The facts and the lessons of Listowel: a study of the factors relating to the collapse of Listowel arena and the lessons to be drawn from the tragedy in the interests of public safety

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The Collapse of the Listowel Arena

by

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CANADIAN CONSULTING ENGINEER

The facts and the lessons of Listowel

The facts and the lessons

'Municipal governments have the authority to enact building by-laws, and are responsible for the enactment of such regulations . . . But, because of the many cases where such regulations have not been enacted, of where they are inadequate or improperly enforced, should there not be a responsibility at a higher level of government to ensure the structural adequacy of buildings to be used as places of public assembly . . .'

Fig. 1. An aerial picture of Listowel arena shortly after the roof collapse

# of Listowel

A study of the factors relating to the collapse of Listowel arena and the lessons to be drawn from the tragedy in the interests of public safety

by

W. R. Schriever
D. E. Kennedy and
C. F. Morrison

On Saturday, February 28, 1959 about 9 a.m., a peewee hockey game got under way in the Listowel, Ont. Arena. Suddenly, at 9.27 a.m., without any immediate warning, the roof and the walls of the rink section of the arena collapsed with thundering noise. Seven boys and one adult were killed and thirteen boys were injured. Of those who were on the ice and in the players' box, only two climbed out of the debris unharmed. Only a small part of the arena remained standing, namely the entrance section at the western end which had interior partitions.

How could such a tragic failure happen? What factors led to a condition which could allow such a sudden and near-complete collapse of the roof and the walls of a building which was designed for large public gatherings?

It is the purpose of this report to discuss the facts which led to this tragedy so that the public in general and, more particularly, engineers, architects, public officials and any citizens who might be concerned with community halls and similar structures, might learn from these facts. The authors hope that in this way the lessons which can be learnt from this tragic failure will help to avoid its recurrence.

The building, which was 240′ long and 110′ wide, contained a regulation size hockey rink with seats for slightly more than 1,000 spectators and an "auditorium" section with a hall, a snack room, dressing rooms, etc. at one end. The arena was oriented in an east-west direction, with the main entrance at the west end which was occupied by the auditorium section. The walls were made of concrete blocks, 8″ thick, and approximately 20′ high, with the wall thickness doubled at all truss supports to form pilasters, 16″ by 40″ in plan. The curved roof was supported by bowstring glue laminated timber trusses spaced 20′ apart and spanning 110′. Each truss consisted of a straight lower chord and a curved upper chord of glue laminated timber, connected by diagonal and vertical web members. Between the trusses, the roof was carried by wooden joists measuring 1⅝″ x 13″ in cross section and 20′ long. The roof deck consisted of wooden boards of nominal 1″ thickness covered with rolled roofing. At the top of the roof were a number of sheet metal ventilators which, however, had only been installed some time after the construction of the arena, when it was discovered that the humidity inside the arena sometimes became too high.

After the collapse, which was almost a complete one, leaving only the auditorium section standing, rescue operations were started immediately in an effort to free the injured buried by the debris. Pieces of the wreckage were removed, first by hand and then by cranes and other equipment rushed to the scene of the accident. This naturally caused considerable further damage to the broken parts of the roof, and made it nearly impossible in the later investigation to differentiate between the parts of the roof damaged during the collapse and those damaged subsequently during the rescue operations.

History of the inquest, the collapse, the planning and construction of Listowel arena No consulting engineer or architect was engaged

Soon after the collapse, a number of authorities were charged with the investigation of the causes of the failure, and an inquest into the reasons for the collapse was ordered by the Attorney-General of Ontario. Some of those investigating various aspects of the failure were: the Ontario Provincial Police, the Fire Marshall of Ontario, the Department of Labour of Ontario (Factory Inspection Branch), the University of Toronto; assistance was also requested from the National Research Council of Canada and the Forest Products Laboratories of Canada. In addition, a number of other organizations and individuals concerned visited the site of the collapse and investigated various technical aspects of the failure.

The inquest was held on March 17 to 20 in Listowel and over 100 witnesses were called to testify on various aspects of the collapse and the history of the arena building. The inquest was conducted by Mr. Eric Silk, Q.C., Assistant Deputy Attorney General of Ontario acting for the Crown on behalf of the coroner. As would be expected, the inquest received a great deal of attention by the Press, both locally and nationally. At the outset of the inquest, the Crown Attorney emphasized that it was the intention to put all possible evidence before the jury so that the cause of the collapse could be established and also so that a recurrence of this tragedy could be avoided.

The witnesses were divided into a number of groups comprising mainly witnesses identifying official exhibits such as photographs, eye witnesses, rescue workers, local residents familiar with the condition of the arena prior to the collapse, people concerned with the planning and the construction of the arena and, finally the technical witnesses. In the following section of this report, information which was presented by various witnesses at the inquest is summarized in what is hoped to be a logical sequence.

The Collapse

Since much of the evidence which could have been provided by the wreckage on the mode of failure was destroyed by the manhandling of the debris during the rescue operations, the accounts of the eye-witnesses on the collapse were very important. Although some of the eye-witness accounts, particularly those of the boys who had lived through the accident, were somewhat confusing, it appeared that the collapse started with a loud crack approximately over the centre of the rink area, perhaps a little more towards the north side. Some of the boys recalled seeing the bottom member of one of the centre trusses falling towards the ice first, and then the opening of a hole in the roof through which light was visible for a fraction of a second before the entire roof came down. One of the adult witnesses estimated that the entire collapse took about three or four seconds. It was generally agreed that the failure started in the roof and not in the walls, and that the collapse of the wall therefore was, a result of the roof failure.

Some eye-witnesses outside the arena reported that they heard a loud rumble and then saw the walls tumbling outward. The falling roof seemed to have produced considerable air pressure in the building, causing a window-frame from the east wall of the arena to land on the front lawn of a house on the opposite side of the street some 70′ away.

A number of persons living in the neighbourhood testified that they had noticed a great deal of snow on the roof of the arena, particularly along the north side, for some weeks prior to the collapse. No observations on the snow on the south side of the roof were reported by the witnesses, and it must therefore be concluded that there was very little snow on the south side. On the north side, a large overhang of snow, projecting beyond the edge of the roof, had developed early in the snow season. This snow condition was discussed by some of the arena officials and also by local residents on several occasions. Early in January, after an inspection by some members of the arena committee, it was decided to have some snow removed from the roof. This was done on January 7 with a piece of barbed wire used as a "saw". Another effort to remove the snow overhang had also been made two days earlier by directing the stream from a fire hose at the snow, but without much success. It appears, however, that in both cases the roof was cleared mainly of the overhang and only to a minor extent of the main mass of the snow on the roof.

A number of witnesses were also called in connection with the condition of the building prior to the collapse, since some persons had expressed some opinions on this. Some people, including the arena manager for instance, had heard "creaks and groans" coming from the roof; others had also noticed some sagging of joists between the trusses along the north wall. All this was reported to the arena manager or the arena commission but, after an inspection of the roof from the ice, no action was taken apart from the snow removal mentioned earlier. Some residents, however, were concerned enough over the safety of the structure to refuse to enter the arena.

Planning and Construction of the Arena

It was in 1953 that the town of Listowel decided to abandon the old natural ice rink and construct a new arena to be known as the Listowel Memorial Arena with artificial ice. After numerous meetings of interested citizens during the year, the ratepayers finally, on September 14, 1953, voted overwhelmingly in favour of borrowing, on debenture, $50,000 for the new arena. The following day, the town council appointed an arena building committee and an arena finance committee, consisting of councillors and interested citizens. Since the people of Listowel were anxious to have the arena for the coming winter, the building committee proceeded as rapidly as possible with the arrangements for the construction of the arena. Members of the building committee visited a number of other arenas in the area and, on their return, decided upon a tentative design which they believed incorporated the best, or the most economical, features of the other arenas. No consulting engineer or architect was engaged, but a retired local engineer offered his services free of charge for some of the design work or for alteration of plans of other arenas in accordance with the wishes of the committee. A local contractor was hired as a "supervisor" for the construction at a fixed fee.

The building committee decided that the arena should have concrete block walls with a bowstring-truss supported roof. It was also agreed to buy the trusses from a timber fabricating firm with which the supervisor mentioned above had had some contacts. A professional engineer, working for this firm, designed and drew the plans for the trusses, the roof, the walls and the footings of the buildings, whereas the plans for the lay-out of the building in general and the auditorium section of the arena in particular were drawn by the retired local engineer mentioned earlier.

There was some discussion in the building committee at the time on whether the walls should be made of 8" blocks or 10" blocks but, after a visit of the representative of the timber fabricating firm, it was decided that 8" blocks would be adequate. Another recommendation of the representative of this firm to space the trusses 20' apart, rather than 16' as originally envisaged, was also accepted. The supervisor was authorized to order eleven trusses at $14,600 instead of fourteen trusses at $15,400 quoted in an earlier letter of the firm. The reasons for these decisions are not clear and were not recorded. It appears that no minutes were kept of any of the committee meetings, although some notes on motions passed during some of the early meetings of the building committee were kept and typed by one member acting on his own.

Construction of the arena began about two weeks after the debenture vote and, in a little more than three months, the arena was almost completed. Most of the labour was volunteered by citizens who donated their time as a contribution towards their arena, and the work was directed by the supervisor mentioned.

During construction, the work was not inspected by the building inspector of Listowel. In fact, the plans of the arena were never submitted to the building inspector for examination and no building permit was issued. Nobody remembered any discussion about a permit. This was confirmed by the town clerk, that there was no record of minutes that any inspection of the arena either during construction or since was made. Members of the building committee, however, were frequently present during the construction of the arena.

Since the town of Listowel had a building by-law (passed in 1943), a permit should have been issued, but the building inspector stated that there was nothing in this by-law that controlled the erection of an arena, and he thought that members of the building committee and others concerned were much better qualified to inspect the job than he was. It should further be taken into account that the building inspector was also the assessor for the town and had no experience in building construction.

It was stated during the inquest that some of the details had to be decided by the supervisor during the construction, such as the thickness of the footings which apparently were not shown on the drawings. He decided that the footings should be 12" thick under the walls and 18" thick under the pilasters. He also said that he thought that he had put some reinforcing into each of the pilaster bases.

Construction proceeded rapidly and the trusses, the design and construction of which will be discussed later, were erected probably during late October or early November by the timber fabricating firm. It apparently remained unnoticed that the trusses were not as high at the centre as was shown in the drawings and that the top chord was one board lighter than called for. The trusses proved to be very "shaky" during the erection, and some workmen of the timber fabricating firm refused to climb on the trusses again after having been on them once.

The arena was substantially completed early in January and was opened for skating at that time. Final completion of the building was achieved in March.

The total cost of the arena amounted to some $140,000, of which $70,000 had been raised by debenture. Additional funds were raised through a canvass of citizens and companies, and finally a grant of $10,000 was made in 1955 by the Department of Agriculture of Ontario under the Community Centres Act. Although plans of the arena were submitted to this Department in order to obtain the grant, it was stated by its representative that the Department does not check designs for structural adequacy.

The majority of people in the town were proud of their arena and thought that it was a fine building. Some persons were doubtful about the strength of the building, especially with regard to the concrete block walls which showed some cracks after completion of the building. These cracks were noticed and discussed a number of times, particularly since they led to rain leakage through the walls. The cracks were pointed or patched in 1954 and 1955 and the walls were waterproofed with paint. Some further opening of the cracks was noticed after that but most people did not think that the cracks were serious or unusual. Some blocks were pushed out near ground level by wooden stringers supporting seats, probably because of the pressure of the ice inside the arena. Two blocks in the south wall were therefore replaced and all stringers cut off a few inches from the wall.

The design of this arena probably anticipated a snow load much less than that now recommended by the National Building Code

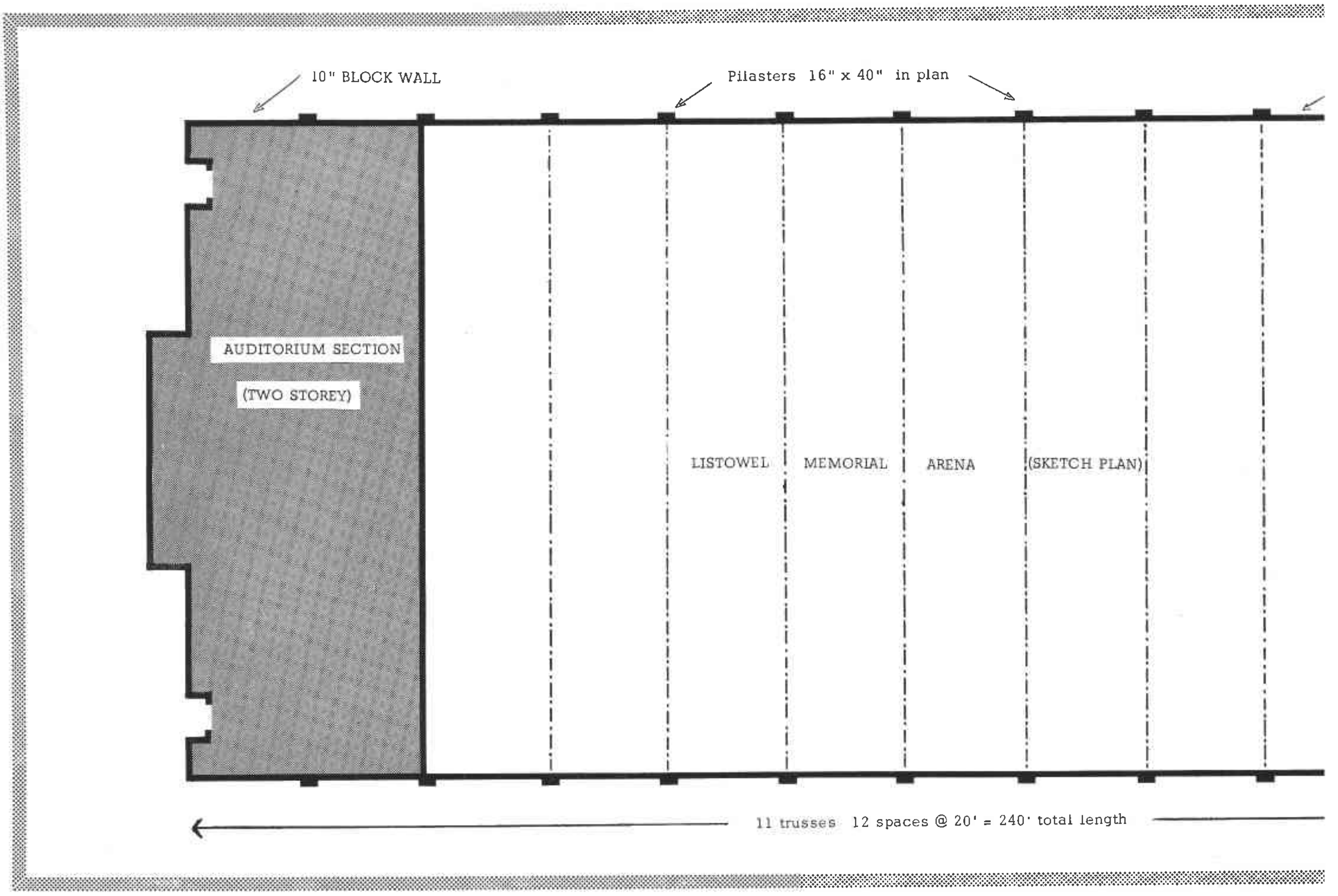
Plans and specifications for the Listowel Memorial Arena which were presented at the inquest were incomplete, and it is necessary to speculate regarding several features of the design and regarding the design criteria used. Presumably the building was constructed using these plans and, if so, it would have been necessary for the construction forces to "ad lib" to a considerable extent.

Figure 2 shows a plan view of the building and an elevation of one of the roof trusses. The building was 110 ft. wide and 240 ft. long. The portion of the building over the ice was 200 ft. long and 20 ft. high at the eaves. The remainder of the building, 40′ x 110′, was two storeys high and housed dressing-rooms, entrance lobbies, services and an assembly hall. The total height of the two storeys was 20 ft. so that the roof trusses were at the same elevation for the full 240 ft. length of the building. Because of the extra strength provided by the partitions and other supplementary framing, the roof trusses over the two-storey portion of the building did not collapse even though they were damaged to some extent by the collapse of the other portion of the roof.

The main load-carrying members in this structure were — roof joists, trusses, walls, pilasters, foundation walls and footings.

Much publicity was given to the fact that, at one stage in the planning, 14 trusses were to have

Fig. 2. Plan view of the building and an elevation of one of the roof trusses

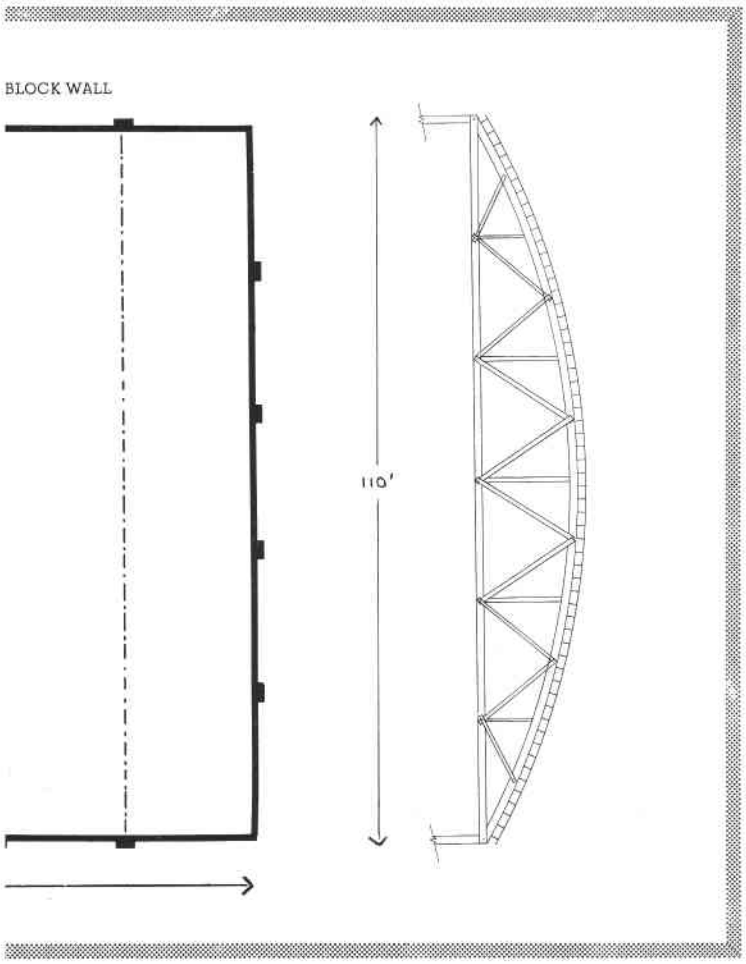


been supplied but that only 11 were installed. Originally it was planned that the trusses were to be spaced 16 ft. apart thus having 15 spaces in the 240 ft. length of the building. This layout would have required 14 trusses. Later it was decided to use twelve 20 ft. bays for the 240 ft. length. This arrangement required only 11 trusses. This is a perfectly reasonable revision in layout and there is good reason to believe that the trusses were designed for the wider (20 ft.) spacing.

When assessing any structural member it is necessary to make two basic decisions in addition to the required calculations. One is the decision as to the magnitude of the load to be anticipated and the other is the decision regarding the load which the member can support safely. This latter decision is usually expressed in terms of specified permissible stress.

Most specifications stipulate the magnitude of loads to be anticipated in design and the value of permissible stresses in structural materials. Accordingly, these two decisions are usually made by selecting the specifications which are to govern the design.

There is no evidence to show what specifications were used in the design of this structure. For the purpose of evaluating the elements used, calculations have been made by the authors assuming that the snow loading anticipated in design was 40 psf. of roof surface, and that the permissible stresses used in the design of the roof joists and the trusses were those stipulated in C.S.A. Specifications 043-1953 "Specification for Structural Timber" and in C.S.A. Specification 0122-1953 "Specification for Glued-Laminated Timber Construction."



It should be noted that there have been significant revisions to these specifications but that it would have been considered "sound practice" to use them at the time when this building was designed.

The calculations show that:

(1) The roof joists would not have been overstressed using the above mentioned values for loading and permissible stress.

(2) The top and bottom chords of the truss would not have been overstressed using the above-mentioned values for loading and permissible stress provided that: (a) the snow loading was assumed to be uniform over the whole area of the roof; (b) the sections used for the chords and the dimensions of the truss were those shown on the designer's drawing, a print of which was presented at the inquest.

(3) Because the top chord section as built consisted of 10 laminations, not 11 as shown on the drawing, and because the truss depth centre to centre of chords was 12'-9¾" instead of the 14'-0¾" shown on the drawing, the top chord would have been overstressed by 6 percent and the bottom chord would have been overstressed by 4 percent using the above-mentioned values for loading and permissible stress.

(4) Information on snow loads which is now available *(Chart 8, Part 2, Climate, National Building Code of Canada (1953) and Building Note # 34, Division of Building Research, N.R.C.)* but which was not generally available in 1953 suggests that 60 psf. would be a more realistic estimate of the snow loading which should be anticipated in the design of structures in Listowel. Using this loading on the truss as shown on the drawing, the top chord would be overstressed by 30 percent and the bottom chord would be overstressed by 18 percent. For the truss as built, the overstress would be 45 percent in the top chord and 24 percent on the bottom chord. A snow load of 60 psf. would overstress the roof joists by 32 percent.

(5) An unsymmetrical distribution of snow load may occur and this condition must also be considered in the design of the truss members. Each diagonal web-member shown on the design drawing and used in these trusses was connected to one face only of each chord. This arrangement is undesirable because the effect of forces in the web-members connected eccentrically is to apply a torque to the chord and, as a consequence, to cause serious second-

ary stresses. This eccentric joint arrangement is shown in Sect. A-A of Fig. 3.

(6) The National Building Code of Canada (1953) stipulates requirements for wall thicknesses and pilaster dimensions for the design of building of this type. According to these requirements, the 8" block wall was not thick enough or the dimensions of the pilaster were not large enough. For a wall 20 ft. high, according to the National Building Code, 1953, using a 16" x 40" pilaster, a wall thickness of 12" is required. Using a 24" x 40" pilaster, a wall thickness of 8" is required. For the dimensions which were used — wall thickness of 8" and pilaster dimensions of 16" x 40" — the maximum permissible height of wall is 16 ft., not the 20 ft. which was used.

It is evident that the wall thickness and pilaster dimensions used were much below the acceptable minimum values which are specified in the National Building Code of Canada (1953).

(7) It is not possible to establish the size of the foundation walls and the footings and the reinforcement in them from the design data shown on the drawing presented.

There was no evidence of serious settlement of the foundations and, accordingly, it might be assumed that there was no major deficiency in this feature of the building as constructed.

In summary, the investigation of the structural features of this building shows that:—

(1) The snow load anticipated in design was probably 40 psf. which is much less than the 60 psf. which is now recommended for the Listowel area by the National Building Code of Canada (1953).

(2) The roof joists had adequate strength for a snow load of 40 psf. but would have been overstressed by a snow load of 60 psf.

(3) The truss chords as shown on the designer's drawing would not have been overstressed by a symmetrical snow load of 40 psf. but would have been seriously overstressed by a snow load of 60 psf. The web-members, particularly because of the eccentricity of the end connections, were undesirable for the pronounced unsymmetrical loading which should have been anticipated and which did occur.

(4) According to the requirements of the National Building Code of Canada, 1953, the thickness of the wall and the dimensions of the pilasters were inadequate.

(5) Design data for the foundation walls and footings were not complete on the drawings which were presented at the inquest.

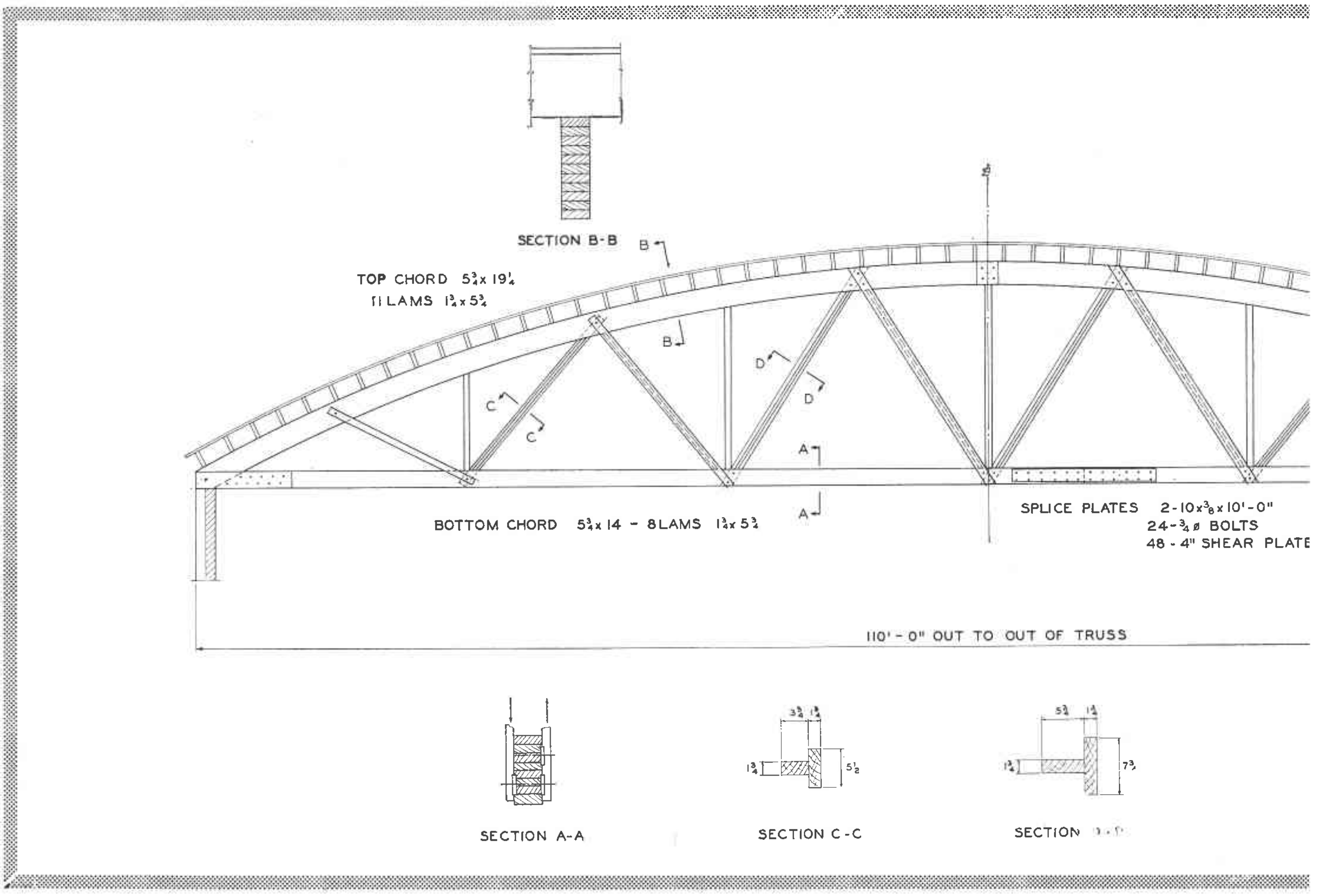
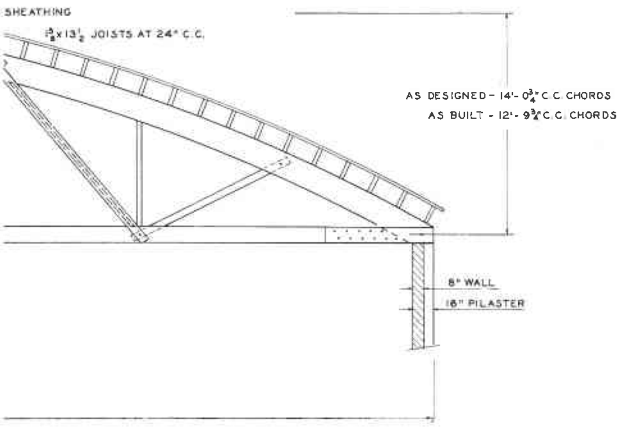


Fig. 3. A drawing of one of the trusses as originally designed

Snow lay more than four feet deep along the north edge of the roof . . . some two feet was removed about seven weeks prior to the collapse

Several witnesses testified on the question of snow loads, both from a general point of view of the severity of the winter 1958/59, and, more specifically, on the amount of snow on the roof at the time of the collapse. Available climatic records showed that the winter of 1958/59 resulted in a total snow fall which was approximately 30% above normal in the Listowel area at the end of February. This was no record snow fall for Listowel. The winter, however, had been unusually cold as illustrated by the fact that, in the period November 26 to February 28, the temperature had risen above freezing point only on three days whereas temperatures of below 0° Fahrenheit had been recorded on 28 days.



Climatic records also indicated that frequent strong winds were experienced during that winter. Although no wind records are kept in Listowel, at London, Ont., the average wind speed for January and February exceeded the normal values (approx. 12 m.p.h.) by about 2 m.p.h. In Listowel the observer recorded blustery winds or blowing and drifting snow on a number of days in January and February.

On February 25, the total fresh snow fall had reached close to 70" and the depth of snow on the ground was 24". Areas to the south and north, and particularly to the west, of Listowel had recorded even greater total snow falls. Stratford to the south 90", Walkerton to the north 112" and Clinton to the west 140". February 28 was a relatively mild day in Listowel and the temperature rose to the freezing point. Some rain and freezing rain had fallen during the previous night and during the morning of February 28.

As was already mentioned, the snow had accumulated to considerable depth along the north edge of the roof early in the winter, and it was along this edge that some snow was removed on January 7 by a group of four people who used a length of barbed wire to saw off the overhang of snow. From the statements of these people, it appeared that the snow was approximately 4½' deep at the eave, tapering to practically zero at the peak, and also that the depth of snow was approximately the same along the entire length of the roof. One of the witnesses stated that the snow was removed from the eave to a point approximately 20' back where the snow left was about 2½' deep. No snow was removed, however, after January 7.

After the collapse, the importance of measuring the actual weight of snow at the site was fortunately realized by a professor from the University of

Fig. 4. Snow condition on north side of roof over Auditorium section of the arena shortly after the collapse. Photo by courtesy M. Rice, Listowel

Waterloo who heard of the collapse over the radio. Since he thought that even approximate observations taken with improvised equipment as soon after the catastrophe as possible could be more valuable than accurate determinations made some time later, he rushed to the scene at 2.30 p.m. and took two samples of snow from the standing part of the roof just before a crew of men were about to shovel the snow off as a precautionary measure.

A sample taken from a location 30′ from the west wall and 10′ from the north wall on the roof indicated a weight of the snow layer of 55 p.s.f. At this spot, the snow was 2′ deep with 2″ of ice at the bottom. A second sample of snow, which had remained undisturbed during the collapse, was taken from the north-east corner of the building on a piece of fallen roof. This sample indicated a load of 75 p.s.f., the snow being 28″ deep with 2½″ of ice. No information, however, was given on the distribution of the snow cover over the roof. Some indication can be obtained from Fig. 4.

Later, on March 3, a member of the investigating team from the National Research Council also took a sample of snow in the north-east corner indicating a weight of snow cover between 70-80 p.s.f.

A comparison of these actual snow loads with the design snow loads of the National Building Code of Canada (1953) indicates that the observed maximum are somewhat greater than the design load of approximately 60 p.s.f. shown on the National Building Code map (chart 8 in Part 2 "Climate" of the National Building Code 1953) for the Listowel area. The lines in this map indicate the computed maximum snow load on a horizontal surface, and are based on snow depth measurements made on the ground at meteorological stations across the country. The values shown represent the maximum recorded in a ten-year period prior to the publication of the National Building Code in 1953. The loads given include an allowance for the weight of the greatest 24 hour rainfall which might fall into, and be retained by, the snow cover during the late part of the winter.

At the time of the collapse the snow cover on the ground was, as mentioned, approximately 24″, which represents a snow load on the ground of the order of 30-40 p.s.f. Since the load on the roof along the north eave was 55-80 p.s.f., it must be concluded that there was very little snow along the south side and the upper parts of the roof because some of the snow naturally had blown off the roof and, furthermore, because some snow had been removed early in January.

It thus appears that, at the time of the collapse, there was a strip of heavy snow along the north side of the roof which would exert a strong unbalanced force on the bowstring trusses, a force to which this type of truss is particularly susceptible, since it tends to produce high compressive and tensile stresses in the web members and considerable bending in addition to the axial forces in the top and bottom chords.

Although the Listowel Arena was not built under the National Building Code, it is worth noting that the Code makes allowance for the well known fact that, under certain conditions, non-uniformly distributed loads will accumulate on roofs. Thus the National Building Code (1953) stated: "The non-uniformly distributed load shall be based on concentrations which are liable to occur due to the effect of wind and the shape of the structure". A further clarification of this point has since been made in the National Building Code in the form of a revision slip.

Fabrication and workmanship of the trusses
There were indications of probable failure of the glued laminated bowstring trusses

The workmanship and materials which go into glued laminated trusses play as important part in the performance of the trusses as the design itself. In view of this fact, the Division of Building Research of the National Research Council, with the approval of the Attorney-General's Department of the Province of Ontario, requested the Forest Products Laboratories of Canada to assist in the investigation of the collapse of the Listowel Arena. There were indications that the collapse may have been caused by failure of the glued laminated bowstring trusses, and that the gluing of the laminated members may have been faulty. As it appeared that the specialized services of the Forest Products Laboratories of Canada could make a worthwhile contribution to the investigation, two of the specialists at the Ottawa Laboratory were assigned to the task and one appeared at the inquest to give expert testimony.

This phase of the investigation consisted of two parts:

(i) an examination of the debris at the site of the collapse,

(ii) a laboratory examination of several lengths cut from the top and bottom chord members of trusses, and shear block tests to determine the strength of the glue bond at several locations.

The first inspection of the collapsed arena by FPLC personnel was made on March 6, 1959, six days after the collapse had occured. A superficial inspection showed considerable evidence of faulty glue bonds, due either to lack of adhesive or lack of sufficient bonding pressure. It was not possible, under the conditions existing at the time of the inspection, to determine which of these two factors was responsible for the poor bonds. It appeared that the gluing in the curved upper chord members was of poorer quality than in the straight lower chord members. There were many open glue lines and delaminations, especially in the upper members.

In one of the laminated web members which had been completely shattered in the collapse, it was observed that the *bonded* area on one glued surface was about 10 percent of the *glued* area. It appeared that the glue had been spread on one contacting surface only and that there had been little or no transfer of glue to the opposite contacting surface.

A 16 ft. length of upper chord was examined. The two upper laminations on this member were delaminated for its complete length. Although it was not possible because of the presence of spikes to separate these laminations so as to inspect the glue lines fully, there were gaps of approximately ½-inch between these laminations and examination indicated very little evidence of wood failure.

Many other indications of poor glue bonds, too numerous to list here, were observed. It remained for the laboratory examination and tests, however, to point out the probable reasons for the glue bonds being as bad as they were.

Poor glue bonds, when found to exist in glued laminated members in a building several years old, may be attributed to one or more of the following factors:

1. Use of glue of an inferior quality
2. Improper fabricating and gluing procedures
3. Deterioration of the glue bonds subsequent to the fabrication of the laminated members.

By a process of elimination, the laboratory examinations and tests served to eliminate factors 1 and 3 as likely causes of the poor glue bonds which appeared to be so general throughout the glued laminated members. Among the interesting points revealed by the laboratory examination were the following. A large number of spikes had been used during the assembly of the members. The number and length of these spikes were such as to suggest that they were intended to provide the gluing pressure. Indeed, they were so closely spaced together that it was impossible to cut a 2-inch section from the top and bottom chord samples without the saw striking a spike. Former employees of the fabricating firm, however, insisted that the gluing pressure had been provided by clamps. If this was so, the spikes could hardly have facilitated the clamping together of the laminations. Small nails are frequently used by good fabricators to hold the laminations in alignment until the clamps can be applied. This calls for something of the order of one nail approximately every 10 or 12 feet. Former employees of the fabricating firm, testifying at the inquest, had no explanation for the large number of spikes that were used. It was revealed, however, that the firm had experienced trouble, in previous contracts, with glulam members delaminating in their yards before shipment. Presumably no visible delamination took place in the Listowel trusses before erection, and the large number of spikes might conceivably have served merely to postpone, until too late, detection of the lack of glue bond between laminations.

There was evidence that the laminating stock had been dressed for a lengthy period before the laminating operation and had undergone further drying before it was laminated. This resulted in some of the laminations being of non-uniform thickness. If the fabricating firm had been following "recommended good practice", this material would have been re-dressed by passing it through a planer. It is virtually impossible to achieve good glue bonds with laminating stock that does not meet the thickness tolerances, no matter how high the clamping pressure may be.

It was quite apparent that the glue had been spread on only one of any pair of contacting surfaces. This practice, in itself, will not necessarily produce a poor glue bond. If the glue is spread generously enough, if the surfaces are truly planed, if the laminations are clamped together within the specified time and under the specified pressure, a good glue bond should result. However, in cutting specimens out of the sample lengths of top and bottom chord members, it was evident that one or more of these conditions were frequently not met. Occasionally, the glue, which had been spread on one of the two contacting surfaces, had never been transferred to the other surface. In some instances this was caused by the two surfaces never coming together. In other instances, it appeared to be caused by the assembly time being too long, allowing the glue to dry on the one surface before the other surface had been brought into contact. In one individual specimen the glue had been spread on one surface and had partially jelled. When the laminations were brought together, only a small percentage of the glue film was pressed by the other lamination and this small area bore the flattened imprint of the other wood surface. No transfer of glue took place, however, even at the area of contact.

Casein glue had been used in the gluing of the trusses. This was determined by a chemical analysis that was carried out by a chemist employed by the Province of Ontario. No confirming chemical analysis was carried out by the Forest Products Laboratory. It was known that laminating plants in the years 1953-54 generally used either casein or resorcinol adhesives, each of which is easily distinguished from the other by appearance. The FPLC did, however, carry out shear block tests to determine the strength of the glue bond between the various laminations in the samples which were taken from the top and bottom chord members.

A total of 43 standard shear blocks were cut from the sample laminated members. Of the 43 shear blocks that were cut, five fell apart before they could be tested. The glue bond, in other words, was non-existent. The remaining 38 shear blocks were tested to failure and the results were extremely variable. The lowest shear strength obtained by test was 103 pounds per square inch. The highest was 2060 pounds per square inch. The average shear strength was not calculated because this figure would serve only to throw an overly optimistic light on the overall quality of the workmanship. Just as a chain is only as strong as its weakest link, the strength of a glued laminated timber may be only as strong as its weakest glue line. By contrast, recent shear block tests on the product of a modern fabricating plant in which quality control procedures were operating, gave a low value of 1090 p.s.i. and an average of 1630 p.s.i.

The results of the FPLC sear tests indicated that (1) the glue used was capable of developing a very satisfactory strength when the conditions of fabricating happened to be right and, (2) there were few grounds to support the suspicion held by some that glue bonds had been seriously weakened by time and by exposure to conditions of high relative humidity. The average moisture content throughout a cross section of one of the members was 16.5 percent and the moisture content of the outer inch was 17.0 percent, indicating, as would be expected at that season of the year, that the trusses were picking up moisture. If moisture had caused any appreciable deterioration of the glue lines, it would be expected that the outer lines would be the weakest. There was, however, no consistent pattern or trend in the shear tests made across any of the cross sections that would indicate this to be true.

From the evidence given at the inquest by former employees of the fabricating company, it was apparent that the conditions which prevailed in the plant at the time that the trusses were assembled were not of the best. The building was said to have been poorly heated by a single oil stove that was moved from place to place as the need arose. The laminating lumber was stored outside and the glued laminated members were also stored outside shortly after the clamps were removed. The only piece of quality-control apparatus reported by the employees was an electric moisture meter which could be used to check the moisture content of the laminating lumber. Whether or not even this piece of equipment was effectively used was not clearly established.

Modern present-day glulam fabricating plants meeting trade association certification standards are required to have considerable quality-control equipment and personnel. Even at the time the trusses were fabricated, however, procedures existed for the checking of glue bonds to ensure uniform and adequate quality. If a professional engineer or architect had been retained by the Listowel arena commission to supervise the erection of the arena, he could have insisted that several inches be cut from the end of each member for testing to determine the shear strength of the glue bond. The trusses which went into the arena could hardly have passed such tests at the time they were fabricated.

Conclusions of the authors

In the planning for buildings where large numbers of people congregate it is essential that proper precautions be taken to obtain safe structures. There are, of course, many other important considerations but it is of prime importance to safeguard the human lives involved.

As a result of the tragic collapse of the Listowel Memorial Arena and the thorough investigation which was made by several agencies of government and by independent consultants the following points deserve special mention:

Administration

Provision should be made for some authority to approve or disapprove the plans and specifications for any proposed structure in this category. The authority having jurisdiction should have a competent official to perform this function. This official might be a full-time employee or might be engaged on a part-time basis or on a temporary basis for a specific project.

Regulations should require that the plans and specifications be prepared under the direction of a registered professional engineer or architect qualified for this work.

Municipal governments have the authority to enact building by-laws and are responsible for the enforcement of such building regulations. However, because of the many cases where such regulations have not been enacted or where they are inadequate or where they are improperly enforced, should there not be a responsibility at a higher level of government to ensure the structural adequacy of buildings to be used as places of assembly?

Design standards

As the authority having jurisdiction should be responsible for the approval or disapproval of the design, it would be necessary to have a standard by which to judge the design and to make this decision. This standard would be the building by-law of the municipality concerned. Any municipality which had not preparel a building code of its own and which did not wish to prepare one could adopt the National Building Code of Canada as its building by-law. Values for loads to be anticipated in design and permissible design stresses for construction materials are given in this code.

Supervision

A sound design is not enough.

Provision must be made to ensure that the structure is built according to the plans and specifications. Supervision by the person who prepared the design is an eminently satisfactory arrangement. In cases where this is not done, some other properly qualified engineer or architect should be in charge of this very important phase of the project.

One of the most significant functions of the supervisor is to ensure that the material supplied is in accordance with the requirements of the specifications.

The authority having jurisdiction also has a responsibility to require that the work is done as designed and specified, in addition to its responsibility in connection with the adequacy of the design of a structure.

Because of the experience with poorly glued members in the roof trusses of the Listowel Memorial Arena, it is appropriate to make special reference to glued-laminated timber. No contract for the fabrication and supply of glued-laminated timber should be let to an organization which is not properly equipped and staffed to turn out a quality product. A plant which has been certified by a recognized construction association and which has a valid certification of qualification from such an associaton is more likely to turn out a quality product than a plant which is not so certified.

In some instances it is desirable for the supervisor to require a report on the manufacturing plant and its process from an independent testing and inspection agency. This report should include results of tests of specimens from members manufactured for the project. Provisions for such inspection and testing of specimens is made in CSA Specification 0122-1959 "Specification for Glued-Laminated Softwood Structural Timber". This specification will be referred to in the National Building Code of Canada (1960).

Higher authority

Many of the existing structures in Canada were built without proper safeguards as to adequacy of design and quality of material and workmanship. Any building used as a place of assembly should be structurally adequate. If evidence is not available to show that proper precautions were taken in its design and construction, the building should be examined by someone competent to make this evaluation and to report on the structural safety. As in the consideration of designs for new assembly buildings, the municipal governments should be responsible for having this evaluation made. But should there not be in addition a responsibility at a higher level of government to require that it be done?

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