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Davenport, A. G.; Handegord, G. O.

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STUDY OF WIND DAMAGE TO RURAL BUILDINGS IN SOUTHERN SASKATCHEWAN

by

A.G. Davenport and G.O. Handegord

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PREFACE

An important major project of the Building Structures Section of the Division of Building Research is a critical assessment of the loads which have to be sustained by structures, particularly under Canadian conditions. The study of wind loads is a part of this general investigation. Theoretical studies in this field can be usefully supplemented by investigations of damage to structures from wind and this report is a further record of such a field investigation.

The report is a joint effort, the field study in Saskatoon having been made by Mr. Handegord who is the Officer-in-Charge of the Division's Prairie Regional Station in Saskatoon. The analysis of his observations has been made by Mr. A.G. Davenport, a member of the Building Structures Section in Ottawa.

When sufficient information such as that contained in this report has been accumulated, it will serve in bridging the gap between results obtained from experimental research in wind tunnels and their application to full-scale structures. In this way safety in structural design should be assisted.

Ottawa,
June 1958.

Robert F. Legget,
Director.

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For some structures the forces exerted by wind are the largest they are called upon to withstand. To a large degree, therefore, the magnitude and action of these forces determine the economy that can be effected in these structures. The difficulty of estimating the wind forces in design, both with regard to their magnitude and manner of action places them among the principal causes of structural failure.

Wind load design requirements, apart from velocity, are based largely on numerous and extensive experiments on models of elementary building forms in wind tunnels; these experiments re-assert the complexity of the pressure contours even around the most elementary shapes. Pressures seldom approximate uniform distribution over any one surface; pressure and suction at times exist simultaneously on the same surface. The pressures themselves vary with every orientation of the building to the wind and are extremely sensitive to changes in building shape, in shielding and in the slope or roughness of the surrounding terrain.

The translation of these factors into design requirements must involve sweeping simplifications. Distinctly non-uniform loads are assumed to be uniform and orientations of the building for which the wind is not normal to a surface are not considered. Thus, even rudimentary refinements in describing the action of the wind's forces are sacrificed to achieve simplicity and convenience of design and specification.

This is inevitable to some degree. It would be impractical to design buildings according to the precise pressures imposed by the wind under all conditions. The consequences of the discrepancy between loads used in design and those actually imposed by the wind will vary for every structure. If all structures were similar, in most cases it would be possible to define a uniformly distributed load whose effect on a structure would be somewhat similar to that imposed by a complex distribution of wind forces.

Structures, however, are not all similar and the effects of representing an actual non-uniform wind load by a simple uniform load will be different for every variation in design. The degree of continuity present in the members subscribing to a surface of a structure will for example largely determine the degree to which the effects of a non-uniform load are distributed among the members.

There is no direct way to determine by observation those structures for which these simplification processes lead to overdesign and consequent lack of economy. It is possible, however, to indicate in a general way those structures for which the simplified specified wind loads lead to underdesign and lack of safety. This is done by observing the failures caused by wind action which occur from time to time. This is the first argument for the type of investigation undertaken in this report.

The second point in favour of observing failures is that in certain cases where the mode of failure is exceptionally simple, it may be possible to obtain some estimate of the actual forces imposed. These cases will not be common but when they do occur they will help either to support present requirements or to cast doubt upon their validity.

Also, a study of this nature may occasionally indicate the existing discrepancies because, generally, the wind load requirements are based on the results obtained in the uniform steady flow of a wind tunnel, whereas the naturally occurring wind, to which full-scale structures are subjected, is turbulent and gusty and seldom achieves the uniform and steady flow of the wind tunnel.

Description of Storm

On July 17, 1957 a severe squall with winds strong enough to cause grave damage to crops, trees and farm buildings, struck southern Saskatchewan (Fig. 1). The areas afflicted by high winds formed a belt approximately 200 miles long, running east-northeast between Moose Jaw and Yorkton. Heavy hail accompanied the squall (reports from Strasbourg and Shamrock), and although this may have caused the more serious property damage this report will deal only with the damage caused by wind.

Most of the wind damage was to farm buildings; two reasons could account for this. The standards of construction and workmanship are probably lower in these buildings than in a town where building code requirements are enforced. Also, the forces of wind in a town may be modified considerably by the effects of ground friction which will be greater there than in open and exposed farm land.

Although the winds appeared to be unusually high, the nature of the storm - a squall wind accompanying the passage of a cold front - is typical. At the front a sharp wind shift occurred; the strongest winds, approximately west-northwest in direction, were experienced after its passage. Although first-class meteorological stations in the area indicated a wind of no more than 56 mph with gusts, newspaper reports suggested that wind velocities reached 80 mph. Judging from the damage it appears that this figure is correct and that the storm actually by-passed all of the first-class weather stations (Fig. 1).

Analysis of Wind Damage

On July 26 and 27 (ten days after the storm) one of the authors who is stationed at the Prairie Regional Station of the National Research Council (G.O.H.), visited the most severely afflicted area in the vicinity of Bethune and Tuxford. A detailed report containing photographs of the damage to four barns and one resort cottage was prepared and this forms the basis for the following analysis, prepared by the other author (A.G.D.).

MOYSEY BARN

Description of Barn and Failure

This barn illustrated in Figs. 2 to 6, was built about 1920 or earlier and it is interesting to note that it survived a severe windstorm in 1953. The barn was of reasonably good construction having 2- by 6-in. studs at 24 in. o.c. (Fig. 5). Horizontal cross-bracing is evident at the east end in this figure. There was, however, no evidence of similar bracing on the west wall.

The failure is shown diagrammatically in Figs. 7(a) and (b). The wind, as in all cases discussed, was approximately west to west-northwest and the ridge of the roof ran east-west. The failure took place in the following way: (a) The west gable end of the barn blew in leaving a gaping hole in the face of the wind. (b) The west portion of the south roof blew outwards.

Analysis of Failure

Wind tunnel studies on similarly shaped buildings with approximately the same orientation to the wind, indicate that the pressure distributions on the end walls and roof are approximately those shown in Figs. 7(a) and 7(b) (1). Initially, with the west gable end intact the greatest pressure will be found on this wall; the suction on the roof decreases from the west to the east end, where it changes to slight pressure. With most of the openings on the sides or to leeward a slight suction will be induced internally; this will augment the effective pressure on the windward gable end wall and balance some of the suction on the roof. Initially, therefore, by far the greatest forces will be on the west gable end which, in fact, was the first portion to fail (Figs. 3 and 4).

The effect of this iritial failure was to increase the opening to the wind with a consequent change in internal pressure from suction to a large positive pressure. Although the <u>external</u> pressure on the roof should not change substantially, this change in the sense and magnitude of the <u>internal</u> pressure will increase the total forces tending to suck the roof off. As before, the suction acting on the east end of the roof will be less than that on the more windward end. Thus, the next portion to fail probably will be the west part of the roof, and this is exactly what happened, a small portion of the roof to the east being left intact (Fig. 2).

Conclusions

(a) The failure as it took place can be adequately explained by the results of wind tunnel studies and suggests nothing to refute the applicability of these results. Quantitative estimates of the wind's force would be difficult to make.

(b) This failure emphasizes the non-uniformity of the pressure exerted by the wind. The roof might have been able to resist the average pressure had it been uniformly distributed, but with the lack of longitudinal continuity in the roof it was vulnerable where the non-uniform pressure distribution was greatest.

(c) The marginal pressure which produced the failure of the roof is attributable to the large change in the internal pressure regime brought about by the initial collapse of the west wall.

(d) The pressure regime on the side walls is similar to those on the roof suggesting the inadequacy of the horizontal bracing used in the structure (Fig. 5). This bracing does no more than transfer some of the pressure from the west end wall to the side walls (tending to push them outward) when these are already under high suction outward. Had the west end wall not failed these side walls would have been even more vulnerable.

(e) There are indications that better spiking might have increased the rigidity of the west end wall. It is notable that all boards in the wall are intact and all rupture has occurred at the spiking. In addition there is very little horizontal support in the wall.

FELIX SEIFFERT'S BARN

Description of Barn and Failure

This barn (Figs. 8, 9, 10 and 11) was constructed of 2 x 4's on 24-in. centres sheathed on one side with shiplap. The rafters were spiked to the top plate with four toe nails and the top plate was in turn nailed to the bottom plate with one 3-in. spike at approximately 16 in. o.c. (Fig. 9).

The orientation of the barn to the wind and the manner in which it failed are almost identical to those of the Moysey barn. The wind was from the west, blowing almost parallel to the ridge of the roof. As in previous cases the sequence of failure was: (i) collapse of west wall; (ii) collapse of west portion of south roof, leaving the east portion of the roof intact.

The roof was left lying some 50 to 100 feet from the barn.

Analysis of the Failure

In spite of the fact that the roof in this case is a simple pitched construction as opposed to the gambrel roof of the Moysey barn, the pressures on the roofs of the two barns for this orientation to the wind should be very similar. The failure is identical and the analysis of Figs. 7(a) and (b) for the Moysey barn may apply equally well in the present case.

Conclusions

(a) As in the case of the Moysey barn this failure indicates nothing contrary to wind tunnel results; these explain very adequately the manner in which the barn failed.

(b) The conclusion drawn previously concerning the inadequacy of the spiking is again underlined in this failure. It is noticed in Fig. 9 that the top plate has come away from the bottom plate. Had more spikes been used to join these members the performance of this structure undoubtedly would have been improved. Also it is noticed in Fig. 10 that the boards on the roof have been so securely spiked to the rafters that the latter have split instead of severing at the spiking. This might also have been due to the toe nailing of the rafters to the top plate, and on these grounds the inadequacy of this type of connection is indicated.

BUNDAS BARN

Description of the Barn and Failure

Although this barn (Figs. 12, 13 and 14) had been standing for about 30 years, it appeared to be of substandard construction with 2-by 4-in. studs and rafters on 24-in. centres and one side sheathed with 8-in. shiplap.

Although it was similar in shape to the Moysey barn with the conventional gambrel roof, the orientation of the barn to the wind differed; the ridge of the roof was at right angles to the wind rather than parallel to it. The two upper panels of the gambrel roof were completely removed by suction.

Analysis of Failure

Figure 15 shows the external pressures exerted on a roof of this type when the wind is at right angles to the ridge of the roof according to the wind tunnel studies (2). This diagram clearly indicates that failure occurred in the region of highest suction, evident on the uppermost panels of the roof.

If the large doorway on the west (windward) side (Fig.12) had been open during the storm, the increased internal pressure would have contributed greatly to the forces tending to blow the roof off.

Conclusions

(a) Once again the failure suggests nothing contrary to what might be anticipated from the distribution of pressures as determined by wind tunnel studies.

(b) The evident lack of splintering around the failure again suggests that the structure might have withstood the storm had the spiking been better.

MUTCHA BARN AND MACHINE SHED

General Comments

As can be seen from Figs. 16 and 17 very little remains of either of these buildings. Figure 16 shows what was partly a two-story barn. Some observations, however, can be made:

(a) With the wind coming from the left hand side of Fig. 16, this photograph shows the consequences of inadequate shear bracing to resist racking.

(b) The pressures which would be exerted on the side walls (facing the camera in Fig. 16) would have been suction which could account for the removal of the boards, (unless the owner had removed them!).

(c) The roof on the far side was close to horizontal and the high suction on this type of roof is indicated by the angle of the remaining rafters on the far side.

Figure 17 shows the spot where a machine shed once stood. A failure of this nature where the whole building has been toppled over might yield a numerical value for the overturning moment which would be helpful for comparison with wind tunnel results.

COOK'S BEACH COTTAGE

Description of Cottage and Failure

This cottage (Figs. 18 and 19) faces southwest so that the wind was almost at right angles to the front wall. The dimensions were about 21 ft by 24 ft in plan; it was of frame construction with plywood "ranch wall" comb sheathing siding, furnished, but with a minimum of interior partitioning. Judging by the position of the front floor beam (Fig. 19) in relation to the concrete blocks on which the cottage rests, (and also the one at the rear), the wind moved the cottage back about 1 1/2 in. The cottage was not tied down.

Analysis of Failure

This is an example where some quantitative estimate of the wind's force can be made. The building is a simple shape for which wind tunnel results are available. The wind struck the building almost normal to the front wall. The failure was simple and straightforward.

The pressure distribution under these circumstances is similar to that shown in Fig. 20 where the figures indicate the number of velocity pressures in action (2).

Consider the forces on this structure if it is just about to slide (Fig. 21). Let the resultant vertical uplift force (calculated from the area of the roof with the pressure distribution of Fig. 20) be Q_z; let the resultant pressure and suction forces on front and rear walls be Q_x and Q'_x respectively; let the total weight of the cottage be W and the coefficient of friction between wood and concrete block be μ.

If the cottage is just about to move

$$Q_x + Q'_x = \mu\ (W - Q_z)$$

If the value of μ were obtained in the laboratory or by actually moving the cottage mechanically, and if the value of the wind velocity were known, this equation could be used to check whether the factors indicated by wind tunnel studies are compatible with the results. It would also be possible to compare the design forces prescribed for this area with those which must have been imposed to move the cottage.

References

1. Irminger and Nokkentved, Wind pressures on buildings. (Ingeniorvidenskabelige skrifter), 1st series, NR. 23, Copenhagen 1930. 88p.

2. Schoemaker, R.L.A. and I. Wouters, Wind loads on buildings. National Research Council Tech. Translation TT-488, Ottawa, 1954. 27p.

APPENDIX

A description of wind damage to structures, if it is to be of practical use in the problem of wind loads should include the following items of interest:

1) A description of the structure

(a) all major dimensions and orientation
(b) construction, including details such as bracing, size of main members, nailing (size of spacing of nails), etc.
(c) the roof pitch

2) A description of the failure

(a) the manner of failure: was the surface sucked out or blown in, torn off or lifted off?
(b) if possible the order in which failure occurred, if portions failed consecutively (general and detail photographs should be taken and labelled)
(c) description of material left around perimeter of failure: did the failure occur at joints or through fracture of some material?

3) Direction and velocity of the wind

Wind direction can be estimated roughly from the direction in which the debris has blown; wind velocity is available from the nearest meteorological station or (with caution) from newspaper reports.

4) If it is considered practical by the observer to make a numerical calculation from the failure, give

(a) weights of materials that have failed
(b) the dimensions
(c) any other information

5) A description of the surrounding terrain - is it flat, hilly; is it in a valley? Is there any shielding nearby such as buildings or trees?

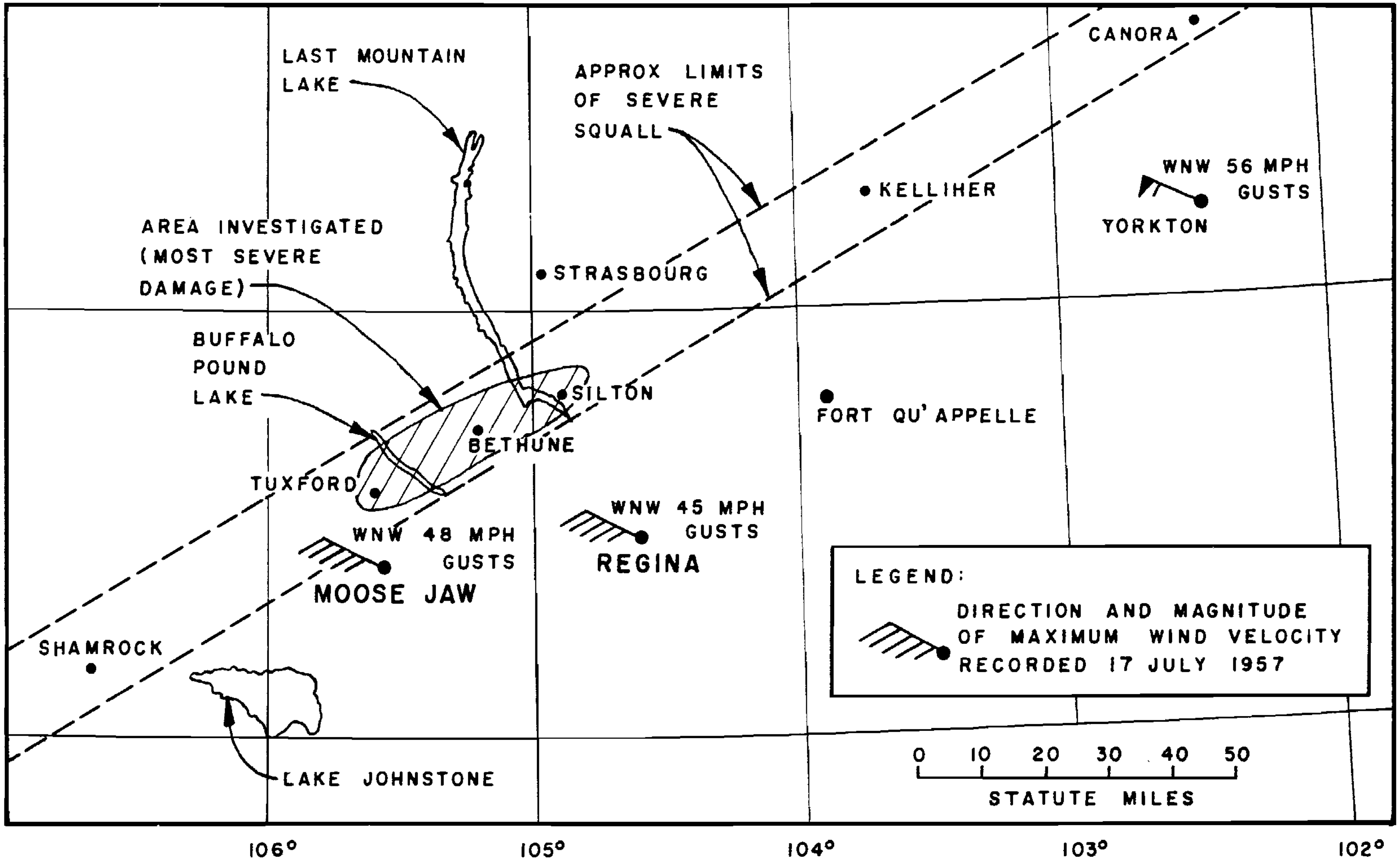

FIGURE I

MAP SHOWING AREA OF SOUTH SASKATCHEWAN AFFECTED BY SEVERE WINDS: JULY 17, 1957

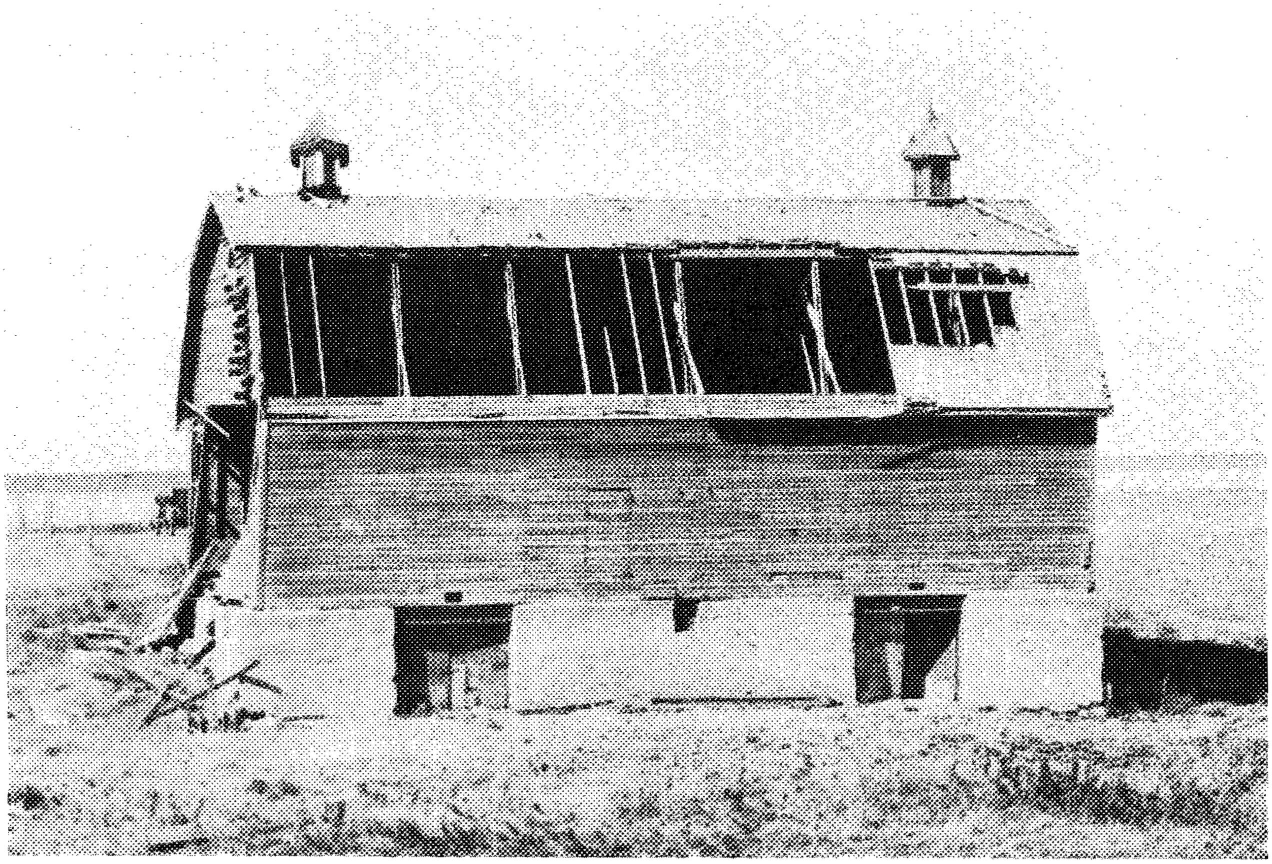

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Figure 2 Moysey Farm, Tuxford, Sask. Damaged barn. (July 27, 1957).

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Figure 3 Moysey Farm, Tuxford, Sask. West wall of damaged barn. (July 27, 1957).

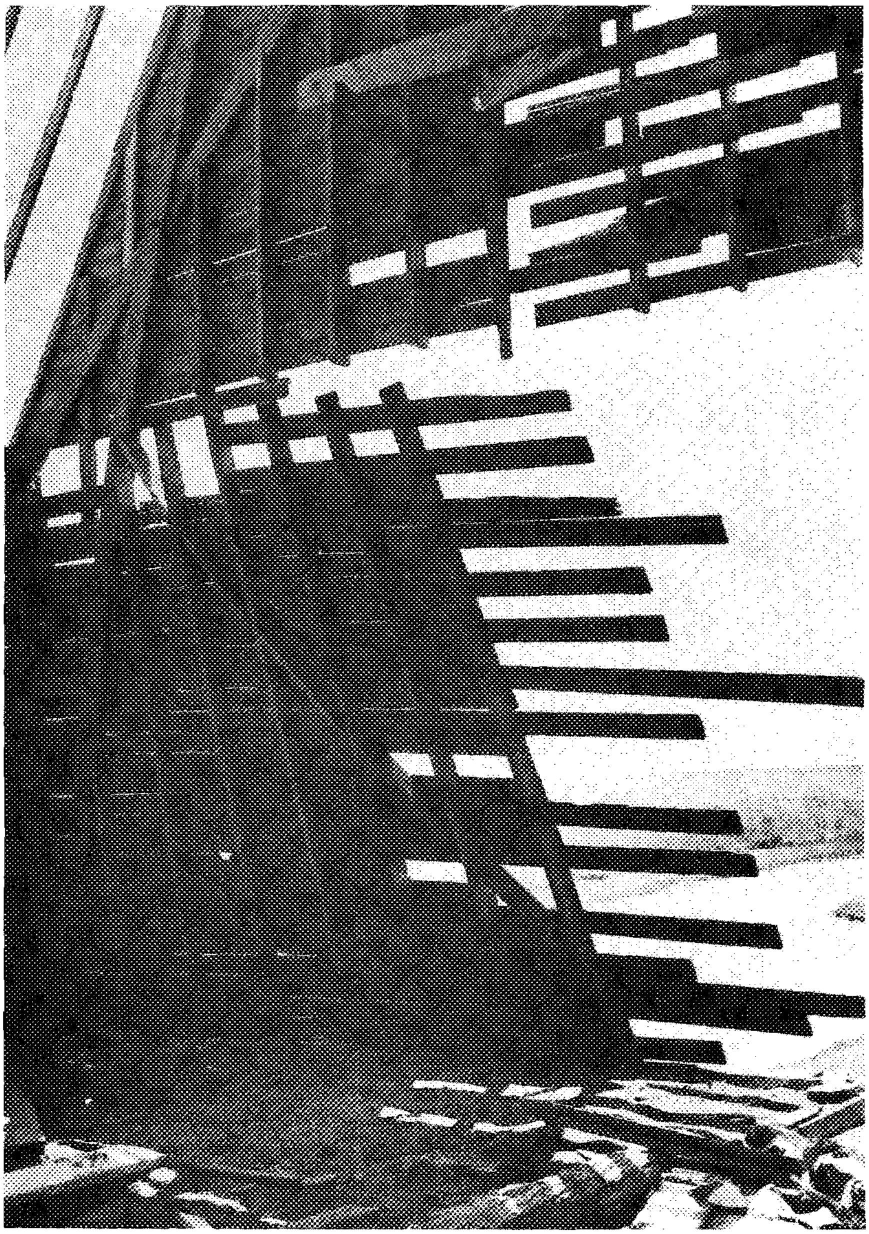

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Figure 4 Moysey Farm, Tuxford, Sask. West wall of damaged barn. (July 27, 1957).

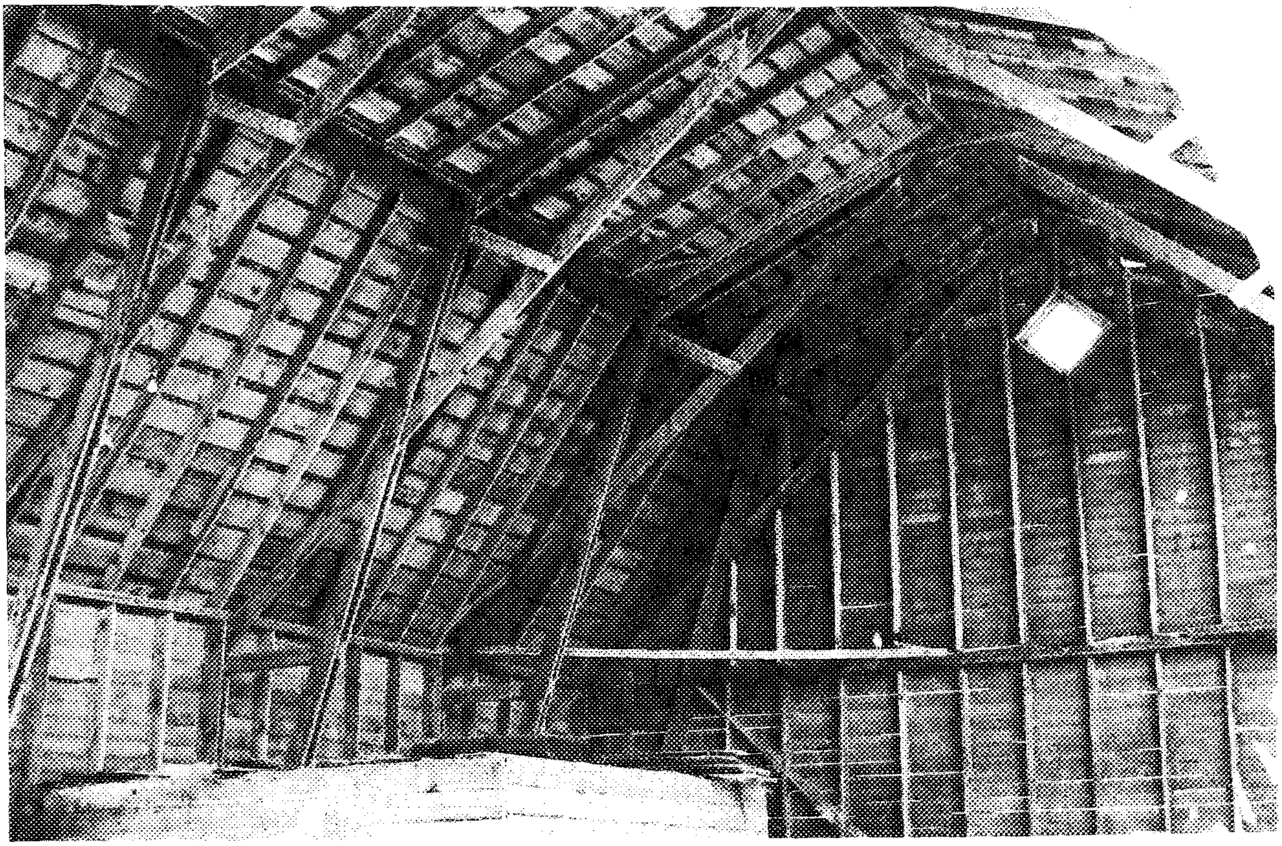

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Figure 5 Moysey Farm, Tuxford, Sask. Undamaged portion of barn roof (looking northeast). (July 27, 1957).

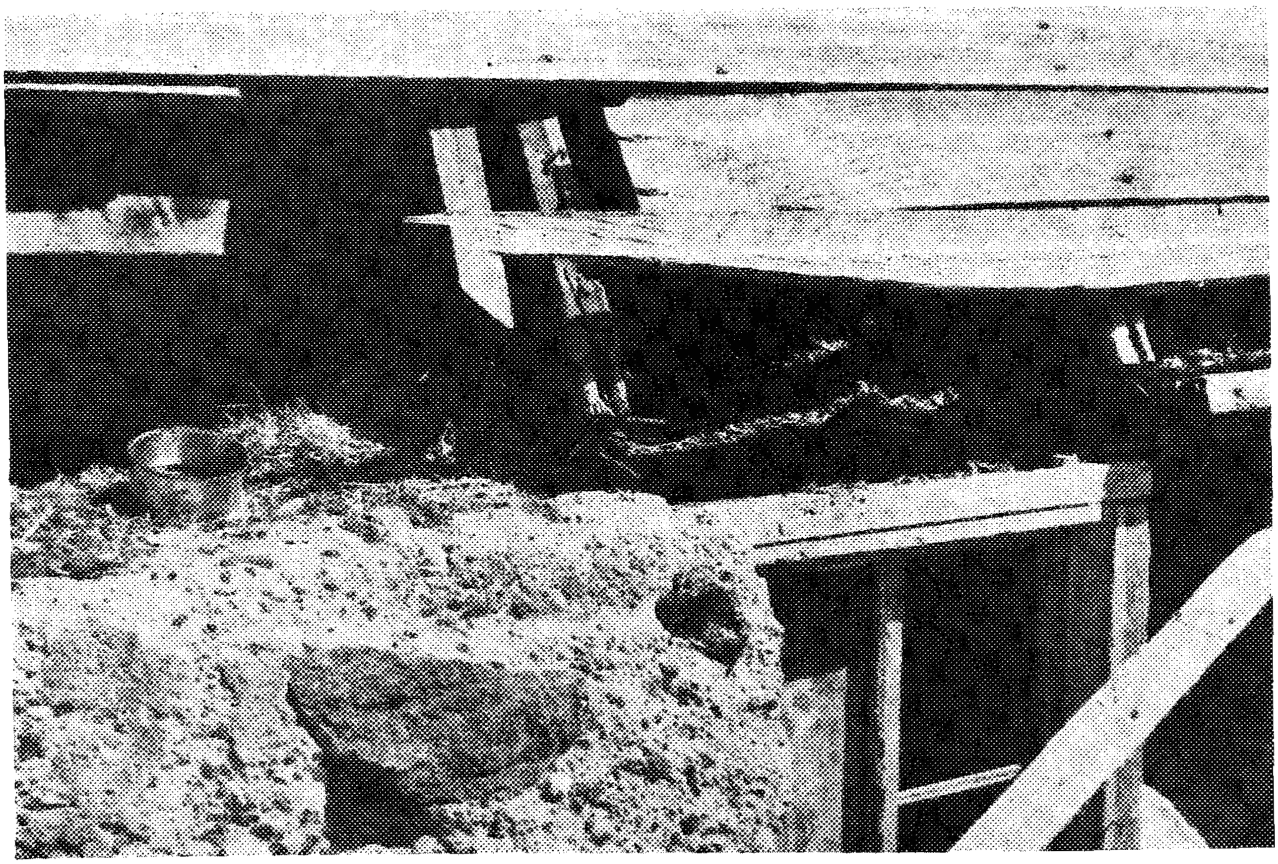

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Figure 6 Moysey Farm, Tuxford, Sask. Plate detail at centre of west end of barn. (July 27, 1957).

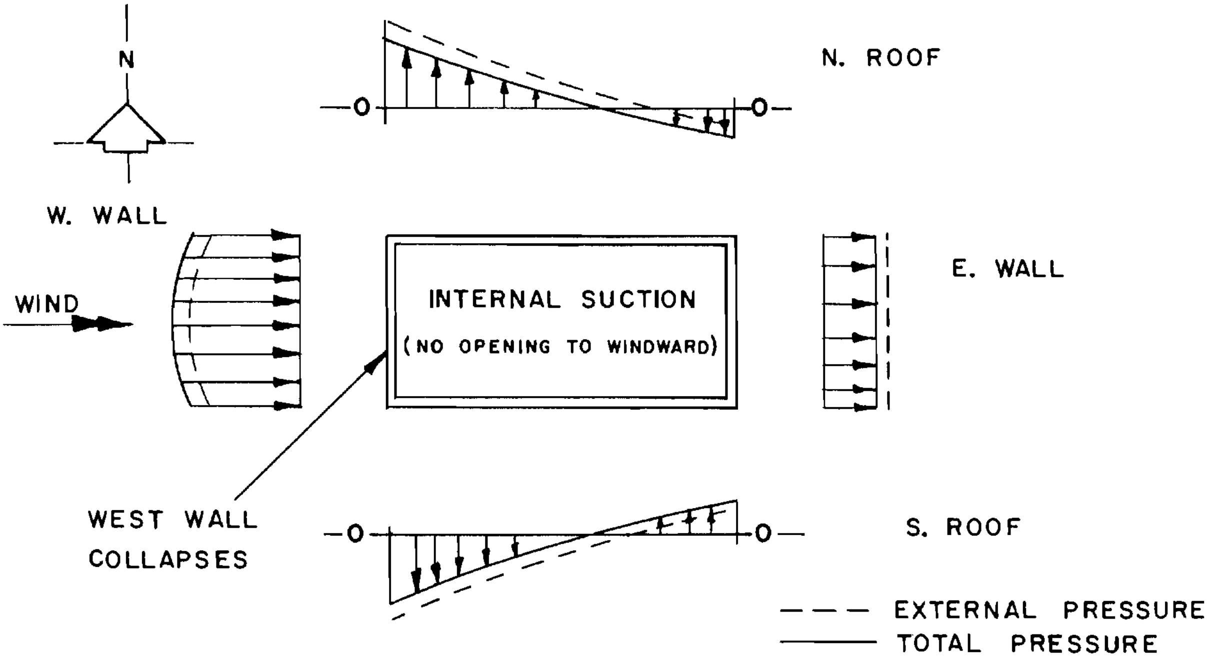

FIGURE 7a

SHOWING RELATIVE PRESSURE DISTRIBUTION ON MOYSEY BARN BEFORE COLLAPSE OF WEST WALL

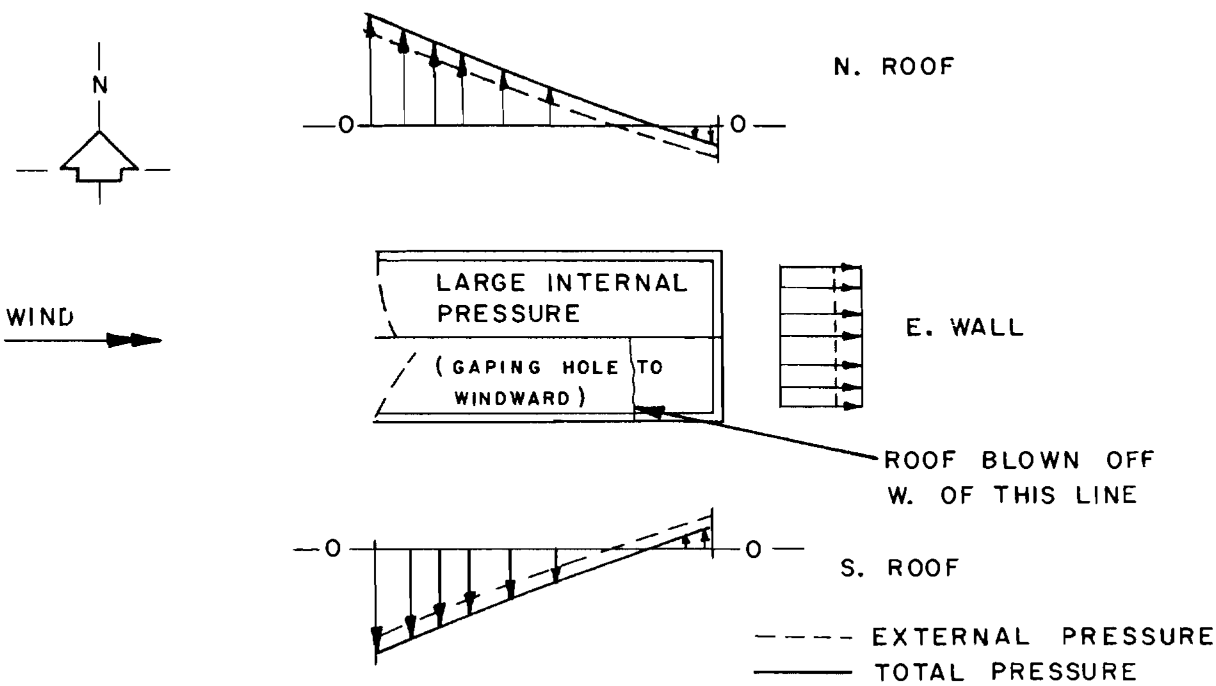

FIGURE 7b

SHOWING THE CHANGE IN THE PRESSURE DISTRIBUTION DUE TO THE COLLAPSE OF WEST WALL

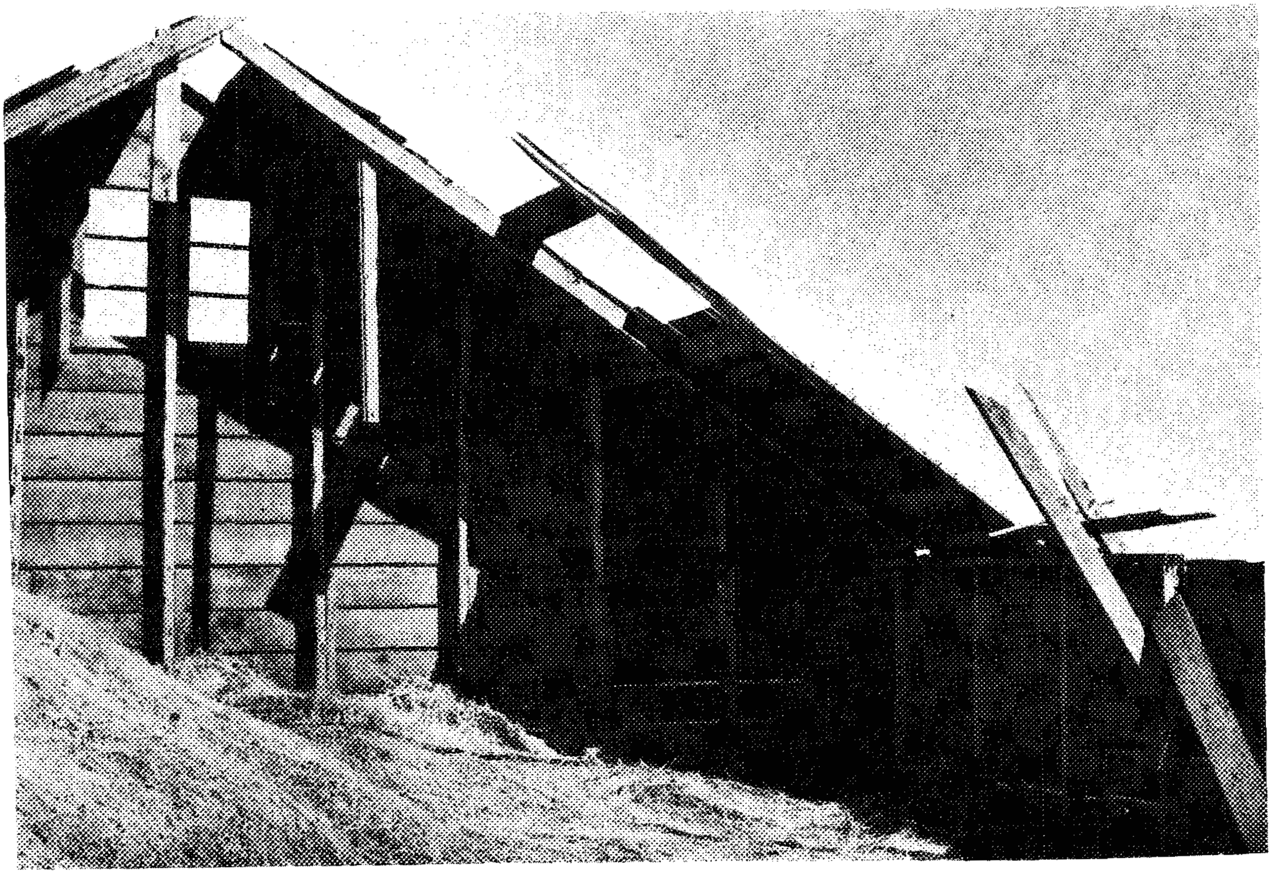

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Figure 8 Seiffert Farm, Bethune, Sask. East gable roof blown off barn. (July 26, 1957).

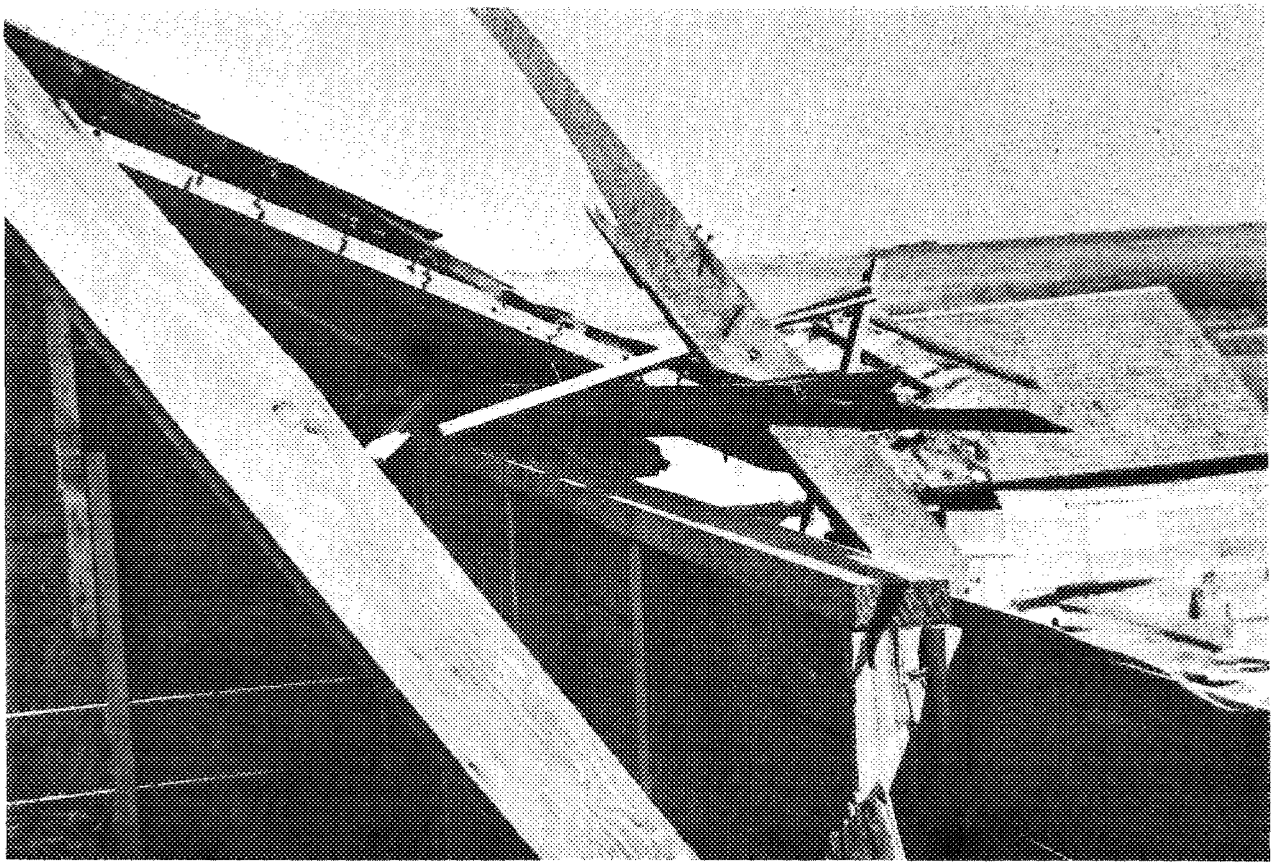

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Figure 9 Seiffert Farm, Bethune, Sask. Detail of Fig. 8. (July 26, 1957).

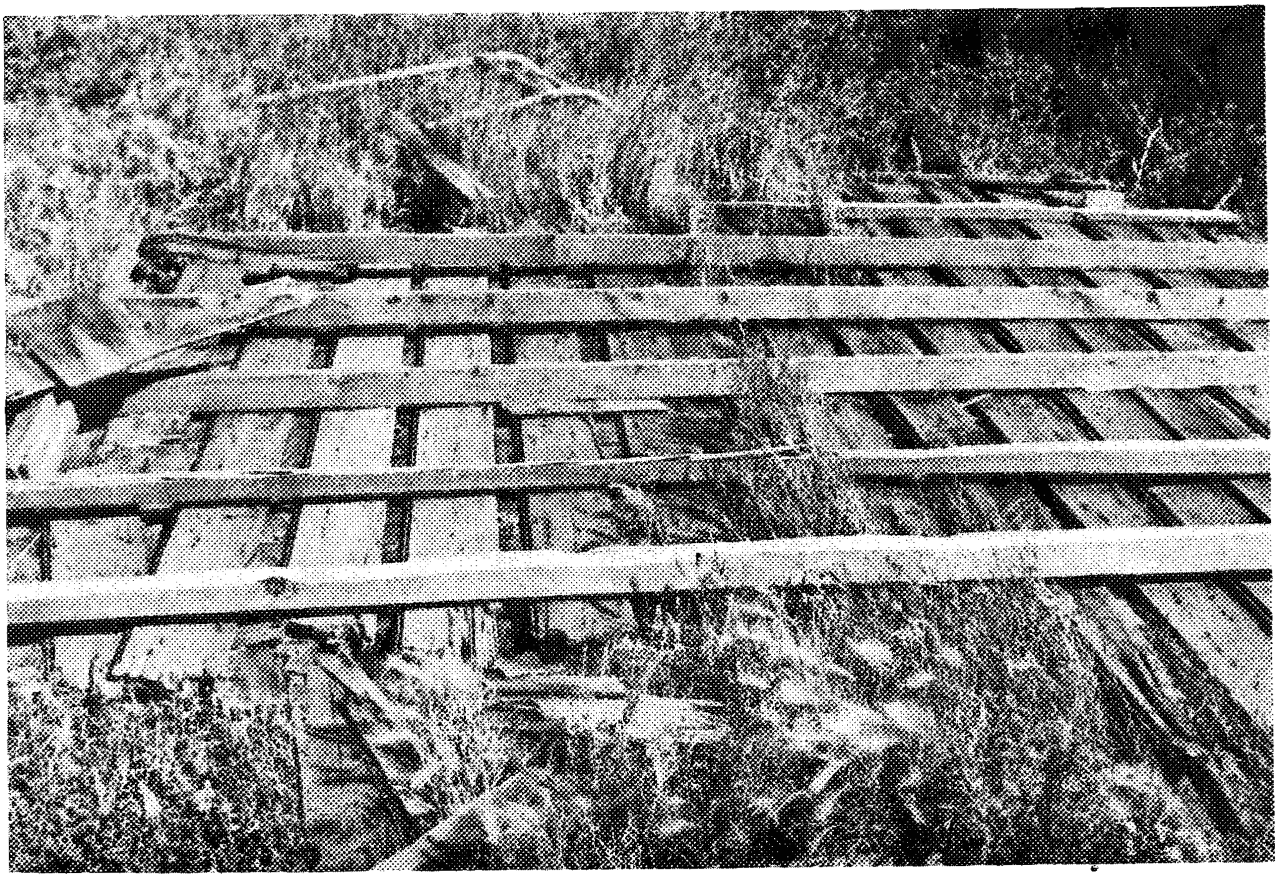

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Figure 10 Seiffert Farm, Bethune, Sask. Roof of barn showing split rafters. (July 26, 1957).

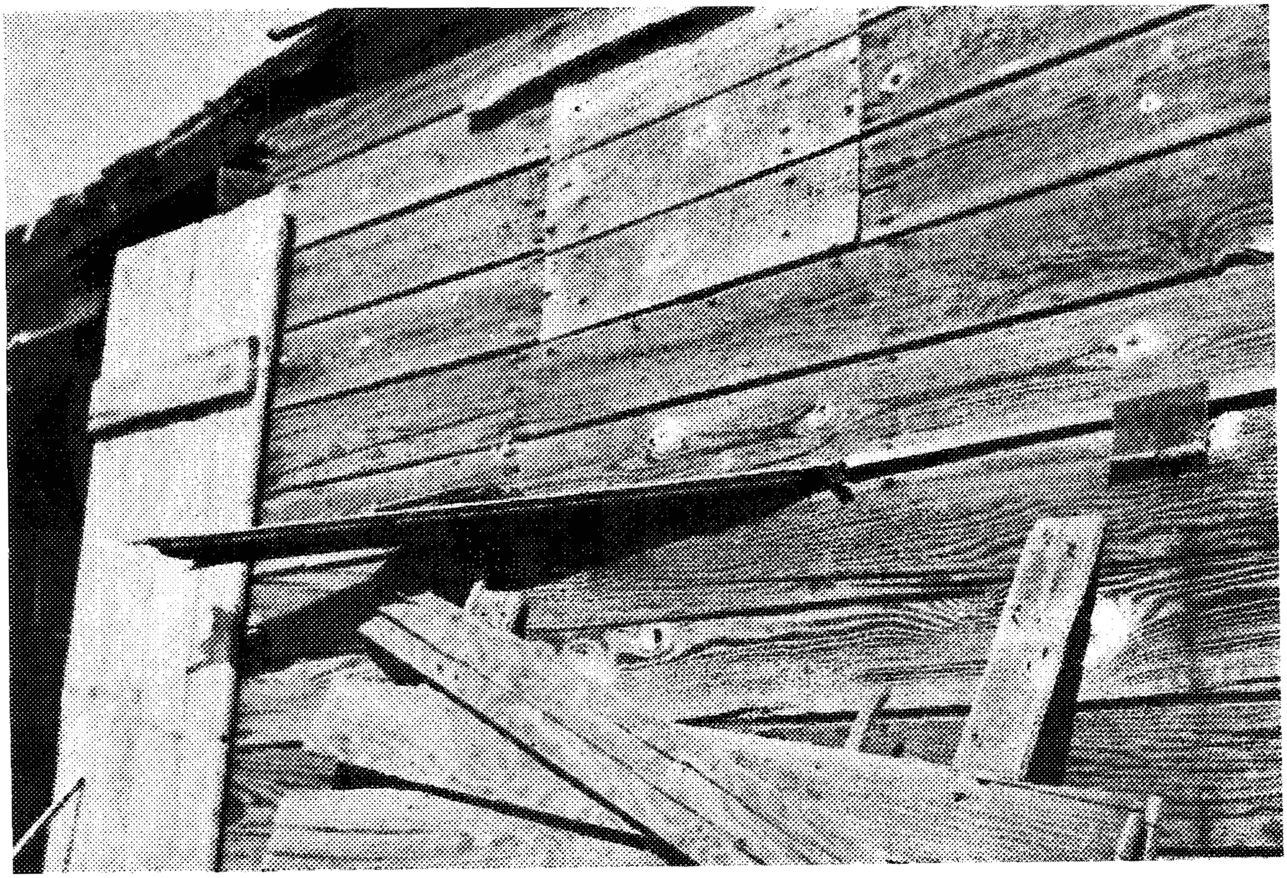

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Figure 11 Seiffert Farm, Bethune, Sask. Barn siding board embedded in granary wall. (July 26, 1957).

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Figure 12 View from south barn on farm of Mr. Bundas, 1 mile north of Silton, Sask. (July 26, 1957).

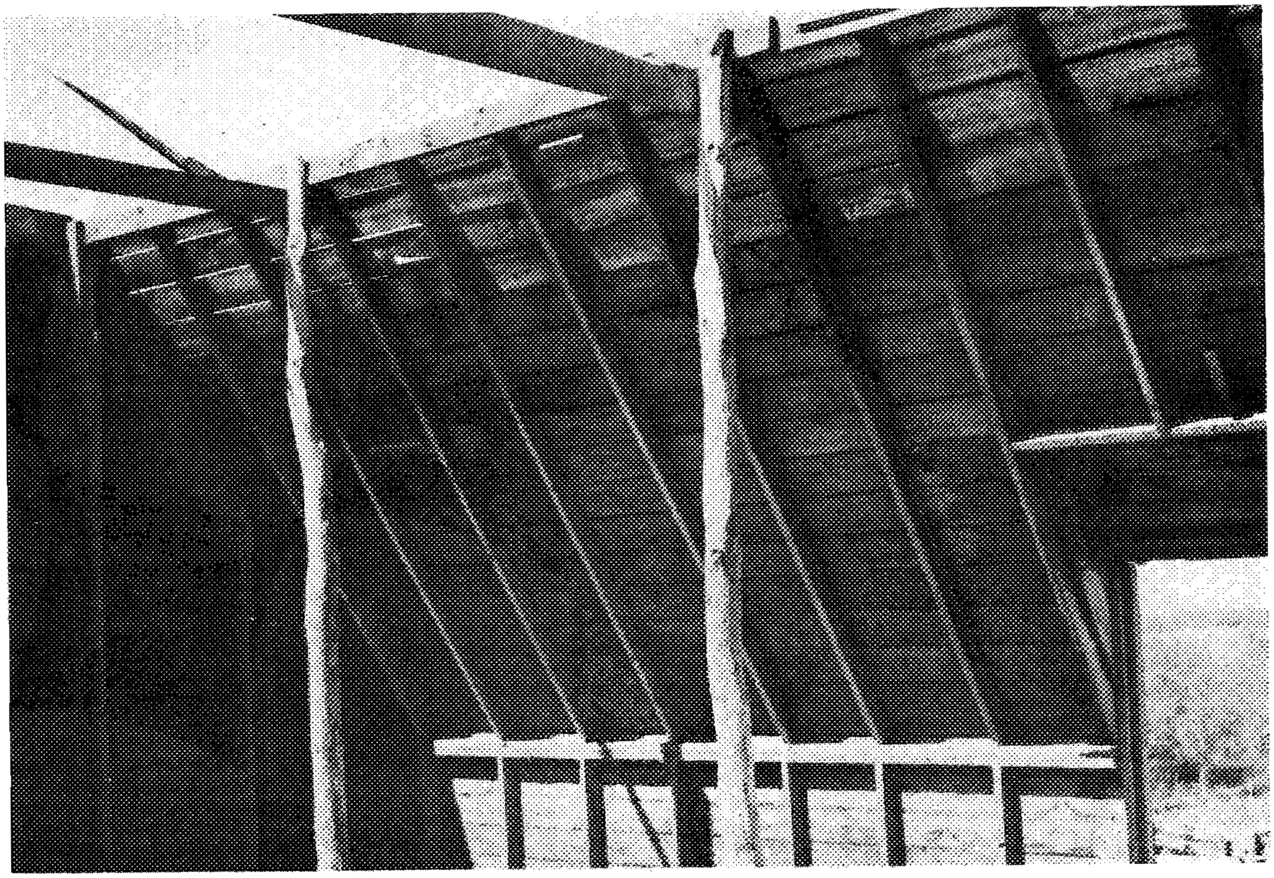

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Figure 13 Interior of barn loft on Bundas Farm north of Silton, Sask. (southwest corner). (July 26, 1957).

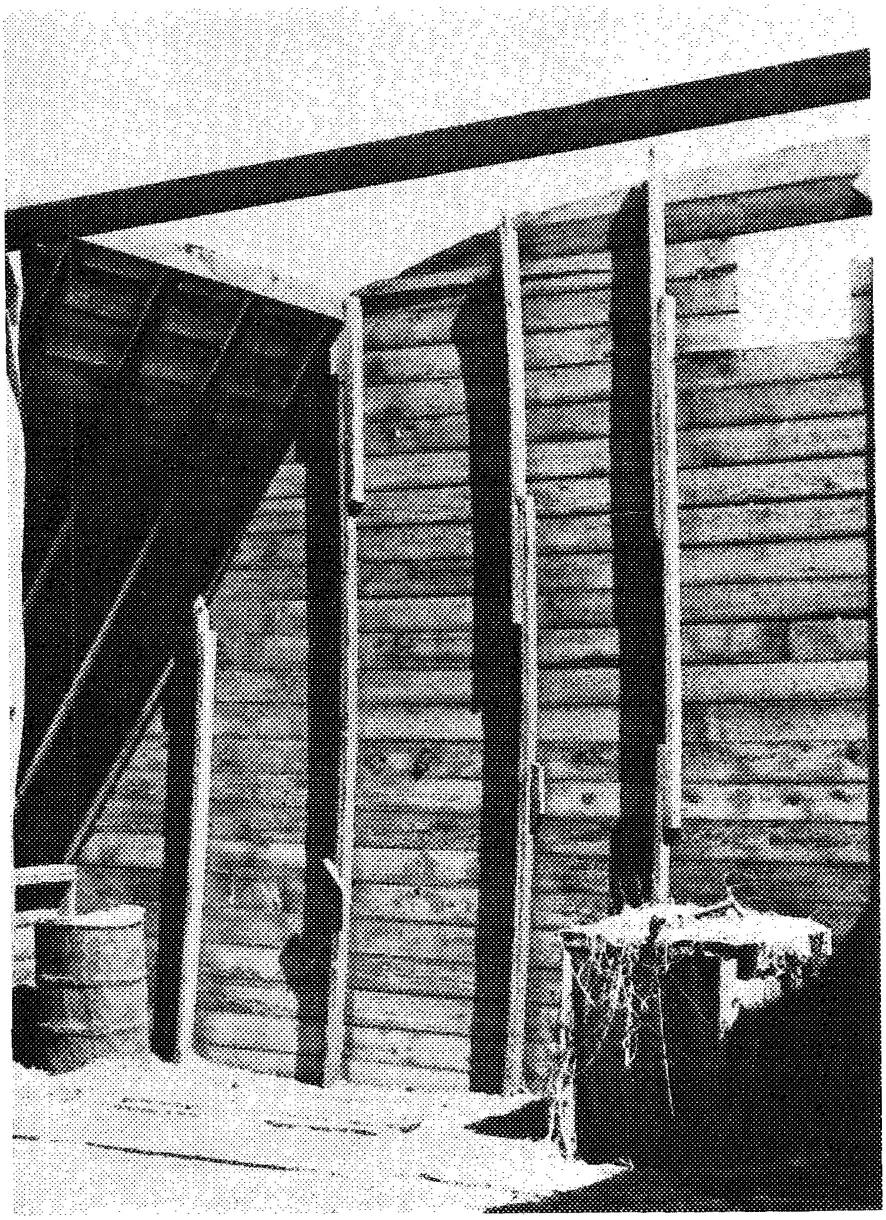

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Figure 14 Bundas Farm, Silton, Sask. Interior of barn loft (northwest corner). (July 26, 1957).

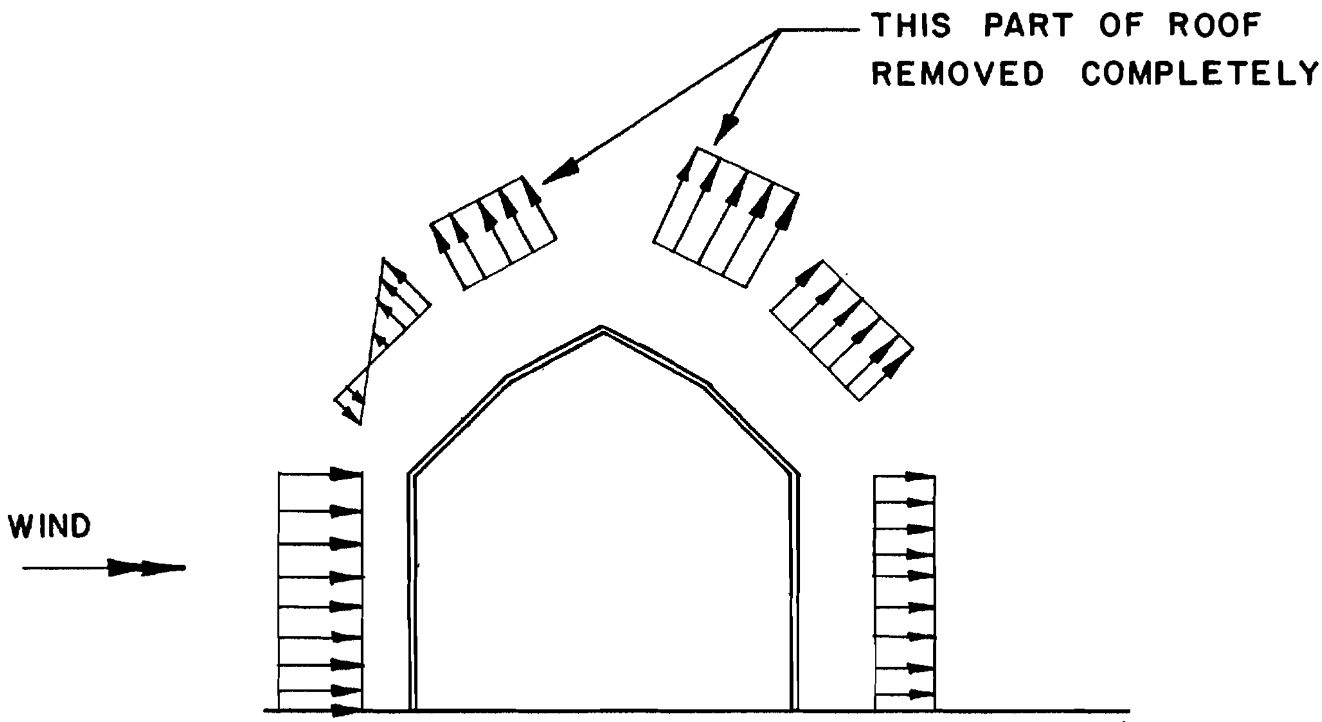

FIGURE 15

PRESSURE DISTRIBUTION FOR DUNDAS BARN ACCORDING TO WIND TUNNEL RESULTS

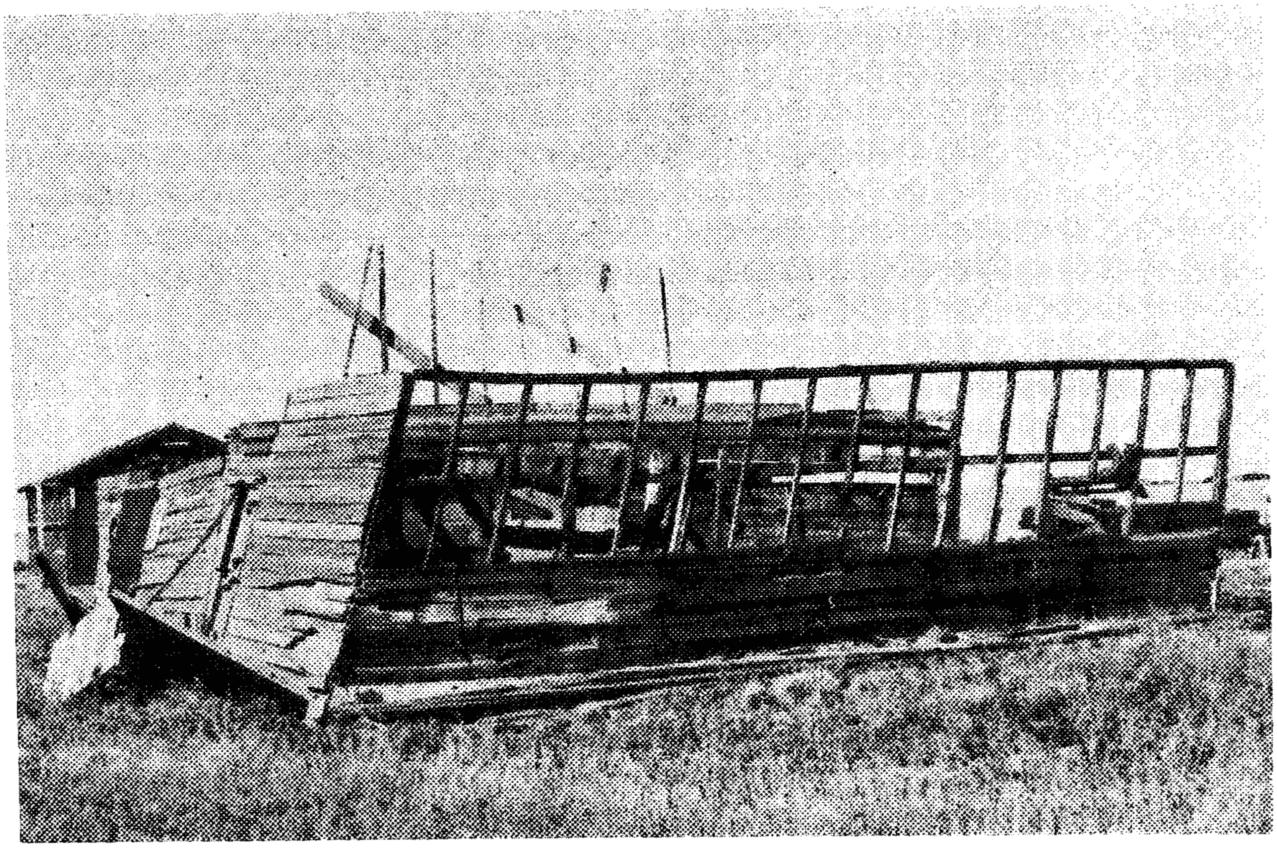

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Figure 16 Mutcha Farm, Bethune, Sask. Small barn demolished by storm. (July 26, 1957).

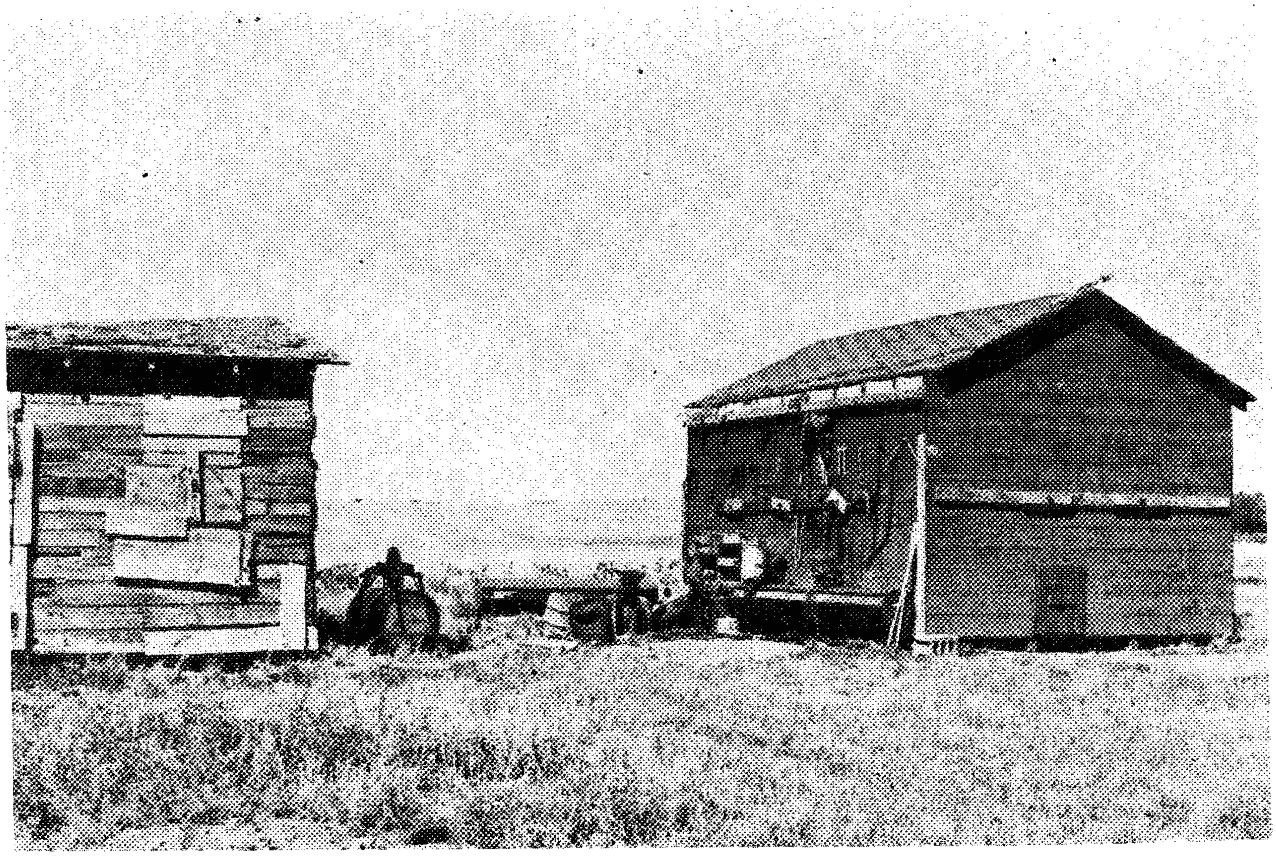

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Figure 17 Mutcha Farm, Bethune, Sask. Machine shed which was blown away from side of granary. (July 26, 1957).

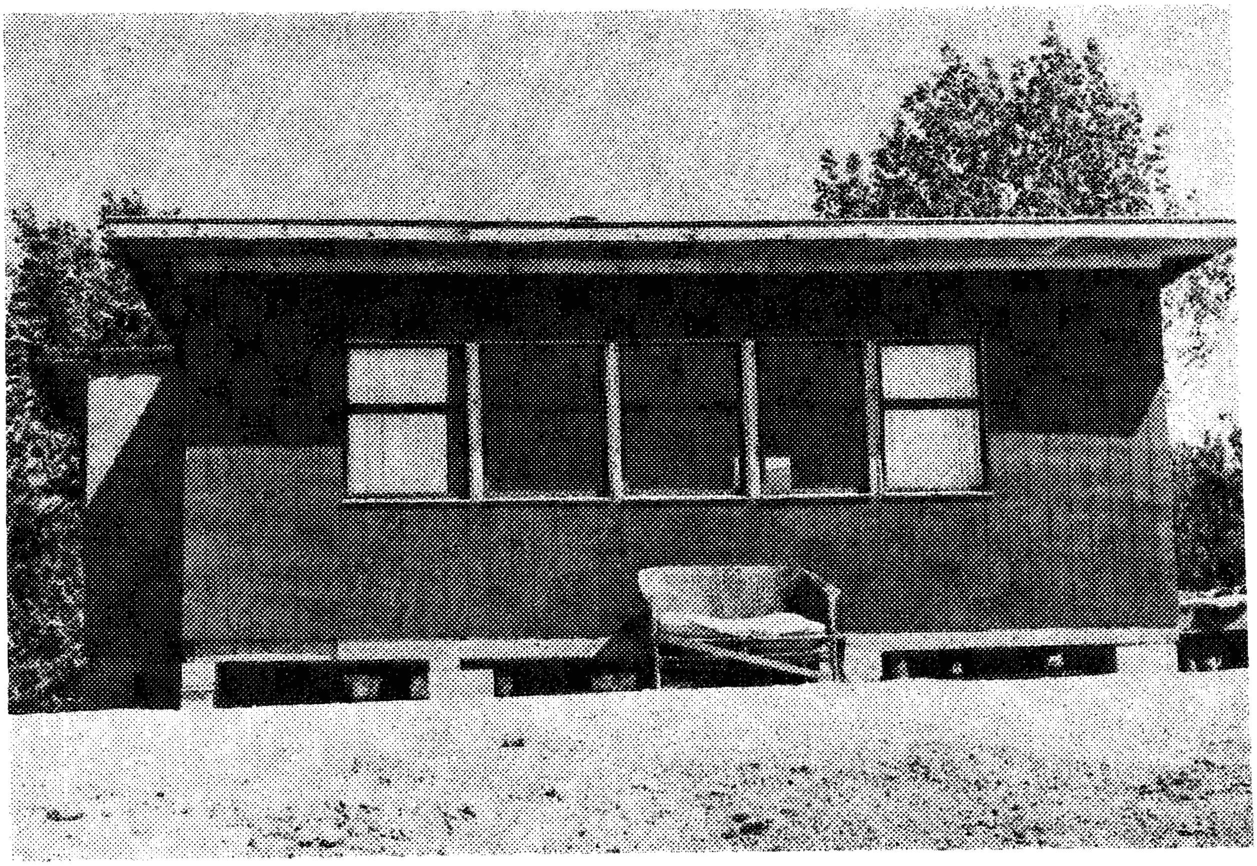

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Figure 18 Beach cottage, Cook's Beach (near Sask. Beach, Sask.). View from southwest. (July 26, 1957).

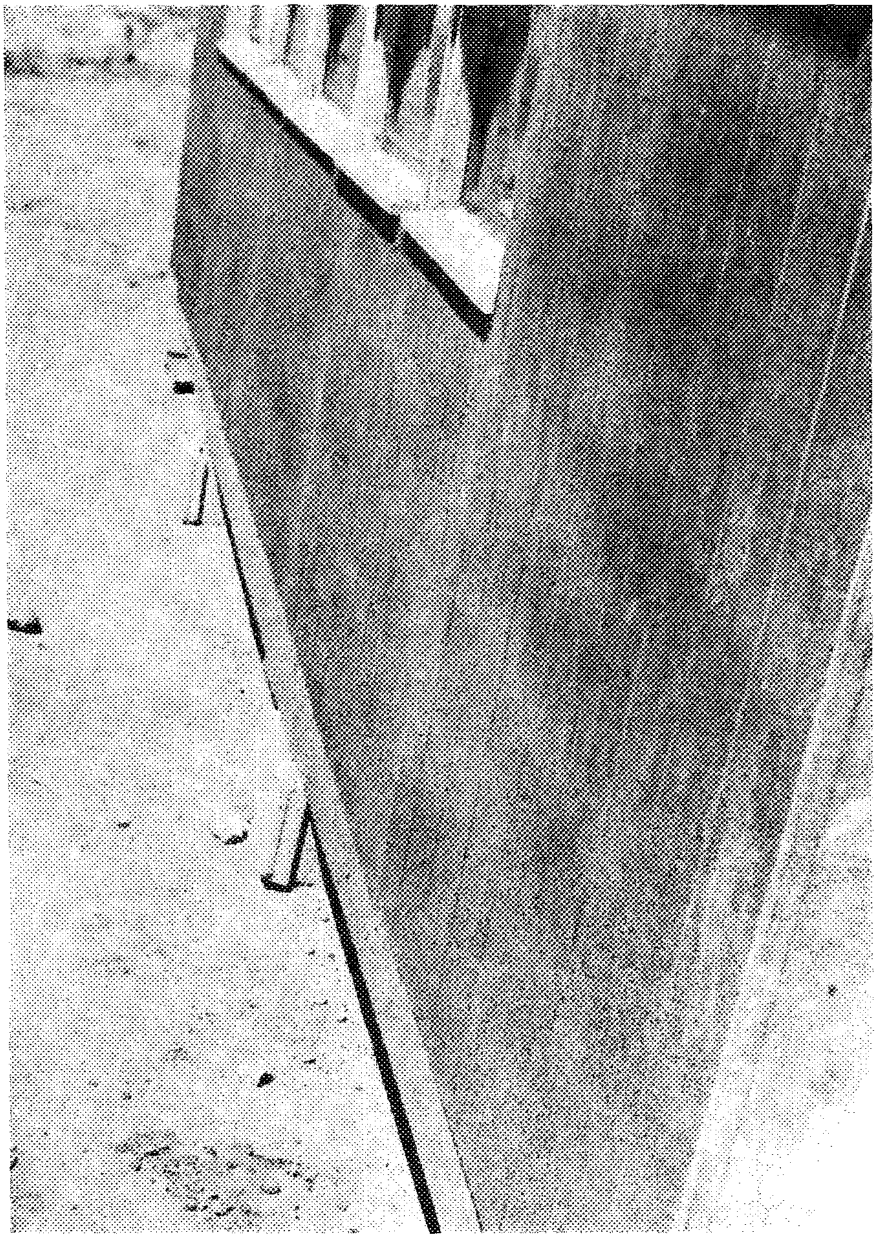

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Figure 19 Beach Cottage, Cook's Beach (near Sask. Beach, Sask.). Wall facing southwest shifted back on foundations. (July 26, 1957).

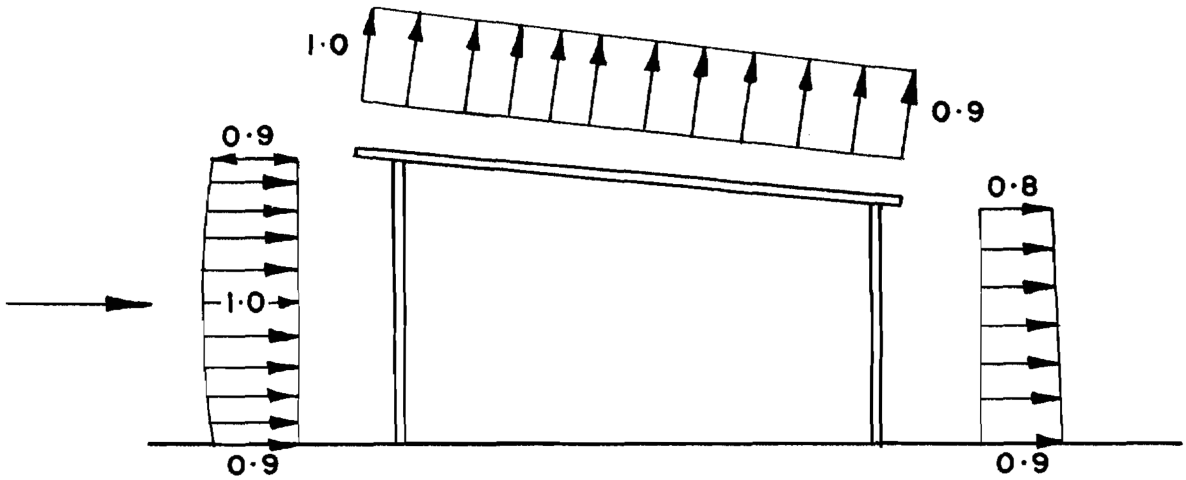

FIGURE 20

SIDE ELEVATION OF COOK'S BEACH COTTAGE SHOWING PRESSURE COEFFICIENTS FOUND FROM WIND TUNNEL RESULTS FOR A BUILDING OF THIS SHAPE

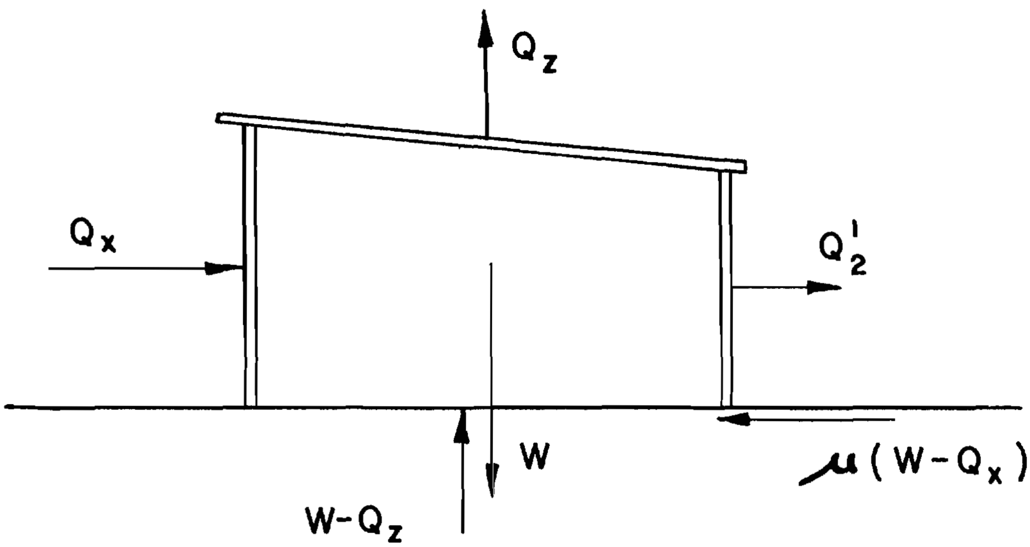

FIGURE 21

FREE BODY DIAGRAM OF FORCES ACTING ON COTTAGE AT POINT OF SLIDING