Seismic Considerations and Evaluation Approach for "Isolated" Rooftop PV Arrays

Mason Walters, SE, Senior Principal
Russell Berkowitz, SE, Senior Associate
Dennis Lau, PE, Engineer
Won Lee, PhD, PE
Forell/Elsesser Engineers, Inc.
San Francisco, CA

Jack Baker, PhD
Stanford University, College of Engineering
Palo Alto, CA

Abstract

Recently, some photovoltaic (PV) equipment manufacturers have developed and implemented non-anchored or "isolated" PV array support on relatively flat rooftops on large commercial and institutional buildings. This technique saves significant time and expense over conventional PV array installation methods, and has the potential to decrease the risk of roof membrane failure. However, concerns regarding possible seismically-induced horizontal movement and wind uplift of PV arrays surround the introduction of this new technique, which currently is required to be considered as an "alternative means of compliance" for rooftop PV array implementation. The isolated approach explicitly relies upon friction between a PV array and its supporting roof membrane, which in principle is similar to the use of friction in a seismic isolation system.

This paper describes the key seismic considerations related to this innovative method of PV installation on flat or near-flat building rooftops, and presents a rational approach for the evaluation of PV array seismic sliding displacements and determination of corresponding gaps for seismic movement.

Introduction & Background

According to the U.S. Energy Information Administration, the fastest growing component of the US renewable energy sector in 2010 was solar PV arrays. Total shipments of PV modules in 2010 more than doubled compared to total module shipments in 2009, corresponding to a rise in capacity from nearly 1.2 peak gigawatts to more than 2.6 peak gigawatts (Figure 1). This surge in growth was supported in part by a rapid decline in the price of PV cells and modules and by government incentives and policies at the federal, state, and local levels. Solar PV energy has been established as a small but important component of the renewable energy supply in the U.S. Over half of the recent growth in PV energy capacity has taken place in the commercial sector, where many PV arrays are located on large, relatively flat building rooftops.

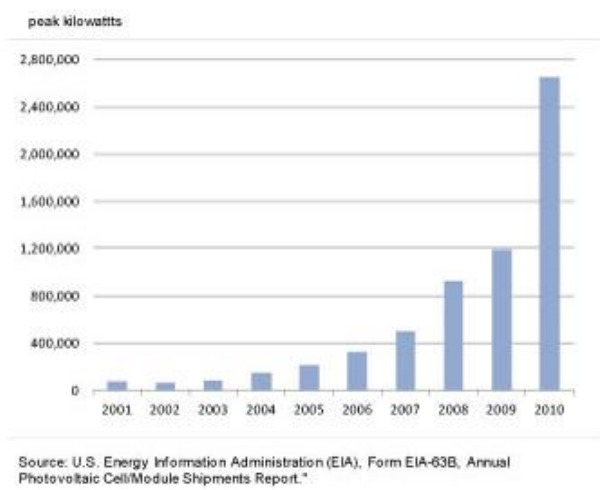

Figure 1: Annual Photovoltaic Shipments, 2001 - 2010

The International Building Code (IBC) and California Building Code (CBC) currently do not explicitly address the seismic requirements for rooftop PV arrays. The conventional method of supporting PV arrays on rooftops is to anchor them to the roof structure or to adhere them to the roof membrane itself to prevent hazards arising from wind and earthquake loads. Design forces for this purpose are derived from Chapter 13 of ASCE 7.

There are significant drawbacks to conventional PV array anchorage to rooftops, including the time and labor costs for PV installers and roofing contractors to deploy solar arrays, the possible need for strengthening of roof structures for increased vertical and lateral loads caused by the arrays, the cost of anchorage hardware, and the potential for future leakage and consequential repairs at numerous penetrations required to install rooftop fasteners. Furthermore, additional seismic inertial mass is introduced at the uppermost level of a structure due to the self-weight of the arrays.

In 2008, the State of California Division of the State Architect first issued Interpretation of Regulations (IR) 16-8, which allowed the use of "ballasted only" (non-anchored) PV arrays for resisting wind forces, but maintained the requirement for positive attachment to resist seismic forces for non-structural components prescribed by Chapter 13 of ASCE 7. These basic requirements have not changed with subsequent revisions.

Recently, several PV equipment manufacturers have developed and, to a limited extent, implemented non-anchored or "isolated" PV array support on relatively flat rooftops of large commercial and industrial buildings such as "big box" and department stores, warehouses, industrial and office buildings, and gymnasiums. This installation technique saves significant time and expense over conventional PV array installation methods, and has the potential to decrease the risk of roof membrane failure. However, concerns regarding possible earthquake and wind induced horizontal movement and wind uplift of PV arrays complicate the introduction of this new technique, which currently must be regarded as an "alternative means of compliance" for rooftop PV array implementation. In addition, Section 13.4 of ASCE 7 explicitly requires that friction shall not be relied upon for seismic lateral resistance. The isolated approach relies upon friction between a PV array and its supporting roof membrane, which in principle is similar to the use of a friction in a seismic isolation system, as addressed in Chapter 17 of ASCE 7. Refer to the photos in Figure 2 and Figure 3 for a comparison of anchored vs. isolated arrays.

Figure 2: Anchored PV array on rooftop

Figure 3: Isolated PV array on rooftop

This paper describes the key seismic considerations related to this innovative method of PV array installation on flat or near-flat building rooftops, and presents a rational approach for the evaluation of earthquake-induced PV array sliding displacements and determination of corresponding clearance requirements for seismic movement. Development of the approach described herein began in 2003 as a generic means for a single manufacturer to obtain installation approval from Authorities Having Jurisdiction (AHJ) for PV applications on rooftops of large commercial buildings with relatively short fundamental periods at potential sites including those areas with "worst-case" expected ground motion intensity in California. This approach has since evolved to include multiple levels of anticipated ground motion intensity in seven Western U.S. states: Arizona, California, Hawaii, Nevada, Oregon, Utah, and Washington. Refer to Figure 4 for a map indicating the western states considered in the current study. Forell/Elsesser has completed or is in the process of completing similar studies for other PV manufacturers.

Figure 4: Western States Included in Rooftop Isolated PV Array Study

Seismic Performance Goals for Isolated PV Arrays

The seismic performance goal for PV arrays on a given building depends on building function and desired PV array status following an earthquake. For most buildings, the standard building code goal of life safety would be applicable. This goal translates to the following specific objectives with respect to PV arrays:

- Prevention of falling hazards: PV arrays must be prevented from falling off the edges of the supporting roof, including perimeter edges, skylights, hatchways, and any other openings in roof surfaces.
- Prevention of collision with fixed rooftop equipment units, ductwork, significant electrical conduits, and other PV arrays: Although hazard arising from collision is not as apparent as falling hazards, significant damage to PV arrays or equipment units under power could result in a fire hazard.
- Prevention of breaks in PV electrical continuity: An interruption of electrical continuity caused by a broken conductor or a disintegrating array could give rise to an electric arc or short circuit, either of which could result in a fire.
- Prevention of emergency personnel access: In the event of a fire following an earthquake, emergency rooftop access could be required. Access paths between adjacent arrays, and between arrays and parapets or other physical constraints, are normally required by building officials for PV array installations for this purpose. Residual seismic translation of PV arrays could effectively block access if adequate pathways around and through arrays do not remain after major earthquakes.

The prevention of falling hazards, equipment collision, and access for emergency personnel can be provided by providing sufficient seismic "gaps" between arrays and roof edges, equipment, and parapets. Preventing breaks in electrical continuity requires the addition of sufficient "slack" length for conduit and conductors, together with the use of flexible conduit between arrays and fixed junction boxes, electrical panels, or inverters, and between arrays themselves.

Parapets may be able to provide an obstacle to sliding ballasted arrays if they have sufficient strength to resist the consequent impact force without failure: However, a realistic evaluation of a parapet's capability may not be possible, due to a lack of knowledge of the array speed at impact and the resulting forces.

Higher seismic performance goals are possible if continuous function or damage limitation is required. However, most seismic arrays are directly connected to the general electrical grid, rather than to the building electrical service or an "off grid" battery storage application. Consequently, most rooftop arrays may not be useful in any case following a general grid failure, and should not be regarded as a source of emergency power.

In effect, for a normal life safety objective the most appropriate design approach is the provision of an adequate gap, based on a reasonably conservative estimate of computed sliding displacement that considers appropriate ground motions.

Rooftop Conditions and Assessment of Applicable Friction

The friction between a PV array and the supporting roof membrane is one key determinant of seismic movement of PV arrays. Both the static ("breakaway") and dynamic (or sliding) friction values will affect the seismic displacement response. Numerous conditions have the potential to affect friction values, including:

- Roof membrane material
- Water (rain, condensation, etc.)
- Snow, frost, or ice
- Dirt or other debris
- Degradation of roof membrane

The friction coefficient that is used to predict earthquake-induced sliding displacement cannot be obtained from standard friction tests in which samples (or coupons) of two types of material are pulled against each other under the application of normal force. Rather, the determination of the "effective" friction between an array and roof membrane requires the implementation of a testing configuration representative of the actual sliding response of an array across a roof membrane. Such representative testing is required because the friction value is affected by the dynamic characteristics of the array itself as it slides across a roof membrane. This is because a flexible array can respond in a "walking" (or shimmying) mode as it slides, which could yield a significantly different (and likely lower) "effective" friction than simple friction tests using two specimens of material. Consequently, the realistic evaluation of an applicable friction coefficient requires testing of a segment of a full-scale array, including actual support framework of the array itself. Refer to Figure 5 for illustrations of example full-scale array pull-testing to evaluate "effective" friction coefficient.

Figure 5: Example of full scale array pull testing.

Typically, such full-scale testing is done for multiple potential membrane types on a zero-slope surface, which allows the identification of purely frictional resistance and is unrelated to membrane slope. Tests on each membrane are conducted in both primary orthogonal directions of a solar panel or array ("N-S" and "E-W" directions), since dynamic "walking" characteristics of arrays commonly vary in the respective directions.

Roof slope is also critically important in determining potential seismic movements of an array. Numerical studies conducted by the authors have shown that the net seismic movement of a PV array, not surprisingly, is almost always in the down-slope direction. Appropriate rooftop slopes for isolated array deployment vary from a minimum of ¼:12 (1.2 degrees or 2%) to a maximum slope of about 1.5:12 (7 degrees or 12.5%); slopes exceeding this range generally result in very large down-slope sliding displacements, and are therefore recommended to be anchored instead of "isolated".

Other conditions of rooftop support of PV arrays require consideration in determining realistic predictions of seismic movement. These include:

- Building dynamic behavior: Most buildings with large-rooftop areas are relatively stiff, low-rise buildings with correspondingly low fundamental periods of vibration. Depending on the lateral strength of the structure (including both vertical and horizontal elements of the lateral system), the effective dynamic period of the building may vary with lateral system yielding.
- Orientation of roof slope with respect to the array N-S and E-W directions: For efficiency reasons, most PV arrays are oriented to face in the cardinal north-south direction. The array N-S and E-W directions may have differing effective friction coefficients. It is conservative to assume that the lower of the two effective friction coefficients is oriented in the downhill direction.
- Direction of applied ground motion with respect to roof slope orientation: Ground motion records are typically stronger in one component direction than in the other. Since directionality is seldom certain, it is conservative to orient the stronger component in the direction with the downhill roof slope.
- Roof diaphragm vertical (out-of-plane) and/or (in-plane) horizontal flexibility: The horizontal flexibility of the roof diaphragm, taken in series with that of the vertical elements of the lateral system, tends to lengthen the global period of the structure and the input to a rooftop array. Similarly the vertical flexibility of a roof diaphragm supporting a ballasted array would affect, to some extent, the vertical excitation of the array, and would thus influence the horizontal motion as explained previously.
- Location of the array on the roof diaphragm: The array behavior would be affected to some extent by its position on a rooftop: For instance, an array located near the mid-span of the roof diaphragm would experience somewhat different rooftop input than an array located near the top of a shear wall or bracing line. The location of an array on a rooftop would also affect the vertical excitation of the array, and would thus influence the horizontal motion as well. For example, an array located near a column or in an area of short rafter spans would experience different vertical excitation than an array in the middle of a long roof span.

Ground Motion Characterization and Rooftop Seismic Motions

Rational evaluation of array sliding displacement requires the use of appropriately derived earthquake time history records. Three-component ground motions are necessary because vertical motion can dynamically affect the normal force exerted on the roof membrane by the array and can thus affect the array sliding "effective" friction and resulting displacement. The evaluation method described herein uses free-field ground motions which are transmitted through a model representing the supporting structure as the input, as opposed to recorded rooftop motions.

For the present study, a suite of seven three-component ground motion records was selected to represent an entire range of possible ASCE 7 site classes (in two groups: A – D, and E) for each of five different ground motion intensities - *Intensity Levels* 0, 1, 2, 3, and 4 - based on USGS S_s/S_1 mapped parameter pairs, wherein "*Level 4*" represents the highest intensity, and "*Level 0*" represents the lowest. The S_s/S_1 values corresponding to the upper limit of each Intensity Level category are listed in Table 1, and a spectral comparison for the various levels is shown in Figure 6. A map of Intensity Levels for the continental United States portion of the study region is shown in Figure 7.

Table 1: Maximum S_s and S_1 values associated with each *Seismic Intensity Level*, and the percentage of the sites in the study region falling into each Level.

Seismic Intensity Level	Maximum S_s (g)	Maximum S_1 (g)	% of sites in study area
0	0.45	0.18	40%
1	0.70	0.27	21%
2	1.95	0.80	35%
3	2.60	1.10	3%
4	3.70	1.38	1%

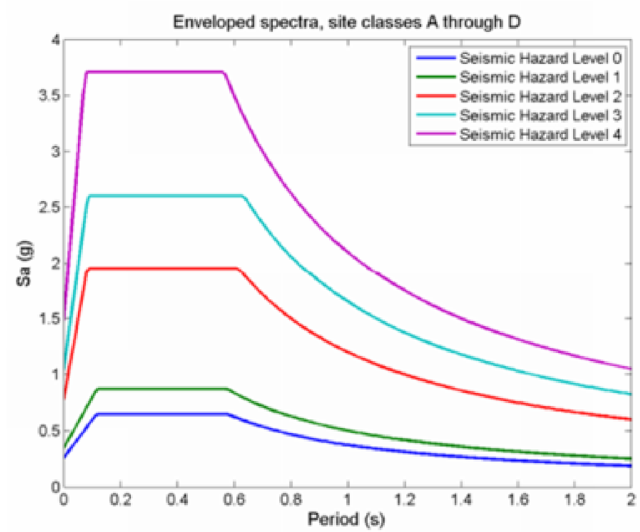

Figure 6: Design spectra obtained by enveloping over site classes A through D for the upper bound S_s amd S_1 values associated with each Seismic Hazard Level.

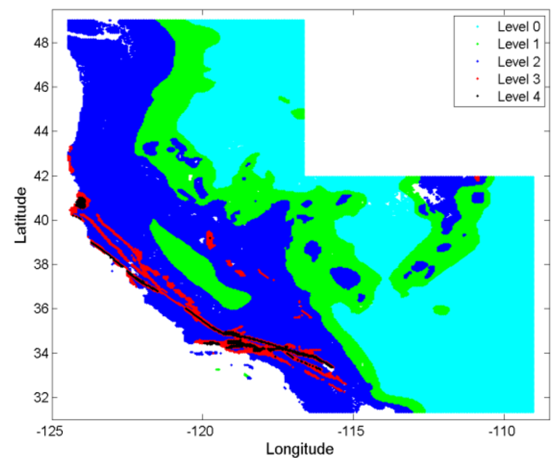

Figure 7: Map of the portion of the study region in the continental US, with the seismic hazard level indicated by the color of shading.

The five selected graduations of *Intensity Levels* correspond to convenient "cutoff" levels of rooftop sliding displacement, as opposed to ground acceleration. For example, the case of an array on a roof membrane with low-to-medium friction and a minimal slope of ¼:12 would be expected to experience near-zero calculated relative displacement on a roof under *Intensity Level 0* ground motions. A separate set of ground motions was selected for Site Class E for each *Level 0* through *4*. Accordingly, a total of ten suites of seven scaled three-component ground motions were prepared for the current study.

The selection and scaling of MCE ground motions for a given *Seismic Intensity Level* represents the strongest MCE ground motion expected in any of the geographic areas included under the respective Level.

Once the above *Intensity Levels* were defined, three component time histories were selected and scaled to represent the ground motions expected in each region. First, the S_s and S_1 values identified above were multiplied by the ASCE 7 site coefficients to determine MCE spectra for site classes B through E. The site class E spectra were used for selection of a suite of ground motions for each *Intensity Level*, and the class A through D spectra at each *Intensity Level* were enveloped to produce a single spectrum used to represent loading across all of those site classes. The spectrum for a given seismic hazard level may thus be significantly higher than the site-specific MCE spectrum at any given location falling in a given level, as the spectrum computed here envelopes the MCE spectra for all locations having that seismic hazard level, and (for site classes A through D) envelopes a range of site classes.

With the above calculations defining response spectrum targets, deaggregation calculations were performed to identify typical earthquake scenarios for each *Seismic Intensity Level*. Deaggregation calculations identify the earthquake scenarios most likely to cause MCE-level shaking at a given site. These calculations vary by period, as different scenarios are sometimes responsible for short-period and long-period portions of the MCE spectrum, so here 0.2s was chosen for the calculations to focus primarily on the short-period portion of the spectrum. Deaggregation results were obtained for populated cities in each *Intensity Level* (Phoenix, Las Vegas, Portland, San Bernardino and Palm Springs for *levels 0* through *4*, respectively). These results were used to determine typical ranges of earthquake scenarios that should be matched when selecting time histories and were used to guide the ground motion selection.

Recorded ground motions were then selected and scaled to represent each analysis case of interest. A few notable features of the selection and scaling are noted below:

- The magnitudes and distances defining the selected ground motions were constrained to reflect typical earthquakes controlling the seismic hazard in each Seismic Hazard Level category. These constraints are summarized in Table 2.
- Recordings were selected from locations with site conditions similar to the target site condition for a given ground motion set. This was done by matching the shear wave velocity in the top 30m of the recording site (V_{s30}) to the V_{s30} associated with the site class range, though it was not possible to obtain perfect matches and also satisfy the other selection criteria. The range of V_{s30} values in each ground motion set are summarized in Table 2.
- Individual ground motions were selected and scaled so that their SRSS spectra closely matched the target MCE spectrum between 0.0 and 2.0 seconds while also ensuring that the average of the SRSS spectra exceeded the target MCE spectrum over this period range.
- The two horizontal components of the ground motion recordings were oriented in the fault-normal and fault-parallel directions.
- All three components of each ground motion were scaled by the same scale factor. Scale factors of the ground motions were minimized to the extent possible while also satisfying other ground motion selection requirements. Maximum scale factors for each ground motion set are reported in Table 2.

Table 2: Properties of selected ground motions for each *Seismic Intensity Level* and Site Class range.

Hazard Level	Site Class	Min M	Max M	Min R	Max R	Min V_{s30}	Max V_{s30}	Max Scale Factor
0	A - D	5.9	6.9	10	75	340	900	3
0	E	6.0	6.9	10	60	190	280	3
1	A - D	5.9	6.9	5	50	260	600	3
1	E	5.8	6.9	5	30	190	280	4
2	A - D	6.7	7.6	5	50	260	800	4
2	E	6.5	7.6	5	50	115	285	4
3	A - D	6.7	7.6	0	15	320	800	5
3	E	6.5	7.6	0	30	190	280	5
4	A - D	6.7	7.6	0	25	270	660	6
4	E	6.5	7.6	0	25	210	280	5

As an example, the horizontal SRSS spectra of the scaled ground motions representing *Intensity Level 2*, site classes A-D are shown in Figure 8. This analysis case pertains to significant areas of both Northern and Southern California. The ground motions are significantly larger than the MCE on average at some periods (e.g., 0.2s), but this was necessary to satisfy the above selection requirements and ensure that the average of the ground motions' spectra were larger than the MCE over a broad range of periods (i.e., 0.1s and 0.7s were controlling periods in this case).

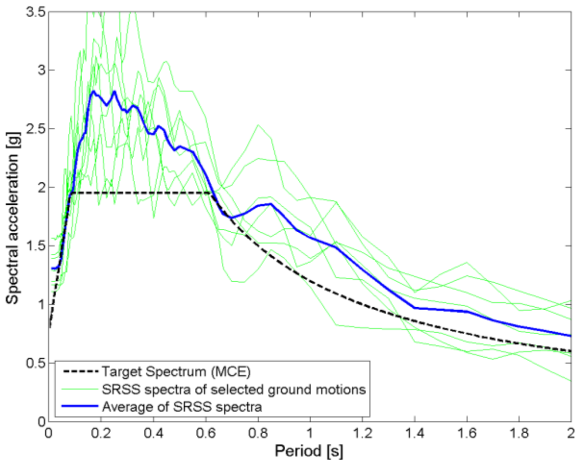

Figure 8: Target MCE for *Intensity Level 2*, site classes A-D, horizontal SRSS spectra of the scaled ground motions representing this analysis case, and the average of the selected and scaled spectra.

Scaling of ground motions for application to the case of a "generic" supporting building must consider a relatively broad range of possible building periods, because:

- The potential range of initial periods of buildings that typically support rooftop PV arrays, such as warehouses, industrial buildings with large footprint area, and big-box retail outlets.
- The possibility of building lateral system yielding may cause the "effective" period of a supporting building to change.
- The relative displacement of the array on the rooftop introduces an additional source of "period" variation.

The current study utilized a range of periods of 0 to 2.0 seconds for scaling periods.

The scaled MCE records were uniformly factored by 2/3 to obtain DBE-level records for response history analyses.

Generically Applicable Analysis Approach

The ultimate goal of the dynamic analysis to facilitate a PV array installation is to determine life-safe movement clearance values; that is, clearance values that protect against array falling hazards and fire risk.

As for other generically applicable analysis approaches, the objective of PV array displacement computation is to obtain a result that considers the influential conditions that may occur, but that is reasonably simple, reasonably conservative to implement, and yet broadly applicable.

Possible sources of deviation from assumed rooftop conditions that should be considered include:

- Uncertainty of building modal response.
- Possible occurrence of nonlinear response of the supporting building.
- Actual effective friction differences from measured test values.
- Directionality of ground motions.
- Effects of roof diaphragm vertical and horizontal stiffness variation, as well as location of array on a rooftop.
- Mass of the array relative to the reactive seismic mass of the building supporting it.

Forell/Elsesser Engineers developed a generically applicable displacement determination approach, which is applicable to the western U.S., for isolated PV arrays on near-flat rooftops, and has assisted several PV manufacturers to implement and obtain AHJ approval for isolated PV installations since 2003. The approach utilizes a simplified building and array modeling approach that addresses the *Seismic Intensity Levels* discussed above and various rooftop conditions.

Scope Limitations for Generic Method

This generic analysis method is limited to a finite number of conditions or parameters, chosen to address the most commonly encountered cases. The parameters have been selected to provide a range of conditions commonly encountered at typical installations. The parameters considered, and the limitations inherent in parameter selection are summarized herein.

Building Type and Modal Behavior: The building types considered in the study are characterized by the range of dynamic structural periods assumed. The assumed period range represents the dynamic characteristics of most buildings of the type that would be expected to support significant PV array installations and would therefore be of most interest to commercial PV systems manufacturers. Six values of building period (T) are considered: T=0.20 sec, 0.40sec, 0.70sec, 1.00sec, 1.50sec, and 2.00sec, with the smallest and largest values actually being more extreme than would be expected for such buildings. Average peak horizontal displacement values used for establishing safe seismic gaps consider the responses obtained from all the above period values, in order to bracket for period shift due to the effects of possible building nonlinear behavior and diaphragm flexibility, without actually modeling the supporting building specifically.

The vertical structural period of the building is taken as 0 seconds, corresponding to a vertically rigid superstructure. The predominant periods of vertical ground motion are normally short (0.2 second or less). Rooftop periods vary between column locations and span locations, so they can be between about 0.1 second and 0.5 second or more. Consequently, it is important to study the sensitivity of the lateral displacement response of the array to possible variations in vertical structure period.

The roof diaphragm is assumed to be horizontally rigid, with the effect of potential diaphragm flexibility (and corresponding period growth) being accounted for by using a structural period range that extends up to 2.0 seconds. Although it is possible for array displacement to be affected by being near a lateral bracing element as opposed to midway between two such elements, this is not viewed as an important difference over the span of a diaphragm.

Structural Behavior: The supporting structure is presumed to behave linear-elastically during a DBE-level event. Most buildings of the type considered would actually experience some degree of nonlinearity in a DBE event, varying from minor to significant nonlinearity, depending on the attributes of the structural system and other factors. However, the assumption of linearity is based on the likelihood that a linear-elastic building will most often induce more extreme rooftop acceleration input to the supported array, and will thus normally be more conservative. This assumption is true for most cases but not all. The relatively small underestimate of displacement with this assumption are offset by taking the displacement response over entire range of building periods discussed above for any application.

Damping: A relatively high value of effective viscous damping (5%) is assumed to complement the conservative assumption of no structural yielding discussed above. Depending on the type of structure and on the materials used, higher values of effective damping are likely during a DBE-level event due to inelastic material deformation.

Roof Slope: Three roof slope magnitudes are considered: a slope of 1/4 inch per foot (1/4:12) representing a "flat" roof condition and slopes of 1/2 inch per foot (1/2:12) and 1 inch per foot (1:12) representing a normal slope range for such roof systems. The slope values are each considered for two possible sloping arrangements, leading to a total of six roof slope permutations. The two sloping conditions are:

- The roof slope occurring in the E-W axis only (flat in the N-S axis).
- The roof slope occurring in the N-S axis only (flat in the E-W axis)

The direction of the slope is important because the ground motions used in the study consist of two orthogonal, horizontal components that can cause different responses in each direction. A truly flat roof condition is not considered.

Roof Membrane Types and Friction: Full-scale friction testing was performed for a four-panel (2x2) array specimen on a specific support system by pulling it across several commonly used types of roof membranes in each direction. The array effective frictional force was measured digitally with respect to time, and the coefficient of effective friction was computed as it varied with displacement. The tests were conducted independently in the two principal directions. Refer to Fig. 5 for a photo of such a test procedure, and **Error! Reference source not found.** for a sample friction-displacement plot. The analysis process conservatively uses the mean value of friction for each test direction minus two standards of deviation. The types of membranes tested include common varieties for large commercial buildings: PVC, EPDM, TPO, and Modified Bitumen.

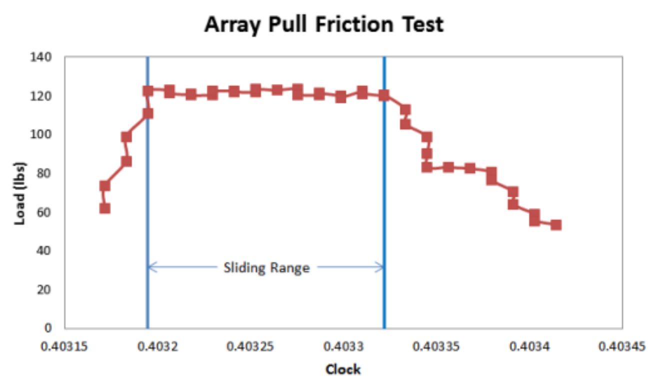

Figure 9: Sample test plot from example friction pull test of a PV array

Seismic Mass of Array: The array mass has been considered to be at or between the values of 5% and 10% of the building seismic mass. The 5% value is reasonable for a ballasted array deployed on a large single-story warehouse building with tilt-up or CMU walls. The 10% value corresponds to the same array supported on a smaller building with light-framed walls, such as a school gymnasium.

Ground Motion: The ground motions used include all motions derived as derived in the above section. The effect of including the vertical ground motion is significant.

Simplified Structure/Array Model:

The building is modeled as an inverted-pendulum-type structure with linear-elastic behavior. The stick is an axially rigid, zero-mass beam element of rectangular cross section and fixed height, with a lumped mass at the top of the stick representing the total seismically reactive building weight. The mass of the array rests on a nonlinear friction isolator link element. The top of the structure (representing the roof) is fixed rotationally on all three axes, and is therefore capable of deformation in both horizontal directions via fixed-fixed deformation of the stick. The stiffness of the stick is varied to produce the desired fundamental period. For the stick representing the building, all behavior is linear-elastic throughout each analysis.

See Figure 10 for a schematic representation of the analytical model. The roof slope is represented explicitly within the model of the isolator element.

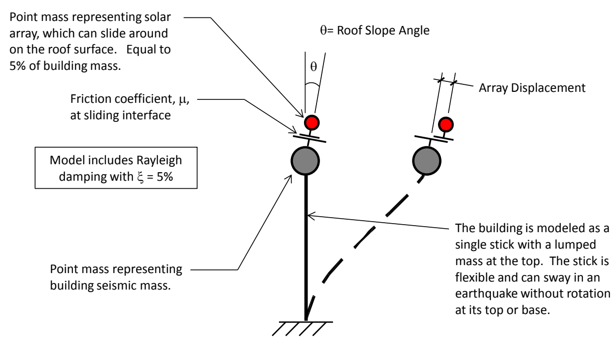

Figure 10: Schematic Representation of Analysis Model

The array is represented as a lumped mass in order to determine displacement demands. The actual components of the array would be subject to lateral loading based on their own inertia and their own support friction. Thus, if all components (solar modules or panels) were similar and friction was the same throughout a roof diaphragm, the connective forces between the array components themselves would be zero. Although these perfect conditions would not occur, the connective forces between array components would not be significant. Interconnection forces between panels in an anchored array would generally be significantly higher where the modules are only intermittently anchored to the roof, as is normally the case for anchored PV arrays.

Displacement Computations: Array displacements, which are taken relative to the rooftop mass, are computed in SAP using only the "Direct Integration" process, as more expedient approaches do not correctly calculate sliding displacements of this type of system. For each ground motion record, the "maximum displacement" is conservatively taken as the vector sum (SSRS) of the maximum displacement in each direction instead of the maximum considering the vector resultant for each time step of the analysis. The value of displacement reported for each building period is the average of all maximums for the entire suite of ground motions. This approach is the same as that prescribed by ASCE 7 for response-history analysis results.

Recommended Seismic Displacement Clearance: The recommended array seismic clearance is taken as 1.1 times the reported average maximum displacement. The value of 1.1 is an arbitrary factor to add conservatism to the result.

Non-Seismic Clearance Considerations: The actual gap provided must consider the requirements of firefighting access and OSHA clearance requirements. Typically, these requirements result in a gap of 4 to 6 feet. A question may occur about whether the displacement clearance requirements should be additive to the OSHA/fire access clearance. For many (perhaps most) cases, the array seismic clearance will be significantly less than the other required clearance, implying that the access clearance can absorb the seismic displacement without significant hindrance to rooftop circulation. However, the AHJ may require them to be additive.

Specific Application Example

As a specific example, a maintenance warehouse building near downtown LA is to have a ballasted PV array installed on it, and the safe seismic clearance is desired. Refer to Figure 11 for the site location. The coordinates are established using Google Earth, then the USGS spectral values are obtained from the USGS website, as one would do for a building design project. The S_s and S_1 values are compared with the *Seismic Intensity Levels* discussed herein, and it is found that the building is near the upper limit of *Level 2*. The precise ASCE 7 site class is not known, but it is known that the site class is not "E." The *Level 2* DBE motions for site classes A-D are therefore selected. Refer to Figure 8 for an illustration of the selected motions and the target spectra, which is the envelope of code spectra for site classes A-D. Refer to
Table **3** for a listing of scale factors used.

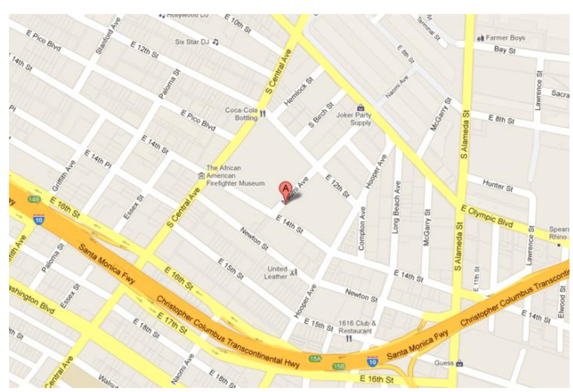

Figure 11: Displacement Calculation Example Building Site.

Table 3: Earthquake Ground Motion Suite

Station	Earthquake	M	Distance	Scale Factor
Gukasian	Spitak, Armenia	6.8	36.2	4.0
Saratoga - Aloha Ave	Loma Prieta	6.9	8.5	2.5
LA - Wadsworth VA Hospital North	Northridge-01	6.7	23.6	3.8
Sylmar - Converter Sta East	Northridge-01	6.7	5.2	1.4
Gebze	Kocaeli, Turkey	7.5	10.9	4.0
TCU122	Chi-Chi, Taiwan	7.6	9.4	3.5
Bolu	Duzce, Turkey	7.1	12.0	1.4

The period of the building is not certain; ASCE 7 formula 12.8-7 provides a rough estimate of 0.2 seconds, which is well within the 0-2.0 second applicability range.

The selected ground motions are listed in
Table **3**. As a comparison, the distance to fault and magnitude values are compared with the results of the de-aggregation plot in Figure 12. It is verified that the magnitude and distance of the selected motions are comparable.

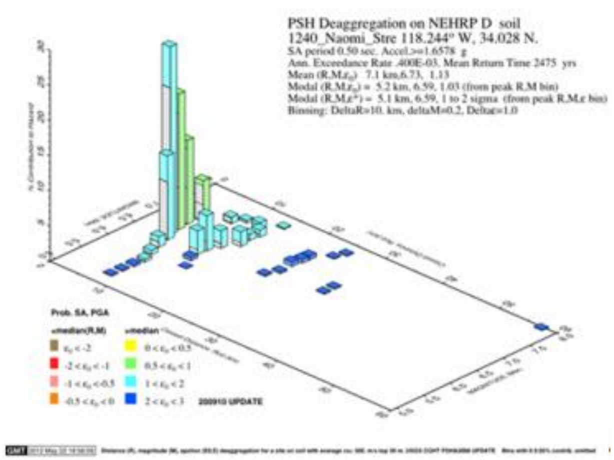

Figure 12: DBE Deaggregation for Example Site.

The array friction coefficient for the roof membrane type used is 0.47. This is the average test value minus two standard deviations. The applicable displacement response curve is shown in Figure 13, with the rough period of 0.2 seconds marked. Note the maximum response result of 10.21 inches corresponds to a period of 1.0 seconds. This value is used as a basis for the clearance calculation. The recommended clearance is taken as 1.1*10.21 = 11.23 inches.

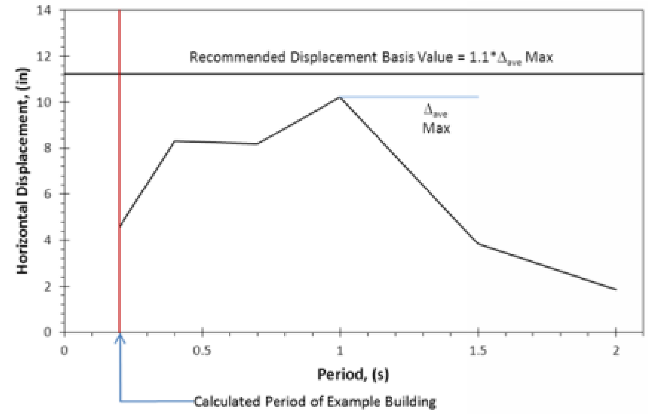

Figure 13: Calculated response vs. recommended seismic clearance

Other Considerations

"Threshold" of Seismic Intensity Causing Movement: Figure 14 is a plot of movement vs. *Seismic Intensity Level* for site classes A through D. The basis of this plot is a roof with a slope of ¼:12, a friction coefficient of 0.47, and a building period of 0.2 seconds. The implication of this plot is that, for the above conditions, significant movement is not expected at *Seismic Intensity Levels* below *Level 2*, which represents very strong motion input. Indeed, for many sites even in California, no array movement would be expected for DBE level ground motion. It should be noted that the comparison made here is even more extreme for larger building periods.

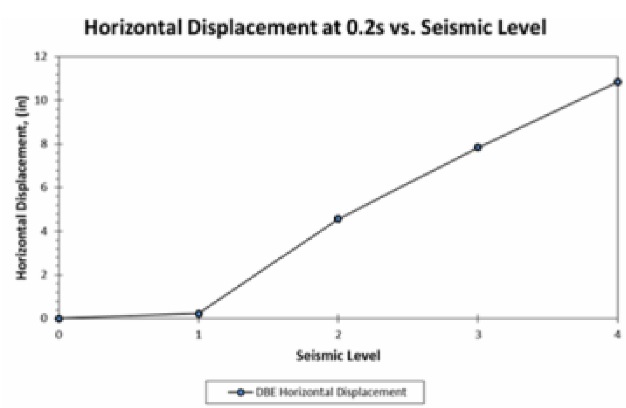

Figure 14: Displacement Occurrence Threshold Corresponding to *Seismic Intensity Level*

Horizontal Yielding of Building: The effect of lateral system yielding (based on varying R values) can be seen from Figure 19, for *Level 3* ground motions, a slope of ¼:12, an initial structural period of 0.5 seconds, and a friction coefficient of 0.5. The array mass is 5% of the building weight. Except for the results for R value of 2, the occurrence of yielding significantly reduces the sliding response of the array.

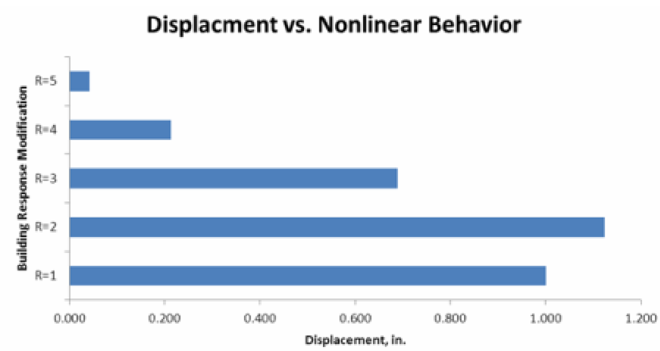

Figure 15: Horizontal displacement comparison at varying response modification coefficient values. Site Class A through D with slope at 1/4": 12". Normalized to R = 1

Building Base Shear Study: A limited study was done to examine the effect of base shear of a building with an isolated rooftop array and with the case of an anchored array. Refer to **Error! Reference source not found.** for a graphic comparison of the results of this study, which assumed a coefficient of friction of 0.50, *Intensity Level 2* site class A-D ground motions, slope of ¼:12, and an array-building mass ratio of 5%. The isolated array mass results in lower building base shear than the fixed array case. To further this study, the base shear of the building without an array was computed and compared with the base shear of the building models with an array. It is interesting to note that the isolated array resulted in lower building base shear than even the building without the array. This finding implies that the isolated array produces a side benefit of a tuned-mass damping effect.

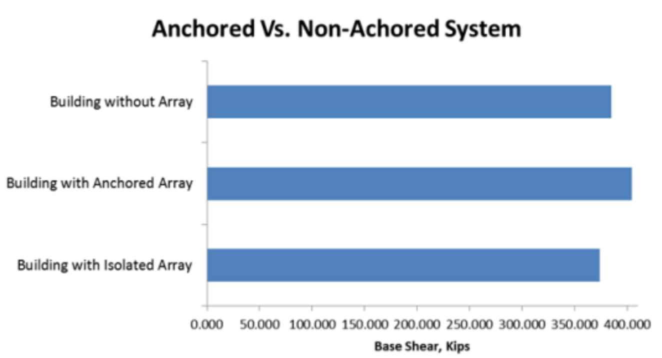

Figure 16: Building horizontal base shear comparison between anchored, non-anchored and building without array. Site Class A through D with slope at 1/4": 12"

Effect of Roof Slope: Numerous comparisons have (not surprisingly) indicated roof slope to be a dominant variable in determining the magnitude of seismic sliding displacement. From the example comparison in Figure 17, it can be seen that maximum array displacement increases rapidly as slope increases. From such studies, it is evident that a roof slope of 1:12 to 1.5:12 (depending on available friction) should be regarded as a maximum for deployment of isolated rooftop PV arrays.

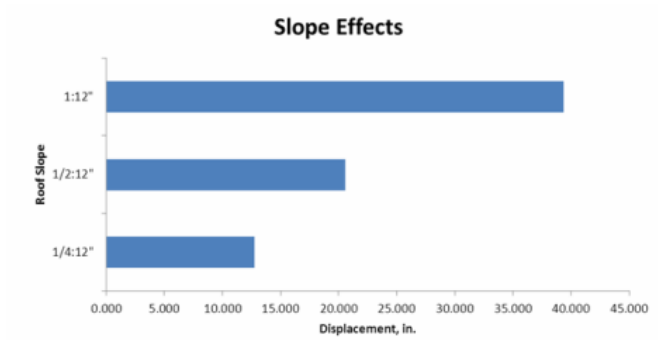

Figure 17: Horizontal displacement comparison at varying roof slopes. Site Class A through D with slope at 1/4": 12"

MCE vs. DBE: A comparison of DBE and MCE displacements was made to determine the potential effect of occurrence larger than expected ground motion. The results of one example are shown in Figure 18. The ASCE 7 approximate building period for this example is 0.20 seconds. It is interesting to note that the peak DBE response (at T=0.70 seconds) is close to the MCE response at the building period. This observation, which was observed repeatedly, is one reason it is recommended to use the peak DBE displacement as a seismic clearance basis, given the possibility of period shift as the building lateral system begins to experience yielding. Refer also to Figure 15, which indicates the increase in displacement that occurs with slight building nonlinearity.

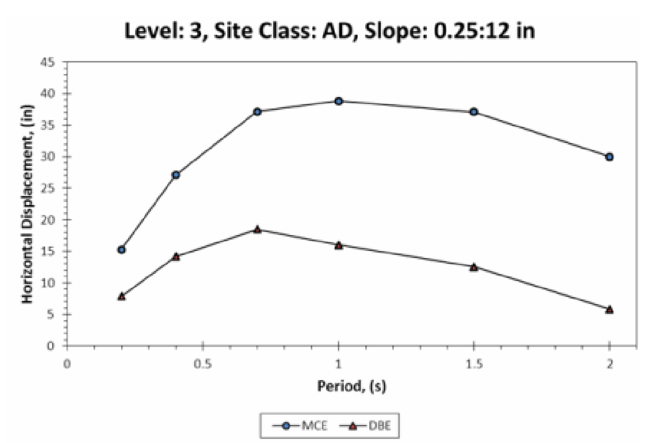

Figure 18: Horizontal displacement comparison at MCE/DBE levels. Site Class A through D with slope at 1/4": 12

Validation by Shake Table Test

The basic approach described herein has been validated by shake table testing. The process of shake table testing requires the input of a roof-response motion into the table. Most shake tables are capable of ground motion-level accelerations, but may not be capable of amplified motions that would occur on a rooftop of a building. Refer to Figure 19 for an illustration of a shake test on a ballasted array that was done at the Seismic Response Modification Device Lab at University of California San Diego in 2010. Seismic sliding displacements were measured for rooftop motions corresponding to ground motions in the range of *Intensity Levels 2* and *3*. Displacement measurements were reasonably close to the values determined through the analysis process described herein. Refer to Figure 19 for an example comparison of shake table results and analysis results.

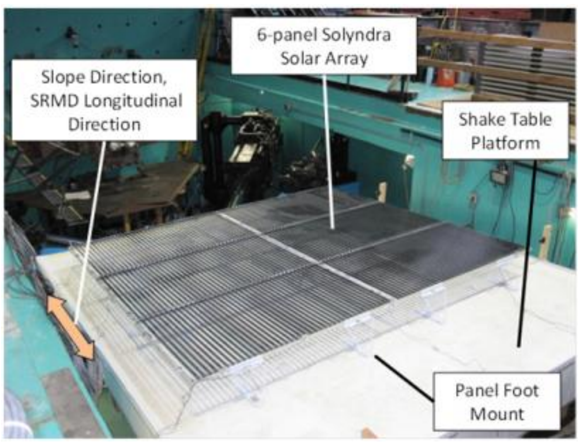

Figure 19: Photograph of assembled array on shake table platform.

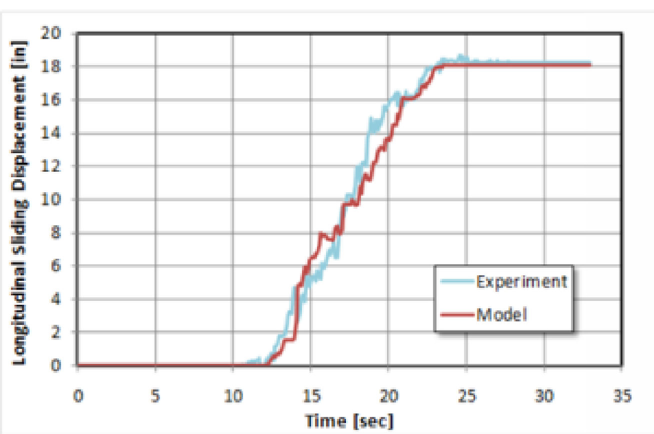

Figure 20: Comparison of shake test results with analysis.

Conclusion

A rational approach to evaluating potential seismic displacement between an isolated rooftop PV array and the roof of the supporting building has been developed to inform the generalized determination of safe seismic clearance values for such array installations. The developed approach utilizes empirically derived friction coefficients from full-scale array specimens and uses conservatively derived ground motion assumptions currently developed for seven western U.S. states. The approach has been verified through the use of shake table testing using simulated three component rooftop seismic motions. Important conclusions of the study described herein are:

- Significant rooftop motions are required to cause PV arrays to displace. Such motion is characteristic only of relatively high seismicity, such as in the coastal areas of California.
- Roof slope has a dominant effect on sliding displacement expectations. A maximum roof slope in the range of 1:12 to 1.5:12 is suggested for the use of isolated PV installations in highly seismic areas, depending on the available coefficient of friction.
- The use of isolated PV arrays can result in lower building base shear than in the use of an anchored PV array. Furthermore, isolated PV arrays may result in tuned-mass damping effect that could actually lower building base shear below that expected without any PV array.

It is concluded that the use of isolated PV arrays can be safe as well as economical, and could encourage the growth of renewable energy in highly seismic areas.

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