Design of a Solar Parking Canopy Array

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Abstract

The significance of developing earth conscience technologies has never been so apparent. While the abundance of available annual sunlight is a largely underutilized energy resource, the extreme heat generated from the solar rays can also cause wear on vehicles subjected to continuous exposure. The objective of this project is to offset the energy consumption of the university while also extending the life of the university fleet vehicles, by taking advantage of the open space created by parking lots to construct solar canopy arrays. We seek to make a modular design that is easy to replicate in the future on other parts of campus. The arrays will reduce electrical demand on the municipal power grid and help to promote awareness and education for renewable energy use.

Design of a Solar Parking Canopy Array

The University of Texas at El Paso Department of Environmental Health and Safety has proposed a project to construct parking canopies fitted with a solar array. The available parking lot space is approximately twenty thousand square feet, containing seventy-two parking spaces. The entire complex consists of the following buildings and projected power consumptions: warehouse, 185 kWh; facility services, 500 kWh; university police, 10 kWh; and an auto shop using 10 kWh. This yields a projected total consumption of about 704 kWh. The desired output of the photovoltaic panel array should offset approximately twenty to thirty percent of the projected consumption, putting the minimum desired electrical production at around 141 kWh.

Figure 1: *Aerial View of Physical Plant Complex*

Site Plan Reconfiguration

When first considering the layout of the Physical Plant parking lot, the desired configuration for the parking canopies was that which best allowed the panels to follow the sun from east to west and maximize the number of hours in full, unobstructed daylight. Given the available open space and low building profiles, the only restriction was to allow for a proper turning radius for a semi-trailer to clear the parked vehicles.

Solar Panel Selection

The panels initially sought by the team were those with the highest ratings in both efficiency and power. The constraints of the site dimensions as well as the desired output of seven twenty-five kilowatt arrays meant it was necessary to maximize the power per square footage. Given these factors, it became clear only panels with a power rating near two hundred kilowatts would be able to produce the desired electrical output within the allotted space. The following panels were given the most serious consideration for inclusion in the final design.

HIT Sanyo 195W Panels were first looked at for their additional performance in hot climates. They are designed to utilize both sides of the panel by exposing the back face to reflected sunlight. These panels achieved gains of five to thirty percent more power production per square foot above the stated power rating (SANYO Energy (USA) Corp, 2010). However they were ultimately not chosen because the additional performance rating is dependent on sunlight reflecting off a high albedo surface, which does not include our parking lot's paving surface.

The SunPower E18/225W Panels were analyzed for their increased efficiency and power production. The design of these panels increases power production by placing the

electrical contact gridlines on the backside of the monocrystalline silicon solar cells (SunPower Corporation, 2010). This configuration completely exposes the cells to sunlight and eliminates losses attributed to shading from the gridlines. It also permits enlarging the contact gridlines. This minimizes the resistivity losses found within the thinned gridlines of other panels that are compensating for shade loss (SunPower Corporation, 2008). Finally the backside placement of the electrical contacts means the panels are all black. This added some aesthetic appeal to the technical attraction, as the panels act as black ″mirrors, ″ softly rendering the images of clouds floating by. Yet this model was also abandoned once the following were discovered.

SunPower E20/327W Panels were selected for their superb efficiency and power production capabilities. The anti reflective coating provides for more capture of sunlight and minimizes the directional dependence for maximum absorption (SunPower Corporation, 2011). This means rotating the panels would provide an insignificant increase in electrical production. Also the panels are designed with a back layer of silicon dioxide and metal on oxide layer that reflects unabsorbed light back to the solar cells (SunPower Corporation, 2008). Finally, the low voltage temperature coefficient gives these panels greater power output in high temperature conditions (SunPower Corporation, 2011). All this increased efficiency allows for maximum energy production per square foot, which means fewer panels are needed for the same output. This will reduce cost initially on installation and ultimately on maintenance over the life of the panels. The total daily output will be approximately 164 kW, or 1,082 kWh with average insolation and accounting for an efficiency variance of about thirty percent.

Structural Design

Material selection was a major aspect in the structural design. Steel was the first structural material to be considered; yet producing a structure out of aluminum has its benefits as well. Per Conservatek, an aluminum structure company, aluminum is corrosion resistant, does not rust, self-heals if scratched, has design flexibility, and is lightweight, which would reduce the assembly and installation costs. While all these characteristics were beneficial for the task, we were too inexperienced to design an aluminum structure. Steel, on a per weight basis, is also much less expensive than aluminum, is much heavier to counteract uplift from wind, and was chosen as the primary structural material.

Table 1

Variables Used to Calculate Main Wind Force Resisting System Design Pressure

Wind Load Variables		**ASCE 7-10 Reference***
Risk category	I	Table 1.5-1
Basic wind speed, V	105 *mph*	Figure 26.5-1C
Wind directionality factor, K_d	0.85	Table 26.6-1
Exposure category	Which ever results in maximum load	Section 26.7.4.1
Topographic factor, K_{zt}	1	Section 26.8.2
Gust effect factor, G	0.85	Section 26.9.1
Velocity pressure exposure coefficient, K_h	1.65	Table 27.3-1
Velocity pressure at mean roof height h, q_h	39.58 *psf*	Section 27.3.2
Net pressure coefficient, C_N	1.2	Figure 27.4-4
Net design pressure, p	40.38 *psf*	Section 27.4.3

**From* (ASCE, 2010)

Note: It is clear this is an overdesign, but given extreme weather conditions occurring in greater frequency, designing closer to worst case seems more practical for 25 year life (Rogash, 2011).

The wind load, equivalent to 40.38 psf per ASCE 7-10, was determined to be the controlling load in our design (see Table 1 above and Appendix A and B). While the weight of the structure itself would help lessen the uplift caused by these aerodynamic forces, our goal was to minimize costs by choosing the lightest steel members, while still maintaining stiffness. Steel joists addressed those two characteristics, maintaining an adequate moment of inertia and a low weight. Two rows of bridging were required for our span as shown on Figure 2.

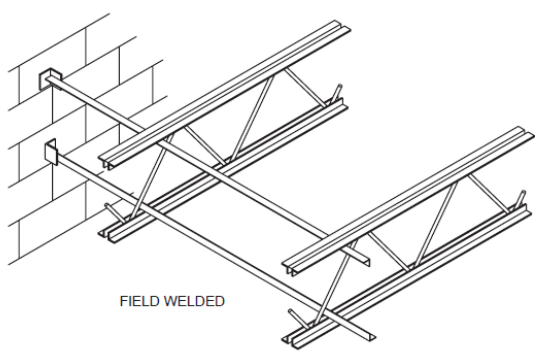

Figure 2: *Typical Bridging Set-up for Steel Joists*

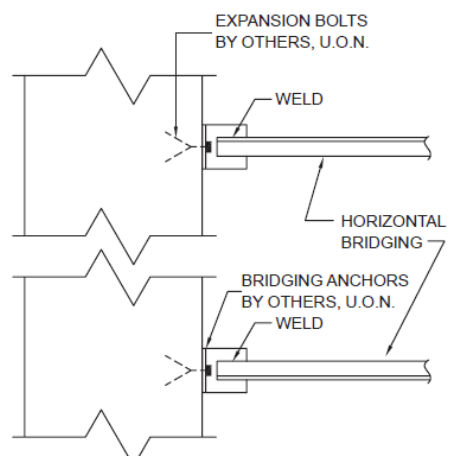

Figure 3: *Bridging Connections*

The Steel Joists Institute makes it clear that the bridging should not be welded onto the joist web members, and to not hang any equipment whatsoever, including mechanical or electrical equipment, from the joists. The joists must be welded to the end girders as shown on Figure 3. A minimum bearing length of 2.5 inches is also required for the joists seat, as shown on Figure 4.

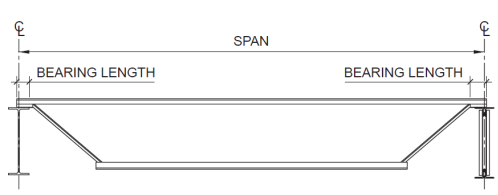

Figure 4: *Typical Steel Joists Configuration*

The selection of the supporting members was based on the capability of handling the induced bending moment while also checking for shear as well. The program StaadPro V8.1 was used to analyze our structure with American Institute of Steel Construction (AISC) method with the chosen members. Once the analysis was performed, we realized that the deflection was too high. It was decided to design for a maximum deflection of one inch. The deflection of one inch was chosen due to the fact that our structure was going to be supporting solar panels. It was uncertain how much bending the solar panels could handle due to deflection before they crack. Being conservative, the one-inch aggregate deflection was chosen.

We then began the iterative process of the selecting different members for the girders. The column at the end span where the pickaxe is to be located was causing major deflection. The column chosen was angled at 12 degrees and was causing the whole structure to rotate. After many attempts to solve this issue, it was finally decided to place two columns spaced 36 inches apart from center to center. This reduced our deflection by 40% and still kept the aesthetics of the pickaxe and the desired angled beam-column. Once the desired deflection was achieved, the selected members were still able to handle the bending moments and shear. Spot checks were checks were performed on random members to make sure the nominal bending moment and nominal shear were higher the induced bending moments and shear. Refer to Appendix B for spot check calculations.

Following the AISC method for designing the base plate connections from the column to the footing the base plate size was chosen. A plate two inches larger on all sides

was chosen to allow for bolts to be connected and not shear off. The recommended design for structures with uplift by the AISC is shown on structural plans.

Architectural Design

The preliminary design was based off the University of Texas at El Paso's logo, which is a pickaxe. We took the basic idea from the shuttle stops on campus to incorporate into the parking canopy. Figure 5 below shows the first designs as they were originally rendered on Google Sketch Up.

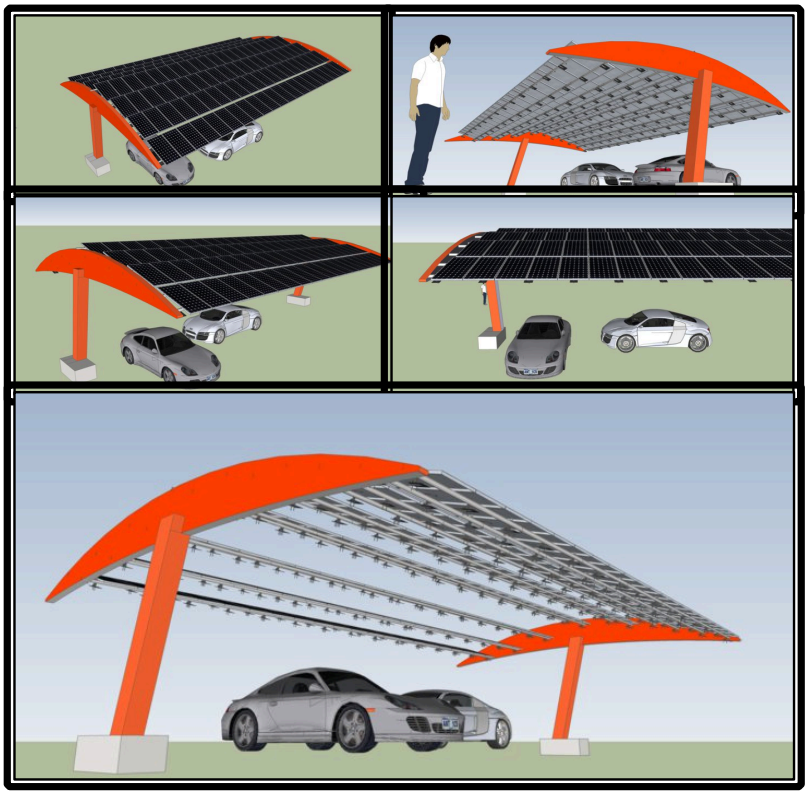

Figure 5: *Preliminary Design*

In reconsidering the overall aesthetics, several aspects of modern design were looked to for inspiration. The raw inner works of mechanics on display, such as exposed mullions and structural members of a building, or gears on a watch, make for clean modern design. Figure 6 shows some examples that shed light onto the design.

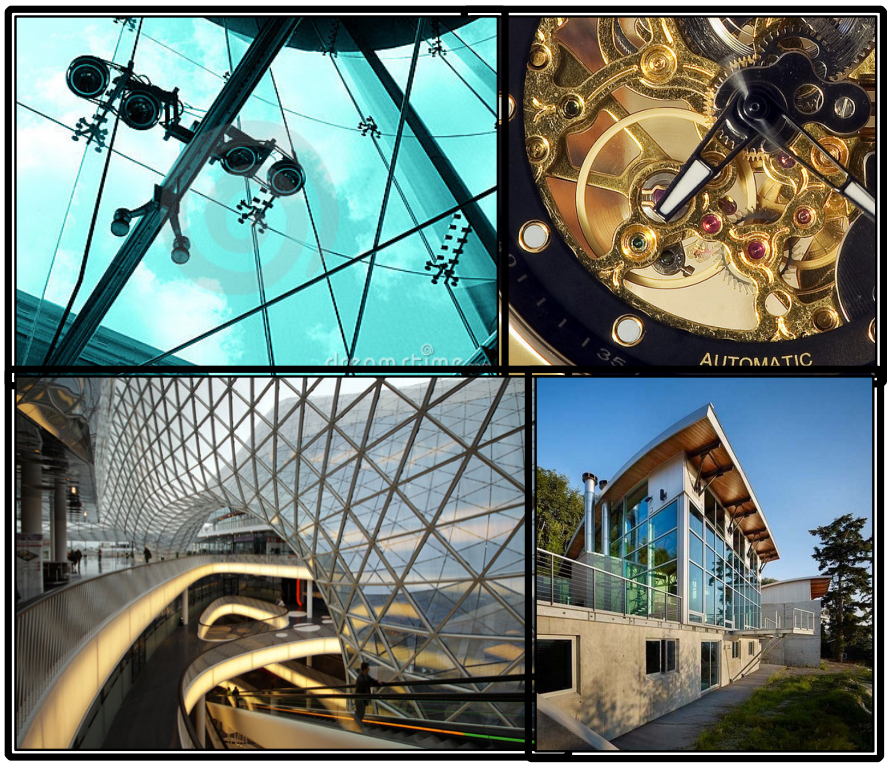

Figure 6: *Elements of Design*

The second design tried to incorporate a daily panel rotation and seasonal shift. Figure 7 shows the design concept for the rotating pickaxe, which was ultimately abandoned for its mechanical/maintenance costs, as they were deemed unnecessary due to the outstanding power production capabilities of the panels.

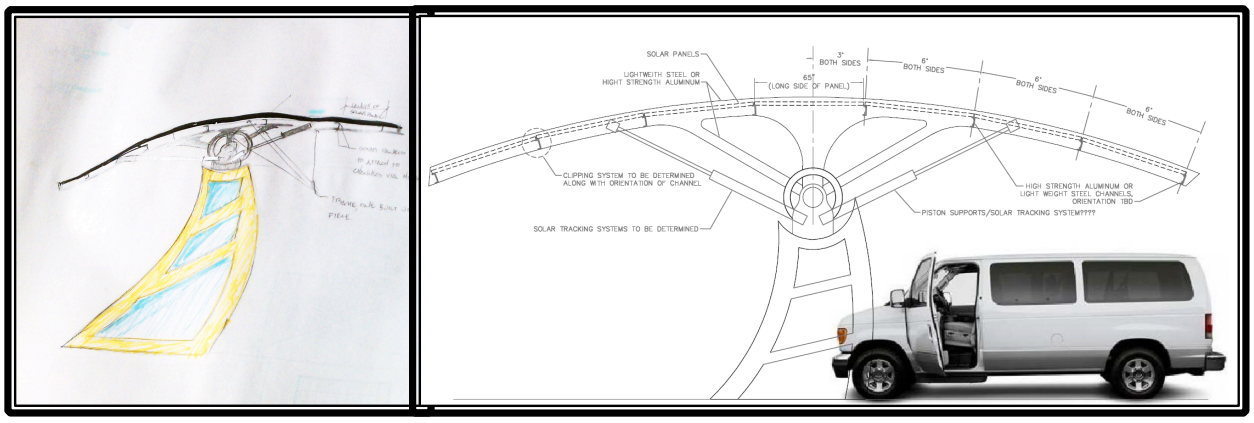

Figure 7: *Second Design Concept*

The latest design integrates mix of materials. It also incorporates a bhutanese symbol typically found in the architecture throughout the campus, shown in Figure 8.

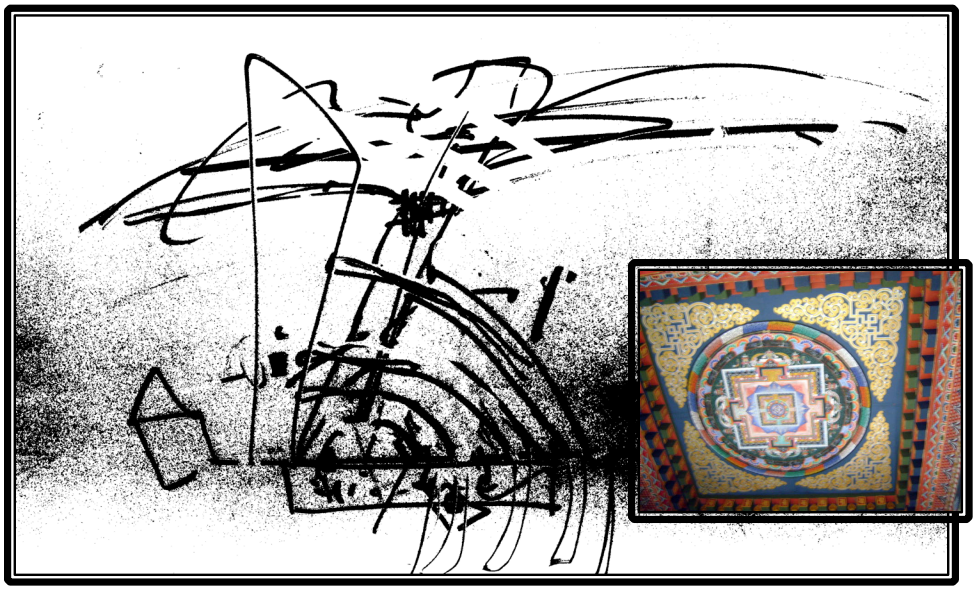

Figure 8: *Latest Pick Design Concept*

The steel canopy frame is supported by a cement column as seen in the Revit renderings in Figure 9. However during the design process, it was found to be an economically unfeasible and impracticle structure. The final architectural design will be presented during the presentation on December 2nd.

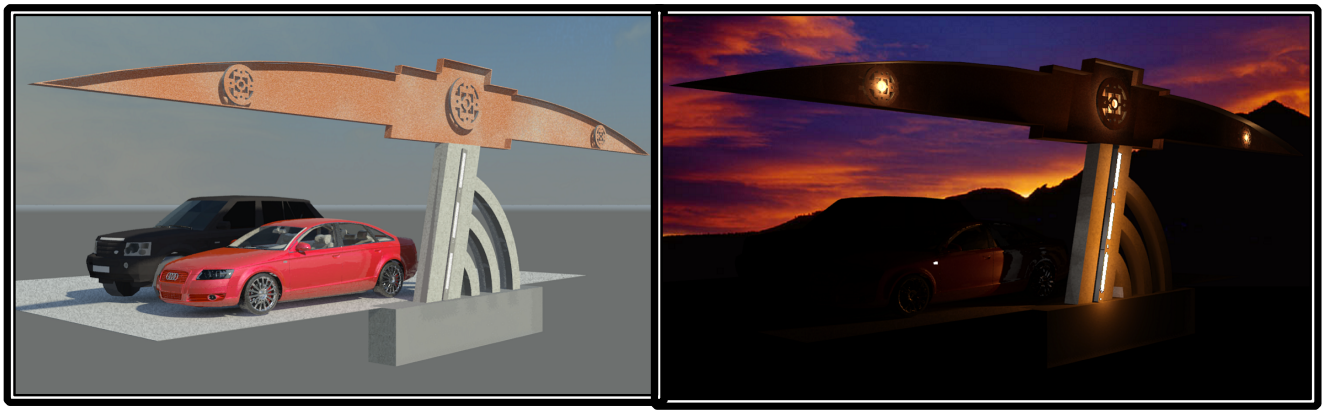

Figure 9: *Latest Rendered Pickaxe Design*

Cost Analysis

Looking at the estimated cost of the project, we analyzed the approximate price of labor, materials, and equipment, and yielded the values noted below in Table 2. Based on these values, the annualized return was calculated to be $33592.85. The rate of return and payback period were also found to be 3.3 and 30 years, respectively.

Table 2
Project Cost Estimate

Division	Labor	Materials	Subtotals	Equipment	Other	Grand Total
General Conditions	28,260.00	3,162.01	208,446.11	4,440.00	11,885.00	256,193.12
Sitework	1,634.66	3,030.54	1,284.76	407.78		6,357.73
Concrete	242.51	167.66				410.17
Steel	9,400.00	127,101.58		595		137,096.58
Equipment			5,042.00			5,042.00
Special Construction			570,900.63			570,900.63
Electrical			24,000.00			24,000.00
Grand Total	39,537.17	133,461.79	809,673.50	5,442.78	11,885.00	1,000,000.24

Conclusion

This assignment was conceptualized and redesigned numerous times. We trust that our meticulous attention to detail has yielded a safe structure serving the purpose it was intended, and if implemented could exceed the desired results.

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Appendix – A

Snow Loads:

- Ground, p_g

For $3500ft$ < elevation < $5000ft$

$$p_g = 5lb/ft^2$$

- Flat Roof, p_f

$$p_f = 0.7C_eC_tI_sp_g,$$

where

Exposure factor for Category B: $(C_e) = 0.9$

Thermal factor for unheated and open air structures: $(C_t) = 1.2$

Importance Factor for risk level I: $(I_s) = 0.80$,

then

$$p_f = (0.7)(0.9)(1.2)(0.80)(5lb/ft^2) = 3.02lb/ft^2$$

Wind Loads – Main Wind Force Resisting Structures:

- Velocity pressure exposure coefficient, K_h

$$K_h = 1.03,$$

or

$$K_h = 2.01\left({}^{z}/_{z_g}\right)^{2/\alpha} = 2.01\left({}^{15ft}/_{30ft}\right)^{2/7} = 1.65, \qquad \therefore K_h = 1.65$$

- Topographic factor, K_{zt}

$$\frac{H}{L_n} = \frac{100ft}{1200} = 0.083 < 0.2, \qquad \therefore K_{zt} = 1$$

- Velocity pressure at mean roof height h, q_h

$$q_h = 0.00256K_hK_{zt}K_dV^2$$

where

Wind directionality factor: $K_d = 0.85$

Basic wind speed for Risk Category I: $V = 105mph$,

then

$$q_Z = 0.00256(1.65)(1)(0.85)(105)^2 = 39.58\ lb/ft^2$$

- Net design wind pressure for MWFRS, p

$$F = q_hGC_N$$

where

Gust-effect factor: $G = 0.85$

Net pressure coefficient: $C_N = 1.2$,

then

$$p = (39.58\,lb/ft^2)(0.85)(1.2) = 40.38\,lb/ft^2$$

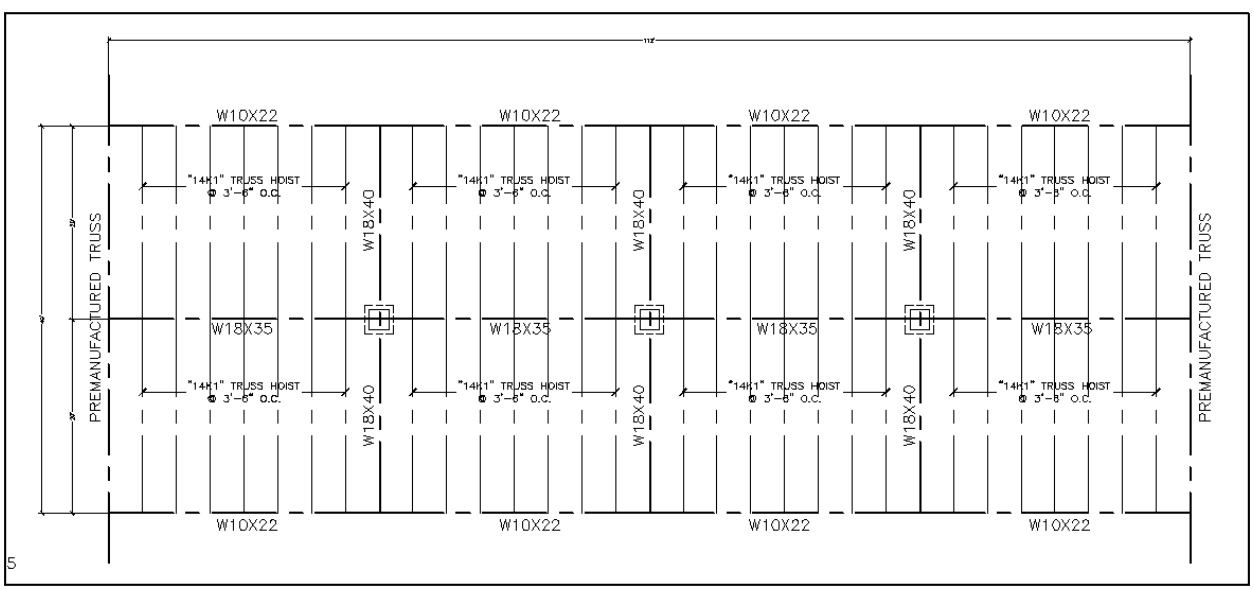
Appendix – B

Loads

Dead	2.34 *psf*
Snow	3.10 *psf*
Wind	40.38 *psf*
Total	157.16 *psf*

Frame Spot Check

	M_u (*kip·ft*)	V_u (*kips*)	Z_{req} (in^3)	ΦM_n (*kip·ft*)	ΦV_n (*kips*)	Z (in^3)
W10X22	–	8.93	17.75	–	73.44	26
W18X35	–	17.86	35.50	–	159.30	66.5
W18X40	181.18	31.83	–	217.39	169.16	–

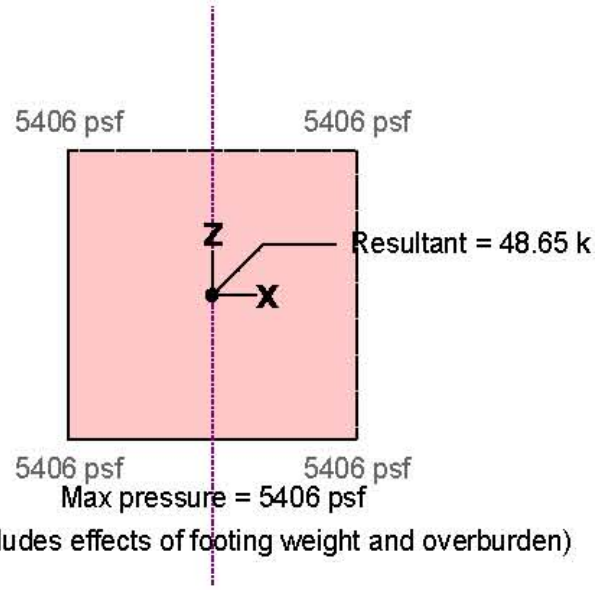


Appendix – C

Footing Stability Check

Factored Loads

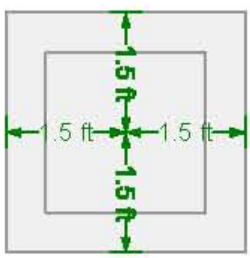
Axial Force	45.8 k
Moment X	0 in·k
Moment Z	0 in·k
Shear X	0 k
Shear Z	0 k
Overburden	0 psf
Footing Weight	1.35 k
Pedestal Weight	1.5 k



(Gross: Includes effects of footing weight and overburden)

Bearing Pressure

q_{allow} = 5500 psf – q_{gross} = 5406 psf W



Overturning

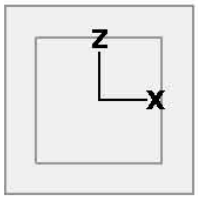
W_f = 1.35 k (weight of footing)

W_p = 1.5 k (weight of pedestal)

$F_{ob} = q_{overburden} (A_{ftg} - A_{ped})$ = (0 psf) [(9 ft²) - (4 ft²)] = 0 k

F.S. against overturning about X axis is infinite (no applied moment)

F.S. against overturning about Z axis is infinite (no applied moment)



Sliding

W_f = 1.35 k (weight of footing)

W_p = 1.5 k (weight of pedestal)

$F_{ob} = q_{overburden} (A_{ftg} - A_{ped})$ = (0 psf) [(9 ft²) - (4 ft²)] = 0 k

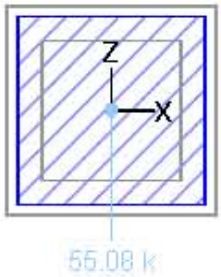
$F_{resist} = C_f (W_f + W_p + F_{ob} + P) + F_{passive}$ = (0.40) [(1.35 k) + (1.5 k) + (0 k) + (45.8 k)] + (0 k) = 19.46 k

F.S. against sliding in X direction is infinite (no applied force)

F.S. against sliding in Z direction is infinite (no applied force)

Appendix – C (con't)

Footing Stability Check



Punching Shear Check (ACI 318-08 Ch 11.12.1.2, 11.11.2.1)

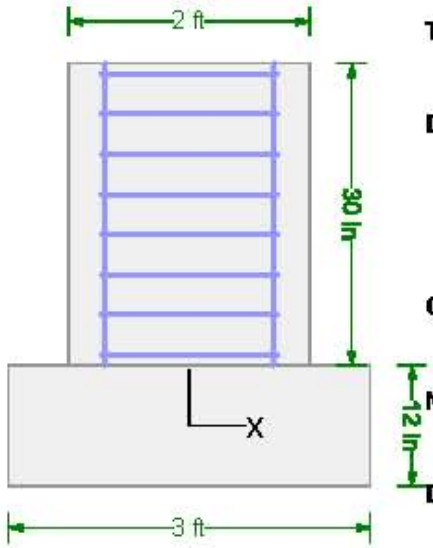
$P_{punching} = P_{total} + W_p - P_{perimeter} = (64.12\ k) + (2.1\ k) - (55.08\ k) = 11.14\ k$

$$v_u = \frac{V_u}{b_o\ d} + \frac{h_{vx}\ M_{ux}\ e_z}{J_x} + \frac{h_{vz}\ M_{uz}\ e_x}{J_z}$$

$$= \frac{(11.14\ k)}{(72\ in)(8.38\ in)} + \frac{(0.40)(0\ in{\cdot}k)(18\ in)}{(40456\ in\ ^\wedge\ 4)} + \frac{(0.40)(0\ in{\cdot}k)(18\ in)}{(40456\ in\ ^\wedge\ 4)}$$

$= 18.47\ psi$

$\phi v_n = 164.3\ psi \quad - \quad v_u = 18.47\ psi$ W



Compressive Force Transfer (Footing) (ACI 3

$\phi P_{nb} = 1569\ k \quad - \quad P_{ub} = 66.22\ k$ W

Tension Force Transfer (ACI 318-08 15.8.1.2

$\phi T_n = 190.1\ k \quad - \quad T_u = 0\ k$ W

Dowel Development (Footing) (ACI 318-08 12

$P_{us} = 0.0$ (concrete bearing is sufficient: $\phi P_{nc} - P_{ub}$)

ratio = $P_{us} / \phi P_{ns} = (0\ k) / (137.3\ k) = 0.0$

$l_d = 9\ in \quad - \quad l_{dreq\_dow} = 0\ in$ W

Compressive Force Transfer (Pedestal) (ACI

$\phi P_{nb} = 1092\ k \quad - \quad P_{ub} = 66.22\ k$ W

Minimum Steel Across Joint (ACI 318-08 15.8

$A_s = 3.52\ in^2 \quad - \quad A_{smin} = 2.88\ in^2$ W

Dowel Development (Pedestal) (ACI 318-08 1

$P_{us} = 0.0$ (concrete bearing is sufficient: $\phi P_{nc} - P_{ub}$)

ratio = $P_{us} / \phi P_{ns} = (0\ k) / (137.3\ k) = 0.0$

$l_d = 27\ in \quad - \quad l_{dreq\_dow} = 0\ in$ W

Axial/Flexure (ACI 318-08 Ch 10)

$\phi P_{nmax} = 776.9\ k \quad - \quad P_u = 66.22\ k$ W

$$\text{BiaxialUnity} = \frac{M_{nz}}{M_{0z}} + \frac{M_{nx}}{M_{0x}}\left[\frac{1-c}{c}\right] = \frac{(0\ in{\cdot}k)}{(983.2\ in{\cdot}k)} + \frac{(0\ in{\cdot}k)}{(983.2\ in{\cdot}k)}\left[\frac{1-(0.650)}{(0.650)}\right] = 0.0$$

Shear Check (ACI 318-08 11.2.1.2)

$$\phi V_c = \phi\, 2\left[1 + \frac{N_u}{2000\ A_g}\right] \lambda \sqrt{F'_c}\ b_w\ d = (0.750)\ 2\left[1 + \frac{(66.22\ k)}{2000\ (4\ ft^2)}\right](1.0)\sqrt{3000\ psi}\ (24\ in)(20.44\ in) = 42.62\ k$$

ial (k)

$\phi V_n = \phi V_c + \phi V_s = (42.62\ k) + (50.58\ k) = 93.2\ k$

$\phi V_n = 93.2\ k \quad - \quad V_u = 0\ k$ W

Shear Check (ACI 318-08 11.2.1.2)

$$\phi V_c = \phi\, 2\left[1 + \frac{N_u}{2000\ A_g}\right] \lambda \sqrt{F'_c}\ b_w\ d = (0.750)\ 2\left[1 + \frac{(66.22\ k)}{2000\ (4\ ft^2)}\right](1.0)\sqrt{3000\ psi}\ (24\ in)(20.44\ in) = 42.62\ k$$

$\phi V_n = \phi V_c + \phi V_s = (42.62\ k) + (50.58\ k) = 93.2\ k$

$\phi V_n = 93.2\ k \quad - \quad V_u = 0\ k$ W