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by D.A. Taylor

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Snow loads for the design of cylindrical curved roofs in Canada, 1953–1980

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Since 1975 recommended design loads on cylindrical curved roofs in the Commentaries on Snow Loads in National Building Code documents have undergone significant changes. This paper presents data and some of the background to the changes, and discusses the problems that have arisen with the recommendations published in the 1977 Commentary. The intent is to make recommended design snow distributions and loads as realistic as possible without undue complication. Further improvements are suggested.

Depuis 1975 les charges de calcul sur les toits en voûte cylindrique dans les Commentaires sur les charges dues à la neige dans les documents qui accompagnent le code national du bâtiment ont subi des changements significatifs. Cette communication présente des données et une partie de l'historique des changements, et discute des problèmes soulevés par les recommandations publiées dans le Commentaire 1977. Le but est de rendre le calcul de la répartition et des charges de neige aussi réalistes que possible sans complications injustifiables. D'autres améliorations sont suggérées.

Can. J. Civ. Eng., 8, 63–76 (1981)

Introduction

Arch-shaped structures are used frequently in Canada for farm and storage buildings and sometimes also for arenas, community halls, and aircraft hangars. Apart from dead weight, the major loadings are usually from snow and wind. Snow will be considered in this paper. The external geometry of a structure, its exposure to wind and sun, and its thermal characteristics are most relevant to the way snow accumulates on it.

The snow loading recommended for the design of arches in Commentaries¹ on Part 4 of the National Building Code of Canada (NBC) 1975 (p. 126) was derived for moderately curved long-span roofs such as those covering arenas. The applicable rise-to-span ratios were largely between $\frac{1}{8}$ and $\frac{1}{10}$. Lately smaller, higher curvature arches have become common, and because the load distributions recommended in 1975 seemed to be unrealistic a re-evaluation of loading on arches was undertaken. This paper presents a short history of the development of snow loading for the design of arches in the NBC, some of the material used in recommending reassessment of the 1977 Commentary (NBC 1977a), and suggestions for further improvements in 1980.

History of design snow loads on arch shapes

Snow on the ground

During the past 20–30 years an attempt has been

¹Henceforth referred to as the Commentary, by year, in this case the 1975 Commentary. This paper is concerned only with commentaries and supplements on snow load requirements in the National Building Code of Canada.

made in Canada to develop a rational approach to the prediction of design snow loads for roofs. Data were required on the snow accumulations for disparate areas of the country. As records of snow depths on the ground had been kept for many years, primarily for predicting spring run-off to rivers and reservoirs and for flood control, it was reasonable to use them as a basis for snow loads on roofs.

Although surveys of snow on roofs have been carried out since 1956, it is still not possible to take into account in calculations many factors such as the thermal properties of roofs, shape, shelter, surface texture, and wind. Roof snow loads are still determined by a simple procedure that uses ground loads obtained from measurements of snow depth on the ground. Factors relating roof to ground accumulations have been derived and are in common use (NBC 1977c).

The basic ground load for code purposes is the value obtained by calculating the ground snow depth having a 30-year return period (i.e., a 1-in-30 chance of being exceeded in any 1 year, or the chance of being exceeded once in 30 years, on the average) from many years of records at meteorological stations across Canada. From this and an assumed specific gravity of about 0.2, the 30-year return snow load, g , was computed. The reasoning behind the choice of a specific gravity of 0.2 was that the 1-in-30-year snow load would probably be composed of old snow with a specific gravity (SG) of about 0.3 and a large fall of new snow with an SG of about 0.1 (Boyd 1961).

The weight of the maximum rainfall occurring in a 24 h period in January, February, or March during a 5–31 year measurement period was added to the

30-year return ground snow (Boyd 1961; NBC 1977b) with the stipulation that the contribution of rain would not be greater than 50% of the total load.

From the predictions made in this manner for some 480 stations across Canada, tables of the 30-year return ground snow loads at 646 stations were established by Boyd and accepted for use in the NBC (1977b). These have been used by most designers as the basis of all roof snow calculations in lieu of better local information on actual roof loads.

Snow on roofs

Many factors make the accumulation of snow on roofs different from the snow on the ground. Climate clearly affects roof and ground snow; shelter from the wind, orientation, shape and height of building, drainage, heat loss, and smoothness and colour, all affect snow on roofs and on the ground to different degrees. It is therefore not surprising that roof loads predicted using the 30-year return *ground* snow loads are not the 30-year return *roof* loads. A universal direct index relating roof to ground loads does not exist! Experience confirms, however, that the C_s coefficients, commonly used in Canada to relate roof loads to the 30-year return ground loads, usually give acceptable and conservative designs (C_s = roof load/ g). From the available data the C_s values calculated in this way appear to be better than those obtained by using the ground snow recorded at the same time as that on the roof. Though a method taking into account the local climate—melting, rainfall, and especially wind velocity and duration, would be preferable, a climatological analysis is not warranted for each building, so the much simpler and cruder C_s values, based on g only, are used. These values are regularly reviewed; new data, practical experience, and case studies of failures are taken into account in this process.

Arch loading in the National Building Code and the Commentary on Snow Loads

The following section will show how the recommended design loads on arch shapes in the NBC have changed since 1953. In the NBC 1953 and 1960 designers were required to consider uniformly and nonuniformly distributed loads on arch shapes, although no assistance in choosing distributions was given until 1960. At that time it was specified that the nonuniform load must be 125% of the uniform load and located on either side of the crown, with zero load on the opposite side. Designers were allowed to use the slope reduction factor for slopes between 20 and 60° in the 1953 Code and 30 and 70° in the 1960 Code for uniformly distributed loading.

Although there was no supplement on snow loads issued in 1960, one was published giving wind load coefficients. The natural progression was to write one on snow loads for publication with the 1965 Code

FIG. 1. Snow loading on curved roofs in 1965 (Schriever and Peter 1965). For roofs exposed to the wind on all sides, all values of C_s marked with an asterisk may be reduced by 25%.

(Schriever and Peter 1965). As there was little information available on snow loading on arches in Canada, except for loads recorded after the tragic collapse of the Listowel Arena in February 1959 (Morrison *et al.* 1960), the distribution in use in the USSR (Schriever and Otstavnov 1967) was recommended (Fig. 1). This recognized the importance of unbalanced and nonuniform drifts deposited by wind on either side of the roof. The NBC remained silent on the use of the slope-reduction relation for the uniform or case I loading (Fig. 1), and by default it was no longer recommended.

In July 1965, shortly after its publication, the arch loading published with the 1965 Code was revised to that shown in Fig. 2. The distribution of loads on the Listowel Arena and on other arches, as shown in Figs. 3 and 4 (excerpted from Schriever *et al.* 1967), convinced the code committees that experience in the USSR was not directly applicable and that this limited Canadian experience should be used. It was apparent that solar radiation, sliding, and scouring action of the wind could leave one side of a roof clear and the other heavily loaded.

FIG. 2. Snow loads on curved roofs as revised July 1965, and in the 1970 Commentary (Schriever *et al.* 1970) and 1975 Commentary. For roofs exposed to the wind on all sides, all values of C_s marked with an asterisk may be reduced by 25%.

The arch loading in the 1970 Code (Schriever *et al.* 1970) and 1975 Commentary remained unchanged from that used in 1965 (Fig. 2), with no allowance for slope reduction, but evidence for further change was accumulating.

Current observations on arch loading

The provisions until 1975 were meant for fairly flat, long-span arches. Gradually, however, it became evident that the short-span, steeper variety was becoming very common and that provisions were inappropriate for it. Many arches were almost semi-circular in cross section and it was not reasonable to assume the full triangular load because large drifts would not cling to their steep sides, especially to the sides of steep arches built on vertical side walls.

The unbalanced triangular drift used in the 1975 Commentary (Fig. 2) attained a maximum of twice the 1-in-30 year ground load at the steepest part of the arch where it was indeed most likely to slide off. Because of the apparent limitations of the provisions in the 1975 Commentary, a pilot study of snow on arch-shaped

buildings was started during the winter of 1975–76 in the Ottawa area. Although this study was far from complete, on the basis of judgement and the few new data obtained, new distributions that seemed to give reasonably conservative accumulations on arch-shaped buildings were derived for the 1977 Commentary.

Distributions on cylindrical arch-shaped structures, 1977

Uniformly distributed load, case I

A uniformly distributed snow load of $0.8g$ (where g is the ground snow load²) was recommended for most locations, or $0.6g$ if the roof was exposed to wind on all sides and was to remain free of shelter, including that due to trees or new buildings, for the life of the structure. The slope reduction factor, β , as defined below, allowed for the reduced accumulation of snow on slopes over 30° . Hence the distribution used in the 1977 Commentary was as shown in Fig. 5.

$C_s = 0.8$, or 0.6 (if exposed), where the slope reduction relation for β is

$$\beta = 1.0 \quad \text{for} \quad 0^\circ < \alpha_x \leq 30^\circ$$

$$\beta = 1.0 - \frac{\alpha_x - 30^\circ}{40^\circ} \quad \text{for} \quad 30^\circ < \alpha_x \leq 70^\circ$$

$$\beta = 0.0 \quad \text{for} \quad 70^\circ < \alpha_x \leq 90^\circ$$

and where α_x is the slope in degrees at any point on the curved surface.

Unbalanced load, case II

The widely used triangular distribution (Fig. 2) was found, as mentioned earlier, to be unrealistic in some cases and in need of improvement. It was largely insensitive to the geometry of the roof and spanned the half width, giving a maximum load of $2.0g$ at the edge regardless of curvature, edge slope, or width of structure. In addition, in locations with moderate-to-high ground snow loads the distribution resulted in quite improbable drift shapes on small arches, drifts which by protruding upward far above the crown of the arch were incompatible with the likely pattern of air flow over the building (Fig. 6). As noted, it put the deepest snow on the steepest part of the arch where it would be most likely to slide off.

Observations by the author during the first 2 years of the pilot study pointed to the need for a more realistic distribution to describe unbalanced loading. Data from a preliminary study of the collapse of Quonset-type

²In the 1980 NBC g is replaced by S_0 to avoid confusion with $g = 9.81 \text{ m/s}^2$.

Date 2 January 1964

Location St. Catharines, Ontario

ROOF AND BUILDING DESCRIPTION

Type of structure and use: Large arch roof with canopy on one side.
Bowling alley.

Building heated: Yes Roof insulated: Yes

Shelter conditions: Building exposed.

ROOF SNOW LOADS (psf)

NBC (1960): 30

Av Obsd: 10

Max Obsd: 35

GROUND SNOW LOADS (psf)

NBC (1960): 38

Av Obsd: --

SNOW LOAD DISTRIBUTION

Unbalanced load on arch with maximum depth along eave (snow fell under calm conditions, producing 8 in. uniform snow cover, and was redistributed by a 15 mph wind the next day as shown)

Date 3 January 1964

Location London, Ontario

ROOF AND BUILDING DESCRIPTION

Type of structure and use: Large arch roof with raised section on one side near middle of building. Curling rink.

Building heated: - Roof insulated: -

Shelter conditions: -

ROOF SNOW LOADS (psf)

NBC (1960): 34

Av Obsd: 20

Max Obsd: 75

GROUND SNOW LOADS (psf)

NBC (1960): 42

Av Obsd: 20

SNOW LOAD DISTRIBUTION

Unbalanced load on arch
(Information not complete)

FIG. 3. Case histories of snow on two large-diameter arches in Ontario (Schriever *et al.* 1967).

buildings and the snow-drifting model experiments reported by Isyumov (1971) confirmed that a change was desirable. The photographs (Figs. 7-12), taken (except for Fig. 11) within 30 km of Ottawa (where $g = 2.9$ kPa or 60 psf), illustrate drifting mechanisms

accommodated by case II or the unbalanced loading of Fig. 13. Because of the poor quality of the photograph, Fig. 11 has been touched up slightly to highlight the shape of the drift near the peak.

A new distribution was therefore proposed for case II

Date 25 February 1965

Location Sarnia, Ontario

ROOF AND BUILDING DESCRIPTION

Type of structure and use: Large arched roofs. Warehouse.

Building heated: No Roof insulated: No

Shelter conditions: Building exposed.

ROOF SNOW LOADS (psf)

NBC (1960): 24

Av Obsd: --

Max Obsd: 40 to 90

(estimated)

GROUND SNOW LOADS (psf)

NBC (1960): 30

Av Obsd: 13

SNOW LOAD DISTRIBUTION

Snow accumulation on leeward side of building No. 5 caused collapse.

Collapse occurred after about 10 in. of snow had fallen in about 12 h.

Date 25 February 1965

Location Sarnia, Ontario

ROOF AND BUILDING DESCRIPTION

Type of structure and use: Large arched roof. Warehouse.

Building heated: No Roof insulated: No

Shelter conditions: No

ROOF SNOW LOADS (psf)

NBC (1960): 24

Av Obsd: 12

Max Obsd: 50

(estimated)

GROUND SNOW LOADS (psf)

NBC (1960): 30

Av Obsd: 13

SNOW LOAD DISTRIBUTION

Drift up to 4 or 5 ft on leeward side of the warehouse. No failure occurred in this warehouse, but some bowing of the truss members was observed.

FIG. 4. Case histories of snow on two large-diameter arches in Sarnia, Ont. (Schriever *et al.* 1967).

loading as follows.

- (1) The windward side of the roof was assumed to be clear of snow;
- (2) the upper surface of the snow drift on the leeward

side was assumed to be parallel to the ground and no higher than the crown of the arch;

- (3) the maximum load at any point on the surface was assumed to be not greater than twice the 1-in-30

FIG. 5. Uniformly distributed load, case I.

FIG. 6. NBC triangular loading on semi-circular arch with radius 4.5 m (≈ 15 ft) and $g = 2.9$ kPa (≈ 60 psf).

year ground snow load (there is no data on drifts with maximum $C_s > 2.0$ occurring between crown and edge (Taylor 1979)); and

(4) the slope-reduction factor, β , as previously described, was applied at all points on the surface after the application of (3) above.

In algebraic terms (referring to Fig. 13), on the windward side $C_s = 0$, and on the leeward side $C = \gamma h_x / g$. When $\gamma h_x / g > 2.0$ and $C = 2.0$, then $C_s = C\beta$, where C_s is the snow load coefficient, i.e., the load is $C_s g$, γ is the snow density of 15 lb/ft^3 (or $240 \text{ kg/m}^3 \times 9.81 \approx 2.35 \text{ kN/m}^3$) unless better, local data are available, and h_x is the distance measured vertically down from a horizontal through the crown to any point on the surface.

Using this new distribution it was evident that on circular arches the maximum of $2g$ would not be reached if the radius was less than $15g/\gamma$ because the reduction due to slope was greater than the effect of increasing h_x (Fig. 13). On the other hand, on some larger radius arches in moderate-to-low snow load regions it was apparent that the total load obtained using this distribution might have been higher than that recommended in the 1975 Commentary. This happened because the peak load, $2g$, started closer to the crown (Fig. 13) and was sustained over a significant portion of the arch, especially when the slope at the eaves was less than 30° .

This "extra" conservatism was not supported by available data. Although Taylor (1979) found in a survey of snow on arena-type buildings that 2 out of 14 arches carrying unbalanced loads had maximum

FIG. 7. Unbalanced loading on circular arch of span 11.7 m (38.5 ft) and height 5 m ($16\frac{1}{2}$ ft), Ottawa, 20 Dec. 1977. Maximum depth ≈ 70 cm ($2\frac{1}{4}$ ft). Ground snow load on 22 Dec. was 1.1 kPa (22 psf).

FIG. 8. Unbalanced loading on circular arch of span 12.8 m (42 ft) and height 6.4 m (21 ft). Ottawa, 20 Dec. 1977. Maximum depth ≈ 90 cm (3 ft), building unheated. Ground snow load on 22 Dec. was 1.1 kPa (22 psf).

FIG. 9. Unbalanced loading on circular arch of span 11.6 m (38 ft) and height 4.3 m (14 ft), 25 Jan. 1978. Maximum depth ≈ 30 cm (2 ft), building unheated. Ground snow load on 24 Jan. was ≈ 1.6 kPa (33 psf).

FIG. 10. Unheated arch warehouse 30 km south of Ottawa, 13 Jan. 1977, span 23 m (75 ft), height 9 m (30 ft), length 92 m (300 ft). Ground drift up to 300 cm (10 ft) deep and roof drift 45 cm ($1\frac{1}{2}$ ft) deep.

FIG. 11. "Cathedral Arch," Lethbridge, Alberta, 1960. Maximum snow depth approximately 120 cm (4 ft) ($g = 1.5$ kPa or 31 psf). Ground snow load unknown. The photograph has been touched up to highlight the drift outlines.

FIG. 12. Snow drift on cathedral arch 20–30 km south of Ottawa, 13 Jan. 1977. Depth of drift only about 13 cm (6 in.). Ground snow load in Ottawa was ≈ 1.0 kPa (20.5 psf).

FIG. 13. Unbalanced loading in 1977 Commentary, case II.

C_s coefficients of over 2.0 (i.e., 2.90 and 2.61, both at the eaves) and 4 greater than 1.8, the data were still not plentiful enough to justify more conservatism. The case II loading, as outlined and adopted for the 1977 Commentary, has therefore been reconsidered by the NBC committees since that time.

There seems little reason to abandon the use of $2g$ as the maximum load until more data supporting a change is collected. Further, the triangular load distribution in case II of the 1975 Commentary (Fig. 2) seems appropriate for larger arches in moderate to low ground load areas (Taylor 1979). There is evidence from the field, however (Figs. 3 and 4), and from Isyumov's water flume experiments that the peak load on these larger structures may also occur between the crown and edge of the roof (Isyumov 1971; Taylor 1979). In order to accommodate the two distributions the empirical hybrid

TABLE 1. Approximate volume of snow as a percentage of the product gL . The Roman numerals II, III, and IV designate the recommended design distributions in Fig. 14

	Volume (%)			
	$g = 20 \text{ psf}$ (0.96 kPa)	$g = 40 \text{ psf}$ (1.92 kPa)	$g = 60 \text{ psf}$ (2.87 kPa)	$g = 80 \text{ psf}$ (3.83 kPa)
1975 case II	50	50	50	50
1977 case II				
$h/L = 20/40 = 0.50$	41; II	29; II	21; II	16; II
$h/L = 35/70 = 0.50$	49; II	39; II	32; II	26; II
$h/L = 20/80 = 0.25$	63; III, IV	52; III, IV	44; II	37; II
$h/L = 13/124 = 0.105$	69; III, IV	57; III, IV	47; II	39; II
$h/L = 17/160 = 0.106$	73; III, IV	62; III, IV	54; III, IV	46; II

loading shown in Fig. 14, cases II, III, and IV, was devised. The volumes of snow in cases III and IV are the same when the edge slope is $\leq 30^\circ$. At present, the two new distributions are proposals only and are under consideration for publication in the 1980 Commentary.

Table 1 shows volume of snow as a percentage of volume of full ground load, g , over the whole span, L (Taylor and Schriever 1981). The Roman numerals beside each number, corresponding to those in Fig. 14, illustrate that the reduction in total load will apply to the larger arches in lower snow load areas.

Discussion of limiting h/L ratios

What rise-to-span ratio is sufficient to cause unbalanced loads? When the recommendations for snow loading on arches were first published (Schriever and Peter 1965), the limiting h/L or rise-to-span ratio, differentiating between recommended design distributions of uniformly distributed loading and uniformly distributed loading plus unbalanced loading, was $\frac{1}{5}$. In the revision that appeared shortly afterwards, however, it was changed to $\frac{1}{10}$. Listowel Arena had an h/L of $\frac{1}{8.5}$, the three arches at Sarnia $\frac{1}{7.4}$ each (Fig. 4), and a bowling alley in St. Catharines $\frac{1}{8}$ (Fig. 3) (Taylor 1979). The change seemed justified, and the $\frac{1}{10}$ limit is still recommended.

The central problem of an abrupt change at any such fixed limit remains, however. The transition between design loads at the limit should be more gradual, but field data are not plentiful and the relation will probably not be defined until the problem is studied in a wind tunnel or water flume.

The effect of the abrupt change at $h/L = \frac{1}{10}$ is mitigated somewhat by the requirements for "full and partial loading," as stated in Part 4 of the NBC (1977a, p. 160) and Commentary (NBC 1977c), i.e., that all roof areas ... must be designed for full load (the specified design load) over the entire area or full load distributed on any one portion of the area and half load on the remainder of the area, whichever produces the greatest

effect on the members concerned. These requirements³ ensure that roofs of $h/L \leq \frac{1}{10}$ are still designed for some degree of unbalanced loading.

Drifts at the base of arch-shaped buildings, case V

When an arch-shaped surface intersects the ground or another roof it appears to act more as an obstruction than as a multi-level roof. Snow characteristically accumulates at the intersection, exerting a vertical gravity load on both the curved surface and the ground or supporting structure. Approximately triangular drifts on each side result from snow sliding off the arch and from snow drifting against or over the structure. It is difficult to establish the maximum heights of these triangular drifts, but heights measured on the windward side in the Ottawa area have been as high as $1\frac{1}{2}$ –2 m ($C_s = 1\frac{1}{2}$ – $1\frac{1}{2}$) and on the lee side up to 2 or $2\frac{1}{2}$ m ($C_s = 1\frac{3}{4}$ –2). The horizontal leg of the "triangle" will probably be between 1 and 2 times its height (Fig. 15). For discussion purposes in this paper the distributions shown in Fig. 15 will be called case V, although in the 1977 Commentary the responsible committee decided to deal with this type of load in the text, where the designer was cautioned to consider accumulations at the base of the arch resulting from drifting and sliding.

Severe unbalanced loading can occur as a consequence of these side drifts. One example is that of a long-span Quonset-type arena in Eastern Canada that collapsed when one of two large symmetrical side drifts which was obstructing a road was cleared away during snow removal. A further example is that of snow distributed in a case II – case V combination on a curved shed at a farm some 35 km south of Ottawa. The Quonset-type structure was exposed to west winds

³They do not imply checkerboard loading, however. The probability that the natural variation of the snow cover on roofs will occur in a checkerboard pattern, so as to give the worst stresses in the underlying supporting structure, is too remote to be considered in every design.

FOR $\frac{h}{L} \leq \frac{1}{10}$ USE CASE I ONLY

FOR $\frac{h}{L} > \frac{1}{10}$ USE CASES I AND II

WINDWARD SIDE $C_s = 0$

LEEWARD SIDE $C = \frac{\gamma h_x}{g}$, $\gamma = 15 \text{ pcf}$
(2.35 kN/m^3)

WHEN $\frac{\gamma h_x}{g} > 2.0$ USE $C = 2.0$

THEN $C_s = C \cdot \beta$

IF THE TOTAL SNOW LOAD PER UNIT LENGTH OF BUILDING (PERPENDICULAR TO THE SPAN) IN CASE II EXCEEDS $g \cdot L/2$, CASES III AND IV MAY BE USED INSTEAD OF CASE II

$$C_s = \frac{4x}{L} \cdot \beta$$

FIG. 14. Proposed snow loading on arches, 1980. For roofs exposed to the wind on all sides, all values of C_s marked with an asterisk may be reduced by 25%.

FIG. 15. Drifts at the base of an arch, case V.

blowing almost at right angles to the ridge. The distribution shown in Figs. 16–18 was later recorded by the author on 23 February 1977.

The structure was 24.7 m (81 ft) long, 11.9 m (39 ft) wide at the base, and 4.3 m (14.2 ft) high; drifts were about 2.1 m (7 ft) deep where the arch intersected the ground on the east or lee side and 1.8 m (6 ft) on the windward side (Fig. 19). The 1-in-30 ground load was about 2.8 kPa (58 psf). The ground load measured in Ottawa on the same day was 1.6 kPa (33 psf) and is an upper bound, given because it was not possible to get a good estimate of the ground load at or near the site owing to lack of shelter.

It is useful to compare the measured distributions on this arch, as shown in Fig. 19, with those in Fig. 20, obtained by calculation using cases I, II, V, and VI. Case VI was obtained by adding II and V together. The comparison is favourable and indicates that for this structure the actual snow distributions would have been adequately covered by design cases I to VI (cases I and II as in Figs. 5 and 13, respectively, or indeed Fig. 14, since the calculated volume of snow in the unbalanced case would have been less than $\frac{1}{2}gL$). The requirement in the Code for consideration of “full and partial” loading would also have to be applied.

A further example of a combined roof and ground drift on a Quonset arch hangar at Halifax Airport when it failed in December 1970 is shown in Fig. 21.

Overall stability

In the design of arches analytical and design techniques that provide a check on the overall stability of the structures under unbalanced snow loads should be used. Article 4.1.1.7 of the NBC (1977) is quite explicit: “Provision shall be made to ensure adequate stability of a structure as a whole, and adequate lateral, torsional and local stability of all structural parts.”

Analytical techniques employed to examine the stability of rather flexible structures (arches) will probably need to have the equilibrium equations based on the deformed structure at each load stage and may have to accommodate nonlinear stress-strain behaviour where stresses are high.

FIG. 16. Combined case II — case V loading on lee side of Quonset building 35 km south of Ottawa, Feb. 1977. Ground snow load in Ottawa was 1.6 kPa (33 psf).

FIG. 17. Snow drift on east (lee) side of arch (Fig. 16) looking north. Maximum depth at edge of structure about 2.1 m (7 ft).

Conclusions

(1) Much more data are required for a better description of the magnitude and distribution of unbalanced snow loads on a wide selection of arch sizes and for the full range of climate throughout Canada.

(2) The rise-to-span ratio used as the demarcation

between curved roofs, flat enough to be designed as flat roofs, and roofs on which unbalanced loads are significant, needs more study. The change between the two types of required design conditions should probably be more gradual.

(3) Recommended snow loading on arches in codes

FIG. 18. View of west (windward) side of same arch (Fig. 16) looking north. Maximum snow depth at the edge of the arch was about 1.8 m (6 ft).

FIG. 19. Quonset-type farm building 35 km south of Ottawa. Measured snow distributions.

FIG. 20. Design snow loads on arch of radius = 6.1 m (20 ft), height = 4.3 m (14.2 ft), $g = 2.9$ kPa (60 psf). For each case loads are plotted at the left and the corresponding depths to scale on the structures at the right.

FIG. 21. Collapse of 36.5 m (120 ft) span Quonset arch under case II – case III loading, Halifax Airport, 28 Dec. 1970. Photo taken 6 Jan. 1971. Ground load in Halifax on 27 Dec. was 1.1 kPa (23 psf). (Courtesy D. C. Tibbetts, DBR–NRC, Halifax.)

and commentaries will probably change as more data become available and as the influence of geometric and climatological variables is better understood.

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