HEAVY SNOW FAILURES IN THE SIERRA NEVADA MOUNTAINS OF CALIFORNIA

by

Charles C. Coen, Ph.D.[1]
and
David A. Crane, M.S.,S.E.[2]

INTRODUCTION

This paper will describe in some detail four case histories of recent building failures in the Sierras. These four failures, along with many other commercial, residential, and residential deck failures, have led to a recently adopted snow ordinance by Placer County for heavy snow design. Many of the concepts developed in the Placer County Ordinance were incorporated in the International Conference of Building Officials Ad Hoc Snow Committee recommendations, which form the basis for the 1988 Edition of the Uniform Building Code Snow Load Provisions.

STRUCTURAL FAILURE AT BLYTH ARENA, SQUAW VALLEY, CALIFORNIA - MARCH 1983

Blythe Arena was the site of the 1960 Winter Olympics. The arena roof was a cable supported steel structure covering approximately 6,874.85 square meters (m^2). Steel purlins at 3.35 meter (m) centers spanned to 45.72 meters (m) cantilevered box girders at 9.75m on center. Each girder had a 24.08m back leg and an 18.29m column extending from 10.06m high concrete piers. The cantilevered girders were supported at 18.59m and 37.19m from the piers with high strength galvanized gables tied back to the columns which were tied to the girder back legs at the ground level. The roof decking was galvanized steel with 11.43 centimeters (cm) deep enclosed cells at 15.24cm on center, originally used as ducts for heated air to melt snow on the roof.

Practice at the time of construction required a design live load of only 4.79 kilopascals (kPa). This was further reduced to 2.39 kPa for the arena only, with the addition of a snow removal system which used excess heat generated from ice rink refrigeration units to heat the roof and exterior ground trenches, with the idea that the snow would slide off the roof and melt on the ground. This was allowed under the assumption that the heating system would be used continuously and in the event of a mechanical failure no single storm would produce more than 4.79 kPa of snow. To insure structural integrity the design was checked to insure no steel would be stressed beyond its yield point with 4.79 kPa on the roof.

Shortly after the Olympics the exterior ice rinks were removed. Without the demand on the refrigeration units, they were turned off or replaced with smaller units. A back up system was installed to heat the roof and trenches. There is question as to the adequacy of this replacement system. For many years the snow was removed by hand using labor from the California Youth Authority. Some time in the early to mid-seventies the United States Forest Service added a sprayed-on fiberglass roofing material, apparently to stop leakage occurring at the roof decking joints.

In 1977 a feasibility study was compiled and the arena was sold to a private company in 1982. The study reviewed the structural alternatives available to the owner, but did not consider destruction or additional support to bring the building up to code. There was agreement among the consultants that the building was in good shape; they recommended $280,000.00 (American dollars) in structural repair, of which most went to repair of spalling concrete and the leveling of the ice rink's foundation. Nothing was done to increase the load capacity. The new owners added a ramp extending from the ground to the roof over the heated ground trench. This ramp allowed access to the roof for small snow removal equipment. The snow was removed in this manner up to the time of failure.

---

1 County of Placer, Building Department, 11424 "B" Avenue - Dewitt Center, Auburn, California 95603
2 David A. Crane & Associates, Civil and Structural Engineering, 3327 Longview Drive, Suite 200, North Highlands, California 95660

Based upon information received from the Placer County officials, the failure of March 1983 occurred when the roof structure on the side adjacent to the snow ramp experienced a failure in one girder leading to collapse of three bays on that side. Sometime prior to collapse a cantilevered box girder buckled at the 18.59m location near the single cable support, the yoke on the cable connection failed, and a chain reaction of overstressed members resulted.

Built in 1959 for the 1960 Winter Olympics, the arena was structurally under-designed based upon current design snow loads. Current design codes require a snow load of 11.49 kPa for Squaw Valley at this location. This load has been established by averaging recorded snowfall over many years. It is clear that the original design would not stand up to loads of this magnitude. The snow removal concept never was a long term economical solution to keep the loads within the design range, and it has not been established if it ever worked as intended. In any case, it was not used for most of the arena's life. The addition of the fiberglass roofing material greatly increased the frictional resistance between the snow and the roof, further reducing any chance of snow sliding off the roof. The addition of the ramp and removal of snow by mechanical equipment further increased the loads on the roof locally and helped to reduce structural member capacity.

It seems that the arena was originally designed as a temporary structure and should have been used as such. Subsequent to the 1960 Winter Olympics, it should have been disassembled, as the probable malfunction of the snow removal system to keep the actual roof load within working stress limits could easily cause catastrophic failure, as ultimately occurred. It is also apparent that the snow load criteria of 4.79 kPa which was in effect at time of design for all the Olympic Village construction was totally inadequate.

TRUCKEE SHOPPING CENTER SNOW DAMAGE

This building has 16.76 meter (m) span gable roof trusses of 4 in 12 slope. The ridge of the building runs east to west. The north eave has an inside gutter for purposes of a fascia for commercial signage. The roof is sheathed with standing seam metal roofing.

This project was completed by July 1981. In the spring of 1982 significant snow damage was observed. The south facing roof slope had approximately .76 to .91 meters of snow in a band about .91 to 1.52 meters wide over the eave, while the rest of the slope was largely free of snow. All the vents and snow splitters on the south half of the roof were sheared off.

The end wall parapets were plumb and straight for most of the width of the building. However, at the canopy ends at the north eave, the parapet walls are leaning out from 5.08 to 10.16 centimeters (cm). The problem of the failure of parapet ends over the interior gutters appears to be due to lateral ice pressure. The code does not address this phenomenon.

The north slope of the roof had about .91 meters of snow covering most of the slope. The front canopy had snow cantilevered out from .91 to 1.83 meters (snow cornice). The roof snow depth at the front of the building was about 1.52m. The fascia at the north eave had business signs mounted on the face. Some of these signs showed evidence of damage, due most likely to snow falling off the front of the building or snow removal equipment damage. The front fascia was beginning to lean out and split in places due to the continuous lateral loading of the snow and ice pushing out and off the front portion of the roof.

The roof drains were frozen and much ice had formed alongside the downspouts and columns. The ice movement had probably sheared off the roof drains but they were not observable. There was evidence of water (ice) standing inside the bottom of the canopy. The installation of heat strips might have eliminated the freezing of the ice in the gutter and downspout, but this method is not reliable due to power outages. The design of the front of the canopy was responsible for most of the damage occurring here. The roof slopped down at a constant slope to within .61m of the front, where it slopped back up to form a valley. This valley was the gutter that the ice and snow mass had bridged across, so that even if the gutter and downspout had heat strips, the snow would still bridge across. If no ice formed in the gutter, the pressure of the snow on the front of the gutter (the building fascia) would probably be comparable.

Snow cornicing, and especially ice loads, could produce loading higher than normal design loads. The snow on the roof at the time of investigation was largely dry, powdery and fairly light, except for heavier snow and ice at the gutter. However, when snow and ice bridges, the loads are not uniform. The ice acts as a beam spanning from point to point. The snow on the canopy is bridging from the fascia in front to about one-half or two-thirds of the distance back to the wall, leading to localized loading. The code mentions use of increased loading for this type of condition, but no specific design data was included in the proposed 1988 Edition of the Uniform Building Code.

The snow splitters on the south facing slope could not split the snow because the standing ribs on the roof prevent the snow from moving laterally. Most of the snow splitters and roof vents have sheared off. The 1988 Uniform Building Code will provide design criteria for lateral pressures on roof obstructions such as vents and chimneys. The sheared vents will probably occur again if the same conditions exist.

The roof slope over the canopy should have been continuous to the front fascia and not broken back up to form a gutter. This would have eliminated the problem at the canopy and the split fascia. The constant slope would eliminate bridging across the canopy. An additional improvement would be to allow the heated building air access to the inside of the canopy, which would prevent re-freezing of water flowing down the roof when it hits the unheated portion of roof over the canopy.

Due to the hazard of overhanging snow on the front canopy, the first row of parking was abandoned during the winter snow season to minimize accidents or vehicle damage due to falling snow. One item which should be emphasized in any snow load code or ordinance is a prohibition of any public ingress or egress and especially, required exits under the eave line of a building in heavy snow areas.

STRUCTURAL FAILURE - TWO-STORY, WOOD FRAME DWELLING - MARCH 1983

This failure was the total collapse of a wood frame, two-story dwelling located at Serene Lakes, California, in March 1983. There were several accumulative causes for this total failure, as follows.

The builder installed an 8.89cm x 33.66cm beam at the front porch; a 15.24cm x 53.34cm Glue Laminated beam was required. Shear walls located at the front building wall were required to have hold-downs installed in the concrete foundation at the panel edges; the hold-downs were not installed.

The structure was designed for a roof snow load of 9.78 kPa. The design ground snow for this subdivision, based upon a snow study completed in 1985 by Charles C. Coen and David A. Crane, should be 20.59 kPa with a roof snow load of 16.47 kPa.

The under-sized front porch beam failed thereby pulling the porch roof down. Rotation of the front wall shear panels occurred, due primarily to the absence of the required hold-downs at the panel edges. The corners of the exterior walls separated and the walls were shoved outward by the weight of the roof snow. The failure was progressive and took place over a period of four or five days.

At the time of the failure there was an estimated total snow load on this wood framed structure of 73.93 kPa. The roof area of the structure was 139.36 square meters (m^2). The structure experienced a total failure, with all exterior walls being forced outward and the majority of the roof snow ending up inside the building.

HOFBRAU CONDOMINIUMS, SQUAW VALLEY, CALIFORNIA - POSTTENSIONED CONCRETE SLAB FAILURE

This building is a four-story condominium complex. The first story is Type I-FR construction with a posttensioned concrete slab over a parking garage. The concrete slab is designed to support two three-story towers of wood frame construction, with each tower containing thirty-two (32) residential condominium units. Between the two towers, on top of the slab, is a one-story wood frame club house. The total area of the posttensioned slab is 3,344 square meters (m^2). The current design snow load for this slab is 17.96 kilopascals (kPa).

There was a punching shear failure of the slab at one of the interior columns. Subsequent investigation, after the initial failure, indicated numerous voids and cracks in the slab and column capitals. At the time of the failure there was 14.84 kPa of snow on the slab. One of the wood frame towers had been completed and was ready for occupancy.

The primary causes of the failure of the 241.3 millimeter (mm) thick slab were; numerous voids and cracks due to cold joints, poor concrete, improper aggregates, and improper reinforcement steel and tendon placement. The failure was accelerated and aggravated by the heavy snow and heavy freeze/thaw cycles with water penetrating the cracks in the slab, which was not waterproofed, and then freezing.

Corrective measures involved; the slab beneath the one completed wood frame tower was de-tensioned, repaired, and re-tensioned with additional beams and columns added to reduce all slab spans. The remainder of the posttensioned slab was demolished and replaced with a new slab. Waterproofing was added to the exposed slab.

REFERENCES

American National Standards Institute, American National Standard (1982), Minimum Design Loads for Buildings and Other Structures.

Brown, John Webster (1974), Snow Load Study-Truckee Quadrangle.

Coen, Charles C., Ph.D. (1985), Placer County Snow Load Ordinance.

Crane, David A., M.S.,S.E. (1985), Placer County Snow Study.

International Conference of Building Officials (1987) Accumulative Supplement, Alternate Snow Load Design pp 71-82; and Special Snow Load Consideration pp 82-87.

Perry, Dale C., Ph.D. (1985), Roof Snow Loads.