DSA

IR PC-7

PRE-CHECK (PC) DESIGN CRITERIA FOR STEEL CANTILEVERED CANOPY STRUCTURES: 2025 CBC

Disciplines: All

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Division of the State Architect (DSA) documents referenced within this publication are available on the DSA Forms or DSA Publications webpages.

PURPOSE

This Interpretation of Regulations (IR) clarifies requirements relating to pre-check (PC) submittals to promote uniform statewide criteria for code compliance in the design and plan review of steel cantilevered canopy structures for projects under DSA jurisdiction. The PC Design Criteria documents were created by DSA as a means for the responsible engineer to demonstrate code compliance when developing and submitting construction documents for DSA review.

The provisions of this IR are intended to be a tool to identify and highlight both common and unique, critical and/or overlooked code requirements that must be considered and incorporated into the design, as applicable, to provide a complete and consistent set of construction documents accepted at all DSA regional offices. This IR does not contain all code requirements that may be necessary to provide a complete structural design. Other methods proposed by design professionals to solve a particular issue may be considered by DSA and reviewed for code and regulation compliance, subject to concurrence of the DSA Codes and Standards Unit. For methods not specifically prescribed in the code, see California Building Code (CBC) Section 104.2.3.

Appendix A below is provided as a guide to assist design professionals and DSA plan reviewers when preparing and reviewing site-specific project applications that incorporate PC steel cantilevered canopy structures designed in accordance with this IR.

SCOPE

The provisions of this IR apply to 2025 PC plans for new steel cantilevered canopy structures submitted to DSA under the 2025 CBC. Steel cantilevered canopy structures are defined as exterior single-story structures with open sides and a roof surface consisting of a deck, solar panels or both. These structures are often configured in “T,” offset “T,” or gable geometries and are sometimes referred to as “carports,” “canopies,” “shade structures” or “lunch shelters.”

This IR includes the design of steel cantilevered canopy structures that support PV panels for both elevated and ground-mounted systems. CBC Chapter 2 defines PV support structures as “Photovoltaic (PV) Support Structures, Elevated” when there is usable space underneath and a minimum clear height of 7’-6”, or “Photovoltaic (PV) Panel Systems, Ground-Mounted” when there is no usable space underneath and is installed directly on the ground. Refer to Section 1.6 below for definitions of usable space and occupancy to determine the classification of the PV support structure. Requirements specific to ground-mounted PV panel structures with no occupancy beneath are in Section 9 below. This document does not address moment frame structures that resist lateral loads primarily through the rigidity of beam-to-column connections.

PC DESIGN CRITERIA FOR STEEL CANTILEVERED CANOPY STRUCTURES: 2025 CBC

As noted in Bulletin (*BU) 18-01: Applicability of Pre-Check (PC) Design Criteria for Non-PC Projects*, these provisions shall also be considered and incorporated in site-specific submittals for structures of the same project type, even if not part of a PC application.

BACKGROUND

The PC approval process is intended to streamline DSA plan review by providing a procedure for approving the design of commonly used structures prior to the submission of plans to DSA for a construction project. The PC approval process allows designers to incorporate designs for structures that have already been "pre-checked" by DSA into their plans for actual site-specific construction projects. The design criteria provided in this document are neither regulations nor law and are not appropriate for verbatim inclusion in project specifications. The design professional in responsible charge is responsible for specifying and detailing the requirements for each project.

Additional information regarding the design and site application of PC structures and solar photovoltaic and thermal systems can be found in the following documents:

- Procedure *(PR) 07-01: Pre-Check Approval*
- Policy *(PL) 07-02: Over-the-Counter Review of Projects Using Pre-Check Approved Designs*
- *IR 16-8: Solar Photovoltaic and Thermal Systems Review and Approval Requirements*
- *IR 31-1: Construction and Installation of Free-Standing, Open-Sided Shade Structures on Public School, and Community College Campuses*

1. GENERAL

1.1 Pre-Check Submission Requirements

Refer to PR 07-01 for a detailed list of items required for all PC applications. The documents required to be submitted for PC approval are listed on form *DSA 3: Project Submittal Checklist*. Site-specific information is not necessary as that information will be defined when a specific construction project is submitted for DSA review.

1.2 Cover Sheet and General Notes

1.2.1 In accordance with PR 07-01 Section 1.4.2, the first sheet(s) of the PC drawings shall include a design information section that defines the basis of the PC design. Refer to PR 07-01 Appendices B and C and the remainder of this IR for required content of the design information section.

1.2.2 The PC construction documents shall include complete and comprehensive general notes and/or specifications as required for construction and inspection. It is common for PC construction documents to consist of drawings only without a book specification or project manual. Refer to PR 07-02 Appendix B, Footnote 8. In this case, the PC drawings shall include information that might otherwise be communicated in a project manual or book specification. For each primary material or group of materials, the following information shall be specified in the construction documents when applicable:

1.2.2.1 Required material properties, including compliance with American Society for Testing and Materials (ASTM) specifications when applicable.

1.2.2.2 Proprietary products name, manufacturer, and evaluation report number. Refer to Section 1.12 below.

1.2.2.3 Quality control performed by the supplier.

1.2.2.4 Standards for the execution of the work, including associated tolerances. References to recognized standards are acceptable.

1.2.2.5 Required qualifications of personnel performing the work for each applicable trade.

1.2.2.6 Product and material finish requirements for weather protection or safety.

1.2.2.7 Quality assurance tests not covered by Section 1.3 below, including frequency requirements, (and citation of ASTM standards when applicable).

1.3 Structural Tests and Special Inspections

The PC drawings shall include example form(s) *DSA 103: List of Structural Tests and Special Inspections*. See PR 07-01, Section 1.5 for additional information.

1.3.1 Example form(s) DSA 103 will be used as a guide to develop the form DSA 103 for the site-specific project. Example form(s) on the PC drawings will be crossed out when the form DSA 103 is provided with the site-specific application.

1.3.2 The example form(s) DSA 103 shall include both in-plant and on-site testing and inspection requirements as applicable. Manufacturers shall be involved in the coordination of the in-plant testing and inspection with the project inspector (PI) and Laboratory of Record (LOR) of the site-specific project using the PC design prior to commencing fabrication.

1.3.3 Only the site-specific form DSA 103 can opt to exempt structural tests and special inspections; therefore, the exemptions appendix of the example form(s) DSA 103 shall not be included on the PC drawings. The applicability of exemptions may be discussed during plan review for site-specific project scope, must be justified by the project design professional, and is subject to DSA review and approval. Refer to Appendix A below for additional information.

1.3.4 For projects involving solar installation, add a line item to the example form(s) DSA 103 for installation verification testing, and special inspection of solar panel attachments utilizing pretensioned bolts (e.g., bolts designed for clamp load; see Section 2.3.1 below). Add a line item to the example form(s) DSA 103 for material identification testing of solar panel attachment fasteners (see Section 2.3.1.2 below).

1.4 Options and Variations

The PC drawings shall provide checkboxes for options and variations if there is more than one configuration or design criteria**.** See PR 07-01 Section 2 for more details, including the maximum number of options permitted. Refer to Appendix D below for a guide to assist designers and reviewers with defining an option within a steel cantilevered canopy structure PC.

1.5 Design Parameters

The PC documents shall state design Information on the cover sheet (and subsequent sheets as necessary) as defined in PR 07-01 Section 2.4 and Appendix B. If the PC includes design variations for multiple tiers or levels of the same design parameter that design information should be presented in a checklist format and provide general direction to future users (i.e., design professionals and plan reviewers) for the application of the PC to site-specific projects. Additionally, refer to and coordinate with PL 07-02 Section 3, which summarizes common site-specific parameters to be verified at over-the-counter (OTC) plan reviews.

1.6 Risk Category and Occupant Load

The PC drawings shall indicate the maximum Risk Category (RC) the structure is designed for in the design information section on the cover sheet.

1.6.1 The design information section shall include a note requiring the intended *Use and Occupancy* be specified on the site-specific application drawings, so the DSA plan reviewer can verify the RC of the PC structure as it applies to the site in accordance with CBC Section 1604A.5 and 1604A.5.2. For this purpose, the PC drawings shall include a Code Analysis table with columns for the definition of Use and Occupancy classification per CBC Chapter 3, Occupant Load Factor (OLF) per CBC Table 1004.5, and total occupant load, to be completed by the design professional at time of the site-specific application. The site-specific RC will correspondingly be determined from the site-specific occupant load in accordance with CBC Table 1604A.5. Refer to Appendix A below for additional information.

1.6.2 All steel cantilevered canopy structures with a minimum clear height of 6'-8" shall be considered to have usable space underneath and have a specific occupancy unless the area is fenced to prohibit access by teachers, students, and visitors.

1.6.3 All ground-mounted PV structures shall be assigned to Risk Category II minimum per CBC Section 1604A.5.2, Item 2.

1.7 Flood Zone

The PC design shall comply with CBC Section 1612A and *PR 14-01: Flood Design and Project Submittal Requirements*.

1.7.1 The design information section shall include a note stating that when the site-specific project is located in a flood zone other than Zone X, a letter stamped and signed by a geotechnical engineer is required to validate the applicability of the allowable soil values listed on the PC drawings. This note may include an exemption for the validation letter for projects located in Zone D (undefined) if the applicant provides either of the following:

1.7.1.1 Evidence from the local jurisdiction or a qualified design professional confirming the site is not in a flood hazard zone.

1.7.1.2 Geotechnical report written for improvements on the same campus and in accordance with the current CBC that acknowledges the flood hazard but confirms it does not result in a reduction of soil capacity values. ←

1.7.2 The location of electrical components shall conform to American Society of Civil Engineers (ASCE) Standard 24: Flood Resistant Design and Construction (ASCE 24), Section 7.2.

1.8 Geohazard Report

It is recommended the design information section state that geohazard reports are not required for open metal site structures provided they do not exceed 4,000 square feet (SF) in plan area and are not located within a mapped geologic hazard zone. Refer to *IR A-4: Geohazard Report Requirements*, Sections 3.5 and 4. ←

1.9 Weather Protection

The PC drawings shall specify adequate weather protection for all weather-exposed steel members (i.e., structural steel and cold-formed steel) in accordance with CBC Section 2201A.3.

1.9.1 Structural steel shall comply with one of the following:

1.9.1.1 Hot dip galvanized in accordance with ASTM A123 or ASTM A153.

1.9.1.2 Painted with zinc-rich primer (undercoat and finish coat) or an equivalent paint system specified on the PC construction documents and approved by DSA.

1.9.2 Cold-formed steel members shall be 55 percent aluminum-zinc alloy coated per ASTM A792 standard in accordance with the American Iron and Steel Institute (AISI) S240: North American Standard for Cold-Formed Steel Structural Framing, Table A4-1, (CP 90 coating designation). Tube members formed from uncoated steel shall be hot-dip galvanized in accordance with ASTM A1057 and complying with a minimum ZT 50 coating designation.

1.9.3 Exposed steel fasteners, including cast-in-place anchor bolts/rods, shall comply with one of the following:

1.9.3.1 Stainless steel in accordance with American Institute of Steel Construction (AISC) 379: specification for Structural Stainless Steel Buildings, Section A3.

1.9.3.2 Hot-dip galvanized in accordance with ASTM A153 or ASTM F2329.

1.9.3.3 Protected with corrosion-preventive coating that demonstrated no more than 2 percent of red rust in minimum 1,000 hours of exposure in salt spray test per ASTM B117. Zinc-plated fasteners do not comply with this requirement. Examples of proprietary coatings that do comply with the 1000-hour requirement include, but are not necessarily limited to: Quik Guard by Simpson, Kwik-Cote by Hilti, Stalgard by Elco, vistaCorr by SFS intec, etc.

1.9.4 Post-installed anchors used for exterior exposure shall comply with the requirements of the evaluation report.

1.10 Sheet Index

The PC drawings shall include a sheet index. When a PC includes multiple major options such that not all sheets are applicable to a given site-specific project application based on the option being used, the sheet index shall include check boxes. When the PC drawings are incorporated into a site-specific application, the submitted sheets will be identified by marking the check boxes (i.e., it is not necessary to strike out sheets that are not applicable). See PR 07-01, Appendix E for additional information.

1.11 Stamps

The PC drawings shall include the following**:**

1.11.1 2025 CBC PC Stamp per PR 07-01 Section 1.4.1.

1.11.2 Two blank areas on each PC sheet title block as indicated in *PR 18-04: Electronic Plan Review for Design Professionals*, Section 1: one for the PC Identification Stamp and one for the future site-specific Identification Stamp.

1.12 Structural Product Acceptance

All structural products shall meet the requirements set forth in *IR A-5: Product and Material Acceptance Based on a Valid Evaluation Report*. Code-based engineering calculations to substantiate the adequacy of a manufactured product will be considered by DSA.

1.13 Structural Analysis Software

The design professional shall provide to DSA sufficient documentation to verify and substantiate input and output of all software used for design, including but not limited to:

1.13.1 Electronic structural models.

1.13.2 Input and output data in PDF file format.

Large files can be reduced in size, and print outs may be limited to the worst-case design output for each profile if they are produced with common structural software, and the worst-case design output data is identified in the calc package by the model and load case where it occurs for verification by the reviewer.

Where custom spreadsheets are used, the formulas used shall be adequately identified and verified per Section 1.13.5 below. Worst case designs for each profile shall be included in the calculation package, and all other designs shall be included as a reference document unless other arrangements are coordinated with the DSA review team.

1.13.3 Description of input and output including schematic framing plans, member and joint labeling, member loads, checks of elements, cables, connections, etc.

1.13.4 Derivation of loads.

1.13.5 Hand calculations as necessary to validate results.

2. SOLAR PANEL AND PANEL ATTACHMENT REQUIREMENTS

When a steel cantilevered canopy structure includes PV panels, the panels shall comply with requirements in IR 16-8. See Section 2.1 below for additional solar panel requirements, Section 2.2 below for solar panel attachment design requirements and Section 2.3 below for field testing requirements of solar panel attachments.

2.1 Solar Panel Requirements

Solar panel orientation (portrait and/or landscape layouts), anchorage point location, and installation tolerance range shall be specified on the drawings for each configuration. Panel connection geometry shall be consistent with tests from UL 1703: Flat-plate Photovoltaic Modules and Panels or with tests from both UL 61730-1: Photovoltaic (PV) Module Safety Qualification - Part 1: Requirements for Construction and UL 61730-2: Photovoltaic (PV) Module Safety Qualification - Part 2: Requirements for Testing (and tests from UL 2703: Mounting Systems, Mounting Devices, Clamping/Retention Devices and Ground Lugs for Use with Flat-plate Photovoltaic Modules and Panels, if utilized). If horizontal slip joints (e.g., thermal expansion joints) in framing members are present, solar panels shall not span across nor be connected on opposing sides of the slip joints.

2.1.1 The PC drawings shall specify overall solar panel dimensions and fully dimensioned frame configuration of panel assumed in design of structure, including height, length, and width. ←

2.1.2 The PC drawings shall specify the required solar panel load rating in pounds per square foot (psf) to meet the demand loads for the PC. Refer to IR 16-8 Section 1.2.4 for more information about solar panel load ratings.

Panel anchorage details for each panel-to-purlin connector assembly shall be fully detailed on the PC drawings and in compliance with the requirements in Section 2.2 below. Anchorage details shall specify fasteners, number of fasteners, and anchor product information if used (manufacturer, model number, capacity, etc.) and installation requirements (maximum and minimum torque, tightening of set screws, etc.). Solar panel anchorage design and detailing shall be included in the PC.

2.1.3 The PC drawings shall contain the following notes:

2.1.3.1 "Solar panels shall be listed and labeled in accordance with UL1703 or with both UL 61730-1 and UL 61730-2 per CBC Section 3111.3.1 for the panel orientations shown on the PC drawings."

2.1.3.2 "The load ratings for the solar panels selected by the contractor shall meet or exceed the actual design wind pressures shown on the PC drawings."

PC DESIGN CRITERIA FOR STEEL CANTILEVERED CANOPY STRUCTURES: 2025 CBC

2.1.3.3 "The owner's site professional shall provide product documentation from the solar panel supplier, including panel dimensions and load ratings, to the PC design professional for review prior to submittal to DSA for plan review. Documentation shall identify panel load ratings and safety factor, and the number of fasteners required to achieve the rating. Upon acceptance, the PC design professional shall provide a statement to the owner's site professional that the solar panels are in compliance with the approved PC drawings. The owner's site professional shall submit the statement and panel documentation to DSA with the plan review package. If the solar panel type and size do not meet the approved PC plan requirements, then that panel will not be permitted as a substitution until a revision is made to the PC permitting such panel."

2.2 Solar Panel Attachment Design Requirements

Solar panels installed on open-frame structures without metal-deck diaphragms shall be anchored to the structural member (purlins) for the design wind and seismic forces based upon the panel tributary area to each connector. Fasteners shall resist combined tension (from wind uplift forces and resulting prying action) and shear (from deflection/catenary action of the panel from wind uplift), and tensile fatigue. Unless in-plane roof diaphragm (racking) deflection is limited in accordance with Section 2.2.6 below, the design shall also consider shear due to racking from in-plane roof seismic forces. The attachment design shall comply with Method A, B, or C as noted below:

2.2.1 Method A

Panels attached directly to purlins with high-strength pretensioned bolts.

2.2.1.1 Bolts shall be sized in accordance with Sections 6.4, 6.5 and 6.6 of UL 2703. Vertical (i.e., Out-of-plane) load testing in accordance with Section 21 of UL 2703 of the panel/fastener assembly will not be required.

2.2.1.2 Requirements of a high-strength pretensioned bolt include all of the following: a minimum tensile strength of 95 kips per square inch (ksi), a minimum yield strength of 60 ksi, a minimum elongation of 14 percent, and a minimum reduction in area of 35 percent (ASTM F593C bolts would be an example of a high-strength bolt meeting this criteria).

2.2.1.3 Minimum and maximum torque (or upper- and lower-bound pretensioned force) shall be specified on the PC drawings in accordance with the bolt manufacturer's specifications. Grounding devices as described in Section 8 of UL 2703 may be utilized provided bonding is achieved at or below the maximum torque.

2.2.2 Method B

Panels attached to purlins with standard bolts in oversized holes in purlin flange; or panels attached to purlins with clamps that rely on friction, interlock, or overlap (i.e., solar panels are not directly bolted or screwed to the purlins).

2.2.2.1 For bolts in oversized holes, the hole diameter shall accommodate in-plane deflection due to racking, assuming panel remains rigid, without inducing shear on the bolt. Where clamps are used, they must be anchored to purlins with bolts, pretensioned bolts or screws designed to meet the tension, shear, and fatigue design requirements described above (Compliance with Sections 6.4, 6.5 and 6.6 of UL 2703 will be considered to satisfy these requirements. See note below regarding fatigue). If bolts or clamps are fastened to an angle clip attached to purlin, the design of the clip and attachment to the purlin must also account for shear, tension, and prying.

Note: When fatigue is calculated, allowable tensile stresses of the fastener and base metal shall be evaluated for a minimum of 200,000 cycles. In no case shall the design tensile stress exceed one-half the allowable design stresses of the fastener and base metal.

2.2.2.2 Vertical load testing in accordance with Section 21 of UL 2703 of the panel/fastener assembly is required prior to approval of the PC. The purlin segments used for testing, if not full-length per the PC drawings, shall have a defined stiffness that results in similar (or greater) beam curvature and torsional rotation as the full-length purlins and must extend far enough beyond the panel points of connection to allow deformations in the purlin flanges and panel frame flanges to develop. If blocking between purlins is required on the PC drawings, then blocking may be included in the test specimen to reflect actual use conditions. The panel used in the test shall be representative of the minimum dimensions and frame parameters specified on the PC drawings. The test protocol shall be submitted to DSA for acceptance prior to conducting the test.

2.2.3 Method C

Panels utilizing attachment methods other than Method A or B above must be provided with a safety device (e.g., safety cable) independent of the primary panel/fastener assembly. The safety device must restrain the panels from dislodging from the structure and becoming a falling hazard. The safety device must be designed for all applicable forces when the device is engaged, and the primary attachment method has failed. Vertical load testing in accordance with Section 21 of UL 2703 of the panel/fastener assembly will not be required.

2.2.4 Attachment of solar panels installed over structures with metal deck diaphragms shall comply with requirements of Section 2.2. Alternatively, panels are permitted to be supported by and fastened to a rack or rail system (or equivalent) with vertical load testing in accordance with UL 2703. Attachment of rack or rail system to structure shall comply with recommendations of Structural Engineers Association of California (SEAOC) Report PV-1: Structural Seismic Requirements and Commentary for Rooftop Solar Photovoltaic Arrays (SEAOC PV-1), SEAOC Report PV-2: Wind Design for Solar Arrays (SEAOC PV-2) and the CBC.

2.2.5 For all attachments that include bolts, the design shall include a mechanism for retention of nuts and prevention of loosening thereafter. Nylon lock nuts per ASME B18.21.1 may be used in the connection, but helical spring washers (i.e., split washers) shall not be used anywhere in the fastener assembly.

2.2.6 Cyclic Testing of Solar Panel Fastener Assembly

Each panel fastener assembly described in Sections 2.2.1, 2.2.2 or 2.2.3 above shall require in-plane cyclic (i.e., racking) testing in accordance with a DSA accepted protocol (See Appendix B below). The cyclic test shall demonstrate the connector and panel do not experience slippage, shifting or distress through the calculated differential wind or seismic in-plane roof drift between adjacent purlins for the panel orientation on the PC drawings. If both panel orientations are shown on the PC drawings, the test need only be performed for the orientation that yields the more severe requirements.

Exception: Cyclic testing of the panel fastener assembly is exempt from structures where the roof plane is equipped with metal deck roof diaphragm (See Section 4.3.4 below), in-plane roof diagonal bracing (See Section 4.3.3 below), or deflection-limiting structural framing. In each case, the maximum inelastic response displacement (e.g., elastic deflection x C_d) at the most extreme point in the roof relative to the top of the column is limited to $0.01L_x$. L_x is the dimension perpendicular to the column line (datum zero) to the extreme framing member. This deflection is a measure of the diaphragm in-plane distortion relative from the column line of resistance to the most extreme diaphragm roof edge. See Appendix C below for example calculations.

2.2.7 Alternate Design Method in Lieu of Testing

As an alternate to vertical and/or lateral cyclic testing of specific panel and bolt or clamp assemblies per Section 2.2 above (Method A, B or C), and Section 2.2.6 above, with DSA pre-approval of the method, a Finite Element Method (FEM) analysis of the complete structure/panel assembly under seismic and wind loads accounting for rotations and displacements of all elements of all members, including the solar panels, may be performed to demonstrate adequacy of the structure, solar panels and their attachments.

In lieu of a FEM analysis for wind, a wind tunnel test of an appropriate model of the complete structure/panel assembly may be performed. The FEM analysis shall account for the rotations and displacements of the solar panels to demonstrate the adequacy of the solar panel attachments.

2.2.8 Summary of Testing Requirements

See Figure 2.2 below for a summary of testing requirements as mentioned in Section 2.2 above and Section 4.6 below. Section 2.2.7 above denotes requirements for an alternate design method in lieu of testing.

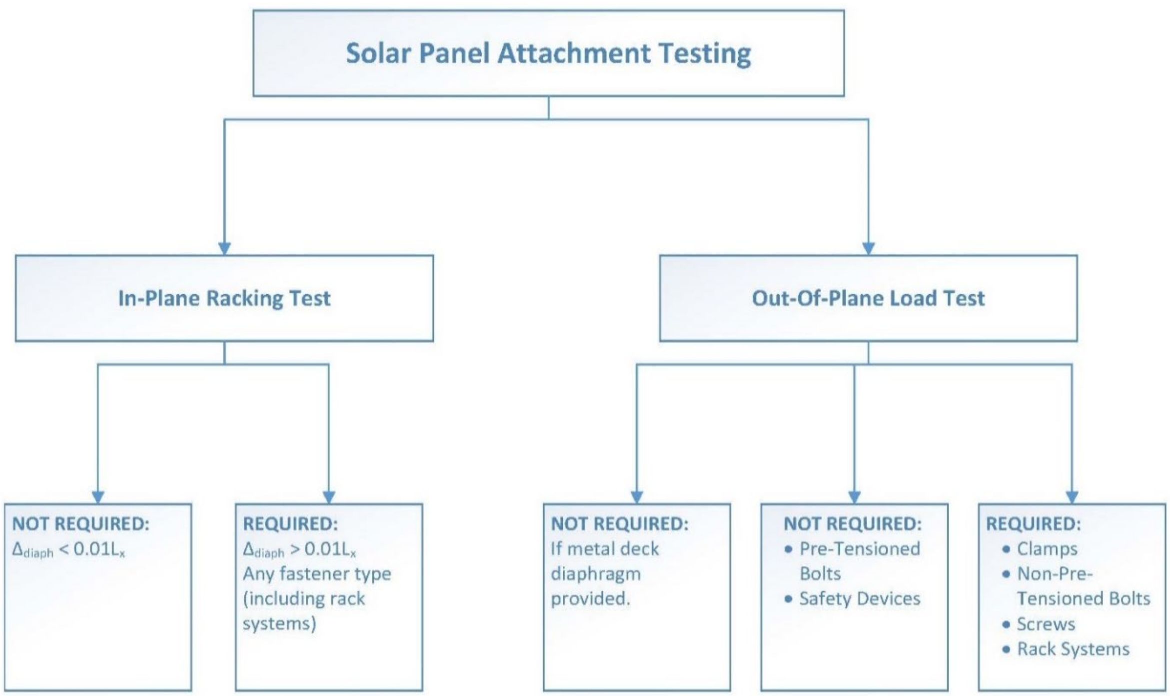


Figure 2.2: Summary of Testing Requirements for Solar Panel Attachments

2.3 Field Testing Requirements for Solar Panel Fasteners

2.3.1 Pretensioned Fasteners

Where pretensioned fasteners are used to attach solar panels to the PV structure, the field testing and special inspection procedures in Appendix E shall be followed and directions included on the PC drawings.

2.3.1.1 Plan Notes

Where pretensioned panel fasteners (e.g., bolts designed for clamp load) are specified, the PC drawings shall clearly state as a block note on the cover sheet with a heading of "PV Panel Fastener Installation Procedure":

"Prior to pretensioned panel fastener installation, the contractor shall submit to the professional in responsible charge for review and acceptance a detailed pretensioned panel fastener installation procedure outlining provisions to ensure all pretensioned panel fasteners are installed and torqued within the specified minimum and maximum torque range. A copy of the responsible design professional-accepted installation procedure shall be provided to the special inspector and project inspector prior to commencing panel fastener installation."

Note: The fastener installation procedure may also be submitted at time of site-specific application and included on the site-specific contract documents.

2.3.1.2 Plan Notes for Installation Verification Testing and Special Inspection

The PC drawings having pretensioned panel fastener connections shall specify on applicable panel connection detail(s):

"Special inspection and torque testing of pretensioned panel fastener installation shall be performed by a qualified representative of the laboratory of record (LOR) in accordance with Appendix E of *IR PC-7: PC Design Criteria for Steel Cantilevered Canopy Structures."*

2.3.2 All Other Connections

Other connections not requiring pretensioned panel fasteners do not require special inspection; all connections not receiving special inspection shall be inspected by the PI. All panel connection detail(s) on the PC drawings that do **NOT** utilize pretensioned fasteners shall include the following note:

"The panel connections detailed here do not require pretensioned fasteners and therefore do not require special inspection. These connections shall be inspected by the Project inspector (PI), who shall provide detailed daily inspection reports in accordance with IR 17-12."

3. GRAVITY LOAD DESIGN

3.1 Dead Load

3.1.1 The design for dead loads shall comply with CBC Section 1606A.

3.1.2 The dead load of solar systems (where occurs), roof decking (where occurs), electrical components, and fire sprinklers (where occurs) shall be considered in the design of the structure. For projects with solar, refer to IR 16-8, and SEAOC Report PV-3: Gravity Design for Rooftop Solar Photovoltaic Arrays (SEAOC PV-3), Section 4.

3.2 Live Load

3.2.1 The design for live loads shall comply with CBC Section 1607A and SEAOC PV-3 Section 5.5.

3.2.2 For structures of open grid framing and no roof sheathing or decking (e.g., canopies over parking and shade structures supporting solar panels), the following two separate live load conditions shall be applied in combination with other applicable loads:

- 12 psf uniform roof live load per CBC Section 1607A.14.3.3 without solar panel dead load. The distributed live load shall be applied to members based on their tributary areas as if sheathing were installed.

- 300 lbs. concentrated roof live load per CBC Table 1607A.1 with solar panel dead load.

3.2.2.1 As a condition for use of this reduced loading condition, the following note shall be shown on the PC drawings: "No future roof decking or sheathing may be applied on the open grid framing."

3.2.3 For structures with roof sheathing or decking, live loads shall be in accordance with CBC Table 1607A.1, including the 300 lbs. concentrated load for maintenance workers, and combined with solar panel dead load.

3.2.4 Unbalanced live load shall be included in design of structure.

3.3 Snow Load

3.3.1 The design information section shall state the snow and ice loads accounted for in the PC design. The PC drawings shall indicate 0 (zero) psf if the design does not account for snow or ice loads.

3.3.2 If the structure is designed for snow load, the design information section of the PC drawings shall include a note the same as or similar to the following: "Site application design professional and DSA plan reviewer shall verify the structure to be located at least "xx" feet from any adjacent higher structure" where the distance "xx" is calculated and stated by the PC applicant. Refer to ASCE 7 Section 7.7. If the horizontal separation from a higher structure is less than 20 feet and six times the vertical dimension separating the roofs, snow drift analysis shall be provided by the PC applicant, and the project is not eligible for OTC submittal.

3.3.3 Attachment of solar panels to the structure shall be designed to resist the shearing force from snow sliding down due to roof slope.

3.3.4 Effective seismic weight shall include snow load in accordance with ASCE 7 Section 12.7.2. See IR 16-8 Section 2.3.

3.3.5 Unbalanced roof snow loads shall be in accordance with ASCE 7 Section 7.6.

3.4 Deflection Limits for Framing Members

Purlins and girders not supporting solar panels shall satisfy deflection limits for gravity, wind and seismic loads per CBC Table 1604A.3 for "Roof members.". Purlins and girders supporting solar panels shall satisfy deflection limits for gravity, wind, and seismic loads per CBC Table 1604A.3 for "Roof members", footnote "c" (glass supports). Per CBC Section 2403, the maximum total deflection from all load combinations shall not exceed L/175. The reduced deflection limits in CBC Table 1604A.3 footnote "a" are not permitted for members supporting loads from solar panels. Members with tributary area less than 700 SF require use of "component and cladding" wind loads. See also footnote "f" and additional requirements of CBC Section 1604A.3.7 for framing supporting glass.

4. LATERAL LOAD DESIGN

This section is based on design requirements for elevated structures with occupancy below. For ground-mounted systems with no occupancy below, see Section 9 below.

4.1 Analysis Methods

A detailed analysis for lateral loads shall be performed on the structure for gravity plus lateral loads for member design and to determine the horizontal drift at the extreme edges of the structure in both orthogonal directions, including rotation of the roof plane. If the structure has eccentricities that would result in a significant torsional response and/or out-of-plane loads from rod bracing that cannot be adequately modeled in a two-dimensional (2D) analysis, a threedimensional (3D) analysis shall be required. The 3D analysis shall minimally include the columns, main beams, collectors, and other elements that provide support for lateral bracing elements. Unless exempted, the drift shall be used to calculate the target deformation to be achieved in the required cyclic test of the bolt or clamp connector/purlin assembly per Section 2.2.6 above.

4.2 Drift and Deflection

Drift and deflection shall be calculated for all structures. Deflection shall not exceed the limits in Sections 4.2.1 through 4.2.4 below. Load combinations that include wind loads shall be based on load cases delineated in Section 4.2.3.1 below. Load combinations that include seismic shall be based on ASCE 7 Section 12.8.6 and may be calculated using a ρ =1.0 per ASCE 7 Section 12.3.4.1, Item 2.

4.2.1 Vertical deflection of members under lateral loads shall not exceed that specified in Section 3.4 above.

4.2.2 RC III and RC IV structures shall comply with the wind drift limit per CBC Section 1609A.1.2 and the seismic drift limits per ASCE 7 Table 12.12-1. Cantilevered column systems shall also comply with seismic drift parameters for moment frame systems in ASCE 7 Section 12.12.1.1. The drift limit shall be applied at the edges of the roof structure which produce the most severe drift.

Exception: Open structures classified as RC I or II, and ground-mounted systems that are designed for seismic based on ASCE 7 Chapter 15, are exempt from these drift limits.

4.2.3 The maximum global vertical deflection due to wind or seismic load combinations at the end of beam and free edges of cantilevers shall not violate minimum clearances as required for the specified use, and in no case shall be less than 7’-0” above highest adjacent grade.

4.2.3.1 The analysis with wind loading shall include the load combinations containing gravity plus vertical wind and gravity plus lateral wind.

4.2.3.2 Load combinations that include wind may use service level loads (0.6W) when determining this deflection.

4.2.4 Structural Separation

Provide minimum structural separations between adjacent cantilevered canopy structures in accordance with ASCE 7 Section 12.12.2.

4.2.4.1 In accordance with ASCE 7 Section 13.6.9, all pipes, conduits, and other utility lines crossing separation joints shall be designed to accommodate, without rupture or distress, differential movements from design displacements between connection points.

4.2.4.2 The PC drawings shall indicate the maximum lateral displacement demand for each cantilevered canopy structure type, configuration, and option in the design information section. This information is required so the following items can be verified by design professionals and plan reviewers in the site-specific application of the PC design(s). Refer to Appendix A below for additional information.

4.2.4.2.1 Adequate separation is provided between adjacent structures.

4.2.4.2.2 Adequate separation is provided relative to existing site structures.

4.2.4.2.3 Utility details provide sufficient compensation for differential movements (e.g., adequate loop/slack, minimum vertical drop in the loop equal to the separation distance).

4.3 Lateral Bracing Systems

4.3.1 Solar panels shall not be considered as providing any diaphragm action for the structure.

4.3.2 When no other system of lateral bracing is provided within a bay, the beams and purlins shall be designed for out-of-plane bending to resist the lateral loads from wind and seismic.

4.3.3 Tension-only rod bracing may be used to resist lateral load forces. When diagonal rod bracing systems are used that rely on pretensioning to reduce the sag of the rod, the design of the structure shall include that pretensioning load in the analysis. DSA will accept the load factor used for dead loads to also be applied to the pretension force. When the analysis is based on a computer model, the pretension force may be applied as a temperature load.

The sequencing of the rod tensioning shall also be specified on the plan. The impact of the rod tensioning on the solar panel connections shall also be considered if the panels are installed prior to the rod tensioning. If vertical supports are added to prevent sag in the rods, they may be installed as snug tight with no additional loads in the structural analysis.

4.3.4 Metal deck may be used to provide diaphragm action. Metal decking shall meet the minimum requirements of CBC Section 2208A.1 with a minimum 20 ga. thickness.

4.4 Column Design For All Lateral Loads

4.4.1 The lateral load design of the column shall include all shear loads due to lateral loads plus the moments due to gravity load eccentricities from the structure above. See Sections 4.7.4 and 4.8.4 below for specific seismic and wind load design requirements.

4.4.2 If column heights vary over the structure, this difference in stiffness shall be accounted for in the distribution of lateral forces.

4.4.3 Built-up columns of different materials shall not be permitted unless approved as an alternate design with supplemental full-scale testing.

4.4.3.1 Built-up columns shall comply with either of the following:

4.4.3.1.1 AISC 360: Specification for Structural Steel Buildings and AISC 341: Seismic Provisions for Structural Steel Buildings for structural steel.

4.4.3.1.2 AISI S100: North American Specification for the Design of Cold-Formed Steel Structural Members and AISI S400: North American Standard for Seismic Design of Cold-Formed Steel Structural Systems for cold-formed steel.

4.4.4 Columns must be designed for P-Δ (first order) effects of the ½" displacement of the foundation if two times the lateral bearing pressure is used per CBC Section 1806A.3.4. The column shall be evaluated for the additional imposed P-Δ effects using a point of inflection at 70 percent of the pier depth as shown in Figure 4.3 below. See Section 5.2 below.

4.4.4.1 If this approach is used, a note shall be included on the PC indicating the ½ inch movement at the base was considered during design; thus, the lateral bearing pressure increase per CBC Section 1806A.3.4 is permitted.

4.4.4.2 The point of rotation for a spread footing shall be the bottom corner of the footing. This P-Δ increase in moment applies to the design of the column and is not required to be included in the drift and deflection calculation in Section 4.2 above.

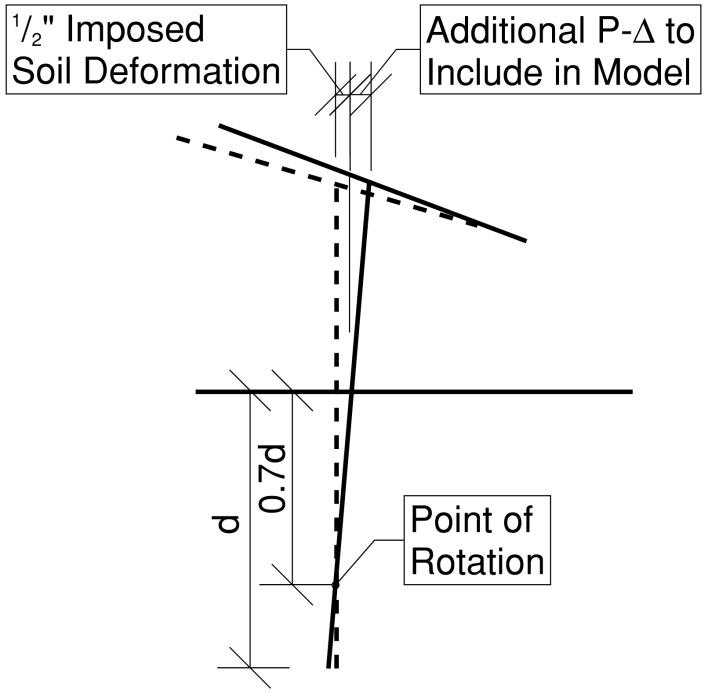


Figure 4.3: Surface Displacement per CBC Section 1806A.3.4

4.5 Pedestal Design

A pedestal is defined as a structural element that extends above grade more than 12” and transfers load from the steel column to the foundation. Concrete pedestals shall be designed and reinforced as required for concrete columns, using the American Concrete Institute (ACI) 318 Chapter 10 for vertical reinforcing, Section 25.7.2 for ties and Section 25.7.3 for spirals. For seismic loads, reinforcing shall also comply with ACI 318 Section 18.13.5.3. The strength shall meet or exceed all load demands and load transfer mechanisms at the column base per Section 5.13 below. In no case shall the pedestal reinforcing and detailing be less than that required for the upper portion of the foundation pier per Section 5.4 below. For pier foundations, the pedestal reinforcing shall extend from the top of the pedestal down to the level where soil passive pressure is assumed to be active. For a spread footing with a pedestal, the reinforcing shall extend from the top of the pedestal to the bottom reinforcement of the spread footing.

When pedestal heights vary within the same structure, the relative stiffnesses of the pedestals shall be considered in the distribution of lateral loads to each column.

4.6 Purlin Design for All Lateral Loads

Purlin design shall comply with the following items as they apply to the project. See Section 4.8.6 below for additional specific wind load design requirements for purlins and Section 4.6 below for additional specific design requirements for purlins.

4.6.1 Purlins shall be designed for weak-axis bending due to seismic and wind loads if a roof diaphragm is not present, (i.e., for an open grid system). Weak-axis deflection of purlins between girders shall be calculated and compared with the in-plane diaphragm roof drift to determine the maximum in-plane differential displacement between panel anchors in order to establish the cyclic testing performance criteria per Section 2.2.6 above.

4.6.2 Purlin top flanges shall be evaluated for localized stresses imposed by solar panel connections. Solar panels shall not be considered as providing top flange bracing, providing lateral torsional buckling resistance to the purlin, or be considered as providing any structural purpose other than transferring solar panel reactions to supporting purlins.

Without a more detailed analysis, it is acceptable to model the top flange as being fixed at the web end and pinned at the lip end with the load at its actual location on the flange, a distance "b" from the center of the bolt to the web. An effective flange strip width may be assumed by projecting a line at a 3:1 slope from the center line of the bolt toward the web on each side of the bolt. Where the bolt is close to the end of the member, or close to an adjacent bolt, that width shall be truncated where the sloped line intersects the end of the member or intersects the projected line of the adjacent bolt. When using this approximate analysis method, no bolt shall be closer than 3b to the end of a purlin that is unsupported, or 2b to the end if the top and bottom flanges of the purlin have positive support against rotation. See Figure 4.6.2 below.

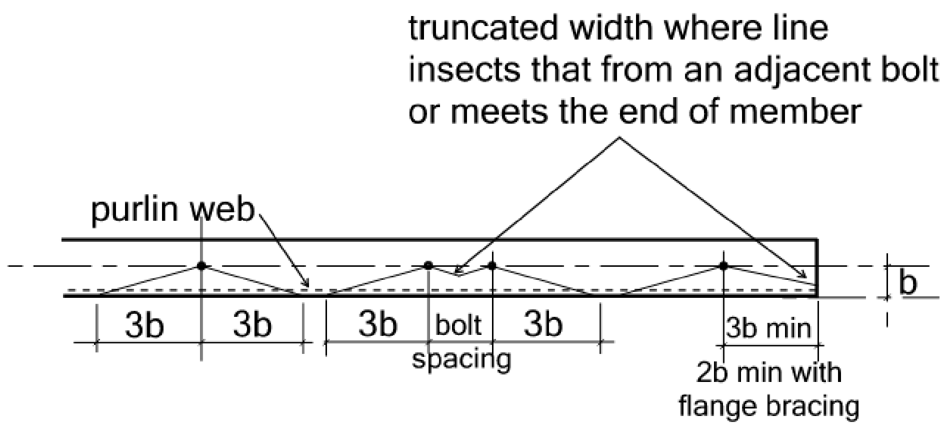


Figure 4.6.2: Effective Flange Width for Solar Panel Fasteners

4.6.3 Purlin to beam connections shall be designed to resist torsion per AISI S100 Section C2.2.1. If there is no torsional bracing along purlins, torsion shall be considered for the full span. If torsional restraint is provided between beams, effective torsional length may be assumed as the distance between such restraints.

4.6.4 Blocking provided to reduce the unbraced length of purlins shall be detailed to restrain both web and flanges of the purlin in accordance with AISI S100 Section C2.2.1 (and Section I6.4.1 if purlin flange is attached to diaphragm).

4.6.5 Where used on structures without metal deck diaphragms, such blocking shall be provided in rows in each bay between purlins across full width of the roof plane to ensure uniform in-plane deflection.

4.7 Seismic Design

4.7.1 Seismic Load Criteria

4.7.1.1 The seismic design criteria upon which the PC design is based shall be stated in the design information section of the PC drawings in accordance with PR 07-01 and CBC Section 1603A.1.5.

4.7.1.2 If the design is based upon the maximum spectral response acceleration parameters occurring in the state of California, the PC can be used at any site in the state. The PC design may be based on lesser values, but doing so will limit the sites where the PC can be used.

4.7.2 Limiting the S_{DS} value in accordance with ASCE 7 Section 12.8.1.3 is not permitted, as this provision no longer applies to structures with a response modification coefficient (R) less than 3. This provision also does not apply to nonbuilding structures designed in accordance with ASCE 7 Chapter 15.

4.7.2.1 Redundancy Factor (ρ)

4.7.2.1.1 For all structures designed to comply with ASCE 7 Chapter 12, "T" and offset "T" configurations or those with no diaphragm shall be designed using ρ = 1.3. For other configurations with more than a single support in the transverse direction or a diaphragm, a ρ = 1.0 may be used if compliance with ASCE 7 Section 12.3.4 is demonstrated. For elements required to be designed for overstrength, the design shall be based on the more severe loading using omega (Ω) or ρ factor, considered independently.

4.7.2.1.2 The 1.2 allowable stress increase factor per ASCE 7 Section 2.4.5 is not applicable to load combinations with ρ and therefore may not be applied to soil pressure increase.

4.7.3 Seismic Force Resisting System

Steel cantilevered canopy structures with occupancy below shall comply with ASCE 7 Chapter 12. These structures shall be designed as cantilevered column systems or other systems permitted in accordance with ASCE 7 Table 12.2-1. ASCE 7 no longer permits steel ordinary cantilever column systems in Seismic Design Categories D, E, and F (refer to errata issued by ASCE); therefore, a special system is required when a steel cantilever column system is used. See Section 9 below for ground-mounted PV structure requirements.

4.7.4 Column Design for Seismic

4.7.4.1 The column design shall comply with all parts of AISC 341 Section E6, as well as ASCE 7 Sections 12.2.5.2 and 12.2.5.3. Columns shall also be designed in accordance with the Orthogonal Directional Combination Procedure per ASCE 7 Sections 12.5.1.2 and 12.5.4(g).

4.7.4.2 Rectangular HSS and wide flange member columns shall be designed in accordance with the stability bracing requirements in AISC 341 Section E6.4b. In the calculation for bracing spacing (L_{bc}), M_1/M_2 shall be considered as 0.5 minimum for inverted pendulum-type structures. Where bracing is provided, it shall meet the requirements per Section E6.4b(b). Bracing may be omitted when the height of the column does not exceed the calculated L_{bc} spacing. Square or round HSS sections are exempt from bracing requirements or limitations on height for stability.

4.7.4.3 Refer to Section 5.13 below for column base connection requirements.

4.7.4.4 The PC design shall demonstrate the adequacy of the column subject to weak axis bending when applicable.

4.7.4.5 The weights, heights, and horizontal offsets of all equipment and fixtures attaching to the columns shall be explicitly accounted for in the column design.

4.7.4.6 Protected Zones as defined in AISC 341 Section E6 shall be permanently marked.

4.7.5 High-Strength Bolts

All bolts that are part of the seismic force resisting system (SFRS) (i.e., moment-resisting beam-to-column connections) shall be pretensioned high-strength bolts that meet the requirements for AISC 341 Section D2.2 slip-critical faying surfaces with a Class A surface or higher. Section

D2.2 lists two conditions eligible for exemption from the Class A faying surface requirement. Bolted end-plate moment connections (1) are eligible for Exemption 2 of that section; bolted side-plate moment connections (2) are not. Beam-to-column connections which are functionally similar to bolted end-plate moment connections (i.e., beam bottom flange bolted to horizontal seat plate atop column; very small shear relative to the moment) are also eligible for exemption from the Class A faying surface requirement.

4.8 Wind Design

The wind design requirements are given in CBC Section 1609A. The main wind force resisting system (MWFRS) for steel cantilevered canopy systems shall be designed using the open building provisions in ASCE 7 Section 27.3.2, including the requirements for structures with a roof slope of < 5 degrees with fascia panels where occurs. The Net Pressure Coefficient C_N shall be determined based upon the specified angle of the roof slope, θ. The Components and Cladding (C&C) wind loads shall be applied to members in accordance with ASCE 7 Section 30.7 as appropriate.

4.8.1 Wind Loads

For the MWFRS design, a minimum horizontal lateral wind load of 16 psf shall be applied to the vertical face of the column and the area of the roof surface (A_f) projected on a vertical plane normal to the wind direction with wind load applied horizontally to the structure per ASCE 7 Section 27.1.5 and Figure C27.1-1. The design wind load Case 1 and Case 3 of ASCE 7 Section 27.3.5 shall also be considered.

4.8.2 Clear and Obstructed Wind Flow

4.8.2.1 Open structures shall be permitted to be designed for Clear Wind Flow per ASCE 7 Figures 27.3-4 through 27.3-7 for MWFRS and Figures 30.7-1 through 30.7-3 for C&C loads if calculations are provided justifying the use of Clear Wind Flow. Without substantiating calculations, structures shall be designed for both Clear and Obstructed Wind Flow. Structures shall be designed for both Clear and Obstructed Wind Flow if located adjacent to a building or other obstruction, or in a bus yard.

4.8.2.2 The design information section of the PC drawings shall include a note stating if the PC structure(s) is approved for both Clear and Obstructed Wind Flow. If the structure has only been designed for Clear Wind Flow, the PC drawings shall also include notes and/or diagrams as necessary to define the required site clearances for verification by the design professionals and plan reviewers of site-specific applications.

4.8.3 Cantilevered Beam Design for Wind

Where the tributary area to a beam is greater than 700 SF, the use of MWFRS loading is permitted per ASCE 7 Section 30.2.3. Cantilevered beams with a tributary area of less than 700 SF shall be designed for C&C wind loads in combination with other loads.

4.8.4 Column Design for Wind

4.8.4.1 The column design shall include the wind load on the projected area of column face.

4.8.4.2 The moment at the bottom of the column shall include the moment from the roof beam eccentricity to the column.

4.8.4.3 The moment at the bottom of the column shall include the horizontal reaction (due to horizontal component of wind load) of the beam to the column.

4.8.5 Solar Panel Fastener Design

Solar Panel fastener connections shall be designed for C&C loading per ASCE 7 Section 30.7.

4.8.6 Purlin and Metal Decking Design for Wind

4.8.6.1 Purlins shall be designed for C&C loading per ASCE 7 Section 30.7

4.8.6.2 All exposed metal decking and purlin framing not enclosed by a soffit system shall be designed for a C&C loading per ASCE 7 Section 30.7, with a minimum load of 16 psf applied normal to the roof surface per ASCE 7 Section 30.2.2. Where an upper roof and lower soffit surface are separate and do not share loading, each face shall be designed for all applicable C&C loads. Refer also to Section 4.8 above for MWFRS wind loads at low slope roofs.

5. FOUNDATION

5.1 Vertical Allowable Soil Pressure

The PC design shall be based on the presumptive allowable soil bearing pressure corresponding to Class 5 soil in CBC Table 1806A.2 unless justified by a site-specific geotechnical report. To base the design on values greater than those stated for Class 5 soil, a statement requiring a site-specific geotechnical report at the time of site application shall be included in the design information section on the PC drawings.

An allowable stress increase in the presumptive load-bearing value is not permitted when using the allowable stress design load combinations per ASCE 7 Section 2.4. An allowable stress increase is permitted in accordance with CBC Section 1806A.2 when using the alternative allowable stress design load combinations per CBC Section 1605A.2 that include wind or seismic loads.

5.2 Lateral Bearing Pressure

The PC design shall be based on the presumptive lateral bearing pressure corresponding to Class 5 soil in CBC Table 1806A.2 unless justified by a site-specific geotechnical report. To base the design on values greater than that stated for Class 5 soil, a statement requiring a site-specific geotechnical report at the time of site application shall be included in the design information section on the PC drawings.

When justified in accordance with Section 4.4.4 above, the presumptive lateral bearing pressure may be increased in accordance with CBC Section 1806A.3.4. This increase is not permitted to lateral bearing values determined by a site-specific geotechnical evaluation. The design information section of the PC drawings shall clearly state if the lateral bearing pressure value used in the design has been increased per CBC Section 1806A.3.4

5.3 Foundation Design Load

The design of foundation elements, including cast-in-place deep foundations (drilled piers) and shallow spread footings, and their connections shall be based on load combinations including the overstrength factor for cantilever column systems per ASCE 7 Section 12.2.5.2. All systems shall comply with the amplified load requirements of CBC Section 1617A.1.15.

5.4 Cast-in-place Deep Foundation (Drilled Pier)

5.4.1 The PC design shall comply with CBC Section 1810A.3.9. The alternate design provisions of *IR 18-5: Foundation Design and Detailing*, Section 1.4.3 are not permitted.

5.4.2 See Section 5.7 below for drilled piers used in combination with shallow spread footings.

5.4.3 In accordance with CBC Section 1810A.2.4, the depth of the drilled pier is permitted to be designed per CBC Section 1807A.3.2 if the drilled pier is assumed to be rigid.

PC DESIGN CRITERIA FOR STEEL CANTILEVERED CANOPY STRUCTURES: 2025 CBC

5.4.3.1 The drilled pier may be assumed to be rigid if the ratio of the specified depth (not the minimum depth required by CBC Section 1807A.3.2) to diameter is equal to or less than 8 and CGS does not otherwise require analysis per Section 5.4.3.2 below.

5.4.3.1.1 The drilled pier shall be designed for the maximum moment and shear below grade based on engineering mechanics. Refer to IR 18-5 Sections 1.4.2.1 and 1.4.2.2 for more information.

5.4.3.2 When the drilled pier does not comply with Section 5.4.3.1 above, the design, including reinforcing, shall consider the nonlinear interaction of the drilled pier and soil (e.g., L-pile analysis or equivalent) per CBC Section 1810A.2.4 with consideration of group effects as required by CBC Section 1810A.2.5.

5.4.4 The design information section shall include a note specifying the minimum clearance required between drilled piers when placing multiple canopy structures adjacent to each other. The design of the drilled piers shall consider group effects per CBC Section 1810A.2.5 if applicable. The drawings may permit drilled pier spacing less than eight times the pier diameter if the PC applicant retains a geotechnical engineer to evaluate group effects for each soil type used.

5.4.5 Transverse reinforcing shall comply with CBC Section 1810A.3.9.4.2 and ACI 318. See ACI 318 Table 18.13.5.7.1 for additional information.

Exception: The transverse reinforcement (i.e., tie or spiral) need not exceed that required in the subsections below when the drilled pier is assumed to be rigid per Section 5.4.3 above and the factored axial force is less than 10 percent of the specified concrete compressive strength multiplied by the gross area of the concrete section (i.e., $P_u < 0.10f'_cA_g$). This exception is only applicable to drilled piers supporting canopy structures. This type of structure is lightly loaded and has a low ductility demand. These exceptions may not be extended to other types of structures.

5.4.5.1 Per CBC Section 1810A.3.9.4.2, the size of transverse reinforcement shall comply with the following:

5.4.5.1.1 Drilled pier diameter 20-inches or less: #3 bar minimum.

5.4.5.1.2 Drilled pier diameter greater than 20-inches: #4 bar minimum.

5.4.5.2 For drilled piers in soil categorized as Site Class A, B, BC, C, CD, D or E, transverse reinforcement spacing shall not exceed the smallest of the following in the top 3d of the drilled pier (where “d” is the drilled pier diameter). Refer to Figures 5.4A and 5.4C in *IR PC-1: Pre-Check (PC) Design Criteria for Freestanding Signs, Scoreboards, and Ball Walls*.

5.4.5.2.1 One quarter the drilled pier diameter: d/4.

5.4.5.2.2 Six times the least Grade 60 longitudinal bar diameter: $6d_b$.

5.4.5.2.3 Five times the least Grade 80 longitudinal bar diameter: $5d_b$.

5.4.5.2.4 Six inches: 6”.

5.4.5.3 For drilled piers in soil categorized as Site Class E, transverse reinforcement spacing shall comply with Section 5.4.5.2 above in the top 7d of the drilled pier (where “d” is the drilled pier diameter). Refer to IR PC-1 Figures 5.4B and 5.4D. In consideration that 7d is 88 percent or more of the overall pier depth and the requirement of Section 5.4.5.5 below, it is recommended this transverse reinforcement spacing requirement simply be specified over the full depth of the drilled pier.

5.4.5.4 Transverse reinforcement spacing shall not exceed the smallest of the following in the remainder of the drilled pier except as required by Section 5.4.5.5 below. Refer to IR PC-1 Figures 5.4A, 5.4B, 5.4C, and 5.4D.

5.4.5.4.1 One half the drilled pier diameter: d/2.

5.4.5.4.2 Twelve times the least longitudinal bar diameter: $12d_b$.

5.4.5.4.3 Twelve inches: 12”.

5.4.5.5 For drilled piers in soil categorized as Site Class E, transverse reinforcement spacing shall comply with Section 5.4.5.2 above at all depths within 7d above and below (where “d” is the drilled pier diameter) interfaces between hard/stiff and soft strata as required by CBC Section 1810A.3.9.4.2.2.

5.4.5.6 For drilled piers with partially embedded columns, the transverse reinforcement spacing shall also not exceed that required by Section 5.12 below. For ground-mounted PV systems, see Section 9 below.

5.5 Allowable Frictional Resistance and Uplift Capacity

The allowable frictional resistance and uplift capacity used in the design of cast-in-place deep foundations (drilled piers) shall be included in the design information section. When a site-specific geotechnical report is not available, CBC Section 1810A.3.3.1.4 can be used to determine the allowable frictional resistance value assuming Class 5 soils as noted in Sections 5.1 and 5.2 above.

5.6 Ground Surface Condition

When CBC Section 1807A.3.2 is utilized, asphalt pavement does not constitute a “constrained” condition and does not justify the use of CBC Equation 18A-2 or 18A-3 to determine the required drilled pier depth. Where the constrained condition is used with concrete pavement, the reaction shall be adequately resisted and justified by calculations. The construction necessary to resist this reaction shall be clearly detailed on the PC drawings.

5.7 Shallow Foundations

Shallow spread footings shall be designed per CBC Chapter 18A and for stability in accordance with CBC Section 1605A.1.1. For cantilevered column systems designed based on ASCE 7 Chapter 12 for seismic, the footings shall also comply with Section 12.2.5.2. If a canopy structure is supported by a combination of deep foundation elements (e.g., drilled pier) and shallow spread footings, all steel columns within the structure shall have the same height unless the column stiffness is accounted for in the design.

5.8 Adjacent Slopes

The PC drawings shall specify minimum setback limits (values are required) of the structure relative to slopes per CBC Section 1808A.7. If the PC drawings define setback limits smaller than the CBC allows, a statement requiring a site-specific geotechnical report at the time of site application shall be included in the design information section. Alternatively, when delineated on the approved PC drawings, the required depth of the cast-in-place deep foundation (drilled pier) can be increased in accordance with Figure 5.8 below.

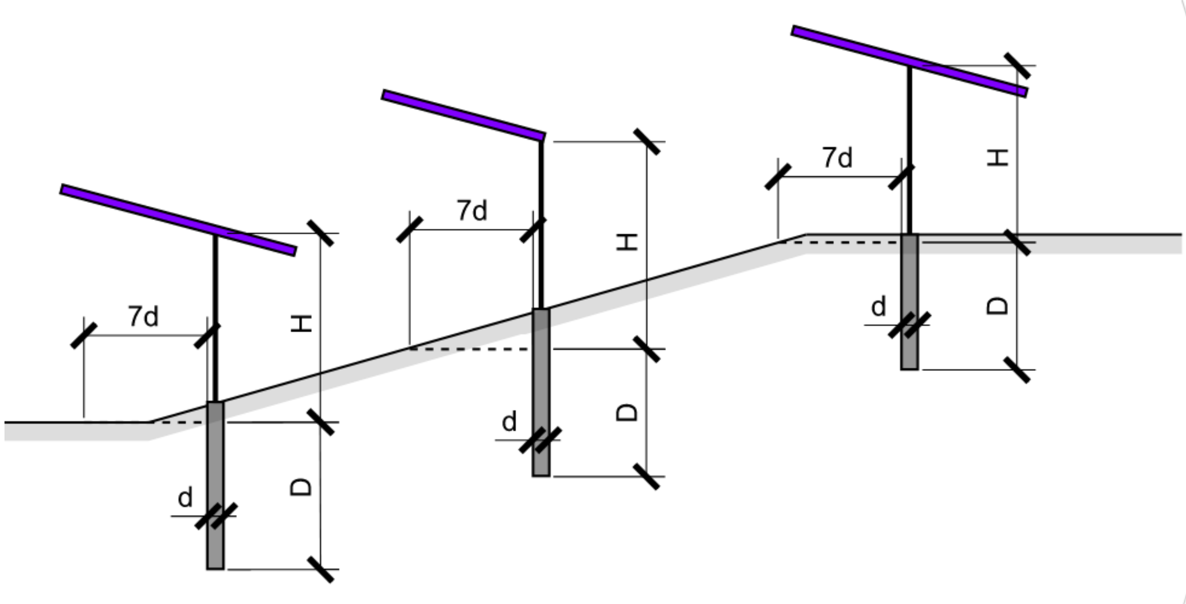


Figure 5.8: Sloped Sites

The pier depth shall be increased such the depth required by analysis (i.e., “D” designated in Figure 5.8) is provided below a horizontal plane projected from a horizontal distance seven times the pier diameter (i.e., “7d” designated in Figure 5.8 above). Additionally, design parameters dependent on column height shall be determined based on a theoretical column height starting from the same horizontal plane (i.e., “H” designated in Figure 5.8 above). If the setback limits are smaller than the CBC requires, a site-specific geotechnical report is required.

5.9 Liquefiable Soil or Site Class F

PC designs will not be approved with an option for construction on sites with liquefiable soil or soil categorized as Site Class F. If the site is not in a mapped liquefaction hazard zone, it may be presumed that no liquefaction hazard exists on that site unless a site-specific geotechnical report identifies such hazard. Refer to IR A-4 Section 4.

5.10 Concrete Mix

In addition to those requirements dictated by the PC design, the concrete used in the foundation elements shall comply with the durability requirements of ACI 318 Section 19.3. The PC drawings shall account for the dependency of these durability requirements on site-specific characteristics.

5.10.1 When the PC drawings do not require a site-specific geotechnical report that quantifies sulfate content in the soils, the PC drawings shall require a concrete mix complying with one of the following per ACI 318 Table 19.3.2.1:

5.10.1.1 Maximum water/cement ratio of 0.45; minimum compressive strength of 4,500 pounds per square inch (psi); Type V cement plus pozzolan or slag cement complying with footnote 7 of the table, and prohibition of admixtures containing calcium chloride.

5.10.1.2 Maximum water/cement ratio of 0.40; minimum compressive strength of 5,000 psi; Type V cement complying with footnote 8 of the table, and prohibition of admixtures containing calcium chloride.

5.10.2 When the PC drawings require a site-specific geotechnical report that quantifies sulfate content in the soil, the PC drawings shall clearly state the exposure class for each category (i.e., F, S, W and C) or combination thereof the PC design is approved for. The maximum water/cement ratio, minimum compressive strength, cementitious material requirements, and admixture limitations shall be stated on the PC drawings for each approved case.

5.10.3 Both approaches given in Section 5.10.1 and 5.10.2 above can be included on the PC drawings as alternate options in accordance with Section 1.4 above.

5.10.4 The PC drawings shall include a note requiring that concrete exposed to freezing-and-thawing cycles be air entrained per ACI 318 Section 19.3.3.

5.11 Conduits in Foundation

The PC drawings shall clearly show the size and number of conduits adjacent to or penetrating the foundation elements (e.g., drilled pier, shallow footing, etc.). The drawings shall include an elevation showing the location of the conduits relative to the foundation element and its reinforcement.

5.11.1 The presence of conduits may require the portion of the foundation above the conduits to be neglected in the structural design. The impact of conduits on the foundation strength, effective column height, and foundation depth shall be justified by calculation.

5.11.2 The base plate design shall also consider holes or notches for conduits. Details of holes and notches in the base plate shall be included in the PC drawings.

5.12 Partially Embedded Columns

When steel columns are partially embedded into cast-in-place deep foundation (drilled pier), the alternate design provisions in IR 18-5 Section 1.4.3 are not applicable.

5.12.1 The load transfer mechanism of partially embedded columns shall include the design of both the column and drilled pier ties or spiral. The transverse reinforcement size and spacing shall be sufficient to transfer the required force based on a rational method and accepted principles of engineering mechanics.

5.12.2 Design of the embedded column shall include local buckling within the concrete based on AISC 341 Section D2.6c(b)(1).

5.12.3 The minimum column embedment depth into the portion of the drilled pier that is designed to resist lateral forces shall be the greater of the following:

5.12.3.1 Seven times the larger dimension of the column section.

5.12.3.2 Minimum development length of the longitudinal pier reinforcing based on ACI 318 Section 25.4 and Section 5.13.2 below.

5.12.4 All embedded columns into drilled piers shall have a mechanical connection to resist uplift. AISC 360 Section I6 provides acceptable criteria for demonstrating the adequacy of the load transfer from the partially embedded column to the drilled pier.

Exception: For steel columns embedded into the drilled pier 4 feet or more, it is permitted to assume an allowable bond stress of 25 psi between the steel column and concrete. The upper 12 inches of the column embedment shall be disregarded and no increase in this allowable bond stress is permitted for wind or seismic loads.

5.13 Column Base Connection

AISC *Design Guide 1: Base Plate and Anchor Rod Design* provides useful guidance on the design of the column base connection.

5.13.1 The column base connection shall comply with AISC 341 Section D2.6 and *IR 18-4: Superstructure to Foundation Connection*, Sections 4.1 and 4.2, except where provisions in this IR specify differently.

5.13.2 The embedment depth of the anchor rods shall be sufficient to lap with the longitudinal drilled pier reinforcement, when applicable. The lap length shall be based on developing the longitudinal reinforcement beyond the projected failure plane of the anchor rod heads. Refer to IR PC-1 Figures 5.4A, 5.4B, 5.4C, and 5.4D and ACI 318 Figure R17.5.2.1a. The lap length is not permitted to be reduced based on providing reinforcement beyond that required for the applied loads.

5.13.3 Anchor rods shall be designed for combined shear and tension. If the maximum grout thickness between the top of the foundation and bottom of base plate exceeds two times the anchor rod diameter, the anchor rods shall be designed for bending in combination with tension and shear. Compliance with Telecommunication Industry Association (TIA) 222-I, Section 4.9.9 is an acceptable design method of designing anchor rods for combined tension, shear, and bending.

5.13.4 When oversized holes are used in the base plate, the design shall comply with CBC Section 2201A.5.1.

6. ACCESS COMPLIANCE REQUIREMENTS

6.1 Protruding Objects in Circulation Areas

Protruding objects such as column-mounted equipment shall comply with CBC Section 11B-307.

6.2 Vertical Clearance in Circulation Areas

Vertical clearance to supporting structures shall comply with CBC Section 11B-307.4.

6.3 Vertical Clearance at Accessible Parking and Electric Vehicle Charging Stations

Vertical clearance at accessible parking spaces, access aisles and vehicular routes serving them shall comply with CBC Sections 11B-502.5 and 11B-812.4.

6.4 Vertical Clearance at Passenger Drop-Off and Loading Zones

Vertical clearance at vehicle pull-up spaces, access aisles and along the vehicular route shall comply with CBC Section 11B-503.5.

7. FIRE AND LIFE SAFETY REQUIREMENTS

7.1 Design

Designs shall comply with the CBC as adopted and amended.

7.2 Type of Construction

Indicate the type of construction in accordance with CBC Chapter 6 in design information on the plan set coversheet.

7.3 Total Area of Structure

Indicate total area of structure in the design information section on the plan set coversheet, and demonstrate that total area does not exceed allowable area in accordance with CBC Table 506.2 based on type of construction and proposed occupancy classification(s).

7.4 Use and Occupancy Classification(s)

Indicate the proposed use and occupancy classification(s) in accordance with CBC Chapter 3 in the design information section on the plan set coversheet.

8. CALIFORNIA ENERGY CODE REQUIREMENTS

Steel cantilevered canopy structures are not subject to California Energy Code review.

9. GROUND-MOUNTED PV STRUCTURES

Ground-mounted PV structures are defined as structures supporting PV panels with no use or occupancy underneath. Per Section 1.6.2 above, any ground-mounted PV structures with a minimum 6‘-8” clearance below shall be confined within an enclosed (i.e., fenced) area to prohibit access by students and teachers to qualify for this section.

9.1 The **General** requirements in Section 1 above shall apply.

9.2 The **Solar Panel and Panel Attachment Requirements** in Section 2 above shall apply.

9.3 The **Gravity Load Design** requirements in Section 3 above shall apply except as follows:

9.3.1 The **Live Load** requirements in Section 3.2 above need not be applied to ground-mounted PV structures per CBC Section 1607A.14.3.4.

9.4 The **Lateral Load Design** requirements in Section 4 above shall apply except as follows:

9.4.1 Section 4.2.2 above shall not apply when the seismic design of the structure is based on ASCE 7 Chapter 15.

9.4.2 Section 4.2.3 above shall not apply to any ground-mounted PV structure.

9.4.3 Section 4.5 above shall not apply when using IR 18-4 Section 1.4 for design of the foundation.

9.4.4 The **Seismic Design** requirements in Section 4.7 above shall be replaced as follows:

9.4.4.1 Ground-mounted PV structure seismic design shall comply with either ASCE 7 Chapter 12 (Refer to Section 4.7.3 above) or ASCE 7 Chapter 15 for nonbuilding structures using a system based on “Inverted pendulum-type structures” or “All other self-supporting structures…” as listed in Table 15.4-2. “Inverted pendulum-type structures” require compliance with ASCE 7 Section 12.2.5.3 for the column and foundation designs.

9.4.4.2 Sections 4.7.2.1, 4.7.4 and 4.7.5 above shall not apply when the seismic design of the structure is based on ASCE 7 Chapter 15.

9.4.5 The **Wind Design** requirements in Section 4.8 above shall be replaced as follows:

9.4.5.1 Wind design for ground-mounted PV structures may be based on ASCE 7 Section 29.4.5 when the system meets the limitations of that section. When the ground-mounted PV structure does not meet the criteria in that section, Section 4.8 above shall apply as for elevated PV structures.

9.4.5.2 Sections 4.8.2 and 4.8.3 above shall not apply when ASCE 7 Section 29.4.5 is the basis of the wind design.

9.4.5.3 Section 4.8.5 above shall be replaced as follows:

9.4.5.3.1 Solar panel fastener connections shall be designed for loading in accordance with ASCE 7 Section 29.4.5 when the system meets the limitations of that section. When the ground-mounted PV structure does not meet the criteria in that section, Section 4.8.5 above shall apply as for elevated PV structures.

9.4.5.4 Section 4.8.6 above shall not apply when ASCE 7 Section 29.4.5 is the basis of the wind design.

9.5 The **Foundation** requirements in Section 5 above shall apply for drilled piers supporting ground-mounted PV systems except as follows:

9.5.1 The alternate design provisions in IR 18-5 Section 1.4.3 can be applied if the steel column is fully embedded in the drilled pier (i.e., to within six inches of the bottom of the drilled pier) and meets all of the parameters noted therein (i.e., nonconstrained, $R \leq 2$, and R/Ω_o=1.0 or less, etc.).

9.5.2 Section 5.3 above shall not apply when the seismic design of the structure is based on ASCE 7 Chapter 15.

9.5.3 Section 5.13.1 above shall not apply when the seismic design of the structure is based on ASCE 7 Chapter 15.

9.5.4 Driven Steel Pile Design for Ground-Mounted PV Structures

9.5.4.1 Design

9.5.4.1.1 Where driven piles are used as the foundation system, the design shall be in accordance with CBC Section 1810A.3 as well as the provisions of this section. The PC shall include notes with site-specific requirements for geotechnical evaluation, design, and testing as noted below.

9.5.4.1.2 Driven steel pile design shall be based on criteria developed by a licensed geotechnical engineer using recognized geotechnical values in accordance with CBC Chapter 18A.

9.5.4.1.3 Site specific applications shall include a site-specific geotechnical report and exploration and site-specific testing to determine which soil values (soil class, lateral resistance, skin friction) from the PC are applicable to the site.

9.5.4.1.4 Driven steel piles shall be designed accounting for the nonlinear characteristics of the soil and pile interaction using software such as L-pile.

9.5.4.1.5 The design vertical and lateral loads for each soil class shall be indicated on the PC drawings.

9.5.4.2 Testing

9.5.4.2.1 Vertical load testing shall be as required by the geotechnical report or where required in accordance with CBC Sections 1810A.3.3.1.2 and 1810A.3.3.1.5. for both downward and uplift loading.

9.5.4.2.1.1 Steel pile sections not included in CBC Table 1810A.3.2.6 (e.g., cold-formed steel members) shall not be exempt from the vertical load testing required by CBC Sections 1810A.3.3.1.2 and 1810A.3.3.1.5.

9.5.4.2.2 Lateral load testing shall be as required by the geotechnical report and CBC Section 1810A.3.3.2. If the geotechnical report can justify lateral capacity based on analysis, then lateral load testing is not required.

9.5.4.2.2.1 Steel pile sections not included in CBC Table 1810A.3.2.6 (e.g., cold-formed steel members) shall not be exempt from the lateral load testing required by CBC Section 1810A.3.3.2.

9.5.4.2.2.2 When required, lateral load testing shall be in conformance with ASTM D3966 tested to twice the design working load. Allowable load shall not be more than one half the load that causes a 1-inch lateral movement at the ground surface.

9.5.4.2.2.3 The required lateral test load values shall be stated on the site-specific drawings, along with the maximum 1-inch deflection limit.

9.5.4.2.2.4 The height of the applied test load on the column shall be equal to the height of the application of the load on the column in the design (i.e., at the connection point of the column and the solar panels). If the test load is applied at the lower point on the column, then the test load shall be increased to apply the same moment at the base of the column.

9.5.4.2.2.5 The lateral test load shall include the moment due to the unequal MWFRS loading on either side of the column in any cases where the forces due to wind govern over seismic forces.

REFERENCES:

2025 California Code of Regulations (CCR) Title 24

Part 1; California Administrative Code (CAC), Section 4-333(b)8.

Part 2: California Building Code (CBC), Chapters 3 and 6, Sections 104.11, 11B-307, 11B-307.4, 11B-502.5, 11B-503.5, 11B-812.4, 1603A.1.8.1, 1604A.3, 1604A.5, 1605A.2, 1605A.1.1, 1607A.14.4.3, 1609A.1.2, 1612A, , 1617A.1.15, 1806A.2, 1806A.3.4, 1807A.3.2, 1808A.7, 1810A.2.4, 1810A.2.5, 1810A.3.2.6,1810A.3.3.4.2, 1810A.3.3.1.2,1810A.3.3.1.4, 1810A.3.3.1.5, 1810A.3.3.2, 2203A.1, 2204A.4, 2403, Tables 506.2, 1004.5, 1604A.3, 1607A.1

PC DESIGN CRITERIA FOR STEEL CANTILEVERED CANOPY STRUCTURES: 2025 CBC

APPENDIX A: SITE-SPECIFIC APPLICATION GUIDE

The following notes are provided as a guide to assist design professionals and DSA plan reviewers when preparing and reviewing site-specific project applications that incorporate PC steel cantilevered canopy structures designed in accordance with this IR. Appendix A is not intended to be an all-inclusive list of design and submission requirements, but rather is an aid to identify aspects of the design criteria described in this IR of particular interest to its site application.

Refer also to PL 07-02 for site-specific requirements that are applicable to both OTC and regular plan review projects utilizing PC project types.

- ☐ Verify site-specific suitability of the PC Steel Cantilevered Canopy Structure including all parameters in PL 07-02 Section 3.
- ☐ Verify site-specific requirements of PL 07-02 Section 4 are met.
- ☐ Verify the Risk Category (RC) and occupancy classification of the site-specific design is compliant with the design information section of the approved PC. RC determination is based on the Occupant Load (OL) of the site-specific code analysis and Occupant Load Factors (OLF) per CBC Table 1004.5. Refer to Section 1.6 above for additional information. The following are some examples of common Use and Occupancy classifications with associated OLF and sizing limits for RC II:
  - Lunch Shelter - Assembly Use ‘A-2’: OLF = 15 square feet (SF)/person or if a combination of table-bench seating is provided, 18 inches/person measured along linear bench length. Group ‘A’ structures with OLF of 15 must not exceed (300 x 15 =) 4,500 SF for RC II.
  - Shade Structure - Concentrated Assembly, Group ‘A’: OLF = 7 SF/person or if fixed seating is provided, calculate per CBC Section 1004.6. Group ‘A’ structures with OLF of 7 must not exceed (300 x 7 =) 2,100 SF for RC II.
  - Shade Structure - Outdoor Instructional Use, Group ‘E’: OLF = 20 SF/person. Group ‘E’ structures with OLF of 20 must not exceed (250 x 20 =) 5,000 SF for RC II.
  - Shade Structure over Playground Equipment, Group ‘E’ (classified same as the campus): OLF = 20 SF/person shall be utilized for purposes of assigning a risk category. Group ‘E’ structures with OLF of 20 must not exceed (250 x 20 =) 5,000 SF for RC II.
  - Shade Structure and/or PV over Parking: Group ‘S-2’ or ‘U’ (determined by design professional): OLF = 200 SF/person. Structures assigned this Use are unlikely to exceed RC II unless utilized for emergency vehicles.
  - Regardless of size, if a structure that would otherwise qualify as RC II provides shelter for emergency vehicles or equipment; or provides required access to, required egress from or shares life safety components with an RC III or IV building, the more restrictive RC must be applied. See CBC Section 1604A.5.1, including the exception for storm shelters constructed in accordance with ICC 500.
  - Review the appendix of the site-specific form DSA 103 for any exemptions from the required structural tests and special inspections. Applicability and consideration of exemptions may be discussed during plan review for site-specific applications and shall be justified by the applicable project design professional for DSA review and approval. Refer to Section 1.3 above for additional information.

PC DESIGN CRITERIA FOR STEEL CANTILEVERED CANOPY STRUCTURES: 2025 CBC

- ☐ If the site is located in a flood zone other than Zone X, verify a validation letter from a geotechnical engineer is provided. Refer to Section 1.7 above for additional information.
- ☐ Geohazard Reports: If the site-specific structure design exceeds 4,000 Sq. Ft. or is located within state or local geologic hazard zones, verify submittal and approval of a geohazard report by CGS in accordance with IR A-4. The structures may be split into multiple seismically separated structures to stay below the 4,000 Sq. Ft. trigger. Refer to Section 1.8 above for additional information.
- ☐ If soil pressure and bearing values exceed Class 5 soil as specified in CBC Table 1806A.2, a site-specific geotechnical report shall be provided at the time of site application to justify values used. Refer to Section 5.1 above for additional information.
- ☐ If drilled pier foundations are used and multiple structures are placed at a site, verify the site-specific drawings comply with the clearance requirements listed on the PC drawings. Refer to Section 5.4 above for additional information.
- ☐ If drilled pier foundations are used and the constrained ground surface condition option is applied, verify the site-specific drawings comply with the ground surface requirements defined on the PC drawings.

  Note: Asphalt concrete is not acceptable. Refer to Section 5.6 above for additional information.
- ☐ If the foundation of the steel cantilevered canopy structure contains both drilled piers and shallow spread footings, verify all columns are the same height unless differing column heights are specifically allowed by the PC drawings. Refer to Section 5.7 above for additional information.
- ☐ If structures are placed adjacent to a slope, verify the site-specific drawings comply with the setback and/or pier embedment requirements defined on the PC drawings. Refer to Section 5.8 above for additional information.
- ☐ If the site has a ground snow load greater than zero, verify the steel cantilevered canopy structure is positioned with sufficient distance from any adjacent structure as defined on the PC drawings. If the horizontal separation is less than 20 ft, snow drift analysis shall be provided by the PC applicant, and the project is not eligible for OTC review. Refer to Section 3.3 above for additional information.
- ☐ Verify the structure location on the site complies with the dimensional requirements for separation from existing structures or other new structures as defined on the PC drawings. Unless a detailed analysis is provided, the movement of an adjacent existing structure shall be assumed to be that corresponding to the maximum drift allowed by the governing code at the time of the existing structure's design or construction. Refer to Section 4.2 above for additional information.
- ☐ Verify utility and services lines crossing structure separation joints are designed to accommodate, without rupture or distress, differential movements between connection points as defined on the PC drawings. Refer to Section 4.2 above for additional information.
- ☐ If the steel cantilevered canopy structure is only approved for Clear Wind Flow (as specified in the design information section), verify the location of the structure(s) on the site meets the clearance requirements defined on the PC drawings. Refer to Section 4.8.2 above for additional information.
- ☐ Verify the solar panel documentation and acceptance letter from the PC design professional is provided. Refer to Section 2.1 above for additional information. Refer to IR 16-8 for more information about load ratings and safety factors required to show the adequacy of the panels for site-specific use.

APPENDIX B: CYCLIC TESTING OF SOLAR PANEL CONNECTORS

In-plane cyclic (i.e., racking) testing shall be performed when required by Section 2.2.6 above. The cyclic test shall demonstrate the connector and panel do not experience slippage, shifting or distress through the calculated differential wind or seismic in-plane roof drift between adjacent purlins for the panel orientation on the PC drawings. If both panel orientations are shown on PC drawings, the test need only be performed for the orientation that yields the more severe requirements.

B1. TEST PROTOCOL

When solar panel connectors are required to be cyclically tested, the following test methodology will be acceptable to DSA. Other test methodologies may be proposed. However, the complete written protocol shall be submitted to DSA for approval in advance of the tests being conducted. The tests shall be performed or witnessed by a nationally recognized laboratory or by a DSA certified laboratory. A written report shall be provided to the responsible PC design professional, who shall include a copy of the report to DSA with the PC plan submittal.

B2. SAMPLE TYPE AND SIZE

The test, performed with materials and fasteners specified on the PC drawings, may be performed using a minimum of one panel mounted on two parallel purlin segments in the orientation intended for use. The solar panel used in the test must be of equivalent or greater load rating than the panel requirements provided on the PC plan. The purlin segments must extend beyond the test panel a distance equal to the adjacent panel's anchors assuming a multi-panel array. One end of one purlin segment shall be restrained in both the longitudinal and transverse direction, and the opposite end restrained in the transverse direction only. The other purlin segment shall be free to move longitudinally and be restrained in the transverse direction at both ends. See Figure B2.1 below.

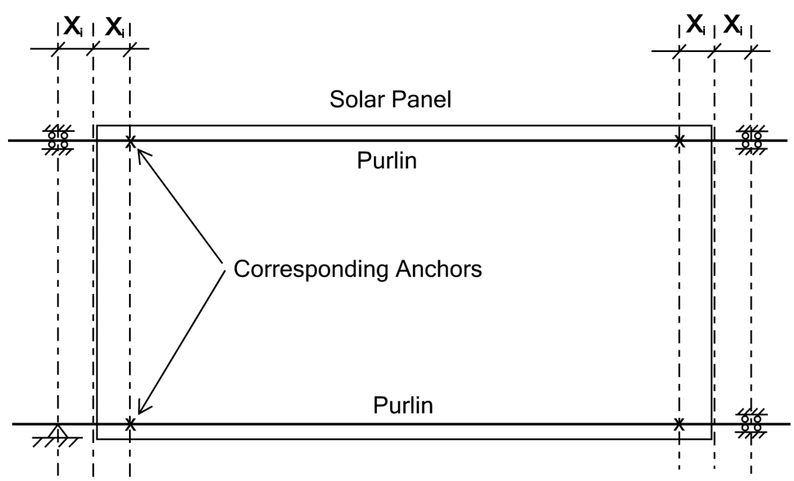


Figure B2.1

B3. APPARATUS

B3.1 The apparatus shall be capable of applying a reverse cycle displacement in increasing step intervals to the unrestrained purlin. Between the end connections, the purlins shall not be restrained from displacing normal to or rotating about their longitudinal axis.

B3.2 The test apparatus shall include a means of recording the applied longitudinal displacement and the corresponding force applied to develop the displacement for each cycle.

B4. LOADING CRITERIA

The cyclic racking load criteria shall increase the displacement in sinusoidal (crescendo) stepped intervals that ramp up to the expected seismic displacement (z) in the horizontal roof plane noted on the PC drawings, similar to AAMA 501.6-09 and ATC 24 (1992). The expected seismic displacement is the calculated differential deflection in the roof plane between corresponding panel connectors on the parallel purlins in the longitudinal axis of the purlins (see Figure B4.1 below). Prior to loading the unrestrained purlin, a starting point shall be located and marked. Each interval shall consist of a push and pull to the assigned step displacement on each side of the starting point, beginning and ending at the starting point. The crescendo intervals shall step up in deflection as follows: 0.25z, 0.5z, 0.75z, 1.0z, 1.25z, 1.25z, 1.25z, 1.25z & 1.5z. Each complete interval in the displacement cycle shall be performed at a frequency not to exceed 60 seconds, unless otherwise approved by DSA in the test protocol.

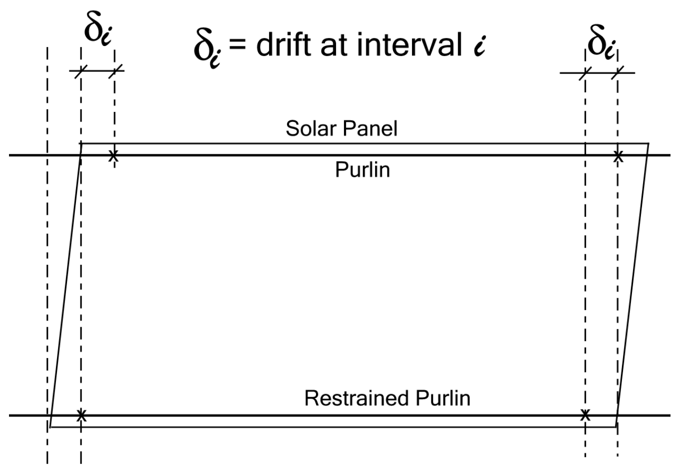


Figure B4.1

B5. ACCEPTANCE CRITERIA

Distress shall include, but not be limited to, in-plane or out-of-plane distortion/deformation or failure of the panel frame or glazing, evidence of shearing, elongation or distortion in the anchor or anchor device, and scoring or galling of the bearing contact surfaces.

B5.1 Connectors allowing slip of panel relative to purlin

B5.1.1 The solar panels may rotate and slip/shift such that no distress in the panel, connector or purlin is observed during the test nor is visible after the test is complete. After test is completed, the panel flange, bolt and purlin flange shall be inspected. No deformation or distress is acceptable in any component of the assembly.

B5.1.2 For clamp connectors that have an internal element (such as a pin, formed or cast tab, etc.) that restrains the panel from dislodging, the panels may shift or slip negligibly such that no visible distress in the panels is observed during or after loading of the displacement. After test is completed, the panel flange, connector device, connector fasteners, and purlin flange shall be inspected. No deformation or distress is acceptable in any component of the assembly.

B5.2 Connectors not allowing slip of panel relative to purlin

B5.2.1 No distress in the panel or connectors shall be visible and no slippage or shifting of panels shall be measurable during or after loading of the displacement. After test is completed, the panel flange, connector device, connector fasteners, and purlin flange shall be inspected. No deformation or distress is acceptable in any component of the assembly.

B5.3 Supplementary Restraining Devices

B5.3.1 For purposes of testing supplemental restraining devices, the panels shall not be connected to the supporting purlins and shall be free to shift, slip or rotate such that no visible distress in the panel flange, glazing, purlins, restraining device and its attachment to the purlins is observed during or after loading of the displacement. No deformation or distress is acceptable in any component of the assembly.

B5.3.2 If the panels can dislodge and potentially fall from the purlins, the supplemental restraining device must be capable of supporting the panels for gravity and/or wind loads.

B6. REPORTING

B6.1 The test report shall include the name and address of the testing laboratory, location of test site, date when test was completed, and date of issuance of report. The laboratory engineering manager shall sign the report.

B6.2 The test report shall also include the following:

B6.2.1 Identification and description of the specimen(s) – panel manufacturer and frame dimensions; connector model, material, type, size, dimensions, and method of attachment to purlins; purlin type, size, length; blocking type size and connection to purlin; detail of purlin end connections to apparatus; and any other pertinent information.

B6.2.2 Dimension of purlins and location of panels and connectors on purlins.

B6.2.3 A drawing of the test panel/connector/purlin assembly indicating location of measuring devices and movement devices.

B6.2.4 Complete description of test measurements and visual characterization of system and components both prior to horizontal displacement and after completion of each specified displacement interval.

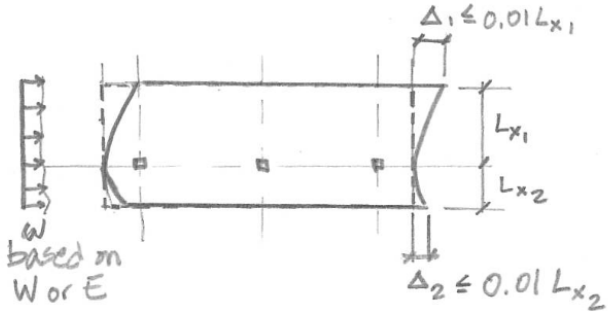
B6.2.5 A clear, definitive, written statement summarizing the observed performance of the panel test specimen in relation to the displacement requirements for the panels.

B6.2.6 Additional observations made by testing agency personnel during testing that may aid the specifier in evaluating system performance.

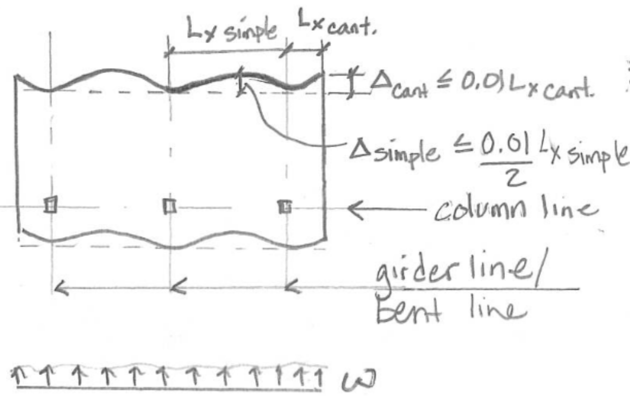
APPENDIX C: CALCULATIONS FOR DIAPHRAGM DEFLECTION

The following is an example per Section 2.2.6.3 above to exempt cyclic (i.e., in-plane racking) testing of the solar panel fastener assembly.

CASE 1 : Structures with diaph. or bracing, all bays:



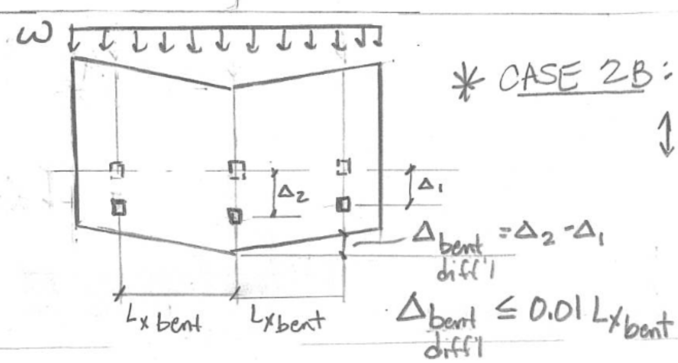
* CASE 1A: load dir. transverse to bents/girders ↔



* CASE 1B: load dir parallel to bents/girders ↕

CASE 2: Structures without diaph. or bracing

* CASE 2A : Same as 1A above ↔
  calculate Δ based upon weak axis bending of girder



* CASE 2B: Check roof plane displacement due to largest differential deflection between 2 adjacent bents – Calculate Δ based on column bending ↕

PC DESIGN CRITERIA FOR STEEL CANTILEVERED CANOPY STRUCTURES: 2025 CBC

APPENDIX D: GUIDELINES FOR DRAWING PRESENTATION

The following are guidelines and preferred practices to help assure a timely review at the OTC submittal. If too many options are presented within a single PC, the review can exceed the specified number of hours allowed for OTC review. The bullet points below are suggested as the best way to ensure a simplified review and approval of drawings and to simplify inspection in the field.

D1. **TABLE OF DRAWING SHEETS**: Include a table identifying the specific drawings applicable to each Profile and Loading Option (see D3 below). This is not the same as an index of drawings.

D1.1 See PR 07-01 Appendix E for more information and an example format that would be similar but is based on a relocatable building with options.

D2. **SITE SPECIFIC PARAMETERS**: Provide an area on the front sheet for site-specific seismic, wind, and snow (as applicable parameters to be filled in for project submittal).

D2.1 Provide Blanks to be filled in on the front sheet

D3. **GRAPHICAL PRESENTATION**: Per Section 1.4 above, each "Profile Option" and "Loading Option" shall be presented on separate drawings. There are no limits to the number of Profile Options that may be presented within a PC; however, the maximum the number of "Loading Options" for each "Profile Option" is 2 wind speeds and 2 seismic loads per PR 07-01.

D3.1 "**Profile Option**" is defined as the number of columns in a cross section and the shape of the structure (i.e., single column T shape, single column L shape sloped up, 2 column, 3 column, etc.). If there are significant dimensional or bay size options within each Profile Option, we recommend these also be separated out on separate charts and drawing sets. Each option drawing subset shall display structural cross section, plans, and details that can be separated into a submittal package without including non-applicable details.

D3.2 **"Loading Option**" is defined as a maximum load parameter (i.e., S_{ds} < 1.4, Wind < 120 mph, Snow< 50 psf).

D3.2.1 Examples of presentation as a separate sub-package of drawings:

- Profile Option: Single Column, T shape, Loading Option: S_{ds} < 1,4, Wind < 100 mph
- Profile Option: Single Column, T Shape, Loading Option: S_{ds} < 2.8, Wind < 120 mph
- Profile Option: Two Column, Loading Option: S_{ds} < 1.4, Wind < 96 mph
- Profile Option: Two Column, Loading Option: S_{ds} < 2.8, Wind < 120 mph

D4. **TABLE FORMAT FOR DESIGN ELEMENT OPTIONS**: Each structural element and detailing option within a Profile Option shall be presented clearly in a tabulated form. No important design features shall be presented in footnotes. DSA review and field personnel shall not have to interpret any design elements. Create a separate tabular entry for all options.

D4.1 Preferred: Table with check boxes

D5. **IDENTIFY DETAIL WITH OPTIONS**: Details that have options within them that shall be chosen at submittal should be clearly identified on the sheet.

D5.1 Preferred: Checkbox symbol next to the detail number

D5.2 Bold box around the detail or similar.

APPENDIX E: SOLAR PANEL PRETENSIONED FASTENER FIELD TESTING

Special inspection and testing of pretensioned panel fastener installation shall be performed by a qualified representative of the LOR as indicated in this Section . Employers of special inspectors shall be as specified in the California Administrative Code.

All special inspection activities shall be recorded by providing detailed daily inspection reports per *IR 17-12: Special Inspection Reporting Requirements* and be transmitted as required by the CAC.

E1. MATERIAL IDENTIFICATION TESTING

All pretensioned fasteners utilized in panel attachment connections shall be received in sealed containers and be readily identifiable for manufacturer, material specification, grade, size and type. Fastener identification shall be documented by a representative of the LOR. Unidentifiable fasteners shall be sampled and tested by a DSA-accepted laboratory at the frequencies prescribed in Section 3.2 of *IR 17-8: Sampling and Testing of High-Strength Structural Bolts, Nuts and Washers*. Identifiable pretensioned fasteners shall be sampled and tested by a DSA-accepted laboratory at the frequencies prescribed in IR 17-8 Section 3.1.

E2. SPECIAL INSPECTION AND TESTING

To ensure compliance with the contractor’s installation procedure and DSA-approved construction documents, special inspection of pretensioned panel fastener installation shall be performed as follows:

E2.1 Verify that specified fasteners are utilized.

E2.2 Verify that installers have access to and follow the installation procedure, including any specified calibrated installation equipment.

E2.3 Verify and document pre-installation qualification for each PV panel fastener installer before continuing with installation beyond this qualification procedure on the project as follows:

E2.3.1 Perform torque testing of 30 randomly selected (as specified in the California Administrative Code [CAC]) panel fasteners from the first 100 fasteners installed.

E2.3.2 If any fastener fails to meet installation torque requirements, test other fasteners not previously tested until 30 consecutive conforming tests are achieved.

E2.3.3 If 30 consecutive conforming tests are not achieved within the first 100 fasteners installed, install another 100 fasteners, and repeat the testing process described until 30 consecutive tests are achieved. All fasteners failing installation torque requirements shall be replaced and/or re-installed correctly by the installer and tested for conformance.

E2.4 Equipment and tools (e.g., torque wrenches) used for verification shall be provided by the LOR and calibrated in accordance with manufacturer's recommendations and pertinent standard (e.g., ANSI/ASME B107).

E3. SPECIAL INSPECTION AND TESTING RATES

After completion of the installer qualification described in Section E2 above, special inspection and torque testing of the remaining installed pretensioned panel fasteners (chosen randomly as specified in the CAC, unless installation concerns suggest otherwise) to verify that minimum torque values are achieved and that maximum values are not exceeded shall occur at least at the rates shown below to verify conformance for each installer:

PC DESIGN CRITERIA FOR STEEL CANTILEVERED CANOPY STRUCTURES: 2025 CBC

Table E1: Special Inspection and Torque Testing Rates

Total Number of Panel Fasteners on the Project	Special Inspection	% of Total to Be Tested
0 – 800	Continuous	50
801 – 1600	Continuous	33
1601 – 3500	Periodic*	20
3501 – 7500	Periodic*	10
More than 7500	Periodic*	5

*For projects with more than 1600 total panel fasteners, the first 1600 shall receive continuous inspection.

If any fastener fails torque testing, all fasteners of the same type and by the same installer, but not previously tested, shall be tested until 20 consecutive fasteners pass, then resume the initial test frequency.