DSA IR PC-7

PRE-CHECK (PC) DESIGN CRITERIA FOR STEEL CANTILEVERED CANOPY STRUCTURES: 2022 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 design and in plan review of steel cantilevered column 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 the 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. 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 DSA Codes and Standards Unit. For methods not specifically prescribed in the code, see California Building Code (CBC) Section 104.11.

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 steel cantilevered column structures designed in accordance with this IR.

SCOPE

The provisions of this IR apply to 2022 PC plans for new steel cantilevered canopy structures submitted to DSA under the 2022 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." They are used for various occupancies. Structural framing provides support for all vertical and lateral loads. These structures may or may not support solar photovoltaic or thermal systems. This document does not address moment frame structures that resist lateral loads primarily through the rigidity of beam to column connections.

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 the submittal is 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 submittal of plans to DSA for construction projects. 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 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

See PR 07-01 for a more detailed list of items that are required for all PC submittals 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 provided 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 2.4 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 6. 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 the 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 were required for weather protection or safety.

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

1.3 Structural Tests and Special Inspections

Provide an example form DSA 103: *List of Required Structural Tests and Special Inspections* on the drawings. See PR 07-01, Section 2.5 for additional information.

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

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

1.3.3 Only the site-specific form DSA 103 can incorporate exemptions from the required structural tests and special inspections in accordance with the appendix of the form DSA 103. 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 Appendix A below for additional information.

1.3.4 For projects involving solar installation, add a line item to the form 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 2.1.8 and 2.1.8.2). Add a line item to the form DSA 103 for material identification testing of solar panel attachment fasteners (see Section 2.7)

1.4 Options and Variations

The PC drawings shall provide checkboxes for options and variations if there is more than one configuration**.** See PR 07-01 Section 3 for more details, including the maximum number of options permitted in a single PC, and the graphical presentation of each option or variation.

1.5 Design Parameters

The PC documents shall provide 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(s), all or part of the Design Information should be presented in a checklist format and provide general direction to future users (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 of the cover sheet. In addition, a Code Analysis shall be shown on the coversheet to indicate the intended Use and Occupancy. The DSA reviewer of the site application shall verify the RC of the PC structure as it applies to the site in accordance with CBC Section 1604A.5. The Code Analysis shall include line items to indicate selection of Use and Occupancy classification per CBC Chapter 3, Occupant Load Factor (OLF) per CBC Table 1004.5, and determination of RC per CBC Table 1604A.5, to be completed by the Design Professional at time of OTC or project submittal. Refer to Appendix A below for additional information.

1.7 Flood Zone

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 from a geotechnical engineer (bearing his/her stamp and signature) 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 a geotechnical report written for improvements on the same campus and in accordance with the current CBC is provided that either (1) confirms the site is not in a flood hazard zone or (2) 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 the American Society of Civil Engineers Standard 24: Flood Resistant Design and Construction (ASCE 24) Section 7.2.

1.8 Geohazard Report

The design information section shall include a note stating when the design requires a geohazard report to be submitted to and approved by the California Geological Survey (CGS) with the site-specific application. See *IR A-4: Geohazard Report Requirements*.

For projects on existing sites outside of a mapped geologic hazard zone, cantilevered canopy structures 4,000 Square Feet (Sq. Ft.) or less complying with the requirements of IR A-4, Section 3.2 are exempt from the requirement to provide a geohazard report if design parameters of the structure are not dependent on geotechnical data. The structures may be split into multiple seismically separated structures to stay below the 4,000 Sq. Ft. trigger.

1.9 Weather Protection

PC drawings shall specify the type of weather protection selected for all weather-exposed steel members (structural steel and cold-formed steel) in accordance with CBC Section 2203A.1.

1.9.1 Structural steel shall comply with one of the following:

1.9.1.1 Hot dip galvanized, minimum ASTM A123 or A153 Class D, as applicable.

1.9.1.2 Painted with zinc-rich primer (undercoat and finish coat) or equivalent paint system.

1.9.2 Cold-formed steel members shall be 55 percent aluminum-zinc alloy coated per ASTM A792/A792M standard in accordance with the American Iron and Steel Institute (AISI) S240 Table A4-1, CP 90 coating designation.

1.9.3 All exposed steel fasteners, including cast-in-place anchor bolts/rods, shall be stainless steel (Type 304 minimum), hot-dip galvanized (ASTM A153, Class D minimum or ASTM F2329), or 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. (Example proprietary coatings that 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.

PC DESIGN CRITERIA FOR STEEL CANTILEVERED CANOPY STRUCTURES 2022 CBC

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 2022 CBC PC Stamp per PR 07-01 Section 6.1.

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

1.12 Structural Products Acceptance

All structural products shall meet the requirements set forth in *IR A-5: Acceptance of Products, Materials, and Evaluation Reports*. Code-based engineering calculations to support a manufactured product will be considered.

2. SOLAR PANEL REQUIREMENTS

2.1 Solar Panel Requirements on PC Plans

2.1.1 PC drawings shall specify: “Solar panels shall be listed and labeled in accordance with UL1703 or with both UL 61730-1 and UL 61730-2 per CBC Section 1511.9 for the panel orientations shown on PC plans.”

2.1.2 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 UL 1703 tests or with both UL 61730-1 and UL 61730-2 (and UL 2703 tests if utilized). If horizontal slip joints (e.g., thermal expansion joints) in framing members are present, solar panels must not span across nor be connected on opposing sides of the slip joints.

2.1.3 PC drawings must specify overall solar panel dimensions and fully dimensioned frame configuration of panel assumed in design of structure, including height, length, width, thickness of each web/flange and material grade. The actual panel to be installed shall be identified on PC plans if known.

2.1.4 PC drawings must specify the required solar panel load rating in pounds per square foot (psf) based on the actual design wind pressure per Section 2.1.6 below based on an effective area equal to the area of one PV panel. The required load rating shall be defined in terms of allowable capacity for use in ASCE 7 load combinations for allowable stress design. Provide a drawing, table or similar means indicating the actual design wind pressure similar to ASCE 7 Figures 30.7-1 to 30.7-3 that are used to determine the point loads.

2.1.5 PC drawings shall specify: “The load ratings for the solar panels selected by the contractor must meet or exceed the actual design wind pressures shown on the PC drawings.”

2.1.6 The PC drawings shall specify: “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, the number of fasteners required to achieve the rating, and whether the load rating is an unadjusted UL test load or a load that is modified by an identified safety actor.” 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 plans. The owner’s site professional shall submit the statement and panel documentation to DSA with the plan review package. If 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.

2.1.7 Panel anchorage details for each panel-to-purlin connector assembly that may be utilized shall be fully detailed on the PC plans. Anchorage details must 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.). Omitting solar panel anchorage design from PC is not permitted.

2.1.8 Connections Utilizing Pre-Tensioned Fasteners

2.1.8.1 Installation Procedure

Where pretensioned panel fasteners (e.g., bolts designed for clamp load) are specified, 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 must 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.

2.1.8.2 Installation Verification Testing and Special Inspection

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 Section 2.1.8 above of IR PC-7 *PC Design Criteria for Cantilevered Canopy Structures.”*

2.1.8.3 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:

2.1.8.3.1 Verify that specified fasteners are utilized.

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

2.1.8.3.3 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:

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

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

2.1.8.3.3.3 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.

2.1.8.3.4 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).

2.1.8.4 Special Inspection and Testing Rates

After completion of the pre-installation qualification described in Section 2.1.8.3.3 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:

Special Inspection and Torque Testing Rates Table

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.

2.1.9 All Other Connections

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

“The panel connections detailed here do not require pre-tensioned 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.”

2.2 Solar Panel Attachment Design Requirements

2.2.1 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 must 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 4.1 below, design must also consider shear due to racking from in-plane roof seismic forces. The attachment design shall comply with one of the following methods A, B, or C:

2.2.1.1 Method A

Panels attached directly to purlins with high-strength pre-tensioned bolts.

2.2.1.1.1 Bolts must be sized in accordance with Sections 6.4, 6.5 and 6.6 of UL 2703. Vertical load testing in accordance with Section 21 of UL 2703 of the panel/fastener assembly will not be required.

2.2.1.1.2 Requirements of a high-strength pre-tensioned bolt include all of the following: a minimum tensile strength of 95 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).

2.2.1.1.3 Minimum and maximum torque (or upper- and lower-bound pretension force) shall be specified on the PC plans 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.

2.2.1.2 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).

2.2.1.2.1 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, pre-tensioned 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). 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.

2.2.1.2.2 Vertical load testing in accordance with Section 21 of UL 2703 of the panel/fastener assembly is required prior to the approval of PC plans. The purlin segments used for testing, if not full-length per the PC plans, 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 plans, 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 plans. The test protocol shall be submitted to DSA for acceptance prior to conducting the test.

2.2.1.3 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.

2.2.2 Attachment of solar panels installed over structures with metal-deck diaphragms shall comply with requirements of this section. Alternatively, panels are permitted to be supported by and fastened to a rack or rail system (or equivalent) satisfying UL 2703. Attachment of rack or rail system to structure shall comply with recommendations of Structural Engineers Association of California (SEAOC) PV-1, SEAOC PV-2 and the CBC.

2.2.3 For all attachments that include bolts, the design shall include a mechanism for retention of nuts and prevention of loosening thereafter.

2.3 Cyclic Testing of Panel/Fastener Assembly

Each purlin/panel fastener assembly described in Sections 2.2.1.1, 2.2.1.2 or 2.2.1.3 above shall require cyclic testing in accordance with a DSA accepted protocol (See Appendix B below) unless the roof plane is equipped with a metal deck roof diaphragm, in-plane roof diagonal bracing, or deflection-limiting structural framing per Section 4.5.3 below. 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 plans. If both panel orientations are shown on PC plans, the test need only be performed for the orientation that yields the more severe requirements.

2.4 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 Sections 2.2 and 2.3 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.

2.5 Summary of Testing Requirements

See Figure 2.5 below for a summary of testing requirements as mentioned in Sections 4.5.3, 4.6.1, 4.6, 2.2 and 2.3 above. Section 2.4 above denotes requirements for an alternate design method in lieu of testing.

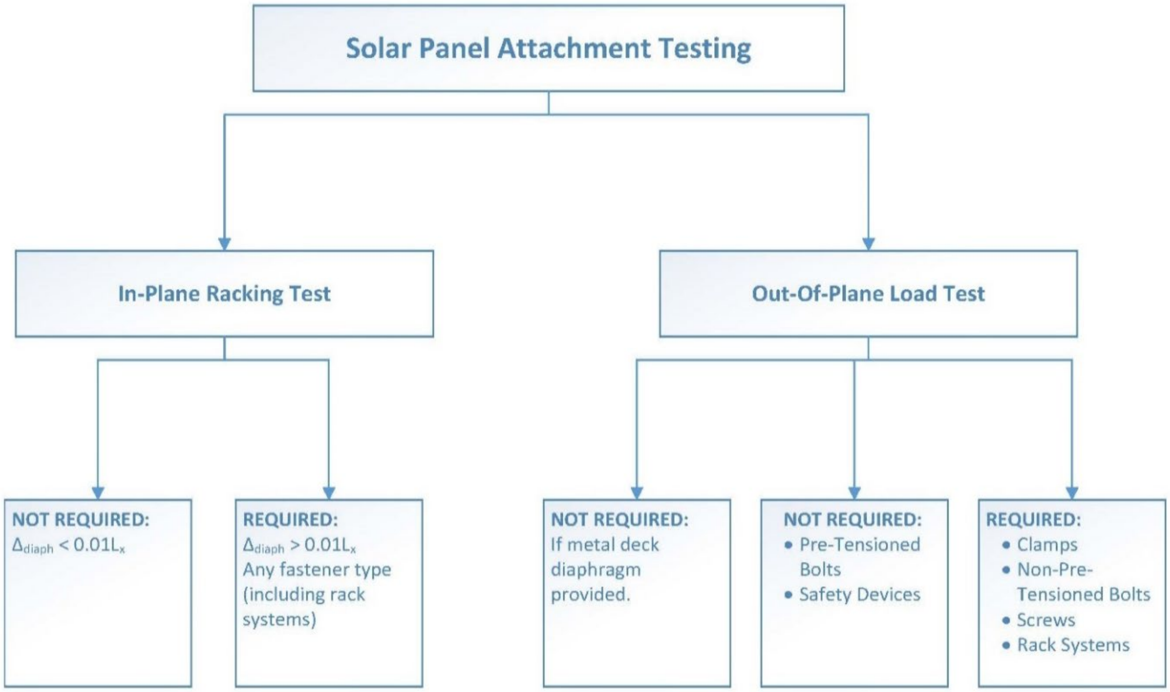

Figure 2.5: Summary of Testing Requirements for Solar Panel Attachments

2.6 Installation Verification Testing and Special Inspection

Special inspection and testing of pre-tensioned panel fastener installation shall be performed by a by a qualified representative of the LOR as indicated in Section 2.1.8 above. 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.

2.7 Pre-tensioned Panel Fastener Material Identification Testing

All pre-tensioned 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 pre-tensioned fasteners shall be sampled and tested by a DSA-accepted laboratory at the frequencies prescribed in IR 17-8 Section 3.1.

3. GRAVITY LOAD DESIGN

3.1 Dead Load

3.1.1 Design shall comply with dead load requirements per CBC 1606A and ASCE 7 Section 3.1.

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, see also IR 16-8, Section 2.1 and SEAOC PV-3, Section 4.

3.1.3 The dead load of rooftop-mounted photovoltaic panel systems, including rack support systems, shall be indicated on the construction documents. See CBC Section 1603A.1.8.1.

3.2 Live Load

3.2.1 Design shall comply with live load requirements per CBC 1607A and ASCE 7 Chapter 4 and SEAOC PV-3 Section 5.5.

3.2.2 For structures of open grid framing and no roof sheathing or decking, such as carports and shade structures supporting solar panels, the following two separate live loading conditions shall be applied in combination with other applicable loads:

- 12 psf uniform roof live load per CBC 1607A.14.4.3 and ASCE 7 Section 4.17.3 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.

3.2.3 As a condition for use of this reduced loading condition, the following note shall be shown on the construction plans: “No future roof decking or sheathing may be applied on the open grid framing.”

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

3.2.5 Snow Load

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

3.2.5.2 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.

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

3.2.5.4 Effective seismic weight shall include snow load per ASCE 7 Section 12.7.2. See IR 16-8 Section 2.3.

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

3.3 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: 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

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 modelled in a two-dimensional analysis, a three-dimensional analysis shall be required. 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 2.3 above.

4.2 Drift and Deflection

4.2.1 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 must also comply with seismic drift parameters for moment frame systems in ASCE 7 Section 12.1.1.1. 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 are exempted from these drift limits.

4.2.2 The maximum global vertical deflection at end of the beam shall not violate minimum clearances as required per specified use, and in no case shall be less than 7’-0” above highest adjacent grade.

4.3 Column Design

4.3.1 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 4.6 and 4.7 below for specific seismic and wind load design requirements.

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

4.3.3 Built-up columns shall comply with the American Institute of Steel Construction (AISC) 360 and 341, or AISI S100 and S213. Built-up columns of different materials shall not be permitted unless approved as an alternate design with supplemental full-scale testing.

4.3.4 If the allowable lateral bearing pressure is increased per Section 5.2 below, the column shall be evaluated for the additional imposed P-Δ (first order) effects using a point of inflection at 70 percent of the pier depth. See Figure 4.3 below. If this approach is used, include a note on the PC indicating the ½ inch movement at the base was considered during design; thus, the lateral bearing pressure increase per 1806A.3.4 is permitted.

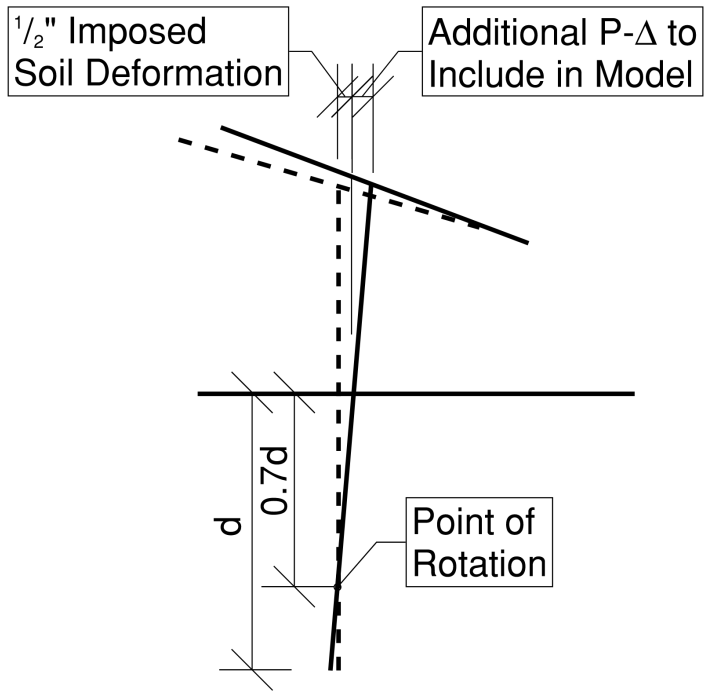

Figure 4.3: Surface Displacement per CBC Section 1806A.3.4

4.4 Pedestal Design

A pedestal is defined as a structural element that extends above the foundation 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 Section 25.7.2 for ties and 25.7.3 for spirals. For seismic loads, reinforcing shall also comply with ACI Chapter 18. 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.

4.5 Lateral Bracing Systems

4.5.1 Solar panels cannot be considered as providing any diaphragm action for the structure.

4.5.2 When diagonal rod bracing systems are used and they rely on pre-tensioning to reduce the sag of the rod, the design of the structure must include that pre-tensioning load in the analysis. 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.

4.5.3 Structures with metal deck roof diaphragms, in-plane roof diagonal bracing or deflection-limiting structural framing will be exempt from the requirement to cyclically test the panel fastener/purlin assembly per Section 2.3 above when 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.

4.6 Purlin Design

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 2.3 above.

4.6.2 Purlin top flanges shall be evaluated for localized stresses imposed by connections. 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, assuming an effective flange strip width of 1.5 times the width of the flange. Solar panels cannot 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.

4.6.3 Purlins to beam connections shall be designed to resist torsion per AISI S100 Section C2.2a, including Appendix A below. 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 unbraced length of purlins shall be detailed to restrain both web and flange of purlin in accordance with AISI S100 Section C2.2 (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.

4.6.6 See Section 4.8.6 below for specific wind load design requirements for purlins.

4.7 Seismic Design

4.7.1 Seismic Load Criteria

4.7.1.1 The design methodology used to analyze a cantilevered canopy structure shall be based on ASCE 7 Chapter 12 *Seismic Design Requirements for Building Structures* (not Chapter 15 *Seismic Design Requirements for Nonbuilding Structures*).

4.7.1.2 The R-factor shall be based on an “Ordinary” system if the concrete pier foundation is designed per Section 3 of *BU 09-06: Minimum Reinforcement of Concrete Piers and Caissons Embedded with Steel Poles* where reinforcing steel provided is less than required in CBC Section 1810A.3.9.

4.7.1.3 Redundancy Factor (ρ)

4.7.1.3.1 “T” and offset “T” configurations or those with no diaphragm shall have $\rho = 1.3$. For other configurations, they shall be justified and contain diaphragms. For elements required to be designed for overstrength, they shall be designed for the more severe loading of Ω or ρ factor considered independently.

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

4.7.1.4 Maximum Seismic Force

If the design is based upon the maximum S_S value for the state of California (ASCE 7-16 data), the PC can be used at any site in the state of California. Other S_S values are permitted but will limit the applicable site locations for the PC.

4.7.1.5 Ground Motion Hazard Analysis

The 2022 CBC adopts ASCE 7 with Supplement 3, which modifies Section 11.4.8. Due to the site-specific ground motion hazard analysis requirements of ASCE 7 Section 11.4.8, the seismic load criteria selected for the PC design per Section 4.7.1 above on Site Class D and E shall consider the Exceptions of ASCE Section 11.4.8, Items 1 and 2.

4.7.1.5.1 The PC option for Site Class D shall include following note in the design information section: "Unless a site-specific ground motion hazard analysis is performed, the S_{M1} value increased by 50% shall be less than the design criteria stated herein."

4.7.1.5.2 The PC option for Site Class E shall state in the design information section whether or not the PC design complies with the conditions of Exception 1 of ASCE 7 Section 11.4.8, Item 2.

4.7.1.6 Maximum S_{DS} Value in Determination of C_s and E_v

The base shear is permitted to be calculated using a cap on the maximum design spectral response acceleration parameter value of S_{DS} in accordance with ASCE 7 Section 12.8.1.3, provided that *all* of the noted criteria are met. The DSA reviewer shall verify any maximum limits applied to the base shear at time of OTC or project submittal. Refer to Appendix A below for additional information.

4.7.1.6.1 The PC design shall demonstrate compliance with the required criteria (e.g., no irregularities, period less than 0.5 s, rho equals 1.0, not Site Class E or F, RC I or II, etc.). The PC drawings shall list these properties in the Design Information section of the coversheet. For the purpose of checking compliance with these criteria, the period shall be determined based on the actual properties of the structure, including foundation flexibility, and not use the approximate period in ASCE 7 Section 12.8.2.1.

4.7.1.6.2 The design information section of the PC drawings shall also contain a note stating the site-specific limitations of the design based on the S_{DS} cap and requiring these to be verified by the site-specific project applicant (e.g., Site Class E not allowed, RC I or II, etc.).

4.7.1.6.3 Per PR 07-01 Appendix C, if a capped value of S_{DS} is used to determine C_S, the Design Information section of the coversheet shall list the S_{DS} (cap) used to determine C_S as well as the S_{DS} (no cap) used for verification of site-specific application and to determine other parameters such as non-structural component anchorage.

4.7.2 Column Design for Seismic

4.7.2.1 The column system design shall conform to AISC 341 Section E5 or E6. The column to foundation connection shall satisfy AISC 341 Section D2.6 for Column Bases. Compliance is required with D2.6a for required axial strength, D2.6b for required shear strength, D2.6c for required flexural strength, including the ductile limit state provision if D2.6c(b)(2) is used as a basis for the design. Base plate and anchor bolt designs shall also satisfy the design requirements of CBC 2204A.4.

4.7.2.2 The masses and heights of all equipment attaching to the columns shall be included in the column design.

4.7.3 High Strength Bolts

All bolts that are part of the seismic load resisting system (SLRS) (i.e., moment-resisting beam to column connections) shall be pretensioned high-strength bolts and shall 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 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 Class A faying surface requirement.

4.7.4 Structural Separation

Provide minimum seismic separations between adjacent cantilevered column structures in accordance with CBC Section 1617A.1.14.

4.7.4.1 Conduits installed across a structural separation shall have adequate loop (slack) to accommodate displacements between structures in accordance with ASCE 7 Section 13.3.2.2 with minimum vertical drop in the loop equal to the separation distance.

4.7.4.2 The Design Information section of the PC drawings shall define the maximum drift demand for each cantilevered column structure type, configuration, and option. This information is required so the following can be verified by design professionals and plan reviewers in the site-specific application of the PC design(s):

- Adequate separation is provided between adjacent structures.
- Adequate separation is provided relative to existing site structures.
- Utility details provide sufficient compensation for differential movement.

Refer to Appendix A below for additional information.

4.8 Wind Design

4.8.1 Wind Loads

The wind design requirements are given in CBC Section 1609A. Freestanding cantilevered column supported systems shall be designed using the open building provisions in ASCE 7 Section 27.3.2. The Net Pressure Coefficient C_N shall be determined based upon the specified angle of the roof slope, θ. An analysis shall be performed on the structure for: 1) gravity plus vertical wind and 2) gravity plus lateral wind loads for member design and to determine the global vertical deflection at the free edges of the cantilevers. See Section 4.2.2 for limits on deflection.

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 and Figures 30.7-1 through 30.7-3 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 must 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 loads is permitted per ASCE 7 Section 30.2.3. Cantilevered beams with a tributary area of less than 700 sf shall be designed for Components and Cladding wind 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 Panel Hold Down Design

Panel hold down connections shall be designed for Component and Cladding loading per ASCE 7 Figures 30.7-1 through 30.7-3.

4.8.6 Purlin Design for Wind

4.8.6.1 Purlins shall be designed for Component and Cladding loading per ASCE 7 Figures 30.7-1 through 30.7-3.

4.8.6.2 All exposed framing not enclosed by a soffit system shall be designed for a minimum load of 16 psf applied perpendicular to the member span direction per ASCE 7 Section 30.2.2 This minimum loading shall be applied simultaneously with the up and down C&C loading.

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. In order to use values greater than that stated for Class 5 soil, a statement requiring a site-specific geotechnical report at the time of site application must 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 Section 2.4. An allowable stress increase is permitted when using the alternative allowable stress 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. In order to use values greater than that stated for Class 5 soil, a statement requiring a site-specific geotechnical report at the time of site application must be included in the design information section on the PC drawings.

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 shall list the lateral bearing pressure value before the increase and indicate whether this value has been increased per CBC Section 1806A.3.4

5.3 Foundation Design Load

The design of the 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 and 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 if the column anchors at the top of the drilled pier (i.e., with base plate and anchor rods) or is partially embedded. See Sections 5.12 and 5.13 below. *BU 09-06* can be applied if the steel column is fully embedded (i.e., to within six inches of the bottom of the drilled pier).

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 when the drilled pier is assumed to be rigid.

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 eight and CGS does not otherwise require analysis per Section 5.4.3.2 below.

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 Provide a note in the design information section 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.

5.4.5 For drilled piers with partial column embedment or a base plate and embedded anchor rods, transverse reinforcing shall comply with CBC Section 1810A.3.9.4.2 and ACI 318.

Exception: The transverse reinforcement (i.e., tie or spiral) spacing need not be less than listed 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 For drilled piers in soil categorized as Site Class A, B, C or D, provide transverse reinforcement spacing not to 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 Design Criteria for Freestanding Signs, Scoreboards, and Ball Walls*.

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

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

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

5.4.5.1.4 Six inches: 6".

5.4.5.2 For drilled piers in soil categorized as Site Class E, provide transverse reinforcement spacing not to exceed that required by Section 5.4.5.1 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 5.4.5.4 below, it is recommended this transverse reinforcement spacing requirement simply be extended over the full depth of the drilled pier.

5.4.5.3 For drilled piers in soil categorized as Site Class A, B, C, D, or E provide transverse reinforcement spacing not to exceed the smallest of the following in the remainder of the drilled pier except as required by Section 5.4.5.4 below. Refer to IR PC-1 Figures 5.4A, 5.4B, 5.4C, and 5.4D.

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

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

5.4.5.3.3 Twelve inches: 12".

5.4.5.4 For drilled piers in soil categorized as Site Class A, B, C, D, or E, transverse reinforcement spacing shall comply with Section 5.4.5.1 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 ACI 318 Section 18.13.5.5.

5.4.6 For drilled piers with partially embedded columns, the transverse reinforcement spacing shall not be greater than that required by Section 5.12.1 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 obtain an allowable frictional resistance value. Assume 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 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. When a canopy/carport structure is supported by a combination of deep foundation element(s) (e.g., drilled pier) and shallow spread footing(s), all steel columns within the structure shall have the same height unless the column stiffness is accounted for in 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. 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.

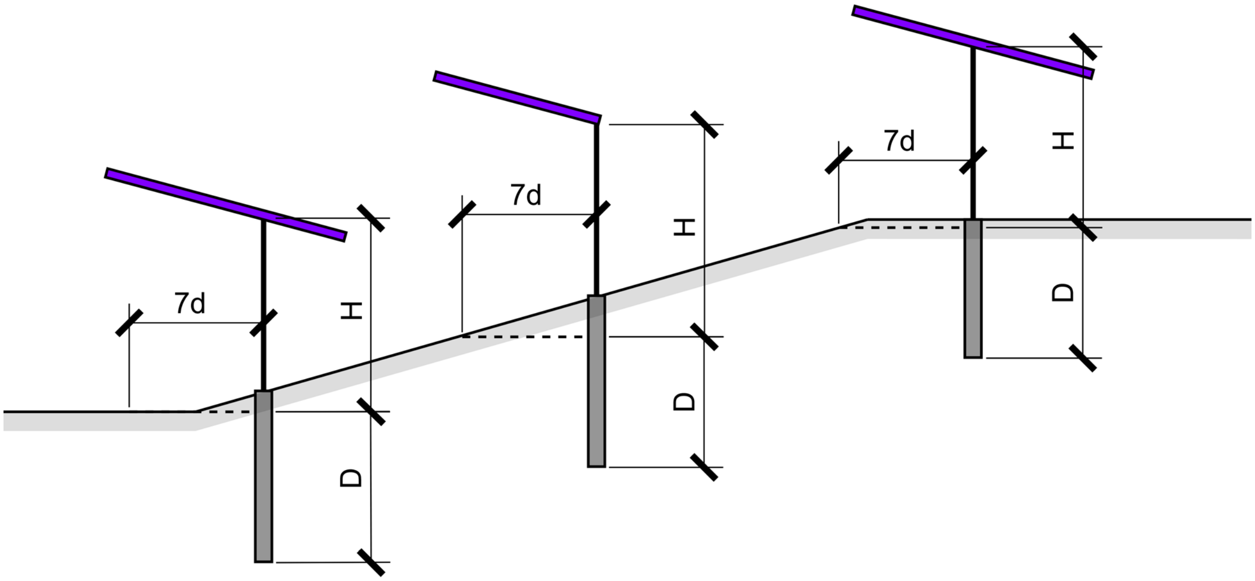

Figure 5.8: Sloped Sites

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). 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). If the setback limits are smaller than the CBC requires, a site-specific geotechnical report is required. If setback limits are smaller than 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 and/or 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 mix 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; 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; 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 Conduit 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 disregarded. 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 cantilevered columns are partially embedded into cast-in-place deep foundation (drilled pier), BU 09-06 is 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 the transfer the required force based on a rational method and accepted principles of engineering mechanics.

5.12.2 The minimum column embedment depth into the drilled pier shall be the greater of the following:

5.12.2.1 Seven times the least dimension of the column section.

5.12.2.2 Minimum development length of the longitudinal pier reinforcing.

5.12.3 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 four 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

5.13.1 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.

5.13.2 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. Refer to Telecommunication Industry Association (TIA) 222-H Section 4.9.9 and AISC *Design Guide 1: Base Plate and Anchor Rod Design*. When oversized holes are used in the base plate, the design shall comply with CBC Section 2204A.4.

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 Type of Construction

Specify type of construction per CBC Chapter 6 in Design Information on coversheet.

7.2 Total Area of Structure

Specify total area of structure in Design Information on coversheet and demonstrate that total area does not exceed allowable area per CBC Table 506.2 based on type of construction and proposed occupancy classification(s).

7.3 Use and Occupancy Classification(s)

Specify proposed use and occupancy classification(s) per CBC Chapter 3 in Design Information on coversheet.

REFERENCES:

2022 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.3.3.4.2,1810A.3.3.1.4, 2203A.1, 2204A.4, 2403, Tables 506.2, 1004.5, 1604A.3, 1607A.1

PC DESIGN CRITERIA FOR STEEL CANTILEVERED CANOPY STRUCTURES 2022 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 column structures designed in accordance with this IR. Appendix A is not intended to be an all-inclusive list of design and submittal 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 2022 CBC

- ☐ In addition to the requirements of PL 07-02 Section 4.9, 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 the site is classified as Site Class D or E and the seismic design of the PC is not based on the short period seismic response parameter SDS as indicated in the Design Information section of the PC drawings, verify if a site-specific ground motion hazard analysis is required. Refer to Section 4.7.1.5 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 column 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 on or 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 a ground motion cap is applied to the value of S_{DS} in determination of the seismic base shear, verify the required criteria per ASCE 7 Section 12.8.1.3 are met at time of OTC or project submittal. In addition, verify the value of S_{DS} for the site-specific application does not exceed the value of S_{DS} (no cap) listed on the PC drawings. Refer to Section 4.7.1.6 above for additional information.
- ☐ If the site has a ground snow load greater than zero, verify the steel cantilevered column 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 3.2.5 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, the differential movement as defined on the PC drawings. Refer to Section 4.2 above for additional information.
- ☐ If the steel cantilevered column 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.

PC DESIGN CRITERIA FOR STEEL CANTILEVERED CANOPY STRUCTURES 2022 CBC

APPENDIX B: CYCLIC TESTING OF SOLAR PANEL CONNECTORS

In-plane cyclic testing shall be performed when required by Section 2.3 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 plans. If both panel orientations are shown on PC plans, the test need only be performed for the orientation that yields the more severe requirements.

1. 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.

2. SAMPLE TYPE AND SIZE

The test, performed with materials and fasteners specified on the PC plans, 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 2.1 below.

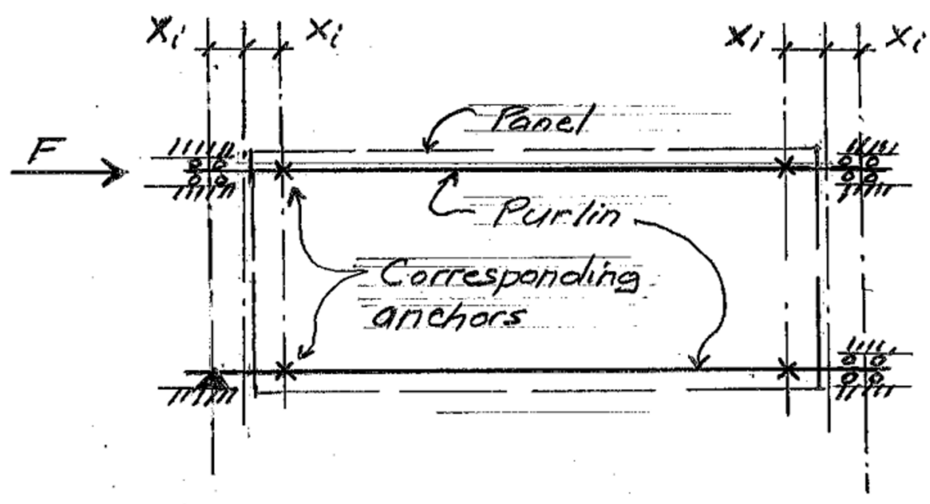

Figure 2.1

3. APPARATUS

3.1 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.

3.2 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.

4. 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 plans, 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 4.1 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.

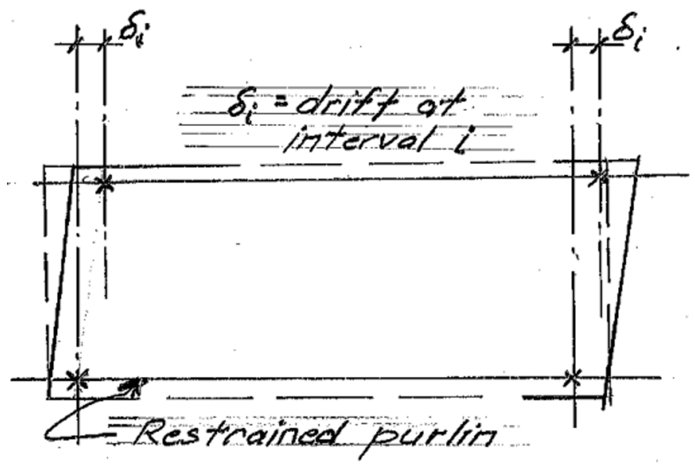

Figure 4.1

5. 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.

5.1 Connectors allowing slip of panel relative to purlin

5.1.1 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.

5.1.2 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.

5.2 Connectors not allowing slip of panel relative to purlin

5.2.1 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.

5.3 Supplementary Restraining Devices

5.3.1 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.

5.3.2 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.

6. REPORTING

6.1 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.

6.2 The test report shall also include the following:

6.2.1 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.

6.2.2 Dimension of purlins and location of panels and connectors on purlins.

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

6.2.4 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.

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

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