Feasibility and Issues with Installing Solar Arrays on Existing Parking Structures in California

Grace Liu

University of California, San Diego

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Section 1: Introduction

In the face of global warming, there is an increased demand for renewable energy sources, such as solar energy. In response to these changes, the California Public Utilities Commission (CPUC) aims to build solar grids in various California deserts, requiring more transmission lines and a large cost of construction, which poses a threat to the surrounding ecosystem.

However, there are many alternatives to large solar grids, such as adding solar panels to residential rooftops, parking lots, and parking structures. The benefit of these local energy alternatives is that the source is much closer to the city and requires fewer costly transmission lines.

Section 2: Research Objective and Scope

The objective of this study is to determine the feasibility of installing solar panels on existing parking structures. In order to assess this, two main factors must be considered, which are the strength of the concrete column (see Section 5.3) and the strength of the anchor bolts (see Section 5.4).

Section 3: Existing Solar Panels

3.1 Overview

Seven solar panels manufactured by various companies were analyzed, as shown in the table below.

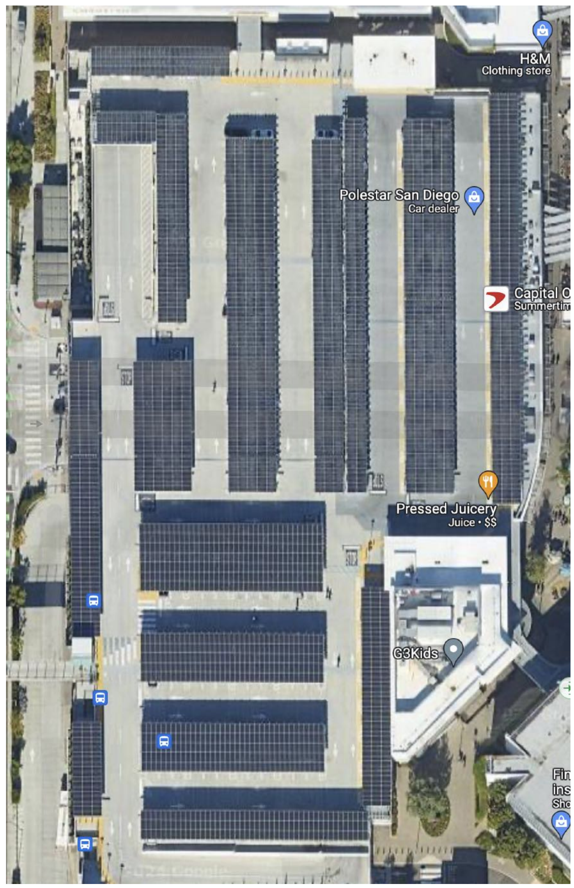
Company	Model	Weight (kg)	Length (ft)	Width (ft)	Height (ft)
LONGi Solar	Hi-Mo 5m	45.86	5.650	3.720	0.098
	Hi-MO 5	70.11	7.474	3.720	0.098
Trina Solar	Vertex S+ Bifacial - NE09RC.05	46.96	5.781	3.720	0.098
	Vertex x - DE0907 (DE09C.07)	46.30	5.755	3.596	0.098
Canadian Solar	TOPBiHiKu7 (690 W ~ 720 W)	83.33	7.822	4.275	0.108
JA Solar	DeepBlue JAM72D42 LB	76.28	8.087	3.720	0.098
	DeepBlue JAM72D40 LB	71.65	7.654	3.720	0.098

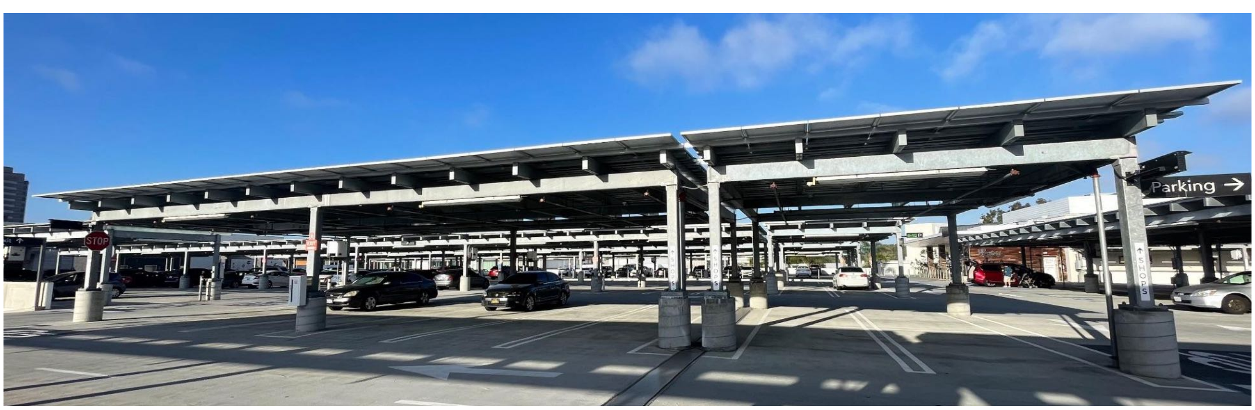
The solar panels were found to have a combined average weight of 58.5 lb, length of 6.50 ft, width of 3.81 ft, and height of 0.10 ft.

Not only are there variations among individual solar panels, but there are also differences among solar canopies, which consists of an array of solar panels. Sections 3.2-3.4 detail case studies of various solar canopies found on California parking structures.

3.2 Westfield UTC Parking Structure

The Westfield UTC Parking Structure, part of the University Town Center shopping mall in San Diego, consists of 5 levels, including the roof. The connected solar canopies vary widely in size and dimension, as shown in the following aerial view figure.





The side length of the square concrete base plate measures 16.1 inches, and the side length of the column measures 7.9 inches. 12 anchor bolts connect these components.

3.3 Gilman Parking Structure

The Gilman Parking Structure, located within the University of California, San Diego, consists of 6 levels. Each solar canopy is supported by a single column, and there are four anchor bolts between column and base plate.


Building 788
Gilman Parking Office
Gilman Parking Structure
ALR
9375
Russell Ln
Elizabeth Barrett-Connor, MD, FACP, FAHA
BluPeak Credit Union

The height of the concrete base is 52.7 inches, and the width and length of the square base plate is approximately 22 inches. The column is fastened by four anchor bolts.

3.4 Scripps Memorial Hospital La Jolla - Parking Lot D

Parking Lot D of Scripps Memorial Hospital consists of five levels. Each solar canopy is supported by 8 columns, with each base being 29.9 inches wide and 24.1 inches long. There are four anchor bolts between the column and base. Each canopy holds an array of 6 solar panels in width and 44 panels in length.

Section 4: Load Analysis

4.1 Load Analysis Overview

To determine the feasibility of installing a solar canopy, two load types must be calculated and compared: the load demand and the load supply. The load demand is the force that the canopy is exposed to, and it is dependent on external factors, including the geographic location, elevation, wind speed, etc. The load supply is the maximum force that the canopy is able to withstand, and it depends on its internal structure (the column diameter, number of anchor bolts, number of rebars, etc). In order for a solar canopy to be feasible, the load supply must exceed the load demand.

The load demand is based on a combination of the wind load W and dead load D. The rain load R, snow load S, live load L, and earthquake load E is assumed to equal zero. According to Section 2.3.1 of ASCE 7-16 (ASCE, 2017), the load capacity of the canopy must equal or exceed five different load combinations:

1. $1.4D$

2. $1.2D + 1.6L + 0.5(L_r \; or \; S \; or \; R)$
3. $1.2D + 1.6(L_r \; or \; S \; or \; R) + (L \; or \; 0.5W)$
4. $1.2D + 1.0W + L + 0.5(L_r \; or \; S \; or \; R)$
5. $0.9D + 1.0W$

The wind and dead load is defined in Sections 4.2 and 4.3, respectively.

4.2 Wind Load Overview

The wind load is the force exerted on the canopy due to wind, and it is calculated using seven steps from Section 29.4.1 and Table 29.1-1 of ASCE 7-16 (ASCE, 2017). See Table 1 for the outline.

Step 1: Determine Risk Category of building or other structure; see Table 1.5-1.
Step 2: Determine the basic wind speed, V, for applicable Risk Category; see Figs. 26.5-1 and 26.5-2.
Step 3: Determine wind load parameters:

- Wind directionality factor, K_d; see Section 26.6 and Table 26.6-1.
- Exposure category B, C, or D; see Section 26.7.
- Topographic factor, K_{zt}; see Section 26.8 and Fig. 26.8-1.
- Ground elevation factor, K_e; see Section 26.9 and Table 26.9-1
- Gust-effect factor, G; see Section 26.11, except for rooftop equipment.
- Combined (GC_r) factor for rooftop equipment; see Section 29.4.1.

Step 4: Determine velocity pressure exposure coefficient, K_z or K_h; see Table 26.10-1.
Step 5: Determine velocity pressure q_z or q_h; see Eq. (26.10-1).
Step 6: Determine force coefficient, C_f, except for rooftop equipment:

- Solid freestanding signs or solid freestanding walls, Fig. 29.3-1.
- Chimneys, tanks, Fig. 29.4-1.
- Open signs, single-plane open frames, Fig. 29.4-2.
- Trussed towers, Fig. 29.4-3.
- Rooftop equipment, using combined (GC_r) factors listed in Section 29.4.1.
- Rooftop solar panels, Fig. 29.4-7 and Eq. (29.4-6), or Fig. 29.4-8.

Step 7: Calculate wind force, F, or pressure, p:

- Eq. (29.3-1) for signs and walls.
- Eqs. (29.4-2) and (29.4-3) for rooftop structures and equipment.
- Eq. (29.4-1) for other structures.
- Eq. (29.4-5) or (29.4-7) for rooftop solar panels.

Table 1 Seven Steps to Determine Wind load (ASCE, 2017)

4.2.1 Step 1: Risk Category

Table 1.5-1 of ASCE 7-16 is used to determine the risk category of solar canopies (ASCE, 2017).

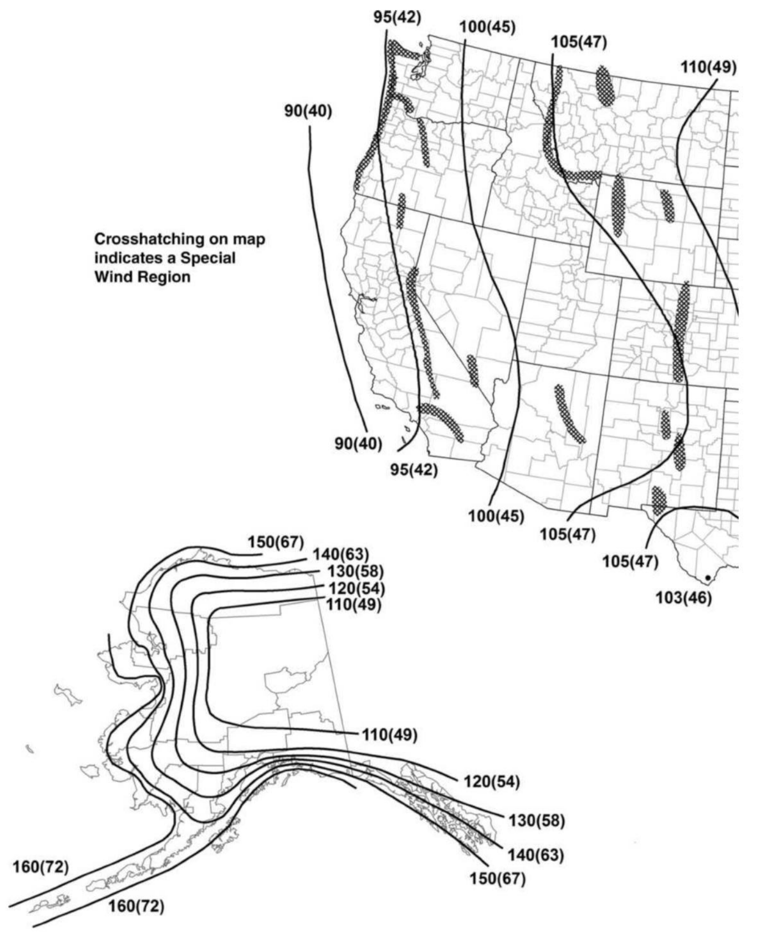
Use or Occupancy of Buildings and Structures	Risk Category
Buildings and other structures that represent low risk to human life in the event of failure	I
All buildings and other structures except those listed in Risk Categories I, III, and IV	II
Buildings and other structures, the failure of which could pose a substantial risk to human life Buildings and other structures, not included in Risk Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure Buildings and other structures not included in Risk Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing toxic or explosive substances where the quantity of the material exceeds a threshold quantity established by the Authority Having Jurisdiction and is sufficient to pose a threat to the public if released[a]	III
Buildings and other structures designated as essential facilities Buildings and other structures, the failure of which could pose a substantial hazard to the community Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing sufficient quantities of highly toxic substances where the quantity of the material exceeds a threshold quantity established by the Authority Having Jurisdiction and is sufficient to pose a threat to the public if released[a] Buildings and other structures required to maintain the functionality of other Risk Category IV structures	IV

Table 2 Risk Category of Structures for Wind Loads (ASCE, 2017)

Because the collapse of a solar canopy would be a possible yet insubstantial risk to human life, the equipment is categorized as Risk Category 2 based on Table 2.

4.2.2 Step 2: Basic Wind Speed

Based on Figure 26.5-1C of ASCE 7-16, the conservative value for basic wind speed for Risk Category 2 is 98 mi/h (ASCE 2017).



Notes

1. Values are nominal design 3-s gust wind speeds in mi/h (m/s) at 33 ft (10 m) above ground for Exposure Category C.
2. Linear interpolation is permitted between contours. Point values are provided to aid with interpolation.
3. Islands, coastal areas, and land boundaries outside the last contour shall use the last wind speed contour.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00143, MRI = 700 years).
6. Location-specific basic wind speeds shall be permitted to be determined using www.atcouncil.org/windspeed.

Figure 1 Basic Wind Speed for California (ASCE, 2017)

4.2.3 Step 3: Wind Load Parameters

Step 3 involves the calculation of multiple variables: the wind directionality factor, exposure category, topographic factor, ground elevation factor, gust-effect factor, and combined factor (ASCE, 2017).

Wind Directionality Factor, K_d

Solar canopies are categorized as equipment; therefore, the directionality factor is equal to 0.85 (ASCE, 2017). See Section 26.66 and Table 26.6-1 of ASCE 7-16 for additional information.

Exposure Category B, C, or D

Section 26.7.3 of ASCE 7-16 details the category for Exposure B as follows:

> buildings or other structures with a mean roof height greater than 30 ft (9.1 m), Exposure B shall apply where Surface Roughness B prevails in the upwind direction for a distance greater than 2,600 ft (792 m) or 20 times the height of the building or structure, whichever is greater *(ASCE, 2017)*

In order to determine the upwind distance, the building height must be calculated. Based on the conservative estimate of a 15 ft floor-to-floor height and 9 levels (including top level), the height of a parking structure is estimated to be 120 feet.

$$max(20 * 90\, ft,\ 2600\, ft) = 2600\, ft$$

Parking garages are more likely to exist in urban and suburban areas due to the benefits of providing parking for a higher number of cars for smaller land area. Therefore, the land 2600 ft upwind of the parking garage is likely to be Exposure Category B. See Section 26.7 of ASCE 7-16 for additional information (ASCE, 2017).

Topographic Factor, K_{zt}

The wind speed-up effect is a phenomenon in which terrain features such as hills or ridges cause the wind speed to increase. It must be accounted for if all of the following conditions detailed in Section 26.8.1 of ASCE 7-16 are fulfilled:

1. The hill, ridge, or escarpment is isolated and unobstructed upwind by other similar topographic features of comparable height for 100 times the height of the topographic feature (100H) or 2 mi (3.22 km), whichever is less. This distance shall be measured horizontally from the point at which the height H of the hill, ridge, or escarpment is determined.
2. The hill, ridge, or escarpment protrudes above the height of upwind terrain features within a 2-mi (3.22-km) radius in any quadrant by a factor of 2 or more.
3. The building or other structure is located as shown in Fig. 26.8-1 in the upper one-half of a hill or ridge or near the crest of an escarpment.
4. H⁄Lh ≥ 0.2.
5. H is greater than or equal to 15 ft (4.5 m) for Exposure C and D and 60 ft (18 m) for Exposure B.

(ASCE, 2017).

The parking structure is assumed to be on a flat terrain; therefore, the conditions are not met, and the topographic factor K_{zt} is assumed to be unity. See Section 26.8 and Fig. 26.8-1 of ASCE 7-16 for additional information (ASCE, 2017).

Ground Elevation Factor, K_e

The ground elevation factor, K_e is based on the air density. To account for the various terrains throughout California, the conservative value of unity is used. see Section 26.9 and Table 26.9-1 of ASCE 7-16 for additional information (ASCE, 2017).

Gust-effect Factor, G; Except For Rooftop Equipment

This does not apply, as solar canopies are categorized as rooftop equipment. See Section 26.11 of ASCE 7-16 for additional information (ASCE, 2017).

Combined GC_r Factor For Rooftop Equipment

The combined factor GC_r is the product of the gust effect factor G and the external pressure coefficient C_r. For a conservative estimate, the combined factor for the horizontal wind force $GC_{r,h}$ is 1.9, and the combined factor for the vertical wind force $GC_{r,v}$ is 1.5. See Section 29.4.1 of ASCE 7-16 for additional information (ASCE, 2017).

4.2.4 Step 4: Determine Velocity Pressure Exposure Coefficient K_z

The mean roof height of a typical parking garage is estimated to be 75 ft tall. Therefore, for Exposure Category B, the velocity pressure coefficient at the height of the parking garage K_h is 0.91, according to Table 3 (ASCE, 2017).

Height above Ground Level, z		Exposure		
ft	m	B	C	D
0–15	0–4.6	0.57 (0.70)[a]	0.85	1.03
20	6.1	0.62 (0.70)[a]	0.90	1.08
25	7.6	0.66 (0.70)[a]	0.94	1.12
30	9.1	0.70	0.98	1.16
40	12.2	0.76	1.04	1.22
50	15.2	0.81	1.09	1.27
60	18.0	0.85	1.13	1.31
70	21.3	0.89	1.17	1.34
80	24.4	0.93	1.21	1.38
90	27.4	0.96	1.24	1.40
100	30.5	0.99	1.26	1.43
120	36.6	1.04	1.31	1.48
140	42.7	1.09	1.36	1.52
160	48.8	1.13	1.39	1.55
180	54.9	1.17	1.43	1.58
200	61.0	1.20	1.46	1.61
250	76.2	1.28	1.53	1.68
300	91.4	1.35	1.59	1.73
350	106.7	1.41	1.64	1.78
400	121.9	1.47	1.69	1.82
450	137.2	1.52	1.73	1.86
500	152.4	1.56	1.77	1.89

[a]Use 0.70 in Chapter 28, Exposure B, when $z < 30$ ft (9.1 m).

Notes

1. The velocity pressure exposure coefficient K_z may be determined from the following formula:
   For 15 ft (4.6 m) $\leq z \leq z_g$ $\quad K_z = 2.01(z/z_g)^{2/\alpha}$
   For $z < 15$ ft (4.6 m) $\quad K_z = 2.01(15/z_g)^{2/\alpha}$
2. α and z_g are tabulated in Table 26.11-1.
3. Linear interpolation for intermediate values of height z is acceptable.
4. Exposure categories are defined in Section 26.7.

Table 3 Velocity Pressure Exposure Coefficient (ASCE, 2017)

See Table 26.10-1 of ASCE 7-16 for more information.

4.2.6 Step 5: Determine Velocity Pressure q_z

The velocity pressure q_z is determined by the Equation 26.10-1 of ASCE 7-16 (ASCE, 2017):

$$q_z = 0.00256 K_z K_{zt} K_d K_e V^2$$

Based on Steps 1 through 4 of Sections 4.2.1 to 4.2.4, the velocity pressure exposure coefficient K_z is 0.91, the topographic factor K_{zt} is 1, the wind directionality factor K_d is 0.85, the ground elevation factor K_e is 1, and the basic wind speed V is 98 mi/h. Therefore, q_z is calculated to be $19.02\ psf$. See Section 26.10.2 of ASCE 7-16 for additional information (ASCE, 2017).

4.2.7 Step 6: Determine Force Coefficient, C_f, Except For Rooftop Equipment:

Step 6 is not applicable for solar canopies. See Steps 3 and 7 for the combined GC_r factors.

4.2.8 Step 7: Calculate Wind Force, F

The wind force consists of the horizontal and vertical force acting on the solar panels. The horizontal force is represented by (ASCE, 2017):

$$F_h = q_h (GC_{r,h}) * A_f \text{ (lb)}$$

The vertical force is similar (ASCE, 2017):

$$F_v = q_h (GC_{r,v}) * A_r \text{ (lb)}$$

In the equations above, F_h and F_v are the horizontal and vertical wind forces, respectively; q_h is velocity pressure evaluated at height h (see Section 4.2.5); $GC_{r,h}$ and $GC_{r,v}$ are the horizontal and vertical combined factors, respectively (see Section 4.2.5); and A_f and A_r

are the vertical and horizontal projected areas of the solar panel, respectively. See Equation 29.4-2 and Equation 29.4-3 of ASCE 7-16 for more information (ASCE, 2017).

Section 5: Structure Analysis of Gilman Parking Structure

In order to determine the load demand of a solar carport, a case study analysis is performed on a solar canopy on the top of Gilman Parking Structure. See Section 3.3 for more information regarding the Gilman Parking Structure.

5.1 Panel Geometry

Each solar canopy comprises an array of 7 by 11 solar panels. The solar panels each have an estimated length (l_{SP}) of $4.875\ ft$ and estimated width (w_{SP}) of $3.31\ ft$. The canopy has a square base with a height of 4.46 feet and side length of 22 inches. The height of both the column and base combined is 9.2 ft. The solar canopy is estimated to be tilted at a 15-degree angle (θ).

The anchor bolt diameter measures 1⅛ inches. Additionally, the distance between each anchor bolt is referred to as the bolt-to-bolt distance (d_{bb}), and it measures 13.5 in. Finally, the distance from the bolt head to the edge of the base is termed the edge-to-bolt distance. In this case, the edge-to-bolt distance is 4.8 in.

5.2 Load Analysis

Sections 5.2.1 and 5.2.2 will determine the wind load and dead load exerted on the Gilman Parking Structure, respectively. These are the two dominating load demands that must be determined in order to assess the feasibility of installing solar canopies. The snow, rain, and live load is assumed to be negligible. Section 5.2.3 will detail the load cases of the wind and dead loads to account for various situations the structures may face.

Figure 2 shows the various representations of the same wind load on the solar canopy. The left representation shows the area of the canopy being exposed to wind in order to achieve the maximum moment due to wind. The middle representation shows the combined force due to wind, and the right representation is the free body diagram for the wind load.

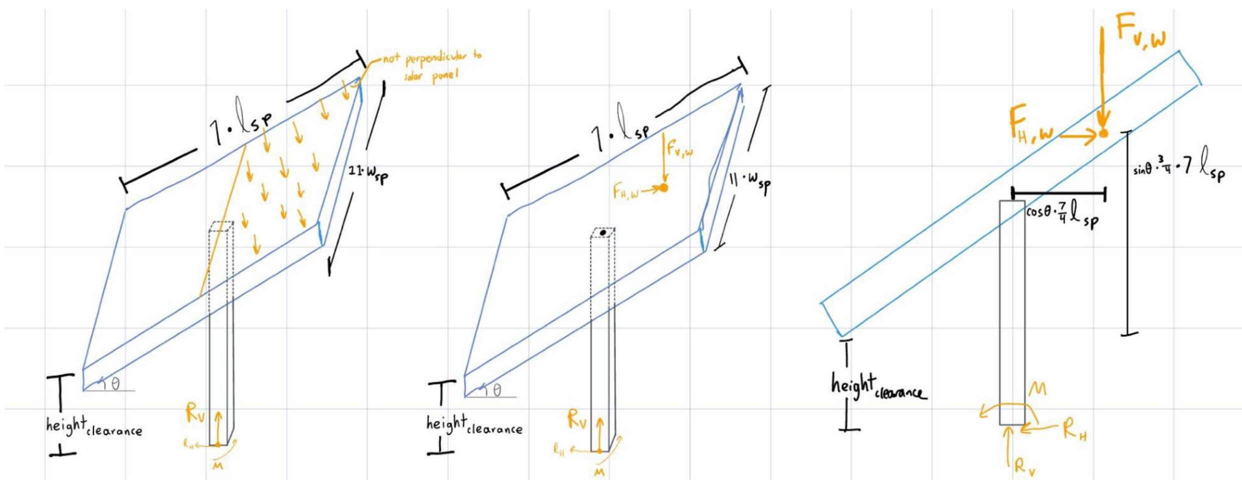


Figure 2 Wind Load on Solar Canopy

5.2.1 Wind Load

The wind load for the Gilman parking structure is determined using Section 4. To determine worst-case load, the wind is assumed to be applied to only one half of the solar panel, as shown in Figure 2. The wind-exposed section of the solar panel (A_{exp}) is calculated to be

621.2 sf. The force equations in Section 4.2.8 are used to derive new horizontal and vertical equations.

$$F_h = q_h (GC_{r,h}) \, sin(\theta) \, A_{exp} \text{ (lb)}$$

The variable q_h is 19.02 lb/ft^2 (see Section 4.2.6), $GC_{r,h}$ is 1.9 (see Section 4.2.4), θ is 15 degrees (see Section 5.1), and A_{exp} is 621.25 sf. For the vertical projected force, $GC_{r,v}$ is 1.5 (see Section 4.2.4):

$$F_v = q_h (GC_{r,v}) \, cos(\theta) \, A_{exp} \text{ (lb)}$$

These equations yield a horizontal force of 5.809 kips and a vertical force of $17.117 \; kips$.

The moment at the floor due to both horizontal and vertical wind loads is based on the equation below (see Figure 1):

$$M_W = F_v * cos(\theta) * \frac{7}{4} * l_{sp} + F_h * \frac{3*7}{4} sin(\theta) * l_{sp}$$

This yields a moment of 179.53 kips-ft.

5.2.2 Dead Load

The weight of each individual solar panel is assumed to be 58.51 lb (see Section 3). To account for the weight of the beams and other components, the weight is doubled to attain a conservative value of 117.02 lb. The number of solar panels is 77.

$$\sum F_v = weight_{sp} * number\ of\ solar\ panels$$

Based on the above calculation, the net vertical force is 9.01 kips. Therefore, the reaction forces at the floor are determined below, where M_D, $R_{V,D}$, and $R_{H,D}$ represents the moment due to dead load, vertical reaction due to dead load, and horizontal reaction due to dead load, respectively.

$$M_D = 0\, lb \quad \text{(basis of symmetry)}$$
$$R_{V,D} = 9.01\, kips\ \ upward$$
$$R_{H,D} = 0\, lb$$

5.2.3 Load Cases

The load cases must then be evaluated (see Section 4.1). The greatest load case is used to evaluate whether the concrete column strength is sufficient (see Section 5.3). The calculations for the five cases are shown below, where the D, W, S, R, and L represent the dead, wind, snow, snow, rain, and live load. Additionally, M, V, and H represent the moment, vertical force, and horizontal force, respectively, for the load case:

1. $1.4D$

   $M = 1.4 * M_D = 1.4 * 0 = 0\, lb * ft\, CW$

   $V = 1.4 * R_{V,D} = 1.4 * 9010.54 = 12.61\, kips\, upward$

   $H = 1.4 * R_{H,D} = 0\, lb\, rightward$

2. $1.2D + 1.6L + 0.5(L_r\ or\ S\ or\ R)$

   Case 2 has a lower load than Case 1.

3. $1.2D + 1.6(L_r\ or\ S\ or\ R) + (L\ or\ 0.5W)$

   Because dead load and wind load are in the same direction, Case 3 has a lower load than Case 4; see Case 4 for additional results.

4. $1.2D + 1.0W + L + 0.5(L_r\ or\ S\ or\ R)$

   $M = 1.2 * M_D + M_W = 1.2 * 0 + 179.53 = 179.53\, kips * ft\, CW$

   $V = 1.2 * R_{V,D} + R_{V,W} = 1.2 * 9.01 + 17.117 = 27.93\, kips\, upward$

   $H = 1.2 * R_{H,D} + R_{H,W} = 1.503\, kips\, rightward$

5. $0.9D + 1.0W$

Because dead load and wind load are in the same direction, Case 5 has a lower load than Case 4. See Case 4 for additional results.

Out of all 5 load cases, Case 4 had the greatest loads with respect to the horizontal force, vertical force, and moment.

5.3 Column Analysis

In order for the concrete column to have sufficient load capacity to hold the solar canopy, the supply load of the column must equal or exceed the maximum demand load. In this case, the maximum demand load is Case 4 (see Section 5.2.3) and must be compared to the design strength of the column.

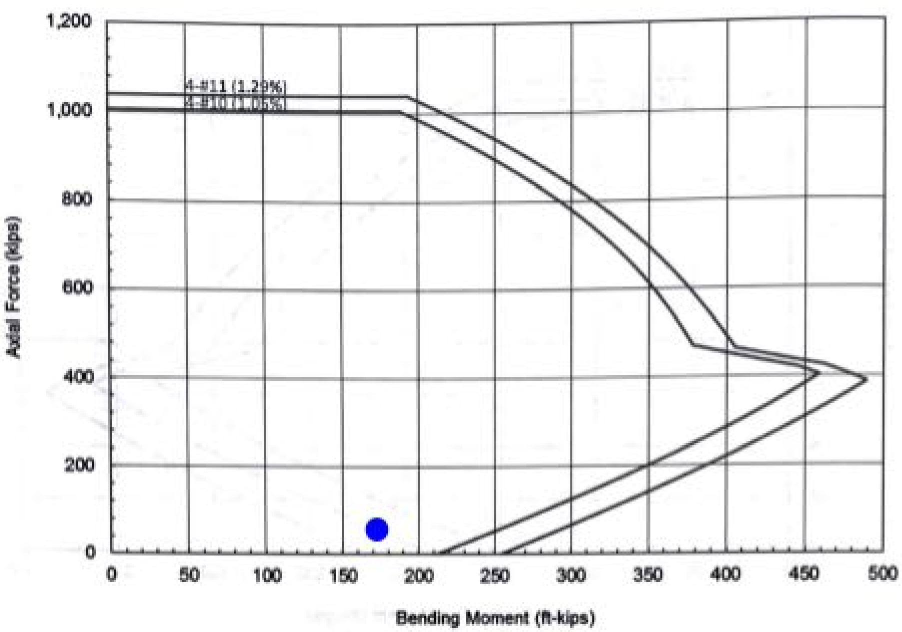


Figure 3 Design Strength Interaction Diagram (Figure E.11, ASCE 7-16)

Figure 3 shows the design strength for a 22x22-inch column with a 4 bar arrangement. The blue circle represents the axial force and bending moment demand of Case 4, and the two lines represent the load capacity for a column with 4 rebars and 11/8 inches or 10/8 inch diameter for the top line and bottom line, respectively. The region enclosed by the lines is recognized as the safe region, where loads can safely reach these values. According to the figure, the demand load is within the safe region; thus, the column has a sufficient load capacity.

5.4 Anchor Bolt Analysis

5.4.1 Anchor Bolt Strength

Two cases regarding the edge distance of the bolts must be considered. Edge distance is the distance from the anchor shaft to the edge of the concrete. The conditions for Case 1 are that the minimum edge distance must be 4 inches and that the minimum embedment depth must be 8.0 inches (Caltrans, 2021). The conditions for Case 2 are stated by The California Department of Transportation in the memo, "5.51 Anchorage to Concrete: Cast-In Anchors." The memo reads as follows:

1. For tensile capacity, the edge distance must equal or exceed 1.5 h_{ef} in both c_{a1} and c_{a2} directions, where h_{ef} is effective embedment depth of anchor, c_{a1} is the edge distance in one direction, and c_{a2} is the edge distance perpendicular to c_{a1}.
2. For shear strength, the minimum edge distance is 6.5 inches, and the minimum perpendicular edge distance is 9.8 inches.

(Caltrans, 2021)

The Gilman Parking Structure fulfills Case 1 with an edge distance of 4.8 and an embedment depth that is assumed to fulfill the requirement. Case 2 is not fulfilled.

Case 1 is fulfilled for both tensile capacity and shear strength. The bolt diameter is above 1 inch; therefore, a conservative estimate of a 1 inch diameter is used. Because the anchor bolts are arranged in a square formation, the design tensile strength is determined using the 2-edge condition.

Bolt Diameter (in)	Minimum Embedment Depth, h_{ef} (in)	Minimum Edge Distance, c_{a1} (in)	Design Tensile Strength, φN_n (kips)	
			1-Edge Condition ($c_{a2} \geq 1.5c_{a1}$)	2-Edge (Corner) Condition ($c_{a2}=c_{a1}$)
1/2	4.0	2.5	4.7 CB	3.3 CB
5/8	5.0	3.0	6.5 CB	4.5 CB
3/4	6.0	3.0	7.9 CB	5.3 CB
7/8	7.0	4.0	10.5 CB	7.3 CB
1	8.0	4.0	12.2 CB	8.1 CB

Note: CB: Concrete Breakout failure governs.

Figure 4 Design Tensile Strength (Caltrans, 2021)

Figure 4 shows the design tensile strength of a case 1, cast-in, hex head anchor bolt, which is 8.1 kips for a 1 inch diameter bolt (Caltrans, 2021). See Table 5.51.5.2.1 of Memo 5.51 by Caltrans for more information.

Bolt Diameter (in)	Minimum Embedment Depth, h_{ef} (in)	Minimum Edge Distance, c_{a1} (in)	Design Shear Strength, φV_n (kips)	
			1-Edge Condition	2-Edge (Corner) Condition (c_{a2}=c_{a1})
1/2	4.0	2.5	1.1 CB	0.9 CB
5/8	5.0	3.0	1.6 CB	1.4 CB
3/4	6.0	3.0	1.8 CB	1.5 CB
7/8	7.0	4.0	2.7 CB	2.3 CB
1	8.0	4.0	2.7 CB	2.3 CB

Note: CB: Concrete Breakout failure governs.

Figure 5 Design Shear Strength (Caltrans, 2021)

Figure 5 shows the design shear strength of a case 1, cast-in, hex head anchor bolt, which is 2.3 kips for a 1 inch diameter bolt (Caltrans, 2021). See Table 5.51.5.2.3 of Memo 5.51 by Caltrans for more information.

5.4.2 Tensile Force Due to Wind Load

The tensile stress due to the moment from the wind load can be determined using beam theory. The area of the bolt (A_b) can be calculated below:

$$A_b = \frac{\pi d_b^2}{4}$$

In this equation, d_b is the bolt diameter, which is taken to be 1 inch. Given that the bolt-to-column center distance (s) is 2.4 inches, and the number of bolts (n) is 4, the moment of inertia is calculated using the equation:

$$I = A_b (s)^2 n$$

This yields a moment of inertia of 18.095 in^4. The bending stress (f) is then calculated using the equation below, where the moment at the base plate is 125.2 kips-inch.

$$f = M / I_x$$

The bending stress is determined to be 6.9 ksi. Therefore, the tension force due to the moment from the wind load ($L_{W,M}$) is calculated using the equation:

$$L_{W,M} = f A_b$$

The tension is determined to be 5.42 kips upward. It is assumed for the anchor bolt calculations in Section 5.4 that the vertical force from the wind load ($F_{v,W,AB}$) is $17.117\ kips$ upward. See Section 5.2.1 for more information regarding the magnitude of the vertical wind force. The upward direction is chosen to account for the maximum tensile load. Therefore, based on the equation below, the vertical force due to wind on each anchor bolt ($L_{W,V}$) is determined to be 4.28 kips upward.

$$L_{W,V} = \frac{F_{v,W,AB}}{n}$$

5.4.3 Tensile Force Due to Dead Load

Because the vertical force due to the dead load is 9.01 kips, the compressive force is distributed across four anchor bolts and is 2.25 kips on each bolt.

5.4.4 Total Tensile Force

To determine the total tensile load on the anchor bolts, the wind and dead loads must be superimposed.

$$Total\ tensile\ load\ =\ L_{W,M} +\ L_{W,V}\ +\ L_D$$

In the above equation, $L_{W,M}$ represents the wind load due to moment of 5.42 kips upward, $L_{W,V}$ represents the wind load due to vertical force of 4.28 kips upward, and L_D represents the dead load of 2.25 kips downward. The total tensile force is calculated to be 7.45 kips.

5.4.5 Total Shear Stress

The shear load on the anchor bolt is exerted by the horizontal force due to wind. The dead load does not exert a horizontal force, which means that it does not cause a shear force in ideal circumstances. The horizontal force due to wind is 5.809 kips. Assuming that the force is evenly distributed across the four anchor bolts, each bolt is acted upon by a 1.45 kips shear force. In the equation below, F_{shear} is the shear force acting on each bolt, A is the cross sectional area of the bolt, and τ is the shear stress on each bolt.

$$\tau\ =\ \frac{F_{shear}}{A}$$

Based on the equation above, the shear stress on each bolt is 1.85 ksi. Therefore, the shear stress is below the corner condition design shear strength of 2.3 kips, which leads to the conclusion that the anchor bolts are able to withstand the shear stress.

Section 6: Summary

Based on the wind and dead loads exerted on the solar canopy of the Gilman Parking Structure, the concrete column was determined to be strong enough to withstand load demand. Additionally, the tensile strength of 8.1 kips exceeds anchor bolt tensile load of 7.45 kips. Thus, both the column and anchor bolts have the strength to support the solar canopy, which is confirmed by the fact that the Gilman Parking Structure is still in operation today.

The method used to analyze the Gilman Parking Structure is also useful for determining the feasibility of adding solar canopies to other parking garages. While the case study analysis was performed on a solar canopy already installed, the calculations based on the ASCE 7-16 and Caltrans Memo 5.51 can also be used to predict whether a future installation on a parking garage without solar panels would be feasible.

Though each parking structure has varying strengths and conditions, many factors can be modified to increase feasibility. For instance, a solar canopy of smaller area will decrease the demand load. Additionally, the column can be strengthened by increasing the diameter and number of rebars. The number of anchor bolts can be increased to decrease the individual load exerted on each bolt. For instance, while the Gilman Parking Structure had 4 anchor bolts, the Westfield UTC Parking Structure had 12 anchor bolts. Finally, other factors such as the concrete strength, bolt diameter, and number of columns can be modified to enable the feasibility of solar canopies. Thus, many variables can be calculated to make the installation of solar panels more feasible.

Section 7: References

American Society of Civil Engineers (ASCE). (2017). *Minimum design loads and associated criteria for buildings and other structures* (ASCE/SEI 7-16). Reston, VA: ASCE.

The California Department of Transportation (Caltrans). (2021). *Anchorage To Concrete: Cast-In Anchors* (Memo 5.51). Caltrans, Sacramento, CA.