IR PC-7

PRE-CHECK (PC) DESIGN CRITERIA FOR STEEL CANTILEVERED COLUMN STRUCTURES (ORDINARY & SPECIAL): 2019 CBC

Disciplines: Structural

History: Issued 07/23/20

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 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 2019 PC plans for new steel cantilevered column structures submitted to DSA under the 2019 CBC after January 1, 2020. Steel cantilevered column 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”, 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*

1. GENERAL

1.1 Pre-Check Approval Requirements: See PR 07-01 for a more detailed list of items that are required for all PC submittals (e.g., design information on coversheet, example form *DSA 103: List of Required Structural Tests and Special Inspections* on the drawings, PC sheet index, etc.). 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 Design Information on Coversheet: See PR 07-01 Section 2.4 and Appendices B & C Require a note in the Design Information section requiring cut sheets of any boards, boxes and equipment to be mounted on the structure, including weights and dimensions.

Indicate snow and ice loads allowed. Note 0 pounds per square foot (psf) if not used in design.

1.3 Structural Tests and Special Inspections

1.3.1 Provide example DSA-103 on the drawings that will be used as a guide to develop a site-specific DSA-103 for the site-specific project. Example forms on the PC drawings will be crossed out when a site-specific DSA-103 is provided during plan review. See PR 07-01, Section 2.5.

1.3.2 A qualified representative of Laboratory of Record (LOR) or approved Special Inspector shall verify all steel identification per CBC Section 2202A.1 and IR 17-3: *Structural Welding Inspection*, Section 3.2.3.

1.3.3 For projects involving solar installation: Add a line item to the DSA 103 for installation verification testing and special inspection of solar panel attachments utilizing pretensioned bolts (e.g., bolts designed for clamp load; see Sections 6.1.8 and 6.6). Add a line item to the DSA 103 for material identification testing of solar panel attachment fasteners (see Section 6.7)

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

1.4 Options and Variations: Provide checkboxes of 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.

1.5 Design Parameters: Provide on the coversheet (and subsequent sheets as necessary) Design Information 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: PC drawings must indicate the maximum Risk Category (RC) the structure is designed for in the Design Information section of the coversheet. 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 for additional information.

1.7 Flood Zone

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

1.7.2 Provide a note in Design Parameter Information section indicating that when a site-specific project is located in a flood zone other than Zone X, a letter stamped and signed from a geotechnical engineer is needed to validate the allowable soil values specified in the PC drawings are still applicable. Unless the bottoms of foundations are raised above the design flood elevation, a validation letter from the geotechnical engineer shall be provided, even if the presumptive load bearing values per CBC Section 1806A.2 are used.

This note may include an exemption for the validation letter for projects located in Zone D (undefined) if the applicant provides evidence from the local jurisdiction or a qualified design professional confirming the site is not in a flood hazard zone. Refer to Appendix A for additional information.

1.7.3 Location of electrical elements shall conform to the American Society of Civil Engineers (ASCE) 24 Section 7.2 as required per *PR 14-01* Section 1.2.1.

1.8 Geohazard Reports: Provide a note in the Design Information section indicating that, for projects on existing sites outside of a mapped geologic hazard zone, cantilevered column structures 4,000 Square Feet (Sq. Ft.) or less complying with the requirements of *IR A-4: Geohazard Report Requirements*, Section 3.2 are exempt from the requirement to provide a geohazard report. The structures may be split into multiple seismically separated structures to stay below the 4,000 Sq. Ft. trigger.

1.9 Weather Protection

1.9.1 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.

1.9.2 Structural steel shall be hot-dip galvanized (minimum American Society for Testing Materials (ASTM) A123 or A153 Class D, as applicable) or painted with zinc-rich primer, undercoat, and finish coat; or equivalent paint system. Cold-formed steel members shall be 55 percent aluminum-zinc alloy coated per ASTM A792/A792M standard in accordance to 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.

1.10 PC Sheet Index: Provide a PC sheet index**.** See PR 07-01, Appendix E.

1.11 DSA ID Stamp and PC Stamp: Provide 2019 CBC PC Stamp per PR 07-01 Section 6.1.

Provide two blank areas on each PC sheet title block as indicated in procedure *PR 18-04: Electronic Plan Review for Design Professionals of Record Using Bluebeam*, Section 1.2.2.2: one for the PC ID stamp and one for the future site-specific DSA Identification Stamp. (See policy *PL 18-02: Record Sets of DSA-Approved Construction Documents*).

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. GRAVITY–ROOF

2.1 Dead Load: Design shall comply with dead load requirements per CBC 1606A and ASCE 7 Section 3.1.

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 Structural Engineers Association of California (SEAOC) PV-3, Section 4.

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.

2.2 Live Load: Design shall comply with live load requirements per CBC 1607A and ASCE 7 Chapter 4.

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.13.5.2.1 and ASCE 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.

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.”

See SEAOC PV-3 Section 5.5

Include unbalanced live load in design of structure.

2.3 Snow Load: Effective seismic weight shall include snow load per ASCE 7 Section 12.7.2. See IR 16-8, Section 2.1.3.

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

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

If the structure is designed for snow load, add note to the design parameters checklist: "Site application DSA reviewer shall verify the structure to be located at least xx (xx to be calculated by PC applicant) feet from any adjacent structure". See ASCE 7 Section 7.7. If the horizontal separation is less than 20 feet, snow drift analysis shall be provided by the PC applicant, and the project is not eligible for Over-the-Counter (OTC) review. Refer to Appendix A for additional information.

2.4 Deflection limits for girders and purlins: 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.

3. LATERAL–SEISMIC

3.1 ASCE 7 Chapter 12: Design of Steel Cantilevered Column Structures: Must utilize ASCE 7 Chapter 12 *Seismic Design Requirements for Building Structures* (not Chapter 15 *Seismic Design Requirements for NonBuilding Structures*).

3.1.1 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.

3.1.2 A 3D analysis shall be performed on the structure for gravity plus seismic 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. Unless exempted per Section 6 below, 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 6 below.

3.1.3 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 6 below 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.

3.2 Redundancy Factor (ρ) : "T" and offset "T" configurations or those with no diaphragm shall have ρ =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.

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.

3.3 Seismic Load Criteria

3.3.1 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.

3.3.2 Ground Motion Hazard Analysis: Due to the site-specific ground motion analysis requirements of ASCE 7 Section 11.4.8, PC designs shall be based on the short period seismic response parameter S_{DS} and ASCE 7 Equations 12.8-2 and 12.8-5. Where a PC design is provided for Site Class E, the short-period site coefficient as required by ASCE 7 Section 11.4.8 Exception 1 shall be used. Alternatively, if the PC design is not based on the short period seismic response parameter S_{DS}, the PC design shall comply with the requirements of ASCE 7 Section 11.4.8, and the Design Information section shall state the fundamental period of the structure(s) and include notes alerting the site specific user of the PC to the conditions requiring a site specific ground motion hazard analysis.

3.3.3 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 for additional information.

3.3.3.1 The PC drawings shall demonstrate compliance with the required criteria (e.g., no irregularities, period less than 0.5 sec, rho equals 1.0, not Site Class E or F, RC I or II, etc.) in the Design Information section of the coversheet. For purposes 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.

3.3.3.2 The PC drawings shall indicate the mapped spectral response acceleration parameter at short period, S_S in the Design Information section of the coversheet.

3.3.3.3 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.

3.4 Column Design

3.4.1 The column needs to conform to American Institute of Steel Construction (AISC) 341, Sections E5 or E6. Column to foundation connection must have sufficient capacity to develop the flexural strength of the steel column, utilizing the expected yield stress, $1.1R_yF_y$ per AISC 341, Sections A3.2 and D2.6, or from the load combinations with overstrength factor per ASCE 7 Section 12.4.3.2.

3.4.2 Columns must include P-Δ (first order) effects of the ½" displacement of footings if two times the lateral bearing pressures per CBC Section 1806A.3.4 is used. See Figure 1 below.

3.4.3 Built-up columns shall comply with 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.

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

3.5 Purlin Design

3.5.1 Purlins shall be designed for weak axis bending due to seismic loads if roof diaphragm is not present, i.e., 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 to establish the cyclical testing performance criteria per Section 6.3 below.

3.5.2 Top flange of the purlin 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 location on the flange and assumes an effective length of the flange as three times the distance from the fastener to the web. Solar panels cannot be considered as top flange bracing, provide lateral torsional buckling resistance to the purlin, or be used for any structural purpose other than delivering solar panel reactions to purlins.

3.5.3 Purlins to beam connection must be designed to resist torsion per AISI S100 Section D3.2, including Appendix B. If there is no torsional bracing along purlins, then the full torsion for the entire span needs to be resisted. If there is torsional bracing of the purlins between beams, then use distance to brace points.

3.5.4 Blocking provided to reduce unbraced length of purlins shall be detailed to restrain both web and flange of purlin in accordance with AISI S100-07 Section D3.2 (and Section D6.3.1 if purlin flange is attached to diaphragm).

3.5.5 Such blocking, where used on structures without metal deck diaphragms, shall be provided in rows in each bay between purlins across full width of roof plane to ensure that the roof plane deflects uniformly.

3.6 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 pretension 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 are eligible for Exemption 2 of this section; bolted side plate moment connections 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.

3.7 Structural Separation: Provide minimum seismic separations between adjacent cantilevered column structures in accordance with CBC Section 1617A.1.15.

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

3.7.2 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 for additional information.

3.8 Drift: RC III and IV structures must comply with the drift limits per ASCE 7 Table 12.12-1. This drift limit shall be applied at the edges of the roof structure which produce the most severe drift. Exception: per footnote "c", open structures classified as RC II are exempted from the drift limits of this table.

4. LATERAL–WIND

4.1 Wind Load: 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 27.4.3. Net Pressure Coefficient CN shall be determined based upon the specified angle of the roof slope, θ. A 3D 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. Maximum global vertical deflection at end of beam shall not violate minimum clearances as required per specified use, and in no case less than 7'-0" above highest adjacent grade.

4.2 Clear and Obstructed Wind Flow: Open structures shall be permitted to be designed for Clear Wind Flow per ASCE 7 Figures 27.4-4 through 27.4-7 and Figures 30.7-1 through 30.7-3 if calculations are provided justifying the use of Clear. If not, then must design 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.

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.

4.3 Cantilevered Beam Design: Where the Tributary Area is greater than 700 SF, the use of MWFRS loads is permitted per ASCE 7 Section 30.2.3; otherwise design the cantilevered beams for components and cladding wind loads.

4.4 Column Design

4.4.1 Must include wind load from projected area of column on column.

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

4.4.3 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.

4.4.4 Columns must include P-Δ (first order) effects of the ½" displacement of footings if two times the lateral bearing pressures per CBC Section 1806A.3.4 is used. See Figure E below.

4.5 Panel Hold Down Design: Panel hold down connection shall be designed for Component and Cladding loading per ASCE 7 Figures 30.7-1 through 30.7-3.

4.6 Purlin Design: Purlins shall be designed for Component and Cladding loading per ASCE 7 Figures 30.7-1 through 30.7-3.

All requirements noted in Section 3.5.1 above for Purlin Design for seismic loads are also applicable for wind loads. All exposed framing not enclosed by a soffit system requires a minimum of 16 psf per ASCE 7 Section 30.2.2 acting normal to the weak axis surface. This minimum loading needs to be applied simultaneously with the up and down C&C loading.

4.7 Drift: For open structures, there is no drift limit for RC II structures; however, there are drift limits for RC III and IV structures per CBC Section 1609A.1.2.

5. FOUNDATION

5.1 Allowable Soil Pressure and Bearing: Maximum soil pressure and bearing values shall be in accordance with Class 5 soil in CBC Table 1806A.2 unless justified by a site-specific geotechnical report. In order to use values above stated maximums 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.

5.1.1 An allowable stress increase is not permitted for Basic Allowable Load Combinations including footings per CBC Section 1605A.3.1. An allowable stress increase may be permitted with Alternative Basic Load Combinations per CBC Section 1605A.3.2.

5.2 Lateral Bearing Pressure Increase: Allowed per CBC Section 1806A.3.4.

5.2.1 Under Design Information section, list the tabulated soils lateral bearing pressure value before the increase and indicate whether this value has been increased per CBC Section 1806A.3.4 (which permits using two times tabular values) for pole footing design.

5.2.2 If two times the tabular value is used, the column shall be evaluated for the additional imposed P-delta (first order) effects using a point of inflection at 70 percent of the pier depth. Refer to Figure E. 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.

5.3 Foundation Design Load: The design of the foundation elements (both pier footings and shallow spread footings) and connections shall include overstrength factor for cantilever column systems per ASCE 7 Section 12.2.5.2 and CBC Section 1617A.1.16.

5.4 Pier Footing: Comply with CBC Section 1810A.3.9 if the column anchors at the top of the pier, or is partially embedded. See Sections 5.7, 5.13 and 5.14. See also BU 09-06, which can be followed if the steel columns are embedded to within six inches of the bottom of the pier.

See Section 5.8 for pier footings used in combination with shallow spread footings.

In accordance with CBC Section 1810A.2.4, the depth of pier footing is permitted to be designed per CBC Section 1807A.3.2 when the pier footing is assumed to be rigid. The pier footing 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 (8)). Otherwise, the pier footing design including its reinforcing shall consider the nonlinear interaction of the pier and soil (e.g., L-pile analysis including group effects as required by CBC Section 1810A.2.5).

Maximum pier embedment depth for the purpose of computing lateral pressure (S_1) is 12 feet per CBC Section 1807A.3.2.1.

Provide a note in the Design Information section specifying minimum clearance required when placing multiple steel cantilevered column structures. Design must consider group effects if applicable per CBC Section 1810A.2.5.

5.5 Allowable Pier Footing Frictional Resistance and Uplift Capacity: The allowable frictional resistance and uplift capacity used in the design 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 allowable frictional resistance value. Presume Class 5 soils as noted in Section 3.1 above. For uplift capacity per CBC Section 1810A.3.3.1.5, a Factor of Safety of 3 shall be applied to the frictional resistance value obtained in CBC Section 1810A.3.3.1.4; where uplift is due to wind or seismic loading, a Factor of Safety of 2 can be used.

5.6 Unconstrained vs. Constrained 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 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 Pier Reinforcing: For piers with partial column embedment or a base plate with anchor bolts, tie reinforcing shall comply with CBC Section 1810A.3.9.4.2, except the tie spacing need not be less than the following where $P_u<0.10f'_cA_g$ (see Figures A through D below). These exceptions are only applicable to steel cantilevered column structures. This type of structure is lightly loaded and has a low ductility demand. These exceptions may not be extended to any other types of structures.

- **Site Class A – D:** For the top 3d of the pier (where "d" is the pier diameter) provide the **smallest** of d/4, $6d_b$ (where "d_b" is the diameter of the longitudinal bars), and 6". For the remainder of section, provide smallest of d/2, $12d_b$, and 12". Refer to Figures A and C.
- **Site Class E:** For the top 7d of the pier (where "d" is the pier diameter) and 7d above and below the interface of each soil stratum, provide the smallest of d/4, $6d_b$ (where "d_b" is the diameter of the longitudinal bars), and 6". For the remainder of section, provide smallest of d/2, $12d_b$, and 12". Refer to Figures B and D.

For partially embedded columns, the ties shall be designed for the transfer force based on a rational method and accepted principles of engineering mechanics**.**

5.8 Shallow Spread Footing: Shallow spread footings shall be designed per CBC Chapter 18A and for stability in accordance with CBC Section 1605A.1.1.

When the cantilevered column structure is supported by a combination of pier footing(s) and shallow spread footing(s), all steel columns within the structure shall have the same column heights/lengths unless the column stiffness is accounted for in design.

5.9 Foundations on or Adjacent to Slopes: PC drawings shall specify minimum setback limits (values are required) of the structure per CBC Section 1808A.7.

Alternatively, the depth of required pier embedment can start from an elevation that corresponds with a horizontal clear distance of seven times (7x) the pier diameter that intersects with the slope (daylighting). If setback limits are smaller than CBC requires, a site-specific geotechnical report is required. Refer to Appendix A for more information.

If the pier is lowered per the above requirements, then the design height of the column for the purpose of calculating pier depth and moment must be increased to the theoretical top of the required pier embedment depth.

5.10 Liquefiable Soil or Site Class F: PC options shall not include liquefiable soil nor site class F. If the structure is located in an area with liquefiable soil or Site Class F, OTC submittal is not allowed and site-specific project submittal is required. 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.

5.11 Concrete Mix

5.11.1 Unless concrete exposure is classified per the American Concrete Institute (ACI) 318 Section 19.3.1.1 whereby the concrete mix design can comply with ACI 318 Section 19.3.2.1, concrete mix shall consist of Type V cement with minimum compressive strength of 4,500 psi and maximum water/cement ratio of 0.45.

5.11.2 If ACI 318 Section 19.3.2.1 is used for concrete mix design, the PC shall clearly indicate the exposure levels applicable in the Design Information section. This may be tabulated for multiple categories or limited to specific exposure categories with conservative values.

5.11.3 Concrete exposed to thaw and freeze cycles shall be air entrained per ACI 318 Section 19.3.3.1.

5.12 Conduit in Piers: The PC drawings shall clearly show the size and number of the conduits adjacent to or penetrating through the pier. Provide elevation showing the location of the conduits relative to the piers. The presence of conduits may cause the portion of pier above the point of entry of conduits to be disregarded. The impact of conduit on the foundation strength, effective column height, and depth of embedment for these affected piers shall be justified by calculation.

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

5.13 Partially Embedded Columns: When cantilevered columns are partially embedded into **concrete** foundation piers, BU 09-06 is not applicable.

5.13.1 The load transfer mechanism of partially embedded columns shall include the design of both the column and concrete pier ties. Minimum column embedment depth into concrete pier shall be the greater of the following:

- Seven times the least dimension of the column section.
- Minimum development length of the longitudinal pier reinforcing.

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

Exception: for steel columns with minimum 48” embedment into concrete, an allowable bonding stress of 25 psi between steel columns and concrete is permitted to be utilized. The first 12” of embedment must be disregarded and no increase in this allowable bonding stress value is permitted for wind or seismic loads.

5.14 Column Base Connection

5.14.1 Embedment depth of the anchor rods shall be sufficient to lap with the longitudinal pier reinforcement. The lap length shall be based on developing the longitudinal pier reinforcement beyond the projected failure plane of the anchor bolt heads. Refer to Figures A through D and ACI 318 Figure R17.4.2.9. The lap length is not permitted to be reduced based on providing reinforcement beyond that required for the applied loads.

5.14.2 Anchor rods shall be designed for combined shear and tension. If the maximum grout thickness between the top of footing 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 American National Standards Institute/Telecommunication Industry Association (ANSI/TIA)-222-H Section 4.9.9 and Design Guide 1: Base Plate and Anchor Rod Design published by AISC. When oversized holes are used in the base plate, the design shall comply with CBC Section 2204A.4.

6. SOLAR PANELS

6.1 Solar Panel Requirements on PC Plans

6.1.1 PC drawings shall specify: "Solar panels shall be listed and labeled in accordance with UL1703 per CBC Section 1510.7.2 for the panel orientations shown on PC plans."

6.1.2 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 (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.

6.1.3 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.

6.1.4 PC drawings must specify the required solar panel load rating (in psf) based on the actual design wind pressure per Section 5 above based on an effective area equal to the area of one PV panel. The required load rating shall be defined in terms of both allowable and strength level capacity and clearly label both. 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.

6.1.5 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."

6.1.6 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 as allowable or strength level and specify whether the loads listed are design or test values from the UL 1703 tests." Upon acceptance, the PC design professional shall provide a statement to 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.

6.1.7 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 fastener 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.

6.1.8 Connections utilizing pre-tensioned fasteners

6.1.8.1 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 responsible design engineer 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.

6.1.8.2 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 6.1.8 of DSA IR PC-7 *PC Design Criteria for Steel Cantilevered Column Structures (Ordinary and Special): 2019 CBC.*"

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:

- Verify that specified fasteners are utilized.
- Verify that installers have access to and follow the installation procedure, including any specified calibrated installation equipment.
- Each installer must demonstrate, with at least three distinct examples, acceptable initial fastener installation for each panel installation configuration that they install for the project.
- For projects with up to 1600 total panel fasteners, provide continuous special inspection for each installer (unless the panel fastener testing frequency is increased as described below).
- For projects with more than 1600 total panel fasteners, the first 1600 fasteners shall follow the requirements for 1600 or fewer total panel fasteners. Special inspection frequency may be reduced from continuous to periodic for the remaining panel fasteners.

Torque testing of installed pretensioned panel fasteners (chosen randomly as specified in the California Administrative Code (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:

- Up to 1600 total panel fasteners: 20 percent (continuous special inspection for all fasteners required unless the testing frequency is increased to achieve at least a 95 percent confidence level and 5 percent margin of error.)
- 3500 or more total panel fasteners: 10 percent (linear interpolation is permitted for panel fasteners between 1600 and 3500).
- More than 7500 total panel fasteners 5 percent (linear interpolation is permitted for panel fasteners between 3500 and 7500).

Exception: The above-noted torque testing minimums may be reduced to that which achieves a 95 percent confidence level and 5 percent margin of error if the compliance rate is above 50 percent for the first 30 tested for each installer and installation configuration.

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 twenty consecutive fasteners pass, then resume the initial test frequency. 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).

6.1.9 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."

6.2 Solar Panel Attachment Design Requirements

6.2.1 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 above, 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:

- **<u>Method A:</u> Panels attached directly to purlins with high-strength pre-tensioned bolts:** 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.

  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).

  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.

- **<u>Method B:</u> 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):** 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). 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.

  *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.

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.

- **Method C: Panels utilizing attachment methods other than 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.

6.2.2 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 SEAOC PV-1, SEAOC PV-2 and CBC.

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

6.3 Cyclic Testing of Panel/Fastener Assembly: Each purlin/panel fastener assembly described in Sections 6.2.1a, b or c above shall require cyclic testing in accordance with a DSA accepted protocol (See Appendix B, this document) unless the roof plane is equipped with a metal deck roof diaphragm, in-plane roof diagonal bracing, or deflection-limiting structural framing per Section 3.1.3 above. 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.

6.4 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 6.2 and 6.3 above, with DSA pre-approval of the method, a 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.

6.5 Summary of Testing Requirements: See Figure F for a summary of testing requirements as mentioned in Sections 3.1.3, 3.5.1, 4.6, 6.2 and 6.3. Section 6.4 denotes requirements for an alternate design method in lieu of testing.

6.6 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 6.1.8. 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 California Administrative Code (CAC).

6.7 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. Undentifiable 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.

7. ACCESS COMPLIANCE REQUIREMENTS

7.1 Protruding Objects in Circulation Areas: Protruding objects such as column-mounted equipment shall comply with CBC Section 11B-307.

7.2 Vertical Clearance in Circulation Areas: Vertical clearance to supporting structures shall comply with CBC Section 11B-307.4.

7.3 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.

7.4 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.

8. FIRE AND LIFE SAFETY REQUIREMENTS

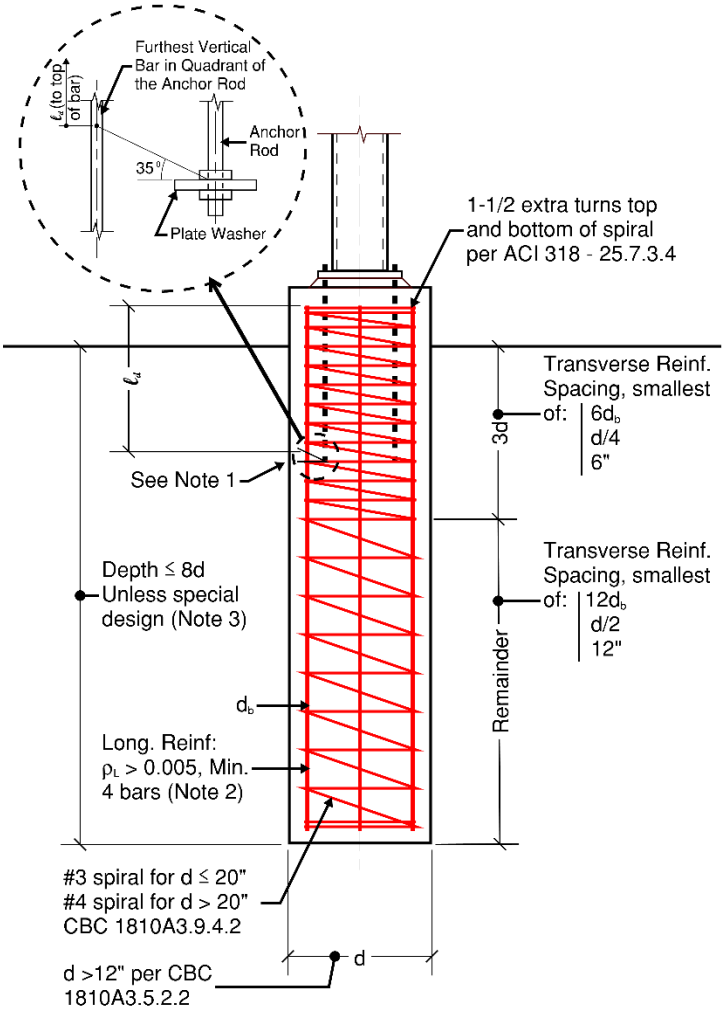
8.1 Type of Construction: Specify type of construction per CBC Chapter 6 in design information on coversheet.

8.2 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).

8.3 Use and Occupancy Classification(s): Specify proposed use and occupancy classification(s) per CBC Chapter 3 in Design Information on coversheet.

PC DESIGN CRITERIA FOR STEEL CANTILEVERED COLUMN STRUCTURES (ORDINARY & SPECIAL): 2019 CBC

Figure A



Note 1: Development length of vertical reinforcing steel that engages the anchor rod, ℓ_d, must meet ACI 318 Section 25.4. It is measured from the top of vertical bar to the point shown in the enlarged detail. Alternatively, the embedment depth of the anchor rod may be determined as the development length of the vertical bar plus one quarter of the pier diameter (ℓ_d + d/4).

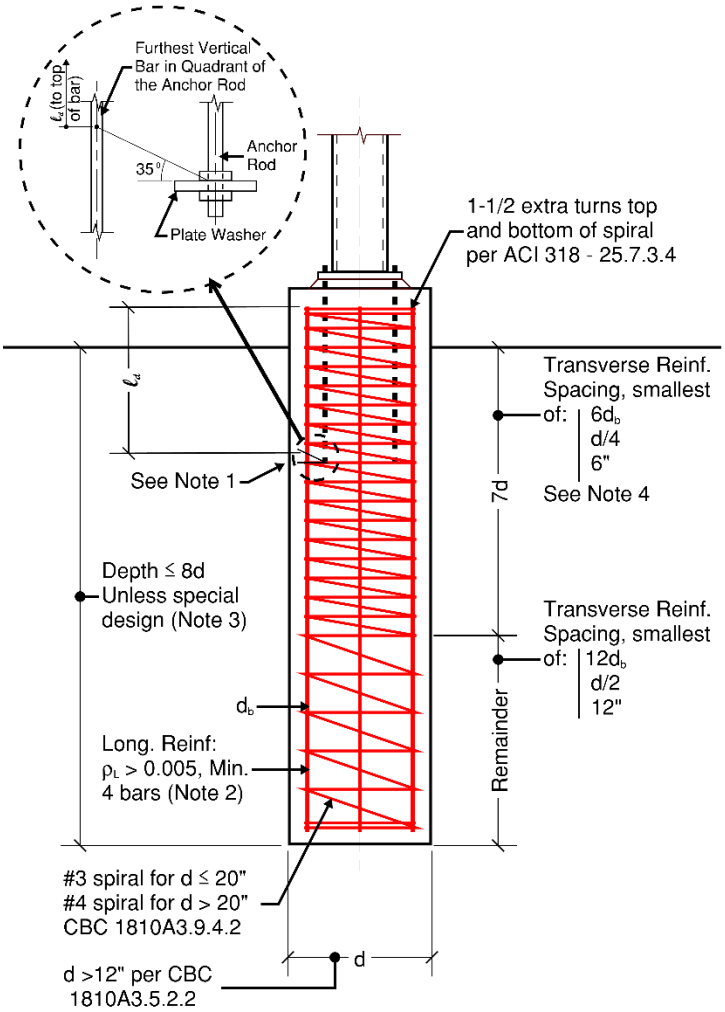
Note 2: The total longitudinal reinforcing steel area (A_s) shall be enough to transfer the loads from the total required anchor rod area ($A_{rod\text{-}req}$) taking into account relative yield stress, as follows: $A_s \geq A_{rod\text{-}req} \times (f_{y\text{-}rod} / f_{ys})$
However, if the spacing between vertical bars is 6" or less, the steel area (A_s) need not exceed 33% more than that required to resist the column design moment in the pier section.

Note 3: The prescriptive longitudinal and transverse reinforcing requirements shown may be used when the embedded pier depth to diameter ratio ≤ 8. If this ratio is exceeded, the pier must be analyzed and designed as required per CBC Section 1810A.2.4.

Spiral Reinforcing in Site Classes A-D

PC DESIGN CRITERIA FOR STEEL CANTILEVERED COLUMN STRUCTURES (ORDINARY & SPECIAL): 2019 CBC

Figure B



Note 1: Development length of vertical reinforcing steel that engages the anchor rod, ℓ_d, must meet ACI 318 Section 25.4. It is measured from the top of vertical bar to the point shown in the enlarged detail. Alternatively, the embedment depth of the anchor rod may be determined as the development length of the vertical bar plus one quarter of the pier diameter (ℓ_d + d/4).

Note 2: The total longitudinal reinforcing steel area (A_s) shall be enough to transfer the loads from the total required anchor rod area ($A_{rod\text{-}req}$) taking into account relative yield stress, as follows: $A_s \geq A_{rod\text{-}req} \times (f_{y\text{-}rod} / f_{ys})$
However, if the spacing between vertical bars is 6" or less, the steel area (A_s) need not exceed 33% more than that required to resist the column design moment in the pier section.

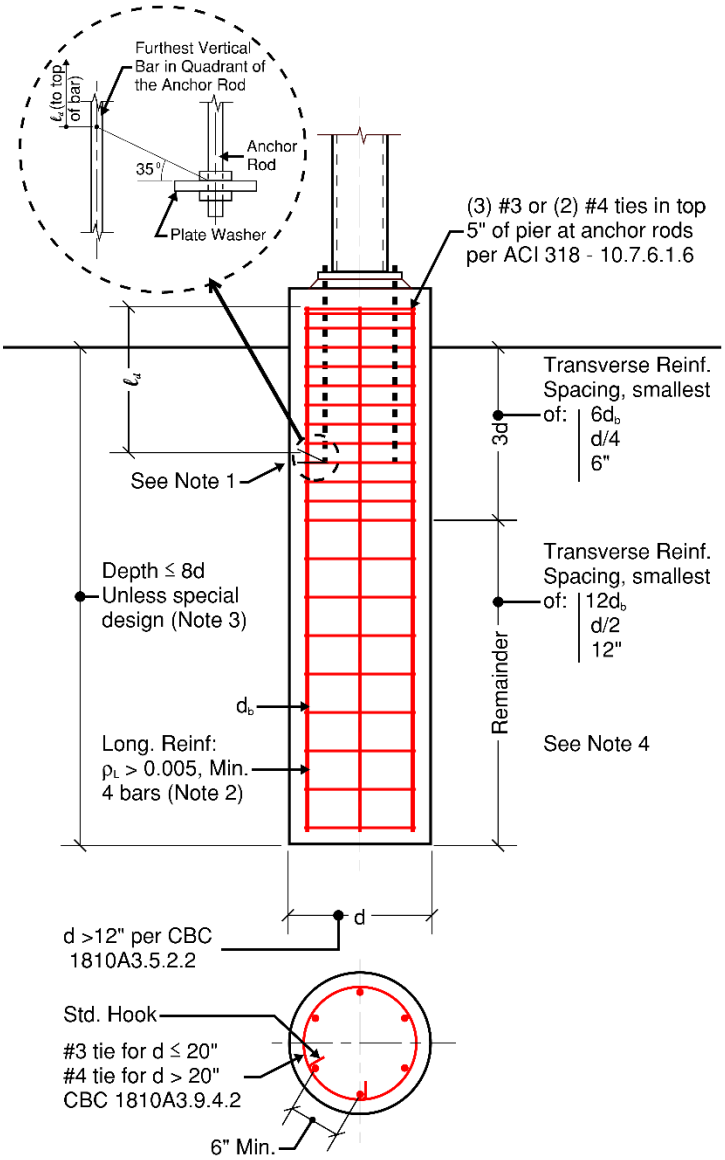
Note 3: The prescriptive longitudinal and transverse reinforcing requirements shown may be used when the embedded pier depth to diameter ratio ≤ 8. If this ratio is exceeded, the pier must be analyzed and designed as required per CBC Section 1810A.2.4.

Note 4: The 7d distance is also required above and below soil strata identified as liquefiable or hard or stiff as required per CBC Section 1810A.3.9.4.2.2.

Spiral Reinforcing in Site Class E

PC DESIGN CRITERIA FOR STEEL CANTILEVERED COLUMN STRUCTURES (ORDINARY & SPECIAL): 2019 CBC

Figure C



Note 1: Development length of vertical reinforcing steel that engages the anchor rod, ℓ_d, must meet ACI 318 Section 25.4. It is measured from the top of vertical bar to the point shown in the enlarged detail. Alternatively, the embedment depth of the anchor rod may be determined as the development length of the vertical bar plus one quarter of the pier diameter (ℓ_d + d/4).

Note 2: The total longitudinal reinforcing steel area (A_s) shall be enough to transfer the loads from the total required anchor rod area ($A_{rod\text{-}req}$) taking into account relative yield stress, as follows: $A_s \geq A_{rod\text{-}req} \times (f_{y\text{-}rod} / f_{ys})$
However, if the spacing between vertical bars is 6" or less, the steel area (A_s) need not exceed 33% more than that required to resist the column design moment in the pier section.

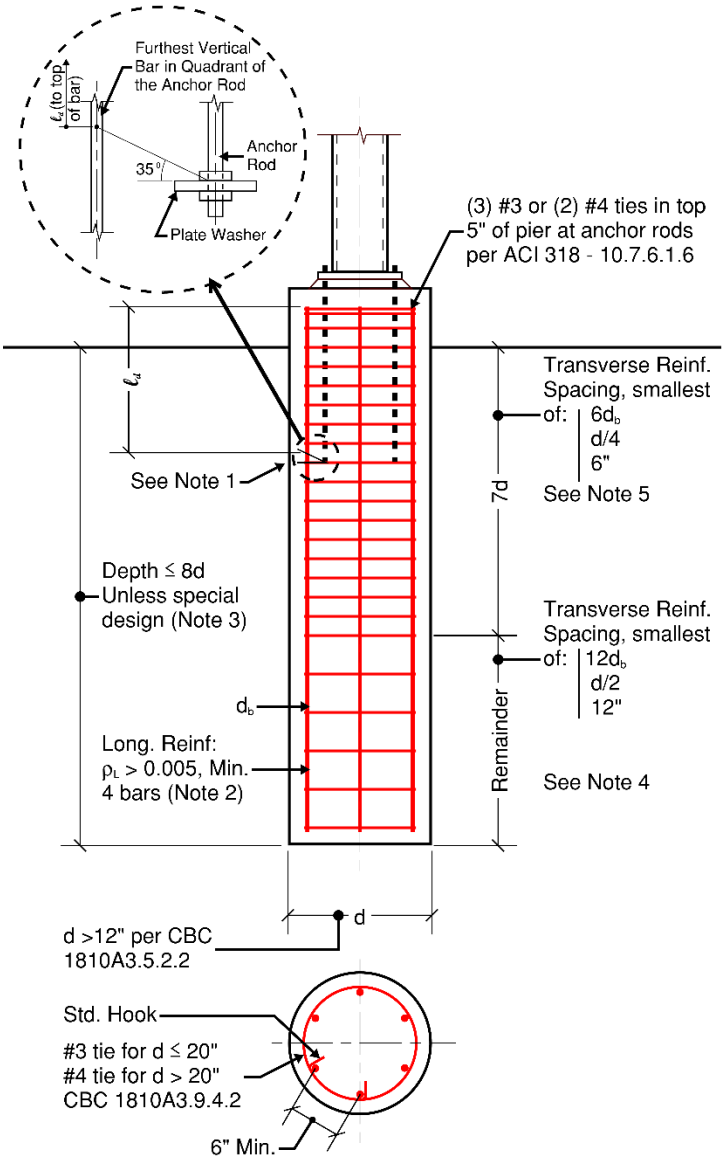
Note 3: The prescriptive longitudinal and transverse reinforcing requirements shown may be used when the embedded pier depth to diameter ratio ≤ 8. If this ratio is exceeded, the pier must be analyzed and designed as required per CBC Section 1810A.2.4.

Note 4: Ties shall comply with ACI 318 Section 25.7.2.4.1 including: (a) ends overlap by at least 6", (b) ends terminate with a standard hook engaging a longitudinal bar, and (c) overlaps of adjacent ties are staggered around the perimeter.

Tie Reinforcing in Site Classes A-D

PC DESIGN CRITERIA FOR STEEL CANTILEVERED COLUMN STRUCTURES (ORDINARY & SPECIAL): 2019 CBC

Figure D



Note 1: Development length of vertical reinforcing steel that engages the anchor rod, ℓ_d, must meet ACI 318 Section 25.4. It is measured from the top of vertical bar to the point shown in the enlarged detail. Alternatively, the embedment depth of the anchor rod may be determined as the development length of the vertical bar plus one quarter of the pier diameter (ℓ_d + d/4).

Note 2: The total longitudinal reinforcing steel area (A_s) shall be enough to transfer the loads from the total required anchor rod area ($A_{rod\text{-}req}$) taking into account relative yield stress, as follows: $A_s \geq A_{rod\text{-}req} \times (f_{y\text{-}rod} / f_{ys})$
However, if the spacing between vertical bars is 6" or less, the steel area (A_s) need not exceed 33% more than that required to resist the column design moment in the pier section.

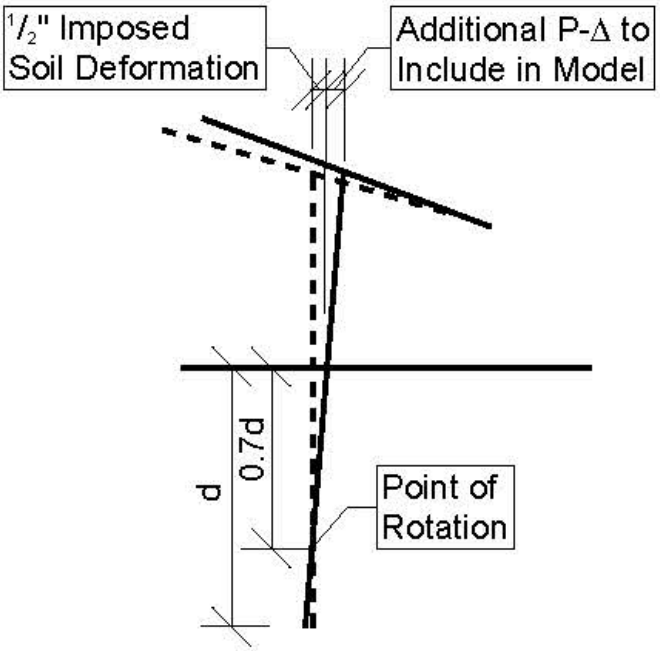
Note 3: The prescriptive longitudinal and transverse reinforcing requirements shown may be used when the embedded pier depth to diameter ratio ≤ 8. If this ratio is exceeded, the pier must be analyzed and designed as required per CBC Section 1810A.2.4.

Note 4: Ties shall comply with ACI 318 Section 25.7.2.4.1 including: (a) ends overlap by at least 6", (b) ends terminate with a standard hook engaging a longitudinal bar, and (c) overlaps of adjacent ties are staggered around the perimeter.

Note 5: The 7d distance is also required above and below soil strata identified as liquefiable or hard or stiff as required per CBC Section 1810A.3.9.4.2.2.

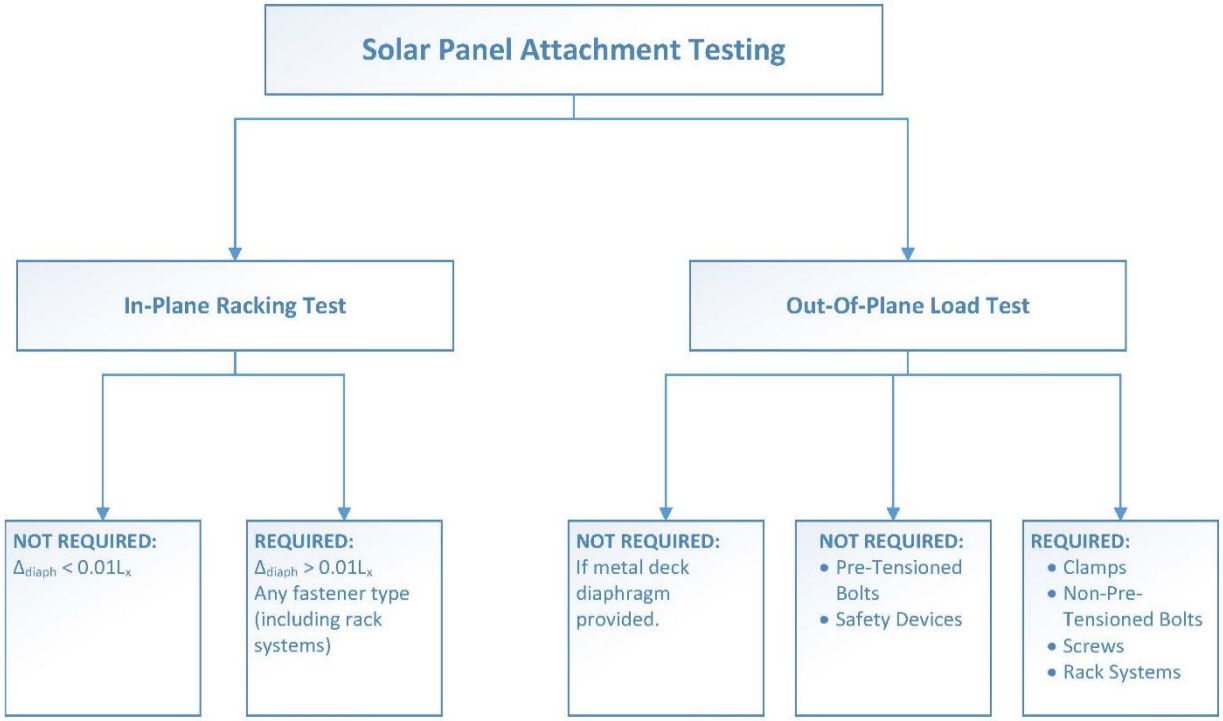
Tie Reinforcing in Site Class E

Figure E



½" surface displacement per CBC Section 1806A.3.4

Figure F



Summary of Testing Requirements for Solar Panel Attachments

REFERENCES:

2019 California Code of Regulations (CCR) Title 24

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

Part 2: Californa Building Code (CBC), Chapters 3 and 6, Sections 104.11, 11B-307, 11B-502.5, 11B-503.5, 11B-812.4, 1510.7.2, 1603A.1.8.1, 1604A.5, 1605A.1.1, 1605A.3.1, 1605A.3.2, 1609A, 1612A, 1806A.3, 1807A.3, 1808A.7, 1810A.2, 1810A.3, 1812A, 2202A.1, 2203A.1, 2204A.4, Tables 506.2, 1004.5, 1604A.3, 1604A.5, 1806A.2.

This IR is intended for use by DSA staff and by design professionals to promote statewide consistency for review and approval of plans and specifications as well as construction oversight of projects within the jurisdiction of DSA, which includes State of California public schools (K–12), community colleges and state-owned or state-leased essential services buildings. This IR indicates an acceptable method for achieving compliance with applicable codes and regulations, although other methods proposed by design professionals may be considered by DSA.

This IR is subject to revision at any time. Please check DSA's website for currently effective IRs. Only IRs listed on the webpage at https://www.dgs.ca.gov/dsa/publications at the time of project application submittal to DSA are considered applicable.

PC DESIGN CRITERIA FOR STEEL CANTILEVERED COLUMN STRUCTURES (ORDINARY & SPECIAL): 2019 CBC

APPENDIX A SITE-SPECIFIC APPLICATION OF PC STEEL CANTILEVERED COLUMN STRUCTURES

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. This Appendix 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.

1. Verify site-specific suitability of the PC Steel Cantilevered Column Structure including all parameters in PL 07-02 Section 3.

2. Verify site-specific requirements of PL 07-02 Section 4 are met.

3. 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:

3.1 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.

3.2 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.

3.3 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.

3.4 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.

3.5 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.

4. 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.

5. Review the Appendix of the site-specific 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.

6. 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.

7. 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.

8. 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 3.3.2 above for additional information.

9. 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.

10. 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.

11. 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.

12. 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 5.8 above for additional information.

13. 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 5.9 above for additional information.

14. 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 3.3 above for additional information.

15. 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 2.3 above for additional information.

16. 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 3.8 above for additional information.

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17. 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 3.8 above for additional information.

18. 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 4.2 above for additional information.

19. Verify the solar panel documentation and acceptance letter from the PC design professional is provided. Refer to Section 6.1 above for additional information.

APPENDIX B CYCLIC TESTING OF SOLAR PANEL CONNECTORS

In-plane cyclic testing shall be performed when required by Section 6.3 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

1.1 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

2.1 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.

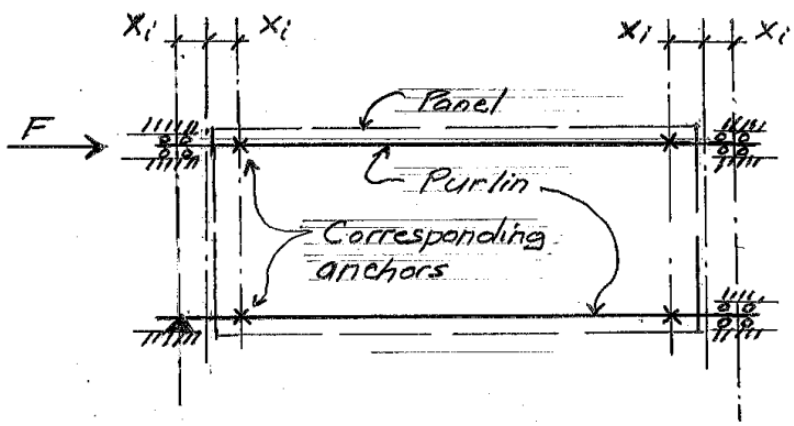

Fig. 2.1

APPARATUS

2.2 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.

2.3 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.

3. LOADING CRITERIA

3.1 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). 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.

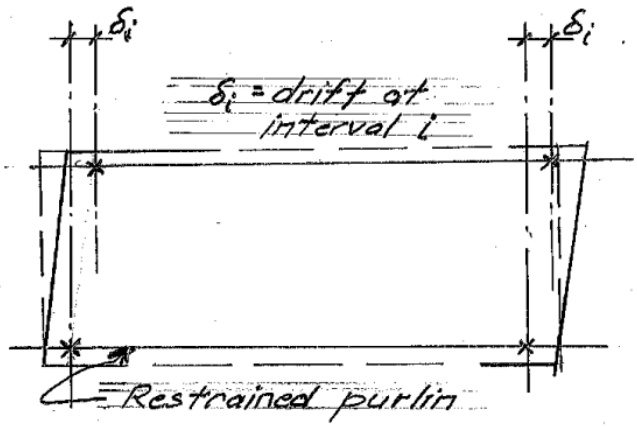

Fig. 4.1

ACCEPTANCE CRITERIA

Definition: 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.

3.2 Connectors allowing slip of panel relative to purlin

3.2.1 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.

3.2.2 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.

3.3 Connectors not allowing slip of panel relative to purlin

3.3.1 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.

3.4 Supplementary Restraining Devices

3.4.1 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.

3.4.2 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.

4. REPORTING

4.1 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 report shall be signed by the laboratory engineering manager.

4.2 The test report shall also include the following:

4.2.1 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.

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

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

4.2.4 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.

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

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