INSTALLATION DESIGN OF THE AVALANCHE IMPACT PYLON FACILITY ALTA, UTAH

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ABSTRACT

In order to research avalanche forces, an impact pylon was installed at Alta Ski Area, Little Cottonwood Canyon, Utah. The pylon is located beneath Mount Baldy in an avalanche path which releases frequently both naturally and as a result of Alta Ski Lift's operational control program. It consists of one primary member standing six meters normal (perpendicular) to the slope, which is the impact face, and three braces down slope. On the impact face of the pylon there are six pressure gauges and two load cells with pressure plates to register avalanche forces. For the design of the footing, the computer modeling program; SAP 80[5] was used to calculate the base reactions (shear and moment) resulting from the avalanche design forces. Based on these reactions, the pylon's leading three base plates were secured by a common reinforced concrete footing. The rear base plate is anchored to a boulder with epoxy anchor bolts.

INTRODUCTION

The avalanche impact pylon facility was undertaken as a joint Japanese/U.S. effort

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involving the National (Japan) Institute of Earth Science and Disaster Prevention, the USDA Forest Service National Avalanche Center, the Department of Civil Engineering at the University of Utah and the Center for Snow Science at Alta. Tombstone (the chute beneath Mount Baldy) was selected as the location for the pylon facility primarily because of the high frequency of artificially released avalanches. A situation where avalanches could be set off "at will" was desirable so that very high rate data recorders could be started just prior to the avalanche event.

The pylon was designed and fabricated in Japan before being shipped to Salt Lake City, Utah. It was built from Japanese steel similar to the all-purpose carbon steel (ASTM A36) used commonly in the U.S. The minimum yield stress is 234 MPa (34 ksi) and the tensile strength is 400 MPa (58 ksi).

The pressure gauges on the impact face are phase lagged in order to determine avalanche velocity as well as record avalanche pressure. The possibility of placing strain gauges on the pylon in addition to the pressure gauges and load cells is being investigated. In addition to the basic research products associated with avalanche dynamics, the data collected from the pylon facility will be used in Japan for the design of future permanent avalanche defense structures. Permanent avalanche defense structures are, by design, impacted by avalanches. These installations typically cost in the tens of millions of dollars.[6]

The focus of this paper is the design of the footing and the installation of the pylon.

PRELIMINARY CALCULATIONS

A design avalanche loading case was provided by the Japanese researchers in the form of force per unit length of the pylon. This is shown in Figure 1.

Computer Analysis

In order to determine the base reactions resulting from the avalanche loading, a three dimensional computer structural modeling program, SAP 80, was used. Several parameters of the original loading case were varied to represent a variety of different possible avalanches.

The computer program used required input designating the bases of the pylon to be either fixed (no translation or rotation possible) or pinned (rotation possible but not translation). In reality, the fixity of the bases is somewhere in between these ideals. For comparison, two configurations for each loading case discussed below were run, one with the bases fixed and one pinned.

The baseline loading was established using a typical midwinter snowpack depth of two meters (see fig. 1). To compare the effects of an early season avalanche, loading cases were run with snowpack depths of zero and one meter. These loading cases are shown in figures 2 and 3.

Because the two struts that angle out to the sides also receive impact forces, loading was applied to them equal to one half the magnitude of the loading on the main impact member at the same height. This was based on the fact that the diameter of the side struts is close to half the diameter of the main impact member.

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[6] Decker, J.D. and R. Decker, 1990, "Permanent Avalanche Defense Structures in Japan," Proceedings of the International Snow Science Workshop, Bigfork, Montana, p.200.

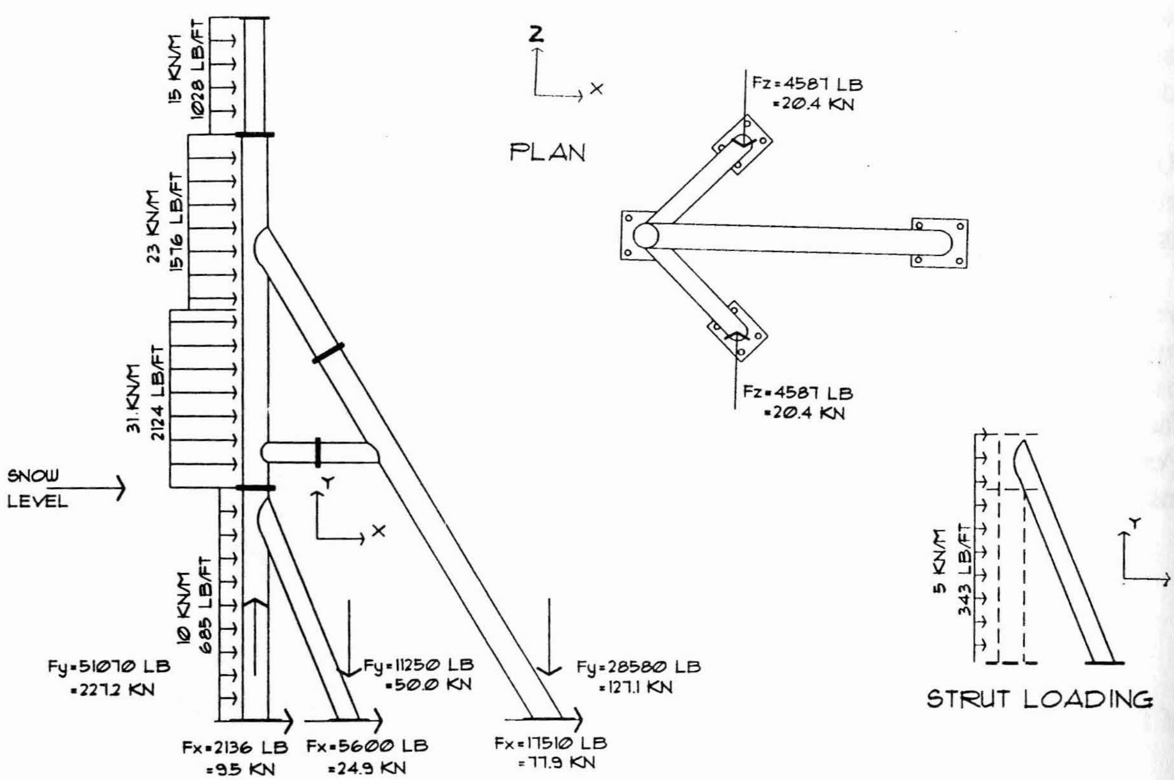

Figure 1- Baseline Loading Case and Reactions

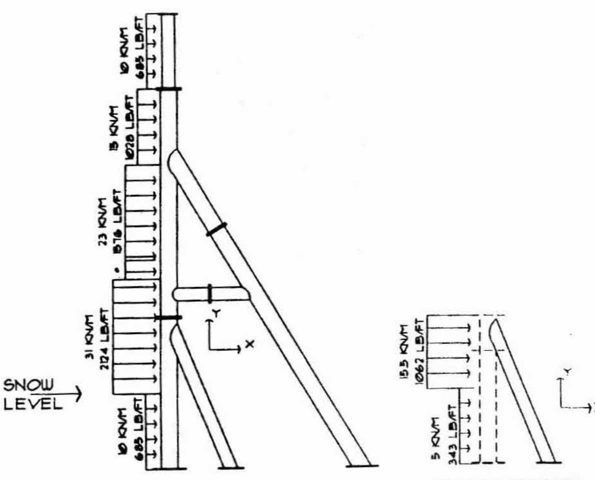

Figure 2- Loading Case for Snow Depth at One Meter

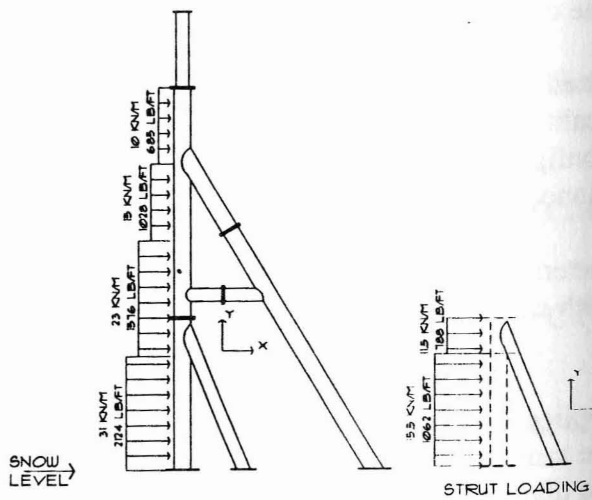

Figure 3- Loading Case for No Snow at Pylon Location

One concern was that if the snowpack depth was too great, the instruments would be buried. To investigate the possibility of extending the height of the pylon and raising the instruments, an additional loading case was executed for a late season avalanche with the snow depth at three meters and a one meter extension added to the top of the pylon. Figure 4 illustrates these conditions.

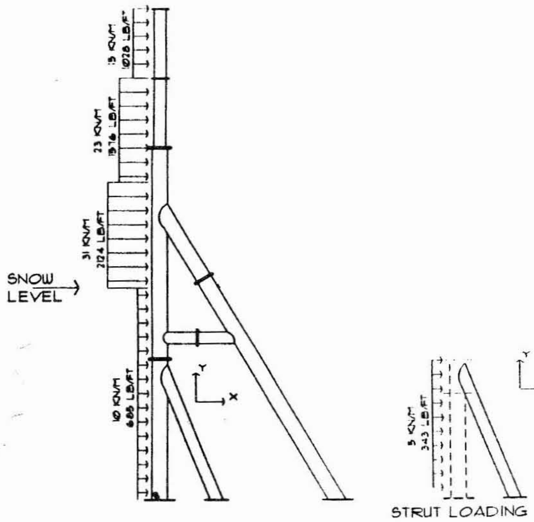

Figure 4- Loading Case for Snow Depth at Four Meters

The original load case was also varied for an oblique angle of impact. This configuration was investigated for the various snowpack depths cited above with an off axis impact angle of 20 degrees.

Results

The results of the computer analysis were very interesting in that there was not one loading case in particular that was the worst case scenario. The variations in snow depth, base fixity and impact angle mentioned above resulted in a total of 16 loading combinations that were considered. Each load case produced up to 24 base reactions that were compared (forces and moments in the x, y, and z directions for each of the four base plates).

There was a curious relationship between snowpack depth and base reactions; 80% of the controlling (largest) reactions were produced by the two extremes of zero and three meter snow depths. As the snow depth increased, the overturning moment also increased because the majority of the force was applied at a greater distance from the bases. One half of the design governing reactions resulted from a snow depth of three meters at the pylon. The critical member for this case was the main impact member. In contrast, an additional 30% of the governing reactions came from the loading cases where the pylon was not buried in the snowpack at all. In these cases, the loading on the side struts was the heaviest. The critical members for these cases were the struts.

As for the variation in impact angle, two thirds of the governing reactions resulted from an impact angle of 20 degrees, the remaining third were generated from a head-on impact. The reason for this was partly because a head-on impact only produced a moment in one direction (about an axis across the slope) and forces in two directions (perpendicular to and down slope). When the impact forces were applied at an angle, moments and forces were produced in all three directions.

Allowable Stress Calculations

In order to determine if a one meter extension to the pylon was possible, the length of the cantilever (the unbraced portion of the main impact member) was studied. The results of calculations showed that a one meter extension would create stresses in the cantilever three times greater than the allowable bending stress.

For the loading case based on a snow depth of three meters and a 20 degree impact angle, the stresses (combined axial and bending) were the greatest in all members of the structure. Upon further investigation, it was discovered that every member of the pylon was overstressed for this scenario by 200% to 300%.

The possibility of the pylon failing under certain circumstances was acknowledged and found to be acceptable. The pylon is a test facility and the chance of injury or loss of life as a result of structural failure of the pylon during an avalanche is negligible. This raised the question of whether to design a footing that would survive an avalanche event that could be expected to damage or destroy the pylon or to design a footing that would most likely fail with the pylon during such an extreme event. Additional design constraints included an upper limit placed on the amount of concrete to be used for the footing. These constraints were primarily economics, but there was also a concern based on environmental (visual) impact.

The more extreme loading cases required roughly 60% more concrete than the baseline loading case. The footing designs based on the extreme loading were rejected.

Creep and Glide

Forces produced by the creeping and gliding motion of the snowpack could potentially affect the design of the footing. This issue was studied using the Swiss Guidelines for Design of Supporting Structures (Mellor, 1968). Forces were estimated and found to be roughly one-tenth the magnitude of the avalanche design forces. The time scales of the application of creep and glide forces relative to avalanche forces are dramatically different. Hence, the two force systems were not considered additive. Consequently, the influence of creep and glide was not considered further in the footing design.

FOOTING DESIGN

Concrete

Figure 1 shows the base reactions used in the final design of the footing. With the main pylon in tension and all three of the struts in compression, an overturning moment of 386 kN-m (285 ft-kips) is produced at the rear base plate. This was the main criteria for the footing design;

shear and flexure were also taken into consideration.

The most common shear failure mechanism for footings is two way shear. This failure scenario would result from an inclined crack forming in a conical or pyramidal shape around the base plate. Failure occurs when the cone "punches" through the concrete. In order to prevent shear failure, there must be enough shear capacity in the concrete to resist the stresses; this requires the development of a large enough shear cone. Calculations indicated that if separate footings were used for the main impact member and the two small struts, the required shear cones would overlap. For this reason, it was decided to design a single monolithic footing for these three leading members. This footing design is illustrated in figures 5 and 6.

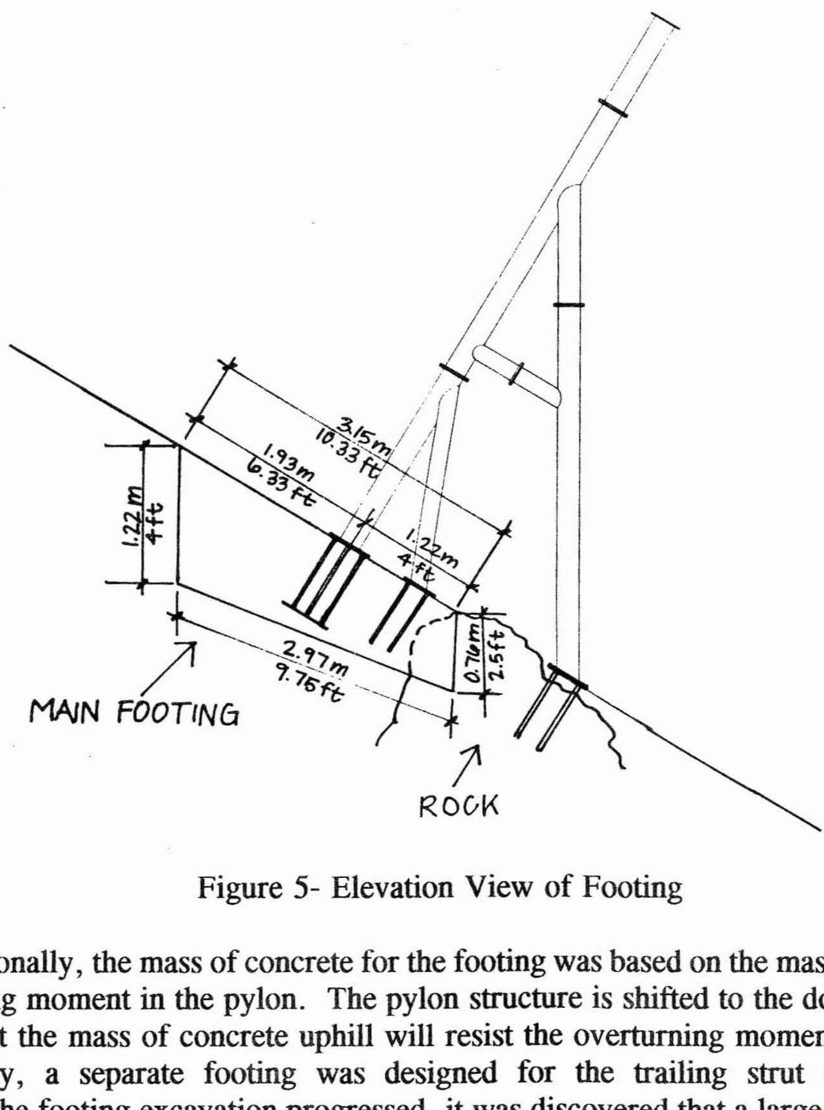

Figure 5- Elevation View of Footing

Additionally, the mass of concrete for the footing was based on the mass required to resist the overturning moment in the pylon. The pylon structure is shifted to the downhill side of the footing so that the mass of concrete uphill will resist the overturning moment.

Initially, a separate footing was designed for the trailing strut (in compression). However, as the footing excavation progressed, it was discovered that a large boulder existed in the trailing strut bearing area. Because the trailing strut is in compression, it was decided to secure the base plate to this boulder with anchor bolts. Refer to figures 5 and 6.

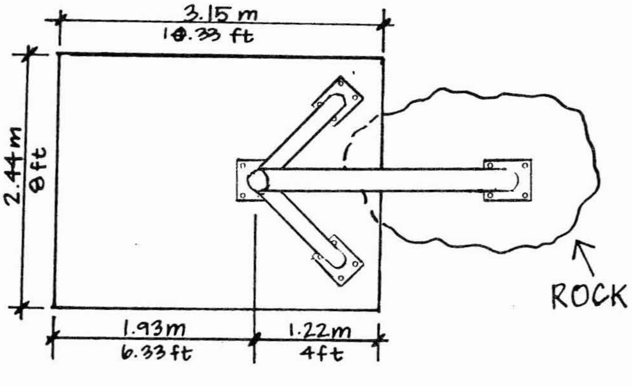

Figure 6- Plan View of Footing

Reinforcing Steel

The ACI (American Concrete Institute) code was used as a basis for selecting reinforcing steel for the main footing. It was determined that the concrete had more than adequate shear and flexural capacities to resist all calculated reactions. Reinforcing was required only for temperature and shrinkage- to minimize cracking and to tie the structure together. The amount of steel was based on a ratio of reinforcement area to gross concrete area of 0.0018. The rebar was designed to fit like a cage inside the footing. Precut number 5 bars (5/8 inch diameter) were selected for their ease of handling.

Bolts

Standard ASTM A307 bolts, one inch in diameter were chosen to secure the base plates. According to calculations, smaller bolts would have been adequate, but as a factor of safety and for possible future use, the bolt holes in the base plates of the pylon were enlarged and one inch bolts were used.

In order to develop as large of a shear cone as possible, a steel plate matching the base plate of the main impact member was bolted to the bottom of the anchor bolts. This also ensured that the bolts would remain parallel and match the base plate exactly during the placement of the pylon on the footing. The bolts of all three base plates extended almost to the bottom of the footing, leaving just enough concrete cover to prevent corrosion. Taking into consideration the possibility of the bolt threads being damaged during placement of the pylon, 15.2 cm (6 inches) of the bolts extended out of the footing. These top threads of the bolts could be cut off and discarded if the potential damage occurred.

INSTALLATION

Excavation

The pylon is located on a 28 degree slope approximately 70 meters uphill from a cabin which is to be the instrumentation site. Due to the steepness of the slope and the possibility of scarring with heavy machinery, the footing site was excavated by hand. The location was selected partly because there was already a depression from an old mining excavation and a sizable boulder that could be incorporated in the footing. The material removed was a mixture of skree and soil.

Once the footing was excavated and it was decided to anchor the rear base plate to the boulder, part of the boulder had to be removed because the top surface was too high. This was accomplished through blasting and chipping.

Steel Placement

One concern regarding the anchor bolts was that they would settle in the concrete to an angle other than the necessary slope normal angle. There were a total of 16 bolts that had to be oriented precisely or else the task of placing the pylon with its pre-drilled base plates upon these bolt sets would be impossible.

The solution to this problem called for the bolts to be rigidly secured at the correct angle and spacing before the concrete was poured. Three steel plates identical to the pylon's leading base plates were cut and fixed to the anchor bolts at what would become the surface level of the footing. The steel plate used to develop the shear cone for the leading base plate would fix the bolts at the bottom. Channel sections were used to weld the three sets of bolts together to create one rigid piece that could be placed inside the steel reinforcing cage. The entire rigid assembly of bolts and steel plates was tied to the reinforcing steel at an angle normal to the slope before the concrete was poured.

Concrete Placement

Concrete with a 28 day strength of 20.7 MPa (3000 psi) was ordered. Two trucks were required to haul 9.17 cubic meters (12 cubic yards) from the Salt Lake Valley, at an elevation of 1371 m (4500 ft), a distance of 25.7 km (16 miles) to the site at 2896 m (9500 ft). The trucks had to stop approximately 60 meters short of the excavation. The concrete was pumped this distance from the trucks to the footing site. A vibrator was used to ensure there were no voids in the concrete.

Pylon Placement

A Bell 214 helicopter transported the 750 kg (1650 lb) pylon from the village of Alta, Utah at 2652 m (8700 ft) to the site. It was intended to place the pylon directly on the bolts but this proved too difficult. Due to an exceedingly long sling line, when the helicopter maneuvered slightly left or right, the swinging of the pylon was exaggerated and created a dangerous situation for the construction personnel trying to place it on the bolts. Consequently, the helicopter had to set the pylon just below the bolts. The pylon was secured by cables until it could be installed and secured on the bolts.

In order to place the pylon on the bolts, it had to be raised by the amount that the bolts were extending out of the footing and moved up hill until the bolt holes coincided with the bolts. Wedges were driven under the base plates to raise the elevation and timbers stacked underneath to temporarily support the pylon. This process was repeated until the pylon was high enough to be dragged over the bolts. A hand winch was then used to pull the pylon uphill until the bolts and bolt holes coincided. After the damaged bolt threads were filed, the leading three base plates were bolted down.

Rock Anchor

The rear base plate was anchored to the boulder. Holes were drilled in the rock and the bolts were secured inside with epoxy glue.

CONCLUSION

There was no single loading case that governed the footing design. The design criteria, overturning or base reactions, varied with snowpack depth and impact angle. Different loading conditions caused the critical member in the pylon structure to shift from the main impact member to the struts. This is not typical of structures under gravity loads, where one case usually governs.

ACKNOWLEDGMENTS

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