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**Serviceability Analysis and Feasibility Study of Rooftop PV System on Commercial Buildings in New York City**

Daniel A. Shedo

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SERVICEABILITY ANALYSIS AND FEASIBILITY STUDY OF BALLASTED ROOFTOP PV SYSTEM ON COMMERCIAL BUILDINGS IN NEW YORK CITY

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Daniel Adamu Shedo
University of New Haven
West Haven, Connecticut

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SERVICEABILITY ANALYSIS AND FEASIBILITY STUDY OF ROOFTOP PV SYSTEM ON COMMERCIAL BUILDINGS IN NEW YORK CITY

APPROVED BY

Byungik Chang, Ph.D., P.E., M.B.A.
Co-Thesis Adviser

Hyungjoo (Jay) Choi, Ph.D., P.E.
Co-Thesis Adviser

Goli Nossoni, Ph.D.
Committee Member

Byungik Chang, Ph.D., P.E., M.B.A.
Program Coordinator

Ronald Harichandran, Ph.D., P.E.
Dean of College of Engineering

Nancy Savage, Ph.D.
Interim Provost
ABSTRACT

Concrete Masonry Unit (CMU) blocks (Ballasts) to counterbalance an uplift wind pressure on Photovoltaic (PV) panels have been used widely. In contrast to mechanically attaching PV panels through penetration of a roof slab, the use of CMU blocks is preferred by many building owners, as penetrating and attaching the panels to a roof slab will pose a serious threat to the warranty of roof membranes, which typically last for 30 years. On the other hand, placing CMU blocks along with PV panels and other system components on a rooftop slab adds additional dead load, which may raise a concern regarding the long-term serviceability of the rooftop slab.

This study is mainly focused on the uplift wind calculation to determine the optimal number of required ballasts, roof slab serviceability analysis to determine sustainability in the form of long-term deflections, and a feasibility study on the economic benefits of PV installation. A building with ballasted PV which has typical material and section properties located in New York City was selected and analyzed.

The initial pre-PV long term deflection was found to be 0.83 in. and 1.05 in. for roof live loads of 60 psf and 100 psf, respectively, while the allowable deflection limit for the given slab geometry is found to be 1.15 in. in accordance with the local code design requirements. Both live load conditions were applied along with self-weight and other dead loads to calculate the initial long-term deflection. The long-term deflection results from post-PV serviceability analysis were found to be below the pre-PV deflection. Hence, it is concluded that installing PV systems on a rooftop slab does not have a significant effect on its service life.

Economic feasibility analysis is also conducted by using various project profitability indicating parameters. Simple Payback Period, Net Present Value, Discounted Cash Flow, Internal Rate of Return, Simple Cash Flow, and Profitability Index were found to be 5 years, $165,173, $135,486, 18%, and $402,772, respectively. This confirms that installing PV systems on rooftop slabs of existing buildings in New York city is an attractive investment.

Key words: PV panels, CMU blocks, Serviceability, Sustainability, Long term deflection, Exposure Class, Risk Category, FE modeling, Feasibility.
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CHAPTER 1: INTRODUCTION

While energy is key to the day-to-day activities of a society, a large portion of it is obtained from non-renewable sources. According to the U.S Energy Information Administration (EIA) 2021 report, the U.S needs a total of 97.33 Quads (2.85×10⁸ GWh) per year to meet its energy demand. Out of this, only 12% is covered by renewable energy sources. The rest of the energy gap is filled by non-renewable sources like petroleum, natural gas, coal, and nuclear electric power (see Figure 1.1). The production and usage of energy from non-renewable sources has noxious impacts. Climate change, acid rain, air pollution and many more are factors that are associated with non-renewable energy production and consumption.

![Diagram](image)

**Figure 1.1 U.S Primary Energy Consumption by Energy Source, 2021 [1]**

Generating energy from renewable sources such as photovoltaic (PV) solar panels can effectively address these issues. Solar radiation is one of the most abundant renewable resources that can be utilized to generate clean energy without a major effect on climate. On average, 1.73 × 10⁵ Terra Watts (TW) of solar radiation strikes the earth’s surface while global electric demand is about 2.7 Terra Watts (TW). Likewise, if only 0.6% of the U.S land is covered by photovoltaic panels, the nation’s energy would be met [2]. To meet the energy demands in metropolitan cities such as New York City, installing ballasted PV panels on rooftop slabs of buildings could be considered as an alternative solution.
This study addresses the serviceability and sustainability issue of existing buildings in New York City and focuses on the deflection due to loading from ballasted PV systems on rooftop slabs. Additionally, the economic feasibility of the ballasted rooftop PV system is also studied.

1.1 Problem Statement

Installing PV panels on existing buildings is one of the common ways to harvest solar energy. However, the building structure might be exposed to risk due to the PV system. The existing building may not have the required capacity to withstand sustained loads transferred from PV panels and CMU blocks (Ballasts), especially in the case that regular maintenance work is not performed. Thus, a thorough serviceability analysis of the building needs to be conducted. Additionally, as shown in Figure 1.2, rooftop slabs usually have membranes, and the membranes prevent water infiltration to the inner surfaces of the slab and the building. More often, these membranes have a 30-year warranty period. Consequently, property owners probably are not willing to mechanically attached PV panels as they will lose the warranty of membranes. Moreover, this installation method can potentially cause leakage and insulation issues in the future. In addition, the effect of ballasted PV installation on the long-term serviceability of the building needs to be studied.

![Figure 1.2 Rooftop Slab and Membranes](image)

The economic feasibility of PV systems in comparison to the conventional electric usage is among the forefront factors which inhibits property owners from going solar. Due to the increased rate of inflation, direct and indirect costs related to PV systems have increased, and this
could be another factor that pushes away building owners from installing PV systems. On the contrary, the production of solar PV panels has been increasing over the years and consequently the installation cost has significantly decreased. In addition, the extension of Investment Tax Credit (ITC) and the Production Tax Credit (PTC) with increased incentives by the U.S congress provides the potential to attract more users to install PV systems. Thus, a detailed feasibility study is needed.

The long-term deflection prediction model that is currently in practice has been developed half a century ago [3]. Since various advancements have been made in material and design process, this prediction model needs to be updated. In the year of 2019, ACI 318-19 [3] updated the equation for the effective moment of inertia for long-term deflection. Apart from this, the model and formula that has been developed in the 1960’s is still in use. Studying the works of scholars in this regard is worthy.

1.2 Objectives

The primary objective of the study is to determine if placing ballasted PV system on a commercial building will increase the long-term deflection of the rooftop slab. The other associated objectives of the study are to:

- Determine the uplift wind pressure on PV panels based on building height, location and exposure class using various specifications.
- Calculate the required CMU blocks which are used to resist uplift wind pressure.
- Conduct a computer-aided (Finite Element Modeling) structural analysis to obtain long-term deflection of the rooftop slab due to ballasted PV system.
- Study long-term deflection prediction models available in current codes and literature.
- Conduct economic feasibility analysis for PV systems in New York City.

1.3 Research Significance

The major significance of this research is to show that placing ballasted PV system does not have a major impact on the serviceability of buildings. In addition, by conducting feasibility analysis, this research can be an input to encourage the wider public to go solar. By doing so, carbon emission to the environment could be reduced.

The emission per kWh can be used as a tool to compare how much carbon dioxide or equivalent greenhouse gases can be reduced from the atmosphere by using renewable sources like solar energy instead of the non-renewable sources. According to studies on PV systems [4, 5, 6,
7], an approximate amount of 40g/kWh of CO₂ equivalent is emitted to the atmosphere during the lifetime of PV system from raw material extraction to disposal. On the other hand, up to 1,000g/kWh of CO₂ equivalent is released into the environment from non-sustainable sources like coal. The total Green House Gases (GHG) emitted from PV during it’s service life as per the Life Cycle Assessment (LCA) model of National Renewable Energy Laboratories (NREL) [8] is equivalent to other renewable sources but much smaller than the non-sustainable energy production.

1.4 Thesis Outline

Chapter 2 presents the historical development of PV systems, wind analysis provisions for PV panels from various codes, previous researches related to long-term deflection modification factor and economic feasibility analysis. Chapter 3 describes the procedures of calculation for uplift wind pressure and required CMU blocks. The methods and input data used for economic feasibility analysis are addressed as well. Chapter 4 discusses finite element modeling to predict the long-term deflection with the ballasts. Chapter 5 presents results obtained from wind analysis, finite element modeling, and economic feasibility study. Conclusions in Chapter 6 are drawn from the results of the study. Chapter 6 also includes recommendations for future works.
CHAPTER 2: LITERATURE REVIEW

This chapter discusses the provisions from various specifications to calculate uplift wind pressure and CMU blocks. In addition, the summary of research work from various scholars is provided. A special emphasis is on the long-term deflection modification factor. At the end of the chapter, existing research on economic feasibility analysis of PV systems is discussed.

2.1 Historical Background

In the early generations of high-rise buildings in the U.S. (i.e., the home insurance building), which was completed in Chicago in 1885, structures required a relatively little operational energy demand as technologies like air conditioning and fluorescent lighting were not yet developed [9]. As technologies that have substantially transformed human life continue to evolve, the energy demand of buildings continued to grow in an unprecedented manner. This resulted in a significant shift on how buildings are designed and thus affected the architectural and structural design procedures. During the 1930s, Massachusetts Institute of Technology (MIT) researchers were striving to design a house that can be powered by solar energy and were able to complete the first house with air conditioning that could be powered by solar energy in 1939 [10].

After World War II, when there was frequent energy shortage, it became clear that there should be a viable alternative energy solution for buildings and homes. Engineers have turned their focus on further development of efficient PV systems that can be installed on roof and ground surfaces and provide adequate energy for users. The 1970s could be considered as the golden age for research in solar energy generation including PV. In 1974, the congress passed a legislature into law called ‘Solar Energy Research and Development Act’, which enabled the country to enter into more intensive research for solar energy development. In 1977, the U.S Department of Energy (DOE), launched the solar energy institute which is dedicated to obtaining energy from the sun.

The years in 1980s and 1990s were accompanied by pioneering developments. Since 1992, Ascension Technology has been installing PV arrays on top of roofs of variety of structures like schools, factories, and warehouses with the U.S. DOE. The company claims to be the pioneer of using ‘low-cost ballasted approach’ for mounting PV arrays although there is no verification from other sources. There were two instances where the solar panels were blown away by wind. In 1999, the tallest skyscraper commercial was completed. This building was also one of the most energy efficient buildings at that time. The building includes Building Integrated Photovoltaic (BIPV) panels from the 37th floor to 43rd floor. In the year of 2000, a family in Morrison, Colorado installed
the largest residential solar panels in the U.S. In 2001, Home Depot started selling residential solar systems in their three branches and a year later 61 branches began to sell the system. In the same year, a company, Power light Corporation, installed the largest rooftop solar power system in the U.S. Afterwards, significant advancements have been made in the efficiency of PV systems.

2.2 Specifications for PV Design

2.2.1 ASCE 7-05 provisions for solar PV

ASCE 7-05 [11] provides three major procedures to calculate the wind pressure on components and cladding. These are a simplified method (specified in section 6.4), an analytic method (specified in section 6.5), and a wind tunnel procedure (specified in section 6.6). For a structure to qualify to be analyzed by a simplified procedure, the mean height of the building shall be less than or equal to 60 ft. It also provides that the wind pressure on component and cladding from the simplified method should not be less than 10lb/ft². The analytic procedure is popular among structural engineers. For this study, the analytic procedure has been followed and the formula for the wind pressure on component and cladding of a structure assuming an open condition is given by Equation 2.1 below.

\[ P = q_h \cdot G \cdot C_N \]  

(2.1)

where,

\( q_h \)= Velocity Pressure

\( G \)= Gust Effect Factor

\( C_N \)= Net Pressure Coefficient

The velocity pressure can be determined by Equation 2.2.

\[ q_h(q_z) = 0.00256K_zK_{zt}K_dV^2I \text{ (lb/ft}^2) \]  

(2.2)

where,

\( K_z \)= Velocity Exposure

\( K_{zt} \)= Topographic Factor

\( K_d \)= Wind Directionality Factor

\( V \)= Basic Wind Speed
I=Importance Factor

Additionally, ASCE 7-05 [11] provides a definitive procedure to calculate snow load on rooftop of buildings. For a flat roof building with a slope less or equal to 5°, Equation 2.3 applies.

\[ P_f = 0.7 C_e C_t I P_g \]  

(2.3)

where,

- \( C_e \)=Snow Exposure
- \( C_t \)=Thermal Factor
- \( I \)=Importance Factor
- \( P_g \)=Ground Snow Load but not less than the following minimum values for low slop, roofs:

where,

- If \( P_g \leq 20 \text{ lb/ft}^2 \), \( P_f = I \cdot P_g \)
- If \( P_g > 20 \text{ lb/ft}^2 \) (0.96 kN/m²), \( P_f = 20 \cdot I \)

The list of coefficients and factors for all exposure classes is provided in Appendix-A.

As obtained by field tests, wind coming from various directions to the edge of a building separates into the lines of the building corner. After that, it will move up and create a zone where the air is swirling for a certain distance until the point of reattachment. After the reattachment point, the wind starts to flow parallel to the roof surface [12]. The distance from the edge of the roof to the point of reattachment is where the wind has a severe effect on the panels. It is estimated that the wind uplift pressure in this zone is 1-2 times larger than the pressure on other zones in the other areas of the roof slab.

ASCE 7-05 [11] does not take this effect into account and uplift pressure is assumed to be uniform throughout the rooftop slab. Thus, ASCE 7-10 [13] introduced zones for the edge and the corresponding effects of wind. As per ASCE 7-10, the edge zones are located within an area of 0.4 times the mean height of the building. Further elaborations on different codes will be discussed in the next sections.
2.2.2 New York City Building Code provisions for solar PV

Chapter 16 of the 2014 New York City Building Code [14] has various provisions for wind pressure on component and cladding elements. It allows engineers to follow chapter 6 of ASCE 7-05 [11], but it also provides some quick procedures for engineers to follow when sufficient data is not available. The following addresses some unique provisions in 2014 NYC Building Code [14] for PV system design.

1) Roof live load is allowed to be zero for an area on the roof covered by PV system with clear space between PV panel and the top of roof surface is 24 in. or less.

2) Snow load exerted on PV system shall be considered.

3) Roof structures that support PV systems shall be designed and checked against deflections and ponding as per section 1607.13.5.1. But due to the limited effect of ponding and for the purpose of simplicity the effect of ponding is not included in the study.

4) The design wind pressure (factored wind pressure) shall not be less than 32 psf as per section 1609.1.3 acting perpendicular to the surface of the PV panel.

2.2.3 ASCE 7-16 provisions for solar PV

ASCE 7-16 [15] provides a comprehensive detail for calculation of wind pressure against various structural and non-structural elements on a building. The code added the effect of edge factor which was a major setback in previous code editions. Thus, the panels on the edge have a greater uplift pressure than the rest of the panels on the interior area of the slab. This will require more ballast numbers in the edge zone than the middle zone, which is in line to what has been obtained in field testing.

The roof wind zones on the edges of the building are also increased. In ASCE 7-05 [11], there was no roof wind area at the corners of the building. ASCE 7-10 [13], has provided a roof wind zone equal to 0.4h at the corners of the rooftop slab. But as per the ASCE 7-16 [15], the wind corner wind zone area has been widened (See Figure 2.1). As per ASCE 7-16, for a tilted solar panel on a flat roof, the corner roof zone can be extended to 2-h. Note that h denotes the building height.
Figure 2.1 Wind Roof Zone for Component and Cladding on a Flat Roof

The calculations for an uplift pressure using this code result in lower uplift pressure than the previous code editions. This is because redundant force is developed to resist uplift pressure since the panels are connected to each other. This decreases the required number of ballasts.
ASCE 7-16 [15] provides the general requirements which are used to determine the velocity pressure on a structure. Figure 2.2 shows various factors to determine design uplift pressure. The formulas and coefficients used are discussed in the next sections.

Figure 2.2 Factors for Uplift Wind Pressure on Rooftop with $\theta \leq 70^\circ$ (ASCE 7-16 [15])
To use Figure 2.2, the following conditions must hold true:

1) \( L_p \leq 6.7 \text{ ft} \)
2) \( \omega \leq 35^\circ \)
3) \( h_1 \leq 2 \text{ ft} \)
4) \( h_2 \leq 4 \text{ ft} \)

where,

- \( L_p \) = Panel Chord Length (ft)
- \( \omega \) = Tilt Angle (deg.)
- \( h_1 \) = Height of the Solar Panel above the Roof at Lower Edge of the Panel (ft)
- \( h_2 \) = Height of the Solar Panel above the Roof at Upper Edge of the Panel (ft)
- \( h_{pt} \) = Height of the Parapet (ft)

The minimum gap between panels can be taken as 0.25 in. and the distance between the rows or arrays of panels must not exceed 6.7 ft. The minimum distance between the edge of the slab and the array of the panels can be taken as the larger of \( 2(h_2 - h_{pt}) \) and 4 ft (1.22 m) for the design pressures in this section to apply. The uplift wind pressure can be calculated with the Equations 2.4 and 2.5.

\[
p = q_h \left( G_{Crn} \right) \left( \text{lb/ft}^2 \right)
\]  

\[
G_{Crn} = (\gamma_p)(\gamma_c)(\gamma_E)(G_{Crn})_{n}\text{om}
\]  

where,

\( q_h = 0.00256K_xK_wK_dK_eV^2 \), in psf

\( G_{Crn} \) = Net Pressure Coefficient

\( K_x \) = Velocity Exposure Coefficient

\( K_d \) = Topographic Factor

\( K_e \) = Wind Directionality Factor

\( K_w \) = Ground Elevation

\( V \) = Basic Wind Velocity

\( h \) = Mean Height of the Building
\[ \gamma_p = \text{Parapet Factor} = \min(1.2, 0.9 + \frac{h_{pt}}{h}) \]

\[ h_{pt} = \text{Parapet Height} \]

\[ \gamma_c = \text{Panel Chord Factor} = \max(0.6 + 0.06 \, L_p, 0.8) \]

\[ \gamma_E = \text{Edge Factor} = 1.5 \text{ for exterior panels, 1.0 elsewhere.} \]

The following condition can also be considered. Linear interpolation is allowed for tilt angle \( \omega \) between 5° and 15°.

\[ A_n = \left[ \frac{1000}{\text{Max}(L_b, 15 \text{ ft})^2} \right] A \]

where,

\[ A = \text{Effective Wind Area (ft}^2) \]

\[ A_n = \text{Normalized Wind Area for Rooftop Solar Panels (ft}^2) \]

\[ W_s = \text{Shortest Side of the Building Plan (ft)} \]

\[ L_b = \text{Normalized Building Length= the minimum of } 0.4 \, (h \, W_s)^{0.5} \text{ or } h \text{ or } W_s \text{ in ft} \]

### 2.2.4 SEAOC-17 provisions for solar PV

Structural Engineers Association of California SEAOC-17 [12] provides a manual that aids in the structural analysis of buildings supporting PV systems. This manual is basically a further elaboration of ASCE 7-16 [15] with recommendations and best practices that could be employed while analyzing structures with PV systems.

In this regard, sections 4.2 and 4.3 of SEAOC-17 [12] provides optional recommendations and refinements that are not included in ASCE 7-16 [15]. Among those recommendations, the below three recommendations can be applicable to this study:

1) Section 4.3.4 has an alternate calculation of roof edge factor.

   Figure 2.3 shows the alternative way of calculating the edge factor. In this study, \( d_2 = 8.26 \) in. and \( h_2 = 0.9 \) ft (10.8 in.). So, when the ratio \( d_2/h_2 \) is calculated, it becomes 0.76. This makes the edge factor as per the alternate procedure to be equal to 1.0.

   Section 4.3.4 also gives elaboration that if \( d_1 < 0.5h \), then a designer can have an option to take the edge factor as 1.0. For this thesis work, \( d_1 = 6 \) ft and \( h = 80 \) ft, 60 ft, and 40 ft. Thus, the edge factor can be taken as 1.0 in this case as well.
Figure 2.3 Alternative Edge Factor Calculation as per SEAOC-17 [12]

where,

\[ d_1 = \text{Distance from the Edge of the Building to the Edge of the First Panel of the Array in the Row.} \]

\[ d_2 = \text{Distance Between Rows of Panels in an Array.} \]

2) SEAOC 17 [12] section 4.2.1 also provides the additional zone called zone1’ which is equivalent to 5h.

Figure 2.4 can be used to determine nominal uplift pressure coefficient for zone 1’. For other zones, Figure 2.4 can be used to determine the minimum nominal uplift pressure coefficient which can be used to calculate the minimum uplift pressure. ASCE 7-16 [15] does not provide minimum uplift pressure calculation.

Figure 2.4 Nominal Wind Pressure Coefficient for Zone 1’ [12]

3) For tilted panels on flat roofs with a clear distance of 0.5\( h_2 \), between each row of panels, it is not necessary to provide the 0.25 in. distance among the panels along the length of the row. This is an optional provision and for this study a gap of 0.25 in. is considered.
2.3 Roof Sustainability and Long-Term Deflection

Structures should be designed both for strength and serviceability requirements to prevent failure and to meet the intended purpose. Usually, failure to address the serviceability requirement will lead to cracks, accompanied by aesthetics problems, door and window closure issues and many more. These issues prevent the structure from furnishing the basic function. Due to the increased use of high strength materials and other factors, slender elements are becoming more common to structures and consequently deflection is more encountered [16]. Moreover, the serviceability requirement is as important as strength criteria and that is why many codes have included provisions to address these issues. PV installation on an existing building cannot be a different scenario.

One of the most critical factors that will govern the long-term service life of a structure is long-term deflection. Therefore, it has been a point of interest for many scholars and various researches have been conducted in this regard to effectively predict the long-term deflection of structural elements. Some of the researches are currently incorporated in various codes. Although the long-term deflection is a major part of the study, research publications in short term deflection have also been reviewed thoroughly. This is because the current existing codes usually amplify the short-term deflection with certain factor to obtain long-term deflection and one way or the other, short-term deflection has an impact on the long-term deflection.

1) Branson

1.1) (Instantaneous and Time Dependent Deflections of Simple and Continuous Reinforced Concrete Beams) [17]:

Branson [17] provided an equation to calculate effective moment of inertia and obtain immediate deflection, which was incorporated in ACI 318 until when equation formulated by Bischoff [16] has been added to replace it. Various research [8, 14] showed that this equation underestimates deflections for members with low reinforcement ratios like slabs and that is why it is replaced with a new equation. The equation provided by Branson is given by:

\[
I_e = I_g \left( \frac{M_{cr}}{M} \right)^3 + I_{cr} \left( 1 - \left( \frac{M_{cr}}{M} \right)^3 \right) \leq I_g \quad \text{when } M_a \geq M_{cr} \tag{2.6}
\]

\[
I_e = I_g \quad \text{when } M_a < M_{cr} \text{ and } M_{cr} = \frac{E I_g}{y_i}
\]
where,

\[ I_g = \text{Second Moment of Inertia of the Gross Section (Ignoring the Reinforcement)} \]

\[ M_c = \text{Cracking Moment} \]

\[ M = \text{Applied Moment} \]

\[ I_{cr} = \text{Cracked Moment of Inertia} \]

\[ f_r = \text{Rupture Modulus of the Section} \]

\[ y_r = \text{Distance from the Section Centroid to the Extreme Tension Fiber} \]

**1.2) (Compression steel effect on Long Term Deflections) [18]:**

Branson [18] introduced a long-term deflection multiplier to calculate additional long-term deflection caused by creep and shrinkage which is included in ACI 318. The long-term deflection multiplier from Branson is given by the following formula.

\[
\lambda = \frac{\xi}{(1+50\rho')} \tag{2.7}
\]

where,

\[ \xi = \text{Time Dependent Factor for Sustained Loads (shown in Figure 2.5)} \]

\[ \rho' = \text{Compression Bar Ratio} \]

![Figure 2.5 Time Dependent Factor for Sustained Loads vs Time [19]](image)

One of the limitations of this equation comes from the time dependent factor for sustained loads. For the years greater than 5, this factor is set to be 2. For a long-term deflection, which is a function of time, it is reasonable to expect that the factor should also increase with time.
1.3) (Deformation of concrete structures) [20];

Branson [20] proposed long-term deflection as the sum of creep and shrinkage deflection. The creep and shrinkage deflection are given in Equation 2.8.

\[ \delta_{cr} = \lambda_c \delta_{sust}, \quad \delta_{sh} = k_{sh} \varphi \cdot l^2 \]  

(2.8)

where,

\[ \delta_{sust} = \text{Initial Deflection Due to Sustained Load} \]

\[ \lambda_c = \text{Long-term Deflection Multiplier} \]

\[ k_{sh} = \text{Shrinkage Deflection Constant} \]

\[ \varphi = \text{Cross Section Curvature} \]

\[ l = \text{Span Length} \]

ACI 435R-95 has incorporated and recommends it when part of the live load is considered as sustained load.

2) Bischoff (Reevaluation of Deflection Prediction for concrete Beams reinforced with Steel and Fiber Reinforced Polymer Bars) [16]:

Bischoff [16] proposed a new equation for calculation of the effective moment of inertia because the equation from Branson [17] (Equation 2.6) has been shown that it underestimates deflections for members with low flexural reinforcement ratio like slabs. Bischoff also added that Equation 2.6 has limited range of applicability, with 1%-2% of tensile reinforcement ratio. Especially for elements with tensile reinforcement ratio of less than 1%, the equation underestimates the immediate deflection and thus the equation shall be updated. This conclusion is also supported by various scholars. The equation is incorporated in ACI 318-19 [3], and is given by,

\[ I_e = \frac{I_{cr}}{1 - \left( \frac{2M_{cr}}{M_a} \right) \left( \frac{k_{cr}}{l_f} \right)} \text{, when } M_a > \frac{2}{3} M_{cr} \]  

(2.9)

\[ I_e - I_g \text{, when } M_a \leq \frac{2}{3} M_{cr}, \]

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where,

\( I_e \) = Second Moment of Inertia of the Gross Section (Ignoring the Reinforcement)  
\( M_{cr} \) = Cracking Moment  
\( M \) = Applied Moment  
\( I_{cr} \) = Cracked Moment of Inertia  
\( f_c \) = Rupture Modulus of the Section

3) Gilbert (Discussion of Reevaluation of Deflection Prediction for concrete Beams reinforced with Steel and Fiber Reinforced Polymer Bars 2006) [21]:

This is a commentary and verification work done for Equation 2.9. Gilbert conducted a series of tests for verifying and used eight simply supported singly reinforced one-way slabs with tensile reinforcement ratio ranging from 0.0018 to 0.01. He compared Equation 2.6, Equation 2.9, and actual deflections. In all the cases, he found that Equation 2.6 underestimates deflection and works fine with higher reinforcement ratio. In contrast, Equation 2.9 goes well with measured deflection. He strongly urged that various code publications include Equation 2.9.

4) Bischoff (Rational Model for Calculating deflection of reinforced concrete Beams and Slabs 2007) [22]:

This paper presents as to why Branson’s equation [17] (Equation 2.6) is not applicable to all reinforced concrete elements with varying reinforcement ratio. Branson’s equation [17] (Equation 2.6) worked well for elements with uncracked to cracked moment of inertia (\( I_e/I_{cr} \)) between 2 or 3. In addition, Branson’s equation overestimates stiffness for members with \( I_e/I_{cr} \) greater than 3 and this results in underestimation of deflection. This paper presents a rational approach that can work well for all kinds of reinforcements and FRP reinforced concrete. The equation is given by:

\[
I_e = \frac{I_{cr}}{1 - \eta \left( \frac{M_{ef}}{M_a} \right)^2} \leq I_g, \quad (2.10)
\]

\[
\eta = 1 - \frac{I_{cr}}{I_g}
\]

where,

\( I_g \) = Second Moment of inertia of the gross section (ignoring the reinforcement),
M_{cr} = Cracking Moment
M = Applied Moment
I_{cr} = Cracked Moment of Inertia
f_{r} = Rupture Modulus of the Section

5) Mari, A., Bairan, J., and Duarte, N. (Long Term Deflections in Cracked Reinforced Concrete Flexural Members 2009) [23]:

Mari, Bairan and Duarte [23] studied the long-term deflection of cracked reinforced concrete flexural elements. The authors also have proposed two equations for estimating the long-term deflection by integrating the curvature of the beam along its length. The equations developed work for beams, one-way slabs regardless of support condition. Even though the equations developed are intended for rectangular sections, it works for T and double T sections. Compared to the ACI318 method the ACI method has resulted in deflections in a more scatter manner.

If the elements are not cracked, meaning if the applied moment is less than the cracking moment of the section, the total long-term deflection is obtained by Equation 2.11.

\[ y = y_{g} + \Delta y_{g} = y_{g}(1 + \psi) \]  \hspace{1cm} (2.11)

If the section is cracked with the most critical service load combination, the long-term deflection is obtained by Equation 2.12.

\[ \Delta y_{g} = \Delta y_{cr} + \Delta y_{sh} \]  \hspace{1cm} (2.12)

Where,
\[ y_{g} = \text{Instantaneous Deflection due to Permanent Loads} \]
\[ \Delta y_{g} = \text{Delayed Deflection} \]
\[ \Delta y_{cr} = \text{Delayed Deflection due to Creep as given on the paper} \]
\[ \Delta y_{sh} = \text{Delayed Deflection due to Shrinkage as given on the paper} \]
\[ \psi = \text{Curvature at } t > t_{0} \]

Because it is hard to obtain an accurate estimation of moments for two-way slabs using a spreadsheet, it will be difficult to apply it for two-way slabs and has applicability limitation. It is also not possible to compare and plot these equations with other equations on a spreadsheet.
6) Scanlon and Supernant. (Estimating two-way Slab Deflections) [24]:

Scanlon and Supernant [24] provided an alternative formula for the additional long term deflection multiplier caused by creep and shrinkage. The accuracy of the formula has been tested on data from Gilbert and Guo [25], which comprises test results of deflection of seven two-way slabs. The formula is based on ACI 209 [26] creep coefficient, a correction accounting for concrete age loading and the creep factor provided by ACI 318 [27]. The proposed formula is given in Equation 2.13:

\[
\lambda = \frac{(t_i - t_j)^{0.6}}{10 + (t_i - t_j)^{0.6}} \cdot k_j \cdot \frac{\xi}{(1 + 50\rho')}
\]

(2.13)

where,

- \( t_i \)=Age of Concrete When Deflection is Calculated [28]
- \( t_j \)=Age of Concrete When Load is Applied [28]
- \( k_j \)=Factor Accounting Concrete Age at Loading,
  \[ = 2.3 \cdot t_j - 0.25 \leq 1.75 \]
- \( \xi \)=Time Dependent Factor for Sustained Loads (from ACI 318) [27]
- \( \rho' \)=Compression Bar Ratio

The intention of this research was to predict the long-term deflection of two-way slabs which are subjected to construction loads. As there is frequent loading and unloading in construction phase especially with shoring and un-shoring of scaffolds, the loads during this stage are usually equal or sometimes surpass the service loads. The proposed model has resulted a reasonable agreement with the test data provided from test program conducted by Gilbert and Guo [25]. A comparison of this formula with other proposed equations has been provided in the following sections.

7) Gribniak, Cervenka, and Kaklauskas (Long-term deflections of reinforced concrete elements: accuracy analysis of predictions by different methods) [29]:

Gribniak, Cervenka, and Kaklauskas [29] compared ACI 318 [27], ACI 435R-95 [30], Eurocode 2 and the Russian Building Code based on the accuracy of long-term deflection prediction using test data from 322 reinforced concrete members with 27 test programs listed on various literatures. Thus, as per ACI 435R-95 [30], the long-term deflection is the sum of
deflections from creep and shrinkage which are obtained from a separate two formulas. The long-
term deflection caused by creep is given by,

$$\delta_{cr} = \lambda_c \cdot \delta_{sust}$$  \hspace{1cm} (2.14)

$$\lambda_c = 0.85 \cdot \varphi(t, t_o)/(1 + 50 \cdot \rho')$$

where,

$\lambda_c$=Modification Factor for Additional Deflection from Creep

$\delta_{sust}$=Immediate Deflection Due to Sustained Load

$\varphi(t, t_o)$=Creep Coefficient as Defined by ACI 209 2008

$\rho'$=Compression Reinforcement

After conducting various tests on ACI 318 [27] and ACI 435R-97 [30] modification factors
and prediction models for long term deflection, the researchers have concluded that even though
the methods lack consistency, the predictions from these models are safe when the applied moment
is equal or greater than twice of the cracking moment.

A comparison has been made for a plot of long-term deflection modification factor $\lambda$ against time, using slab data listed in Table 2.1.

<table>
<thead>
<tr>
<th>Table 2.1 Section and Material Properties of the Slab Used</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section and Material Properties</td>
</tr>
<tr>
<td>Width (b)</td>
</tr>
<tr>
<td>Thickness (h)</td>
</tr>
<tr>
<td>28-day compressive strength ($f_{c'}$)</td>
</tr>
<tr>
<td>Yield strength of steel ($f_y$)</td>
</tr>
<tr>
<td>Tension Reinforcement ($A_s$) (#5@12/o/c)</td>
</tr>
<tr>
<td>Compression Reinforcement ($A_{s'}$) (#5@12/o/c)</td>
</tr>
<tr>
<td>Effective Depth ($d$)</td>
</tr>
<tr>
<td>Concrete Cover</td>
</tr>
</tbody>
</table>
Figure 2.6 Comparison of $\lambda$ vs Time

The summary of parameters used by Gribniak et al. [29] Scalonon and Supernant [29] is provided in Table 2.2.

Table 2.2 Comparison Among Papers for Long-Term Deflection Modification Factor

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>28-day compressive strength</td>
<td>4,000-6,000 psi</td>
<td>2,379-8,974 psi</td>
</tr>
<tr>
<td>Tensile Bar Ratio</td>
<td>N/A</td>
<td>0.0016-0.0023</td>
</tr>
<tr>
<td>Support Condition</td>
<td>Simply supported, Cantilever, Continuous</td>
<td>Simply supported, Cantilever, Continuous</td>
</tr>
<tr>
<td>Time Range</td>
<td>1-1,825 days</td>
<td>1-2,853 days</td>
</tr>
<tr>
<td>Applicability</td>
<td>Two-way slabs</td>
<td>Beams, one-way slabs, two-way slabs</td>
</tr>
</tbody>
</table>

Figure 2.6 shows the plot for long term deflection modification factor ($\lambda$) vs time. The time considered is 30 years to match with the typical warranty period of the roof slab membranes. Except at the early periods of the analysis, the formula by Scanlon and Supernant is shown to provide higher long-term deflection modification factor than the other two formulas. But in the early period of the time considered (0-6 months), it has resulted lesser value than the ACI 318 $\lambda$ (deviating by -21% to -2.83%). For time 6 months to 30 years, this formula resulted in higher $\lambda$. 

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than the ACI 318 $\lambda$ (deviating by +4.3% to +26.58%). The formula from Gribniak is also less than ACI 318 $\lambda$ for the whole duration considered.

10) Walkup and Musselman (Effects of Service Load Loading On long term Deflection Multiplier) [31]:

Walkup and Musselman [31] studied the effect of service load on long-term deflection modification factor ($\lambda$) by varying applied moment to cracking moment ($M_s/M_{cr}$) ratio and $M_{sus}/M_{cr}$ ratios. The researchers used a single cross section of beam in which three of them are steel reinforced and the rest nine are glass fiber polymer reinforced. Each beam is 4 in. wide and 8 in. deep and the steel reinforced beams had two No. 3 reinforcement bars with clear cover of ¾ in. and none of the specimens had compression reinforcement or transverse shear reinforcement. Each of the sets of specimens were subjected to a varying applied moment to cracking moment ratio ($M_s/M_{cr}$) but a constant sustained to cracking moment ratio ($M_{sus}/M_{cr}$). This is to evaluate the effect of applied moment on long term deflection multiplier ($\lambda$).

After conducting several experiments on Glass fiber Reinforced Polymer concrete (GFRP) and steel reinforced concrete, for constant $M_{sus}/M_{cr}$ ratio, the long-term deflection multiplier decreases with increasing $M_s/M_{cr}$ ratio. This is because the sustained load influences the long-term deflection multiplier greater than the applied load.

11) Gribniak and Kaklauskas (Improving the Deflection Prediction Model from ACI 318, 2014) [32]:

Gribniak and Kaklauskas [32] proposed an alternative procedure for the immediate deflection model of reinforced concrete in lieu of the ACI318 model. Previous publications written by the same authors [33] showed that the ACI 318 method underestimates short term deflections for members with low reinforcement ratio like slab. For long term deflection, the ACI 318 method is highly dependent on load intensity and deflections near the cracking moment are underestimated.

The authors address that these discrepancies occur because ACI 318 does not consider shrinkage to calculate effective moment of inertia and thus immediate deflection. Additionally, the authors argue that even though shrinkage is considered as a long-term factor, it will impact cracking resistance of the concrete in the early days of loading of the concrete. Thus, it should be
considered in immediate deflection calculation. The proposed method inserts a fictitious moment into the effective moment of inertia calculation used by ACI 318 to reduce the cracking moment.

The model has been tested with 3,500 immediate and long-term deflections and the proposed equation is:

\[ M_{sh} = \varepsilon_{sh} \cdot S_{s,el} \cdot E_s \]  \hspace{1cm} (2.15)

Where,

- \( \varepsilon_{sh} \) = Shrinkage Strain as Defined by ACI 209 [26]
- \( S_{s,el} \) = First Moment of Area of the Reinforcement
- \( E_s \) = Elastic Modulus of Reinforcement

Although the proposed model provides adequate prediction for elements with low reinforcement ratio, the model does not fit well for elements with higher tensile reinforcement ratio.

12) Zaki (Predicting the Long-Term Deflection of Flexural Members Using Artificial Neural Networks 2018) [34]:

Zaki [34] proposed an equation for long-term modification using artificial neural network method. The equation contains the most critical parameters including creep coefficient, time, shrinkage, and compression bar ratio in the proposed equation. Nevertheless, it does not indicate where the creep coefficient and shrinkage strains are obtained. The author also concluded that the equation is too simplistic.

After comparing the proposed equations from the research works, the ACI 318 long term deflection modification is found to be conservative. Thus, it has been used for this thesis to determine the long-term deflection of the rooftop slab.
2.4. Feasibility Study on PV Systems

1) Lee, et al. (Economic feasibility study of Campus wide Photovoltaic Systems in New England [35]:

This paper presents the economic feasibility of an installed PV system in one of the buildings located at the University of New Haven (UNH). It compares the installed capacity of PV system on Celentano Hall, which is a residential Hall for undergraduate students at the university, with a predicted PV electric generation on the building. The total floor area of the building is 23,529.91 ft$^2$ (2,186 m$^2$) and the total panel area is 4736.12( ft$^2$). The life expectancy (service period) of the PV system was assumed to be 25 years.

According to the paper, the PV system will generate 82,800 kWh/year in its first year and the unit installation cost is around $4.29/W. Net Present Value (NPV), Internal Rate of Return (IRR), Simple Payback Period (SPBP), Discounted Payback Period (DPBP), Discounted Cash Flow (DCFy), Simple Cash Flow (SCFy) and profitability index (PI) were used as profitability indicating factors. The PI is calculated to be 1.28, which is greater than 1.0, indicating that the project is profitable. The NPV, IRR, SPBP, SCFy, DCFy are calculated to be $81, 996, 8.74%, 11 years, $360,290, $106,395, respectively. The project is found to generate a positive cash flow after a simple payback period of 11 years. The system is found to generate $360,000 by the end of its service life.

26 other buildings in the University campus were also assessed for feasibility study and of which none of them have a PV system installed at the time of the study. It was found that the average annual cost savings from PV electric generation on these buildings is $250,000, which is quite significant as compared to the annual electric spending by the University which was around $3 million. The payback periods of the proposed systems range from 8-12 years. For the 25-year service life period, assuming a constant rate of inflation for electric bills, these buildings would save $6.3 million.

The Celentano Hall was awarded for a government incentive called ZREC and the authors of the paper assumed that most of the buildings will be eligible for this incentive too. This has a huge impact on the profitability calculations of PV systems on the buildings and the conclusion of the research. Although the paper can serve as an initial tool for studying feasibility of buildings
with PV across universities in Connecticut, the feasibility of buildings that are not eligible for
government incentives remained unanswered on the paper.

2) **Karnam and Chang (A feasibility study for PV installations in higher education
institutions-A case study)** [36]:

This is about feasibility and profitability of installed PV systems at the University of New
Haven (UNH) and other 14 higher education institutions in the state of Connecticut. The Celentano
Hall PV installation was studied for its economic feasibility and the same procedure was followed
for other installations in different Universities.

It is found that by the year 2026, the PV installation at the UNH will start yielding a positive
cash flow. The study at the UNH found out that the PV installation is a profitable investment by
various profitability and economic indicators like NPV, IRR and payback period. The values for
these indicators are $121, 134, 9.19% and 10.5 years, respectively.

The paper can serve as a stepping board for decision makers in higher education institutions
to see the profitability of PV system. However, the study for profitability for other universities is
simplistic and can be taken as the setback of the study.
CHAPTER 3: METHODOLOGY

3.1 General Guideline

The basic engineering principle used in designing the required ballasts to resist uplift wind pressure (or force) is equilibrium. The weight from CMU blocks (ballast) are designed to counteract the uplift wind pressure. Determination of the basic wind speed is the first step to calculate the uplift pressure.

Various coefficients are combined with the basic wind speed to obtain uplift pressure. Dead loads from the weight of the PV as well as racking system are used to calculate the required ballast. The weight of a single panel is around 40 lb. for X-22-360, and the racking system has a weight of 8.4 lb. [37]. A single ballast which counteracts the uplift pressure has 34 lb. of weight. The dead loads are summed up and multiplied by the appropriate load factor in the Load Resistance and Factored Design (LRFD) load combination. The net load coming after the load combination was used to calculate the required ballast.

It is recommendable that the designer maintains uniformity with respect to ballast arrangement. Uniformity in ballast arrangement can decrease the time and effort required by PV installation contractors while placing ballast. Snow load is another part of the analysis as the roof is assumed to have an open exposure. Ground snow load along with various coefficients is combined to obtain the design snow load.

3.2 Uplift Wind Pressure Calculation and Ballast Design Using ASCE 7-16 and SEAOC-17

Designing the number of solar panels that can be placed on the slab area is performed as the first step of the study. ASCE 7-16 [15] section 29.4.3 guides designers to place panels with a minimum and maximum gap of 0.25 in. and 6.7 ft, respectively. The array must also be placed at a distance maximum of 2(h2-hp) or 4 ft from the edge of the rooftop slab, where h2 is height of a solar panel above the roof at the upper edge of the panel and hp is parapet height. An 8.2 in. gap between two consecutive rows of panels (arrays) is provided in this study. This matches the installation specifications of the panel product used (X-22-360) [37].

ASCE 7-16 [15] outlines the procedures to obtain the uplift wind pressure for various wind exposure categories. The first step according to the code is to determine the basic wind speed. The basic wind speed for a building in New York City under Risk Category II is 115 mph. Various coefficients are combined with the basic wind speed to obtain velocity pressure and uplift pressure. The number of ballasts to resist uplift wind pressure is computed by applying appropriate load
multipliers from load combination. ASCE 7-16 considers a suction (uplift) force as a negative value.

SEAOC-17 [12] as mentioned in Chapter 2 (Literature Review) is a further explanation and elaboration for ASCE 7-16 [15]. In the last parts of the manual, logarithmic equations to be used for calculating the nominal net pressure coefficient are provided. This can make a designer more accurate in determining the uplift pressure. The coefficients used for calculating uplift wind pressure and snow load are provided in Appendix-A.

Uplift pressure is calculated using Sections 4.1 and the recommendations provided in Sections 4.3.1 through 4.3.5 of SEAOC-17 [12]. Although, SEAOC-17 manual does not provide load combinations to calculate required ballast, the basic load combinations in ASCE 7-16 [15] can be referred. Figure 3.1 is a diagrammatic step-by-step procedure for uplift pressure and required ballast calculations using ASCE 7-16 and SEAOC-17.

Figure 3.1 Uplift Wind Calculation and Ballast Design Procedures Using ASCE 7-16 and SEAOC-17 [15, 12]
3.3 Uplift Wind Pressure Calculation and Ballast Design Using ASCE 7-05 and 2014 NYC Building Code

ASCE 7-05 [11] has three methods to determine the wind pressure on structural and non-structural elements. These elements are divided into Main Wind Force Resisting Systems (MWFRS) and Components and Cladding (C&C). PV panels are considered as components and cladding because it does not have impact on resisting wind force as a structural element of the building. Section 6.4 of ASCE 7-05 shows the simplified procedure, Section 6.5 elaborates the analytical procedure, and Section 6.6 addresses the wind tunnel procedure. For calculating wind pressure on component and cladding using simplified procedure, the code lists the following five major criteria. These are:

- The height of the building must be less than or equal to 60 ft.
- The building must be enclosed as defined in the code.
- The building must be regular in shape as defined in the code.
- The building must have a flat roof, gable roof with θ≤45° or a hip roof with θ≤27°.
- The building does not have response characteristics as defined in the code.

For this study, the analytic approach is employed because it is more applicable to the typical building height used. Additionally, this procedure is commonly used by designers. Like the simplified method, there are some criteria in this procedure, as shown below:

- The building must be regular in shape as defined in the code.
- The building must not have a response characteristic as defined in the code.

Assuming both conditions hold true, the typical building is analyzed using this approach. Equation 6-15 and Sections 6.5.4 through 6.7.5.1 are used to apply the appropriate coefficients and calculate velocity pressure. Figures from 6-19A through 6-19C in ASCE7-05 [11] are used to calculate the net pressure coefficient (Cn). Using the net pressure coefficient, uplift pressure was determined.

The 2014 New York City Building Code [14] allows designers to follow ASCE 7-05 [11] provisions to design wind pressure on PVs. It also provides two types of simplified wind load calculations. The first one is called the simplified design method I, which is for low rise buildings (with roof height less than 60 ft). The other one, which can be applicable to this study, is simplified method II. According to this method, the design wind pressure cannot be taken less than 30 psf
(both positive and negative), except at corners of the building with a width equivalent to 10 percent of the building’s width at its side. Additionally, the net wind pressure on component and cladding cannot be less than 45 psf, for buildings in between 200 ft and 300 ft. 40 psf for portion of the building in between 100 ft to 199 ft. The step-by-step illustrations of procedures from both codes are shown in Figure 3.2. The various coefficients used for calculating uplift wind pressure and snow loads using these two codes are shown in Appendix-A.

**Figure 3.2 Uplift Wind Calculation and Ballast Design Procedures Using ASCE 7-05 and 2014 NYC Building Code [11, 14]**
3.3 Building Specification

In this study, the building geometry was selected to be a typical commercial building structure with a 50 ft × 50 ft dimension and having a total of nine columns. A 6 ft distance is provided from the edge of the building to the edge of the PV array. The reinforced concrete slab of the building has a 10 in. thickness with a compressive strength of 5,000 psi and reinforced with #5 tension rebar at 12 in. spacing on center in both (longitudinal and transverse) directions. As a practical industry standard for the roof slab design, the clear span length between the columns is set to be equal to 23 ft in both ways. Table 3.1 shows the summary of material and section properties used for the slab analyzed.

Figure 3.3 graphically illustrates the layout of roof plan of the building with a newly installed PV system. This plan dimension is determined because it can be a representative rooftop floor plan across the state of New York for buildings with ballasted PV. In addition, in this study, three different building heights are considered to study the effect of height on uplift wind pressure. 80 ft, 60 ft and 40 ft building heights were investigated, and these heights represent 8, 6 and 4 story buildings, respectively.

<table>
<thead>
<tr>
<th>Table 3.1 Section and Material Properties of the Slab</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section /Material Property</td>
</tr>
<tr>
<td>-----------------------------</td>
</tr>
<tr>
<td>Thickness (h)</td>
</tr>
<tr>
<td>28-day compression strength of concrete</td>
</tr>
<tr>
<td>Yield strength of the steel</td>
</tr>
<tr>
<td>Tension Reinforcement (A_t) (#5 12o/c)</td>
</tr>
<tr>
<td>Compression reinforcement (A_c) (#5 12 o/c)</td>
</tr>
<tr>
<td>Effective Depth</td>
</tr>
<tr>
<td>Concrete Cover</td>
</tr>
</tbody>
</table>

3.4 PV Panel Specification

The PV panel used in the study is SunPower X-22-360 [37] panel which is widely used for a rooftop solar PV system. The dimensions are shown in Figure 3.4 below and each panel has a plan area of 17.57 ft² with a tilt angle of 10 deg. It should be noted that the panel tilt angle is an important factor in terms of uplift wind pressure calculations as well as efficiency of electricity generation.
Legend

- PV Panel

Panel Chord Length, \( L_p = 5.12 \text{ft} \)
Panel Width = 3.43 ft
Panel Area = 17.57 sq ft
Panel Weight = 41.0 lbs

- Pans for placing CMU Blocks (Ballast)

Weight = 8.14 lbs

- Corner Column
- Middle Column
- Edge Column

Figure 3.3 Typical Building Plan View
Figure 3.4 Dimensions of Panel Used [37]

Assuming there is a minimum distance of 0.25 in. between the panels and 8.2 in. distance between the array of the line of panels, the typical plan area of the building takes a total number of 63 panels with 7 rows and 9 columns. Table 3.2 summarizes the important panel dimensions and parameters according to the manufacture’s cut sheet [37].

Table 3.2 Parameters from Panel Product Data Sheet [37]

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel Chord Length, $L_p$</td>
<td>5.12 ft</td>
</tr>
<tr>
<td>Width of Panel</td>
<td>3.43 ft</td>
</tr>
<tr>
<td>Panel Area</td>
<td>17.57 ft$^2$</td>
</tr>
<tr>
<td>Panel to Panel Distance, $d_2$</td>
<td>8.20 in.</td>
</tr>
<tr>
<td>Panel Weight</td>
<td>41.0 lb</td>
</tr>
<tr>
<td>Racking Weight</td>
<td>8.14 lb</td>
</tr>
<tr>
<td>Solar Panel Base Height, $h_1$</td>
<td>0.1 ft</td>
</tr>
<tr>
<td>Solar Panel Top Height, $h_2$</td>
<td>0.9 ft</td>
</tr>
<tr>
<td>Panel Tilt Angle, $\phi$</td>
<td>10 deg.</td>
</tr>
</tbody>
</table>

3.5 Long-Term Deflection Analysis Procedures

3.5.1 Design engineering vs existing building structural evaluation

As compared to the procedures followed for conducting analysis and design of a new building, which is going to have a PV system on rooftop slab, the serviceability analysis of an existing building has different aspects. The load case along with magnitude and load pattern will
change for an existing building before and after installing the PV system. This is because the roof slab live load can be made zero (eliminated) and instead additional dead load from the PV system is added. In addition, differences in material strength need to be considered.

Equation 3.1 represents the formula used to model rooftop slab for long term deflection for designing a new building with PV while Equation 3.2 is for an existing building proposed to have PV system on the roof top slab.

\[
\Delta_{T LT} = (1 + \lambda_{30})(\Delta_{T DL} + \alpha\Delta_{LL}) + \Delta_{LL} \quad (3.1)
\]

\[
\Delta_{T LT} = (1 + \lambda_{t+30})(\Delta_{T DL} + \alpha\Delta_{LL}) + \Delta_{LL} \quad (3.2)
\]

where,

\(\Delta_{LL}\)=Immediate Deflection from Live Load

\(\Delta_{T DL}\)=Immediate Total Dead Load Deflection

\(\alpha\Delta_{LL}\)=Immediate Sustained Live Load Deflection

\(\alpha\)=Percentage of Live Load Considered as Sustained

\(=50\%\) [15]

\(\lambda_{30}\)=Long-Term Deflection Multiplier for New Building Design

\(\lambda_{t+30}\)=Long-Term Deflection Multiplier for Existing Building

Since the new building has not yet existed, the long-term deflection modification factor (i.e., represented as \(\lambda_{30}\)) is different than the one to be used for an existing building. Because the analysis period of this study to account for the warranty period of roof slab membranes is 30 years, the period of serviceability analysis for the new building design can be 30 years as well. Meanwhile, since the existing building has been serving for time period of \(t\), the analysis period can be made \(t + 30\) years. Thus, the deflection modification factor is also different.
3.5.2 Long term deflection analysis procedures in finite element modeling

In a typical reinforced concrete flat slab design, long term deflection can be a governing factor. To represent its actual behavior, this analysis is performed as non-linear cracked analysis using iteration procedure. Therefore, this study utilizes finite element modeling using SAFE by CSI America [38]. SAFE is widely used in concrete slab and footing design within structural engineering industry practice. There are three kinds of deflection analysis:

- Elastic (non-cracked) analysis,
- Immediate cracked analysis, and
- Long term deflection (cracked with creep and shrinkage).

The focus of this thesis work is on long term deflection (cracked with creep and shrinkage effect considered). Modeling and analyzing long term deflection on software is not limited to a single approach and designers can use various procedures to analyze the long-term deflection.

After sketching or importing and defining the layout of the slab including its thickness and material property, load patterns were defined both for immediate and long-term deflection. The load patterns used are dead load, live load and superimposed loads, with a self-weight multiplier equal to 1, 0, 0, respectively. The dead load is the self-weight of the structure while the superimposed load is the weight from Ballast, PV and the racking system. Load cases are created from the combinations of load patterns. In all the load cases, the software must be set for non-linear (cracked) analysis.

As shown in Equations 3.1 and 3.2, the long-term deflection is the function of immediate deflection from live load, total dead load, and sustained part of the live load. Designers use various percentages for dividing the live load into sustained and non-sustained parts. According to ASCE 7-16 [15] Equation CC 2-2, 50% of the live load is applied to sustained live load and remaining live load will be applied to non-sustained loads.

After setting the load combination, before running the model, it is important to set the reinforcement source from the finite element-based design. The designer can set the minimum tension and compression reinforcement ratios. Figure 3.5 describes analysis procedure for the long-term deflection in SAFE v20 [38]. Load case, reinforcement ratio definition as well as load application on the software are shown in Appendix-B.
3.3 Discussion on Pre-PV Finite Element Modeling

Before the building is analyzed for long-term deflection after PV placement, it is necessary to first analyze the Pre-PV long term deflection of the rooftop slab. This is to know the initial deflection and compare it with the post-PV placement condition. As per Table 1607.1 of 2014 NYC Building Code and Table 4.3-1 of ASCE 7-16 [15], the Pre-PV condition is analyzed with two live load conditions. The 100 psf is used to account if the rooftop slab is used for public purposes and 60 psf is used to account the usage of the roof space for private purposes.
The long-term deflections were found to be 1.05 in. and 0.83 in. for live loads of 100 psf and 60 psf, respectively. To compare the effects of PV system placement on the long-term deflection of the rooftop slab, the more conservative deflection (0.83 in.) was used. More discussion and comparison of results for the finite element modeling will be provided in Chapter 5 (Results) of this thesis. Figure 3.6 shows the deflected and non-deflected shape of the rooftop slab before the PV system is applied on the building when it is analyzed on finite element Modeling software (SAFE).

![Images of deflected and non-deflected shapes of the rooftop slab]

(a) Non-Deflected

(b) Deflected

Figure 3.6 Pre- PV SAFE Model of the Slab
3.4 Feasibility Analysis Procedures

As shown in Figure 3.7, calculating the electric energy generated from the PV system is the first step to study the economic feasibility. Since PVWATTS [39] does not require specific address of a building to calculate electric generated, it can be used to estimate the average electricity generated from a PV system installed on rooftop slab of a building in a city. It can be used by inserting the appropriate amount of system size on the website.

![Diagram](image)

**Figure 3.7 Procedures for Feasibility Study**

Table 3.3 shows the inputs used for estimating annual energy production using PVWATTS. It should be noted that the system is a grid connected system (not a standalone system) and thus
no battery backup is needed. Because the building is a commercial one, it is assumed that the energy produced from the PV system will not be enough to run the entire building.

**Table 3.3 Inputs Used for PVWATTS**

<table>
<thead>
<tr>
<th>PV System Information (Inputs)</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Module Type</td>
<td>Standard</td>
</tr>
<tr>
<td>Array Type</td>
<td>Roof Mounted</td>
</tr>
<tr>
<td>System Loss (default)</td>
<td>14.080</td>
</tr>
<tr>
<td>Tilt Angle</td>
<td>10 deg.</td>
</tr>
<tr>
<td>Azimuth Angle</td>
<td>0-360 deg.</td>
</tr>
<tr>
<td>Module Efficiency</td>
<td>22.2 %</td>
</tr>
<tr>
<td>Number of Panels</td>
<td>63</td>
</tr>
<tr>
<td>Area of Single Panel</td>
<td>17.57 ft$^2$</td>
</tr>
<tr>
<td>Total Array Area</td>
<td>1106.91 ft$^2$</td>
</tr>
<tr>
<td>Total Area of the Slab</td>
<td>2,500 ft$^2$</td>
</tr>
<tr>
<td>DC System Size</td>
<td>22.829 kW</td>
</tr>
<tr>
<td>GCR</td>
<td>0.443</td>
</tr>
<tr>
<td>Albedo</td>
<td>0.00</td>
</tr>
<tr>
<td>Inverter Efficiency</td>
<td>96%</td>
</tr>
</tbody>
</table>

Helioscope [40], on the other hand, is a very useful tool for calculating the annual estimated amount of electric energy produced from a specific location. It can take plan drawings so that it is possible to design PV system using a plan drawing (for non-existing buildings). Because it requires a specific location to estimate the annual power production, two locations, Manhattan (i.e., 244-250 west 49th Street) and Brooklyn (527 Grand Street), were selected. Since there are many high-rising buildings in Manhattan, the shading effect on PV systems placed on medium height buildings is assumed to be higher than the rest of boroughs of NYC. Thus, it is reasonable to assume the most conservative result will be obtained in Manhattan and Brooklyn is added for comparison purposes. In addition, these locations are added to verify the results from PV WATTS, which gives a rough estimate of electric generation of PV system installed on a building in a city. The results obtained are reasonably close and will be discussed more in Chapter 5.

To calculate the annual cash flow, annual expenses and incomes must be determined. Expenses like operation and maintenance cost, insurance cost, annual revenue from electric production, need to be calculated. Factors affecting these incomes and expenses must also be considered. General inflation rate as indicated by Consumer Price Index (CPI), fuel inflation rate,
electric inflation rate over the years, annual degradation rate of the system, various state and federal incentives, discount rate etc. are some of the parameters considered in this thesis work. The company manufacturing the product used in this thesis provides a warranty for defects associated with age and thus costs associated with age and resulting from system failure are assumed to be covered by the company.

In relation to incentives, the Investment Tax Credit (ITC) and Production Tax Credit (PTC) are currently in play. Even though these incentives have been available since 2006 (supposed to end in 2023), in August 2022 the congress extended it to 2033 with even more benefits for eligible PV projects. The ITC is a one-time tax credit which covers a base credit of 30% with additional 20% credit. The additional credit is added if the PV and other steel components are produced in the U.S and if the location of the project is in one of energy states, making the incentives from federal government to 50% of the initial installation cost [41]. Since the products and components used in this thesis work and its manufacturing company are based in the U.S., this project is eligible for the 10% additional credit. Moreover, since NYC is one of the energy states, with direct employment of 4% in energy sector and 1.4% directly employed in fuel [34], it is eligible for additional 10% on ITC, making the total ITC for this project to be 50%.

In addition, the Modified Accelerated Cost Recovery System (MACRS) is also considered as an incentive since it will provide PV system owners with depreciation tax relief. A straight-line method is used to calculate the annual depreciation cost. The Internal Revenue Service (IRS) states that for PV systems, the recovery period is 5 years with 10 % of the installation cost going to the first year and 20% will be assigned for the next 4 years.

The state of New York also has various incentives which are provided for commercial and residential property owners who are willing to go to solar PV. The state incentive includes a loan with a much minimum interest rate (2%) and with a loan repayment period of 10 years [42]. The state provides a sales tax relief, but New York City is not eligible for this benefit and thus it is not considered in the analysis. In addition, the state of New York has a cash reward for commercial and residential property owners who are willing to go solar, thus this project is eligible to $0.6/Watt cash reward since New York City is in Con Edion’s service territory [43]. The amounts of the benefits for this project will be discussed in Chapter 5.
The installation cost is one of the significant factors in the analysis of feasibility study. The cost of installation of a PV system per watt is provided by various institutes like the National Renewable Energy Laboratories (NREL), energy sage and Lawrence Berkley National Laboratories etc. As indicated by various publications from these institutes, the cost of installation per Watt is decreasing year-to-year [44]. The cost of installation reported is a turnkey cost which includes cost for design, permit, labor and various direct and indirect costs.

At the end of the service life of the PV system, it is assumed that the PV panels will be sold out and will be recycled and thus it has a salvage value. The decommissioning cost is associated with removal and detachment of the system from the rooftop slab.
CHAPTER 4: RESULT OF FINITE ELEMENT MODELING ANALYSIS

4.1 Line Dead Load Calculation and Modeling

After the quantity of the ballast required to resist uplift pressure is determined, the next step is to calculate the dead load that will be exerted on the slab by the CMU blocks, PV panel weight, racking system weight and the slab dead load itself. The total number of CMU blocks in a row of panels (array) times the weight of each block (34 lb) will give the total dead load across the row from CMU blocks. Dividing this with width of the slab minus clear distance from the edge to end of panel on both sides, will give the line dead load coming from Ballast weight. The weight of a single panel when converted to area load is determined to be 2.276 psf, and the racking weight is determined to be about 0.379 psf. Equation 4.1 is used to calculate the total weight from CMU blocks. Equation 4.2 is used to convert the weight from CMU blocks into line load. Equation 4.3 is used for calculating the line dead load from PV weight for edge arrays, which are in the first and last rows of the PV arrangement. To calculate the dead load for middle arrays, Equation 4.3 is multiplied by 2.

\[
W_{\text{total(CMU, row)}} = \text{No. of CMU on a single panel} \times (\text{No. of PV in a row} + 1) \times 34\text{lb} \quad (4.1)
\]

\[
\text{Line dead load from CMU blocks} = \frac{\text{Equation 4.1}}{\{\text{Slab width} - (2 \times \text{edge distance, d}_1)\}} \quad (4.2)
\]

\[
\text{Dead load}_{\text{line,PV weight}} = \frac{\text{No. of PV panel in a row} \times 17.57 \text{ ft}^2 \times \frac{2.276 \text{ psf}}{2}}{\{\text{Slab width} - (2 \times \text{edge distance, d}_1)\}} \quad (4.3)
\]

Similarly, the line dead load from the racking system is calculated as shown in Equation 4.4.

\[
\text{Line dead load from the racking system} = \frac{0.379 \text{ psf} \times \text{width of panel}}{2} \quad (4.4)
\]

Figure 4.1 shows the dead load application for FE modeling to analyze the long-term deflection.
4.2. Line Snow Load Calculation and Modeling

The snow load to be presented here is for design and stability check. In other words, it is to check if the structure can withstand the probable loads that will be exerted. Since snow load is also an area load, to apply it on a SAFE v20 software [38], it must be changed to line load. Equation 4.5 shows how to convert the area snow load for middle array into line snow load to be applied on the modeling software.

\[
\text{Line Snow Load} = \text{Area snow load calculated} \times 17.57 \text{ ft}^2/5.12 \text{ ft} \quad (4.5)
\]

To calculate the line snow load for the edge panels, which are in the first and last rows of the PV arrangement, Equation 4.5 is divided by 2. Figure A in Appendix-B shows snow load application in finite element modeling software SAFE v20.

4.3 Wind Line Load Calculation and Modeling

The wind load must also be applied on the finite element modeling, to perform stability and design checks. The line wind load that will be applied on the software is calculated in a similar manner as the snow load is calculated. Since it is an area load, to make it similar with other loads and apply it on the software, it must be converted to a line load. Equation 4.6 is used to convert it
into line load. Figure B in Appendix-B illustrates the application of wind line load on SAFEv20 [38].

\[
\text{Line Wind Load} = \text{Area wind load calculated} \times 17.57 \text{ ft}^2 / 5.12 \text{ ft} \quad (4.6)
\]

4.4 Load Cases

As described in Section 3.5, the load cases defined are the dead load, live load and superimposed load which are obtained from the basic load pattern definition. Additional load cases are the “Immediate dead load”, “Immediate total dead load”, “Immediate sustained”, “Immediate total load” and “Façade load” on the perimeter of the slab.

These load cases must be combined with the appropriate scale factor to obtain the load combination used for long term deflection analysis, which is discussed in the next section. Figure C in Appendix B shows the load cases used for this thesis.

4.5 Load Combination

The load combination used for calculating the required ballast are those from LRFD design model, which are already discussed in Chapter 3. These combinations target the strength requirement of the building. Three load combinations, which are comprised of load cases were created. The first one is “Immediate non-sustained”, using load cases called “Immediate total dead” and “Immediate sustained, which have a scale factor of +1 and -1.

The other load combination defined on the software is “Long term live load” for live load effects including the long-term effects. Load cases “Total dead load”, “Immediate sustained” and “Immediate total dead load” with respective scale factor is added. Figure D in Appendix-B shows the load combinations used for this thesis.

4.6 Analysis Option and Reinforcement Ratio

Before running the model on the software, a reinforcement ratio has been inserted based on the previously defined slab information in Chapter 3. A cracked analysis option has been selected. A tension ratio of 0.0031 has been inserted. The reinforcement ratio source and ratio definition is shown in Figure E in Appendix-B.
CHAPTER 5: RESULTS

In this chapter, the results from calculations for velocity pressure, uplift wind pressure, required CMU blocks and long-term deflections are presented. A hypothetical building with specifications described in Chapter 3 with three different heights of 80 ft, 60 ft and 40 ft is considered. For 80 ft building, ASCE 7-05 [11], ASCE 7-16 [15], 2014 New York City Building Code [14] and SEAOC-17 [12] are considered as design specifications for comparison purposes. ASCE 7-16 is used for 60 ft and 40 ft building heights.

5.1 Velocity Pressure Results

5.1.1 80 ft tall building

As described in Chapter 3, the first step of calculating the uplift pressure is determining the velocity pressure (the wind pressure without nominal uplift coefficients). Location and height of the building are some of the factors that affect the value of the velocity pressure. Table 5.1 summarizes the velocity pressure for the typical building used in this thesis for height equal to 80 ft.

As shown in Table 5.1, the wind velocity pressure is increased in ASCE 7-16 [15] and SEAOC-17 [12]. This is because the basic wind speed is increased in ASCE 7-16. The 2014 NYC building code [14] states that the basic wind speed for a building is equal to 98 mph while ASCE 7-16 states that for a building in New York City under Risk Category II, the basic wind velocity is 115 mph. In addition, one can notice that the basic wind speed is set to be the same for all buildings despite risk category variation in ASCE 7-05 and 2014 NYC building code [11, 14]. Although the basic wind speed and velocity pressure are increased in ASCE 7-16, the uplift pressure on PV systems is lower than uplift pressure obtained from ASCE 7-05 and 2014 NYC building code. Further explanation on the difference between these codes has been discussed in Chapter 2.

The variation in velocity pressure across exposure class is caused by the change in velocity pressure coefficient ($k_v$). For wind exposure classes B, C and D, the velocity pressure coefficient is 0.93, 1.21 and 1.38, respectively.
Table 5.1 Velocity Pressure for 80 ft Tall Building

<table>
<thead>
<tr>
<th>Building Codes</th>
<th>Velocity Pressure (psf/panel)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Exposure Class B</td>
</tr>
<tr>
<td>ASCE 7-16 [15]</td>
<td>26.76</td>
</tr>
<tr>
<td>SEAOC -17 [12]</td>
<td>26.76</td>
</tr>
</tbody>
</table>

5.1.2 60 ft tall building
Compared to the velocity pressure calculated for 80 ft tall building, the velocity pressure for 60 ft building is less. This is because the velocity pressure coefficient ($k_v$) is now lowered to 0.85, 1.13 and 1.31 for exposure classes of B, C and D, respectively. Table 5.2 presents the velocity pressure for a typical building with a 60 ft height.

Table 5.2 Velocity Pressure for 60 ft Tall Building

<table>
<thead>
<tr>
<th>Building Codes</th>
<th>Velocity Pressure (psf/panel)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Exposure Class B</td>
</tr>
<tr>
<td>ASCE 7-16 [15]</td>
<td>24.46</td>
</tr>
</tbody>
</table>

5.1.3 40 ft tall building
For comparison purpose, the 40 ft building height is also considered. Table 5.3 shows the velocity pressure of the typical building for wind exposure class B, C and D. As compared to the other two cases discussed above, the velocity pressure for this case is lower. Velocity pressure coefficient ($k_v$) has a direct relation with height and will reduce as height is reduced.

Table 5.3 Velocity Pressure for 40 ft Tall Building

<table>
<thead>
<tr>
<th>Building Codes</th>
<th>Velocity Pressure (psf/panel)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Exposure Class B</td>
</tr>
<tr>
<td>ASCE 7-16 [15]</td>
<td>21.87</td>
</tr>
</tbody>
</table>
5.2 Uplift Pressure Results

5.2.1 80 ft tall building

After the velocity pressure and the appropriate uplift pressure coefficients are determined, the next step is to calculate uplift pressure. As discussed in Chapter 2, the 2014 New York City Building Code and ASCE 7-05 [14, 11] do not have an edge factor and the equations from these codes provide a uniform result for interior and exterior panels, which does not match to what has been found in various tests [12].

In the recommendation part of SEAOC-17 [12], as shown in Chapter 2, the edge factor can be calculated as 1.0 but due to practical purposes (to prevent sag or connection problem for middle panels and to be more conservative for the deflection analysis), an edge factor of 1.5 has been used on perimeter panels. Even with this being applied to the rooftop slab, the long-term deflection decreased from its initial value (Pre-PV condition), and this will be discussed more in the coming sections of this chapter. Tables 5.4 through 5.6 provide uplift pressure on interior and exterior panels using various specifications. Figure 5.1 is a figurative illustration of these results. The uplift pressure on exterior panels calculated using ASCE 7-16 [15] and SEAOC-17 [12] is 33% greater than uplift pressure on interior panels calculated using similar code. Moreover, uplift pressure results from ASCE 7-05 [11] and 2014 New York City Building code [14] are 8.95% greater than uplift pressure on exterior panels calculated using ASCE 7-16 and SEAOC-17. Similarly uplift pressure calculated using ASCE 7-05 and 2014 New York City Building code is 39% greater than interior panels calculated using ASCE 7-16 and SEAOC-17.

### Table 5.4 Uplift Pressure on Exterior Panels (80 ft, ASCE 7-16 and SEAOC-17)

<table>
<thead>
<tr>
<th>Building Codes</th>
<th>Exposure Class B</th>
<th>Exposure Class C</th>
<th>Exposure Class D</th>
</tr>
</thead>
</table>

46
Table 5.5 Uplift Pressure on Interior Panels (80 ft, ASCE 7-16 and SEAOC-17)

<table>
<thead>
<tr>
<th>Building Codes</th>
<th>Exposure Class B</th>
<th>Exposure Class C</th>
<th>Exposure Class D</th>
</tr>
</thead>
<tbody>
<tr>
<td>SEAOC -17 [12]</td>
<td>11.03</td>
<td>14.35</td>
<td>16.37</td>
</tr>
</tbody>
</table>

Table 5.6 Uplift Pressure on Interior and Exterior Panels (80 ft, ASCE 7-05 and 2014 New York City Building Code)

<table>
<thead>
<tr>
<th>Building Codes</th>
<th>Exposure Class B</th>
<th>Exposure Class C</th>
<th>Exposure Class D</th>
</tr>
</thead>
</table>

5.2.2 60 ft tall building
The uplift pressure for the 60 ft building height is calculated using ASCE 7-16 [15]. Since exposure class D has the lowest wind obstruction, it has the highest velocity pressure and consequently the highest uplift pressure. Tables 5.7 and 5.8 show the uplift pressure on exterior and interior panels for 60 ft building. As shown in Figure 5.2, the exterior panels experience a 33% higher uplift pressure than the pressure on the interior panels.

Table 5.7 Uplift Pressure on Exterior Panels (60ft, ASCE7-16)

<table>
<thead>
<tr>
<th>Building Codes</th>
<th>Exposure Class B</th>
<th>Exposure Class C</th>
<th>Exposure Class D</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASCE 7-16 [15]</td>
<td>14.10</td>
<td>18.75</td>
<td>21.73</td>
</tr>
</tbody>
</table>
### Table 5.8 Uplift Pressure on Interior Panels (60 ft, ASCE7-16)

<table>
<thead>
<tr>
<th>Building Codes</th>
<th>Exposure Class B</th>
<th>Exposure Class C</th>
<th>Exposure Class D</th>
</tr>
</thead>
</table>

![Bar chart showing uplift pressure for different building codes and exposure classes.](chart)

(a) Exterior Panels

![Bar chart showing uplift pressure for different building codes and exposure classes.](chart)

(b) Interior Panels

**Figure 5.1 Uplift Pressure for 80 ft**
5.2.3 40 ft tall building

Since uplift pressure is dependent on height, the 40 ft height has resulted in the lowest uplift pressure calculation as compared to the other two cases. Tables 5.9 and 5.10 present the uplift pressure for exterior and interior panels. Figure 5.3 is the graphical illustration of these results.

![Figure 5.2 Uplift Pressure for 60 ft](image)

**Table 5.9 Uplift Pressure for Exterior Panels (40 ft, ASCE7-16)**

<table>
<thead>
<tr>
<th>Building Codes</th>
<th>Exposure Class B</th>
<th>Exposure Class C</th>
<th>Exposure Class D</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASCE 7-16 [15]</td>
<td>11.37</td>
<td>15.56</td>
<td>18.25</td>
</tr>
</tbody>
</table>

**Table 5.10 Uplift Pressure for Interior Panels (40 ft, ASCE7-16)**

<table>
<thead>
<tr>
<th>Building Codes</th>
<th>Exposure Class B</th>
<th>Exposure Class C</th>
<th>Exposure Class D</th>
</tr>
</thead>
</table>
5.3 CMU Block

5.3.1 80 ft tall building

The ballast number calculation follows from uplift wind pressure. Basic load combinations in respective codes have been used to calculate CMU blocks. The ballast number will then be shown on working drawings that will be handed to the PV contractor. Tables 5.11 and 5.12 present the number of ballasts for exterior and interior panels using ASCE 7-16 [15] and SEAOC-17 [12]. Table 5.13 shows the number of ballasts for interior and exterior panels using ASCE 7-05 [11] and 2014 New York City building code [12]. Figure 5.4 demonstrates the results from the tables. CMU blocks for exterior panels calculated using ASCE 7-16 and SEAOC-17 are 37% greater than CMU blocks for interior panels calculated using the same case.

**Table 5.11 Number of CMU Blocks for Exterior Panels (80 ft, ASCE7-16 a SEAOC-17)**

<table>
<thead>
<tr>
<th>Building Codes</th>
<th>Exposure Class B</th>
<th>Exposure Class C</th>
<th>Exposure Class D</th>
</tr>
</thead>
<tbody>
<tr>
<td>SEAOC -17 [12]</td>
<td>7.12</td>
<td>9.66</td>
<td>11.20</td>
</tr>
</tbody>
</table>
Table 5.12 Number of CMU Blocks for Interior Panels (80 ft, ASCE 7-16 & SEAOC-17)

<table>
<thead>
<tr>
<th>Building Codes</th>
<th>Exposure Class B</th>
<th>Exposure Class C</th>
<th>Exposure Class D</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASCE 7-16 [15]</td>
<td>4.31</td>
<td>6.01</td>
<td>7.03</td>
</tr>
<tr>
<td>SEAOC -17 [12]</td>
<td>4.31</td>
<td>6.01</td>
<td>7.03</td>
</tr>
</tbody>
</table>

*** Note: The standard ballast pan can take up to eight ballasts at a time. However, a confirmation of technical support staff of the product used for this thesis has confirmed that an adjustment (modification) can be made to the pan to accommodate additional ballast (CMU Blocks).

Table 5.13 Number of CMU Blocks for Interior Panels (80 ft, ASCE7-05 & 2014 New York City Building Code)

<table>
<thead>
<tr>
<th>Building Codes</th>
<th>Ballast Number (Exterior and Interior Panels, piece/panel)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Exposure Class B</td>
</tr>
<tr>
<td>2014 NYC Building Code</td>
<td>13.50</td>
</tr>
</tbody>
</table>

Figure 5.5 shows the layouts for CMU blocks on roof top slab along with it’s sample calculation using various specifications and codes. The results provided in the layouts are rounded up to the nearest whole number. The sample calculation for ASCE 7-16 [15] Exposure Class B is shown below (all line loads in other exposure follow the same procedure).

Length of Panel=5.12 ft  Weight of 1 panel=2.33 psf  Weight of 1 Ballst=34 lbs
Width of Panel=3.43 ft  Weight of Racking System=0.379 psf

A) Axis A=Axis I

Total Number of Ballast=40
Line Dead Load on Axis A=

\[(40 \times 34/38) + (1.851 \times 7 \times 17.55/2)/38 + (0.379 \times 3.43/2) = 0.040 \text{ k/ft}\]
Line Snow Load = (12.60 psf × cos 10 × 17.55/2)/5.12 = 0.0212 k/ft
Line Wind Load = (16.55 psf × Cos 10 × 17.55/2)/5.12 = 0.02779 k/ft

- Wind load is different for exterior and interior panels based on edge factor
- Axis B = Axis C = Axis G = Axis H
  - Total number of ballast = 58
  - Line Dead load Across Axis B = (58 × 34/38) + (1.851 × 7 × 17.55/38) + (0.379 × 3.43) = 0.060 k/ft
  - Line Snow Load = (12.60 psf × Cos 10 × 17.55)/5.12 = 0.0424 k/ft
  - Line Wind Load = (16.55 psf × Cos 10 × 17.55/2)/5.12 = 0.056 k/ft
- Axis D = E = F
  - Total number of ballast = 52
  - Line Dead load Across Axis B = (52 × 34/38) + (1.851 × 7 × 17.55/38) + (0.379 × 3.43) = 0.055 k/ft
  - Line Snow Load = (12.60 psf × Cos 10 × 17.55)/5.12 = 0.0424 k/ft
  - Line Wind Load = (11.03 psf × Cos 10 × 17.55/2)/5.12 = 0.0372 k/ft

(a) Exterior Panels
Figure 5.4 Number of CMU Blocks (pieces/panel), 80ft
(b) Exposure Class C
Figure 5.5 Ballast Arrangement Layout (80 ft, ASCE 7-16)

The layout for other codes and specifications and other building heights are provided in Appendix-C.

5.3.2 60 ft tall building

The number of CMU blocks calculated for exterior panels have decreased by 17.41%, 14.69% and 12.85% for exposure classes of B, C and D as compared to the results of 80 ft building calculated using the same specification. Similarly, CMU blocks for interior panels have decreased by 19.0%, 15.80% and 13.5% for exposure classes B, C and D as compared to the 80 ft case. Tables 5.14 and 5.15 present the CMU blocks calculated for 80 ft building. Figure 5.6 is the graphical representation of these results.
Table 5.14 Number of CMU Blocks for Exterior Panels (60 ft, ASCE7-16)

<table>
<thead>
<tr>
<th>Building Codes</th>
<th>Exposure Class B</th>
<th>Exposure Class C</th>
<th>Exposure Class D</th>
</tr>
</thead>
</table>

Table 5.15 Number of CMU Blocks for Interior Panels (60 ft, ASCE 7-16)

<table>
<thead>
<tr>
<th>Building Codes</th>
<th>Exposure Class B</th>
<th>Exposure Class C</th>
<th>Exposure Class D</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASCE 7-16 [15]</td>
<td>3.49</td>
<td>5.06</td>
<td>6.08</td>
</tr>
</tbody>
</table>

![Figure 5.6 Number of CMU Blocks (pieces/panel) (60 ft, ASCE 7-16)](image)

5.3.3 40 ft tall building

As compared to the previous two cases, the number of CMU blocks calculated for the 40 ft building is lower. This is due to the reduction of uplift pressure. Tables 5.16 and 5.17 and Figure 5.7 show the number of CMU blocks (Ballasts) calculated to resist uplift pressure on interior and exterior panels.
### Table 5.16 Number of CMU Blocks for Exterior Panels (40 ft, ASCE 7-16)

<table>
<thead>
<tr>
<th>Building Codes</th>
<th>Exposure Class B</th>
<th>Exposure Class C</th>
<th>Exposure Class D</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASCE 7-16 [15]</td>
<td>5</td>
<td>7</td>
<td>8</td>
</tr>
</tbody>
</table>

### Table 5.17 Number of CMU Blocks for Interior Panels (40 ft, ASCE 7-16)

<table>
<thead>
<tr>
<th>Building Codes</th>
<th>Exposure Class B</th>
<th>Exposure Class C</th>
<th>Exposure Class D</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASCE 7-16 [15]</td>
<td>3</td>
<td>4</td>
<td>5</td>
</tr>
</tbody>
</table>

![Graph showing the number of CMU Blocks for Interior and Exterior Panels]

**Figure 5.7 Number of CMU Blocks (pieces/panel) (40ft, ASCE 7-16)**
5.4 Long Term Deflection Results

5.4.1 80 ft tall building

Building codes like the 2014 New York City building code [14] allow designers to use ballasted PV installations for rooftop slabs until the building height of 100 ft. Moreover, the most repetitive heights in New York City with ballasted PV systems are 80 ft, 60 ft and 40 ft. That is why the typical building in this thesis is analyzed using these heights. The initial (Pre-PV) condition is analyzed with a roof live load of 100 psf and 60 psf. For the 100 psf roof live load, without the PV system being applied on the slab, the long-term deflection is obtained to be 1.01 in.

The corresponding initial Pre-PV long term deflection for a live load of 60 psf is 0.83 in. The general trend of long-term deflection after PV installation shows decrements as compared to the initial condition. Tables 5.18 through 5.20 present the summary of results for long term deflection across wind exposure classes of B, C and D for building height of 80 ft.

<table>
<thead>
<tr>
<th>Wind Exposure Class</th>
<th>Long Term Deflection (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Exposure Class B</td>
<td>0.79</td>
</tr>
<tr>
<td>Exposure Class C</td>
<td>0.80</td>
</tr>
<tr>
<td>Exposure Class D</td>
<td>0.84</td>
</tr>
</tbody>
</table>

Table 5.19 Long Term Deflection (80 ft, ASCE 7-16)

<table>
<thead>
<tr>
<th>Wind Exposure Class</th>
<th>Long Term Deflection (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Exposure Class B</td>
<td>0.76</td>
</tr>
<tr>
<td>Exposure Class C</td>
<td>0.78</td>
</tr>
<tr>
<td>Exposure Class D</td>
<td>0.79</td>
</tr>
</tbody>
</table>
Table 5.20 Long Term Deflection (80 ft, SEAOC-17)

<table>
<thead>
<tr>
<th>Wind Exposure Class</th>
<th>Long Term Deflection (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Exposure Class B</td>
<td>0.76</td>
</tr>
<tr>
<td>Exposure Class C</td>
<td>0.77</td>
</tr>
<tr>
<td>Exposure Class D</td>
<td>0.78</td>
</tr>
</tbody>
</table>

5.4.2 60 ft tall building
The long-term deflections as compared to initial Pre-PV condition (60 psf roof live load condition) has decreased by 9.54%, 8.82% and 8.3% for wind exposure classes of B, C and D respectively. Table 5. 21 present the long-term deflection for 60 ft building.

Table 5.21 Long Term Deflection (60 ft, ASCE 7-16)

<table>
<thead>
<tr>
<th>Wind Exposure Class</th>
<th>Long Term Deflection (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Exposure Class B</td>
<td>0.75</td>
</tr>
<tr>
<td>Exposure Class C</td>
<td>0.76</td>
</tr>
<tr>
<td>Exposure Class D</td>
<td>0.76</td>
</tr>
</tbody>
</table>

5.4.3 40 ft tall building
As compared to the initial Pre-PV deflection, the long-term deflection for the 40 ft building case is reduced by 10.63%, 9.30% and 8.82 %. Table 5.22 illustrates the long-term deflection for 40 ft.
Table 5.22 Long Term Deflection (40 ft, ASCE7-16)

<table>
<thead>
<tr>
<th>Wind Exposure Class</th>
<th>Long Term Deflection (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Exposure Class B</td>
<td>0.74</td>
</tr>
<tr>
<td>Exposure Class C</td>
<td>0.76</td>
</tr>
<tr>
<td>Exposure Class D</td>
<td>0.76</td>
</tr>
</tbody>
</table>

5.5 Results and Discussion on Feasibility Analysis

5.5.1 Energy production estimate

As per the procedures described in Section 3.6 of this thesis work, the first step for the economic feasibility of ballasted PV system on rooftop is to calculate the amount of estimated electricity. Four different methods have been followed. The maximum electricity is estimated using PVWATTS [39], which is a government website. The result from PVWATTS is reasonably close when verified by another website called Helioscope at two locations in two different boroughs of New York city. A total of 24 different cases have been studied using Helioscope based on the direction in which the panels face. The two manual calculations also provide a fairly close result. The discussions and results are provided in subsequent sections.

5.5.1.1 PVWATTS

PVWATTS [39] is a website prepared by the National Renewable Energy Laboratories (NREL) to estimate annual energy production from PV systems. Since the website does not need a specific location (street address), it is ideal for getting a rough annual energy production estimate for a city. After inserting the inputs such as tilt angle, system size, panel efficiency and other factors, the results of the annual power production can be obtained. Since the PV system employed for this thesis work is a fixed one, eight different cases based on azimuth angle are formed. Ideally, for the northern hemisphere the PV system will yield maximum energy when it has an azimuth angle of 180 degrees facing south. But this can change due to the local weather pattern. For the compass headings listed on the website, the energy estimate is calculated as shown in Table 5.23. System loss from the combination of the PV panel and other system components is provided in Appendix-D.
Table 5.23 Annual Electric Production Estimate Using PVWATTS

<table>
<thead>
<tr>
<th>Azimuth Angle (deg.)</th>
<th>Heading</th>
<th>Annual production (kWh)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>N</td>
<td>22,978</td>
</tr>
<tr>
<td>45</td>
<td>NE</td>
<td>23,848</td>
</tr>
<tr>
<td>90</td>
<td>E</td>
<td>25,675</td>
</tr>
<tr>
<td>135</td>
<td>SE</td>
<td>27,342</td>
</tr>
<tr>
<td>180</td>
<td>S</td>
<td>27,952</td>
</tr>
<tr>
<td>225</td>
<td>SW</td>
<td>27,179</td>
</tr>
<tr>
<td>270</td>
<td>W</td>
<td>25,442</td>
</tr>
<tr>
<td>315</td>
<td>NW</td>
<td>23,677</td>
</tr>
</tbody>
</table>

As shown in Table 5.23 the maximum estimated electric energy for this specific system is 27,952 kWh annually and it is obtained when the azimuth angle is set to equal to 180 degrees. The monthly electric power production is shown in Figure 5.8. As shown in Figure 5.8 the maximum electric power is generated during May to July due to the increase in solar incident on solar panels.

Figure 5.8 Monthly Electric Energy Generated Using 180 Degrees Azimuth Angle
Inverter sizing is also completed by this website. Inverters are components of the PV system that change Direct current (DC) into alternate current (AC) and need to be sized so as work well with the system. Assuming DC-AC ratio of 1.2, the inverter is sized to be 19 kW.

5.5.1.2 Helioscope

Helioscope [40] is a very useful tool for estimating electric power generation. It enables designers to insert building plans and obtain potential electric generation from a building which is not yet constructed. As per the reasons indicated in Section 3.6, a location in Manhattan or Brooklyn was selected and the electricity generated from the system was estimated. Because the azimuth of the building is unknown and dependent on the direction in which the panel is facing and instead of guessing it, an automatic function of Helioscope to set azimuth has been used as per training videos and manuals from Helioscope. Since the direction to which the panels should be facing is not yet known, all the four faces of the building have been checked and the direction which resulted in maximum electric generation has been used to be compared with the results from PVWATTS [39]. Thus, this will result in a total of 24 cases. Tables 5.24 to 5.29 show the amount of energy estimated by Helioscope at the location and heights specified in Section 3.6.

<table>
<thead>
<tr>
<th>Panel Facing side</th>
<th>Array Azimuth (deg.)</th>
<th>Total Power to Grid (kWh)</th>
<th>Inverter size (kW)</th>
<th>DC-AC Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rear (Case-1)</td>
<td>204.87</td>
<td>25,825</td>
<td>18</td>
<td>1.12</td>
</tr>
<tr>
<td>Right (Case-2)</td>
<td>115.01</td>
<td>28,235</td>
<td>20</td>
<td>1.13</td>
</tr>
<tr>
<td>Front (Case-3)</td>
<td>25.01</td>
<td>25,498</td>
<td>20</td>
<td>1.13</td>
</tr>
<tr>
<td>Left (Case-4)</td>
<td>295.25</td>
<td>26,300</td>
<td>20</td>
<td>1.13</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Panel Facing side</th>
<th>Array Azimuth (deg.)</th>
<th>Total Power to Grid (kWh)</th>
<th>Inverter Size (kW)</th>
<th>DC-AC Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rear (Case-1)</td>
<td>209.23</td>
<td>29,326</td>
<td>20</td>
<td>1.13</td>
</tr>
<tr>
<td>Right (Case-2)</td>
<td>119.23</td>
<td>28,369</td>
<td>20</td>
<td>1.13</td>
</tr>
<tr>
<td>Front (Case-3)</td>
<td>29.69</td>
<td>25,599</td>
<td>20</td>
<td>1.13</td>
</tr>
<tr>
<td>Left (Case-4)</td>
<td>299.23</td>
<td>26,390</td>
<td>20</td>
<td>1.13</td>
</tr>
</tbody>
</table>
Table 5.26 Annual Power Production (40ft, Manhattan)

<table>
<thead>
<tr>
<th>Panel Facing side</th>
<th>Array Azimuth (deg.)</th>
<th>Total Power to Grid (kWh)</th>
<th>Inverter Size (kW)</th>
<th>DC-AC Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rear (Case-1)</td>
<td>209.93</td>
<td>29,311</td>
<td>20</td>
<td>1.13</td>
</tr>
<tr>
<td>Right (Case-2)</td>
<td>119.20</td>
<td>28,374</td>
<td>20</td>
<td>1.13</td>
</tr>
<tr>
<td>Front (Case-3)</td>
<td>30.22</td>
<td>25,611</td>
<td>20</td>
<td>1.13</td>
</tr>
<tr>
<td>Left (Case-4)</td>
<td>299.98</td>
<td>26,367</td>
<td>20</td>
<td>1.13</td>
</tr>
</tbody>
</table>

Tables 5.27 through 5.29 show the amount of electricity estimated by Helioscope when the typical building is in Brooklyn, NYC.

Table 5.27 Annual Power Production (80ft, Brooklyn)

<table>
<thead>
<tr>
<th>Panel Facing side</th>
<th>Array Azimuth (deg.)</th>
<th>Total Power to Grid (kWh)</th>
<th>Inverter Size (kW)</th>
<th>DC-AC Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rear (Case-1)</td>
<td>234.12</td>
<td>28,588</td>
<td>20</td>
<td>1.13</td>
</tr>
<tr>
<td>Right (Case-2)</td>
<td>144.24</td>
<td>29,184</td>
<td>20</td>
<td>1.13</td>
</tr>
<tr>
<td>Front (Case-3)</td>
<td>54.33</td>
<td>28,036</td>
<td>21.5</td>
<td>1.17</td>
</tr>
<tr>
<td>Left (Case-4)</td>
<td>324.11</td>
<td>25,707</td>
<td>20</td>
<td>1.13</td>
</tr>
</tbody>
</table>

Table 5.28 Annual Power Production (60ft, Brooklyn)

<table>
<thead>
<tr>
<th>Panel Facing side</th>
<th>Array Azimuth (deg.)</th>
<th>Total Power to Grid (kWh)</th>
<th>Inverter Size (kW)</th>
<th>DC-AC Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rear (Case-1)</td>
<td>199.57</td>
<td>29,718</td>
<td>20</td>
<td>1.13</td>
</tr>
<tr>
<td>Right (Case-2)</td>
<td>109.60</td>
<td>28,029</td>
<td>20</td>
<td>1.13</td>
</tr>
<tr>
<td>Front (Case-3)</td>
<td>19.26</td>
<td>25,404</td>
<td>20</td>
<td>1.13</td>
</tr>
<tr>
<td>Left (Case-4)</td>
<td>289.37</td>
<td>26,710</td>
<td>20</td>
<td>1.13</td>
</tr>
</tbody>
</table>

Table 5.29 Annual Power Production (40ft, Brooklyn)

<table>
<thead>
<tr>
<th>Panel Facing side</th>
<th>Array Azimuth (deg.)</th>
<th>Total Power to Grid (kWh)</th>
<th>Inverter Size (kW)</th>
<th>DC-AC Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rear (Case-1)</td>
<td>241.21</td>
<td>28,324</td>
<td>20</td>
<td>1.13</td>
</tr>
<tr>
<td>Right (Case-2)</td>
<td>151.24</td>
<td>29,320</td>
<td>20</td>
<td>1.13</td>
</tr>
<tr>
<td>Front (Case-3)</td>
<td>61.24</td>
<td>29,334</td>
<td>20</td>
<td>1.13</td>
</tr>
<tr>
<td>Left (Case-4)</td>
<td>61.24</td>
<td>29,334</td>
<td>20</td>
<td>1.13</td>
</tr>
</tbody>
</table>
5.5.1.3 Theoretical calculation

Equation 5.1 is considered to be a universal formula to estimate energy production from a PV system and is used by many references [30].

\[ E = A \cdot r \cdot h \cdot pr \]  \hspace{1cm} (5.1)

where,

\( E \) = Annual Energy Estimate,
\( A \) = Total Area of Panels in m\(^2\),
\( r \) = Panel Efficiency,
\( h \) = Annual Average Solar Radiation on Tilted Panels in kWh/m\(^2\),
\( pr \) = System Performance Ratio.

The annual average solar radiation for New York city is 3.93 kWh/m\(^2\)/day as per National Solar Radiation Data Base (NSRDB) [45] and other sources [46, 47]. The power incident on solar PV panel is dependent on solar radiation and the angle between the sun and PV. The power incident on the PV system will be maximum when the angle between the panel and sun is perpendicular. It can be calculated using the following formula.

\[ S_{\text{module}} = S_{\text{horizontal}} \cdot \frac{\sin(\alpha + \beta)}{\sin \alpha} \]  \hspace{1cm} (5.2)

where,

\( \alpha \) = Elevation angle
\( \beta \) = Tilt Angle
\( S_{\text{horizontal}} \) = Horizontal Incident (Average Solar Radiation Obtained for a City or a Location).

The tilt angle is already determined to be 10 degrees and the elevation angle \( \alpha \) can be determined using the equation below.

\[ \alpha = 90 - \varnothing + \delta \]  \hspace{1cm} (5.3)

where,
$\varnothing =$ The Latitude which is equal to 40.73 degrees for New York city [39]

$\delta =$ The Declination Angle which is Calculated Using Equation 5.4.

$$\delta = 23.45 \sin (360/365) \cdot (284 + d) \quad (5.4)$$

where,

$d =$ Specific Day in a Year

Performance ratio is the ratio between actual and theoretical energy generated by the PV system. For this thesis work the performance ratio is set equal to 0.75.

After inserting the required inputs, the estimated annual energy generated using the above shown formula is determined to be 25,011 kWh/year. This is an estimated amount of energy produced for the first year of installation. A degradation factor of 0.25% must be for accounting decrease in efficiency for accounting panel age [37]. The input data used for this formula is provided in Appendix-D.

Another formula which can be used to estimate the average annual electric power generated from a PV system is shown in Equation 5.5. This formula is obtained from a training course from continuing education and development [48].

$$\text{Energy Yield} = \text{Peak Sun Hours} \times \text{Module rated power} \times \text{Total Derating Factor} \quad (5.5)$$

The peak sun hours for New York city as per National Renewable energy Laboratories is 4.08 hrs. [45] and the module rated power for a single panel used in this thesis equals 360 Watts [37]. The module rated power is obtained from the product data sheet. This is a power that will be generated when the PV system is tested under standard testing conditions (STC). There will also be a power production tolerance of +/-5% and this has been considered. Thus, the energy yield after inserting all the inputs is calculated to be 24,064 kWh. Inputs used to estimate energy production using this equation are provided in Appendix-D of this thesis.

Although all the websites and formulas have merits and demerits, estimates from PVWATTS [39] are used for conducting feasibility study due to conservative approach followed.
In addition, the results from PVWATTS [39] and Helioscope [40] are reasonably close and hence verifying the accuracy of the estimation. The results from the two theoretical calculations are also close. Given the fact that PVWATTS website is designed by NREL and to be more conservative on the estimation, results from PVWATTS (27,952 kWh) is used for economic models used.

5.5.2 Results and discussion on economic feasibility tools

The profitability of a project is indicated by various economic tools and an investment for implementing the project into reality is checked for its worthiness. To calculate and find out these tools, various inputs must be determined first. The first step is to determine the annual energy production which has been discussed in the previous section. Another input is the installation cost which has been reported by various sources. For this thesis, a turnkey price has been assumed. The reported installation costs from NREL and various sources include prices for design, permit, direct and indirect costs and other administrative costs. The national average installation cost reported by NREL is $1.84 - $3.6/kW [49] while other independent sources like energy sage make the installation cost in New York City to be $3.21/kW [50]. The installation cost used for this thesis work is $3.21/kW and since the system is 22.7 kW, the total installation cost is calculated to be $72,867.0. NREL also estimates that annual insurance cost to be 25% of the annual operation and maintenance cost [51]. Table 5.30 lists the inputs and sources used in the feasibility study.

Various important data must also be determined to calculate the feasibility tools. The input data used in analyzing these tools is shown in Table 5.30. Data such as inflation rate, electricity inflation rate, fuel escalation rate are averaged after the data is being extracted. The sources from which this data is extracted is indicated in the references section of this thesis. The time considered for the analysis is 25 years. The ownership model assumed is a direct ownership where the owner purchases the system.
Table 5.30 Input Values Used for Feasibility Analysis

<table>
<thead>
<tr>
<th>Input</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>DC Nameplate Size</td>
<td>22.7 kW</td>
</tr>
<tr>
<td>Year 1 Electric Generated</td>
<td>27,952 kWh</td>
</tr>
<tr>
<td>Installation cost [50]</td>
<td>$72,867 ($3.21/kW)</td>
</tr>
<tr>
<td>Annual operation and maintenance cost [52] *</td>
<td>$23/kW/year</td>
</tr>
<tr>
<td>Annual Electric degradation rate [37]</td>
<td>0.25%</td>
</tr>
<tr>
<td>Average Electric inflation Rate [53]**</td>
<td>1.86%</td>
</tr>
<tr>
<td>Fuel Inflation Rate [54]**</td>
<td>6.33%</td>
</tr>
<tr>
<td>General Inflation Rate [55]</td>
<td>2.93%</td>
</tr>
<tr>
<td>Future Value Discount Rate [35, 56, 57]</td>
<td>5%</td>
</tr>
<tr>
<td>Decommissioning cost [58]</td>
<td>$23,061 ($300/kW)</td>
</tr>
<tr>
<td>Salvage value of PV [58]</td>
<td>$3,075</td>
</tr>
<tr>
<td>Investment tax credit (base and other benefits) [41]</td>
<td>50% of the installation cost</td>
</tr>
<tr>
<td>New York State Incentive (for Con-Edison’s Service territory) [43]</td>
<td>$13,620($0.6/W)</td>
</tr>
<tr>
<td>Leverage Rate [42]</td>
<td>50% of the installation cost</td>
</tr>
<tr>
<td>Debt Repayment period [42]</td>
<td>10 years</td>
</tr>
<tr>
<td>Interest rate of the debt [42]</td>
<td>2%</td>
</tr>
<tr>
<td>Current Electricity price for commercial [53]</td>
<td></td>
</tr>
<tr>
<td>Buildings in New York City</td>
<td>$0.19/kWh</td>
</tr>
<tr>
<td>Total Value to be claimed using Modified Accelerated Cost-Recovery System (MACRS) [59]</td>
<td>$65,580</td>
</tr>
</tbody>
</table>

*The literature has data from 2016 and adjustment is made to account for time value of money

** Source Data are averaged

Economic analysis is fundamentally dependent on the principle of time value of money. The economic feasibility indicators for PV system are Net Present Value, Internal Rate of Return, Simple payback period, Profitability Index, Simple Cash Flow and Discounted Cash Flow [30,31,33]. Thus, these indicators are used in this thesis.

5.5.2.1 Net Present Value (NPV)

The Net Present Value is one of the most widely used economic analysis tools to judge the financial profitability of a project. It is the difference between the present cash inflows and outflows [35, 56]. If a net present value of an investment is calculated to be positive, the project is worth investing on and vice versa.

\[
NPV = \sum_{n=0}^{N} \frac{c_n}{(1 + r)^n}
\]  

(5.6)
Where $C_n$ is the annual cash flow and is determined by Equation 5.7

$$C_n = C_{ener}(1 + r_e)^n - \{C_{op}(1 + r_l)^n + D\} \tag{5.7}$$

where,

$C_{ener}$= The Annual Energy and Other Revenue from Electric Resulting a Positive Income

$r_e$= The Rate of Inflation for Electricity

$C_{op}$= The Annual Operation and Maintenance Cost

$r_l$= The Inflation Rate. $D$ is the Annual Debt Repayment Given by Equation 5.8

$$D = C_{fd}(\frac{r_d}{1 - (1 + r_d)^{N'}}) \tag{5.8}$$

where,

$C$ = Total Initial Cost

$f_d$ = The Leverage Rate

$r_d$ = Interest Rate of Debt

$N'$ = Debt Repayment Period

After calculating the annual cash flow and the provided inputs from Table 5.29, the net present value of this project is calculated to be about $165,174.

5.5.2.2 Internal Rate of Return (IRR)

The Internal rate of return is an economic model which indicates the profitability of an investment by comparing it with the discount rate. The more the IRR is greater than the discount rate, the more favorable the investment on the project is.

The formula to determine the internal rate of return is shown in Equation 5.9.

$$0 = \sum_{n=0}^{N} \frac{C_n}{(1+IRR)^n} \tag{5.9}$$
The internal rate of return is determined to be 18%. This is more than 3 folds of the discount rate of 5% and this proves that the investment is favorable.

5.5.2.3 Simple Payback Period (SPBP)

The Simple Payback Period is the time in which the project will break even. In other words, it is a period of time in which the project starts to generate positive income. The early days of the investment project has more expenses than incomes and thus the annual cash flow is negative. The following is the formula that is used to calculate the simple payback period.

\[
SPBP = \frac{\text{Initial Investment}}{\text{Annual Savings}}
\]  

(5.10)

The Simple Payback Period for this specific project based on the inputs described above is calculated to be 4.75 years. As per a study by U.S Department of Energy in 2015 [60], solar systems on Multifamily buildings can pay for themselves 5-10 years at incentive levels at that time. Since the investment tax credit used for this project is 50% and the efficiency of the panel is high, the payback period obtained from the feasibility analysis is in line with the study from DOE.

5.5.2.4 Profitability Index (PI)

The Profitability Index is the ratio between the present value of future cash flows by the initial investment. An investment having a PI greater than 1.0 is considered as a profitable investment and projects with higher values of PI are seen as attractive to invest on. A PI of 3.26 was obtained for this system.

\[
PI = \frac{\text{NPV}}{\text{Initial Investment}} + 1
\]  

(5.11)

5.5.2.5 Discounted Cash Flow (DCF)

Discounted Cash Flow (DCF) analysis helps to determine the value of an investment based on the future cash flow it will generate. If the discounted cash flow is greater than the current initial investment, then the project is worth investing on. If the discounted cash flow is less than the investment, the project is not worthwhile. The following equation explains how to calculate the discounted cash flow. The DCF for this project is calculated to be about $135, 486.

\[
DCF = \frac{\text{Annual Cash Flow}}{(1+r)^n}
\]  

(5.12)
5.5.3 Feasibility analysis without incentives.

The incentives, both local and federal, that are currently in place have expiration date. Thus, it is worth studying the feasibility of ballasted PV without these incentives. Accordingly, the feasibility of installing ballasted PV on roof top of buildings in New York City with zero incentive condition has been investigated.

A NPV value of $46,638, Internal Rate of Return (IRR) of 5%, Profitability Index (PI) of 1.64, Discounted Cash Flow of $10,140, Simple Payback Period of 18 years and Simple Cash Flow of $266,206 were obtained. The results, except for Discounted Cash Flow (DCF), show that investment in the PV system can still be feasible without government incentives. But it is important to note that though the investment is feasible it may not be attractive for property owners. Since an IRR of 5% is not greater than the current discount rate (i.e., 5%). Although the simple payback period, which is found to be 18 years, is within the service life of the PV (i.e., 25 years), it is close to the end of the analysis period.

A study from the University of Connecticut [61], which is published in 2012, concludes that for a PV system to be feasible with in the service life period (usually 25 years), government incentives are required. Market changes and progress in the production of more efficient PV has yielded at least a feasible result although it is not an attractive one. This shows incentives play a crucial part in the feasibility of PV systems in New York City.

5.5.4 Environmental benefits

One of the merits of generating electric energy from solar radiation is the reduction of carbon dioxide (CO₂) emission from the atmosphere. Various methods have been developed to estimate how much carbon dioxide will be avoided from the atmosphere due to the usage of PV system instead of using the conventional electric. The recent prominent method to estimate the avoided carbon dioxide and other harmful substances from the environment which is developed by EPA is Avoided Emissions and Generation Tool (AVERT).

This tool has been used to estimate the amount of carbon dioxide that will be avoided from the environment by using the ballasted PV in this thesis. Consequently, 20 tons of carbon dioxide and 10 lbs. of Nitrogen Oxides will be avoided annually. During the lifetime, the PV system will avoid 500 tons of carbon dioxide and 250 lbs. of nitrogen oxides (NOₓ).
CHAPTER 6: SUMMARY AND CONCLUSION

6.1 Summary
This thesis work encompasses wind load determination as well as analysis to evaluate uplift wind pressure and consequently determine the effectiveness of CMU blocks to resist uplift wind pressure. Four different specifications along with three wind exposure classes and three different building heights were considered. This resulted in 18 different cases. Finite element modeling is used to obtain pre- and post-PV installation long term deflections for all the cases considered. Economic analysis to determine the profitability of installing ballasted PV system on rooftop slabs with and without incentives is also conducted. To validate calculations from each energy estimation, three methods of energy production estimating have been used, forming a total of thirty-two cases.

6.2 Conclusions
Based on this study, the following conclusions are drawn:

- The ballasted PV system does not influence the serviceability (long term deflection) of a building.
- The latest specifications (i.e., ASCE 7-16 and SEAOC-17) resulted in lower uplift pressure and ballast as compared to the old specifications (i.e., ASCE 7-05 and 2014 New York City Building code). This is due to the introduction of the array effect concept, which enables the connected PV on rooftop slab to have more redundant forces to resist the uplift pressure.
- While ASCE 7-05 and 2014 New York City Building code resulted in uniform uplift pressure (with no difference between exterior and interior panels), ASCE 7-16 and SEAOC-17 have an edge factor which brings differences in uplift pressure and ballast required among exterior and interior panels.
- Calculations based on ASCE 7-16, and SEAOC-17 show that for an 80 ft building, uplift pressure on exterior panels is 33% higher than on the interior panels, and CMU blocks on exterior panels are 37% greater than on interior panels. Additionally, CMU blocks for exterior panels calculated using ASCE 7-16 and SEAOC -17 are 45% less than those calculated using ASCE 7-05 and 2014 New York city building code.
- The investment for PV installation on rooftops of buildings in New York City is feasible as per economic indicators used in this thesis with the current level of incentives. The simple payback period is found to be 5 years.
Based on the literature search covered under this thesis work, the long-term deflection modification factor $\lambda$ from ACI 318 is found to be conservative.

6.3 Recommendations for Future Work

The following points can be seen as recommendations for future work and/or expansion of this study:

- Investigating on the time dependent factor ($\xi$) and long-term deflection modification multiplier for better estimation of long-term deflection.
- Examining additional case study based on different structural arrangements (i.e., based on different slab types, span, etc.) for considering the effect of ballasted PV on serviceability.
- Investigating the rigidity of the connection of PV arrays based on array effect concept.
- Exploring incentive policies and their impact to identify the best policies and modify those with limited impact.
- Investigating the production of high efficiency PV systems that can be made from cheaper materials for better feasibility results.
REFERENCES


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[27] ACI Committee 318, Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary, Farmington Hills, MI, 2008.; American Concrete Institute, 2008.


[43] NC Clean Energy Technology Center, NC State University, "DSIRE, NY-Sun PV Incentive Program (Residential, Low-Income, and Small Business)," 2019. [Online]. Available:


[51] B. Speer, M. Mendelsohn and K. Cory, "Insuring Photovoltaics; Challenges and Possible Solutions," 2010.


APPENDIX-A: Coefficients for Snow Load and Uplift Wind Pressure Calculations

Table A-1) Uplift Pressure in ASCE 7-05 and 2014 NYC Building Code.

<table>
<thead>
<tr>
<th>Coefficients</th>
<th>ASCE 7-05</th>
<th>2014 NYC Building Code</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Exposure class</td>
<td>Exposure class</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>C</td>
</tr>
<tr>
<td>Basic Wind Velocity (mph)</td>
<td>98</td>
<td>98</td>
</tr>
<tr>
<td>Importance Factor (I)</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Velocity Pressure Coefficient (Kv)</td>
<td>0.93</td>
<td>1.21</td>
</tr>
<tr>
<td>Topographic Factor (Kzt)</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Wind Directionality Factor (Kd)</td>
<td>0.85</td>
<td>0.85</td>
</tr>
<tr>
<td>Gust Effect Factor (G)</td>
<td>0.85</td>
<td>0.85</td>
</tr>
</tbody>
</table>
Table A-2) Coefficients to Calculate Uplift Pressure in ASCE 7-16

<table>
<thead>
<tr>
<th></th>
<th>ASCE 7-16</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Exposure Class B</td>
</tr>
<tr>
<td>Basic Wind Velocity (mph)</td>
<td>115</td>
</tr>
<tr>
<td>Importance Factor (I)</td>
<td></td>
</tr>
<tr>
<td>Velocity Pressure Coefficient (K_v)</td>
<td>0.93</td>
</tr>
<tr>
<td>Topographic Factor (K_t),</td>
<td>1.0</td>
</tr>
<tr>
<td>Wind Directionality Factor (K_d),</td>
<td>0.85</td>
</tr>
<tr>
<td>Gust Effect Factor (G)</td>
<td></td>
</tr>
<tr>
<td>Ground Elevation Factor (k_e)</td>
<td>1.0</td>
</tr>
<tr>
<td>Nominal Pressure Coefficient (G_{cm, hom}, after interpolation for 10 deg. panel tilt)</td>
<td>0.51</td>
</tr>
<tr>
<td>Net Pressure Coefficient (G_{cm, net}, for Exterior Panels)</td>
<td>0.67</td>
</tr>
<tr>
<td>Net Pressure Coefficient (G_{cm, net}, for Interior Panels)</td>
<td>0.45</td>
</tr>
<tr>
<td>Array Edge Factor γ_E for Exterior Panels</td>
<td>1.5</td>
</tr>
<tr>
<td>Array Edge Factor γ_E for Interior panels</td>
<td>1.0</td>
</tr>
<tr>
<td>Panel Chord Factor, γ_c</td>
<td>0.91</td>
</tr>
<tr>
<td>Parapet Height Factor, γ_p</td>
<td>0.96</td>
</tr>
</tbody>
</table>
Table A-3) Coefficients to Calculate Uplift Pressure in SEAOC-17

<table>
<thead>
<tr>
<th></th>
<th>Exposure Class B</th>
<th>Exposure Class C</th>
<th>Exposure Class D</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basic Wind Velocity (mph)</td>
<td>115</td>
<td>115</td>
<td>115</td>
</tr>
<tr>
<td>Importance Factor (I)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Velocity Pressure Coef. ($K_x$)</td>
<td>0.93</td>
<td>1.21</td>
<td>1.38</td>
</tr>
<tr>
<td>Topographic Factor ($K_a$),</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Wind Directionality Factor ($K_d$),</td>
<td>0.85</td>
<td>0.85</td>
<td>0.85</td>
</tr>
<tr>
<td>Gust Effect Factor (G)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ground Elevation Factor ($k_e$)</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Ground Elevation Factor ($k_e$)</td>
<td>0.51</td>
<td>0.51</td>
<td>0.51</td>
</tr>
<tr>
<td>Nominal Pressure Coefficient ($G_{cm}$)$_{nom}$, after interpolation for 10 degrees panel tilt</td>
<td>0.67</td>
<td>0.67</td>
<td>0.67</td>
</tr>
<tr>
<td>Net Pressure Coefficient ($G_{cm}$)$_{net}$, for Exterior Panels</td>
<td>0.45</td>
<td>0.45</td>
<td>0.45</td>
</tr>
<tr>
<td>Net Pressure Coefficient ($G_{cm}$)$_{net}$, for Interior Panels</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
</tr>
<tr>
<td>Array Edge Factor $\gamma_E$ for Exterior Panels</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Panel Chord Factor $\gamma_c$</td>
<td>0.91</td>
<td>0.91</td>
<td>0.91</td>
</tr>
<tr>
<td>Parapet Height Factor, $\gamma_p$</td>
<td>0.96</td>
<td>0.96</td>
<td>0.96</td>
</tr>
<tr>
<td>------------------------------</td>
<td>---------------</td>
<td>------------------------</td>
<td>--------------</td>
</tr>
<tr>
<td>Exposure class</td>
<td>Exposure class</td>
<td>Exposure Class</td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>20</td>
<td>25</td>
<td>20</td>
</tr>
<tr>
<td>C</td>
<td>20</td>
<td>25</td>
<td>20</td>
</tr>
<tr>
<td>D</td>
<td>20</td>
<td>25</td>
<td>20</td>
</tr>
<tr>
<td>Snow Exposure Factor, $C_e$</td>
<td>0.90</td>
<td>0.90</td>
<td>0.90</td>
</tr>
<tr>
<td></td>
<td>0.90</td>
<td>0.90</td>
<td>0.90</td>
</tr>
<tr>
<td></td>
<td>0.80</td>
<td>0.90</td>
<td>0.90</td>
</tr>
<tr>
<td>Thermal Factor, $C_t$</td>
<td>0.90</td>
<td>0.90</td>
<td>0.90</td>
</tr>
<tr>
<td></td>
<td>0.90</td>
<td>0.90</td>
<td>0.90</td>
</tr>
<tr>
<td></td>
<td>0.90</td>
<td>0.90</td>
<td>0.90</td>
</tr>
<tr>
<td>Snow Importance Factor, $I_s$</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>
APPENDIX-B: Load Application in Finite Element Modeling

Figure B-1: Snow Load Applied for Design Stability Check
**Figure B-2:** Wind Load Applied for Design Stability Check.

**Figure B-3:** Load Cases Used
**Figure B-4:** Load Combination for Total Long-Term Load Combination

**Figure B-5:** Reinforcement Source and Ratio Definition in SAFE V20
APPENDIX-C: Ballast Arrangement Layout

A) 80 ft Tall Building using SEAOC-17

Figure C-1: Wind Exposure Class B
Figure C-2: Wind Exposure Class C
Figure C-3: Wind Exposure Class D
B) 80 ft Tall Building using ASCE 7-05 and 2014 NYC Building Code

![Wind Exposure Class B Diagram]

**Figure C-4:** Wind Exposure Class B

Number of Ballast = 14
Figure C-5: Wind Exposure Class C
Figure C-6: Wind Exposure Class D
C) 60 ft Tall Building Using ASCE 7-16

**Figure C-7: Wind Exposure Class B**
Figure C-8: Wind Exposure Class C
Figure C-9: Wind Exposure Class D
A) 40 ft Tall Building Using ASCE 7-16

Figure C-10: Wind Exposure Class B
Figure C-11: Wind Exposure Class C
Figure C-12: Wind Exposure Class D
APPENDIX-D: Inputs Used for Energy Estimation

Table D-1) System Loss Breakdown in PVWATTS

<table>
<thead>
<tr>
<th>System Loss</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soiling</td>
<td>2</td>
</tr>
<tr>
<td>Shading</td>
<td>3</td>
</tr>
<tr>
<td>Snow</td>
<td>0</td>
</tr>
<tr>
<td>Mismatch</td>
<td>2</td>
</tr>
<tr>
<td>Wiring</td>
<td>2</td>
</tr>
<tr>
<td>Connection</td>
<td>0.5</td>
</tr>
<tr>
<td>Light Induced Degradations</td>
<td>1.5</td>
</tr>
<tr>
<td>Nameplate Rating</td>
<td>1</td>
</tr>
<tr>
<td>Age</td>
<td>0</td>
</tr>
<tr>
<td>Availability</td>
<td>3</td>
</tr>
<tr>
<td>Total</td>
<td>14</td>
</tr>
</tbody>
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Table D-2) Input Data for Theoretical Calculation using Equation 5.1.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of Panels</td>
<td>63 pieces</td>
</tr>
<tr>
<td>Area of 1 Panel</td>
<td>17.57 ft²</td>
</tr>
<tr>
<td>Total Area of Panels</td>
<td>1106.91 ft²</td>
</tr>
<tr>
<td>Panel Efficiency</td>
<td>22.2%</td>
</tr>
<tr>
<td>Declination Angle (δ)</td>
<td>-23.432 to 23.432 deg.</td>
</tr>
<tr>
<td>Elevation Angle (α)</td>
<td>56.550 to 103.45 deg.</td>
</tr>
<tr>
<td>Latitude</td>
<td>40.73 deg.</td>
</tr>
<tr>
<td>Tilt angle</td>
<td>10 deg.</td>
</tr>
<tr>
<td>Performance Ratio</td>
<td>0.75</td>
</tr>
</tbody>
</table>

Table D-3) Input Data for Theoretical Calculation using Equation from CED (Equation 5.5)

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of Panels Used</td>
<td>Sun Power X-22-360</td>
</tr>
<tr>
<td>Nominal Power from Each Panel (at STC)</td>
<td>360 Watts</td>
</tr>
<tr>
<td>System Loss</td>
<td>14%</td>
</tr>
<tr>
<td>System Efficiency</td>
<td>86%</td>
</tr>
<tr>
<td>Power Tolerance</td>
<td>-5%</td>
</tr>
<tr>
<td>Peak Sun hours for NYC</td>
<td>4.08 hrs/day</td>
</tr>
<tr>
<td>Derating Factor</td>
<td>0.75</td>
</tr>
</tbody>
</table>