Solar Power for

Deployment in Populated Areas

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ABSTRACT

Solar Power for Deployment in Populated Areas

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The thesis presents background on solar thermal energy and addresses the structural challenges associated with the deployment of concentrating solar power fields in urban areas. Two potential structural systems and urban locales of deployment are proposed and investigated to determine whether they have the potential to be a cost-effective renewable energy solution for urban areas. The structural issues explored in the thesis include flutter, the wind loading of open frame structures, performance-based design, and the design of flexibly mounted equipment on a building.
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LIST OF NOMENCLATURE

a_{xm} – Modal story acceleration at level x for mode m (g)

A_s – The gross area of the solid wall as defined in ASCE 7-05

A_g – The gross area of the wall including all openings as defined in ASCE 7-05

B – Frame width, measured from outside edge to outside edge

C_{Dg} – Force coefficient on the gross area of the wall

C_f – Force coefficient as defined in ASCE 7-05

C_s – Seismic response coefficient as defined in ASCE 7-05 (g)

F_p – Design force applied to solar tower (force)

G – Gust Factor as defined in ASCE 7-05

I – Importance Factor as defined in ASCE 7-05

Kzt – Wind topographic factor as defined in ASCE 7-05

N – Number of framing lines normal to the nominal wind direction

p_s – Net design wind pressure as defined in ASCE 7-05 (lb/ft²)

p_{S30} – Simplified design wind pressure as defined in ASCE 7-05 (lb/ft²)

P_{EY} – Probability of exceedance (expressed as a decimal) in time Y (years) for the desired earthquake hazard level

P_R – Return period of seismic event (year)

PF_{xm} – Modal Participation Factor at level x for mode m

S_{am} – Spectral roof response acceleration of mode m

S_{DS} – Design spectral response acceleration response parameter, 5 percent damped, at short periods as defined in ASCE 7-05 (g)
SD1 – Design spectral response acceleration response parameter, 5 percent damped, at a period of 1 second as defined in *ASCE 7-05* (g)

SMS – The MCE spectral response acceleration response parameter, 5 percent damped, at short periods adjusted for site class effects as defined in *ASCE 7-05* (g)

SM1 – The MCE spectral response acceleration response parameter, 5 percent damped, at a period of 1 second adjusted for site class effects as defined in *ASCE 7-05* (g)

SF – Frame spacing, measured from centerline to centerline

Sfa – Spectral roof response acceleration (g)

Ta – Period of solar power tower or other flexibly mounted equipment (sec)

Tm – Building period for mode m (sec)

To – .2 SD1/SDS as defined in *ASCE 7-05* (sec)

TS – SD1/SDS as defined in *ASCE 7-05* (sec)

V – Seismic base shear as defined in *ASCE 7-05* (force)

wi/g – Mass assigned at level I (k-sec²/ft)

W – Effective seismic weight as defined in *ASCE 7-05* (force/g)

Wp – Effective seismic weight of the solar tower (force/g)

Y – Years for the desired earthquake hazard level (years)

α – Wind angle of attack

Φim – Amplitude of mode m at level I (dimensionless)

ε – Solidity ratio (Ae/Ag)

λ – Wind adjustment factor for building height and exposure as defined in *ASCE 7-05*
1.0 PURPOSE

The purpose of the thesis is to present a single document exploring the structural challenges and issues that arise when integrating solar thermal energy (STE) in an urban environment through the use of concentrating solar power (CSP) fields. To facilitate the purpose, the thesis has been organized into the following sections:

- The Introduction provides a synopsis of both the thesis and the overall project.
- The Background discusses solar thermal energy and the concentrating solar power systems that have the potential to be deployed in urban areas.
- The Research reveals the key structural issues investigated in relation to the deployment of concentrating solar power systems in urban areas.

Many terms related to alternative energy methods may be unfamiliar to individuals outside of the field. As a result, a Glossary section (6.0) has been provided for all bold words within the body of this thesis.
2.0 INTRODUCTION

The objective of the thesis is to explore the deployment of concentrating solar power (CSP) fields in urban areas in order to provide solar thermal energy (STE). Specifically, the thesis investigates the engineering-related issues of two structural systems that have the potential to support the mass deployment of CSP fields in populated areas. In addition, the thesis aims to provide a foundation for future research and offers suggestions for where that research is best directed.

For an individual unfamiliar with solar thermal energy, the Background section (3.0) should be referenced, as needed, for a more comprehensive understanding of the presented material.

2.1 Focus of the Thesis

Populated regions provide only a limited number of areas that are both commonly available and large enough to deploy a CSP field. Two areas that were deemed appropriate to deploy CSP fields in urban areas were over urban parking lots and on top of urban industrial buildings. Not only are these two types of locale commonly available, but they also provide an area large enough to integrate an efficient CSP field.

Each potential locale of urban deployment gives rise to its own unique structural system. In urban parking lots, the CSP field must be elevated so as not to interfere with the vehicular use of the lot, whereas on industrial buildings, the CSP field can be constructed directly on the rooftop.
The structural issues associated with the design of an elevated CSP field are explored in the Research section (4.0) of the report. Figure A, below, provides an illustration of how the CSP field would appear deployed over an urban parking lot.

![CSP Field over Urban Parking Lot](image)

**Figure A – CSP Field over Urban Parking Lot**  
Source: Nathan Hicks

Another aspect of the thesis is the investigation of the deployment of a concentrating solar power system on an existing industrial building. For this portion of the thesis, the Digital West Networks™ building on Sacramento Drive in San Luis Obispo serves as a case study to investigate the implementation of a CSP field in an urban area. Figure B, below, provides an illustration of how the solar concentrator field would appear deployed over an existing urban industrial building.
2.0 Introduction

2.2 Focus of the Overall Project

The thesis’ investigation is but one part of a broader project being investigated in two departments (Physics and Architectural Engineering) at California Polytechnic State University, San Luis Obispo. To avoid confusion, the words “thesis” or “report” will refer to the specific contributions herein presented, whereas references to the “project,” “research project,” or “overall project” will refer to the broader scope of work being conducted at Cal Poly.

The objective of the project is to research and develop solar thermal energy (STE) integrated within urban areas. Specifically, the research aims at advancing alternative energy solutions by bringing alternative energy systems closer to the end-user. Presently,
urban end-users are offered a limited scope of on-site solar energy solutions, and the solutions themselves are not economically feasible. Therefore, the project’s goal is to investigate whether intermediate scale CSP fields have the potential to be the on-site solar energy solution for urban areas and commercial buildings.

2.3 Relevance of Research

The fossil fuel trio of coal, oil, and natural gas provides more than three-quarters of the world’s energy, today. Figure C, below, displays the predominant use of fossil fuels worldwide as compared to other energy sources for 2004.

![WORLDWIDE ENERGY CONSUMPTION](image)

**Figure C – Worldwide Energy Consumption**
Source: US Energy Information Administration

Despite this demand, intermittent concerns have been raised ever since the oil crisis of the 1970s over the world’s continued dependence on fossil fuel. The expressed concerns have dealt not only with the environmental impacts of fossil fuel use, but also with the
finite nature of supplies (Boyle 2004), fueling an interest in finding a renewable energy source for a **sustainable** future.

The goal of the overall project is to address the need for renewable energy sources through the design of a solar concentrator field that provides cooling, heating, and power services at a price consistent with present competitive technologies. In an effort to reach this end, both the solar field and the structural system must be innovative in design. While some individuals and corporations are willing to transition to renewable energy sources out of concern for our natural resources and ecosystems, the majority will wait until the solution is cost-effective. The wide-scale use of the solar concentrator field in urban areas has the potential to be that cost-effective solution.

With the wide availability of suitable urban sites and the increasing scarcity of conventional fuel sources, the solar concentrator field system could compete with fossil fuel-based power in the future.
3.0 BACKGROUND

The background section presents information on solar thermal energy (STE) that is necessary for a complete understanding of this thesis.

Solar Thermal Energy (STE) is a technology for harnessing the sun’s solar energy for thermal energy purposes. A majority of the solar thermal collectors currently produced each year are low-temperature collectors that use water or air as a medium to transfer heat to a final destination. The low-temperature collectors are common in residential applications for space heating or the heating of swimming pools, but are inefficient, in terms of return on investment, when used in commercial or large-scale applications. As a result, the thesis will not deal with the low-temperature collectors, but will focus on high-temperature collectors. Figure D, below, illustrates the conversion of solar radiation into electricity for high-temperature collectors.

![Figure D – Solar Thermal Energy](image)

Source: Nathan Hicks
In the CSP field illustrated in the figure a mirror or lense is used to concentrate the sun’s radiation onto a collector, creating a multi-sun effect. The working fluid in the collector is then super heated to between 200° – 1000° C. As the fluid expands to a gaseous state, the released energy powers a turbine that in turn produces electricity. In a vast majority of solar thermal collectors, excess heat is wasted; however, by moving the solar thermal collector near the end-user the waste heat can be used in heat storage, hot water generation, and even air conditioning through the use of an absorption chiller.

STE technology should not be confused with the photovoltaic (PV) cells commonly used in solar panels. Rather than converting the solar radiation into thermal power, photovoltaic cells convert solar energy directly into electricity. While photovoltaic panels are often adequate to meet the energy demands of a residential building, they are not presently efficient for commercial use.

3.1 Early Solar Thermal Energy Systems

The idea of concentrating solar radiation in order to produce solar thermal energy has been around for over 100 years.

In the late nineteenth century, France was struggling to meet energy demands as it lacked an economical supply of coal. Addressing this lack of coal, Augustin Mouchot, a French mathematics professor, began production on the first high-temperature solar concentrators in the 1870s and 1880s. Over these years Mouchot and his assistant, Abel Pifre, constructed and displayed a series of parabolic concentrators with a steam boiler mounted at the focus of each concentrator. Solar radiation incident to the surface of the parabolic dish was concentrated on the boiler, producing steam. The steam traveled down
from the boiler through a series of pipes to a **reciprocating engine**, powering mechanical work (Boyle 2004). Figure E, below, displays one of Pifre’s solar concentrators, which was used to power a printing press.

![Figure E – Pifre’s Solar Concentrator](image)

**Figure E – Pifre’s Solar Concentrator**  
*Source: Boyle 2004*

Although the concentrating solar power systems were widely acclaimed, it became clear by the 1890s that the solar concentrators would be unable to compete with coal in France. The parabolic dishes produced by Mouchot and Pifre were unable to generate a concentration ratio high enough to create a competitive overall efficiency (Boyle 2004).
During this same era other attempts were made to mass-produce high-temperature solar collectors, most notably by American entrepreneur Frank Shuman. After building several prototypes and raising a substantial financial backing, Shuman began planning the construction of 20,000 square miles of parabolic trough collectors in the Sahara Desert. World War I broke out before construction could begin, and immediately after the war the era of cheap oil began, effectively killing interest in high-temperature solar collectors for half a century (Boyle 2004).

### 3.2 Contemporary Solar Thermal Energy Systems

Over the last few decades, interest in high-temperature solar collectors to use solar thermal energy has increased once again. While these collectors are conceptually the same, modern technology has allowed this new era of solar concentrators to reach much higher overall efficiencies. The following sections will briefly overview three concentrating solar power (CSP) systems that have the potential to be deployed in urban areas.

#### 3.2.1 Solar Power Towers

The first CSP system under consideration is the solar power tower. Solar power towers use a large array of flat mirrors to concentrate solar radiation on a collector tower. In the early 1980s the first solar power tower, Solar One, was constructed in Barstow, California. The plant used synthetic oils to carry away the heat from the collector tower to a steam boiler. In the 1990s Solar One was rebuilt to include heat storage, allowing the production of electricity on a 24-hour basis. In 2005 a new tower project was completed in Seville, Spain to explore the use of super-heated air as a transfer medium to a
conventional steam turbine (Boyle 2004). Figure F, below, displays the solar power tower in Seville and illustrates the intense concentration of the sun’s rays that can be achieved with an array of flat mirrors.

![Figure F – Seville Solar Power Tower](image)

**Figure F – Seville Solar Power Tower**  
Source: NewEnergyDirection

Presently BrightSource Energy is developing solar power tower complexes in both California’s Mojave Desert and Israel’s Negev Desert. Figure G, below, shows an aerial view of the Ivanpah Solar Power Complex in the Mojave Desert.

![Figure G – Ivanpah Solar Power Tower](image)

**Figure G – Ivanpah Solar Power Tower**  
Source: BrightSource Energy
The 5-square-mile facility, illustrated above, began construction in 2009 and will be completed in 2011, generating enough electricity to power 140,000 homes per year.

Solar power towers in an urban context would be on a much smaller scale than the plants currently being built on desert floors. As a part of the thesis and the overall project, the solar power tower is being explored as a possible method of deploying a solar concentrator on the roof of an industrial building in an urban setting. The idea is that an array of flat mirrors are added to the roof of existing industrial buildings of an adequate size. The mirrors would then focus the sun’s rays onto a collector tower beginning, the production of solar thermal energy.

From a structural perspective many issues must be addressed for the practical integration of a collector tower in an urban area. First of all, the gravity system of the industrial building on which the tower is being deployed must be investigated to ensure that the building can resist the additional load of the mirrors, mechanical equipment, and the tower itself. Additionally, in a seismically active region the tower must be engineered to withstand the roof accelerations that would result from the ground accelerations of an earthquake. As a relatively flexible column with a mass on the end, even small accelerations at the base of the tower can lead to large and potentially destructive displacements.

3.2.2 Solar Troughs

The second CSP system under consideration is solar concentrator troughs. Solar concentrator troughs use long parabolic mirrors to focus the sun’s radiation on a
continuous **Dewar tube**. The concentrated radiation heats up the fluid, commonly synthetic oil, inside the tube as is shown in Figure H, below.

![Figure H – Solar Trough](source: trec-uk.org)

The heat transfer fluid is then used to heat steam in a standard turbine generator (Boyle 2004). Often the troughs rotate to track the sun throughout the day and increase the efficiency of the system. In such instances the solar troughs are oriented along a North-South axis so that they can efficiently follow the sun from the east to the west horizon. However, if the troughs are stationary and lack a tracking method, they are often oriented along an East-West axis. With this stationary setup there is no need for tracking motors, leading to a lighter, less mechanically complicated overall system, but this system is consequently far less efficient (Patel 2006).
As a part of the thesis, the potential of an elevated solar trough system over urban parking lots is being explored. The idea is that columns would support a frame-like structure that would in turn carry the parabolic mirrors, tubing, and other mechanical equipment for the solar concentrator system. Not only would the parabolic mirrors provide solar thermal energy to the surrounding community, but the structure itself would provide shielding from both sunlight and precipitation in the urban areas of deployment.

Structurally, the integration of a solar trough concentrator system over urban parking lots raises key issues that are investigated in this thesis, including the wind loading of open frame structures, the potential for destructive flutter behavior, and the potential for destructive collapse. The key issues for both the industrial building and parking lot structural systems will be discussed in greater detail in the Research section (4.0) of the report.

### 3.2.3 Concentrating Photovoltaics and Thermal (CPVT)

The third CSP system under consideration is concentrating photovoltaics (CPV). Concentrating photovoltaics use lenses to concentrate the solar radiation onto a small area of photovoltaic cells, as is shown on the following page in Figure I. To maximize the concentration ratio, the CPV systems are often designed with tracking systems to stay in line with the sun. While the photovoltaic cells convert the solar radiation into electricity in the same manner as conventional panels, the photovoltaics in the concentrating system are far more efficient; thus substantially fewer PV cells are required to produce the same amount of electricity (Boyle 2004).
As a result of the high concentration of radiation, the cells in the concentrating photovoltaic system have been plagued by overheating. In the past, the cells often needed to be cooled either passively or actively to prevent this overheating. In order to address this issue, Concentrating Photovoltaics and Thermal (CPVT) has emerged as a technology that combines the electricity production of CPV systems with the thermal heat production of STE systems. As the photovoltaic cells heat up, the system allows the flow of a fluid to cool off the cells. Not only does the fluid absorb heat to allow the PV cells to operate without overheating, but the heated fluid is also used in the production of solar thermal energy. In this way both electricity and thermal energy are generated in a CPVT system.
The CPVT system has the potential to be integrated into both the elevated solar trough system and the solar power tower system. As a part of the overall project, the CPVT system is being explored by the physics team in order to establish whether or not concentrating photovoltaics and thermal have the potential to be deployed in urban environments.
4.0 RESEARCH

The Research section investigates the key structural issues associated with the integration of a solar concentrator field in an urban area, which are:

- Flutter
- Wind Loading of Open Frame Structures
- Performance-Based Design

In each of the sections, the key issue will be defined; the reason for an investigation of the key issue will be explored; the research on the key issue will be presented; and finally a conclusion will be made concerning the key issue’s impact on the thesis and overall project.

4.1 Flutter

Flutter develops as the heaving (vertical) and torsional (twisting) motion of an object are unified at single frequency. As the aerodynamic forces couple with the object’s natural mode of vibration, a resulting rapid periodic motion emerges (Jakobsen and Tanaka 2003). If the energy from the aerodynamic excitation exceeds the natural dampening of the system, the level of vibration will increase. In such instances the self-starting coupled flutter can result in potentially destructive vibrations (Jakobsen and Tanaka 2003).

4.1.1 Reason for Investigation

Flutter was investigated to determine whether or not it was a phenomenon that needed to be taken into account when designing the structural system for a CSP field. The
thought was that flutter due to high winds might cause destructive behavior in the elevated CSP structural model to be deployed over urban parking lots. In addition, there was also a concern that flutter could cause destructive vibrations within the individual elements, such as the mirrors, of the concentrating solar power field. As an open frame structure, the interaction between the wind forces and the behavior of the building system is more complex than for a similar closed structure.

4.1.2 Early Investigation of Flutter

The first exploration into flutter occurred in the first half of the nineteenth century. On November 29, 1836 a section of the Brighton Chain Pier in East Sussex, England failed in a storm. As local columnist George Bishop wrote,

About half-past twelve in the day the centre bridge seemed to have acquired through the force of the wind, a vibratory motion, which soon after, more or less affected the whole structure. At times the platform was raised to the level of the protecting iron rails at the sides of the Pier. Eventually one of the Towers began to rock, and the piles also to twist; and finally the platform of the third bridge was lifted up from its bed several feet, and, falling again — the suspension rods being unable to bear the stupendous strain — plunged into the stormy waters below (Bishop 1897).

Figure J, on the following page, displays an illustration that accompanied the newspaper article from 1836. It clearly shows the wild behavior of the section of Brighton Chan Pier that failed during this storm. As is evidenced in Bishop’s account, it was known at the time that torsional oscillations coupled with large vertical displacements were to blame for the collapse. However, it would take the better part of a century before bridges and other structures were designed to resist flutter (Cook 1990).
It wasn’t until the early 1900s with the development of the airplane that the problem of flutter was readdressed. Individuals in the aeronautical field began investigating the phenomenon of flutter after an aircraft designed by Samuel Pierpont Langley broke apart shortly after takeoff on December 9, 1903. It was decided that a complex torsional interaction between the airflow and the plane was at fault for the craft breaking up, an interaction which we have come to know as flutter (Cook 1990).

The issue of instability in structures due to flutter wasn’t raised again until the Tacoma Narrows Bridge Failure in 1940 (Matsumoto, et al. 2007). In the Tacoma Narrows Bridge, as well as other long span bridges, the weak torsional rigidity ultimately made the bridges susceptible to a strong wind flow. Over the years, as bridges have employed the use of ever stronger and lighter building materials, there has been a significant decrease in the natural frequency of these structures. In addition, the evolution to stronger and lighter materials has resulted in a decrease in the ratio between the
fundamental torsional and vertical mode frequencies, making long span bridges even more vulnerable to flutter instability (Bartoli and Righi 2006). Research by bridge engineers, such as Farquharsen and Karman, over the last half-century has led to the formulation of high order differential equations to calculate the critical wind speeds at which lift and pitching moment are maximized (Matsumoto, et al. 2008). These differential equations have enabled the design of bridges less likely to experience unstable torsional, **dynamic** responses. Furthermore, research in bridge engineering has revealed that in the wind velocity range of interest for bridge design, the flow around **bluff body** bridge sections (bridge deck) is not agreeable with the quasi-steady flow theory. The use of turbulent flow as opposed to steady flow in bridge design actually reduces the amplitude of the motion, since the lack of correlation of the incoming wind introduces an aerodynamic damping effect (Bartoli and Righi 2006).

### 4.1.3 Current Investigation of Flutter

Outside of aeronautics and bridge engineering, very little research has been done on the issue of flutter instability. One of the compelling reasons to investigate flutter as a part of this thesis is due to the similarities between the cross section of the proposed elevated CSP field and the cross section of a typical bridge. In bridge engineering the ratio of the length or breadth of a bridge section (B), to the depth of the section (D) is an indicator of how susceptible a structure is to flutter. A higher B/D ratio often translates to a weak torsional **rigidity** and a structure potentially more susceptible to destructive flutter. As Figure K, below, illustrates, the elevated CSP field for deployment over urban parking lots has a B/D ratio larger than that of a bridge section.
While this may seem to be cause for concern, differences between the elevated CSP field and the bridge section need to examined before arriving at any conclusions.

The first key difference is that a major contributor to the weak torsional rigidity of many bridges is their relatively long span in relation to the breadth of the section. The elevated CSP field’s span to breadth ratio is not as severe as that seen in bridges, and therefore more torsional rigidity is provided. However, the defining difference between the solar field model and a long span bridge is the fact that the CSP field is anchored to the ground by columns. Although not perfectly rigid, these columns limit the overall heaving and torsional displacements of the structure to infinitesimal amounts. As Figure L, on the following page, displays a bridge develops flutter as the vertical and torsional motion are unified at a single frequency. In the elevated CSP field the vertical and torsional motion do not develop. The columns that support the structural system restrict the CSP field from any large displacements in the vertical direction.
4.1.4 Conclusion on Flutter

The thesis concludes that flutter is not an issue that need be addressed in the modeling of the overall structural system for the elevated CSP field being deployed over urban parking lots. The columns supporting the CSP field provide enough vertical rigidity to resist large vertical displacements due to dynamic wind loading. Future testing of the mirrors, solar troughs, and supporting infrastructure in moderate wind speeds will be needed to establish what type of support is necessary to resist the potentially destructive vibrations of flutter within individual components of the CSP field.

4.2 Wind Loading of Open Frame Structures

The basis and procedures for calculating wind induced forces on conventional and enclosed structures are well documented in the engineering literature, perhaps most notably in the *ASCE 7-05* and its predecessor documents. The provisions set forth in the
ASCE 7 have been adopted by organizations in the development of their building codes. One such building code to adopt the ASCE 7 is the International Building Code, which is the model building code adopted throughout most of the United States.

The scope of the ASCE 7 states, “This standard provides minimum load requirements for the design of buildings and other structures that are subject to building code requirements.” However, even a task committee from the American Society of Civil Engineers (ASCE) acknowledged that the ASCE 7 does not adequately address open frame structures. Lacking a uniform method, the industry has developed numerous design practices to calculate the wind loading on open frame structures, all of which can vary greatly in the resulting wind induced forces (ASCE 1997).

4.2.1 Reason for Investigation

The concentrating solar power field deployed over urban parking lots calls for an elevated structural system. The elevated CSP system consists of solar troughs running over a frame-like structure that is supported above the parking lot by columns. The structure would likely lack any exterior cladding and thus would behave as an open frame structure. Due to the structure’s lightweight construction, it becomes increasingly likely that the lateral forces in the system would be governed by wind rather than earthquake even in high seismic regions of California. It is therefore essential that in the design of the lateral force resisting system an acceptable method be found for calculating the wind forces on an open frame structure.
4.0 Research

4.2.2 Current Investigation of Wind Loading

Chapter 6 of the ASCE 7-05 presents methods for calculating the wind-induced lateral forces on a building. The basic principles and procedures behind the methods are similar, and for the purpose of this thesis are presented in a simplified manner. The first step in the design procedure is to determine the basic wind speed, $V$, in accordance with the values shown on Figure M, below.

![Figure M - Basic Wind Speed](image)

**Figure M – Basic Wind Speed**
Source: *ASCE 7-05*

The basic wind speed can then be translated into a simplified design wind pressure, $p_{S30}$, using an accompanying figure in the chapter. The final design wind pressure for the system, $p_s$, can be calculated using Equation 1, below:
\[ p_s = \lambda (K_{zt})(I)(p_{S30}), \]  \hspace{1cm} \text{Eq. 1}

where \( \lambda \) is the adjustment factor for building ht. and exposure (dimensionless), 
\( K_{zt} \) is the wind topographic adjustment factor (dimensionless), 
\( I \) is the importance factor (dimensionless), and 
\( p_{S30} \) is the simplified design wind pressure (psf).

The final design wind pressure, \( p_s \), is then applied to projections of the building surfaces in order to determine the total base shear (ASCE 7-05).

The shortcoming of the ASCE 7-05 wind design procedure is illustrated in plan view in Figure N, below.

![Figure N – Wind Direction](source: Nathan Hicks)

As the solidity ratio, \( \varepsilon \), of the frame decreases, the maximum force on the open frame structure no longer occurs when the wind direction is normal to the set of frames. The solidity ratio is defined in Equation 2, below:

\[ \varepsilon = \frac{A_s}{A_g}, \]  \hspace{1cm} \text{Eq. 2}

where \( A_s \) is the gross projected area of the solid wall (ft\(^2\)), and 
\( A_g \) is the gross projected area of the wall including openings (ft\(^2\)).
At a wind angle of attack, $\alpha$, equal to 0° in an open frame structure, the columns at the front of the structure in effect shield the back columns in the frame from experiencing any wind pressure. As a result the design wind pressure is only applied to the gross projected of the solid wall, $A_g$. As $\alpha$ increases beyond 0° the lateral force component normal to the frame will decrease, but the projected area that the wind pressure acts on will likely increase. Eventually an $\alpha$ is found that maximizes the lateral force on the open frame structure. In a guideline for the wind loading of open frame structures published by the ASCE, the task committee states

Although the wind direction is nominally considered as being normal to the set of frames under construction, the maximum force coefficient occurs when the wind is not normal to the frames. The angle at which the maximum force coefficient occurs varies with the dimensions of the structure, the solidity, number of frames, and frame spacing (ASCE 7-05).

The ASCE task committee, realizing the deficiency in the ASCE 7-05 for wind loading of open frame structures, began work on a uniformly accepted design guideline for calculating the wind-induced forces on open frame structures. Rather than using the maximum $\alpha$ in order to calculate the wind-induced base shear, the task committee devised a method in which the wind pressure calculated from the ASCE 7-05 was modified by a force coefficient and a gust effect factor. The resulting Equation 3, below, gives the base shear due to wind, $F$, on an open frame structure:

$$F = p_s \left( G \right) \left( C_f \right) \left( A_s \right),$$

Eq. 3

where $p_s$ is the final design wind pressure (psf),
$G$ is the gust effect factor (dimensionless),
$C_f$ is the force coefficient (dimensionless), and
$A_s$ is the gross projected area of the solid wall (ft$^2$).
The gust effect factor, $G$, can be calculated using the ASCE 7-05, however the force coefficient, $C_f$, is calculated using the ASCE task committee design guide and is presented in Equation 4:

$$C_f = \frac{C_{Dg}}{\varepsilon},$$  \hspace{1cm} \text{Eq. 4}

where $C_{Dg}$ is the force coefficient on the wall’s gross area (dimensionless), and $\varepsilon$ is the solidity ratio (dimensionless).

Taking into account the frame spacing ratio ($S_r/B$), the solidity ratio ($\varepsilon$), and the number of frames ($N$), the force coefficient ($C_{Dg}$) can be tabulated from a series of graphs in the recommended guidelines. Figure O, below, helps clearly identify the nomenclature ($S_r$, $B$, $N$) that the task committee adopted in the guidelines.

Figure O – Plan View of Framing
Source: Nathan Hicks
Figure O provides a hypothetical open frame structure in plan view and identifies the variables that are needed in order to interpret the ASCE task committee’s design guidelines force coefficient ($C_{Dg}$) graphs.

Figure P, on the following page, displays the force coefficient graphs for four different frame spacing ratios ($S_f/B = .1, .2, .33, .5$). For frame spacing ratios in between the four frame spacing ratios, linear interpolation can be used in order to identify the force coefficient on the gross area ($C_{Dg}$). Each graph in Figure P plots the force coefficient ($C_{Dg}$) as a function of the frame solidity ratio ($\varepsilon$) for structures with anywhere from 2 – 12 frames ($N$).
Figure P – Force Coefficient
Source: (ASCE 1997)
4.2.3 Conclusion on Wind Loading

It is the recommendation of this thesis that the wind-induced forces on an open frame structure deployed over an urban parking lot be calculated using the ASCE 7-05 in combination with the recommended guidelines set forth by the ASCE task committee in Wind Loads and Anchor Bolt Design for Petrochemical Facilities. An additional consideration with the wind loading of open frame structures is that the design load cases must take into account that the maximum wind load occurs when $\alpha > 0$. As a result it is the recommendation of both the task committee and this thesis that the designer take the total wind force acting on the structure in a given direction and simultaneously apply 50% of the total wind force along the other axis.

4.3 Performance-Based Design

Presently in the United States, design is regulated based on national model building codes. When adopted and enforced by local authorities, building codes are intended to establish minimum requirements for providing safety to life and property from hazards such as wind, earthquake, and fire. The building code’s goal is accomplished through prescriptive requirements, developed over the years by the performance assessment of buildings after a hazard (FEMA 445). The prescriptive criteria of building codes provide an assurance that design professionals will avoid repeating mistakes. In addition, building codes facilitate a simple and relatively rapid design, permit, and construction process, in which the liability of the design professional is minimized (Hamburger 2009).
Although the prescriptive criteria of model building codes are intended to result in buildings capable of providing a minimum of life safety performance, actual performance of individual building designs is not traditionally assessed. The lack of an assessment of building performance in the codes has led to a public misconception of how buildings will respond to a major seismic event. While the public may expect limited structural damage after a large seismic event, this expectation is not in accord with the intent of the building code. Historically, the intent of building code seismic provisions has been to provide buildings with an ability to withstand intense ground shaking without collapse. However even without collapse there could potentially be significant structural and nonstructural damage (FEMA 445). As the 1997 Uniform Building Code (UBC) states, the purpose of the code is “…to safeguard against major structural failures and loss of life, not to limit damage or maintain function.”

Earthquakes at the end of the twentieth century, such as the 1994 Northridge Earthquake, led to the recognition that “the level of structural and nonstructural damage that could occur in code-compliant buildings may not be consistent with public notions of acceptable performance” (FEMA 445). While the Northridge Earthquake resulted in only fifty-seven deaths, the earthquake caused an estimated $20 billion in damage, according to Pacific Earthquake Engineering Research Center (PEER). While the code was relatively successful in safeguarding against loss of life, the money and time lost from the damage and loss of function of building, was unacceptable. As a result, the engineering community has moved toward predictive methods for assessing seismic performance, and
the development of what is known in the engineering community as performance-based design (PBD).

4.3.1 Reason for Investigation

Performance-based design evaluates how a building is likely to perform, given a potential hazard. The performance, which is detailed further in Table A, can be measured in terms of deflections, plastic rotations, and nonstructural appearance, among other things. PBD was investigated to determine if it was an appropriate method to use in the design of CSP fields deployed directly on the roof of urban industrial buildings. The use of PBD allows the design professional to clearly communicate to an owner the expected performance of the rooftop solar power tower to a range of seismic events.

4.3.2 Early Investigation of Performance-Based Design

Traditionally in performance-based design, the target building performance levels have been defined as:

- Operational Performance
- Immediate Occupancy Performance
- Life Safety Performance
- Collapse Prevention Performance

Table A, on the following page, reveals the expected post-earthquake structural and nonstructural damage associated with specific performance levels.
The post-earthquake damage state of the building ranges from severe for collapse prevention performance, to very light for operational performance.

Building performance objectives are formed when selecting target building performance levels for a broad range of earthquake hazard levels. For instance in PBD, the performance objectives might dictate that a building be designed to an operational performance level for relatively minor or frequent earthquakes, and a collapse prevention performance level for major or very rare earthquakes. The earthquake hazard levels are based upon a probabilistic approach to earthquake severity and are measured in terms of the mean return periods of the earthquakes, or in other words the average number of years between events of similar severity. Table B, on the following page, provides the probabilistic earthquake hazard levels commonly used in FEMA and ASCE documents (FEMA 356).
Table B – Earthquake Hazard Levels
Source: FEMA 356

<table>
<thead>
<tr>
<th>Earthquake Having Probability of Exceedance</th>
<th>Mean Return Period (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50% - 50 years</td>
<td>72</td>
</tr>
<tr>
<td>20% - 50 years</td>
<td>225</td>
</tr>
<tr>
<td>10% - 50 years (BSE - 1)</td>
<td>474</td>
</tr>
<tr>
<td>2% - 50 years (BSE - 2)</td>
<td>2475</td>
</tr>
</tbody>
</table>

In the Earthquake Hazard Levels Table, the 474-year return period and the 2475-year return period, are designated by FEMA 356 as Basic Safety Earthquake 1 (BSE-1) and Basic Safety Earthquake 2 (BSE-2), respectively. The BSE-1 and BSE-2 earthquake hazard levels are used in the FEMA 356 to establish the basic safety objective (BSO), which is “intended to approximate the earthquake risk to life safety traditionally considered acceptable in the United States” (FEMA 356). The BSO is a performance objective that achieves the dual goals of life safety performance for the BSE-1 earthquake hazard level and collapse prevention performance for the BSE-2 earthquake hazard level. Table C, on the following page, provides a visual representation of the BSO as well as additional building performance objectives that might be desired.
Table C – Building Performance Objectives
Source: Vision 2000

As is shown in the Performance Objectives Table, the basic safety objectives outlined in the FEMA 356 are consistent with the minimum performance objectives outlined in other performance-based design standards. If the building is designed to a performance exceeding the BSO, it is termed an enhanced performance objective, and if the building is designed to a performance less that that of the BSO, it is termed a limited performance objective (FEMA 356).

After performance objectives for a building are determined collaboratively by the owner, design professionals, and building officials a preliminary building design is
developed. After the preliminary design PBD becomes an iterative process. The flow diagram for PBD is illustrated in Figure Q, shown below.

![Performance-Based Design Flow Diagram](source: FEMA 445)

Following the preliminary building design, the performance of the building is assessed, and if it meets the objectives the process is complete. If the building does not meet the objectives, the design is revised until it is able to meet the objectives (FEMA 445).

### 4.3.3 Current Investigation of Performance-Based Design

In a structure the seismic **base shear**, $V$, from an earthquake can be determined by multiplying the mass of the structure, $W$, by the acceleration at the base of the structure, $C_s$. Although earthquakes result from a rupture in the earth’s crust miles under the ground, $C_s$ considers the motion at the surface of the earth as well as the potential for
resonance in the structure. Equation 5, below, presents the seismic base shear equation from the *ASCE 7*:

\[ V = C_s W, \quad \text{Eq. 5} \]

where \( V \) = Seismic base shear (k), 
\( C_s \) = Seismic response coefficient (g), and 
\( W \) = Effective seismic weight (k/g).

The ground acceleration, \( C_s \), is based upon ground motion data from the United States Geological Survey (USGS) for a 474 year return period earthquake (BSE-1) modified by the *ASCE 7* to take into account the soil conditions and structure period (ASCE 7-05).

The structural challenge associated with deploying a CSP field on the rooftop of an urban industrial building is that the roof accelerations acting at the base of the solar power tower cannot be calculated directly from the USGS ground motion data. The *ASCE 7* does not provide procedures for calculating accelerations at floor or roof levels, and thus an alternative design guideline was needed to address the seismic design of flexibly mounted equipment (solar power tower) on a building. The *Seismic Design Guidelines for Essential Buildings*, a technical manual published by Office of the Chief of Engineers, United States Army, was found to provide a basic performance-based procedure for designing flexibly mounted equipment on a building, and will be drawn upon in the design of the solar power tower. The premise of the guideline is quantifying the accelerations at the roof from the ground accelerations at the foundation. Figure R, below, displays the relationship between the ground accelerations, the amplified roof accelerations, and the solar power tower, for an urban industrial building.
Chapter 6 of *Seismic Design Guidelines for Essential Buildings*, “prescribes the criteria for non-structural elements that must remain intact or functional after a major seismic disturbance. The provisions include the determination of the seismic forces to be applied to the elements and the determination of the deformations that the elements will withstand” (United States 1986). The military design guideline provides two performance objectives for the flexibly mounted equipment. In the event of a 50% in 50 years earthquake (72-year return period) the element will be designed for an operational performance level, and in a 10% in 100 years earthquake (950-year return period) the element will be designed for a collapse prevention performance level. In the design guideline, the 72-year return period earthquake is designated as EQ-1 and the 950-year return period earthquake is designated as EQ-2.
In order to design a solar power tower for both the 72-year (EQ-1) and 950-year (EQ-2) earthquake hazard levels, the corresponding seismic base shear for each event must be calculated. As Equation 5 (on page 37) illustrates, the seismic base shear of the flexibly mounted equipment (solar power tower) is based upon the acceleration at the base of the tower and the effective seismic weight of the tower. The effective seismic weight can be calculated by summating the weights of the structural and mechanical systems, including the working fluid in the solar power tower. However, to calculate the design acceleration at the base of the tower (roof of the industrial building), a roof response spectrum for each seismic event needs to be developed. The roof response spectrum plots the maximum design acceleration at the roof, $S_{fa}$, as a function of the solar tower period.

In order to generate the roof response spectra, the basic procedure from Chapter 6 of the *Seismic Design Guidelines for Essential Buildings* will be employed, with modifications to take advantage of up-to-date ground motion data and three-dimensional computer modeling. To clearly convey the design procedures for generating a roof response spectrum, the Digitial West Networks™ industrial building on Sacramento Drive in San Luis Obispo will serve as a case study for deploying a solar power tower.

**4.3.4 Performance-Based Design Case Study**

A basic floor plan of the industrial building on Sacramento Drive is provided in Figure S, on the following page. The figure displays the dimensioned floor plan, with a legend that designates the location of both the 8” and 12” thick masonry walls on the plan.
drawing. In addition to the floor plan, the figure also includes some of the basic design assumptions, such as material strengths, floor/roof weights, and rigid diaphragms.

The first step in developing a roof response spectrum is to generate a three-dimensional model of the building upon which the solar power tower will be deployed. For the purpose of this thesis the building design software ETABS was used for the computer modeling to run an elastic structural analysis. A key assumption of the ETABS model is that the heavily shear walled industrial building will remain elastic for both EQ-1 and EQ-2. If the building of interest is not expected to remain elastic, alternative design procedures will need to be developed. Figure T, on the following page, shows the completed ETABS model of the two-story industrial building. In addition to the masonry

**Figure S – Industrial Building Plan**

Source: Nathan Hicks
walls along the exterior of the building, gravity framing was included at the intersection of all grid lines to restrain the vertical movement of the diaphragm and allow the model to display distinct mode shapes.

Figure T – ETABS Model
Source: Nathan Hicks

After completing the model and running the analysis, the next step is to record the mass that was assigned to each level of the structure, $m_1$, and the period of all modes, $T_m$. Table D, on the following page, displays the mass assigned to both the floor level and the roof level for the case study industrial building. The mass assigned to level 1, or the floor level, is based upon a dead load of 100 psf, and the mass assigned to level 2, or the roof
level, is based upon a dead load of 70 psf. Within the model the mass of the exterior masonry walls as well as the gravity framing has been lumped into the dead load applied to the diaphragm.

### Table D – Mass Assignment
Source: Nathan Hicks

<table>
<thead>
<tr>
<th>( \frac{w_i}{g} ) (k-sec^2/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( w_1 ) / g = 69.565</td>
</tr>
<tr>
<td>( w_2 ) / g = 48.696</td>
</tr>
</tbody>
</table>

\[ \frac{w_i}{g} = \frac{\text{Mass Assigned to Level } i}{\text{(k-sec}^2/\text{ft)}} \]

Equation 6, below, presents the calculation for the mass at a given level i:

\[ m_i = \frac{w_i}{g} \]

where \( i \) = The designation of the building level,
\( m_i \) = Mass at level i (k-sec^2/ft),
\( w_i \) = Weight at level i (k), and
\( g \) = Gravity (32.2 ft/sec^2).

The six mode periods from the case study industrial building are displayed, below, in Table E. The associated mode shapes for the case study industrial building are presented on pages 43-45 in Figure U.

### Table E – Mode Periods
Source: Nathan Hicks

<table>
<thead>
<tr>
<th>( T_m ) (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( T_1 = 0.0921 )</td>
</tr>
<tr>
<td>( T_2 = 0.0834 )</td>
</tr>
<tr>
<td>( T_3 = 0.0504 )</td>
</tr>
<tr>
<td>( T_4 = 0.0345 )</td>
</tr>
<tr>
<td>( T_5 = 0.0319 )</td>
</tr>
<tr>
<td>( T_6 = 0.0194 )</td>
</tr>
</tbody>
</table>
4.0 Research

Solar Power for Deployment in Populated Areas

\[
T_1 = 0.092 \text{ SECONDS} \\
\text{FIRST MODE IN THE Y-Z PLANE}
\]

**MODE 1** (DISPLACEMENT IN THE X-DIRECTION)

\[
T_2 = 0.083 \text{ SECONDS} \\
\text{FIRST MODE IN THE X-Z PLANE}
\]

**MODE 2** (DISPLACEMENT IN THE Y-DIRECTION)
4.0 Research

Solar Power for Deployment in Populated Areas

$T_3 = 0.050$ SEC

FIRST TORSIONAL MODE IN THE X-Y PLANE

**MODE 3 (TORSIONAL DISPLACEMENT)**

$T_4 = 0.035$ SECONDS

SECOND MODE IN THE Y-Z PLANE

**MODE 4 (DISPLACEMENT IN THE X-DIRECTION)**
As Figure U illustrates, all six mode shapes have either a translational displacement (in the x or y direction) or a torsional displacement. As a result, the
computer program can display the relative amplitude of the displacements, $\phi_{im}$, at all
levels, $i$, for every mode, $m$. The modal displacement amplitudes need to be recorded as
shown in Table F, on the following page, for modes 1 through 6 at both the floor level
and the roof level for the case study industrial building.

<table>
<thead>
<tr>
<th>$\Phi_{im}$</th>
<th>Ampl. of Mode m at Level i</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Phi_{11}$</td>
<td>0.538</td>
</tr>
<tr>
<td>$\Phi_{21}$</td>
<td>1</td>
</tr>
<tr>
<td>$\Phi_{12}$</td>
<td>0.563</td>
</tr>
<tr>
<td>$\Phi_{22}$</td>
<td>1</td>
</tr>
<tr>
<td>$\Phi_{13}$</td>
<td>n/a</td>
</tr>
<tr>
<td>$\Phi_{23}$</td>
<td>n/a</td>
</tr>
<tr>
<td>$\Phi_{14}$</td>
<td>-1.302</td>
</tr>
<tr>
<td>$\Phi_{24}$</td>
<td>1</td>
</tr>
<tr>
<td>$\Phi_{15}$</td>
<td>-1.244</td>
</tr>
<tr>
<td>$\Phi_{25}$</td>
<td>1</td>
</tr>
<tr>
<td>$\Phi_{16}$</td>
<td>n/a</td>
</tr>
<tr>
<td>$\Phi_{26}$</td>
<td>n/a</td>
</tr>
</tbody>
</table>

Table F – Amplitude of Modal Displacements
Source: Nathan Hicks

In the case study, the amplitude of displacements in the x-direction are used for modes 1
and 4, the amplitude of displacements in the y-direction are used for modes 2 and 5, and
the amplitude of rotations are used for the torsional modes 3 and 6.

The amplitude values from Table F are then used in the calculation of the modal
participation factor, $PF_{xm}$, at the roof level for modes 1 through 6. Equation 7, on the
following page, presents the calculation for the modal participation factor from the
United States Army design guideline:
where \( x \) = Building level of interest,
\( n \) = Total number of building levels,
\( i \) = The designation of the building level,
\( w_i \) = Weight at level \( i \) (k),
\( g \) = Gravity (32.2 ft/sec\(^2\)),
\( \phi_{im} \) = Amplitude of mode \( m \) at level \( i \),
\( \phi_{xm} \) = Amplitude of mode \( m \) at level \( x \), and
\( PF_{xm} \) = Modal participation factor at level \( x \) for mode \( m \).

The modal participation factors at the roof level (\( x = 2 \)) for the case study industrial building are shown below in Table G.

<table>
<thead>
<tr>
<th>PF(_{xm})</th>
<th>Modal Participation Factor at Level ( x ) for mode ( m )</th>
</tr>
</thead>
<tbody>
<tr>
<td>PF(_{21})</td>
<td>1.25</td>
</tr>
<tr>
<td>PF(_{22})</td>
<td>1.24</td>
</tr>
<tr>
<td>PF(_{23})</td>
<td>n/a</td>
</tr>
<tr>
<td>PF(_{24})</td>
<td>-0.25</td>
</tr>
<tr>
<td>PF(_{25})</td>
<td>-0.24</td>
</tr>
<tr>
<td>PF(_{26})</td>
<td>n/a</td>
</tr>
</tbody>
</table>

Table G – Modal Participation Factors
Source: Nathan Hicks

The next step in developing the roof response spectra is to calculate the EQ-1 and EQ-2 spectral acceleration, \( S_{am} \), at the ground level, for every mode, \( m \). The 50% - 50 year and 10% - 100 year earthquake hazard levels chosen for the performance objectives by the US Army design guideline do not match up with the design level earthquake (BSE-1) or maximum considered earthquake (BSE-2) that are typically used by the ASCE
7, USGS, or FEMA. The BSE-1 corresponds to a 10% - 50 year earthquake and the BSE-2 corresponds to a 2% - 50 year earthquake.

Using a ground motion calculator provided on the USGS website, the spectral response acceleration at the ground level for short periods and at 1 second for both BSE-1 ($S_{DS}$ & $S_{DI}$) and the BSE-2 ($S_{MS}$ & $S_{MI}$) can be found. The ground motion calculator, as shown in Figure V, below, obtains the four spectral acceleration values after the user inputs the zip code of the building location and the site class.

![Figure V – USGS Ground Motion Calculator](image)

Source: USGS

One of the keys in converting the spectral accelerations from the BSE-1 and BSE-2 hazard level earthquakes ($S_{DS}, S_{DI}, S_{MS}, S_{MI}$) to spectral accelerations for the EQ-1 and EQ-2.
EQ-2 hazard level earthquakes is the establishment of the mean return period of the hazard levels. Equation 8, below, presents the mean return period equation from FEMA 356:

\[ P_R = -Y \ln(1 - P_{EY}), \quad \text{Eq. 8} \]

where \( Y \) = Exposure time for the desired earthquake hazard level (years),
\( P_{EY} \) = Probability of exceedance (expressed as a decimal) in time \( Y \), and
\( P_R \) = Mean return period of the earthquake hazard level (years).

After the mean return period has been obtained for EQ-1 (72 years) and EQ-2 (949 years), the spectral response acceleration at the ground level for these hazard levels can be obtained from equations in section 1.6.1.3 of FEMA 356. If the mean return period is between 475 years (BSE-1) and 2475 years (BSE-2) the spectral acceleration, \( S_{s_{i}} \), can be determined from Equation 9:

\[ \ln(S_{s_{i}}) = \ln(S_{s_{i}}^{BSE-1}) + [\ln(S_{s_{i}}^{BSE-2}) - \ln(S_{s_{i}}^{BSE-1})] \times [\frac{606 \ln(P_R)}{P_R} - 3.73], \quad \text{Eq. 9} \]

where \( P_R \) = Mean return period of the earthquake hazard level (years),
\( i = s \) or 1,
\( S_{s_{i}}^{BSE-1} \) = Spectral acceleration parameter for BSE-1 hazard level (g),
\( S_{s_{i}}^{BSE-2} \) = Spectral accel. parameter for BSE-2 hazard level (g), and
\( S_{s_{i}}^{BSE-2} \) = Spectral acceleration parameter for EQ-1 or EQ-2 hazard level (g).

If the mean return period is less than 475 years the spectral acceleration, \( S_{s_{i}} \), can be determined from Equation 10 on the following page:
\[ S_{xi} = S_{St, BSE-1} \left( \frac{P_R}{475} \right)^n, \quad \text{Eq. 10} \]

where \( P_R \) = Mean return period of the earthquake hazard level (years),

\( i = s \) or 1,

\( S_{Di, BSE-1} \) = Spectral acceleration parameter for BSE-1 hazard level (g),

\( S_{Xi} \) = Spectral accel. parameter for EQ-1 or EQ-2 hazard level (g), and

\( n \) = Value obtained from Table H, which is Table 1-2 in *FEMA 356*.

As a part of Equation 10, a value of exponent \( n \) is provided by Table 1-2 in *FEMA 356*. That table has been reproduced, below, in the thesis as Table H.

<table>
<thead>
<tr>
<th>Region</th>
<th>( S_S )</th>
<th>( S_1 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>California</td>
<td>0.44</td>
<td>0.44</td>
</tr>
<tr>
<td>Pacific Northwest and Intermountain</td>
<td>0.54</td>
<td>0.59</td>
</tr>
<tr>
<td>Central and Eastern US</td>
<td>0.77</td>
<td>0.80</td>
</tr>
</tbody>
</table>

**Table H – Values of Exponent \( n \)**

Source: FEMA 356

After the spectral accelerations have been calculated for the EQ-1 and EQ-2 hazard levels the design response spectra at the ground level of the industrial building can be generated. Figure W, on the following, displays the design response spectra for the case study industrial building.
4.0 Research

Solar Power for Deployment in Populated Areas

Figure W – Design Response Spectrum at Ground Level
Source: Nathan Hicks

Based upon the period of the building, \( T_m \), the design response spectrum can be divided into three regions. If \( T_m < T_0 \), then the building is in the constant velocity region of the design spectra and the modal spectral acceleration, \( S_{am} \), can be calculated using Equation 11 (ASCE 7-05):

\[
S_{am} = S_{xs} \left( 4 + .6 \left( \frac{T_m}{T_0} \right) \right), \tag{11}
\]

where \( S_{am} = \) Spectral acceleration of mode m (g),

\( S_{xs} = \) 5 percent damped, spectral response acceleration parameter at short periods (g),

\( T_m = \) Building period at mode m (sec), and

\( T_0 = .2(S_{DI}/S_{DS}) \).
If $T_0 \leq T_m \leq T_S$ the building is in the constant acceleration region of the design spectra and the modal spectral acceleration, $S_{am}$, can be calculated using Equation 12 (ASCE 7-05):

\[
S_{am} = S_{XS},
\]

Eq. 12

where $S_{am}$ = Spectral acceleration of mode m (g),
$S_{XS}$ = 5 percent damped, spectral response acceleration parameter at short periods (g), and
$T_S = S_{DI}/S_{DS}$ (sec).

If $T_m < T_s$ the building is in the constant displacement region of the design spectra and the modal spectral acceleration, $S_{am}$, can be calculated using Equation 13 (ASCE 7-05):

\[
S_{am} = S_{XI}/T_m,
\]

Eq. 13

where $S_{am}$ = Spectral acceleration of mode m (g),
$S_{XI}$ = 5 percent damped, spectral response acceleration parameter at a period of 1 second (g), and
$T_m$ = Building period at mode m (sec),

Using the building period for modes 1 through 6, $T_m$, and the design response spectrum shown in Figure W (page 51), the spectral acceleration, $S_{am}$, of modes 1 through 6 can be calculated. In the case study industrial building, the period of all modes was less than $T_0$, and thus the modal spectral acceleration at the ground level could be calculated using Equation 11. Table I, on the following page, displays a summary of the modal spectral accelerations for both EQ-1 and EQ-2 for the case study industrial building.
Using the modal spectral acceleration at the ground level, Sam, the modal story acceleration at the roof, $a_{xm}$, ($x = 2$) can be calculated using Equation 14:

$$a_{xm} = PF_{xm}(S_{am}),$$  \hspace{1cm} \text{Eq. 14}$$

where $a_{xm}$ = Modal story acceleration at level x for mode m (g),
$PF_{xm}$ = Modal participation factor (see Eq. 7 on page 47), and
$S_{am}$ = Spectral acceleration of mode m (g).

For each earthquake hazard level, the maximum acceleration at level x, $a_{x\text{max}}$, can be found using the square-root-of-sum-of-squares (SRSS) rule for modal combination. The SRSS rule is expressed, on the following page in Equation 15:

$$a_{x\text{max}} = \sqrt{\sum a_{xm}^2},$$  \hspace{1cm} \text{Eq. 15}$$

where $a_{x\text{max}}$ = Maximum story acceleration at level x (g), and
$a_{xm}$ = Modal story acceleration at level x for mode m (g).

A summary of the modal story accelerations, at the roof, for the case study industrial building, along with the maximum roof acceleration is presented, below, in Table J.

### Table I – Modal Spectral Accelerations
Source: Nathan Hicks

<table>
<thead>
<tr>
<th>Source</th>
<th>$P_R$ (yr)</th>
<th>$S_{x_0}$ (g)</th>
<th>$S_{x_1}$ (g)</th>
<th>$T_0$ (sec)</th>
<th>$T_s$ (sec)</th>
<th>$S_{a_1}$ (g)</th>
<th>$S_{a_2}$ (g)</th>
<th>$S_{a_3}$ (g)</th>
<th>$S_{a_4}$ (g)</th>
<th>$S_{a_5}$ (g)</th>
<th>$S_{a_6}$ (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50% - 50 year (EQ-1)</td>
<td>72.1</td>
<td>0.405</td>
<td>0.221</td>
<td>0.109</td>
<td>0.546</td>
<td>0.366</td>
<td>0.347</td>
<td>0.274</td>
<td>0.238</td>
<td>0.233</td>
<td>0.205</td>
</tr>
<tr>
<td>10% - 100 year (EQ-2)</td>
<td>949.1</td>
<td>1.101</td>
<td>0.601</td>
<td>0.109</td>
<td>0.546</td>
<td>0.997</td>
<td>0.945</td>
<td>0.745</td>
<td>0.649</td>
<td>0.633</td>
<td>0.558</td>
</tr>
</tbody>
</table>
Using the modal story accelerations at the roof a plot of the spectral roof response acceleration, $S_{fa}$, versus the period of the solar tower, $T_a$, can be developed with the aid of Figure X, which is located on the following page. Figure X plots the design magnification factor as a function of the ratio of the solar tower period, $T_a$, over the modal building period, $T_m$ (United States 1986). Equation 16 and 17, shown below, can be used in combination with Figure X in order to calculate the roof response spectrum plot for each mode:

$$T_a = (T_a/T_m)T_m, \quad \text{Eq. 16}$$

$$S_{fa} = a_{xm}(\text{Magnification Factor}), \quad \text{Eq. 17}$$

where $T_a =$ Period of solar tower or other flexibly mounted equipment (sec), $T_m =$ Building period at mode m (sec), $T_a/T_m =$ Period Ratio from Figure X, $S_{fa} =$ Spectral roof response acceleration (g), $a_{xm} =$ Modal story acceleration at level x for mode m (g), and Magnification Factor = Reference y-axis of Figure X on page 55.
4.0 Research

Solar Power for Deployment in Populated Areas

Figure X – Design Magnification Factor vs. Period Ratio
Source: United States 1986

Table K, below, illustrates the tabulation of the pertinent data required for such a plot for the first mode of the case study industrial building.

<table>
<thead>
<tr>
<th>FIRST MODE</th>
</tr>
</thead>
</table>

\[
T_m (s) = 0.0021 \\
\alpha_{20\%} (g) = 0.459 \quad 50\% - 50 \text{ year} \\
\alpha_{20\%} (g) = 1.248 \quad 10\% - 100 \text{ year} \\
\]

<table>
<thead>
<tr>
<th>$T_a/T_m$</th>
<th>0</th>
<th>0.5</th>
<th>0.8</th>
<th>1.2</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>M.F.</td>
<td>1</td>
<td>1</td>
<td>7.5</td>
<td>7.5</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>$T_a$</td>
<td>0</td>
<td>0.046</td>
<td>0.074</td>
<td>0.111</td>
<td>0.184</td>
<td>0.276</td>
</tr>
<tr>
<td>$S_{a,10% - 50\text{yr}}$</td>
<td>0.459</td>
<td>0.459</td>
<td>3.440</td>
<td>3.440</td>
<td>0.459</td>
<td>0.459</td>
</tr>
<tr>
<td>$S_{a,10% - 100\text{yr}}$</td>
<td>1.248</td>
<td>1.248</td>
<td>9.359</td>
<td>9.359</td>
<td>1.248</td>
<td>1.248</td>
</tr>
</tbody>
</table>

Table K – First Mode Spectral Roof Response Acceleration Tabulation
Source: Nathan Hicks
The tabulation shown for the first mode in Table K needs to be repeated for each modal period and modal story acceleration in order to produce a curve on the roof response spectrum for each mode. The roof response spectrum also needs to include a horizontal line intersecting the coordinate at $S_{fa} = a_{x \max}$, where $a_{x \max}$ is the maximum floor acceleration from Eq. 15. The final roof response spectrum is defined by the envelope of the aforementioned curves established by Eq. 16 and 17 as well as the horizontal line established by Eq. 15 (United States 1986). For the case study industrial building, it was necessary to develop a roof response spectrum for the service level earthquake, EQ-1, and the maximum considered earthquake, EQ-2. Figure Y, on the following page, displays the completed roof response spectra of the case study industrial building for EQ-1.
Flexibly mounted equipment at the roof level, such as the solar tower, will be designed to resist forces due to the appropriate spectral roof response accelerations, $S_{fa}$, from Figure Y, at an operational performance level. The design level force for EQ-1 will be in accordance with Equation 18 (United States 1986):

$$F_p = S_{fa} (W_p),$$  \hspace{2cm} \text{Eq. 18}

where $F_p$ = Design force applied to solar tower (force), $S_{fa}$ = Spectral roof response acceleration (g), and $W_p$ = Effective weight of the solar tower (force/g).
Figure Z, below, displays the completed roof response spectra of the case study industrial building for EQ-2.

![Roof Response Spectrum](image)

**Figure Z – Roof Response Spectrum (EQ-2)**
Source: Nathan Hicks

Flexibly mounted equipment at the roof level, such as the solar tower, will be designed to resist forces due to the appropriate spectral roof response accelerations, $S_{fa}$, from Figure Z, at a collapse prevention performance level. The design level force for EQ-2 will be in accordance with Equation 18 on page 57 (United States 1986).

If the solar power tower is to be rigidly mounted at the roof, the tower will be designed to resist forces in accordance with Equation 19 (United States 1986):
\[ F_p = a_{x\text{ max}} (W_p), \quad \text{Eq. 18} \]

where \( F_p \) = Design force applied to solar tower (force),  
\( a_{x\text{ max}} \) = Maximum story acceleration at level x (g), and  
\( W_p \) = Effective weight of the solar tower (force/g).

### 4.3.5 Conclusion on Performance-Based Design

The thesis recommends that design level forces for all performance objectives be calculated according to the modified United States Army design guidelines procedures outlined in Section 4.3.4. In order to avoid resonance in the solar power tower structure the tower should be designed such that the period of the tower, \( T_a \), is at a minimum two times the max modal period, \( T_m \).

This thesis also recommends that further exploration on collapse prevention as it relates to performance-based design. The collapse prevention research could address concerns over the expected damage that would result from the elimination of a column due to a vehicular collision, in the solar field deployed over urban areas.
5.0 CONCLUSION

From a structural perspective, the deployment of concentrating solar power fields in urban areas is a feasible goal for the immediate future. Structural challenges associated with integrating solar thermal energy in an urban environment, such as flutter, wind loading of open frame structures, and design of flexibly mounted equipment at the roof level were investigated and addressed in detail in the Research section (4.0) of the thesis. The Research section provides a foundation for future research on the structural systems deployed over urban parking lots and on top of urban industrial buildings.

More research is needed on both the physics and structural engineering side of the overall project in order to determine whether intermediate scale CSP fields are the cost-effective renewable energy solution for urban areas. Future research on the structural side of the overall project should focus on more detailed modeling of the two structural systems and eventually the construction of prototypes for both the solar power tower and the elevated truss supporting solar troughs. In addition the structural research should look into the potential challenge of transporting an extremely hot working fluid through structural members. Finally, the future structural research should determine whether the progressive collapse of columns due to a vehicular collision needs to be considered in the design of the structural system deployed over urban parking lots.
6.0 GLOSSARY

Absorption Chiller – An air conditioning device that uses a heat source to provide the energy needed to drive the cooling system. A refrigerant with a very low boiling point is heated by a fluid. As the refrigerant evaporates additional heat is taken away with it providing a cooling effect.

ASCE 7-05 – A Standard from the American Society of Civil Engineers that provides a means to determine the minimum design loads for buildings and other structures.

Base Shear – The total lateral force applied to a structure.

Bluff Body – The flow streamlines do not follow the surface of the body, but detach from it leaving a wide trailing wake.

Building Performance – The safety (qualitatively described) afforded the building occupants during and after a seismic event; the cost and feasibility of restoring the building to pre-earthquake condition; the length of time the building is removed from service to effect repairs; and economic, architectural, or historic impacts on the larger community.

Dewar Tube – A tube, with an air cavity between the contents of the tube and the environment, that provides thermal insulation.

Dynamic – The time evolution of physical processes.

Earthquake Hazard Level – Probabilistic approach to quantifying the severity of an earthquake. The hazard levels correspond to the mean return periods (the average number of years between events of similar severity) of the earthquakes.

Elastic – The linear response of the building to forces and displacements. No permanent deformation occurs.

End-user – The consumer of the energy.

Envelope – On a plot of multiple curves the envelope is established by taking the maximum y-value for every x-coordinate of the plot.

Flutter – A phenomenon that develops as the heaving (vertical) and torsional (twisting) motion of an object are unified at single frequency.
Intermediate Scale Solar Concentrators – A system capable of producing 100kW to 10MW of electricity.

Mode – A natural mode occurs when the masses at the different levels of a structure move at the same period, such that the masses all pass through both equilibrium and maximum amplitude simultaneously.

Model Building Code – A code developed and maintained by a standards organization independent of the jurisdiction responsible for enacting the building code.

Normal – Acting in a direction perpendicular to the body of interest.

Oil Crisis – A 1973 oil embargo declared by the Organization of Petroleum Exporting Countries (OPEC) in response to the US support of Israel in the Yom Kippur war.

Performance-Based Design – A design process that evaluates how a building is likely to perform, given a potential hazard.

Performance Objectives – A set of goals based upon the target building performance levels for a broad range of earthquake hazard levels.

Period – The time, measured in seconds, required for the undamped system to complete one cycle of free vibration.

Photovoltaic (PV) Cell – A device that converts sunlight directly into electricity.

Prescriptive Requirements – Explicit and long-standing directions issued by the building code in order to regulate building safety.

Reciprocating Engine – An internal-combustion engine in which the crankshaft is turned by pistons moving up and down in cylinders.

Resonance – The increase in building displacements that occurs when the frequency of seismic waves approaches the natural frequency of the building.

Rigidity – The degree of deforming ability of a solid material to an applied force.

Roof Response Spectrum – A plot of the design spectral acceleration at the roof as a function of the period of the flexibly mounted-equipment at the roof level.

Site Class – A classification of Site Class A, B, C, D, E, or F based on the site soil properties, in accordance with Chapter 20 of the ASCE 7.
Solar Concentrator Field – A system that uses lenses/mirrors and tracking systems in order to focus a large area of sunlight into a small beam. The concentrated light is then used as a heat source for a conventional power plant or is concentrated on photovoltaic surfaces.

Solar Thermal Energy (STE) – Technology for harnessing solar energy for thermal energy.

Sustainable – An energy source that is not substantially depleted by continued use and does not entail significant environmental problems.
### 7.0 ACRONYMS

**ASCE** – American Society of Civil Engineers  
**BSE** – Basic Safety Earthquake  
**BSO** – Basic Safety Objective  
**CPV** – Concentrating Photovoltaics  
**CPVT** – Concentrating Photovoltaics and Thermal  
**CSP** – Concentrating Solar Power  
**FEMA** – Federal Emergency Management Agency  
**IBC** – International Building Code  
**PBD** – Performance-Based Design  
**PEER** – Pacific Earthquake Engineering Research Center  
**PV** – Photovoltaics  
**S.E.** – Structural Engineer  
**SRSS** – Square-Root-of-Sum-of-Squares  
**STE** – Solar Thermal Energy  
**UBC** – Uniform Building Code  
**USGS** – United States Geological Survey
8.0 REFERENCES


9.0 WORKS CONSULTED


