Structural Considerations for PV Installations on Older Row Houses

While Washington, DC, has a vibrant PV market, streamlining the design and permitting process has been a persistent challenge. As a result, many installation companies migrate toward standard designs in terms of system topology and mounting method. This strategy helps companies control costs, minimize unforeseen challenges and achieve predictable engineering and permitting results. It is challenging, however, for companies to develop standardized design approaches suitable for the more than 70,000 row houses that comprise more than 25% of the housing stock in Washington, DC. These row houses are typically two to three stories above grade and 14–20 feet wide. Similar housing stock is found in many other US cities.

As a structural plan reviewer for solar projects, I have evaluated hundreds of engineering plans for retrofitting residential PV systems. Here I present some key considerations for installations on older row houses. This is intended as a high-level overview; some issues are entirely unaddressed and others could stand greater scrutiny. However, PV installation contractors, structural engineers and plan reviewers can apply some of these lessons learned to the older structures prevalent where they work.

Unique Design Challenges

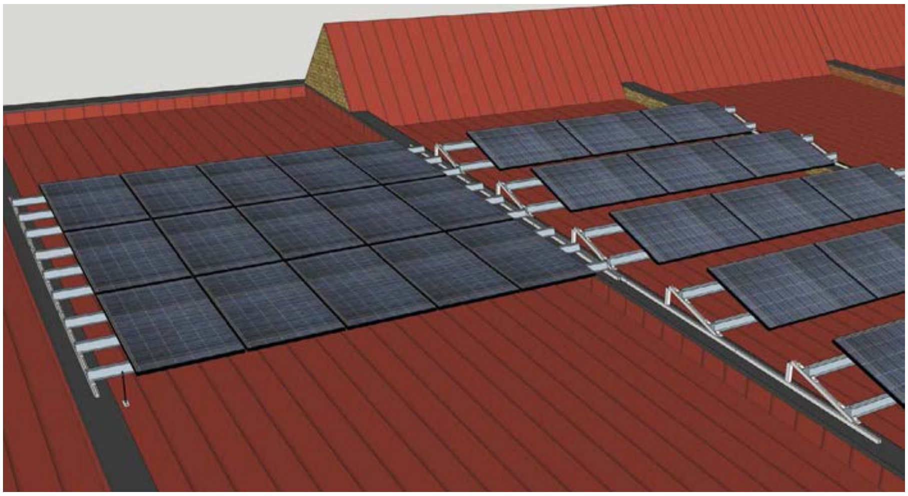
Standard approaches to retrofitting PV systems on residential structures are not well suited to the peculiarities of older row homes. Largely built in the period between 1900 and 1940, row homes in Washington, DC, present a unique set of design challenges. The roof framing is rarely up to code, which raises concerns about connecting rail-mounted PV systems to the rafters. Further, the nearly flat roof slope, typically around 5°, is not ideal for making dozens of roof penetrations. However, the light framing typically also rules out the use of a ballasted mounting system. The alternative mounting solution that many designers commonly propose is to suspend the PV system above the roof by spanning between parapets.

Party walls and parapets. In Washington, DC, row-house roofs are separated by 12-inch-wide party walls that extend above the roof about 6–8 inches as parapets. These are multiwythe brick walls—meaning that continuous vertical sections of brick are laid next to one another to increase the wall thickness. Roofing material generally extends up and over the parapet. While torch-down modified bitumen is a common modern roofing material, many of these structures originally had standing-seam metal roofs with coping (a metal cap flashing) atop the parapet; painted and patched variations of this original roofing are still found in the field. Often the parapet has wood cap board, of uncertain and variable age, on top of the brick and under the roofing or coping. The condition of these components and the wall in general can vary considerably, due in part both to the effects of aging and to a wide variation in the original materials and build quality.

Parapet-to-Parapet Mounting

Because brick walls have very high compressive strength—1,000 pounds per square inch is a reasonable minimum value—parapets in older row houses are attractive to solar contractors and structural engineers as a means of supporting the PV array. The general idea behind this approach is that the array can be mounted on a system of beams that span from parapet to parapet, as shown in Figure 1. As long as the bearing surface of each beam end is at least several square

Figure 1 Examples of parapet-to-parapet mounting schemes are shown here. Typically, anchors embedded in the top of the 12-inch-wide parapet walls provide uplift resistance for the roof-mounted PV array.



inches, compressive loading is unlikely to limit the structural design of the mounting system. The viability of parapet-to-parapet mounting, therefore, depends on whether the design addresses uplift loads in addition to the compressive loads due to dead weight and snow.

Resistance to uplift. I have reviewed many row house projects where the engineer had not properly accounted for wind-related uplift loads. The most commonly proposed method of handling uplift in parapet-to-parapet mounting is to use some arrangement of bolts or threaded rods installed in the top of the parapets. Proposed embedment depths are typically in the 4- to 18-inch range, although some are specified as deep as 50 inches. In nearly every case, the bolts or threaded rods are to be embedded in injectable mortar or hydraulic masonry cement, such as Rockite anchoring cement or Hilti HIT-HY 70. The problem with these designs is that they do not take into account the fact that row homes have unreinforced masonry walls. Section 2.2.4 of the *Building Code Requirements and Specifications for Masonry Structures,* published by the American Society of Civil Engineers (ASCE) as Standard 5-11, effectively states that anchors embedded in the top of a multi-wythe brick wall cannot be considered to provide resistance to uplift: "The tensile strength of [unreinforced] masonry shall be neglected in design when the masonry is subject to axial tension forces."

Upon closer inspection, there are other problems with this proposed approach. First, load tables published for adhesive anchor systems do not apply to the specifics of the application, as shown in Table 1. In addition, other issues in practice may lead to an unreasonably wide range in anchor tensile strength or to unpredictable and nonlinear behavior. For example, it is typically not possible to assess the quality of the wall materials or construction methods, given that roofing materials often cover the parapets. Further, it is difficult to identify and account for all of the layers of building materials—including roofing, cap board and coping—that anchors may penetrate before reaching the brick layers. Finally, drilling holes in a brick wall often creates a certain degree of damage—such as cracked bricks and broken mortar bonds—which is compounded where holes are drilled in close proximity to one another. In a worst-case scenario, this damage could undermine the compressive strength of the brick beyond acceptable levels.

In spite of these issues, embedded anchors may still provide some benefit. For example, these connections counteract lateral sliding forces, which are generally smaller than uplift forces. Since some strength is observed even in tension, it may be reasonable for an engineer to assign modest tensile strength, on average, to a set of anchors—provided these are embedded at a sufficient depth to engage several courses of bricks, such as 10–12 inches of brick embedment *after* penetrating the roofing and cap board. While testing would be helpful in this regard, such testing would likely require many samples in a wide variety of walls, as well as a large safety factor in practice.

Even in the absence of such data, commentary in ASCE 5-11 associated with Section 2.2.4 seems to allow an approach that assigns modest tensile strength to anchors embedded in the top of a multi-wythe wall based simply on the mass of any bricks directly engaged by the anchors: "Net axial tension in unreinforced walls due to axially applied loads are not permitted. If axial tension develops in the walls due to uplift of connected roofs or floors, the walls must be reinforced to resist the tension. *Compressive stress from dead load can be used to offset axial tension*" [emphasis added].

Quantifying the Problem

The loads transferred from a PV system to the underlying building are composed of several elements: the dead load of the system itself (the weight of the various materials and equipment), snow loads, wind loads and the resultant combinations of

Allowable Adhesive Bond Loads for Multi-Wythe Solid Brick Walls

Nominal anchor diameter (inches)	Effective embedment (inches)	Tension (pounds)	Shear (pounds)	Minimum spacing (inches)	Minimum edge distance (inches)
3/8	6	895	680	16	16
	10	1,325	795		
½	6	895	1,075		
	10	1,455	1,115		
5/8	6	1,025	1,405		
	10	1,955	1,445		
¾	8	1,575	1,985		
	13	2,135	1,985		

Data courtesy Hilti

Table 1 Engineering data published by Hilti show very good tensile behavior for threaded rod embedded in multi-wythe brick wall using its HIT-HY 70 injectable mortar. However, these data assume that you have embedded the anchors in the side face of the wall, not in the top of the wall. Further, they assume that you have installed the anchors at least 16 inches from other anchors or the wall edge. The latter is obviously not possible when you are anchoring in the top of a 12-inch-wide wall.

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these loads. While a thorough analysis of these loads is beyond the scope of this article—and is an ongoing topic in the structural engineering community—I detail a schematic example here to help make this discussion more concrete. For more in-depth analyses of wind loads on PV systems, I recommend reading “Wind Loads on Low Profile Solar Photovoltaic Systems on Flat Roofs” by the Structural Engineers Association of California (seaoc.org), “Wind Load Calculations for PV Arrays” by the Solar America Board for Codes and Standards (solarabcs.com) and “Determining Wind and Snow Loads for Solar Panels” by SolarWorld (solarworld-usa.com).

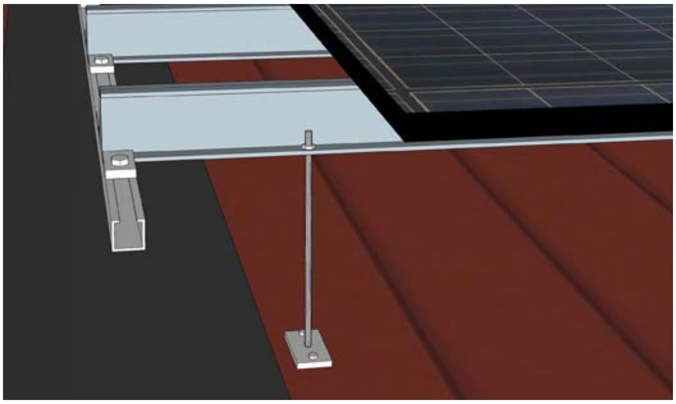


Figure 2 A rafter tie-down, like the one shown here, can provide considerable uplift resistance. To minimize roof penetrations, consider using tie-downs in combination with masonry anchors.

Design principles. According to Section 1509.7.1 of the *International Building Code (IBC)*, 2012 edition, engineers should address uplift calculations for roof-mounted PV systems using the components and cladding (C&C) method described in *IBC* Chapter 16, which is based on ASCE 7-10, “Minimum Design Loads for Buildings and Other Structures.” While the *International Residential Code (IRC)* does not specifically address PV systems, a similar or compatible approach is a reasonable starting point. As illustrated in *IRC* Table R301.2(2), the *IRC* includes the C&C approach for buildings and covers the same applications as the *IBC.* Typically, the C&C loads applied to PV systems are in the range of 15–40 pounds per square foot (psf) of uplift.

While engineers have used ASCE 7-05, published in 2005, as the basis of structural designs for many years, ASCE 7-10, which was published in 2010, is gradually replacing it. The authors of the SolarWorld article referred to earlier note that ASCE 7-05 largely follows the design principles of allowable stress design (ASD), whereas ASCE 7-10 represents a shift in design principles toward load-resistance factor design (LRFD).

According to *A Beginner's Guide to Structural Engineering,* by Bartlett Quimby (bgstructuralengineering.com), one argument in favor of LRFD is that it yields an equivalent factor of safety that is "more consistent with the probabilities of design" compared to ASD. In other words, where a building is subjected to highly predictable loads, LRFD generally results in a lighter structure compared to ASD. However, where a building is subject to highly unpredictable loads—such as live, seismic and wind loads—LRFD generally results in a stronger structure compared to ASD. Sample calculations in SolarWorld's article seem to reinforce this conclusion. Assuming the same system design details, uplift load values are lower when calculated according to ASD per ASCE 7-05 (15–33 psf) compared to uplift load values calculated using LRFD per ASCE 7-10 (26–64 psf).

Load Calculations

The following example is illustrative only, as many variables depend on the site or design, such as array height off the roof. Row houses have features that can result in reduced wind loading of PV systems. For example, if you are considering wind loads on the roof of a mid-row unit, typically only Zone 1 (interior field of roof) and Zone 2 (edge of roof) loading applies, as the length of the row of houses effectively eliminates higher Zone 3 (corner of roof) wind loads. Trees and other obstructions may also disrupt high local forces.

In this case, I assume a wind uplift load of –25 psf. Per ASCE 7, after determining wind uplift force, you incorporate this value into a load combination analysis to identify the dominant combination of forces. The dominant uplift load combination is most likely based on dead load (*D*) and wind load (*W*) as shown in the following equation:

$$\text{Total uplift load} = 0.6D + 0.6W$$

If D = 5 psf and W = –25 psf, then the total uplift load = –12 psf (0.6 x –20 psf). As a comparison, the C&C uplift loads in *IRC* Table R301.2(2) are –13.3 psf for Zone 1 and –15.8 psf for Zone 2, assuming an effective wind

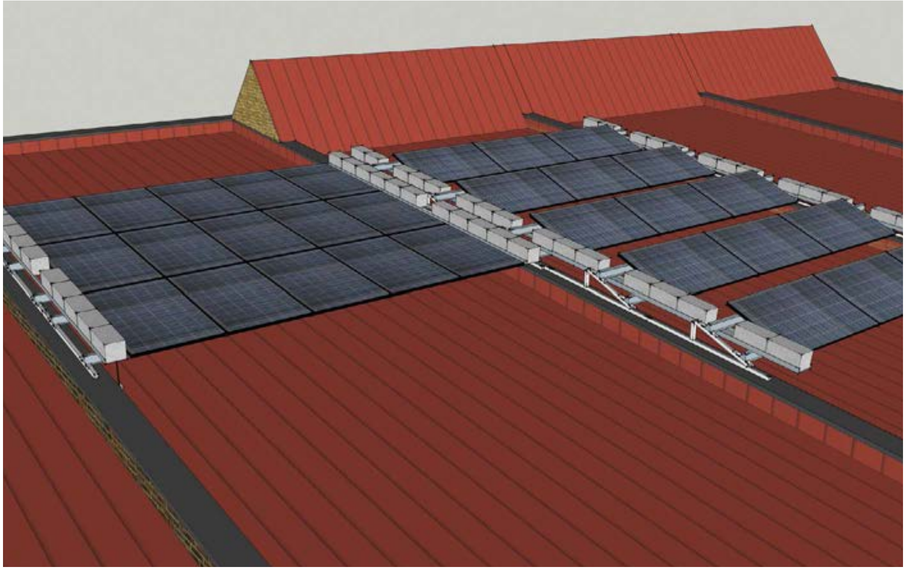


Figure 3 In this example, the mounting systems from Figure 1 (p. 20) are modified so that you accomplish some anchoring using ballast materials.

area of 100 square feet and a basic wind speed of 90 miles per hour per 3-second gust.

Assuming that the PV modules are 3 by 5 feet (15 square feet), then the dominant uplift load combination on a row of five modules equals –900 pounds (75 ft^2 x –12 psf). If the total array consists of three five-module rows, then the total uplift amounts to 2,700 pounds. However, since there are anchors on either side of the array, the uplift loads are 1,350 pounds per side of the array.

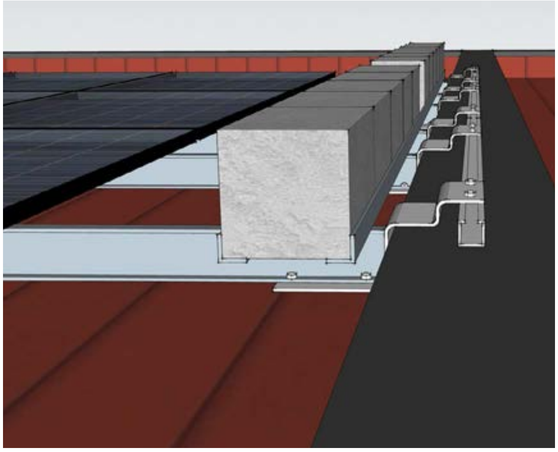
Anchoring the system. Once you calculate uplift loads, you must ensure that the array is sufficiently anchored to the building structure to resist these loads. You can accomplish this in several ways, such as using masonry anchors in the parapets, tie-downs (lag bolts) into the rafters, ballast or a combination of anchor types. Each of these approaches has advantages and limitations.

With the use of masonry anchors, the mass of the brick wall is the limiting design constraint. Therefore, you want to tie a rigid PV structure to the parapets repeatedly, deeply and at reasonable intervals. To engage several layers of brick, you likely need to embed threaded rods at a depth of 10–12 inches. (The choice of anchoring cement is likely not critical.) To reflect best engineering practices—for example, ASCE 5 Section 1.17.3 assumes a 45° projected shear area per anchor bolt—place these anchors at an interval distance of about twice their depth. In this case, that amounts to a spacing of 20–24 inches between anchors.

Per the commentary on ASCE 5-11 Section 2.2.4, it is reasonable to ascribe a net uplift/tensile resistance of 50–80 pounds per anchor based on the compressive weight of the brick above the anchor. Assuming 80 pounds of uplift resistance per anchor, installing ten anchors per side of the array would result in a net uplift resistance of 800 pounds per side. While this offsets a substantial fraction of the calculated uplift load (1,350 pounds per side), it does not entirely counter the uplift. A thoroughly engineered approach will require additional uplift resistance.

While row-house roof rafters themselves are generally not adequate to handle the downward loads associated with a PV array, they provide considerable tensile/uplift strength. For example, a single 5⁄16-inch lag bolt embedded 2 inches into a rafter, as shown in Figure 2 (p. 22), can provide 400–500 pounds of tensile strength. In my example, three or four rafter tie-downs per array side could provide all the necessary uplift resistance. The downside to this approach is that you have to penetrate the roof. To minimize roof penetrations, you could use rafter tie-downs in combination with masonry anchors; in this scenario, you might need only one lag bolt at each corner of the PV array.

Ballast is likely the most reliably characterized design approach. For example, solid 8-by-8-by-16-inch concrete masonry units (CMUs) weigh approximately 75 pounds each, which means that 18 or 19 CMUs per side could fully counteract the calculated uplift. If you use the ballast in combination with masonry anchors, you only need 7 or 8 CMUs per side. (The use of some masonry anchors is advisable to counteract sliding forces.) Some challenges are also associated with using ballast in this application. Doing so likely requires some modification to the



Visibility Where street-level visibility is an issue in historic districts, it may be possible to recess the array and the ballast material within the parapets, as shown here.

parapet-to-parapet mounting system, as shown in Figure 3. The ballast materials may also introduce shading that requires mitigation.

In both Figure 1 (p. 20) and Figure 3, I show two common module mounting approaches. The mounting methods on the left use a monolithic flat array; the mounting methods on the right use a tilted sawtooth design. While you could also tilt up the entire 15-module array as a single flat plane, the wind uplift forces on this "sail" would be considerably larger than those discussed here.

Additional considerations for minimizing the chance of failure. First, because wind loads are uneven, increasing the effective wind area decreases the associated wind loads on a pounds-per-square-foot basis. A larger array provides more area for countervailing forces—where wind uplift in one area is countered by wind downloads in another area—to reduce maximum loads. Note that this applies only if all of the rows are stiffly connected to one another, so that the PV array functions as a single structural element.

Second, a structurally stiff array allows counter-uplift elements—such as ballast, lag bolts or masonry anchors—to load-share, meaning that more anchors can counteract a given uplift point load. This is somewhat analogous to the effective wind area issue, except that it involves spreading net uplift across multiple anchors as opposed to spreading variable wind loads across a greater surface area. Structural rigidity is the key to enabling load sharing.

Third, a PV array is not a continuous membrane. There are gaps between rows and columns through which air pressure behind the modules will equalize with the pressure in front of the modules. To the degree that this equalization can happen quickly, this permeable quality can reduce net loading. Permeability, in this case, is a function of both the size of the gaps in the array and the size of the volume of air behind it. For example, larger gaps will facilitate faster equalization. Similarly, a smaller volume of air will equalize more quickly than a larger one. In other words, an array mounted closer to the roof will generally experience lesser loads than one that is mounted higher off the roof.

The views expressed here are the author's alone, and do not represent the policy of the DC government or agents thereof. The images included are schematic and illustrative in nature and may not reflect all engineering considerations.
—Keith Winston / Green Building Program / Department of Consumer and Regulatory Affairs / Washington, DC / dcra.dc.gov