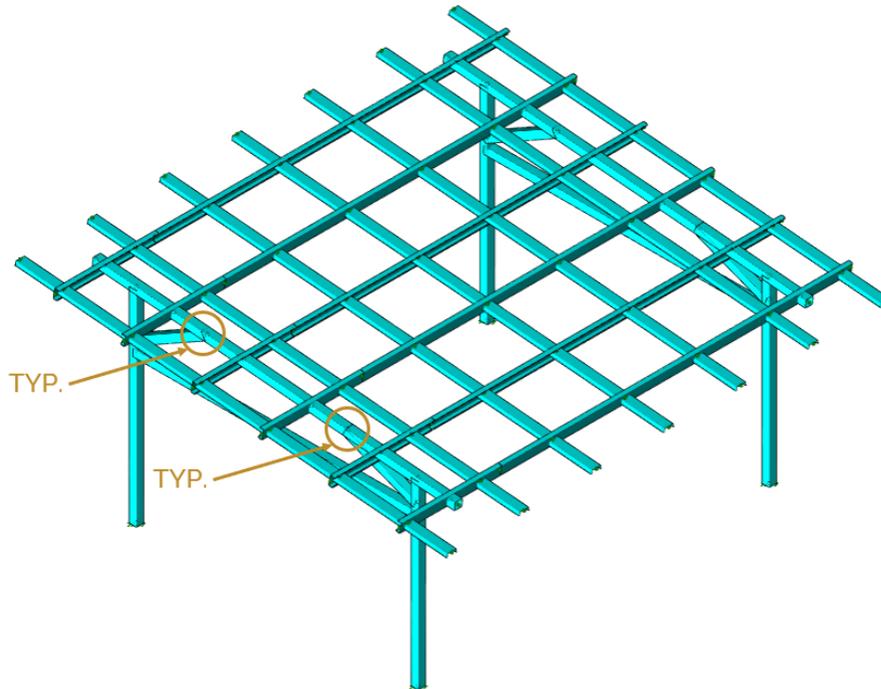


EPS has analyzed the worst-case shear loads from the loading given in this packet. EPS has found the shear loads for the Diagonal Brace to Beam connection to be no greater than 3,500 lbs. and 6,900 lbs. to a single 1/2" dia. and 5/8" dia. bolt respectively. EPS has found the tension loads for the Diagonal Brace to Beam connection to be no greater than 3,500 lbs. 100 lbs. and to a single 1/2" dia. and 5/8" dia. bolt respectively.

- o Connection Hardware: (2) A307 1/2" Dia. Bolt* and (1) A307 5/8" Dia. Bolt*
*Conservative as actual bolt is M14 and M16
- o Bolt Shear Allowable Capacity (Double Shear):
 - 1/2" Dia.: 5,302 lbs. \geq 3,500 lbs. \rightarrow OK
 - 5/8" Dia.: 8,290 lbs. \geq 6,900 lbs. \rightarrow OK
- o Bolt Tension Allowable Capacity:
 - 1/2" Dia.: 4,414 lbs. \geq 3,500 lbs. \rightarrow OK
 - 5/8" Dia.: 6,900 lbs. \geq 100 lbs. \rightarrow OK



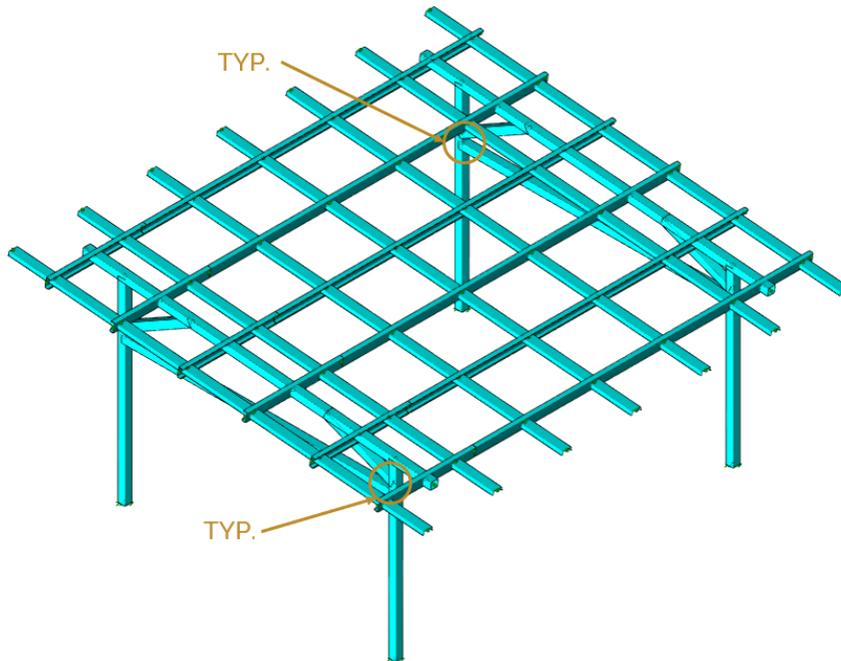
- Diagonal Brace to Beam Continued:



- Bearing Strength at Screw Holes (Eq. J3-6a) Based on Racking Framing (Double Shear)
- Member thickness: 0.09" (2.3 mm) (Diagonal Brace), 0.23" (6 mm) (Bracket)
- Member ultimate strength (F_u): 65ksi* (Diagonal Brace), 45 ksi* Bracket)
*(Conservatively assumed)
 - 1/2" Dia.: $(2.4 \times 0.5 \times 0.09 \times 45 \text{ksi} \times 2 \text{ holes}) = 9,720 \text{ lbs.} / \Omega = 4,860 \text{ lbs.} \geq 3,500 \text{ lbs.} \rightarrow \text{OK}$
 - 5/8" Dia.: $(2.4 \times 0.625 \times 0.09 \times 65 \text{ksi} \times 2 \text{ holes}) = 17,550 \text{ lbs.} / \Omega = 8,775 \text{ lbs.} \geq 6,900 \text{ lbs.} \rightarrow \text{OK}$
 - 5/8" Dia.: $(2.4 \times 0.625 \times 0.23 \times 45 \text{ksi} \times 2 \text{ holes}) = 31,050 \text{ lbs.} / \Omega = 15,525 \text{ lbs.} \geq 6,900 \text{ lbs.} \rightarrow \text{OK}$
 - Where $\Omega = 2.00$ (ASD level)



- Diagonal and Horizontal Brace to Column:

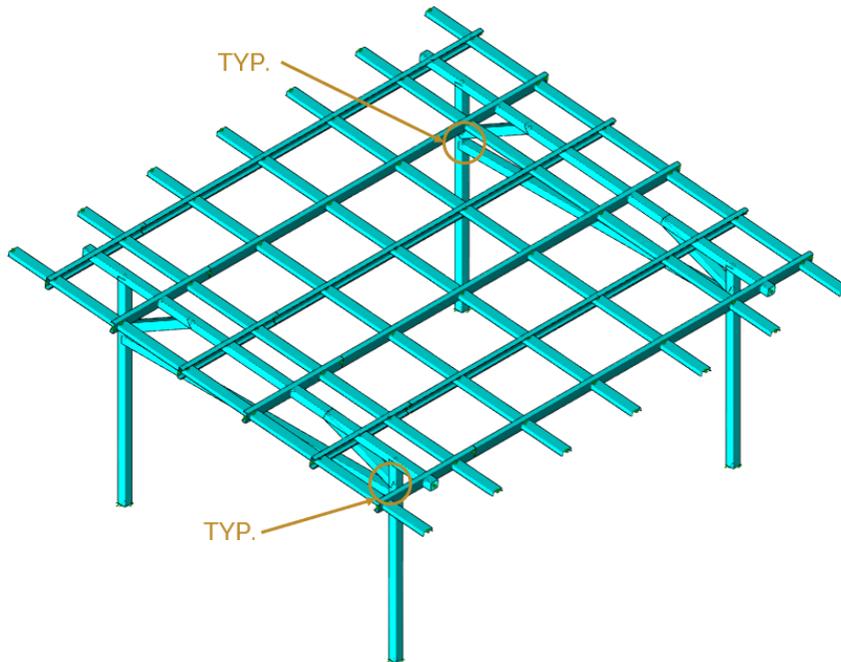


EPS has analyzed the worst-case shear loads from the loading given in this packet. EPS has found the shear loads for the Diagonal and Horizontal Brace to Column connection to be no greater than 3,400 lbs. and 6,900 lbs. to a single 1/2" dia. and 5/8" dia. bolt respectively. EPS has found the tension loads for the Diagonal and Horizontal Brace to Column connection to be no greater than 3,400 lbs. and 100 lbs. to a single 1/2" dia. and 5/8" dia. bolt respectively.

- Connection Hardware: (1-3) A307 1/2" Dia. Bolt* & (1) A307 5/8" Dia. Bolt*
*Conservative as actual bolt is M14 and M16
- Bolt Shear Allowable Capacity (Double Shear):
 - 1/2" Dia.: 5,302 lbs. ≥ 3,400 lbs. → OK
 - 5/8" Dia.: 8,290 lbs. ≥ 6,900 lbs. → OK
- Bolt Tension Allowable Capacity:
 - 1/2" Dia.: 4,414 lbs. ≥ 3,400 lbs. → OK
 - 5/8" Dia.: 6,900 lbs. ≥ 100 lbs. → OK



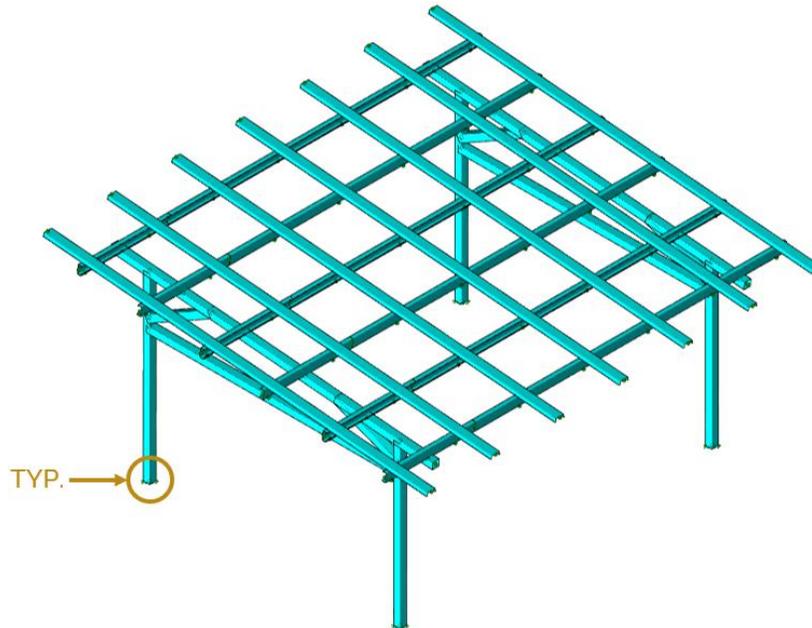
- Diagonal and Horizontal Brace to Column Continued:



- Bearing Strength at Screw Holes (Eq. J3-6a) Based on Racking Framing (Double Shear)
- Member thickness: 0.09" (2.3 mm) (Diagonal & Horizontal Brace), 0.23" (6 mm) (Bracket)
- Member ultimate strength (F_u): 65 ksi* (Diagonal & Horizontal Brace), 45ksi* (Bracket)
(Conservatively assumed)
 - 1/2" Dia.: $(2.4 \times 0.5 \times 0.09 \times 45 \text{ksi} \times 2 \text{ holes}) = 9,720 \text{ lbs.} / \Omega = 4,860 \text{ lbs.} \geq 3,400 \text{ lbs.} \rightarrow \text{OK}$
 - 5/8" Dia.: $(2.4 \times 0.625 \times 0.09 \times 65 \text{ksi} \times 2 \text{ holes}) = 17,550 \text{ lbs.} / \Omega = 8,775 \text{ lbs.} \geq 6,900 \text{ lbs.} \rightarrow \text{OK}$
 - 5/8" Dia.: $(2.4 \times 0.625 \times 0.23 \times 45 \text{ksi} \times 2 \text{ holes}) = 31,050 \text{ lbs.} / \Omega = 15,525 \text{ lbs.} \geq 6,900 \text{ lbs.} \rightarrow \text{OK}$
 - Where $\Omega = 2.00$ (ASD level)



- Column to Baseplate:

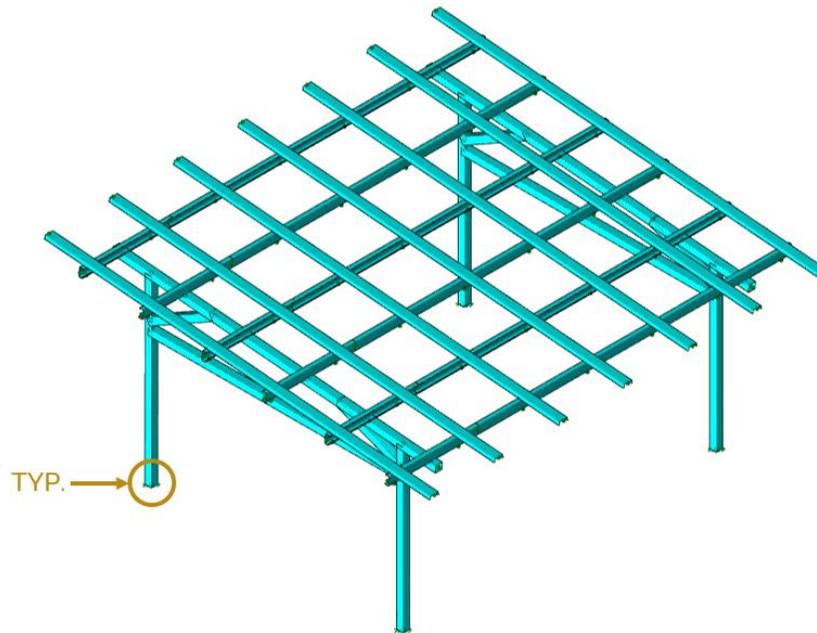


EPS has analyzed the worst-case shear loads from the loading given in this packet. EPS has found the shear loads for the Column to Baseplate connection to be no greater than 5,200 lbs. to a single bolt. EPS has found the tension loads for the Column to Baseplate connection to be no greater than 300 lbs. to a single bolt.

- Connection Hardware: (8) A307 1/2" Dia. Bolt*
*Conservative as actual bolt is M14.
- Bolt Shear Allowable Capacity (Double Shear):
5,302 lbs. \geq 5,200 lbs. \rightarrow OK
- Bolt Tension Allowable Capacity:
4,413 lbs. \geq 300 lbs. \rightarrow OK
- Bearing Strength at Screw Holes (Eq. J3-6a) Based on Racking Framing (Double Shear)
- Member thickness: 0.09" (2.3 mm) (Column), 0.31" (8 mm) (Baseplate)
- Member ultimate strength (F_u): 65 ksi* (Column), 45ksi* (Baseplate)
*(Conservatively assumed)
 - $(2.4 \times 0.5 \times 0.09 \times 65 \text{ ksi} \times 2 \text{ holes}) = 14,040 \text{ lbs.} / \Omega =$
7,020 lbs. \geq 5,200 lbs. \rightarrow OK
 - $(2.4 \times 0.5 \times 0.31 \times 45 \text{ ksi} \times 1 \text{ hole}) = 16,740 \text{ lbs.} / \Omega =$
8,370 lbs. \geq 5,200 lbs. \rightarrow OK
 - Where $\Omega = 2.00$ (ASD level)



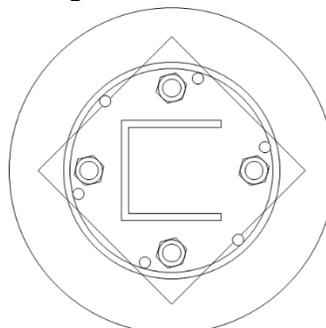
- Baseplate to Concrete Pier:



EPS has analyzed the worst-case shear, tension, and moment loads from the loading given in this packet factored for LRFD level.

EPS has utilized the Hilti Profis software to check the structural capacity of this connection. The worst-case Hilti printouts for wind and seismic forces can be found in the following pages.

From this analysis, EPS has concluded that using (4) 1" anchor bolts with a 1'-9" effective embedment will be acceptable. **All bolts shall be installed within the pier/pedestal circular rebar cage.**



Chiko Baseplate Orientation

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Company:	EPS	Page:	1
Address:		Specifier:	RK
Phone Fax:		E-Mail:	
Design:	Chiko Premier Carport Foundation Wind - DIAGONA	Date:	7/12/2024
Fastening point:			

Specifier's comments:

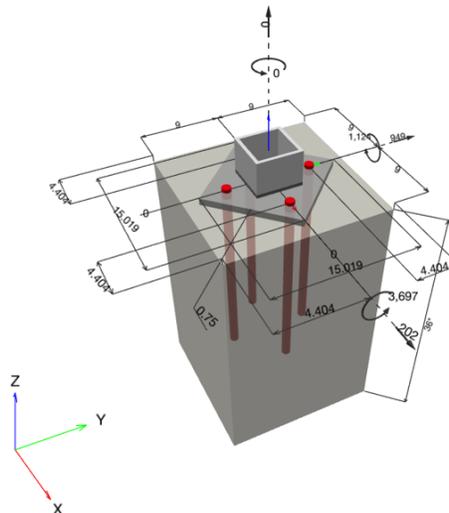
1 Anchor Design

1.1 Input data

Anchor type and diameter:	Hex Head ASTM F 1554 GR. 36 1	
Item number:	not available	
Specification text:	Hilti Hex Head headed stud anchor with 21 in embedment, 1, Steel galvanized, installation per instruction for use	
Effective embedment depth:	$h_{ef} = 21.000$ in.	
Material:	ASTM F 1554	
Evaluation Service Report:	Hilti Technical Data	
Issued Valid:	- -	
Proof:	Design Method ACI 318-19 / CIP	
Stand-off installation:	$e_b = 0.000$ in. (no stand-off); $t = 0.750$ in.	
Anchor plate ^{CBFEM} :	$l_x \times l_y \times t = 15.019$ in. x 15.019 in. x 0.750 in.;	
Profile:	Square HSS (AISC), HSS5X5X.375; (L x W x T) = 5.000 in. x 5.000 in. x 0.375 in.	
Base material:	cracked concrete, 4000, $f'_c = 4,000$ psi; $h = 36.000$ in.	
Reinforcement:	tension: present, shear: present; anchor reinforcement: tension, shear edge reinforcement: > No. 4 bar with stirrups	

^{CBFEM} - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [in.] & Loading [lb, ft.lb]



Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Design:	Chiko Premier Carport Foundation Wind - DIAGONA	Date:	7/12/2024
Fastening point:			

1.1.1 Design results

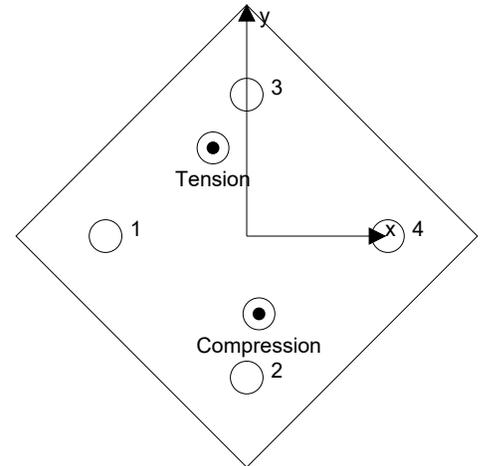
Case	Description	Forces [lb] / Moments [ft.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 0; V _x = 202; V _y = 949; M _x = 3,697.000; M _y = 1,124.000; M _z = 0.000;	no	20

1.2 Load case/Resulting anchor forces

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	2,456	256	56	250
2	-1	234	48	229
3	4,980	226	46	222
4	549	253	53	248



resulting tension force in (x/y)=(-1.097/2.867): 7,984 [lb]
 resulting compression force in (x/y)=(0.399/-2.526): 8,201 [lb]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N _{ua} [lb]	Capacity ϕ N _n [lb]	Utilization $\beta_N = N_{ua} / \phi N_n$	Status
Steel Strength*	4,980	26,361	19	OK
Pullout Strength*	4,980	26,051	20	OK
Concrete Breakout Failure** ¹	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction y+**	4,980	36,045	14	OK

* highest loaded anchor **anchor group (anchors in tension)

¹ Tension Anchor Reinforcement has been selected!



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Fastening point:			

1.3.1 Steel Strength

$$N_{sa} = A_{se,N} f_{uta} \quad \text{ACI 318-19 Eq. (17.6.1.2)}$$

$$\phi N_{sa} \geq N_{ua} \quad \text{ACI 318-19 Table 17.5.2}$$

Variables

$A_{se,N} [\text{in.}^2]$	$f_{uta} [\text{psi}]$
0.61	58,000

Calculations

$N_{sa} [\text{lb}]$
35,148

Results

$N_{sa} [\text{lb}]$	ϕ_{steel}	$\phi N_{sa} [\text{lb}]$	$N_{ua} [\text{lb}]$
35,148	0.750	26,361	4,980

1.3.2 Pullout Strength

$$N_{pN} = \psi_{c,p} N_p \quad \text{ACI 318-19 Eq. (17.6.3.1)}$$

$$N_p = 8 A_{brg} f'_c \quad \text{ACI 318-19 Eq. (17.6.3.2.2a)}$$

$$\phi N_{pN} \geq N_{ua} \quad \text{ACI 318-19 Table 17.5.2}$$

Variables

$\psi_{c,p}$	$A_{brg} [\text{in.}^2]$	λ_a	$f'_c [\text{psi}]$
1.000	1.16	1.000	4,000

Calculations

$N_p [\text{lb}]$
37,216

Results

$N_{pn} [\text{lb}]$	ϕ_{concrete}	$\phi N_{pn} [\text{lb}]$	$N_{ua} [\text{lb}]$
37,216	0.700	26,051	4,980

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Hilti PROFIS Engineering 3.1.1

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Company:	EPS	Page:	4
Address:		Specifier:	RK
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Design:	Chiko Premier Carport Foundation Wind - DIAGONA	Date:	7/12/2024
Fastening point:			

1.3.3 Concrete Side-Face Blowout, direction y+

$$N_{sb} = 160 c_{a1} \sqrt{A_{brg}} \lambda_a \sqrt{f'_c} \quad \text{ACI 318-19 Eq. (17.6.4.1)}$$

$$N_{sbg} = \alpha_{group} N_{sb} \quad \text{ACI 318-19 Eq. (17.6.4.2)}$$

$$\phi N_{sbg} \geq N_{ua} \quad \text{ACI 318-19 Table 17.5.2}$$

$$\alpha_{group} = \left(1 + \frac{s}{6 c_{a1}} \right) \quad \text{see ACI 318-19, Section 17.6.4.2, Eq. (17.6.4.2)}$$

Variables

c_{a1} [in.]	c_{a2} [in.]	A_{brg} [in. ²]	λ_a	f'_c [psi]	s [in.]
4.404	9.000	1.16	1.000	4,000	-

Calculations

α_{group}	N_{sb} [lb]
1.000	48,060

Results

N_{sbg} [lb]	$\phi_{concrete}$	ϕN_{sbg} [lb]	$N_{ua,edge}$ [lb]
48,060	0.750	36,045	4,980



Hilti PROFIS Engineering 3.1.1

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Company:	EPS	Page:	5
Address:		Specifier:	RK
Phone Fax:		E-Mail:	
Design:	Chiko Premier Carport Foundation Wind - DIAGONA	Date:	7/12/2024
Fastening point:			

1.4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_v = V_{ua} / \phi V_n$	Status
Steel Strength*	256	13,708	2	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	970	32,985	3	OK
Concrete edge failure in direction ** ¹	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

¹ Shear Anchor Reinforcement has been selected!

1.4.1 Steel Strength

$$V_{sa} = 0.6 A_{se,V} f_{uta} \quad \text{ACI 318-19 Eq. (17.7.1.2b)}$$

$$\phi V_{steel} \geq V_{ua} \quad \text{ACI 318-19 Table 17.5.2}$$

Variables

$A_{se,V}$ [in. ²]	f_{uta} [psi]
0.61	58,000

Calculations

V_{sa} [lb]
21,089

Results

V_{sa} [lb]	ϕ_{steel}	ϕV_{sa} [lb]	V_{ua} [lb]
21,089	0.650	13,708	256



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1.4.2 Pryout Strength

$$V_{cp,g} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right] \quad \text{ACI 318-19 Eq. (17.7.3.1b)}$$

$$\phi V_{cp,g} \geq V_{ua} \quad \text{ACI 318-19 Table 17.5.2}$$

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$$A_{Nc0} = 9 h_{ef}^2 \quad \text{ACI 318-19 Eq. (17.6.2.1.4)}$$

$$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e_N}{3 h_{ef}}} \right) \leq 1.0 \quad \text{ACI 318-19 Eq. (17.6.2.3.1)}$$

$$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0 \quad \text{ACI 318-19 Eq. (17.6.2.4.1b)}$$

$$\psi_{cp,N} = \text{MAX} \left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0 \quad \text{ACI 318-19 Eq. (17.6.2.6.1b)}$$

$$N_b = k_c \lambda_a \sqrt{f_c} h_{ef}^{1.5} \quad \text{ACI 318-19 Eq. (17.6.2.2.1)}$$

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	2.936	0.000	0.000	4.404
$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f_c [psi]
1.000	∞	24	1.000	4,000

Calculations

A_{Nc} [in. ²]	A_{Nc0} [in. ²]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
239.37	77.58	1.000	1.000	1.000	1.000	7,636

Results

$V_{cp,g}$ [lb]	$\phi_{concrete}$	$\phi V_{cp,g}$ [lb]	V_{ua} [lb]
47,121	0.700	32,985	970

1.5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.191	0.029	5/3	7	OK

$$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} \leq 1$$



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1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to <https://submittals.us.hilti.com/PROFISAnchorDesignGuide/>
- Attention! In case of compressive anchor forces a buckling check as well as the proof of the local load transfer into and within the base material (incl. punching) has to be done separately.
- The design of Anchor Reinforcement is beyond the scope of PROFIS Engineering. Refer to ACI 318-19, Section 17.5.2.1 (a) for information about Anchor Reinforcement.
- The design of Anchor Reinforcement is beyond the scope of PROFIS Engineering. Refer to ACI 318-19, Section 17.5.2.1 (b) for information about Anchor Reinforcement.
- Anchor Reinforcement has been selected as a design option, calculations should be compared with PROFIS Engineering calculations.
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

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1.7 Installation data

Profile: Square HSS (AISC), HSS5X5X.375; (L x W x T) = 5.000 in. x 5.000 in. x 0.375 in.

Hole diameter in the fixture: $d_f = 1.062$ in.

Plate thickness (input): 0.750 in.

Anchor type and diameter: Hex Head ASTM F 1554 GR. 36 1

Item number: not available

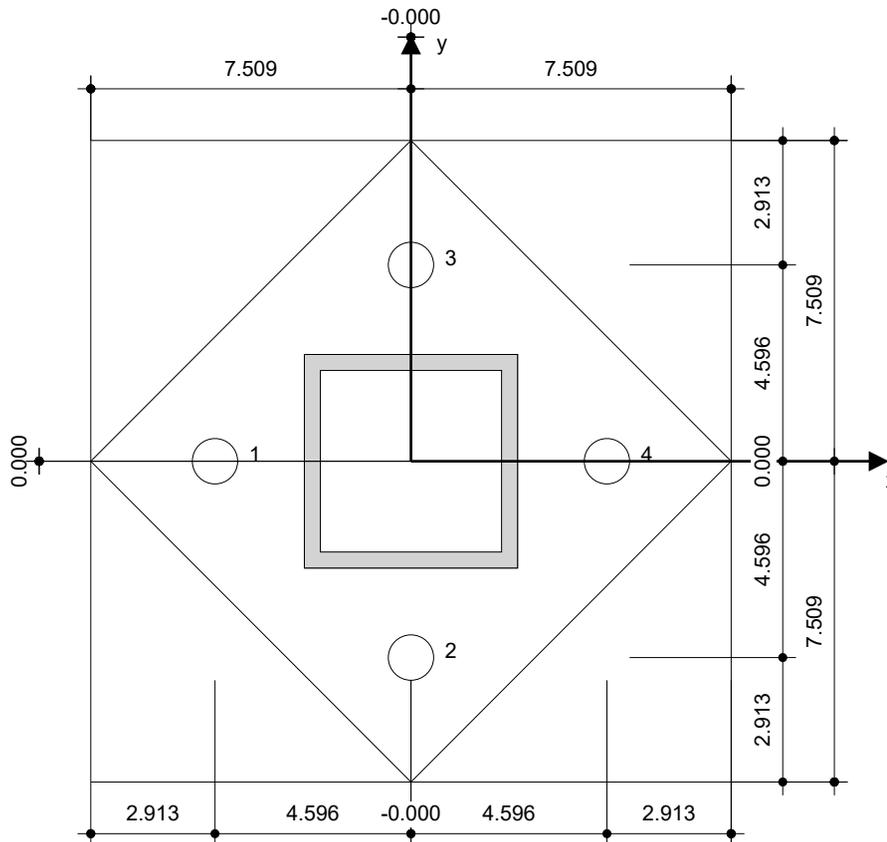
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 21.000 in.

Minimum thickness of the base material: 22.172 in.

Hilti Hex Head headed stud anchor with 21 in embedment, 1, Steel galvanized, installation per instruction for use



Coordinates Anchor [in.]

Anchor	x	y	C _{-x}	C _{+x}	C _{-y}	C _{+y}
1	-4.596	-0.000	4.404	13.596	9.000	9.000
2	0.000	-4.596	9.000	9.000	4.404	13.596
3	-0.000	4.596	9.000	9.000	13.596	4.404
4	4.596	0.000	13.596	4.404	9.000	9.000

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2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Diamond $l_x \times l_y \times t = 15.019 \text{ in} \times 15.019 \text{ in} \times 0.750 \text{ in}$ Calculation: CBFEM Material: ASTM A572 Gr.50; $F_y = 50,000 \text{ psi}$; $\epsilon_{lim} = 5.00\%$
Anchor type and size:	Hex Head ASTM F 1554 GR. 36 1, $h_{ef} = 21.000 \text{ in}$
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	$e_b = 0.000 \text{ in}$ (No stand-off); $t = 0.750 \text{ in}$
Profile:	HSS5X5X.375; (L x W x T x FT) = 5.000 in x 5.000 in x 0.375 in x - Material: ASTM A500 Gr.C Rect; $F_y = 50,000 \text{ psi}$; $\epsilon_{lim} = 5.00\%$ Eccentricity x: -0.000 in Eccentricity y: -0.000 in
Base material:	Cracked concrete; 4000; $f_{c,cyl} = 4,000 \text{ psi}$; $h = 36.000 \text{ in}$
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Anchor plate		Concrete [%]	
		σ_{Ed} [psi]	ϵ_{Pl} [%]	σ_{Ed} [psi]	ϵ_{Pl} [%]	Hole bearing [%]	
1	Combination 1	12,559	0.00	9,395	0.00	1	6

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Combination 1

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	1,587 lb	2,456 lb
Anchor 2	-1 lb	-1 lb
Anchor 3	4,620 lb	4,980 lb
Anchor 4	0 lb	549 lb

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

2.4 Profile/Stiffeners/Plate

Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

2.4.1 Equivalent stress and plastic strain

Part	Load combination	Material	f_y [psi]	ϵ_{lim} [%]	σ_{Ed} [psi]	ϵ_{Pl} [%]	Status
Plate	Combination 1	ASTM A572 Gr.50	50,000	5.00	9,395	0.00	OK

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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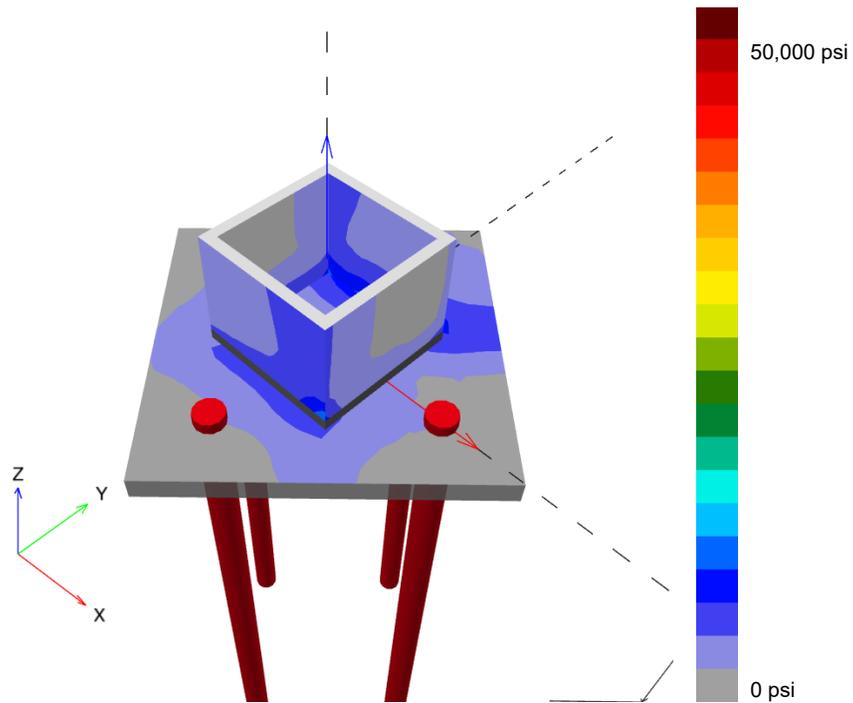
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Part	Load combination	Material	f_y [psi]	ϵ_{lim} [%]	σ_{Ed} [psi]	ϵ_{Pl} [%]	Status
Profile	Combination 1	ASTM A500 Gr.C Rect	50,000	5.00	12,559	0.00	OK
Profile	Combination 1	ASTM A500 Gr.C Rect	50,000	5.00	11,531	0.00	OK
Profile	Combination 1	ASTM A500 Gr.C Rect	50,000	5.00	7,642	0.00	OK
Profile	Combination 1	ASTM A500 Gr.C Rect	50,000	5.00	8,468	0.00	OK

2.4.1.1 Equivalent stress

Results below are displayed for the decisive load combination: 1 - Combination 1

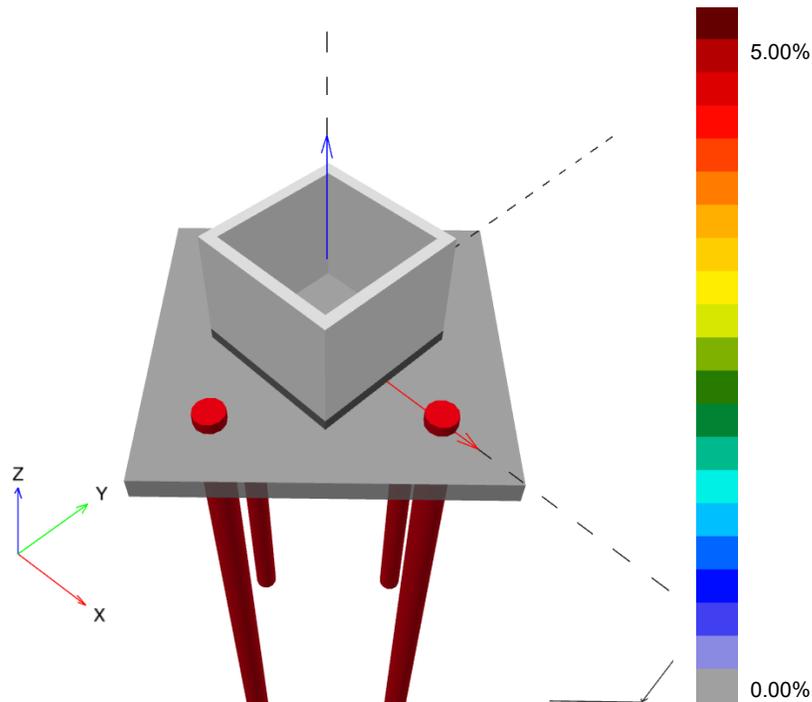


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2.4.1.2 Plastic strain

Results below are displayed for the decisive load combination: 1 - Combination 1



2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 1 - Combination 1

Equations

$$R_n = \min(1.2 l_c t F_u, 2.4 d t F_u) \quad (\text{AISC 360-16 J3-6a, c})$$

$$\Phi R_n = 0.75 R_n$$

$$V \leq \Phi R_n$$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [lb]
Anchor 1	1.909	0.750	65,000	1.000	111,678
Anchor 2	1.929	0.750	65,000	1.000	112,856
Anchor 3	9.718	0.750	65,000	1.000	117,000
Anchor 4	3.247	0.750	65,000	1.000	117,000

Results

	V [lb]	ΦR_n [lb]	Utilization [%]	Status
Anchor 1	257	83,759	1	OK
Anchor 2	234	84,642	1	OK
Anchor 3	226	87,750	1	OK

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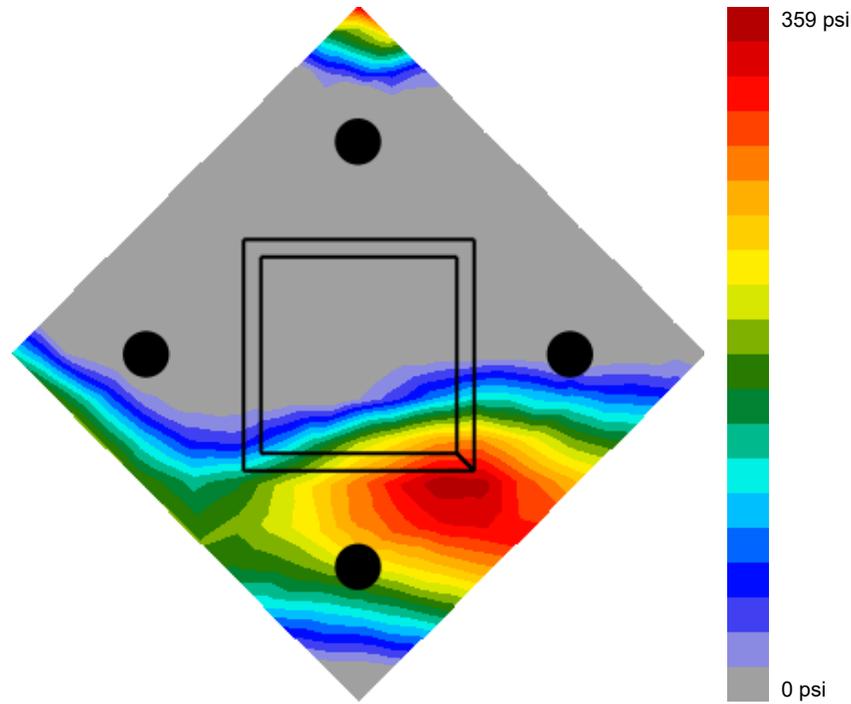
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	V [lb]	ΦR_n [lb]	Utilization [%]	Status
Anchor 4	253	87,750	1	OK

2.5 Concrete

Decisive load combination: 1 - Combination 1

2.5.1 Compression in concrete under the anchor plate



2.5.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

Equations

$$F_p = \Phi f_{p,max}$$

$$f_{p,max} = 0.85 f_c' \sqrt{\left(\frac{2}{A}\right)^2 + 1} \leq 1.7 f_c' \sqrt{\left(\frac{2}{A}\right)^2 + 1} \leq 2$$

$$\sigma = \frac{N}{A}$$

$$\text{Utilization} = \frac{\sigma}{F_p}$$

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Variables

N [lb]	f _c ' [psi]	Φ	A ₁ [in ²]	A ₂ [in ²]
8,201	4,000	0.65	46.30	104.73

Results

Load combination	F _p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	3,324	177	6	OK

2.6 Symbol explanation

A ₁	Loaded area of concrete
A ₂	Supporting area
d	Nominal diameter of the bolt
ε _{lim}	Limit plastic strain
ε _{PI}	Plastic strain from CBFEM results
f _c	Concrete compressive strength
f _c '	Concrete compressive strength
F _u	Specified minimum tensile strength of the connected material
F _p	Concrete block design bearing strength
f _{p,max}	Concrete block design bearing strength maximum
f _y	Yield strength
l _c	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
N	Resulting compression force
σ	Average stress in concrete
σ _{Ed}	Equivalent stress
Φ	Resistance factor
ΦR _n	Factored resistance
t	Thickness of the anchor plate
V	Resultant of shear forces V _y , V _z in bolt.

2.7 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.



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3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Combination 1	20%	OK
Anchor plate	Combination 1	19%	OK
Concrete	Combination 1	6%	OK
Profile	Combination 1	26%	OK

Fastening meets the design criteria!



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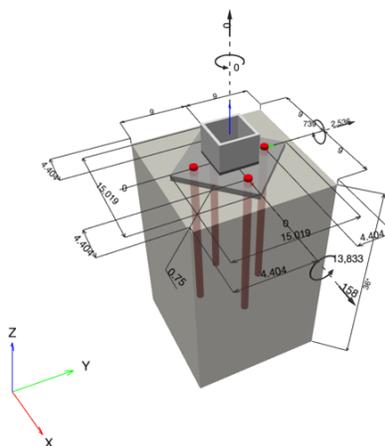
1 Anchor Design

1.1 Input data

Anchor type and diameter:	Hex Head ASTM F 1554 GR. 36 1	
Item number:	not available	
Specification text:	Hilti Hex Head headed stud anchor with 21 in embedment, 1, Steel galvanized, installation per instruction for use	
Effective embedment depth:	$h_{ef} = 21.000$ in.	
Material:	ASTM F 1554	
Evaluation Service Report:	Hilti Technical Data	
Issued Valid:	- -	
Proof:	Design Method ACI 318-19 / CIP	
Stand-off installation:	$e_b = 0.000$ in. (no stand-off); $t = 0.750$ in.	
Anchor plate ^{CBFEM} :	$l_x \times l_y \times t = 15.019$ in. \times 15.019 in. \times 0.750 in.;	
Profile:	Square HSS (AISC), HSS5X5X.375; (L x W x T) = 5.000 in. \times 5.000 in. \times 0.375 in.	
Base material:	cracked concrete, 4000, $f'_c = 4,000$ psi; $h = 36.000$ in.	
Reinforcement:	tension: present, shear: present; anchor reinforcement: tension, shear edge reinforcement: > No. 4 bar with stirrups	
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.10.5.3 (d)) Shear load: yes (17.10.6.3 (c))	

^{CBFEM} - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [in.] & Loading [lb, ft.lb]



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1.1.1 Design results

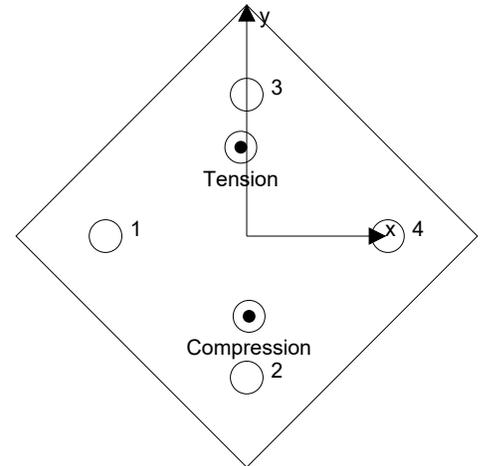
Case	Description	Forces [lb] / Moments [ft.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 0; V _x = 158; V _y = 2,536; M _x = 13,833.000; M _y = 739.000; M _z = 0.000;	yes	95

1.2 Load case/Resulting anchor forces

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	6,088	681	50	679
2	-2	605	37	604
3	18,512	576	37	574
4	4,858	680	34	679



resulting tension force in (x/y)=(-0.192/2.888): 29,456 [lb]
 resulting compression force in (x/y)=(0.077/-2.606): 29,864 [lb]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N _{ua} [lb]	Capacity ϕ N _n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	18,512	26,361	71	OK
Pullout Strength*	18,512	19,538	95	OK
Concrete Breakout Failure** ¹	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction y+**	18,512	27,034	69	OK

* highest loaded anchor **anchor group (anchors in tension)

¹ Tension Anchor Reinforcement has been selected!



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1.3.1 Steel Strength

$$N_{sa} = A_{se,N} f_{uta} \quad \text{ACI 318-19 Eq. (17.6.1.2)}$$

$$\phi N_{sa} \geq N_{ua} \quad \text{ACI 318-19 Table 17.5.2}$$

Variables

$A_{se,N}$ [in. ²]	f_{uta} [psi]
0.61	58,000

Calculations

N_{sa} [lb]
35,148

Results

N_{sa} [lb]	ϕ_{steel}	ϕN_{sa} [lb]	N_{ua} [lb]
35,148	0.750	26,361	18,512

1.3.2 Pullout Strength

$$N_{pN} = \psi_{c,p} N_p \quad \text{ACI 318-19 Eq. (17.6.3.1)}$$

$$N_p = 8 A_{brg} f'_c \quad \text{ACI 318-19 Eq. (17.6.3.2.2a)}$$

$$\phi N_{pN} \geq N_{ua} \quad \text{ACI 318-19 Table 17.5.2}$$

Variables

$\psi_{c,p}$	A_{brg} [in. ²]	λ_a	f'_c [psi]
1.000	1.16	1.000	4,000

Calculations

N_p [lb]
37,216

Results

N_{pn} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	ϕN_{pn} [lb]	N_{ua} [lb]
37,216	0.700	0.750	1.000	19,538	18,512

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1.3.3 Concrete Side-Face Blowout, direction y+

$$N_{sb} = 160 c_{a1} \sqrt{A_{brg}} \lambda_a \sqrt{f'_c} \quad \text{ACI 318-19 Eq. (17.6.4.1)}$$

$$N_{sbg} = \alpha_{group} N_{sb} \quad \text{ACI 318-19 Eq. (17.6.4.2)}$$

$$\phi N_{sbg} \geq N_{ua} \quad \text{ACI 318-19 Table 17.5.2}$$

$$\alpha_{group} = \left(1 + \frac{s}{6 c_{a1}} \right) \quad \text{see ACI 318-19, Section 17.6.4.2, Eq. (17.6.4.2)}$$

Variables

c_{a1} [in.]	c_{a2} [in.]	A_{brg} [in. ²]	λ_a	f'_c [psi]	s [in.]
4.404	9.000	1.16	1.000	4,000	-

Calculations

α_{group}	N_{sb} [lb]
1.000	48,060

Results

N_{sbg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	ϕN_{sbg} [lb]	$N_{ua,edge}$ [lb]
48,060	0.750	0.750	1.000	27,034	18,512



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1.4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_v = V_{ua} / \phi V_n$	Status
Steel Strength*	681	13,708	5	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	2,541	32,985	8	OK
Concrete edge failure in direction ** ¹	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

¹ Shear Anchor Reinforcement has been selected!

1.4.1 Steel Strength

$$V_{sa} = 0.6 A_{se,V} f_{uta} \quad \text{ACI 318-19 Eq. (17.7.1.2b)}$$

$$\phi V_{steel} \geq V_{ua} \quad \text{ACI 318-19 Table 17.5.2}$$

Variables

$A_{se,V}$ [in. ²]	f_{uta} [psi]
0.61	58,000

Calculations

V_{sa} [lb]
21,089

Results

V_{sa} [lb]	ϕ_{steel}	$\phi V_{sa,eq}$ [lb]	V_{ua} [lb]
21,089	0.650	13,708	681



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1.4.2 Pryout Strength

$$V_{cp,g} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right] \quad \text{ACI 318-19 Eq. (17.7.3.1b)}$$

$$\phi V_{cp,g} \geq V_{ua} \quad \text{ACI 318-19 Table 17.5.2}$$

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$$A_{Nc0} = 9 h_{ef}^2 \quad \text{ACI 318-19 Eq. (17.6.2.1.4)}$$

$$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e_N}{3 h_{ef}}} \right) \leq 1.0 \quad \text{ACI 318-19 Eq. (17.6.2.3.1)}$$

$$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0 \quad \text{ACI 318-19 Eq. (17.6.2.4.1b)}$$

$$\psi_{cp,N} = \text{MAX} \left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0 \quad \text{ACI 318-19 Eq. (17.6.2.6.1b)}$$

$$N_b = k_c \lambda_a \sqrt{f_c} h_{ef}^{1.5} \quad \text{ACI 318-19 Eq. (17.6.2.2.1)}$$

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	2.936	0.000	0.000	4.404
$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f_c [psi]
1.000	∞	24	1.000	4,000

Calculations

A_{Nc} [in. ²]	A_{Nc0} [in. ²]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
239.37	77.58	1.000	1.000	1.000	1.000	7,636

Results

$V_{cp,g}$ [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi V_{cp,g}$ [lb]	V_{ua} [lb]
47,121	0.700	1.000	1.000	32,985	2,541

1.5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.947	0.077	1.000	86	OK

$$\beta_{NV} = (\beta_N + \beta_V) / 1.2 \leq 1$$



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1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to <https://submittals.us.hilti.com/PROFISAnchorDesignGuide/>
- Attention! In case of compressive anchor forces a buckling check as well as the proof of the local load transfer into and within the base material (incl. punching) has to be done separately.
- "An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-19, Chapter 17, Section 17.10.5.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, the connection design (tension) shall satisfy the provisions of Section 17.10.5.3 (b), Section 17.10.5.3 (c), or Section 17.10.5.3 (d). The connection design (shear) shall satisfy the provisions of Section 17.10.6.3 (a), Section 17.10.6.3 (b), or Section 17.10.6.3 (c)."
- Section 17.10.5.3 (b) / Section 17.10.6.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. Section 17.10.5.3 (c) / Section 17.10.6.3 (b) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the anchors by a non-yielding attachment. Section 17.10.5.3 (d) / Section 17.10.6.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by ω_0 .
- The design of Anchor Reinforcement is beyond the scope of PROFIS Engineering. Refer to ACI 318-19, Section 17.5.2.1 (a) for information about Anchor Reinforcement.
- The design of Anchor Reinforcement is beyond the scope of PROFIS Engineering. Refer to ACI 318-19, Section 17.5.2.1 (b) for information about Anchor Reinforcement.
- Anchor Reinforcement has been selected as a design option, calculations should be compared with PROFIS Engineering calculations.
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

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1.7 Installation data

Profile: Square HSS (AISC), HSS5X5X.375; (L x W x T) = 5.000 in. x 5.000 in. x 0.375 in.

Hole diameter in the fixture: $d_f = 1.062$ in.

Plate thickness (input): 0.750 in.

Anchor type and diameter: Hex Head ASTM F 1554 GR. 36 1

Item number: not available

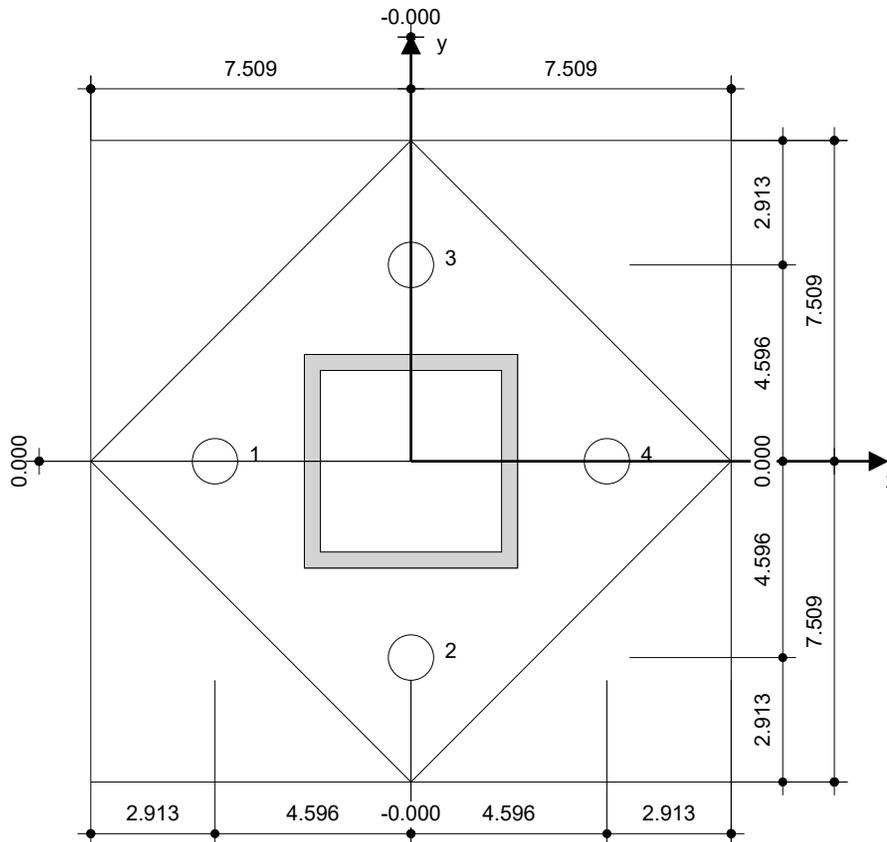
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 21.000 in.

Minimum thickness of the base material: 22.172 in.

Hilti Hex Head headed stud anchor with 21 in embedment, 1, Steel galvanized, installation per instruction for use



Coordinates Anchor [in.]

Anchor	x	y	C _{-x}	C _{+x}	C _{-y}	C _{+y}
1	-4.596	-0.000	4.404	13.596	9.000	9.000
2	0.000	-4.596	9.000	9.000	4.404	13.596
3	-0.000	4.596	9.000	9.000	13.596	4.404
4	4.596	0.000	13.596	4.404	9.000	9.000

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Diamond $I_x \times I_y \times t = 15.019 \text{ in} \times 15.019 \text{ in} \times 0.750 \text{ in}$ Calculation: CBFEM Material: ASTM A572 Gr.50; $F_y = 50,000 \text{ psi}$; $\epsilon_{lim} = 5.00\%$
Anchor type and size:	Hex Head ASTM F 1554 GR. 36 1, $h_{ef} = 21.000 \text{ in}$
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Seismic loads (cat. C, D, E or F):	Tension load: Yes (17.10.5.3 (d)) Shear load: Yes (17.10.6.3 (c))
Stand-off installation:	$e_b = 0.000 \text{ in}$ (No stand-off); $t = 0.750 \text{ in}$
Profile:	HSS5X5X.375; (L x W x T x FT) = 5.000 in x 5.000 in x 0.375 in x - Material: ASTM A500 Gr.C Rect; $F_y = 50,000 \text{ psi}$; $\epsilon_{lim} = 5.00\%$ Eccentricity x: -0.000 in Eccentricity y: 0.000 in
Base material:	Cracked concrete; 4000; $f_{c,cyl} = 4,000 \text{ psi}$; $h = 36.000 \text{ in}$
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Anchor plate			Concrete [%]
		σ_{Ed} [psi]	ϵ_{Pl} [%]	σ_{Ed} [psi]	ϵ_{Pl} [%]	Hole bearing [%]	
1	Combination 1	39,492	0.00	34,752	0.00	1	20

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Combination 1

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	2,412 lb	6,088 lb
Anchor 2	-2 lb	-2 lb
Anchor 3	17,690 lb	18,512 lb
Anchor 4	1,150 lb	4,858 lb

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

2.4 Profile/Stiffeners/Plate

Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

2.4.1 Equivalent stress and plastic strain

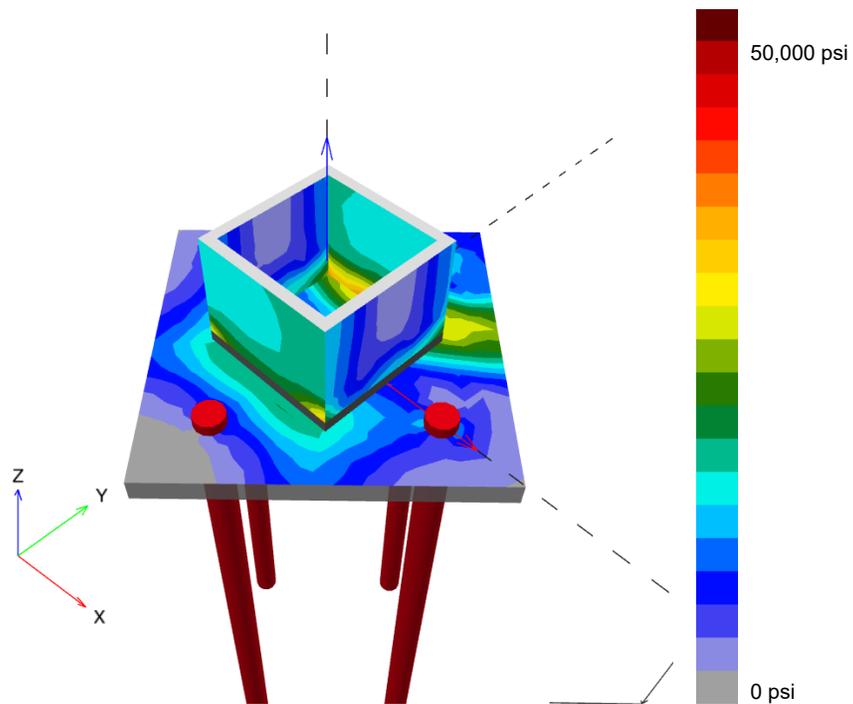
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Part	Load combination	Material	f_y [psi]	ϵ_{lim} [%]	σ_{Ed} [psi]	ϵ_{Pl} [%]	Status
Plate	Combination 1	ASTM A572 Gr.50	50,000	5.00	34,752	0.00	OK
Profile	Combination 1	ASTM A500 Gr.C Rect	50,000	5.00	37,199	0.00	OK
Profile	Combination 1	ASTM A500 Gr.C Rect	50,000	5.00	39,492	0.00	OK
Profile	Combination 1	ASTM A500 Gr.C Rect	50,000	5.00	27,921	0.00	OK
Profile	Combination 1	ASTM A500 Gr.C Rect	50,000	5.00	29,340	0.00	OK

2.4.1.1 Equivalent stress

Results below are displayed for the decisive load combination: 1 - Combination 1

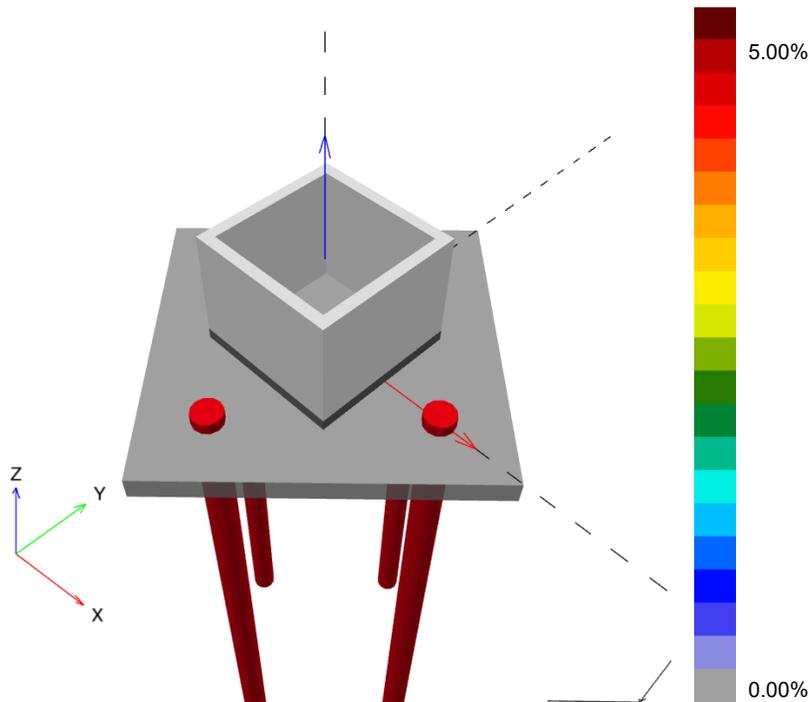


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2.4.1.2 Plastic strain

Results below are displayed for the decisive load combination: 1 - Combination 1



2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 1 - Combination 1

Equations

$$R_n = \min(1.2 l_c t F_u, 2.4 d t F_u) \quad (\text{AISC 360-16 J3-6a, c})$$

$$\Phi R_n = 0.75 R_n$$

$$V \leq \Phi R_n$$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [lb]
Anchor 1	2.189	0.750	65,000	1.000	117,000
Anchor 2	2.220	0.750	65,000	1.000	117,000
Anchor 3	8.130	0.750	65,000	1.000	117,000
Anchor 4	2.539	0.750	65,000	1.000	117,000

Results

	V [lb]	ΦR_n [lb]	Utilization [%]	Status
Anchor 1	681	87,750	1	OK
Anchor 2	605	87,750	1	OK
Anchor 3	576	87,750	1	OK

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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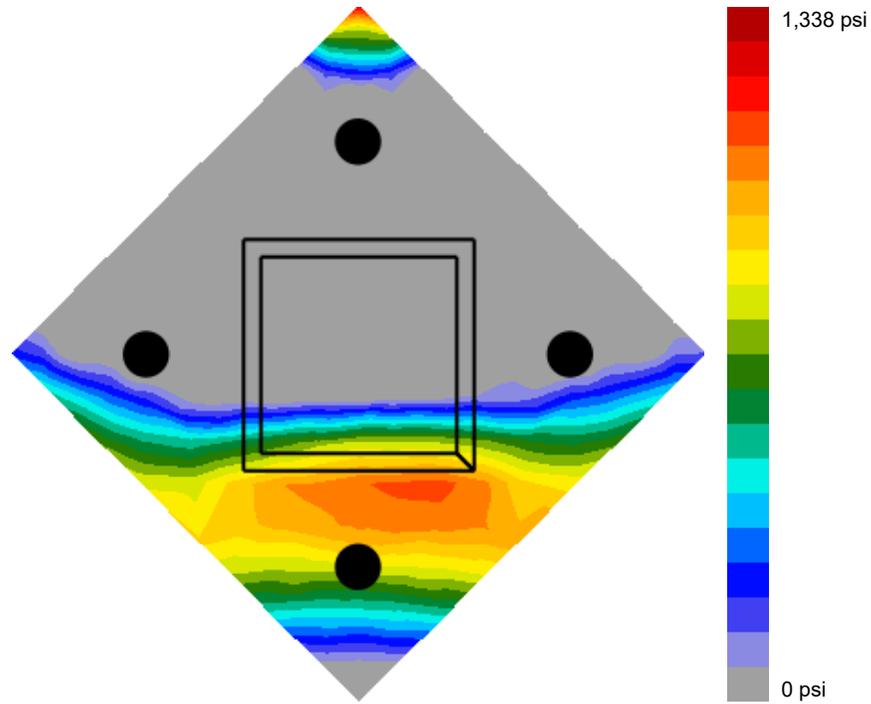
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	V [lb]	ΦR_n [lb]	Utilization [%]	Status
Anchor 4	680	87,750	1	OK

2.5 Concrete

Decisive load combination: 1 - Combination 1

2.5.1 Compression in concrete under the anchor plate



2.5.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

Equations

$$F_p = \Phi f_{p,max}$$

$$f_{p,max} = 0.85 f_c' \sqrt{\left(\frac{2}{A}\right)^2 + 1} \leq 1.7 f_c' \sqrt{\left(\frac{2}{A}\right)^2 + 1} \leq 2$$

$$\sigma = \frac{N}{A}$$

$$\text{Utilization} = \frac{\sigma}{F_p}$$

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Variables

N [lb]	f_c' [psi]	Φ	A_1 [in ²]	A_2 [in ²]
29,864	4,000	0.65	45.48	105.08

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	3,359	657	20	OK

2.6 Symbol explanation

A_1	Loaded area of concrete
A_2	Supporting area
d	Nominal diameter of the bolt
ϵ_{lim}	Limit plastic strain
ϵ_{PI}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f_c'	Concrete compressive strength
F_u	Specified minimum tensile strength of the connected material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	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
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
Φ	Resistance factor
ΦR_n	Factored resistance
t	Thickness of the anchor plate
V	Resultant of shear forces V_y, V_z in bolt.

2.7 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.



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3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Combination 1	95%	OK
Anchor plate	Combination 1	70%	OK
Concrete	Combination 1	20%	OK
Profile	Combination 1	79%	OK

Fastening meets the design criteria!



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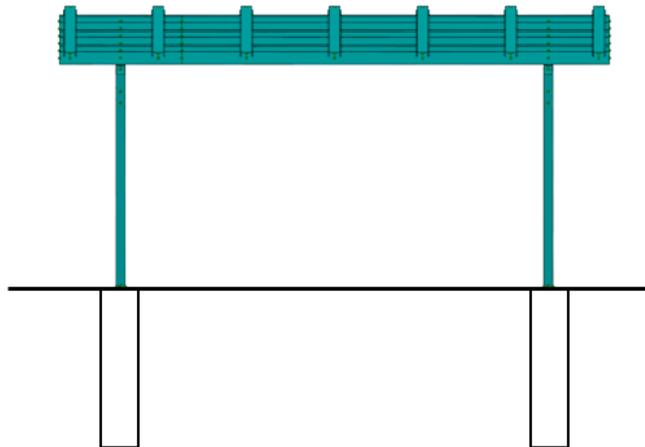
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4 Remarks; Your Cooperation Duties

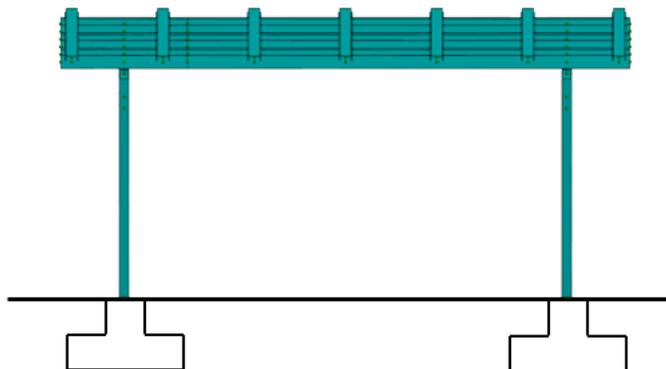
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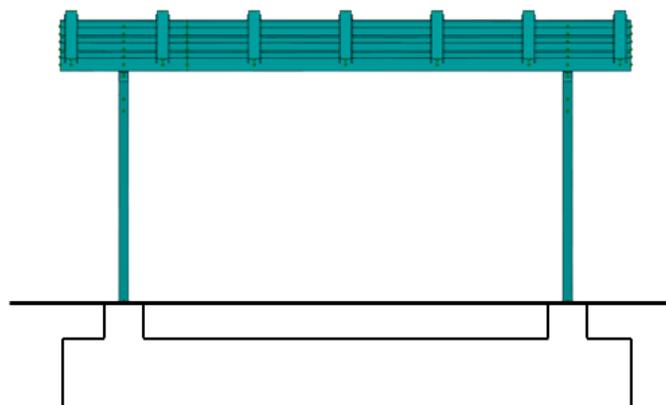
4.0 – Foundation Design and Analysis



Example Pier Elevation (Not to Scale)



Example Spread Footing Elevation (Not to Scale)



Example Grade Beam Elevation (Not to Scale)



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The Chiko Premier Carport has (3) compatible footing types: 16" cylindrical concrete piers, 12" thick square concrete spread footings with an 18" cylindrical concrete pedestal on top, and a 2 ft. wide by 2 ft. thick grade been with an 18" cylindrical concrete pedestal on top. See the following pages for the rebar requirements and details for each foundation type.



16" Dia. Cylindrical Concrete Pier Depth (ft.) per S_{DS} Value
1 Car Carport

$S_{DS} = 1.50g$		Ground Snow Load (pg)					
		10 psf	20 psf	30 psf	40 psf	50 psf	60 psf
ASCE 7-16 Wind Speed (3 sec. gust) (V)	100 mph	6.0	6.0	6.0	6.0	N/A	N/A
	110 mph	6.0	6.0	6.0	6.0	N/A	N/A
	120 mph	6.0	6.0	6.0	6.0	N/A	N/A
	130 mph	6.0	6.0	6.0	6.0	N/A	N/A
	140 mph	6.0	6.0	6.0	N/A	N/A	N/A
	150 mph	6.0	6.0	6.0	N/A	N/A	N/A

$S_{DS} = 1.25g$		Ground Snow Load (pg)					
		10 psf	20 psf	30 psf	40 psf	50 psf	60 psf
ASCE 7-16 Wind Speed (3 sec. gust) (V)	100 mph	5.5	5.5	5.5	5.5	N/A	N/A
	110 mph	5.5	5.5	5.5	5.5	N/A	N/A
	120 mph	5.5	5.5	5.5	5.5	N/A	N/A
	130 mph	5.5	5.5	5.5	5.5	N/A	N/A
	140 mph	5.5	5.5	5.5	N/A	N/A	N/A
	150 mph	5.5	5.5	5.5	N/A	N/A	N/A

$S_{DS} = 1.00g$		Ground Snow Load (pg)					
		10 psf	20 psf	30 psf	40 psf	50 psf	60 psf
ASCE 7-16 Wind Speed (3 sec. gust) (V)	100 mph	5.0	5.0	5.0	5.0	N/A	N/A
	110 mph	5.0	5.0	5.0	5.0	N/A	N/A
	120 mph	5.0	5.0	5.0	5.0	N/A	N/A
	130 mph	5.0	5.0	5.0	5.0	N/A	N/A
	140 mph	5.0	5.0	5.0	N/A	N/A	N/A
	150 mph	5.0	5.0	5.0	N/A	N/A	N/A



16" Dia. Cylindrical Concrete Pier Depth (ft.) per S_{DS} Value

1 Car Carport Continued

$S_{DS} = 0.75g$		Ground Snow Load (pg)					
		10 psf	20 psf	30 psf	40 psf	50 psf	60 psf
ASCE 7-16 Wind Speed (3 sec. gust) (V)	100 mph	5.0	5.0	5.0	5.0	N/A	N/A
	110 mph	5.0	5.0	5.0	5.0	N/A	N/A
	120 mph	5.0	5.0	5.0	5.0	N/A	N/A
	130 mph	5.0	5.0	5.0	5.0	N/A	N/A
	140 mph	5.0	5.0	5.0	N/A	N/A	N/A
	150 mph	5.0	5.0	5.0	N/A	N/A	N/A

$S_{DS} = 0.50g$		Ground Snow Load (pg)					
		10 psf	20 psf	30 psf	40 psf	50 psf	60 psf
ASCE 7-16 Wind Speed (3 sec. gust) (V)	100 mph	4.5	4.5	4.5	4.5	N/A	N/A
	110 mph	4.5	4.5	4.5	4.5	N/A	N/A
	120 mph	4.5	4.5	4.5	4.5	N/A	N/A
	130 mph	4.5	4.5	4.5	4.5	N/A	N/A
	140 mph	4.5	4.5	4.5	N/A	N/A	N/A
	150 mph	4.5	4.5	4.5	N/A	N/A	N/A



16" Dia. Cylindrical Concrete Pier Depth (ft.) per S_{DS} Value
2 Car Carport

$S_{DS} = 1.50g$		Ground Snow Load (pg)					
		10 psf	20 psf	30 psf	40 psf	50 psf	60 psf
ASCE 7-16 Wind Speed (3 sec. gust) (V)	100 mph	7.5	N/A	N/A	N/A	N/A	N/A
	110 mph	7.5	N/A	N/A	N/A	N/A	N/A
	120 mph	7.5	N/A	N/A	N/A	N/A	N/A
	130 mph	N/A	N/A	N/A	N/A	N/A	N/A
	140 mph	N/A	N/A	N/A	N/A	N/A	N/A
	150 mph	N/A	N/A	N/A	N/A	N/A	N/A

$S_{DS} = 1.25g$		Ground Snow Load (pg)					
		10 psf	20 psf	30 psf	40 psf	50 psf	60 psf
ASCE 7-16 Wind Speed (3 sec. gust) (V)	100 mph	7.0	N/A	N/A	N/A	N/A	N/A
	110 mph	7.0	N/A	N/A	N/A	N/A	N/A
	120 mph	7.0	N/A	N/A	N/A	N/A	N/A
	130 mph	N/A	N/A	N/A	N/A	N/A	N/A
	140 mph	N/A	N/A	N/A	N/A	N/A	N/A
	150 mph	N/A	N/A	N/A	N/A	N/A	N/A

$S_{DS} = 1.00g$		Ground Snow Load (pg)					
		10 psf	20 psf	30 psf	40 psf	50 psf	60 psf
ASCE 7-16 Wind Speed (3 sec. gust) (V)	100 mph	6.5	N/A	N/A	N/A	N/A	N/A
	110 mph	6.5	N/A	N/A	N/A	N/A	N/A
	120 mph	6.5	N/A	N/A	N/A	N/A	N/A
	130 mph	N/A	N/A	N/A	N/A	N/A	N/A
	140 mph	N/A	N/A	N/A	N/A	N/A	N/A
	150 mph	N/A	N/A	N/A	N/A	N/A	N/A



16" Dia. Cylindrical Concrete Pier Depth (ft.) per S_{DS} Value
2 Car Carport Continued

$S_{DS} = 0.75g$		Ground Snow Load (pg)					
		10 psf	20 psf	30 psf	40 psf	50 psf	60 psf
ASCE 7-16 Wind Speed (3 sec. gust) (V)	100 mph	6.0	N/A	N/A	N/A	N/A	N/A
	110 mph	6.5	N/A	N/A	N/A	N/A	N/A
	120 mph	6.5	N/A	N/A	N/A	N/A	N/A
	130 mph	N/A	N/A	N/A	N/A	N/A	N/A
	140 mph	N/A	N/A	N/A	N/A	N/A	N/A
	150 mph	N/A	N/A	N/A	N/A	N/A	N/A

$S_{DS} = 0.50g$		Ground Snow Load (pg)					
		10 psf	20 psf	30 psf	40 psf	50 psf	60 psf
ASCE 7-16 Wind Speed (3 sec. gust) (V)	100 mph	6.0	N/A	N/A	N/A	N/A	N/A
	110 mph	6.5	N/A	N/A	N/A	N/A	N/A
	120 mph	6.5	N/A	N/A	N/A	N/A	N/A
	130 mph	N/A	N/A	N/A	N/A	N/A	N/A
	140 mph	N/A	N/A	N/A	N/A	N/A	N/A
	150 mph	N/A	N/A	N/A	N/A	N/A	N/A



16" Dia. Cylindrical Concrete Pier Depth (ft.) per S_{DS} Value
3 Car Carport

$S_{DS} = 1.50g$		Ground Snow Load (pg)					
		10 psf	20 psf	30 psf	40 psf	50 psf	60 psf
ASCE 7-16 Wind Speed (3 sec. gust) (V)	100 mph	7.0	N/A	N/A	N/A	N/A	N/A
	110 mph	7.0	N/A	N/A	N/A	N/A	N/A
	120 mph	7.0	N/A	N/A	N/A	N/A	N/A
	130 mph	N/A	N/A	N/A	N/A	N/A	N/A
	140 mph	N/A	N/A	N/A	N/A	N/A	N/A
	150 mph	N/A	N/A	N/A	N/A	N/A	N/A

$S_{DS} = 1.25g$		Ground Snow Load (pg)					
		10 psf	20 psf	30 psf	40 psf	50 psf	60 psf
ASCE 7-16 Wind Speed (3 sec. gust) (V)	100 mph	6.5	N/A	N/A	N/A	N/A	N/A
	110 mph	6.5	N/A	N/A	N/A	N/A	N/A
	120 mph	6.5	N/A	N/A	N/A	N/A	N/A
	130 mph	N/A	N/A	N/A	N/A	N/A	N/A
	140 mph	N/A	N/A	N/A	N/A	N/A	N/A
	150 mph	N/A	N/A	N/A	N/A	N/A	N/A

$S_{DS} = 1.00g$		Ground Snow Load (pg)					
		10 psf	20 psf	30 psf	40 psf	50 psf	60 psf
ASCE 7-16 Wind Speed (3 sec. gust) (V)	100 mph	6.0	N/A	N/A	N/A	N/A	N/A
	110 mph	6.0	N/A	N/A	N/A	N/A	N/A
	120 mph	6.0	N/A	N/A	N/A	N/A	N/A
	130 mph	N/A	N/A	N/A	N/A	N/A	N/A
	140 mph	N/A	N/A	N/A	N/A	N/A	N/A
	150 mph	N/A	N/A	N/A	N/A	N/A	N/A



16" Dia. Cylindrical Concrete Pier Depth (ft.) per S_{DS} Value
3 Car Carport Continued

$S_{DS} = 0.75g$		Ground Snow Load (pg)					
		10 psf	20 psf	30 psf	40 psf	50 psf	60 psf
ASCE 7-16 Wind Speed (3 sec. gust) (V)	100 mph	5.5	N/A	N/A	N/A	N/A	N/A
	110 mph	5.5	N/A	N/A	N/A	N/A	N/A
	120 mph	5.5	N/A	N/A	N/A	N/A	N/A
	130 mph	N/A	N/A	N/A	N/A	N/A	N/A
	140 mph	N/A	N/A	N/A	N/A	N/A	N/A
	150 mph	N/A	N/A	N/A	N/A	N/A	N/A

$S_{DS} = 0.50g$		Ground Snow Load (pg)					
		10 psf	20 psf	30 psf	40 psf	50 psf	60 psf
ASCE 7-16 Wind Speed (3 sec. gust) (V)	100 mph	5.0	N/A	N/A	N/A	N/A	N/A
	110 mph	5.5	N/A	N/A	N/A	N/A	N/A
	120 mph	5.5	N/A	N/A	N/A	N/A	N/A
	130 mph	N/A	N/A	N/A	N/A	N/A	N/A
	140 mph	N/A	N/A	N/A	N/A	N/A	N/A
	150 mph	N/A	N/A	N/A	N/A	N/A	N/A



16" Dia. Cylindrical Concrete Pier Depth (ft.) per S_{DS} Value
4 Car Carport

$S_{DS} = 1.30g$		Ground Snow Load (pg)					
		10 psf	20 psf	30 psf	40 psf	50 psf	60 psf
ASCE 7-16 Wind Speed (3 sec. gust) (V)	100 mph	7.5	N/A	N/A	N/A	N/A	N/A
	110 mph	7.5	N/A	N/A	N/A	N/A	N/A
	120 mph	N/A	N/A	N/A	N/A	N/A	N/A
	130 mph	N/A	N/A	N/A	N/A	N/A	N/A
	140 mph	N/A	N/A	N/A	N/A	N/A	N/A
	150 mph	N/A	N/A	N/A	N/A	N/A	N/A

$S_{DS} = 1.25g$		Ground Snow Load (pg)					
		10 psf	20 psf	30 psf	40 psf	50 psf	60 psf
ASCE 7-16 Wind Speed (3 sec. gust) (V)	100 mph	7.5	N/A	N/A	N/A	N/A	N/A
	110 mph	7.5	N/A	N/A	N/A	N/A	N/A
	120 mph	N/A	N/A	N/A	N/A	N/A	N/A
	130 mph	N/A	N/A	N/A	N/A	N/A	N/A
	140 mph	N/A	N/A	N/A	N/A	N/A	N/A
	150 mph	N/A	N/A	N/A	N/A	N/A	N/A

$S_{DS} = 1.00g$		Ground Snow Load (pg)					
		10 psf	20 psf	30 psf	40 psf	50 psf	60 psf
ASCE 7-16 Wind Speed (3 sec. gust) (V)	100 mph	7.0	N/A	N/A	N/A	N/A	N/A
	110 mph	7.0	N/A	N/A	N/A	N/A	N/A
	120 mph	N/A	N/A	N/A	N/A	N/A	N/A
	130 mph	N/A	N/A	N/A	N/A	N/A	N/A
	140 mph	N/A	N/A	N/A	N/A	N/A	N/A
	150 mph	N/A	N/A	N/A	N/A	N/A	N/A



16" Dia. Cylindrical Concrete Pier Depth (ft.) per S_{DS} Value
4 Car Carport Continued

$S_{DS} = 0.75g$		Ground Snow Load (pg)					
		10 psf	20 psf	30 psf	40 psf	50 psf	60 psf
ASCE 7-16 Wind Speed (3 sec. gust) (V)	100 mph	6.0	N/A	N/A	N/A	N/A	N/A
	110 mph	6.0	N/A	N/A	N/A	N/A	N/A
	120 mph	N/A	N/A	N/A	N/A	N/A	N/A
	130 mph	N/A	N/A	N/A	N/A	N/A	N/A
	140 mph	N/A	N/A	N/A	N/A	N/A	N/A
	150 mph	N/A	N/A	N/A	N/A	N/A	N/A

$S_{DS} = 0.50g$		Ground Snow Load (pg)					
		10 psf	20 psf	30 psf	40 psf	50 psf	60 psf
ASCE 7-16 Wind Speed (3 sec. gust) (V)	100 mph	5.5	N/A	N/A	N/A	N/A	N/A
	110 mph	6.0	N/A	N/A	N/A	N/A	N/A
	120 mph	N/A	N/A	N/A	N/A	N/A	N/A
	130 mph	N/A	N/A	N/A	N/A	N/A	N/A
	140 mph	N/A	N/A	N/A	N/A	N/A	N/A
	150 mph	N/A	N/A	N/A	N/A	N/A	N/A



12" Thick Square Concrete Spread Footing Dimensions per Carport (LxWxD)

1 Car Carport:

$S_{DS} = 1.50g$	6.0 ft. x 6.0 ft. x 1 ft.
$S_{DS} = 1.25g$	5.5 ft. x 5.5 ft. x 1 ft.
$S_{DS} = 1.00g$	5.0 ft. x 5.0 ft. x 1 ft.
$S_{DS} = 0.75g$	4.5 ft. x 4.5 ft. x 1 ft.
$S_{DS} = 0.50g$	4.5 ft. x 4.5 ft. x 1 ft.

2 Car Carport:

$S_{DS} = 1.50g$	7.5 ft. x 7.5 ft. x 1 ft.
$S_{DS} = 1.25g$	7.0 ft. x 7.0 ft. x 1 ft.
$S_{DS} = 1.00g$	6.5 ft. x 6.5 ft. x 1 ft.
$S_{DS} = 0.75g$	6.5 ft. x 6.5 ft. x 1 ft.
$S_{DS} = 0.50g$	6.5 ft. x 6.5 ft. x 1 ft.

3 Car Carport:

$S_{DS} = 1.50g$	7.0 ft. x 7.0 ft. x 1 ft.
$S_{DS} = 1.25g$	6.5 ft. x 6.5 ft. x 1 ft.
$S_{DS} = 1.00g$	6.0 ft. x 6.0 ft. x 1 ft.
$S_{DS} = 0.75g$	5.5 ft. x 5.5 ft. x 1 ft.
$S_{DS} = 0.50g$	5.0 ft. x 5.0 ft. x 1 ft.

4 Car Carport:

$S_{DS} = 1.30g$	7.5 ft. x 7.5 ft. x 1 ft.
$S_{DS} = 1.25g$	7.0 ft. x 7.0 ft. x 1 ft.
$S_{DS} = 1.00g$	6.5 ft. x 6.5 ft. x 1 ft.
$S_{DS} = 0.75g$	6.0 ft. x 6.0 ft. x 1 ft.
$S_{DS} = 0.50g$	5.5 ft. x 5.5 ft. x 1 ft.



2 ft. Wide by 2 ft. Thick Grade Beam Length per Carport

1 Car Carport:

$S_{DS} = 1.50g$	17.0 ft.
$S_{DS} = 1.25g$	14.0 ft.
$S_{DS} = 1.00g$	11.5 ft.
$S_{DS} = 0.75g$	11.0 ft.
$S_{DS} = 0.50g$	11.0 ft.

2 Car Carport:

$S_{DS} = 1.50g$	27.0 ft.
$S_{DS} = 1.25g$	23.0 ft.
$S_{DS} = 1.00g$	20.0 ft.
$S_{DS} = 0.75g$	20.0 ft.
$S_{DS} = 0.50g$	20.0 ft.

3 Car Carport:

$S_{DS} = 1.50g$	44.0 ft.
$S_{DS} = 1.25g$	37.0 ft.
$S_{DS} = 1.00g$	30.0 ft.
$S_{DS} = 0.75g$	30.0 ft.
$S_{DS} = 0.50g$	30.0 ft.

4 Car Carport:

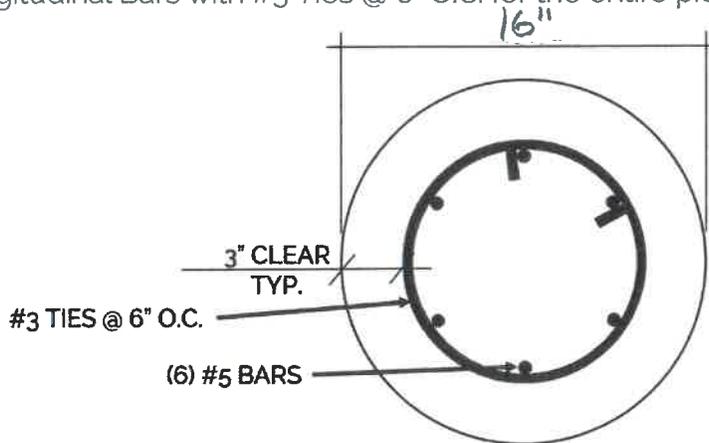
$S_{DS} = 1.30g$	45.0 ft.
$S_{DS} = 1.25g$	43.0 ft.
$S_{DS} = 1.00g$	39.0 ft.
$S_{DS} = 0.75g$	39.0 ft.
$S_{DS} = 0.50g$	39.0 ft.



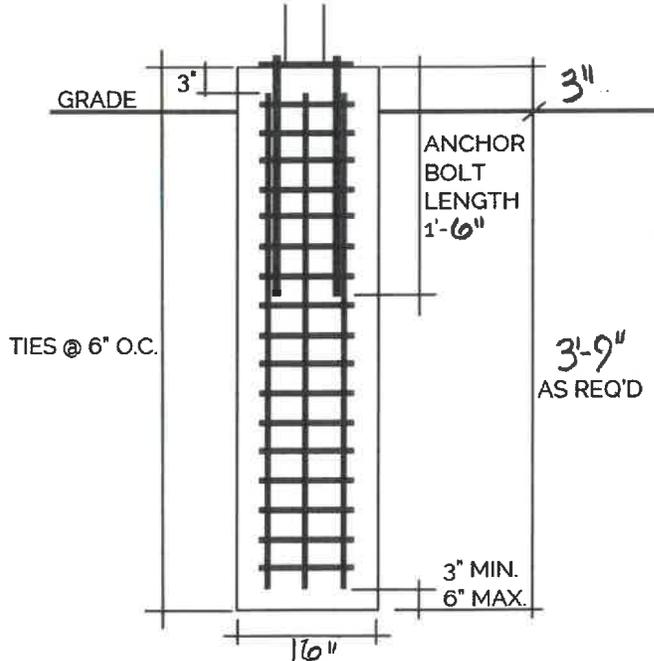
4.1 - Foundation Details

4.1.1 - Cylindrical Concrete Pier Footing Details

18" Diameter Cylindrical Concrete Pier footing Rebar is required to be (6) #5 Longitudinal Bars with #3 Ties @ 6" O.C. for the entire pier depth.



Rebar Requirements (Plan)



Rebar Requirements (Elevation)

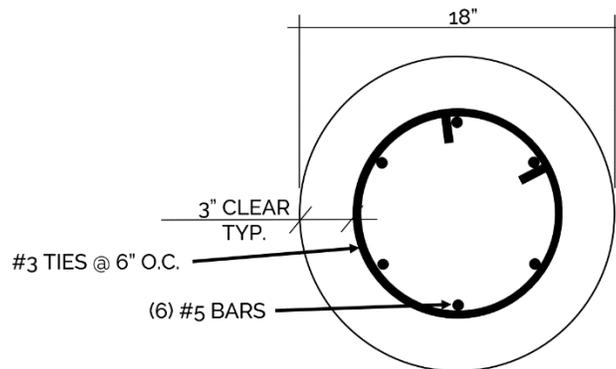


4.1.2 – Square Concrete Spread Footing Details

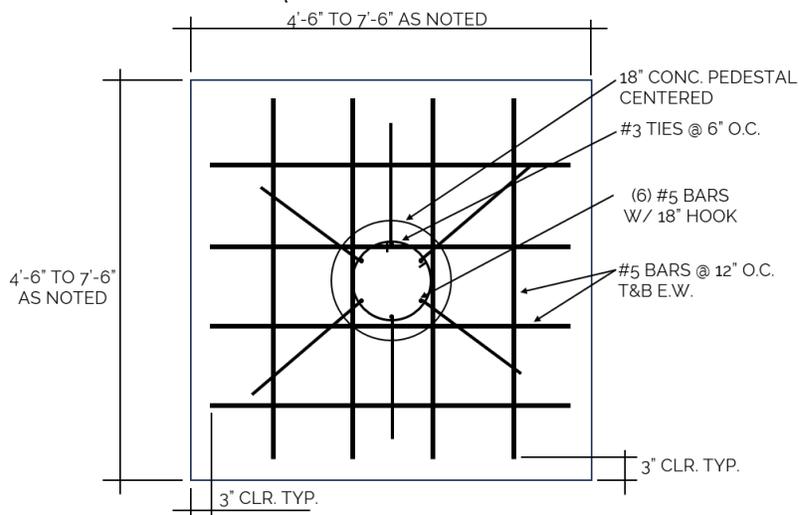
12" Thick Square Concrete Spread Footings with an 18" Cylindrical Concrete Pedestal on top Rebar is required to be:

In Pedestal: (6) #5 Longitudinal Bars with #3 Ties @ 6" O.C. for the entire pedestal depth.

In Spread Footing: #5 Bars @ 12" O.C. Each Way, Top and Bottom



Rebar Requirements in Pedestal (Plan)



Rebar Requirements in Spread Footing (Plan)

Anchor bolts not shown for clarity. Anchor bolts are to be installed in a similar manner to the 18" Cylindrical Concrete Pier.

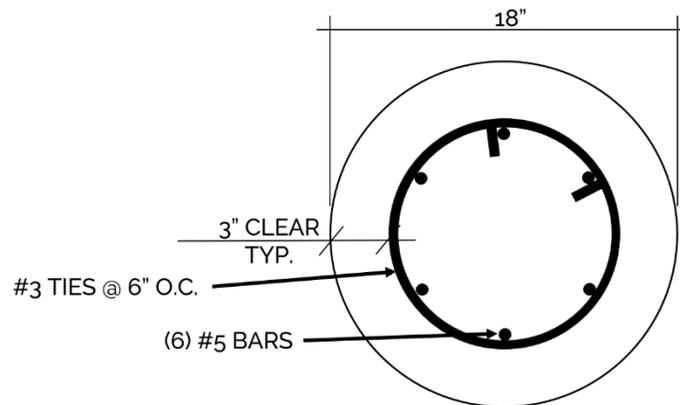


4.1.3 – Rectangular Concrete Grade Beam Footing Details

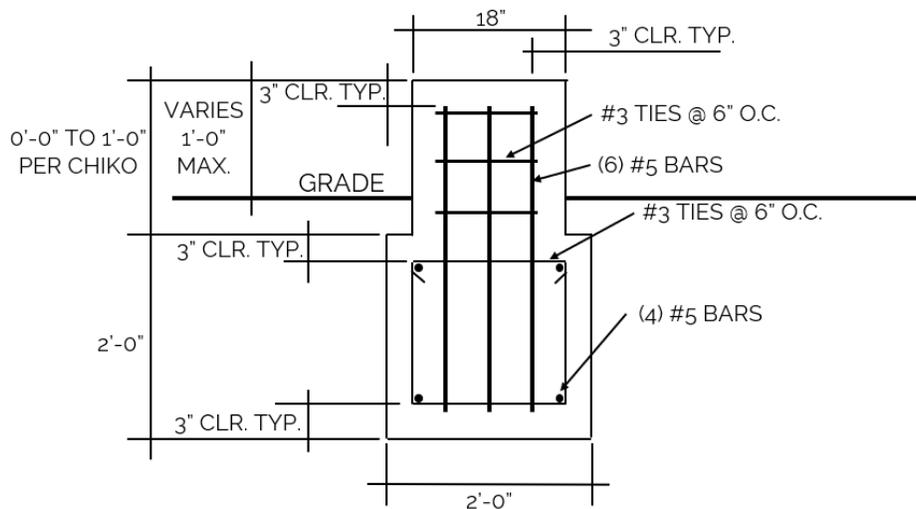
2 ft. Wide by 2 ft. Thick Grade Beam Rebar is required to be:

In Pedestal: (6) #5 Longitudinal Bars with #3 Ties @ 6" O.C. for the entire pedestal depth.

In Grade Beam: (2) #5 Bars Top and Bottom with #3 Ties @ 12" O.C.

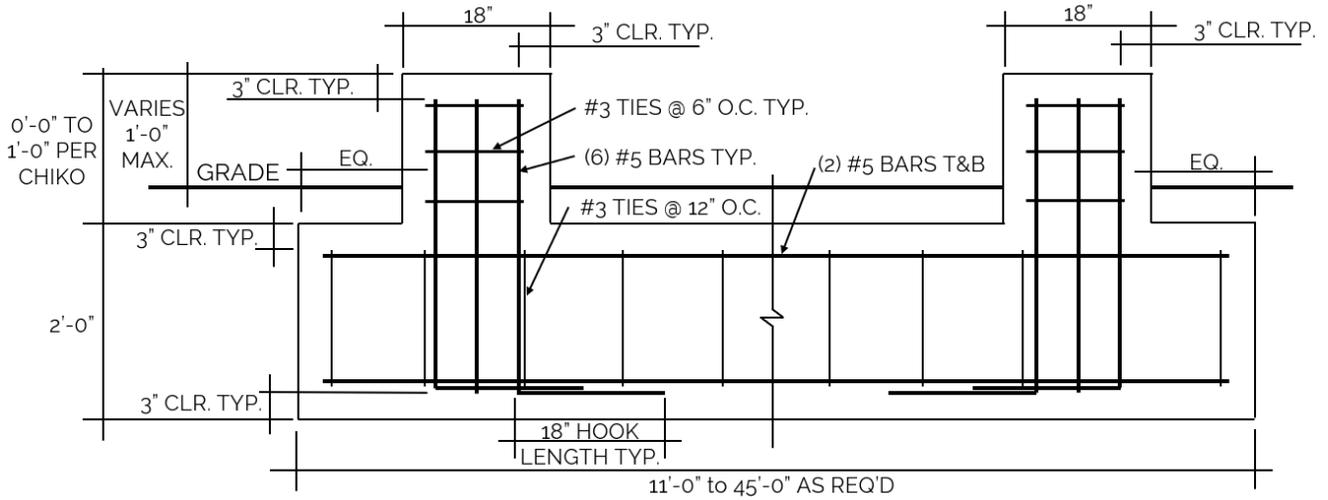


Rebar Requirements in Pedestal (Plan)



Rebar Requirements in Grade Beam (Elevation)

Anchor bolts not shown for clarity. Anchor bolts are to be installed in a similar manner to the 18" Cylindrical Concrete Pier.



Rebar Requirements in Grade Beam (Elevation)

Anchor bolts not shown for clarity. Anchor bolts are to be installed in a similar manner to the 18" Cylindrical Concrete Pier.



The allowable forces listed on the previous RISA printout pages (reaction loads) represent an example of the worst-case foundation forces imposed on the concrete piers under the example loading scenario. While this analysis only shows one carport size and loading scenario, EPS has analyzed all prescribed carport sizes and loading scenarios in a similar manner.

The following load cases and array geometry have been determined to be the governing design loads for the example concrete pier design.

Pier Design Loads are taken at baseplate.

- Pier Design Loads (lbs.):
 - Down: 3,215 lbs.
 - Up: -1,054 lbs.
 - Lateral: 847 lbs.
 - Moment: 4,687 lbs.-ft.

Envelope Node Reactions

	Node Label		X [lb]	LC	Y [lb]	LC	Z [lb]	LC	MX [lb-ft]	LC	MY [lb-ft]	LC	MZ [lb-ft]	LC
1	N146	max	104.859	7	3214.281	11	188.254	24	566.406	24	84.683	30	3063.12	31
2		min	-659.597	31	-1054.449	24	-576.123	30	-3487.114	28	-21.839	24	-550.844	9
3	N148	max	109.068	7	3092.267	13	206.892	26	538.749	26	42.888	26	2930.377	31
4		min	-600.728	31	-950.318	26	-664.521	30	-3864.111	28	-137.702	13	-540.491	7
5	N140	max	96.645	25	3090.772	11	521.886	11	2224.999	11	98.339	32	3116.006	27
6		min	-670.72	27	-1007.865	24	-712.513	32	-4276.792	32	-69.049	13	-517.396	9
7	N143	max	89.979	25	3012.53	13	614.113	13	2397.686	13	73.277	13	2979.745	27
8		min	-609.544	27	-910.834	26	-846.6	32	-4686.907	32	-67.918	32	-494.146	9
9	N287	max	NC		NC		NC		LOCKED		LOCKED		NC	
10		min	NC		NC		NC		LOCKED		LOCKED		NC	
11	N286	max	NC		NC		NC		LOCKED		LOCKED		NC	
12		min	NC		NC		NC		LOCKED		LOCKED		NC	
13	N285	max	NC		NC		NC		LOCKED		LOCKED		NC	
14		min	NC		NC		NC		LOCKED		LOCKED		NC	
15	N288	max	NC		NC		NC		LOCKED		LOCKED		NC	
16		min	NC		NC		NC		LOCKED		LOCKED		NC	
17	N289	max	NC		NC		NC		LOCKED		LOCKED		NC	
18		min	NC		NC		NC		LOCKED		LOCKED		NC	
19	N290	max	NC		NC		NC		LOCKED		LOCKED		NC	
20		min	NC		NC		NC		LOCKED		LOCKED		NC	
21	Totals:	max	381.105	7	10889.824	13	0	11						
22		min	-2538.412	31	-1669.649	26	-2538.412	32						



Depth Required Due to Vertical Loads:

As the location of installation for these Chiko Premier Carports will be variable, EPS has used the code minimum soil values in the IBC for determining the soil capacity.

Presumptive Load-Bearing Values used for Class 5 Soils from IBC Table 1806.2:

- Bearing: 1500 psf
- Lateral Passive: 100 psf/ft (doubled to 200 psf/ft per code deflection allowance section 1806.3.4)
- Skin Friction: 250 psf (divided by 2 for uplift)

Soil conditions are to be field verified (by others). The top 1.0 ft. of soil has been conservatively ignored for skin friction.

- **Compression Design:**
 - Compression Resistance: $4.71 \text{ sf} \cdot 250 \text{ psf} \cdot 6.5 \text{ ft.} = 7,653 \text{ lbs.}$
→ 7,653 lbs. \geq 3,215 lbs. → OK
- **Uplift Design:**
 - Uplift Resistance: $4.71 \text{ sf} \cdot 125 \text{ psf} \cdot 6.5 \text{ ft.} = 3,826 \text{ lbs.}$
→ 3,826 lbs. \geq 1,054 lbs. → OK



- Depth Required Due to Lateral Loads:

Calculations Per IBC Section 1805.7.2.1, Eq. 18-1: Nonconstrained Pile/Pier

$$d = 0.5A[1+(1+(4.36h/A))^{1/2}]$$

$$A = 2.34 P/S_1b$$

$$b = 1.5 \text{ ft.}$$

$$S_1 = [200 \text{ psf} * (6.5 \text{ ft.} / 3)] = 433.33 \text{ psf}$$

$$P = 847 \text{ lbs.}$$

$$A = 2.34 * (847 \text{ lbs.} / (433.33 \text{ psf} * 1.5 \text{ ft.})) = 3.05 \text{ ft.}$$

$$h = 5.53 \text{ ft. (Corresponding height for the moment)}$$

$$d = 0.5 * 3.05 \text{ ft.} * [1+(1+((4.36 * 5.53 \text{ ft.}) / 3.05 \text{ ft.}))^{1/2}] = 6.08 \text{ ft.}$$

$$\rightarrow 6.08 \text{ ft.} \geq 6.5 \text{ ft.} \rightarrow \text{OK}$$

Therefore, use an 18" diameter concrete pier with an embedment depth of 7.5 ft. min. for the example loading scenario.

*EPS does not specialize in corrosion engineering and therefore the corrosion protection requirements of the steel posts, driven piles, and all racking elements/components are not included in EPS's analysis and is the responsibility of others.

Calculations for Spread Footings and Grade Beams can be presented upon request.



4.1.4 – Alternative Foundation Loads

In lieu of the foundation options presented in this packet, alternative foundations (ground screws, helical piles, etc.) may be designed by others using the following loads. The loads presented below have been factored using the ASD load combinations in ASCE 7-16 Section 2.4.1 and Section 2.4.5 and have not utilized any factor of safety. The lateral and moment loads are to be assumed to act in any direction. These loads occur at the base of each carport column. The use and design of these potential alternative foundations is by others and is not within EPS's scope.



1 Car Carport

1.5 Sds	Downward	3,100 lbs.
	Upward	1,200 lbs.
	Lateral	500 lbs.
	Moment	2,300 lbs.-ft.

1.25 Sds	Downward	3,100 lbs.
	Upward	1,200 lbs.
	Lateral	500 lbs.
	Moment	1,900 lbs.-ft.

1.00 Sds	Downward	3,100 lbs.
	Upward	1,200 lbs.
	Lateral	400 lbs.
	Moment	1,600 lbs.-ft.

0.75 Sds	Downward	3,100 lbs.
	Upward	1,200 lbs.
	Lateral	300 lbs.
	Moment	1,200 lbs.-ft.

0.50 Sds	Downward	3,100 lbs.
	Upward	1,200 lbs.
	Lateral	300 lbs.
	Moment	900 lbs.-ft.



2 Car Carport

1.5 Sds	Downward	3,600 lbs.
	Upward	1,400 lbs.
	Lateral	900 lbs.
	Moment	4,800 lbs.-ft.

1.25 Sds	Downward	3,600 lbs.
	Upward	1,400 lbs.
	Lateral	800 lbs.
	Moment	4,000 lbs.-ft.

1.00 Sds	Downward	3,600 lbs.
	Upward	1,400 lbs.
	Lateral	800 lbs.
	Moment	3,100 lbs.-ft.

0.75 Sds	Downward	3,600 lbs.
	Upward	1,400 lbs.
	Lateral	800 lbs.
	Moment	2,700 lbs.-ft.

0.50 Sds	Downward	3,600 lbs.
	Upward	1,400 lbs.
	Lateral	800 lbs.
	Moment	2,700 lbs.-ft.



3 Car Carport

1.5 S _{DG}	Downward	5,200 lbs.
	Upward	2,100 lbs.
	Lateral	800 lbs.
	Moment	3,800 lbs.-ft.

1.25 S _{DG}	Downward	5,200 lbs.
	Upward	2,100 lbs.
	Lateral	700 lbs.
	Moment	3,100 lbs.-ft.

1.00 S _{DG}	Downward	5,200 lbs.
	Upward	2,100 lbs.
	Lateral	600 lbs.
	Moment	2,500 lbs.-ft.

0.75 S _{DG}	Downward	5,200 lbs.
	Upward	2,100 lbs.
	Lateral	500 lbs.
	Moment	2,000 lbs.-ft.

0.50 S _{DG}	Downward	5,200 lbs.
	Upward	2,100 lbs.
	Lateral	400 lbs.
	Moment	1,400 lbs.-ft.



4 Car Carport

1.3 Sds	Downward	5,900 lbs.
	Upward	2,300 lbs.
	Lateral	900 lbs.
	Moment	4,500 lbs.-ft.

1.25 Sds	Downward	5,900 lbs.
	Upward	2,300 lbs.
	Lateral	900 lbs.
	Moment	4,400 lbs.-ft.

1.00 Sds	Downward	5,900 lbs.
	Upward	2,300 lbs.
	Lateral	800 lbs.
	Moment	3,500 lbs.-ft.

0.75 Sds	Downward	5,900 lbs.
	Upward	2,300 lbs.
	Lateral	600 lbs.
	Moment	2,700 lbs.-ft.

0.50 Sds	Downward	5,900 lbs.
	Upward	2,300 lbs.
	Lateral	500 lbs.
	Moment	1,900 lbs.-ft.



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APPENDIX

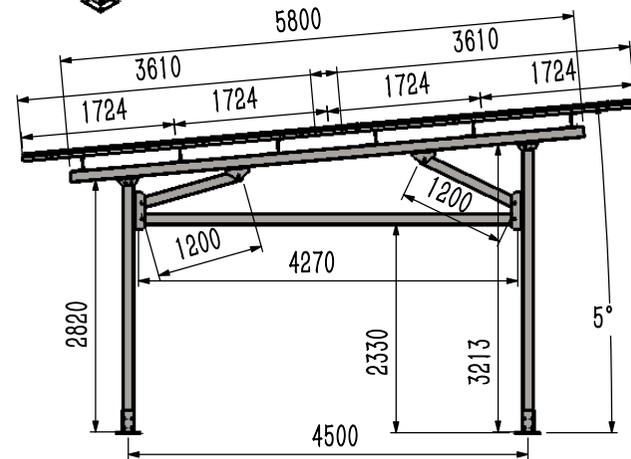
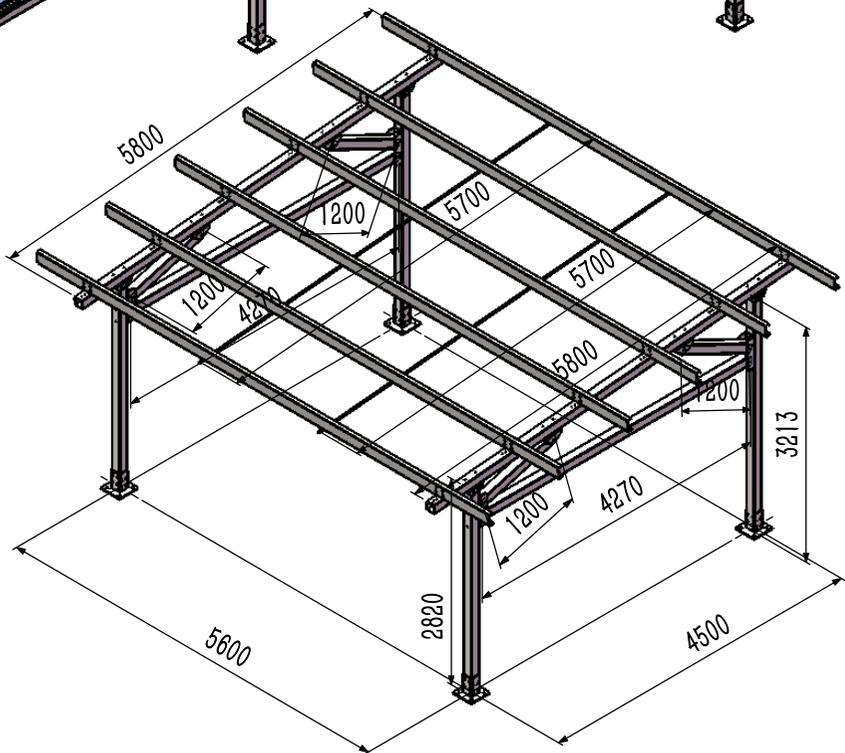
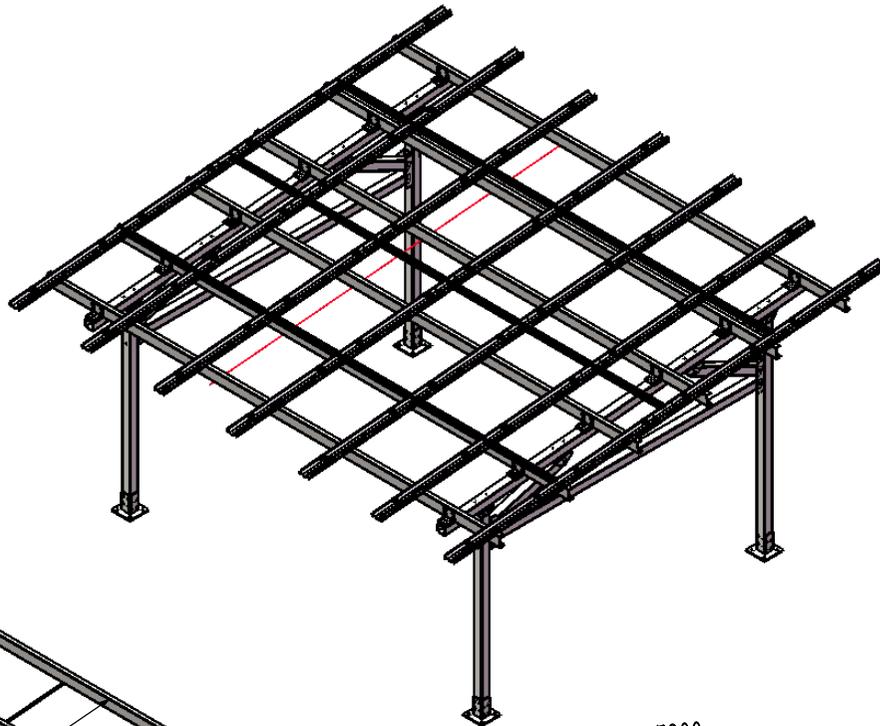
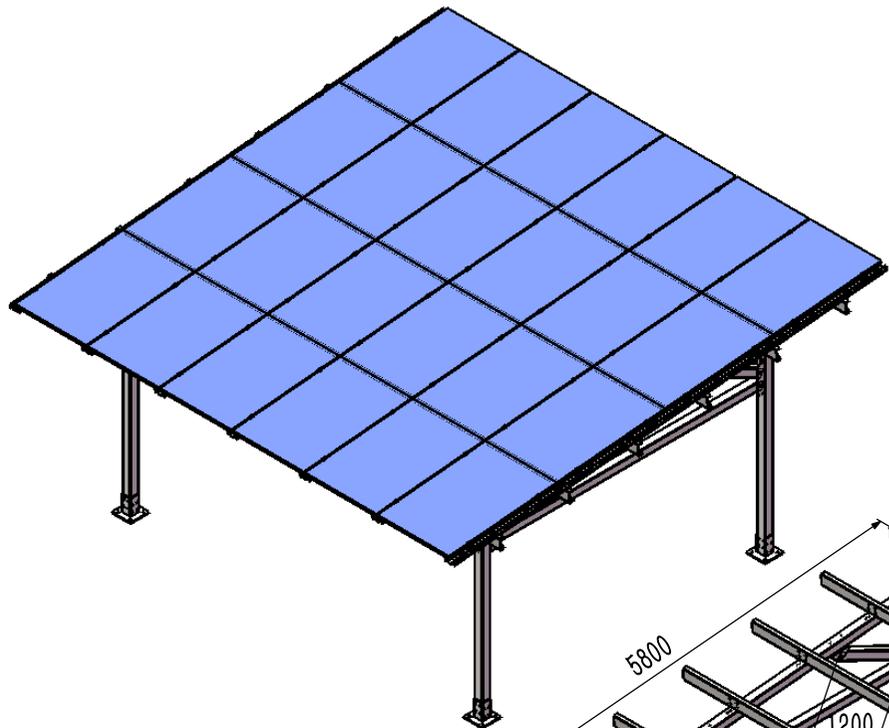
(For Reference Only)

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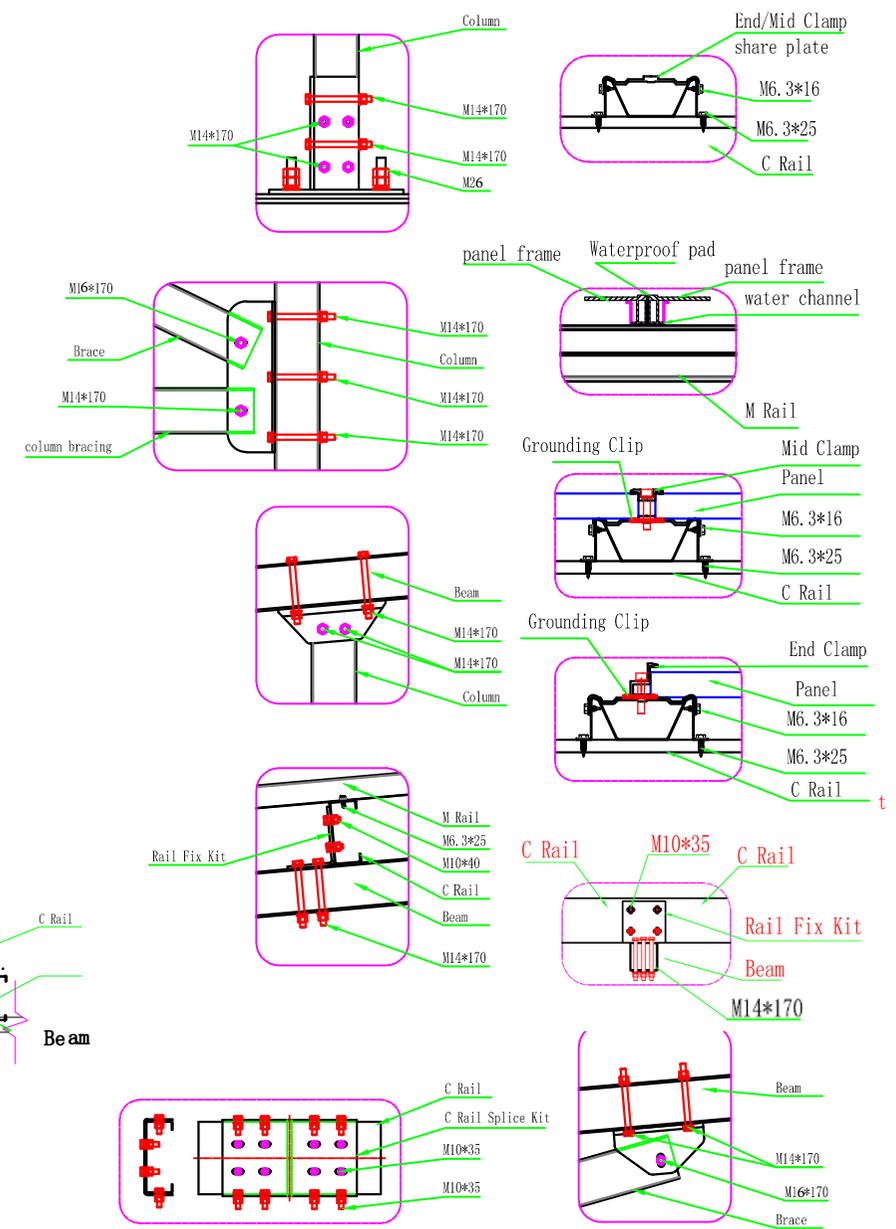
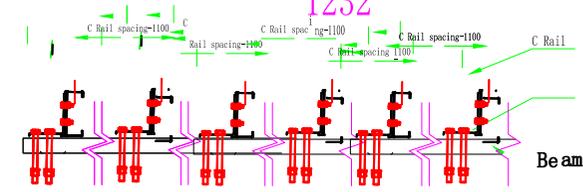
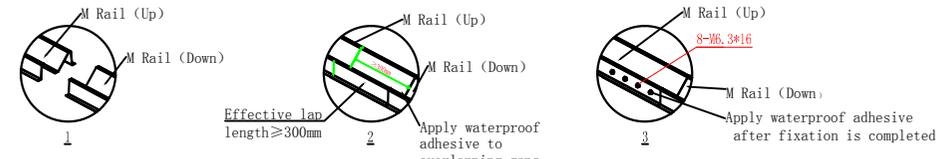
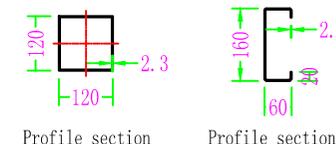
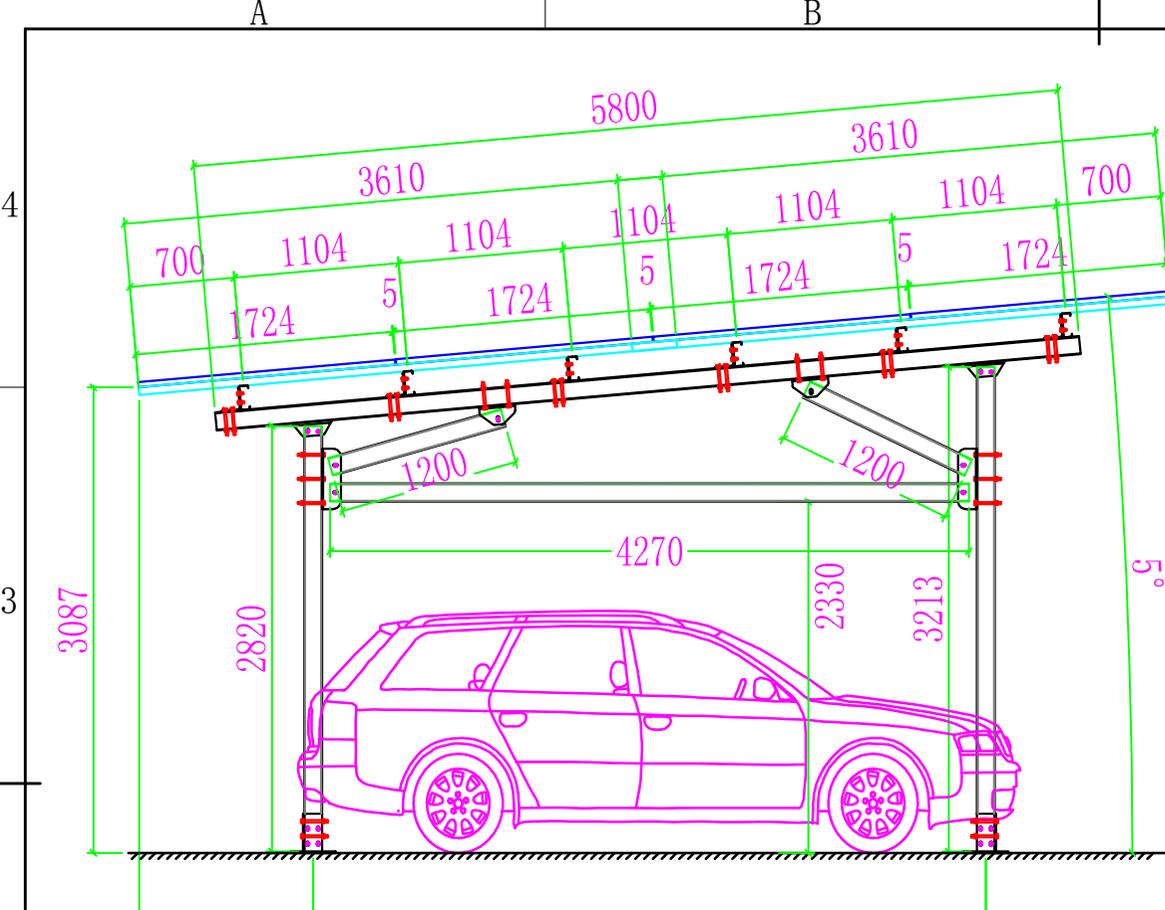
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Craft			4x6-5° -2500	
App			View: 	Rate
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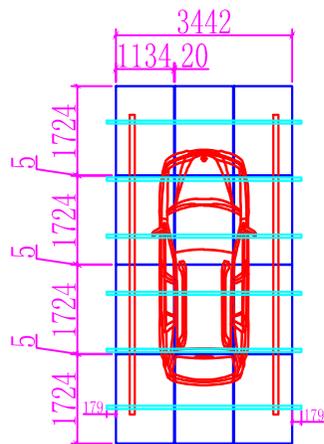
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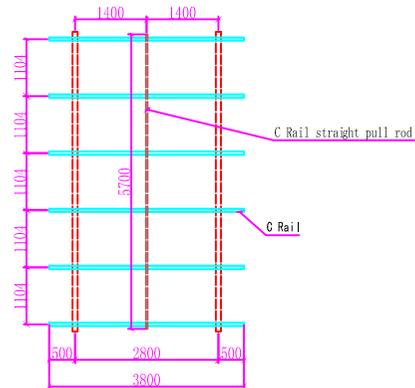
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 Front Clearance: 2500mm
 Wind Speed: --
 Snow Load: --

Modify by	Doc No.	Data

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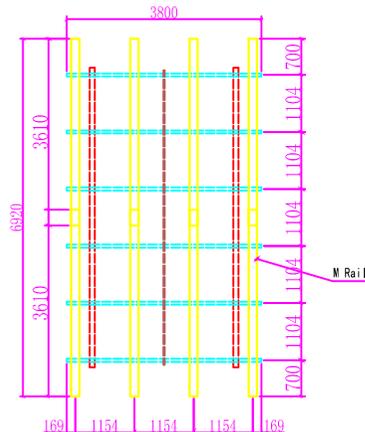
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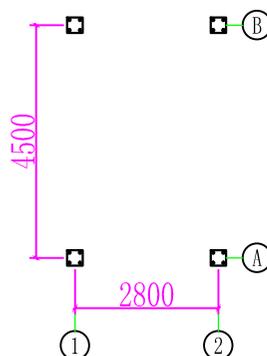
C Rail layout

NO.	Description	QTY
1	Support Rack	2
2	End Clamp	16
3	End Clamp share plate	16
4	Mid Clamp	16
5	Mid Clamp share plate	16
6	C Rail fix kit	12
7	C Rail-3800	6
8	C Rail Splice Kit	8
9	M Rail-3610	8
10	C Rail straight pull rod-5700	1
11	Water channel-1062	9
12	Waterproof pad-1134	9
13	M6.3*16 (M Rail)	32
14	M6.3*16 (End & Mid Clamp share plate)	64
15	M6.3*25 (M Rail & C Rail)	48

layout:4X3



C Rail & M Rail layout



Foundation layout

Design Factors:
 Module Dimensions: 1724*1134*35
 Tilted Angle:5°
 Front Clearance:2500mm
 Wind Speed:--
 Snow Load:--

Modify by	Doc No.	Data

Des	Bowen	231130
Drw		
Check		
Craft		
App		

Shanghai CHIKO Solar Technology Co., Ltd

Support Rack

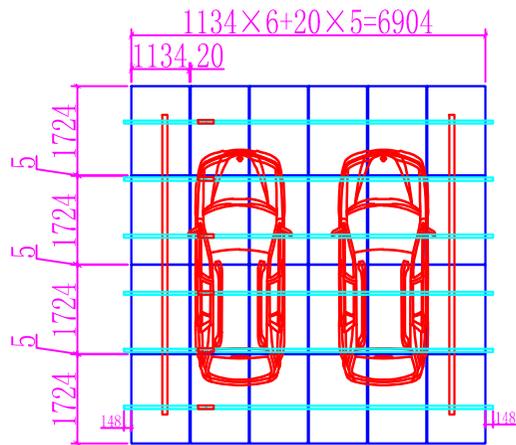
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View

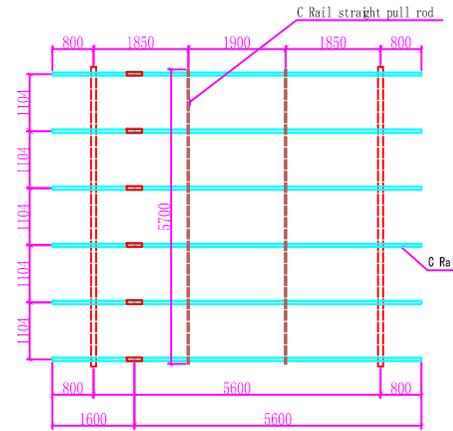


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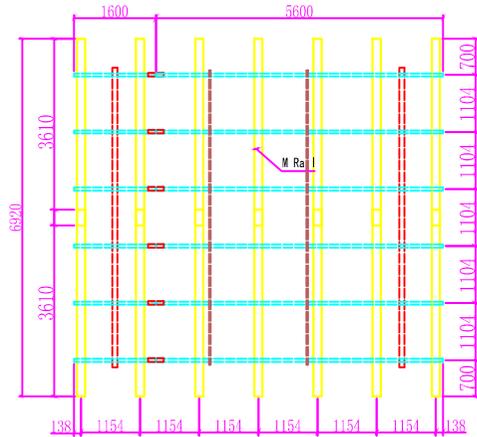
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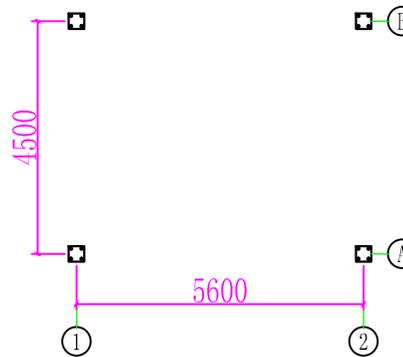
C Rail layout

NO.	Description	QTY
1	Support Rack	2
2	End Clamp	16
3	End Clamp share plate	16
4	Mid Clamp	40
5	Mid Clamp share plate	40
6	C Rail fix kit	12
7	C Rail-1600	6
8	C Rail-5600	6
9	C Rail Splice Kit	6
10	M Rail-3610	14
11	C Rail straight pull rod-5700	2
12	Column bracing fix kit	2
13	Water channel-1062	18
14	Waterproof pad-1134	18
15	M6.3*16 (M Rail)	56
16	M6.3*16(End & Mid Clamp share plate)	112
17	M6.3*25(M Rail & C Rail)	84

layout: 4X6



C Rail & M Rail layout



Foundation layout

Design Factors:
 Module Dimensions: 1724*1134*35
 Tilted Angle: 5°
 Front Clearance: 2500mm
 Wind Speed: --
 Snow Load: --

Modify by	Doc No.	Data

Des	Bowen	231130
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Shanghai CHIKO Solar Technology Co., Ltd

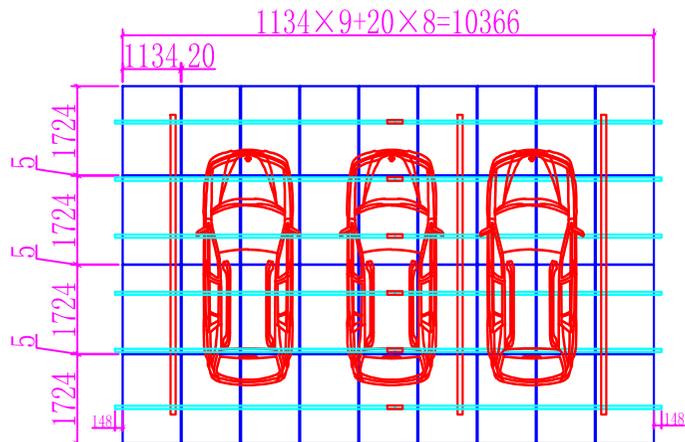
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4x6-2500-5°

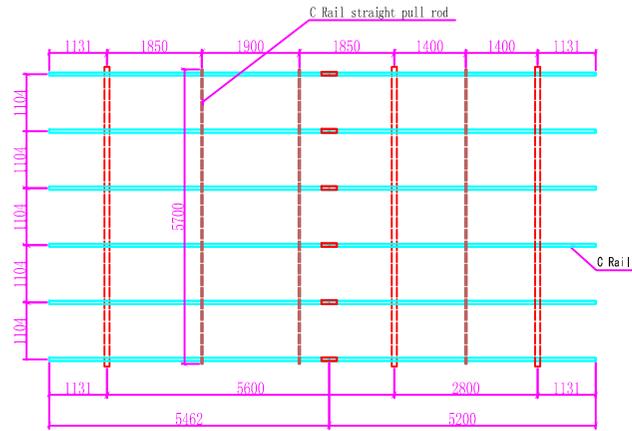


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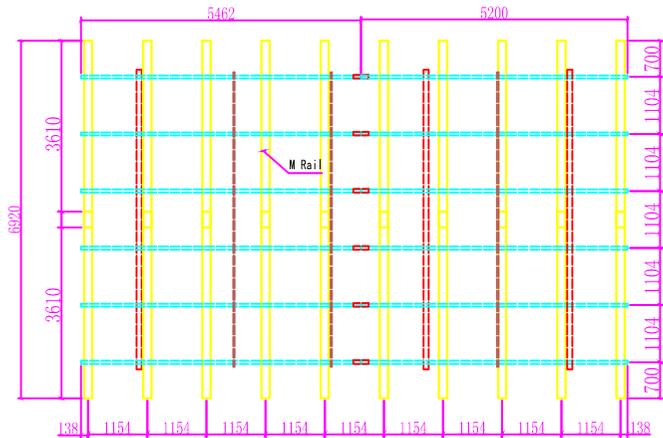
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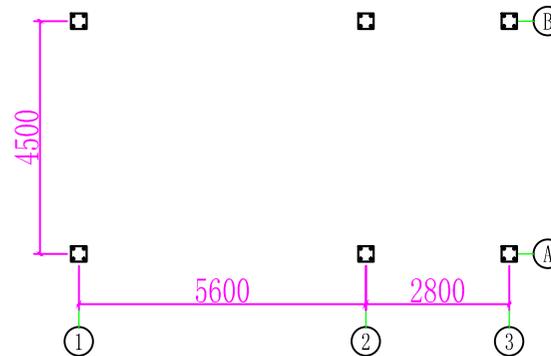
C Rail layout

NO.	Description	QTY
1	Support Rack	3
2	End Clamp	16
3	End Clamp share plate	16
4	Mid Clamp	64
5	Mid Clamp share plate	64
6	C Rail fix kit	18
7	C Rail-5462	6
8	C Rail-5200	6
9	C Rail Splice Kit	6
10	M Rail-3610	20
11	C Rail straight pull rod-5700	3
12	Water channel-1062	27
13	Waterproof pad-1134	27
14	M6.3*16 (M Rail)	80
15	M6.3*16(End & Mid Clamp share plate)	160
16	M6.3*25(M Rail & C Rail)	120

layout: 4X9



C Rail & M Rail layout



Foundation layout

Design Factors:
 Module Dimensions: 1724*1134*35
 Tilted Angle: 5°
 Front Clearance: 2500mm
 Wind Speed: --
 Snow Load: --

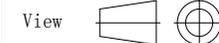
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Des	Bowen	231130
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Shanghai CHIKO Solar Technology Co., Ltd

Support Rack

4x9-2500-5°



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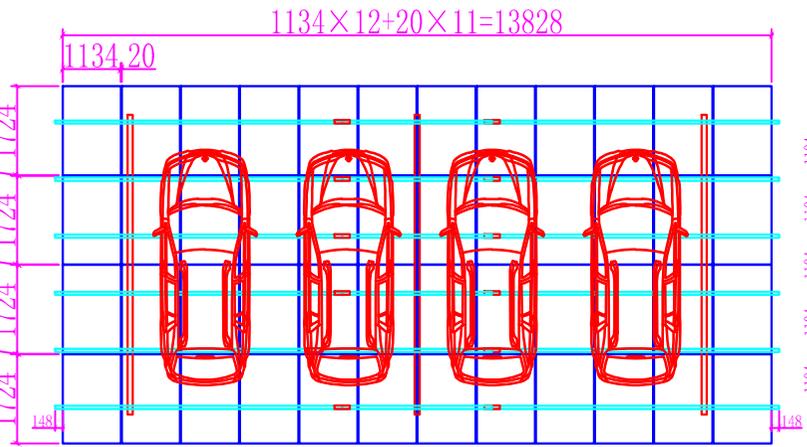
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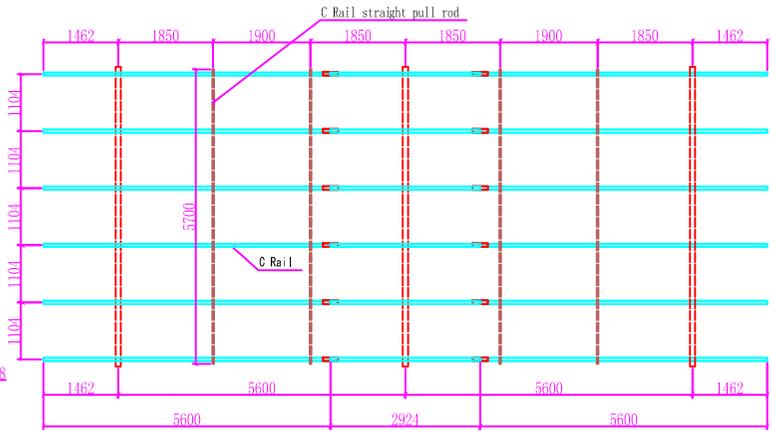
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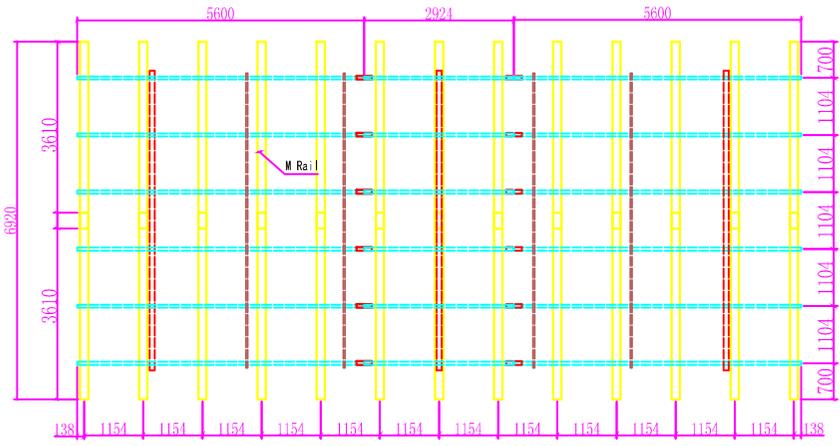
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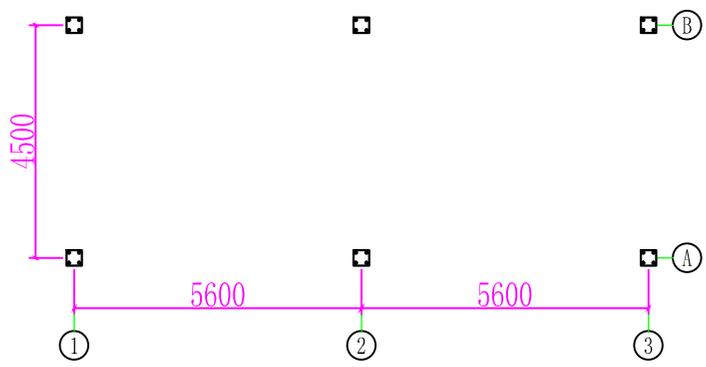
C Rail layout

NO.	Description	QTY
1	Support Rack	3
2	End Clamp	16
3	End Clamp share plate	16
4	Mid Clamp	88
5	Mid Clamp share plate	88
6	C Rail fix kit	18
7	C Rail-5600	6
8	C Rail-2924	6
9	C Rail-5600	6
10	C Rail Splice Kit	12
11	M Rail-3610	26
12	C Rail straight pull rod-5700	4
13	Water channel-1062	36
14	Waterproof pad-1134	36
15	M6.3*16 (M Rail)	104
16	M6.3*16 (End & Mid Clamp share plate)	160
17	M6.3*25 (M Rail & C Rail)	208

layout: 4X12



C Rail & M Rail layout



Foundation layout

Design Factors:
 Module Dimensions: 1724*1134*35
 Tilted Angle: 5°
 Front Clearance: 2500mm
 Wind Speed: --
 Snow Load: --

Modify by	Doc No.	Data

Des	Bowen	231130
Drw		
Check		
Craft		
App		

Shanghai CHIKO Solar Technology Co., Ltd

Support Rack
4x12-2500-5°

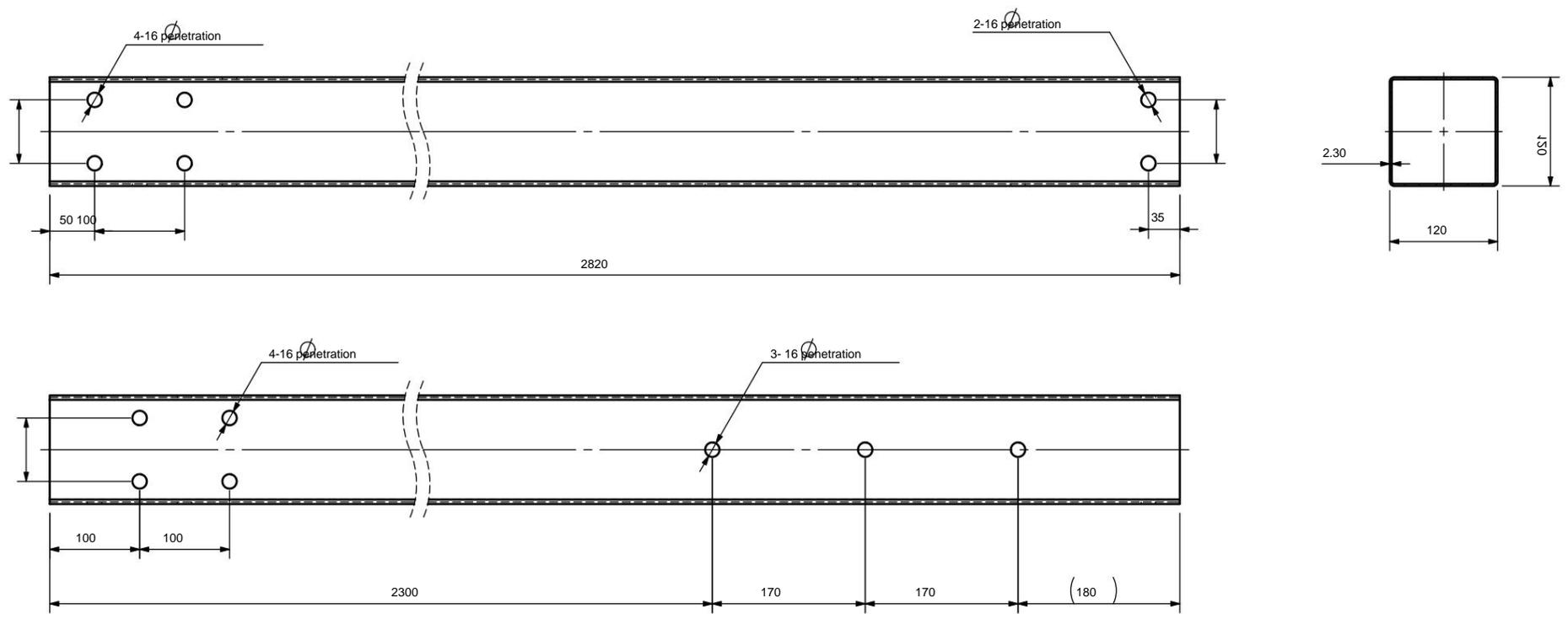
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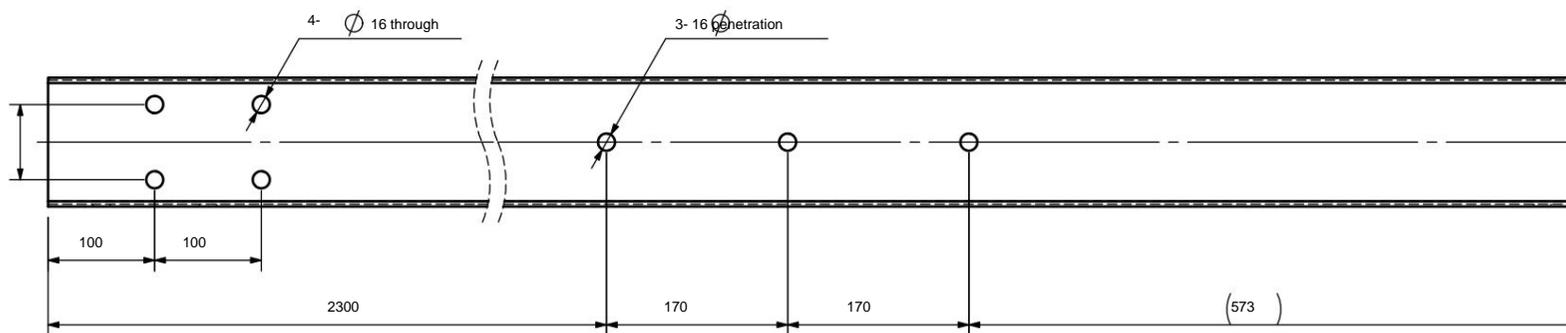
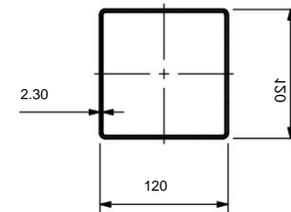
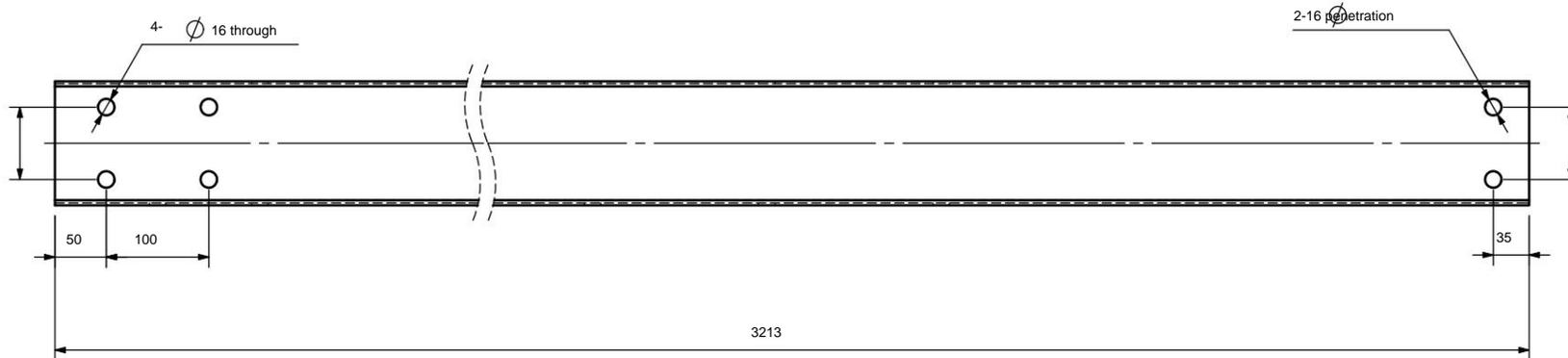
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skills requirement:

1. The weight of double-sided coating of continuous heat zinc, aluminum and magnesium is $\geq 275g/m^2$.
2. Product size, shape, weight and allowable deviations are in accordance with GB/T25052-2010.
3. The surface is clean and free of burrs, bruises, etc.
4. Unmarked dimensions refer to GB/T1804-2000m level.
5. Spray black paint, the paint film will be smooth and smooth, there will be no significant difference in color, and there will be no peeling, bruises, etc. on the surface.

Mark.mark	Modification.Change	Modify by. Modifier Doc	NO. File number Data	Date	Des.Design	 Shanghai Chenke Solar Technology Co., Ltd.	
					Drw.Drawing		Bowen231124
					Check. review		
					Craft. Craftsmanship		
					App. Approval		
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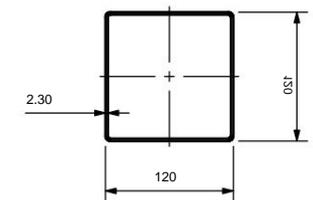
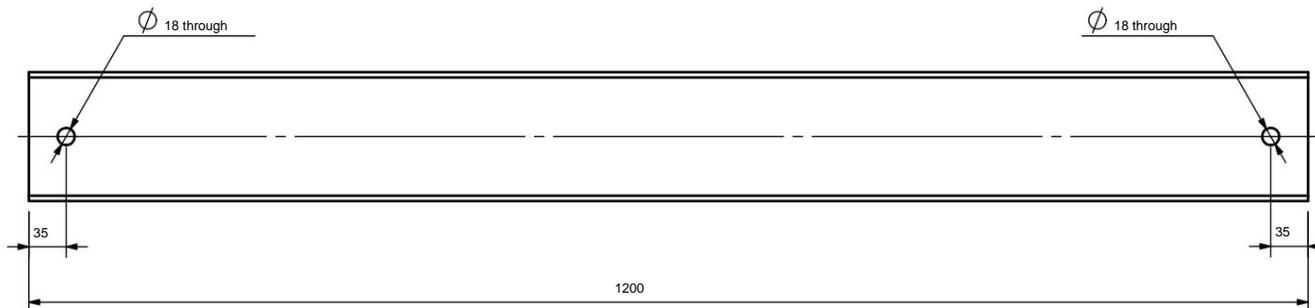


skills requirement:

1. The weight of double-sided coating of continuous heat zinc, aluminum and magnesium is $\geq 275\text{g/m}^2$.
2. Product size, shape, weight and allowable deviations are in accordance with GB/T25052-2010.
3. The surface is clean and free of burrs, bruises, etc.
4. Unmarked dimensions refer to GB/T1804-2000m level.
5. Spray black paint, the paint film will be smooth and smooth, there will be no significant difference in color, and there will be no peeling, bruises, etc. on the surface.

Mark.mark	Modification.Change	Modify by.	Modifier Doc	NO.	File number	Data	Date

Des.Design		CHIKO Solar Mounting Solutions	Shanghai Chenke Solar Technology Co., Ltd.			
Drw.Drawing	Bowen231124		Name.Name:	rear pillar		
Check. review		Mat.Material:	S350GD+ZM275	Type.Model:	CK-GT-001-019-802	
Craft. Craftmanship		View.Perspective:	Scale. Proportion:		Page.Page number:	
App. Approval						

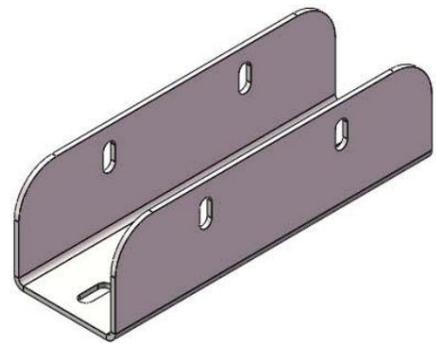
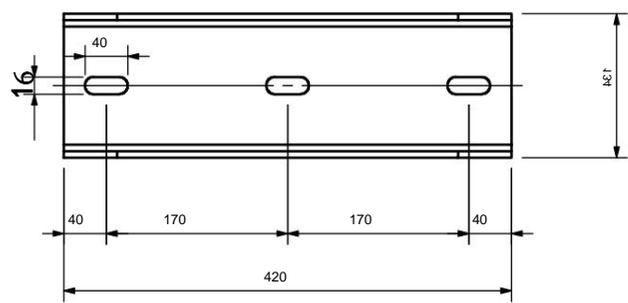
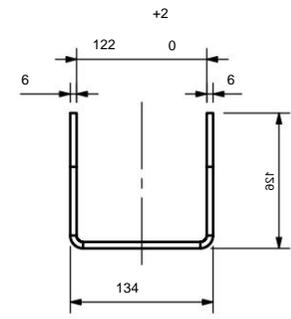
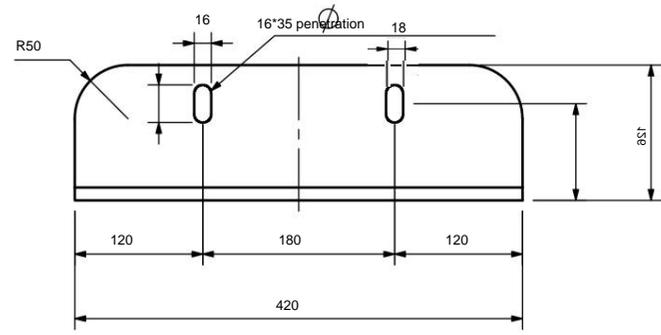


skills requirement:

1. The weight of double-sided coating of continuous heat zinc, aluminum and magnesium is $\gamma 275g/m^2$.
2. Product size, shape, weight and allowable deviations are in accordance with GB/T25052-2010.
3. The surface is clean and free of burrs, bruises, etc.
4. Unmarked dimensions refer to GB/T1804-2000m level.
5. Spray black paint, the paint film will be smooth and smooth, there will be no significant difference in color, and there will be no peeling, bruises, etc. on the surface.

Mark.mark	Modification.Change	Modify by.	Modifier Doc	NO.	File number	Data.	Date

Des.Design		 Shanghai Chenke Solar Technology Co., Ltd.				
Drw.Drawing	Bowen231124					
Check. review		Mat.Material: S350GD+ZM275 <table border="1" style="float: right;"> <tr> <td>Name.Name:</td> <td>Diagonal brace</td> </tr> <tr> <td>Type.Model:</td> <td>CK-GT-001-019-803</td> </tr> </table>	Name.Name:	Diagonal brace	Type.Model:	CK-GT-001-019-803
Name.Name:	Diagonal brace					
Type.Model:	CK-GT-001-019-803					
Craft. Craftsmanship		View.Perspective:  				
App. Approval		<table border="1" style="float: right;"> <tr> <td>Scale. Proportion:</td> <td></td> </tr> <tr> <td>Page.Page number:</td> <td></td> </tr> </table>	Scale. Proportion:		Page.Page number:	
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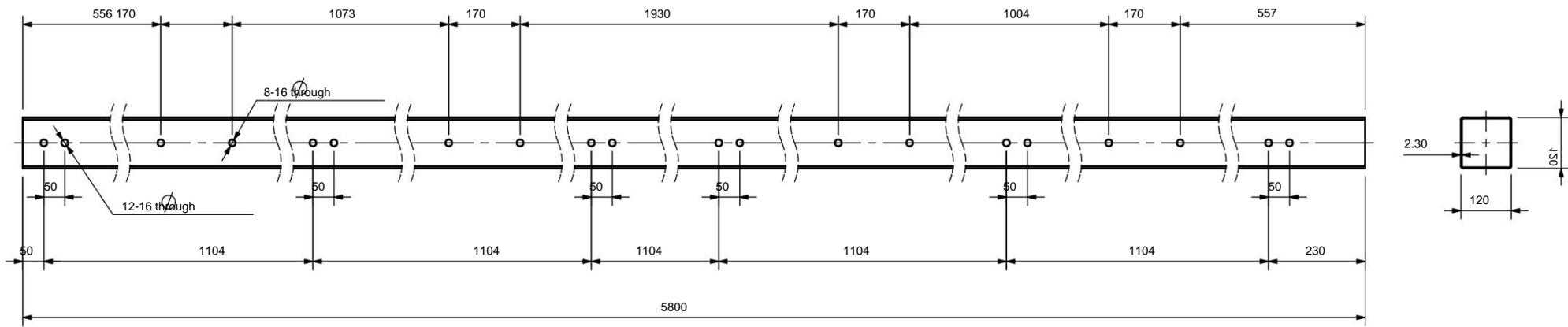


skills requirement:

1. The surface is hot-dip galvanized, and the film thickness is not less than 65µm. For details, please refer to GB/T13912.
2. After galvanizing, it is not allowed to have zinc hanging on holes or other phenomena that affect the processing size.
3. The surface is clean and free of burrs, bruises, etc.
4. Unmarked dimensions refer to GB/T1804-2000m level.
5. Spray black paint, the paint film will be smooth and smooth, there will be no significant difference in color, and there will be no peeling, bruises, etc. on the surface.

Mark.mark	Modification.Change	Modify by.	Modifier Doc	NO.	File number	Data.	Date

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Drw.Drawing	Bowen231124	
Check. review		Mat.Material: Q235B Name.Name: Diagonal brace cross arm adapter Type.Model: CK-GT-001-019-804
Craft. Craftsmanship		View.Perspective:   Scale. Proportion: Page.Page number:
App. Approval		

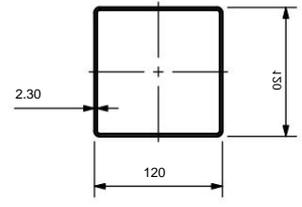
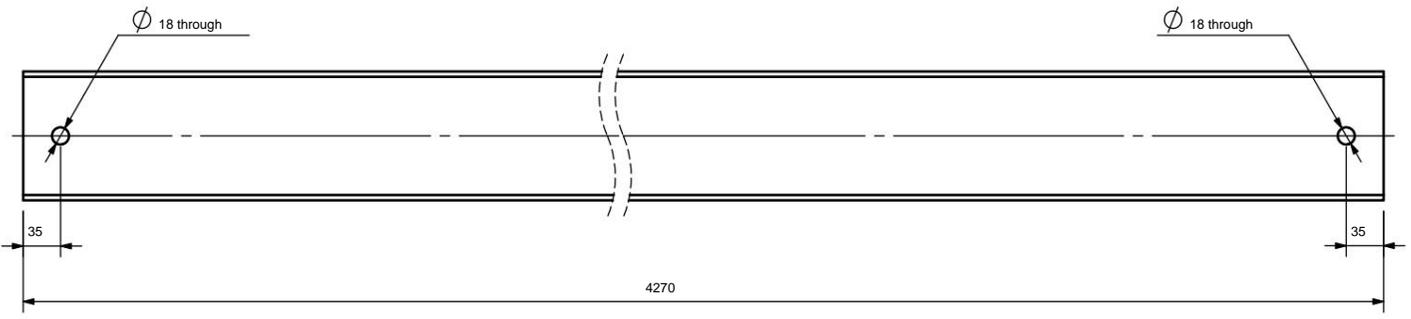


skills requirement:

1. The weight of double-sided coating of continuous heat zinc, aluminum and magnesium is $\geq 275g/m^2$.
2. Product size, shape, weight and allowable deviations are in accordance with GB/T25052-2010.
3. The surface is clean and free of burrs, bruises, etc.
4. Unmarked dimensions refer to GB/T1804-2000m level.
5. Spray black paint, the paint film will be smooth and smooth, there will be no significant difference in color, and there will be no peeling, bruises, etc. on the surface.

Mark.mark	Modification.Change	Modify by	Modifier	Doc NO.	File number	Date	Date

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Drw.Drawing	Bowen231124		Mat.Material: S350GD+ZM275	Name.Name: inclined beam
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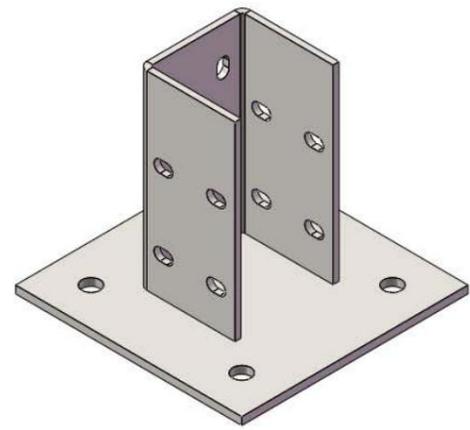
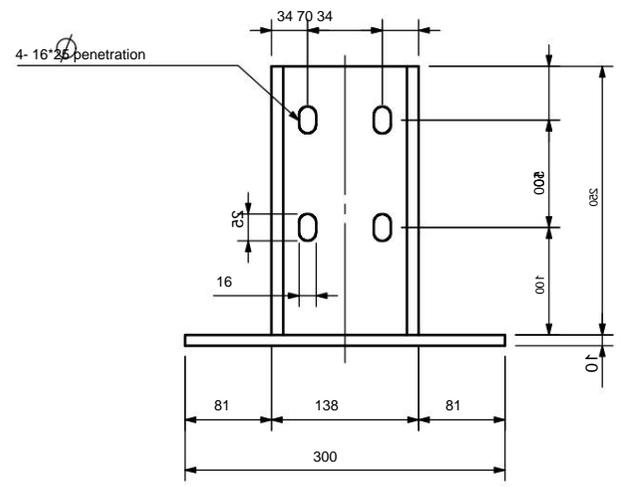
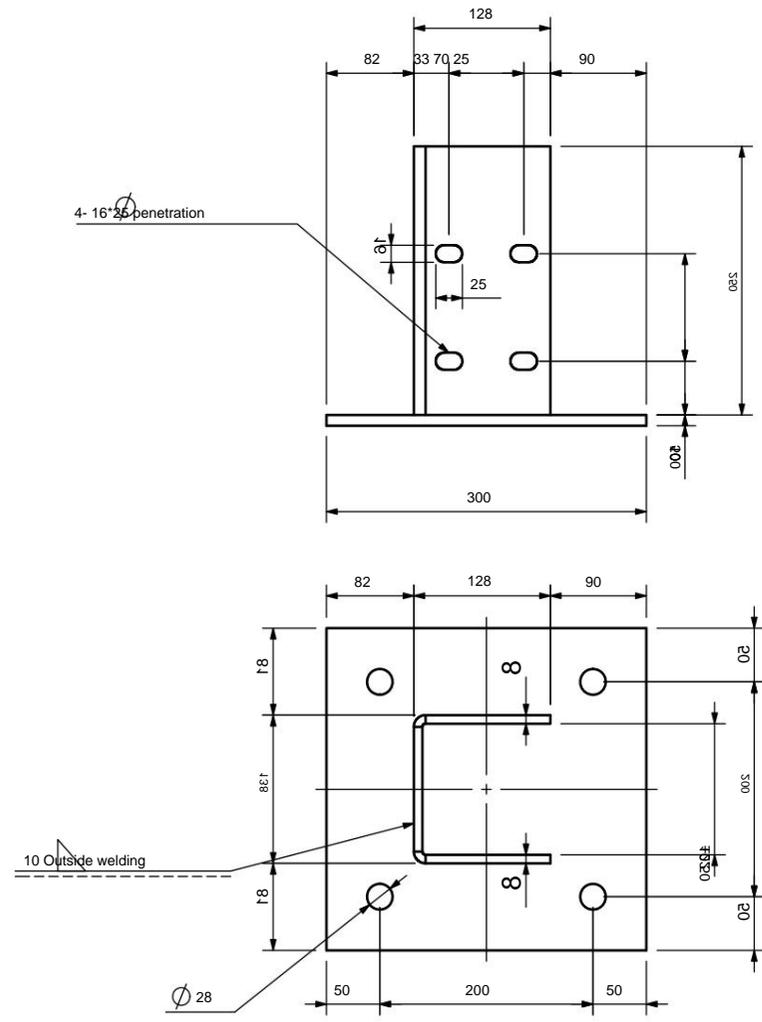


skills requirement:

1. The weight of double-sided coating of continuous heat zinc, aluminum and magnesium is 275g/m².
2. Product size, shape, weight and allowable deviations are in accordance with GB/T25052-2010.
3. The surface is clean and free of burrs, bruises, etc.
4. Unmarked dimensions refer to GB/T1804-2000m level.
5. Spray black paint, the paint film will be smooth and smooth, there will be no significant difference in color, and there will be no peeling, bruises, etc. on the surface.

Mark. mark	Modification. Change	Modify by. Modifier Doc	NO. File number	Data. Date

Des. Design		Shanghai Chenke Solar Technology Co., Ltd.
Drw. Drawing	Bowen231124	
Check. review		Mat. Material: S350GD+ZM275 Name. Name: cross arm
Craft. Craftsmanship		Type. Model: CK-GT-001-019-806
App. Approval		View. Perspective: Scale. Proportion: Page. Page number:

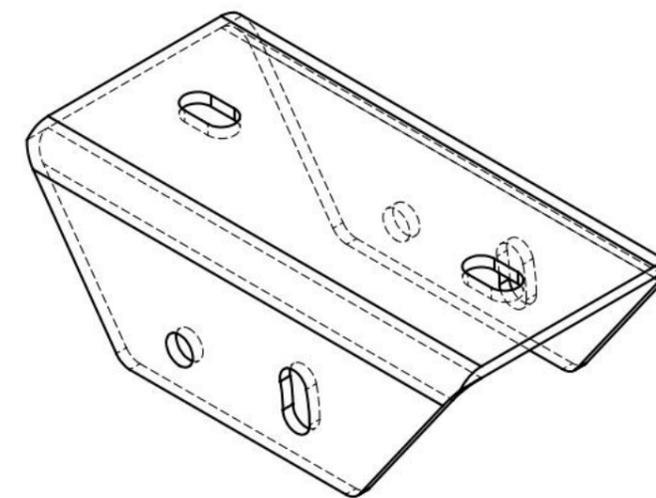
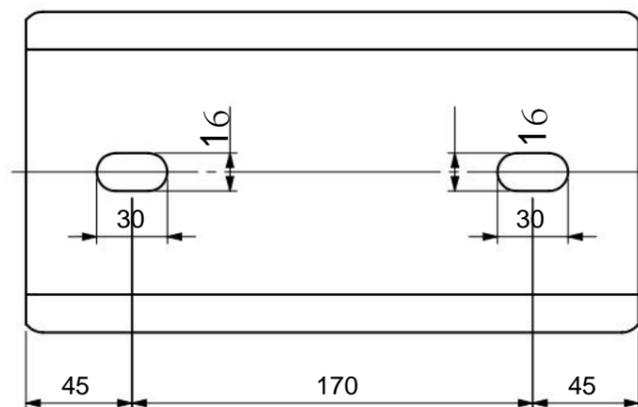
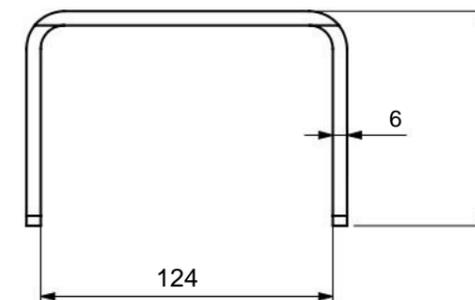
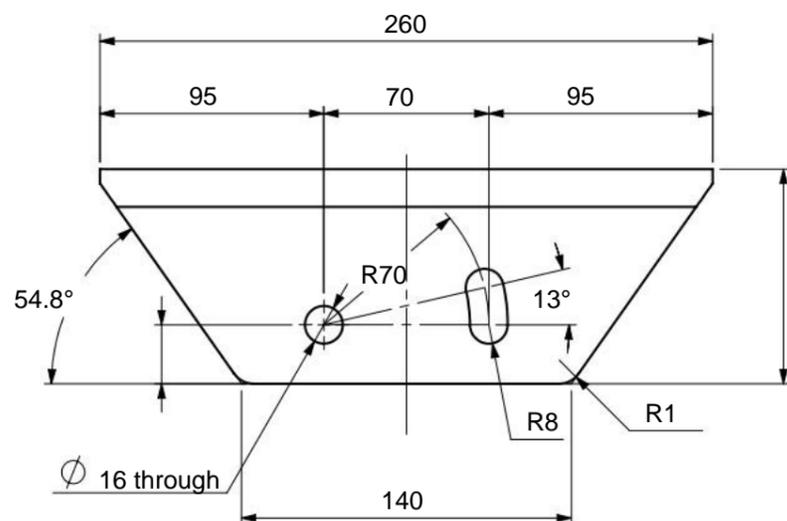


skills requirement:

1. The surface is hot-dip galvanized, and the film thickness is not less than 65 μ m. For details, please refer to GB/T13912.
2. After galvanizing, it is not allowed to have zinc hanging on holes or other phenomena that affect the processing size.
3. The surface is clean and free of burrs, bruises, etc.
4. Unmarked dimensions refer to GB/T1804-2000m level.
5. Spray black paint, the paint film will be smooth and smooth, there will be no significant difference in color, and there will be no peeling, bruises, etc. on the surface.

Mark.mark	Modification.Change	Modify by.	Modifier Doc NO.	File number	Date

Des.Design		 Shanghai Chenke Solar Technology Co., Ltd.				
Drw.Drawing	Bowen231124					
Check. review		Mat.Material: Q235B <table border="1" style="float: right;"> <tr> <td>Name.Name:</td> <td>Base</td> </tr> <tr> <td>Type.Model:</td> <td>CK-GT-001-019-807</td> </tr> </table>	Name.Name:	Base	Type.Model:	CK-GT-001-019-807
Name.Name:	Base					
Type.Model:	CK-GT-001-019-807					
Craft. Craftsmanship		View.Perspective:  				
App. Approval		Scale. Proportion: Page.Page number:				

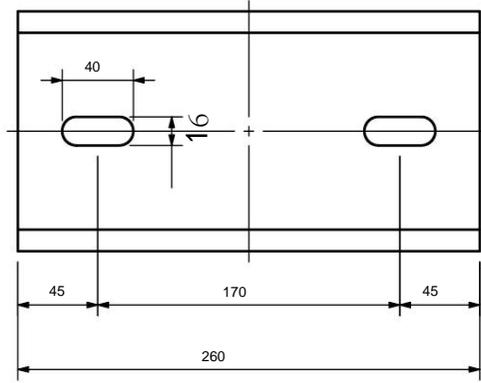
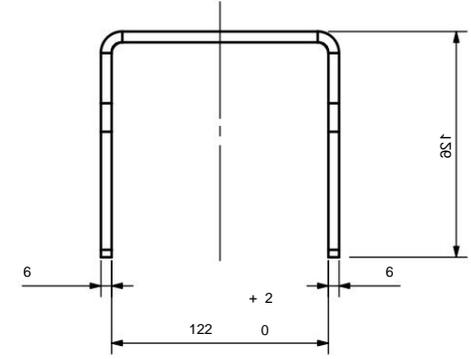
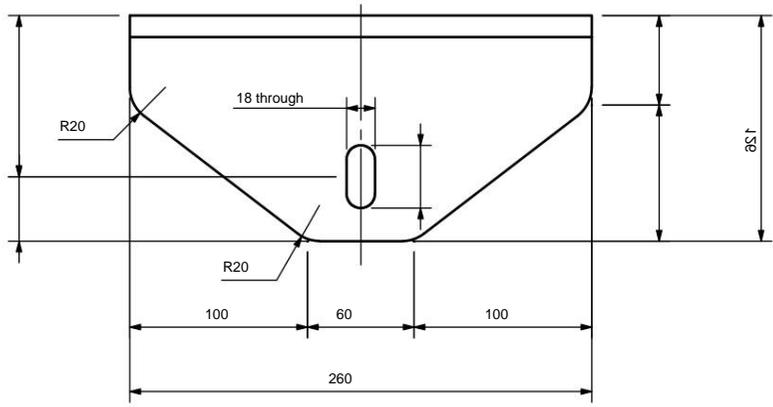


skills requirement:

1. The surface is hot-dip galvanized, and the film thickness is not less than 65UM. For details, please refer to (GB/T13912)
2. After hot-dip galvanizing, it is not allowed to have zinc hanging on holes or other phenomena that affect the processing size.
3. The surface is clean and tidy, without burrs, bruises, etc.
4. Unmarked dimensions refer to GB/T1804-2000m level.
5. Spray black paint, the paint film will be smooth and smooth, with no significant difference in color, and no peeling, bruises, etc. on the surface.

Mark.mark	Modification.Change	Modify by.	Modifier	Doc NO.	File number	Data.	Date

Des.Design		CHIKO Solar Mounting Solutions	Name.Name: Column adapter	
Drw.Drawing	WHL 231116		Type.Model: CK-GT-001-019-808	
Check. review		Mat.Material: Q235B	Scale. Proportion:	
Crsft. Craftsmanship		View.Perspective:		Page.Page number:
App. Approval				

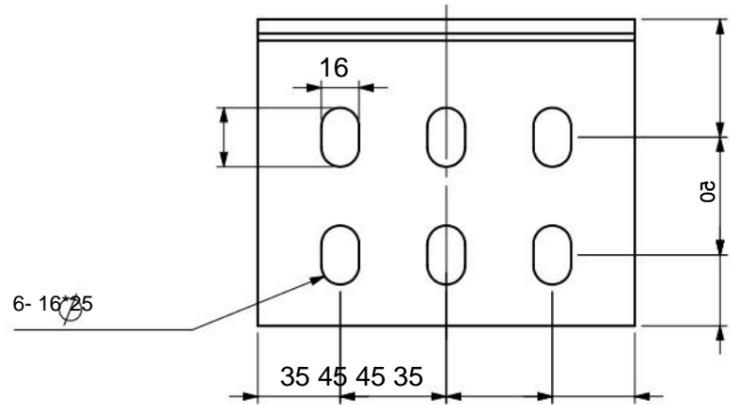
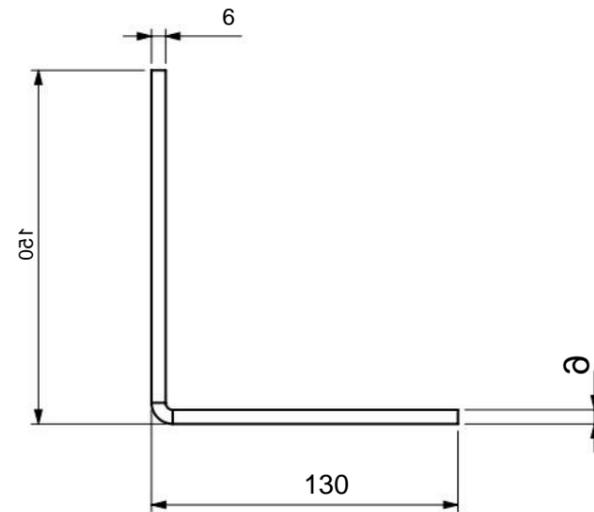
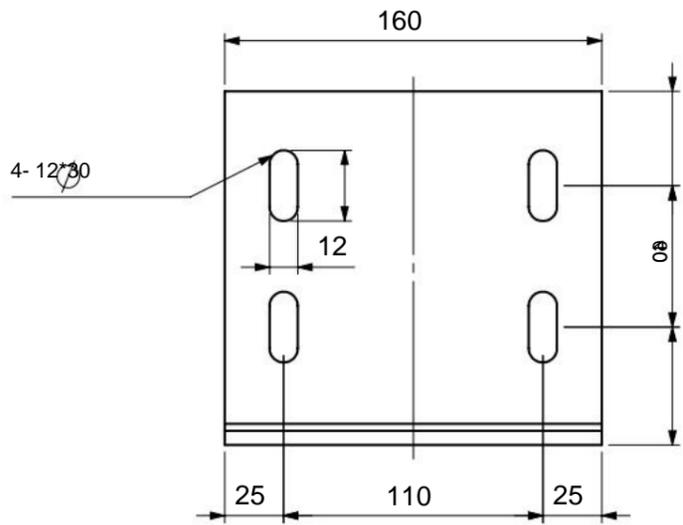


Technical

requirements: 1. The surface is hot-dip galvanized, and the film thickness is not less than 65 μm. For details, please refer to GB/T13912. 2. After galvanizing, it is not allowed to have zinc hanging on holes or other phenomena that affect the processing size. 3. The surface is clean and free of burrs, bruises, etc. 4. Unmarked dimensions refer to GB/T1804-2000m level. 5. Spray black paint, the paint film will be smooth and smooth, there will be no significant difference in color, and there will be no peeling, bruises, etc. on the surface.

Mark.mark	Modification.Change	Modify by	Modifier Doc	NO. File number	Data	Date

Des.Design		 Shanghai Chenke Solar Technology Co., Ltd.
Drw.Drawing	Bowen231124	
Check. review		Mat.Material: Q235B Name.Name: Diagonal brace adapter
Craft. Craftsmanship		Type.Model: CK-GT-001-019-809
App.ApprovalC		View.Perspective:   Scale. Proportion: Page.Page number:

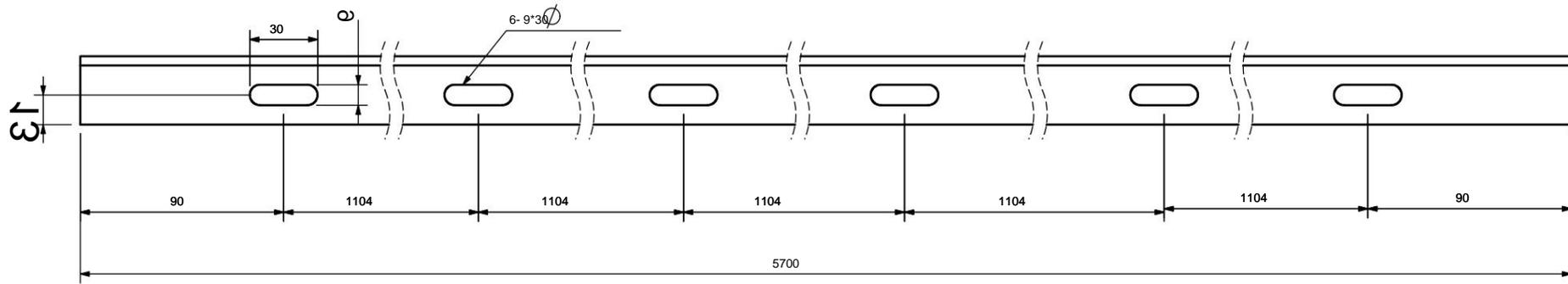


skills requirement:

1. The surface is hot-dip galvanized, and the film thickness is not less than 65UM. For details, please refer to (GB/T13912)
2. After hot-dip galvanizing, it is not allowed to have zinc hanging on holes or other phenomena that affect the processing size.
3. The surface is clean and tidy, without burrs, bruises, etc.
4. Unmarked dimensions refer to GB/T1804-2000m level.
5. Spray black paint, the paint film will be smooth and smooth, with no significant difference in color, and no peeling, bruises, etc. on the surface.

Mark.mark	Modification.Change	Modify by.	Modifier	Doc NO.	File number	Data.	Date

Des.Design				
Drw.Drawing	WHL 231116			
Check. review		Mat.Material: Q235B	Name.Name: purlin	
Crsft. Craftsmanship			Type.Model: CK-GT-001-019-810	
App. Approval		View.Perspective:	Scale. Proportion:	Page.Page number:

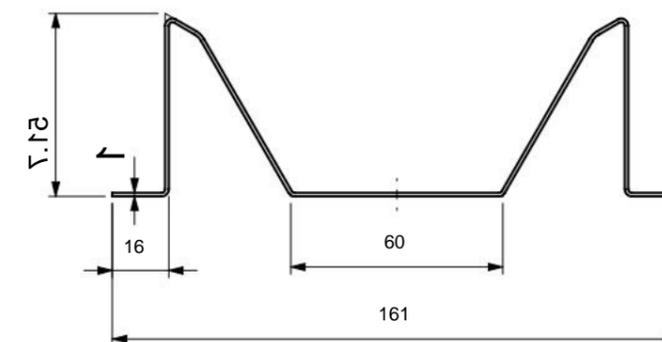
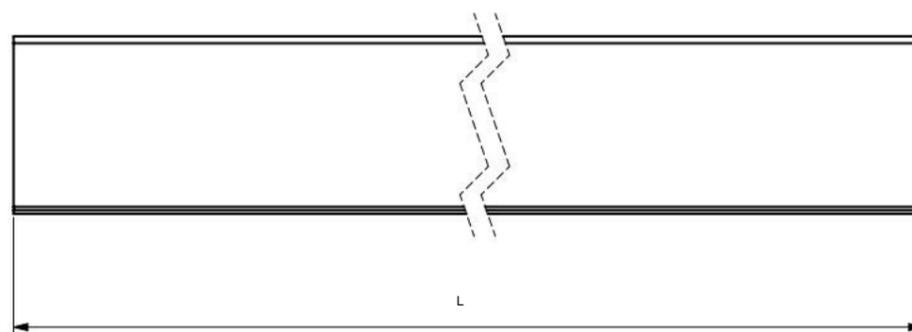


skills requirement:

1. The weight of double-sided coating of continuous heat zinc, aluminum and magnesium is $\geq 275g/m^2$.
2. Product size, shape, weight and allowable deviations are in accordance with GB/T25052-2010.
3. The surface is clean and free of burrs, bruises, etc.
4. Unmarked dimensions refer to GB/T1804-2000m level.
5. Spray black paint, the paint film will be smooth and smooth, there will be no significant difference in color, and there will be no peeling, bruises, etc. on the surface.

Mark.mark	Modification.Change	Modify by.	Modifier Doc	NO. File number	Data.	Date

Des.Design		 CHIKO Solar Mounting Solutions	Shanghai Chenke Solar Technology Co., Ltd.		
Drw.Drawing	Bowen231130		Name.Name:	Purlin straight strips	
Check. review		Mat.Material:	S350GD/S250GD+ZM275	Type.Model:	CK-GT-001-019-811
Craft. Craftsmanship		View.Perspective:		Scale. Proportion:	
App. Approval					Page.Page number:



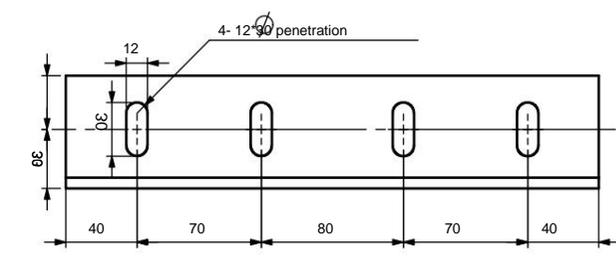
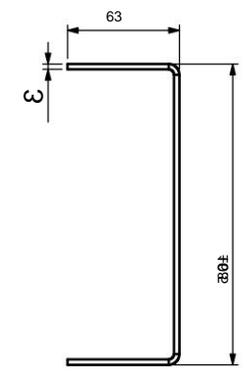
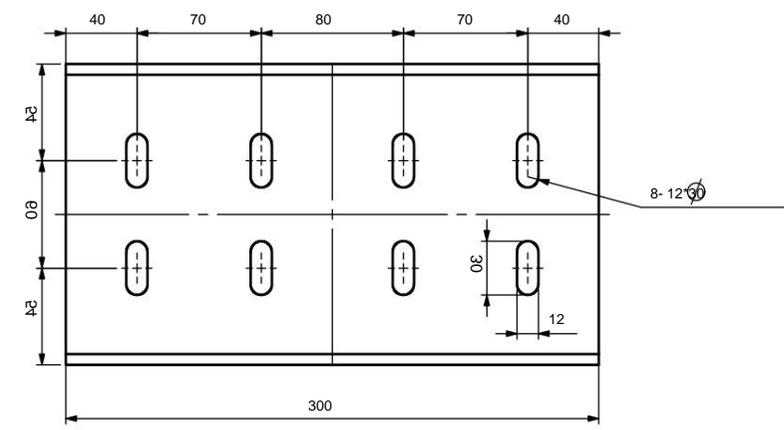
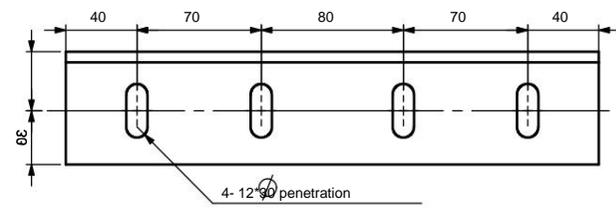
L/mm		
3610		

Technical requirement two:

1. The double-sided weight of the continuous hot-dip galvanized aluminum-magnesium coating of equal thickness is $\geq 275\text{g/m}^2$.
2. Product size, shape, weight and allowable deviations are in accordance with GB/T25052-2010.
Negative tolerances on thickness are not allowed.
3. The surface is clean and tidy, without burrs, bruises, etc.
4. Unmarked dimensions refer to GB/T1804-2000m level.
5. Spray black paint, the paint film will be smooth and smooth, with no significant difference in color, and no peeling, bruises, etc. on the surface.

Mark.mark	Modification. Change A:	Modify by. Modifier	Doc NO.	File number	Data. Date
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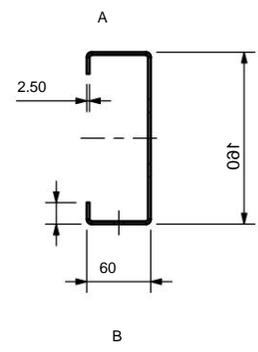
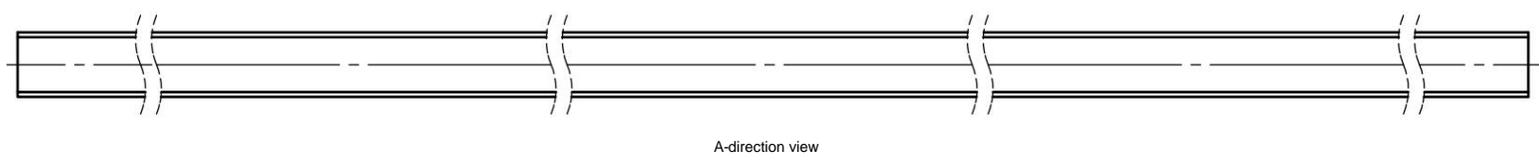
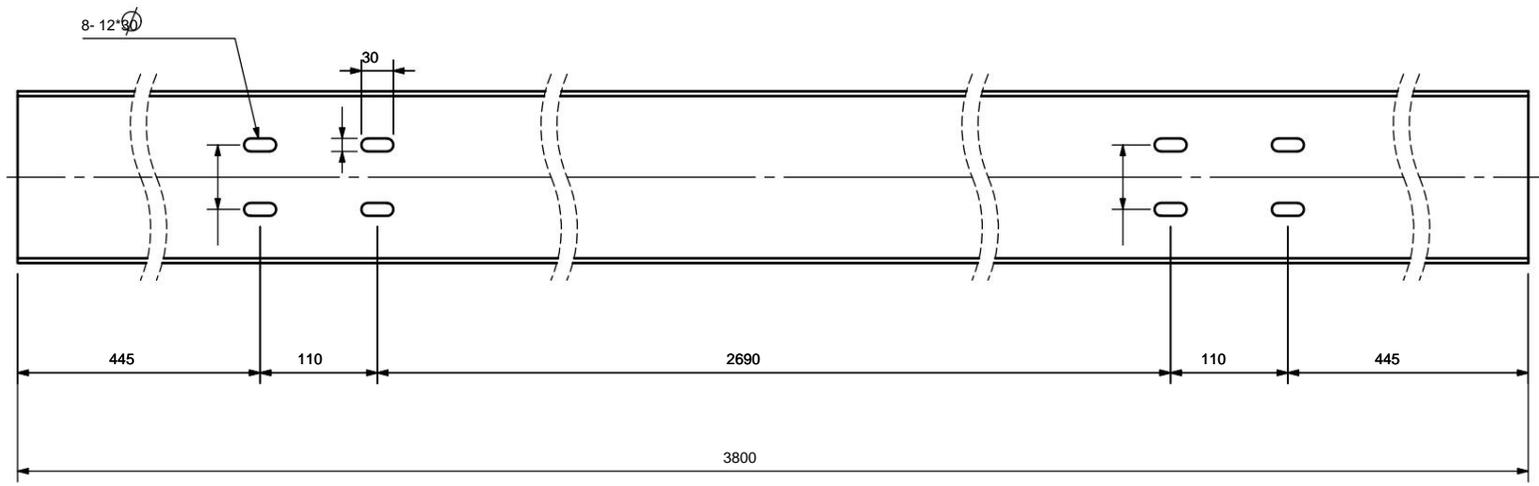
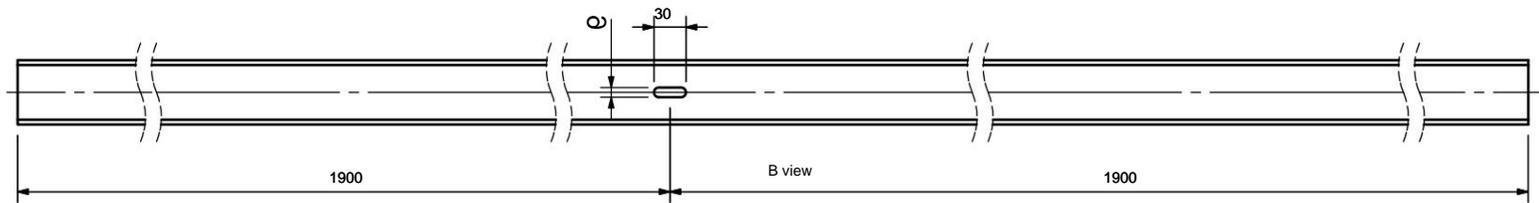
Des.Design	WHL 220610			
Drw.Drawing				
Check. review		Mat.Material: S350GD	Name.Name: Waterproof guide rail	
Craft. Craftsmanship			Type.Model: CK-IR-010-L	
App. Approval		View.Perspective:	Scale. Proportion:	Page.Page number:



- skills requirement:
1. The double-sided weight of the continuous hot-dip galvanized aluminum-magnesium coating of equal thickness is $\geq 275\text{g/m}^2$.
 2. Product size, shape, weight and allowable deviations are in accordance with GB/T25052-2010.
 3. The surface is clean and tidy, without burrs, bruises, etc.
 4. Unmarked dimensions refer to GB/T1804-2000m level.
 5. Spray black paint, the paint film will be smooth and smooth, with no significant difference in color, and no peeling, bruises, etc. on the surface.

Mark.mark	Modification.Change	Modify by	Modifier Doc	NO. File number Data	Date

Des.Design		Shanghai Chenke Solar Technology Co., Ltd.
Drw.Drawing	sweet 231116	
Check. review		Mat.Material: S350GD+ZM275 Name.Name: purlin connections Type.Model: CK-GT-001-019-820
Craft. Craftmanship		
App. Approval		View.Perspective: Scale. Proportion: Page.Page number:

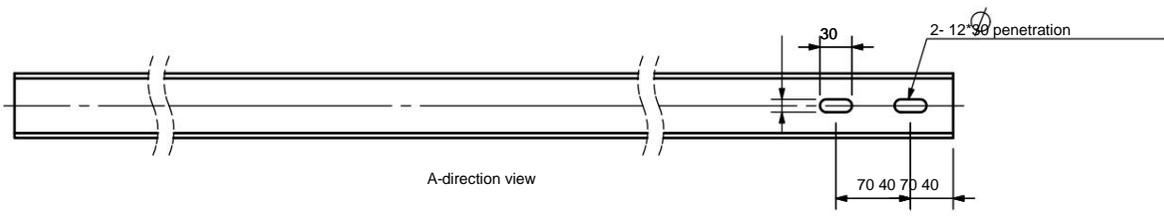
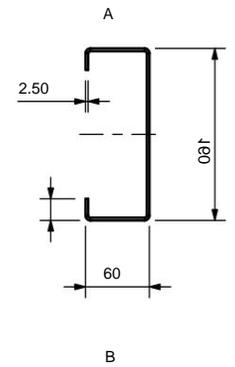
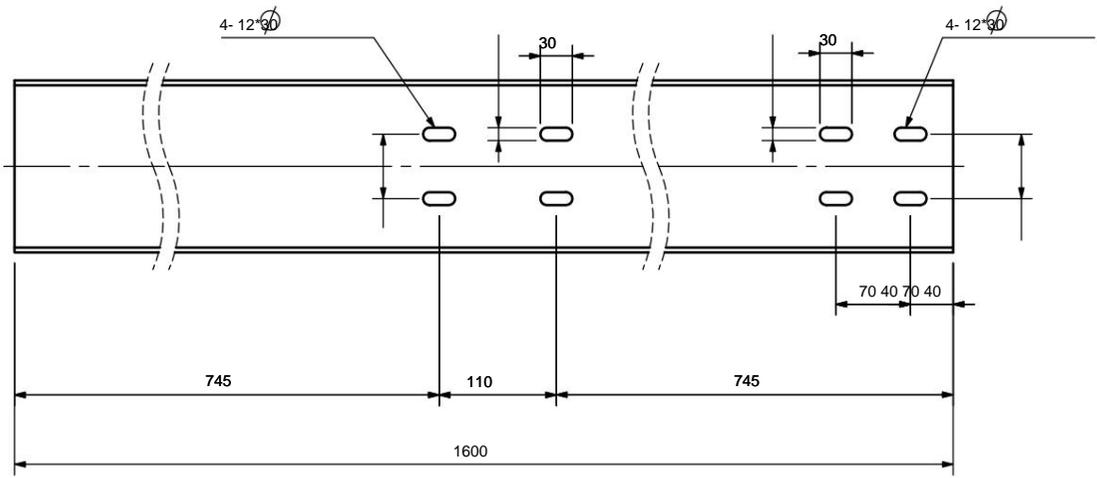
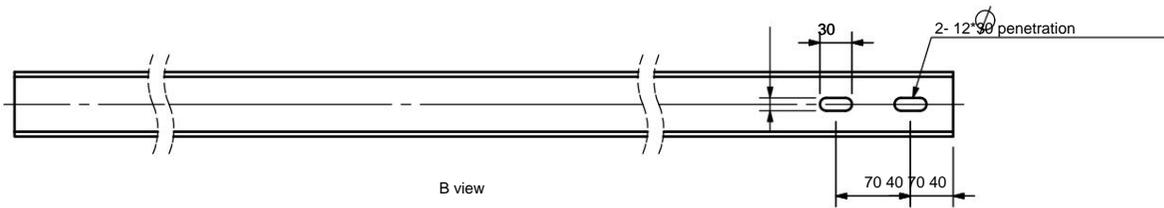


skills requirement:

1. The weight of double-sided coating of continuous heat zinc, aluminum and magnesium is $\geq 275g/m^2$.
2. Product size, shape, weight and allowable deviations are in accordance with GB/T25052-2010.
3. The surface is clean and free of burrs, bruises, etc.
4. Unmarked dimensions refer to GB/T1804-2000m level.
5. Spray black paint, the paint film will be smooth and smooth, there will be no significant difference in color, and there will be no peeling, bruises, etc. on the surface.

Mark.mark	Modification.Change	Modify by	Modifier	Doc NO.	File number	Date	Date

Des.Design		Shanghai Chenke Solar Technology Co., Ltd.
Drw.Drawing	Bowen231124	
Check. review		Mat.Material: S350GD+ZM275 Name.Name: Purlin 1 Type.Model: CK-GT-001-019-821
Craft. Craftsmanship		
App. Approval		View.Perspective: Scale. Proportion: Page.Page number:



Technical

requirements: 1. The double-sided weight of the continuous-heat zinc, aluminum and magnesium coating of equal thickness is $\geq 275g/m^2$. 2. Product size, shape, weight and allowable deviations are in accordance with GB/T25052-2010. 3. The surface is clean and free of burrs, bruises, etc. 4. Unmarked dimensions refer to GB/T1804-2000m level. 5. Spray black paint, the paint film will be smooth and smooth, there will be no significant difference in color, and there will be no peeling, bruises, etc. on the surface.

Mark.mark	Modification.Change	Modify by.	Modifier	Doc NO.	File number	Data.	Date

Des.Design	
Drw.Drawing	Bowen231130
Check. review	
Craft. Craftsmanship	
App.ApprovalC	

Shanghai Chenke Solar Technology Co., Ltd.

Mat.Material: S350GD+ZM275	Name.Name: Purlin 2
	Type.Model: CK-GT-001-019-822
View.Perspective:	Scale. Proportion: Page.Page number:

4

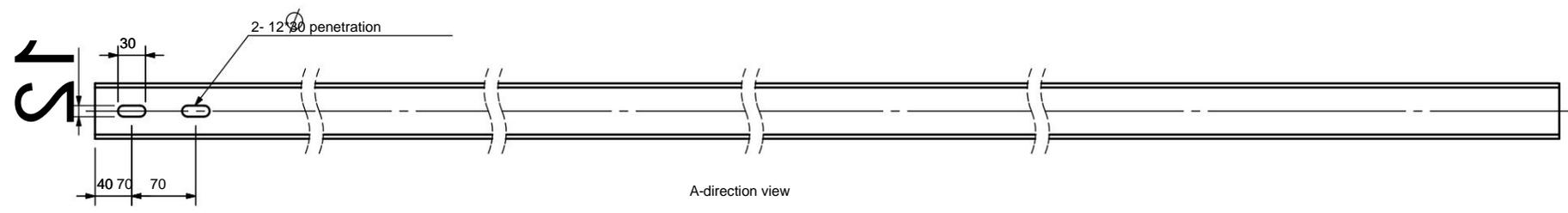
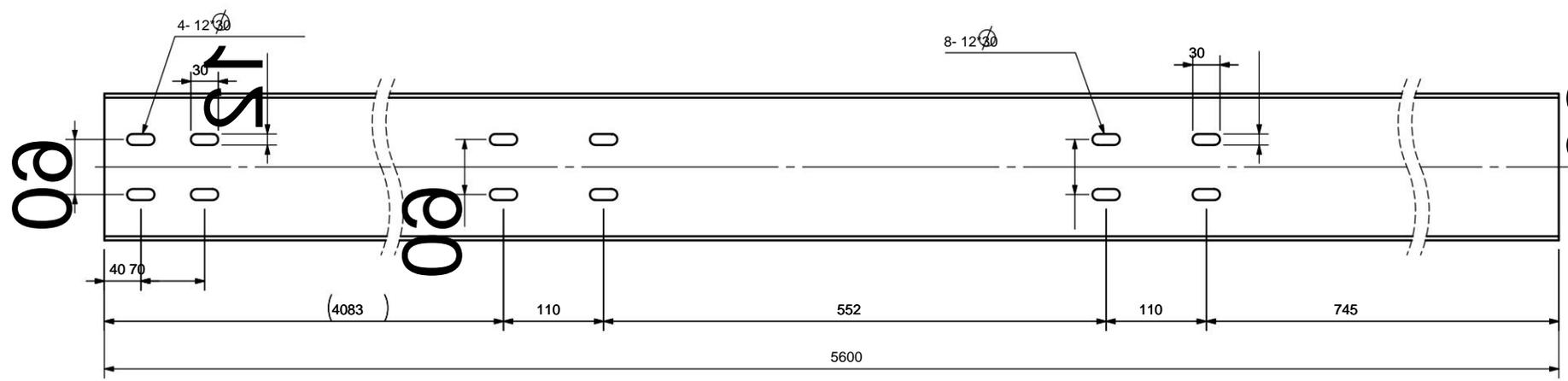
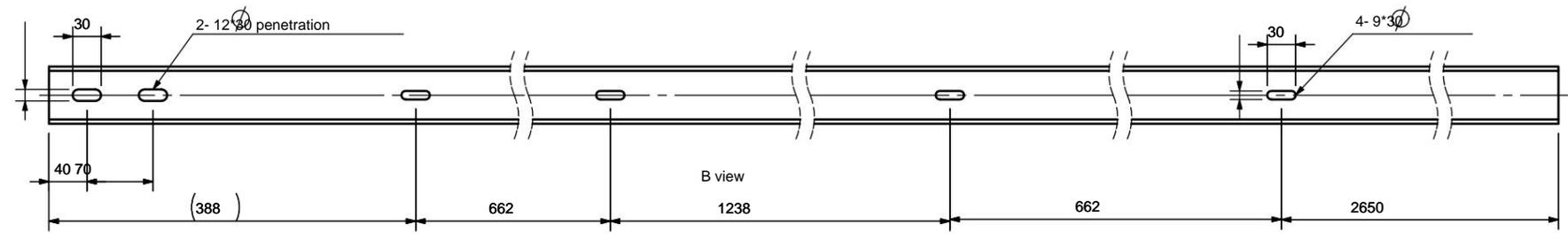
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skills requirement:

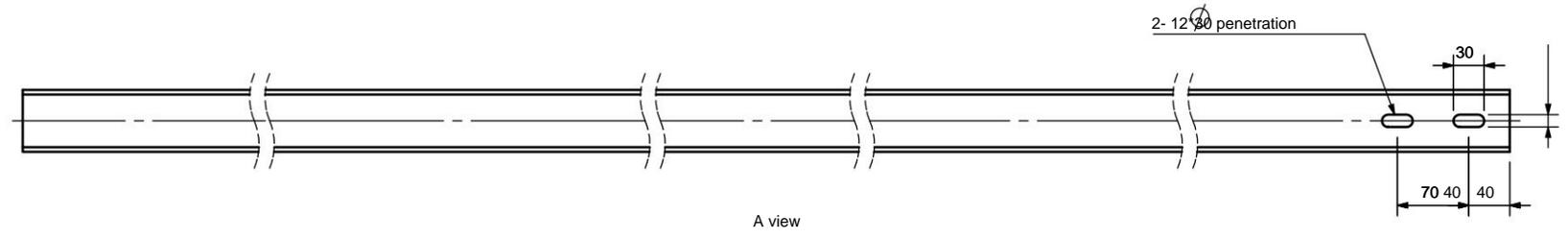
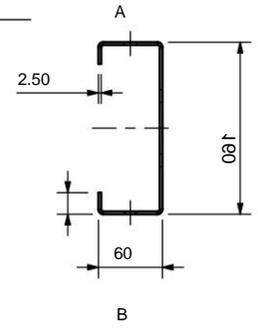
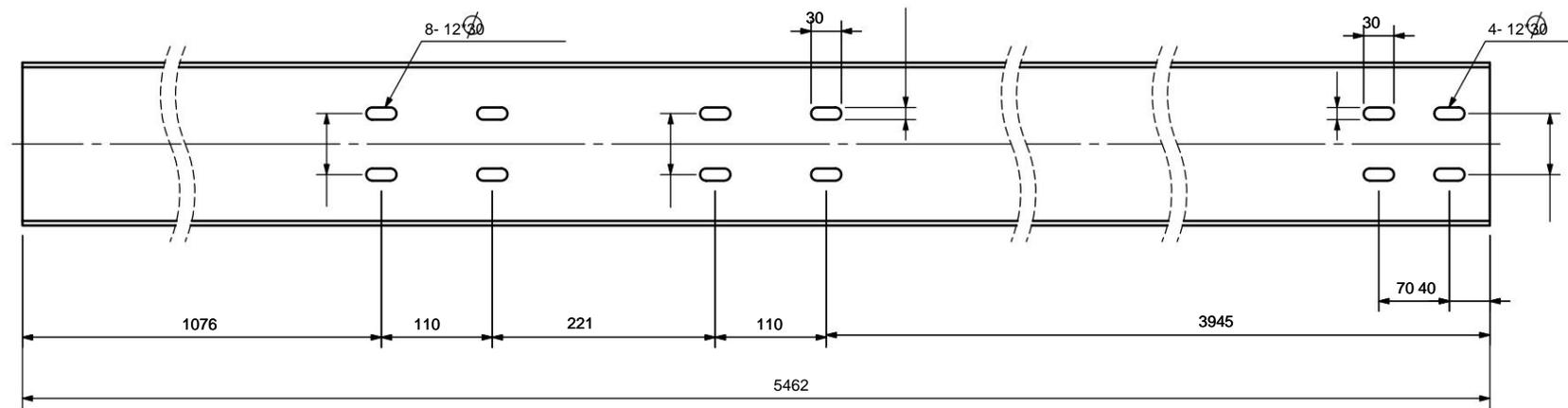
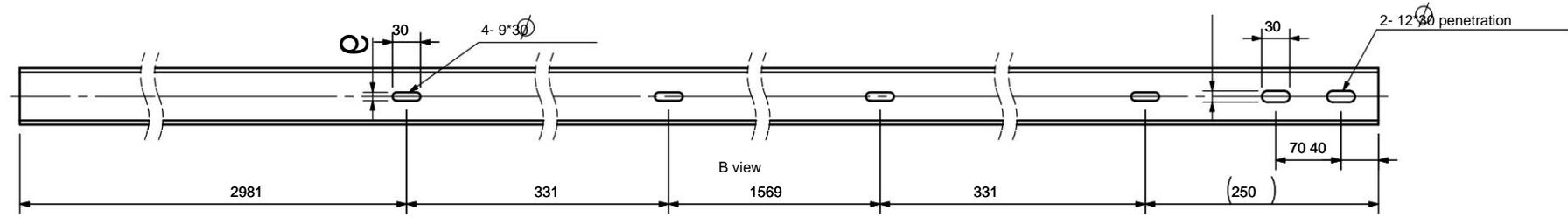
1. The weight of double-sided coating of continuous heat zinc, aluminum and magnesium is $\geq 275\text{g/m}^2$.
2. Product size, shape, weight and allowable deviations are in accordance with GB/T25052-2010.
3. The surface is clean and free of burrs, bruises, etc.
4. Unmarked dimensions refer to GB/T1804-2000m level.
5. Spray black paint, the paint film will be smooth and smooth, there will be no significant difference in color, and there will be no peeling, bruises, etc. on the surface.

Mark.mark	Modification.Change	Modify by. Modifier	Doc NO. File number	Data. Date

Des.Design		CHIKO Solar Mounting Solutions	Shanghai Chenke Solar Technology Co., Ltd.		
Drw.Drawing	Bowen231201		Name.Name:	Purlin 3	
Check. review		Mat.Material:	S350GD+ZM275	Type.Model:	CK-GT-001-019-823
Craft. Craftsmanship		View.Perspective:		Scale. Proportion:	
App. Approval				Page.Page number:	

1

1



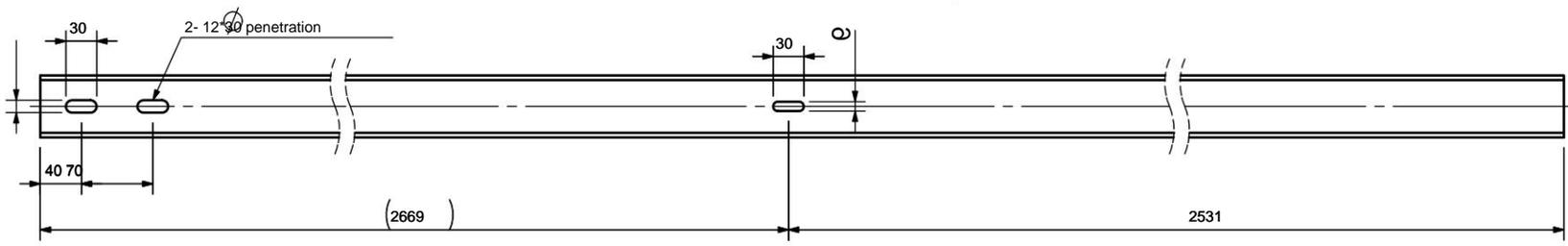
skills requirement:

1. The weight of double-sided coating of continuous heat zinc, aluminum and magnesium is $\geq 275\text{g/m}^2$.
2. Product size, shape, weight and allowable deviations are in accordance with GB/T25052-2010.
3. The surface is clean and free of burrs, bruises, etc.
4. Unmarked dimensions refer to GB/T1804-2000m level.
5. Spray black paint, the paint film will be smooth and smooth, there will be no significant difference in color, and there will be no peeling, bruises, etc. on the surface.

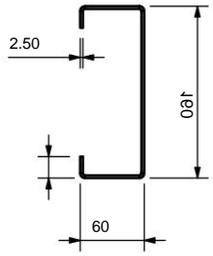
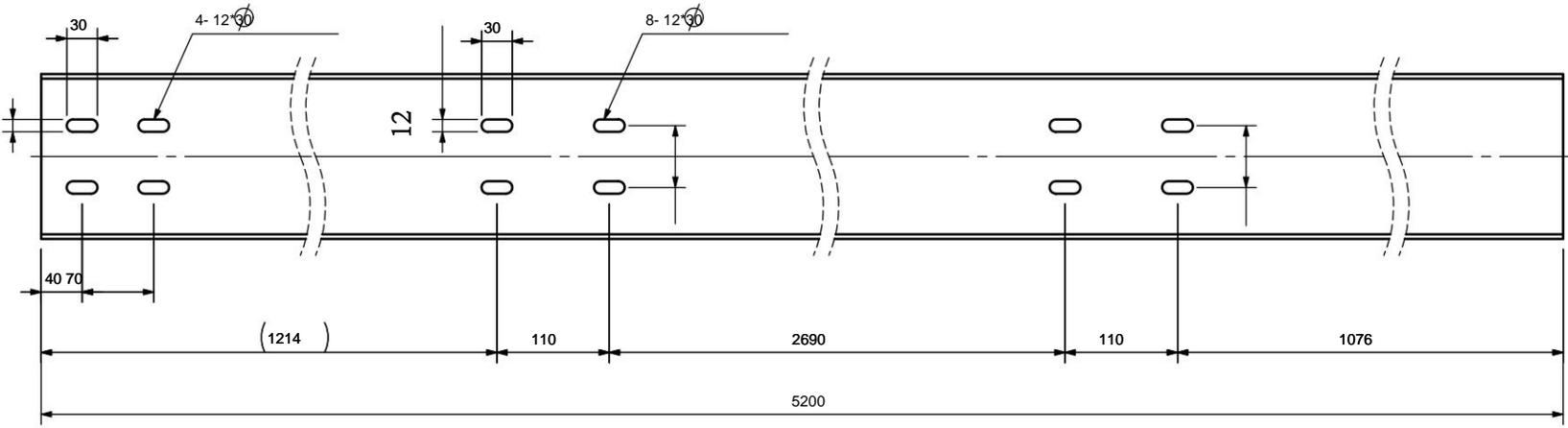
Mark.mark	Modification.Change	Modify by. Modifier	Doc NO. File number	Data. Date

Des.Design		Shanghai Chenke Solar Technology Co., Ltd.
Drw.Drawing	Bowen231201	
Check. review		Mat.Material: S350GD+ZM275
Craft. Craftmanship		Name.Name: Purlin 4
App. Approval		Type.Model: CK-GT-001-019-824
View.Perspective:		Scale. Proportion:
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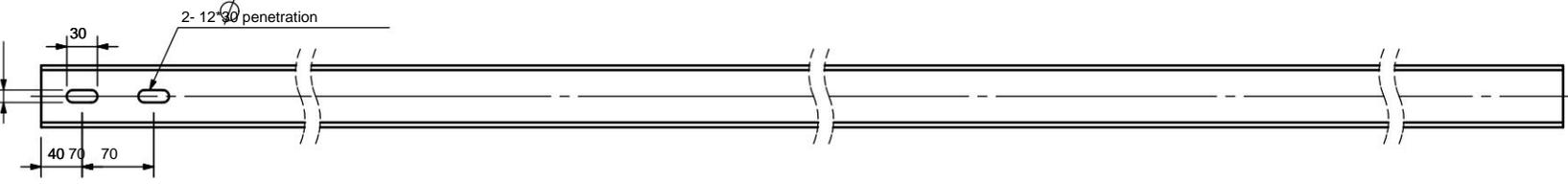
4



3



2



- skills requirement:
1. The weight of double-sided coating of continuous heat zinc, aluminum and magnesium is $\geq 275g/m^2$.
 2. Product size, shape, weight and allowable deviations are in accordance with GB/T25052-2010.
 3. The surface is clean and free of burrs, bruises, etc.
 4. Unmarked dimensions refer to GB/T1804-2000m level.
 5. Spray black paint, the paint film will be smooth and smooth, there will be no significant difference in color, and there will be no peeling, bruises, etc. on the surface.

Mark.mark	Modification.Change	Modify by.Modifier	Doc NO. File number	Data. Date

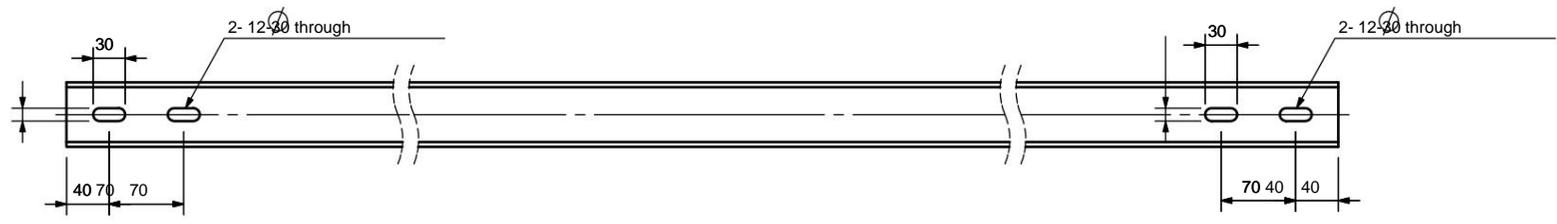
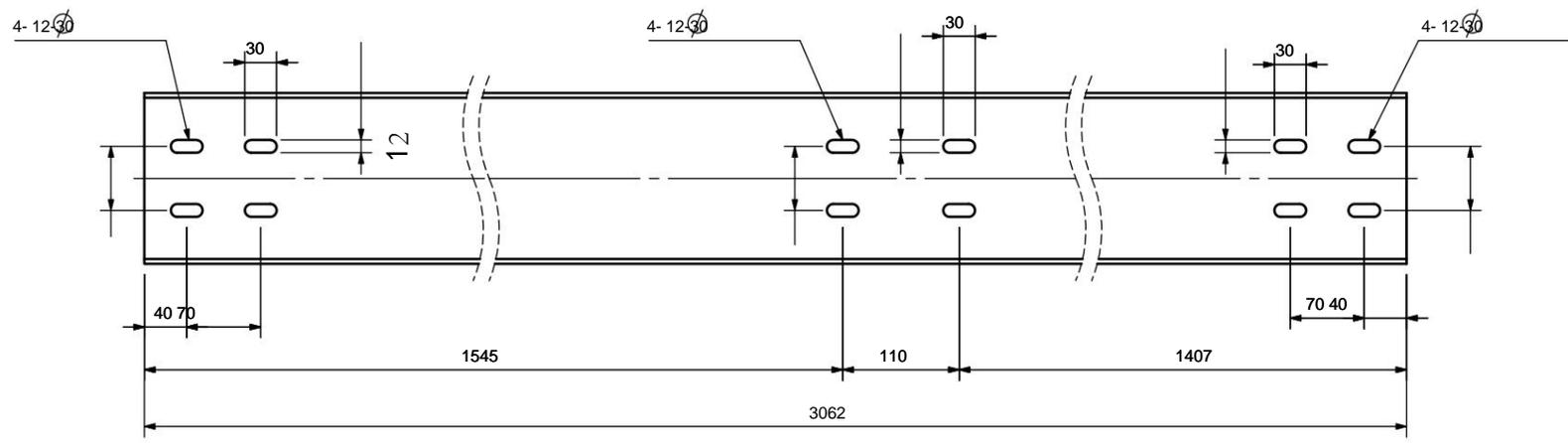
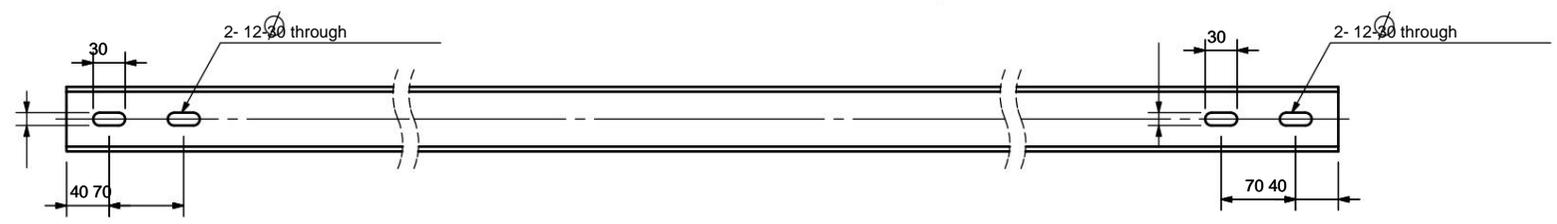
Des.Design		 CHIKO Solar Mounting Solutions	Shanghai Chenke Solar Technology Co., Ltd.	
Drw.Drawing	Bowen231124		Mat.Material: S350GD+ZM275	Name.Name: Purlin 5
Check. review			Type.Model: CK-GT-001-019-825	
Craft. Craftsmanship			Scale. Proportion:	
App. Approval		View.Perspective: 	Page.Page number:	

A

B

C

D



skills requirement:

1. The weight of double-sided coating of continuous heat zinc, aluminum and magnesium is $\geq 275\text{g/m}^2$.
2. Product size, shape, weight and allowable deviations are in accordance with GB/T25052-2010.
3. The surface is clean and free of burrs, bruises, etc.
4. Unmarked dimensions refer to GB/T1804-2000m level.
5. Spray black paint, the paint film will be smooth and smooth, there will be no significant difference in color, and there will be no peeling, bruises, etc. on the surface.

Mark.mark	Modification.Change	Modify by. Modifier	Doc NO. File number	Data. Date

Des.Design		 CHIKO Solar Mounting Solutions	Shanghai Chenke Solar Technology Co., Ltd.	
Drw.Drawing	Bowen231201		Name.Name:	Purlin 6
Check. review		Mat.Material: S350GD+ZM275	Type.Model:	CK-GT-001-019-826
Craft. Craftsmanship		View.Perspective:	Scale. Proportion:	Page.Page number:
App. Approval				



ICC-ES Evaluation Report

Reissued July 2022

ESR-1976

Revised January 2023

This report is subject to renewal July 2024.

DIVISION: 05 00 00—METALS
Section: 05 05 23—Metal Fastenings

REPORT HOLDER:

ITW BUILDDEX

EVALUATION SUBJECT:

ITW BUILDDEX TEKS® SELF-DRILLING FASTENERS

1.0 EVALUATION SCOPE

Compliance with the following codes:

- 2021, 2018, 2015 and 2012 *International Building Code®* (IBC)
- 2021, 2018, 2015 and 2012 *International Residential Code®* (IRC)

For evaluation for compliance with codes adopted by Los Angeles Department of Building and Safety (LADBS), see [ESR-1976 LABC and LARC Supplement](#).

Property evaluated:

Structural

2.0 USES

The ITW Buildex TEKS® Self-drilling Fasteners described in this report are used in engineered or code-prescribed connections of cold-formed steel framing and of sheet steel sheathing to cold-formed steel framing.

3.0 DESCRIPTION

3.1 General:

ITW Buildex TEKS® Self-drilling Fasteners are self-drilling tapping screws complying with the material, process, and performance requirements of ASTM C1513. The screws have either a hex washer head (HWH), an HWH with serrations, or a Phillips® (Type II) pan head. The screws are fully threaded with threads that comply with ASME B18.6.4, and the screws' drill points and flutes are proprietary and are designated as TEKS/1, TEKS/2, TEKS/3, TEKS/4, TEKS/4.5, TEKS/5.0 and TEKS/5. The screws have nominal sizes of No. 10 (0.190 inch), No. 12 (0.216 inch), and 1/4 inch (0.250 inch), and lengths from 1/2 inch to 4 inches (12.7 mm to 102 mm). See Figures 1 through 3 for depictions of the screws. Table 1 provides screw descriptions (size, tpi,

length), nominal diameters, head style, head diameters, point styles, drilling capacity ranges, length of load-bearing area, minimum required protrusion lengths and coatings.

3.2 Material:

ITW Buildex TEKS® Self-drilling Fasteners are case-hardened from carbon steel conforming to ASTM A510, Grades 1018 to 1022, and are heat-treated and case-hardened to give them a hard outer surface necessary to cut internal threads in the joint material. Screws are coated with corrosion preventive coating identified as Climaseal®, or are plated with electrodeposited zinc (E-Zinc) complying with the minimum corrosion resistance requirements of ASTM F1941.

3.3 Cold-formed Steel:

Cold-formed steel material must comply with one of the ASTM specifications listed in Section A3.1 of AISI S100 (Section A2.1 of AISI S100 for the 2015 and 2012 IBC) and have the minimum specified tensile strengths shown in the tables in this report.

4.0 DESIGN AND INSTALLATION

4.1 Design:

4.1.1 General: Selection of screw length must be based on the thickness of the fastened steel members plus the minimum required protrusion past the back of the supporting steel. Point selection must be based on the drilling capacity of the screw. See [Table 1](#) for minimum required protrusion lengths and drilling capacities.

When tested for corrosion resistance in accordance with ASTM B117, the screws meet the minimum requirement listed in ASTM F1941, as required by ASTM C1513, with no white corrosion after three hours and no red rust after 12 hours.

4.1.2 Prescriptive Design: ITW Buildex TEKS Self-drilling Fasteners described in Section 3.1 may be used where ASTM C1513 screws of the same size and head style/dimension are prescribed in the IRC and in the AISI standards referenced in IBC Section 2211 for steel-to-steel connections.

4.1.3 Engineered Design: ITW Buildex TEKS® Self-drilling Fasteners may be used in engineered connections of cold-formed steel construction. Design of the connection must comply with Section J4 of AISI S100 (Section E4 of

AISI S100 for the 2015 and 2012 IBC), using the nominal and allowable fastener tension and shear strength for the screws, shown in Table 5. Allowable connection strength for use in Allowable Strength Design (ASD) for pull-out, pullover, and shear (bearing) capacity for common sheet steel thicknesses are provided in Tables 2, 3, and 4, respectively, based upon calculations in accordance with AISI S100. Instructions on how to calculate connection design strengths for use in Load Resistance Factor Design (LRFD) are found in the footnotes of these tables. The connection strength values are applicable to connections where the connected steel elements are in direct contact with one another. For connections subject to tension, the least of the allowable pullout, pullover, and fastener tension strength found in Tables 2, 3 and 5, respectively, must be used for design. For connections subject to shear, the lesser of the fastener shear strength and allowable shear (bearing) found in Tables 5 and 4, respectively, must be used for design. Design provisions for tapping screw connections subjected to combined shear and tension loading are outside the scope of this report.

Under the 2021 IBC, for screws used in framing connections, in order for the screws to be considered fully effective, the minimum spacing between screws must be 3 times the nominal screw diameter and the minimum edge distance must be 1.5 times the nominal screw diameter. Under the 2018, 2015 and 2012 IBC, for screws used in framing connections, in order for the screws to be considered fully effective, the minimum spacing between the fasteners and the minimum edge distance must be three times the nominal diameter of the screws, except when the edge is parallel to the direction of the applied force, the minimum edge distance must be 1.5 times the nominal screw diameter. When the spacing between screws is less than 3 times the nominal screw diameter, but at least 2 times the nominal screw diameter, the connection shear strength values in Table 4 must be reduced by 20 percent [Refer to Section B1.5.1.3 of AISI S240 (Section D1.5 of AISI S200 for the 2015 and 2012 IBC)].

For screws used in applications other than framing connections, the minimum spacing between the fasteners must be three times the nominal screw diameter and the minimum edge and end distance must be 1.5 times the nominal screw diameter.

Connected members must be checked for rupture in accordance with Section J6 of AISI S100 (Section E6 of AISI S100 for the 2015 IBC, Section E5 of AISI S100-07/S2-10 for the 2012 IBC).

4.2 Installation:

Installation of ITW Buildex TEKS® Self-drilling Fasteners must be in accordance with the manufacturer's published installation instructions and this report. The manufacturer's published installation instructions must be available at the jobsite at all times during installation.

The screws must be installed perpendicular to the work surface, using a screw driving tool. The installation speed for 1/4-inch TEKS/3, 1/4-inch TEKS/5, #12 TEKS/5.0 and #12 TEKS/5 screws should not exceed 1,800 rpm; the installation speed for all other screws should not exceed 2,500 rpm. The screw must penetrate through the supporting steel with a minimum of three threads protruding past the back side of the supporting steel.

5.0 CONDITIONS OF USE

The ITW Buildex TEKS® Self-drilling Fasteners described in this report comply with, or are suitable alternatives to what is specified in, those codes listed in Section 1.0 of this report, subject to the following conditions:

- 5.1 Fasteners must be installed in accordance with the manufacturer's published installation instructions and this report. In the event of a conflict between this report and the manufacturer's published installation instructions, this report governs.
- 5.2 The use of the screws in engineered steel deck diaphragms has not been evaluated and is outside the scope of this evaluation report.
- 5.3 Evaluation of screws subjected to cyclic or fatigue loading is outside the scope of this report. Applicable Seismic Design Categories must be determined in accordance with the code for the entire assembly constructed with the screws.
- 5.4 The allowable load values (ASD) specified in Section 4.1 for screws or for screw connections are not permitted to be increased for short-duration loads, such as wind or earthquake loads.
- 5.5 Drawings and calculations verifying compliance with this report and the applicable code must be submitted to the code official for approval. The drawings and calculations are to be prepared by a registered design professional when required by the statutes of the jurisdiction in which the project is to be constructed.
- 5.6 The screws are manufactured under a quality control program with inspections by ICC-ES.

6.0 EVIDENCE SUBMITTED

Data in accordance with the ICC-ES Acceptance Criteria for Tapping Screw Fasteners Used in Steel-to-steel Connections (AC118), dated January 2018 (editorially revised December 2020).

7.0 IDENTIFICATION

- 7.1 The ICC-ES mark of conformity, electronic labeling, or the evaluation report number (ICC-ES ESR-1976) along with the name, registered trademark, or registered logo of the report holder must be included in the product identification. [Electronic labeling is the ICC-ES web address (www.icc-es.org); specific URL related to the report; or the ICC-ES machine-readable code placed on the aforementioned items.]
- 7.2 In addition, ITW Buildex TEKS® Self-drilling Fastener heads are marked with "BX" as shown in Figures 1 through 3, and each box of fasteners has a label with the fastener description (model, point type, diameter and length) and the lot number.
- 7.3 The report holder's contact information is the following:

ITW BUILDEX
155 HARLEM AVENUE
GLENVIEW, ILLINOIS 60025
(800) 848-5611
www.itwbuildex.com
techsupport@itwccna.com

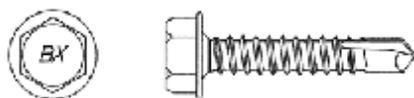


FIGURE 1—HEX WASHER HEAD (HWH)



FIGURE 2—HWH WITH SERRATIONS



FIGURE 3—PHILLIPS PAN HEAD

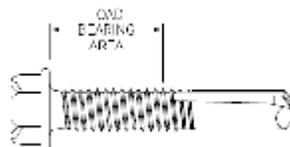


FIGURE 4—LENGTH OF LOAD-BEARING AREA

TABLE 1—TESK® SELF-DRILLING TAPPING SCREWS¹

DESCRIPTION (nom. size-tpi x length)	NOMINAL DIAMETER (inch)	HEAD STYLE	HEAD DIAMETER (inch)	DRILL POINT	DRILLING CAPACITY ³ (in.)		LENGTH OF LOAD BEARING AREA ⁴ (inch)	MINIMUM REQUIRED PROTRUSION (inch)	COATING
					Min.	Max.			
10-16 x 3/4"	0.190	HWH	0.400	TEKS/1	0.018	0.095	0.220	0.530	Climaseal
12-14 x 3/4"	0.216	HWH	0.415	TEKS/1	0.018	0.095	0.205	0.545	Climaseal
1/4-14 x 7/8"	0.250	HWH	0.415	TEKS/1	0.018	0.095	0.380	0.495	Climaseal
10-16 x 1/2"	0.190	Pan	0.365	TEKS/3	0.036	0.175	0.075	0.425	Climaseal
10-16 x 5/8"	0.190	Pan	0.365	TEKS/3	0.036	0.175	0.200	0.425	Climaseal
10-16 x 3/4"	0.190	Pan	0.365	TEKS/3	0.036	0.175	0.325	0.425	Climaseal
10-16 x 1/2"	0.190	HWH	0.400	TEKS/3	0.036	0.175	0.075	0.425	Climaseal
10-16 x 5/8"	0.190	HWH	0.400	TEKS/3	0.036	0.175	0.200	0.425	Climaseal
10-16 x 3/4"	0.190	HWH	0.400	TEKS/3	0.036	0.175	0.325	0.425	Climaseal
10-16 x 1"	0.190	HWH	0.400	TEKS/3	0.036	0.175	0.575	0.425	Climaseal
10-16 x 1"	0.190	Pan	0.365	TEKS/3	0.036	0.175	0.575	0.425	Climaseal
10-16 x 1 1/4"	0.190	HWH	0.400	TEKS/3	0.036	0.175	0.825	0.425	Climaseal
10-16 x 1 1/2"	0.190	HWH	0.400	TEKS/3	0.036	0.175	1.075	0.425	Climaseal
10-16 x 3/4"	0.190	HWH ²	0.435	TEKS/3	0.036	0.175	0.325	0.425	E-Zinc
12-14 x 3/4"	0.216	HWH	0.415	TEKS/3	0.036	0.210	0.200	0.550	Climaseal
12-14 x 1"	0.216	HWH	0.415	TEKS/3	0.036	0.210	0.450	0.550	Climaseal
12-14 x 1 1/4"	0.216	HWH	0.415	TEKS/2	0.036	0.210	0.550	0.700	Climaseal
12-14 x 1 1/2"	0.216	HWH	0.415	TEKS/2	0.036	0.210	0.800	0.700	Climaseal
12-14 x 2"	0.216	HWH	0.415	TEKS/3	0.036	0.210	1.450	0.550	Climaseal
12-14 x 2 1/2"	0.216	HWH	0.415	TEKS/3	0.036	0.210	1.950	0.550	Climaseal
12-14 x 3"	0.216	HWH	0.415	TEKS/3	0.036	0.210	2.450	0.550	Climaseal
12-14 x 4"	0.216	HWH	0.415	TEKS/3	0.036	0.210	3.450	0.550	Climaseal
1/4-14 x 3/4"	0.250	HWH	0.500	TEKS/3	0.036	0.210	0.150	0.600	Climaseal
1/4-14 x 1"	0.250	HWH	0.500	TEKS/3	0.036	0.210	0.400	0.600	Climaseal
1/4-14 x 1 1/4"	0.250	HWH	0.500	TEKS/3	0.036	0.210	0.650	0.600	Climaseal
1/4-14 x 1 1/2"	0.250	HWH	0.500	TEKS/3	0.036	0.210	0.900	0.600	Climaseal
1/4-14 x 2"	0.250	HWH	0.500	TEKS/3	0.036	0.210	1.400	0.600	Climaseal
1/4-14 x 2 1/2"	0.250	HWH	0.500	TEKS/3	0.036	0.210	1.900	0.600	Climaseal
1/4-14 x 3"	0.250	HWH	0.500	TEKS/3	0.036	0.210	2.400	0.600	Climaseal
1/4-14 x 4"	0.250	HWH	0.500	TEKS/3	0.036	0.210	3.400	0.600	Climaseal
1/4-14 x 3/4"	0.250	HWH ²	0.610	TEKS/3	0.036	0.210	0.150	0.600	Climaseal
1/4-14 x 1"	0.250	HWH ²	0.610	TEKS/3	0.036	0.210	0.400	0.600	Climaseal
12-24 x 7/8"	0.216	HWH	0.415	TEKS/4	0.125	0.250	0.325	0.550	Climaseal
12-24 x 1 1/4"	0.216	HWH	0.415	TEKS/4.5	0.125	0.375	0.575	0.675	Climaseal
12-24 x 1 1/4"	0.216	HWH	0.415	TEKS/5.0	0.125	0.500	0.450	0.800	Climaseal
12-24 x 1 1/2"	0.216	HWH	0.415	TEKS/5.0	0.125	0.500	0.700	0.800	Climaseal
12-24 x 1 1/2"	0.216	HWH	0.500	TEKS/5.0	0.125	0.500	0.700	0.800	Climaseal
12-24 x 2"	0.216	HWH	0.415	TEKS/5.0	0.125	0.500	1.200	0.800	Climaseal
12-24 x 1 1/4"	0.216	HWH	0.415	TEKS/5	0.125	0.500	0.375	0.875	Climaseal
12-24 x 1 1/2"	0.216	HWH	0.415	TEKS/5	0.125	0.500	0.625	0.875	Climaseal
12-24 x 1 1/2"	0.216	HWH	0.500	TEKS/5	0.125	0.500	0.625	0.875	Climaseal
12-24 x 2"	0.216	HWH	0.415	TEKS/5	0.125	0.500	1.125	0.875	Climaseal
1/4-28 x 3"	0.250	HWH	0.415	TEKS/5	0.125	0.500	2.150	0.850	Climaseal
1/4-28 x 4"	0.250	HWH	0.415	TEKS/5	0.125	0.500	3.150	0.850	Climaseal

For SI: 1 inch = 25.4 mm.

¹ Screw dimensions comply with ASME B18.6.4 (nom. size = nominal screw size, tip = threads per inch, length = inches).

² HWH with serrations.

³ Drilling capacity refers to the minimum and maximum total allowable thicknesses of steel the fastener is designed to drill through.

⁴ Length of load-bearing area is the total screw length minus the length from the screw point to the third full thread. See Figure 4.

TABLE 2—ALLOWABLE TENSILE PULL-OUT LOADS (P_{NOT}/Ω), pounds-force^{1, 2, 3, 4, 5}

Steel $F_u = 45$ ksi, Applied Factor of Safety, $\Omega=3.0$												
Screw Designation	Nominal Diameter (in.)	Design Thickness of Member Not in Contact with the Screw Head (in)										
		0.018	0.024	0.030	0.036	0.048	0.060	0.075	0.105	0.125	0.187	0.250
10-16	0.190	44	58	73	87	116	145	182	254	303	⁶	⁶
12-14, 12-24	0.216	50	66	83	99	132	165	207	289	344	515	689
¹ / ₄ -14, ¹ / ₄ -28	0.250	57	77	96	115	153	191	239	335	398	596	797

For SI: 1 inch = 25.4 mm, 1 lbf = 4.45 N, 1 ksi = 6.89 MPa.

¹For tension connections, the least of the allowable pull-out, pullover, and fastener tension strength found in Tables 2, 3, and 5, respectively, must be used for design.

²ANSI/ASME standard screw diameters were used in the calculations and are listed in the tables.

³The allowable pull-out capacity for other member thickness can be determined by interpolating within the table.

⁴To calculate LRFD values, multiply values in table by the ASD safety factor of 3.0 and multiply again with the LRFD Φ factor of 0.5.

⁵For $F_u = 58$ ksi, multiply values by 1.29; for $F_u = 65$ ksi, multiply values by 1.44.

⁶Outside drilling capacity limits.

TABLE 3—ALLOWABLE TENSILE PULLOVER LOADS (P_{NOV}/Ω), pounds-force^{1, 2, 3, 4, 5}

Steel $F_u = 45$ ksi, Applied Factor of Safety, $\Omega=3.0$													
Screw Designation	Nominal Diameter (in.)	Head or Integral Washer Diameter (in.)	Design Thickness of Member in Contact with the Screw Head (in)										
			0.018	0.024	0.030	0.036	0.048	0.060	0.075	0.105	0.125	0.187	0.250
Hex Washer Head (HWH)													
10-16	0.190	0.400	162	216	270	324	432	540	675	945	1125	⁶	⁶
12-14, 12-24	0.216	0.415	168	224	280	336	448	560	700	980	1167	1746	2334
12-24	0.216	0.500	203	270	338	405	540	675	844	1181	1406	2104	2813
¹ / ₄ -14	0.250	0.500	203	270	338	405	540	675	844	1181	1406	2104	2813
¹ / ₄ -14, ¹ / ₄ -28	0.250	0.415	168	224	280	336	448	560	700	980	1167	1746	2334
HWH with Serrations													
10-16	0.190	0.435	176	235	294	352	470	587	734	1028	1223	⁶	⁶
¹ / ₄ -14	0.250	0.610	203	270	338	405	540	675	844	1181	1406	2104	⁶
Phillips Pan Head													
10-16	0.190	0.365	148	197	246	296	394	493	616	862	1027	⁶	⁶

For SI: 1 inch = 25.4 mm, 1 lbf = 4.45 N, 1 ksi = 6.89 MPa.

¹For tension connections, the lower of the allowable pull-out, pullover, and fastener tension strength found in Tables 2, 3, and 5, respectively must be used for design.

²ANSI/ASME standard screw diameters were used in the calculations and are listed in the tables.

³The allowable pull-over capacity for other member thickness can be determined by interpolating within the table.

⁴To calculate LRFD values, multiply values in table by the ASD safety factor of 3.0 and multiply again with the LRFD Φ factor of 0.5.

⁵For $F_u = 58$ ksi, multiply values by 1.29; for $F_u = 65$ ksi, multiply values by 1.44.

⁶Outside drilling capacity limits.

TABLE 4—ALLOWABLE SHEAR (BEARING) CAPACITY (P_{Ns}/Ω), pounds-force^{1, 2, 3, 4, 5}

Steel $F_u = 45$ ksi, Applied Factor of Safety, $\Omega=3.0$															
Screw Designation	Nominal Diameter (in.)	Design Thickness of Member Not in Contact with the Screw Head (in)	Design Thickness of Member in Contact with the Screw Head (in)												
			0.018	0.024	0.030	0.036	0.048	0.060	0.075	0.105	0.125	0.187	0.250		
10-16	0.190	0.018	66	66	66	66	66	66	66	66	66	66	---	---	
		0.024	102	102	102	102	102	102	102	102	102	102	102	---	---
		0.030	111	143	143	143	143	143	143	143	143	143	143	---	---
		0.036	120	152	185	188	188	188	188	188	188	188	188	---	---
		0.048	139	168	199	228	289	289	289	289	289	289	289	---	---
		0.060	139	185	213	239	327	404	404	404	404	404	404	---	---
		0.075	139	185	231	251	337	427	564	564	564	564	564	---	---
		0.105	139	185	231	277	356	436	570	808	808	808	808	---	---
12-14 12-24	0.216	0.018	71	71	71	71	71	71	71	71	71	71	71	71	
		0.024	109	109	109	109	109	109	109	109	109	109	109	109	
		0.030	125	152	152	152	152	152	152	152	152	152	152	152	
		0.036	136	170	205	200	200	200	200	200	200	200	200	200	
		0.048	157	190	223	253	308	308	308	308	308	308	308	308	
		0.060	157	210	240	266	362	430	430	430	430	430	430	430	
		0.075	157	210	262	282	375	468	601	601	601	601	601	601	
		0.105	157	210	262	315	402	483	624	919	919	919	919	919	
		0.125	157	210	262	315	420	494	629	919	1094	1094	1094	1094	
		0.187	157	210	262	315	420	525	642	919	1094	1636	1636	1636	
1/4-14 1/4-28	0.250	0.018	76	76	76	76	76	76	76	76	76	76	76	76	
		0.024	117	117	117	117	117	117	117	117	117	117	117	117	
		0.030	142	164	164	164	164	164	164	164	164	164	164	164	
		0.036	156	193	215	215	215	215	215	215	215	215	215	215	
		0.048	182	218	253	283	331	331	331	331	331	331	331	331	
		0.060	182	243	276	300	406	463	463	463	463	463	463	463	
		0.075	182	243	304	322	424	521	647	647	647	647	647	647	
		0.105	182	243	304	365	461	544	694	1063	1063	1063	1063	1063	
		0.125	182	243	304	365	486	560	703	1063	1266	1266	1266	1266	
		0.187	182	243	304	365	486	608	731	1063	1266	1893	1893	1893	
0.250	182	243	304	365	486	608	759	1063	1266	1893	2531	2531			

For SI: 1 inch = 25.4 mm, 1 lbf = 4.45 N, 1 ksi = 6.89 MPa.

¹The lower of the allowable shear (bearing) and the allowable fastener shear strength found in Tables 4 and 5, respectively, must be used for design.

²ANSI/ASME standard screw diameters were used in the calculations and are listed in the tables.

³The allowable bearing capacity for other member thickness can be determined by interpolating within the table.

⁴To calculate LRFD values, multiply values in table by the ASD safety factor of 3.0 and multiply again with the LRFD Φ factor of 0.5.

⁵For $F_u = 58$ ksi, multiply values by 1.29; for $F_u = 65$ ksi, multiply values by 1.44.

TABLE 5—FASTENER STRENGTH OF SCREWS^{1, 2, 3, 4, 5}

SCREW DESIGNATION	DIAMETER (in.)	ALLOWABLE FASTENER STRENGTH		NOMINAL FASTENER STRENGTH	
		Tensile, P_{ts}/Ω (lbf)	Shear, P_{ss}/Ω (lbf)	Tensile, P_{ts} (lbf)	Shear, P_{ss} (lbf)
10-16	0.190	885	573	2654	1718
12-14	0.216	1184	724	3551	2171
12-24	0.216	1583	885	4750	2654
1/4-14	0.250	1605	990	4816	2970
1/4-28	0.250	1922	1308	5767	3925

For SI: 1 inch = 25.4 mm, 1 lbf = 4.45 N.

¹For tension connections, the least of the allowable pull-out, pullover, and fastener tension strength found in Tables 2, 3, and 5, respectively, must be used for design.

²For shear connection, the lower of the allowable shear (bearing) and the allowable fastener shear strength found in Table 4 and 5, respectively, must be used for design.

³See Section 4.1 for fastener spacing and end distance requirements.

⁴Nominal strengths are based on laboratory tests

⁵To calculate LRFD values, multiply nominal strength values by the LRFD Φ factor of 0.5.

DIVISION: 05 00 00—METALS

Section: 05 05 23—Metal Fastenings

REPORT HOLDER:

ITW BUILDDEX

EVALUATION SUBJECT:

ITW BUILDDEX TEKS® SELF-DRILLING FASTENERS

1.0 REPORT PURPOSE AND SCOPE

Purpose:

The purpose of this evaluation report supplement is to indicate that the ITW Buildex TEKS® Self-Drilling Fasteners, described in ICC-ES evaluation report [ESR-1976](#), have also been evaluated for compliance with the codes noted below as adopted by the Los Angeles Department of Building and Safety (LADBS).

Applicable code editions:

- 2020 *City of Los Angeles Building Code* (LABC)
- 2020 *City of Los Angeles Residential Code* (LARC)

2.0 CONCLUSIONS

The ITW Buildex TEKS® Self-Drilling Fasteners, described in Sections 2.0 through 7.0 of evaluation report [ESR-1976](#), comply with the LABC Chapter 22, and the LARC, and are subject to the conditions of use described in this supplement.

3.0 CONDITIONS OF USE

The ITW Buildex TEKS® Self-Drilling Fasteners described in this evaluation report supplement must comply with all of the following conditions:

- All applicable sections in the evaluation report [ESR-1976](#).
- The design, installation, conditions of use and identification of the ITW Buildex TEKS® Self-drilling Fasteners are in accordance with the 2018 *International Building Code*® (IBC) provisions noted in the evaluation report [ESR-1976](#).
- The design, installation and inspection are in accordance with additional requirements of LABC Chapters 16 and 17, as applicable.
- Under the LARC, an engineered design in accordance with LARC Section R301.1.3 must be submitted.

This supplement expires concurrently with the evaluation report, reissued July 2022 and revised January 2023.

DIVISION: 05 00 00—METALS**Section: 05 05 23—Metal Fastenings****REPORT HOLDER:****ITW BUILDEX****EVALUATION SUBJECT:****ITW BUILDEX TEKS® SELF-DRILLING FASTENERS****1.0 REPORT PURPOSE AND SCOPE****Purpose:**

The purpose of this evaluation report supplement is to indicate that the ITW Buildex TEKS® Self-Drilling Fasteners, addressed in ICC-ES evaluation report ESR-1976, have also been evaluated for compliance with the codes noted below.

Applicable code editions:

- 2020 *Florida Building Code—Building*
- 2020 *Florida Building Code—Residential*

2.0 CONCLUSIONS

The ITW Buildex TEKS® Self-Drilling Fasteners, described in Sections 2.0 through 7.0 of ICC-ES evaluation report ESR-1976, comply with the *Florida Building Code—Building* and *Florida Building Code—Residential*. The design requirements must be determined in accordance with the *Florida Building Code—Building* or the *Florida Building Code—Residential*, as applicable. The installation requirements noted in ICC-ES evaluation report ESR-1976 for the 2018 *International Building Code*® meet the requirements of the *Florida Building Code—Building* or *Florida Building Code—Residential*, as applicable.

Use of the ITW Buildex TEKS® Self-Drilling Fasteners in accordance with the High-Velocity Hurricane Zone provisions of the *Florida Building Code—Building* and the *Florida Building Code—Residential* has not been evaluated and is outside the scope of this supplement report.

For products falling under Florida Rule 61G20-3, verification that the report holder's quality assurance program is audited by a quality assurance entity approved by the Florida Building Commission for the type of inspections being conducted is the responsibility of an approved validation entity (or the code official when the report holder does not possess an approval by the Commission).

This supplement expires concurrently with the evaluation report, reissued July 2022 and revised January 2023.