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BY

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STRUCTURAL TEST OF A HOUSE UNDER SIMULATED WIND AND SNOW LOADS

By D. B. Dorey[1] and W. R. Schriever[1]

House structures, in contrast to industrial buildings and other larger structures, do not generally lend themselves to the normal methods of structural design. For this reason the design of wood frame houses in Canada, as well as in other countries with similar climatic conditions, has developed largely through experience or tradition.

It is often thought that conventional house construction is overdesigned and that substantial economies could be achieved if a means could be found of evaluating the structural strength and rigidity of a completed house. This need for evaluating the structural sufficiency of a house design has become more pronounced with the introduction of new materials and new methods of combining both old and new materials in house construction. In some cases, a new construction can be compared structurally with conventional construction with some success, but in other cases there is no direct basis of comparison. Even when a comparison is possible, there remains the question of overdesign as it is frequently not known if conventional construction is overdesigned. The structural assessment of house designs must, therefore, be based in part upon information obtained from full-scale house tests in which simulated live loads are applied to a house.

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Shortly after the formation of the Division of Building Research (DBR) of the National Research Council of Canada, in Ottawa in 1947, it was decided to begin a research project on structural testing of full-scale houses. This paper describes the first such test, conducted by the Division in the summer of 1954.

Work on this project was begun with a literature survey of pertinent work by other research organizations. It was found that work in this field had mainly been done in the United States by the Naval Civil Engineering Research and Evaluation Laboratory at Port Hueneme, Calif.; in Great Britain by the British Building Research Station; and in South Africa by the National Building Research Institute.

DBR Experimental House

The house chosen for the structural test was one that had been built in 1948 for experimental purposes but not particularly for a structural test. The floor plan and cross-section of this house are shown in Figs. 1 and 2, respectively. The house was a basementless, one-story, wood-frame dwelling about 36 ft long and 24 ft wide. It had three bedrooms, a living room, a kitchen, a bathroom, and a utility room.

The foundation walls were of concrete blocks and extended to a depth of 3 or 4 ft below grade to bedrock. Sill plates, 2 by 6 in., were fastened to the top of the foundation wall with anchor bolts.

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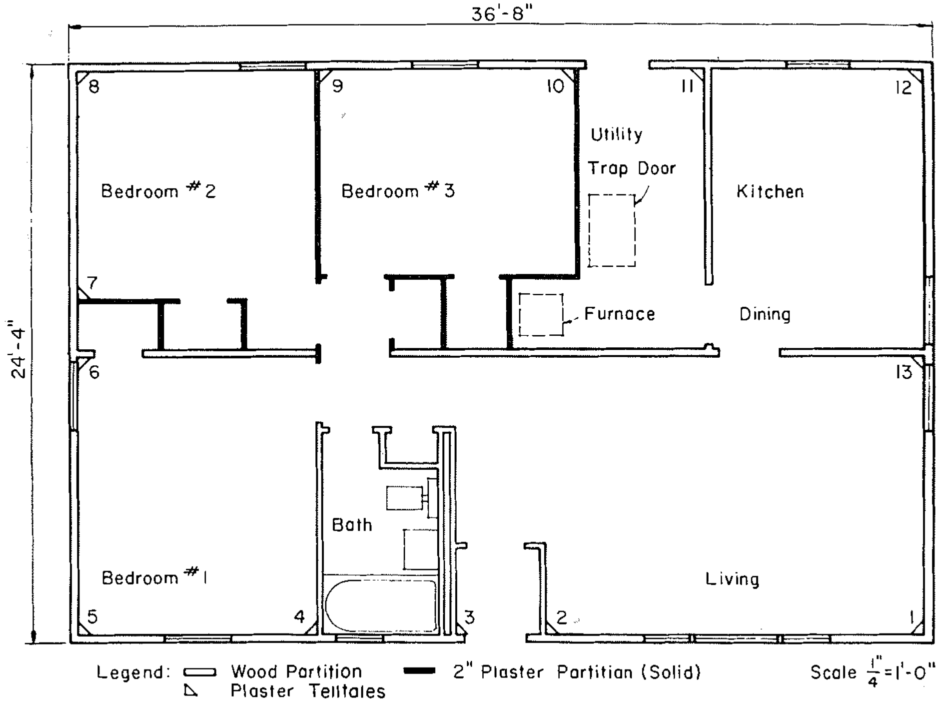


FIG. 1.—Floor Plan of Experimental House No. 2.

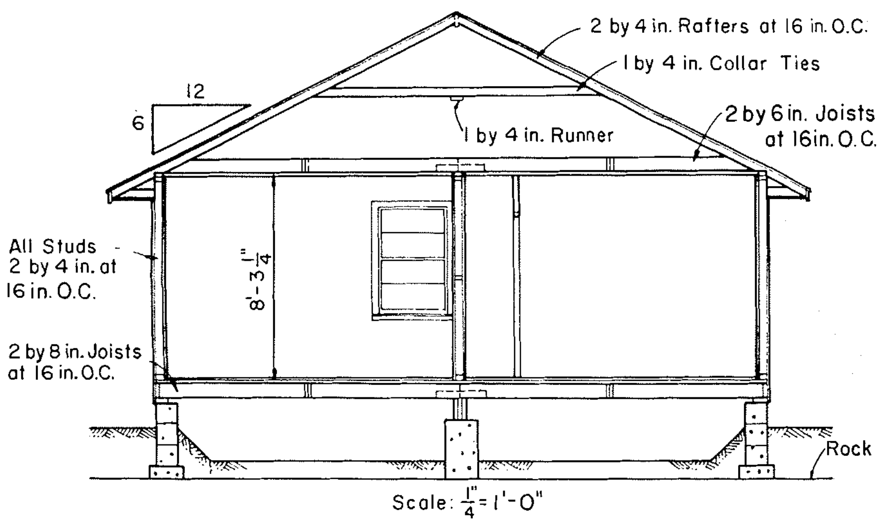


FIG. 2.—Cross-Section of Experimental House No. 2.

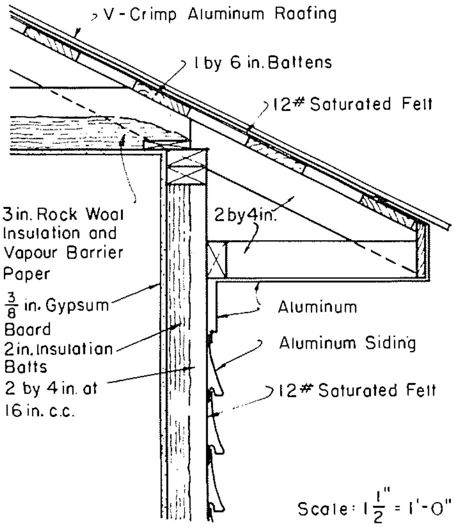


FIG. 3.—Typical Wall Section.

The floor consisted of diagonal subflooring, $\frac{13}{16}$ in. thick, covered generally by hardwood flooring $\frac{3}{4}$ in. thick.

The exterior walls were constructed of 2 by 4-in. wood studding placed on 16-in. centers, and braced by 1 by 4-in. diagonal members let into the studs at each corner of the house. Horizontal 2 by 4-in. girths were placed between the studs at the midheight of the studs. The studding was covered on the outside with 12-lb asphalt-saturated felt paper, over which interlocking aluminum siding, $6\frac{7}{8}$ in. in width, was nailed to the studs (Fig. 3). Figure 4 shows part of the unclad framing and the aluminum siding being applied to the wall studs over the saturated felt.

An important and unusual feature of

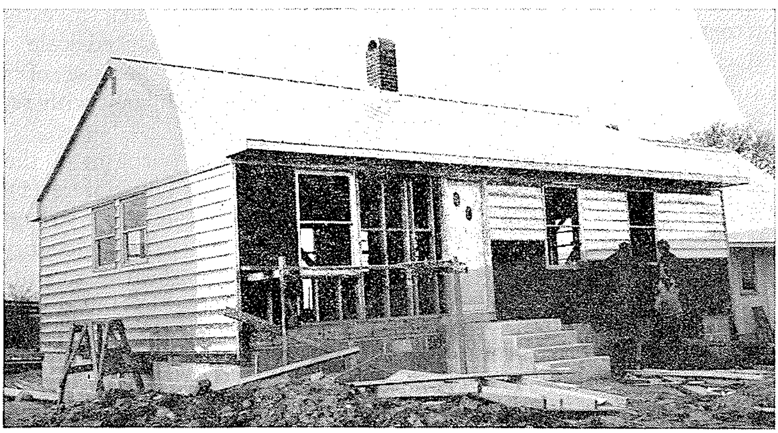


FIG. 4.—View of Exterior Wall during Construction Showing Framing Details and Aluminum Siding being Applied.

Floor joists, 2 by 8 in., 16 in. on centers, spanned each half width of the house and were supported at the center of the house by a wooden beam made up of 2 by 10-in. planks, running lengthwise through the center of the house. This beam was supported in turn by the end walls of the concrete block foundation and by piers at approximately the third points of its span.

the construction of the exterior walls was the omission of exterior sheathing. Exterior sheathing normally contributes to the strength and stiffness of conventional exterior wall construction; it was not included in this structure so that a study could be made of the effect of such an omission.

The roof framing consisted of 2 by 4-in. rafters placed on 16-in. centers on a 6 in 12 slope with 1 by 4-in. collar ties for each pair of rafters. The collar ties were fastened to each other at midspan by a 1 by 4-in. longitudinal member nailed to their underside.

The aluminum roofing was supported by 1 by 6-in. roofing boards which were nailed to the top of the rafters. These boards were spaced approximately 6 in. apart and were covered by 12-lb asphalt-saturated felt. V-crimp aluminum roofing, 30 in. wide, and running the full length of the rafters, was applied parallel to the rafters, over the asphalt-saturated felt. The aluminum sheets overlapped 7 in. along their sides and were double V-crimped at the joints.

The ceiling construction consisted of 2 by 6-in. joists with a 12-ft span, supported near the center of the house by a partition wall. The construction was done according to normal construction practice and does not represent above average workmanship. For example, the lower ends of the rafters were not all nailed directly to the joists at the wall plates, and thus, at some points, the ceiling joists were merely toe-nailed to the top wall plate. The interior finish of the ceiling was gypsum wallboard $\frac{3}{8}$-in. thick.

The central load-bearing partition and most of the other partitions were constructed of 2 by 4-in. studs placed on 16-in. centers. Three partition walls and some closet walls (Fig. 1), however, represented another experimental feature of the house consisting of 2-in. solid plaster walls. Figure 1 also shows the plaster telltales which were installed in a number of interior corners at ceiling level to observe damage resulting from loading. The plaster telltales were made by removing the interior paint from the gypsum wallboard over an area of several square inches on each of the three intersecting planes in each corner and by replacing the paint with returned layers of gypsum plaster to give a better indication of harmful deformations at the junction of the intersecting planes.

PURPOSE AND SCOPE OF STRUCTURAL TESTS

Structural tests that may be conducted on the various elements, components (partial assemblies), and complete assemblies of domestic dwellings are all interrelated. Structural tests on elements such as joists, rafters, studs, or on components such as floors, walls, or roofs, can reveal the strength and stiffness of the specimens but may not always give a proper indication of the strength and rigidity of the complete dwelling, assembled from these components or elements. Full-scale dwellings must be tested, therefore, if a useful attempt is to be made in correlating the results of such investigations. The testing of reduced-scale models would be difficult because of various scale effects, particularly with regard to nailing.

The two main purposes of the structural test on this house were (1) to obtain information on the strength and stiffness of a full-scale single-story house without exterior sheathing, and (2) to obtain experience in full-scale testing and in evaluating the strength of house frames.

A further reason for the test was the hope of obtaining information which would be useful in writing performance requirements for houses in Canada which could be used in connection with the acceptance of new types of construction.

The performance was to be evaluated on the basis of deformation and strength.

Deformations of the load-supporting members under design live load may, and often do, cause unsightly cracks in walls and ceilings. The possibility of such damage seemed particularly real in the house tested, as it had no exterior sheathing. To check this it was decided to apply first the design loadings of the National Building Code of Canada (1953) for wind and snow and to observe deformations and possible damage under these loads.

The strength of the house was to be tested by loading beyond the design live loads to the point of failure. Since, however, it was not the intention to damage the test house beyond the state of possible repair, it was decided to carry the loading only to a point of relatively minor damage.

PREPARATIONS FOR TESTING EXPERIMENTAL HOUSE

Choice of a Loading System:

One of the first problems in the planning for this test was the selection of a loading method. A review of methods used by other research organizations revealed the two most promising methods. The first uses steel cables anchored to the ground at one end and directed over columns through a loading mechanism and a load-measuring device to load-distributing pads on the roof or side wall. The second method makes use of a reaction framework over the test house, inward and outward thrusts being applied by double-acting hydraulic rams.

The first method has the advantage of simplicity of assembly but is not readily adaptable for applying outward thrusts to moderately sloping roof surfaces and has some other disadvantages.

The second method is more complicated to design and assemble but may be justified if the facilities are available. With a rigid reaction framework spanning the test house, simultaneous inward or outward thrusts may be applied and controlled from a central position without altering the test house very much.

After careful consideration, and since the Division had an available stock of Bailey bridging components which could easily be adapted to use, it was decided that the reaction framework combined with hydraulic tension jacks to apply the loads should be used.

Another question that arose during the planning of this test was the choice of the

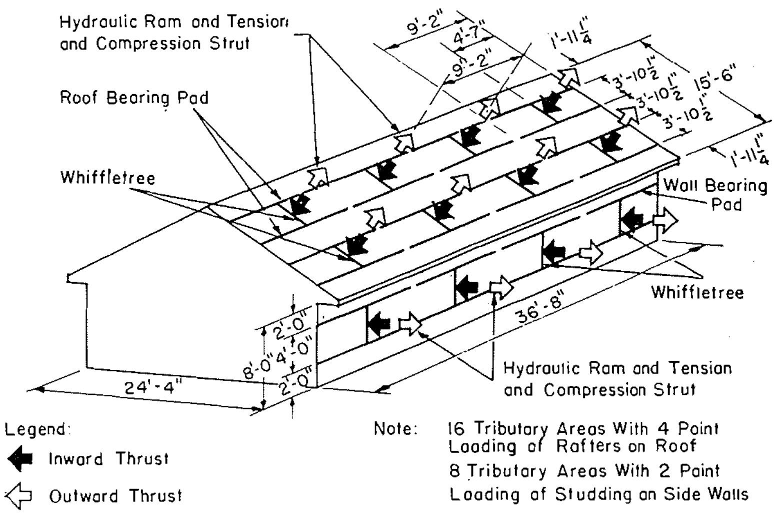


FIG. 5.—Points of Loading.

degree of load distribution to be achieved on the walls and roof. Maximum shears or bending moments could be duplicated in the members by concentrated loads, or a system of loading could be used that would approach uniform loading of the structure as a whole and produce the over-all effect of the uniform loadings assumed in the design. The latter system of loading was decided upon. The roof and side wall loading distribution is shown in Fig. 5.

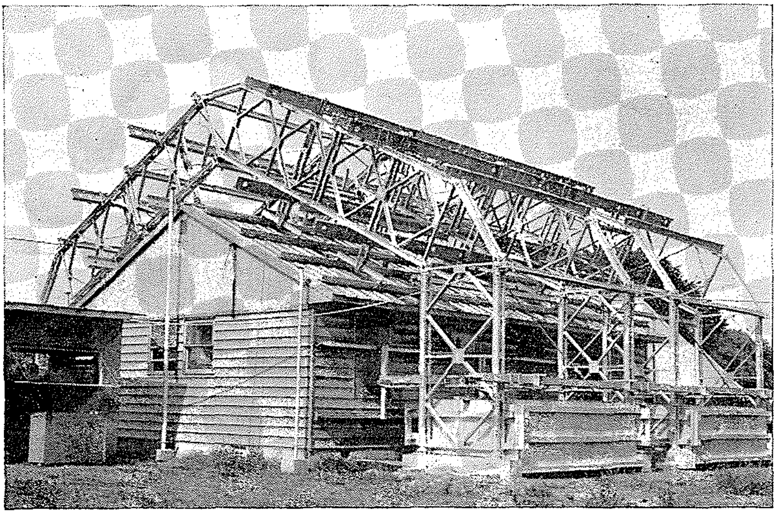


FIG. 6.—General View of Test Assembly.

Figure 5 also shows that the loading system was designed for the various loads to be applied at right angles to the roof and side walls. This system represents the normal wind loading condition but does not represent the correct system for snow loading which should be vertical (loads in pounds per square foot of horizontally projected area). Since, however, the test house did not have exterior sheathing to contribute strength against racking loads, it was originally thought that the wind loads would have the greatest influence on the interior finishes. As the additional work and expense involved in providing a loading system that could be used to apply loads either vertically or at right angles to the roof would have been considerable, it was decided to provide a system that would apply loads at right angles to the roof only and disregard the component of the snow loading parallel with the slope of the roof.

Reaction Framework and Loading Members:

The reaction framework spanning over the top of the test house was composed mainly of Bailey bridging, with additional specially designed components. This framework was supported by special foundation pads and weighted down by large boxes filled with crushed stone. The erection of the steel work was done by an outside firm with the help of a mobile crane.

Each roof slope was divided into 16 equal tributary areas of loading and each side wall into 8 areas, as indicated in Fig. 5. Hardwood loading pads were installed on the side walls and the roof and bolted to the 4-in. I-beams. Sponge rubber $\frac{1}{8}$

in. thick was used between the surface of the roof and the hardwood to provide a compressible medium between the two surfaces. On the roof the pads were bonded by glue to the aluminum sheeting so that both an upward and downward force might be applied. Later during the test a partial breakdown in the glue bond occurred, making it necessary to fasten the pads by wires to the rafters through the roofing. The side wall loading pads were fastened to each stud with steel wire through the wall.

Pairs of loading pads were then connected at their midpoint to a cross-whiffletree which in turn was connected by a pin joint at its center point to a tension and compression strut. Each of the 24 tension and compression struts were restricted to unidirectional movement at right angles to the plane being loaded. Hydraulic rams were later mounted between the channels of the tension and compression struts. Figure 6 shows a general view of the test assembly.

Deflection Measuring Apparatus:

A system of pulleys and wires was used inside the test house to measure the deformations under load. This is a convenient system to measure in a central location deformations at widely separated points of a structure. It can be used when the deformations are expected to be large enough to be read without magnification.

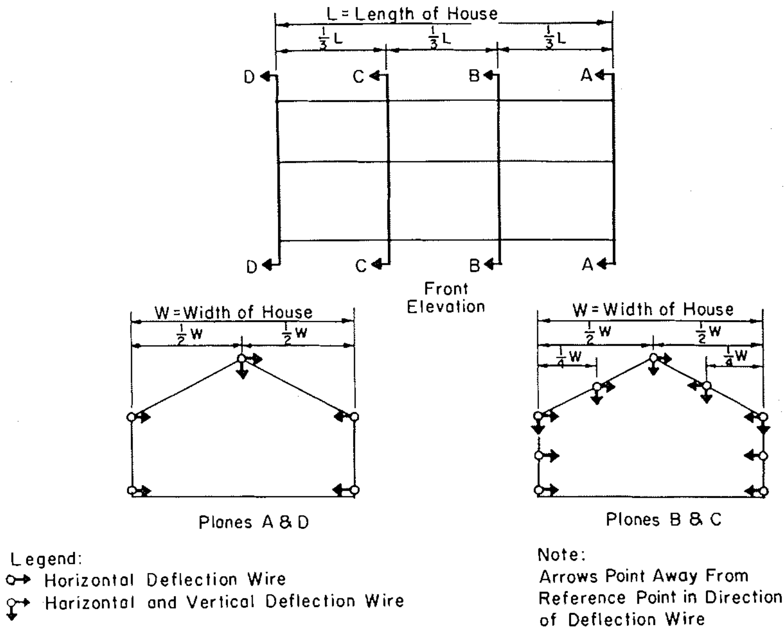


FIG. 7.—Planes of Deformation.

Piano wire, 0.010 in. in diameter, running over a system of $2\frac{1}{2}$-in. low-friction aircraft pulleys, was installed to register horizontal and vertical movements on four deformation planes, one at each of the third points in the interior and one at each end wall on the exterior as shown in Fig. 7. The pulleys, over which the wires are running, must not be affected by the movements of the house during the test. In this case the chimney, which is near the center of the house, was used to support the deflection apparatus inside the house and all connections between the chimney and the house were removed.

Two additional wires were installed on one of the end-wall planes to indicate vertical and endwise movement at the ridge-board level of the test house, and one wire to indicate whether the deflection apparatus had been disturbed. This made a total of 43 wires.

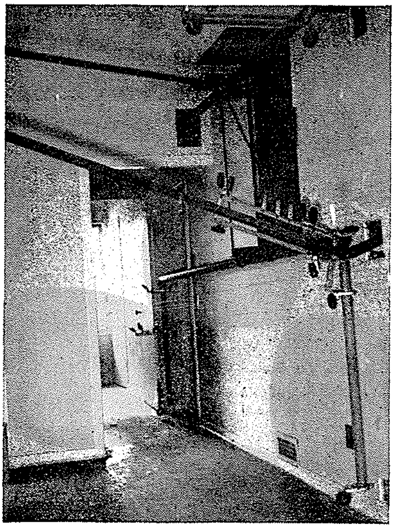


FIG. 8.—Interior Deflection Apparatus.

All the wires leading from the various points in the house were brought through one window opening in the end wall to a deflection board in the instrument hut. Figure 8 shows the deflection apparatus in the living room and hallway and the way in which the system was attached to the chimney through the larger opening made in the partition wall. Figure 9 shows the termination of the 43 wires on the deflection board, each wire being tensioned by a 1-lb weight, the lower edge of which was used in marking successive deflection readings on the board. The instrument hut also housed the hydraulic console unit and the power supply transformers.

Loading Equipment:

The hydraulic equipment for this test included 24 tension jacks, an air accumulator, a console unit, and special lines and fittings.

All hydraulic rams were calibrated for hydraulic pressures ranging from 0 to 2000 psi before the test. The connections to the hydraulic rams on the struts and Bailey transoms were so designed that the direction of the force exerted by the hydraulic rams could be reversed, thus making it possible to apply either a push or a pull to the area of loading.

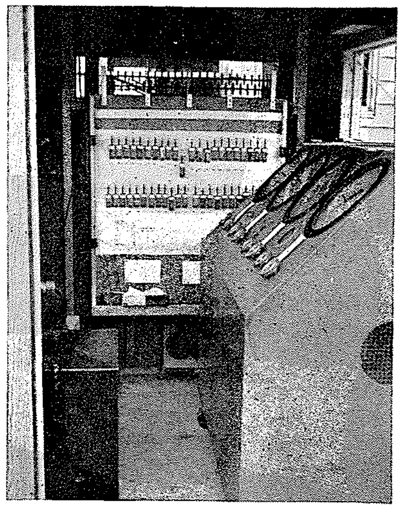
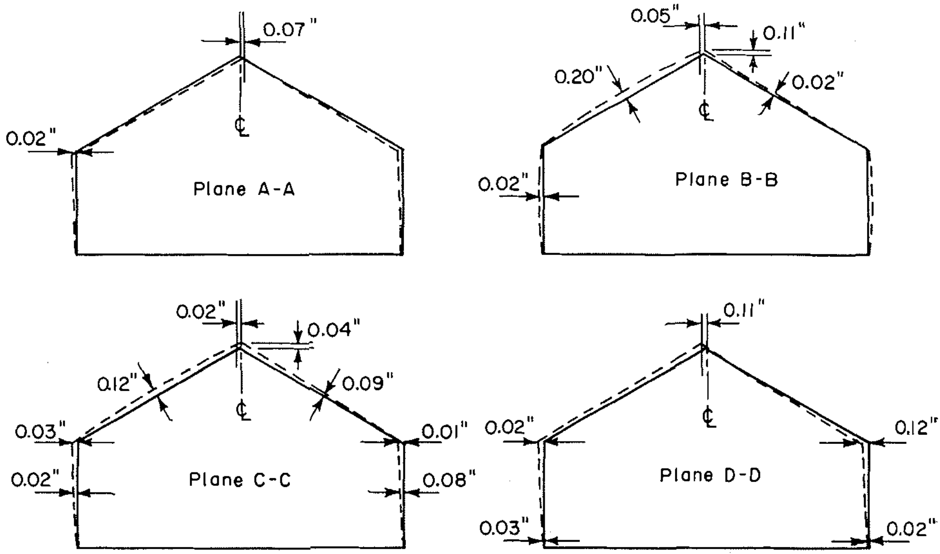
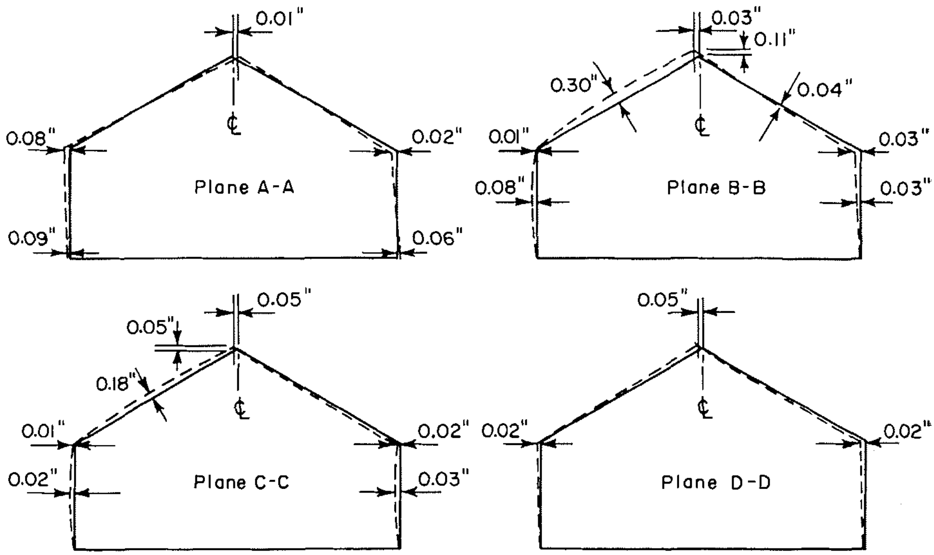


FIG. 9.—General View of Instrument House.

The rams were operated from the console with four pressure channels, each of which could be operated at a different hydraulic pressure, one for each of the four surfaces of the test house. Four rams of equal thrust capacity were installed on each of the side walls, and eight of equal thrust capacity on each of



(*a*) Wind Velocity 90 mph.



(*b*) Wind Velocity 120 mph.

——— Undistorted Frame
— — — Distorted Frame

Fig. 10.—Deformations due to Wind Loads (Internal Suction).

Notes.—Refer to Figs. 5 and 7 for loading and deflection points. Deformations are not drawn to scale.

Wind Direction: From the right.

the two roof slopes. Each of the four pressure channels had its own pressure gage and operating valves in the console. To facilitate accurate loading an air accumulator, which could be charged with a booster pump to pressures in accord with the requirements of the hydraulic rams, was connected to the console unit.

STRUCTURAL TESTING OF EXPERIMENTAL HOUSE

The sequence of loading phases for this test was from the least to the most severe conditions as follows:

(*a*) Wind loads: internal suction,

(*b*) Wind loads: internal pressure,

(*c*) One-half design snow load plus wind loads, and

(*d*) Snow loads.

According to the 1953 edition of the National Building Code of Canada, 90 mph is the design wind velocity for the Ottawa area and 120 mph is the highest wind velocity that might reasonably be expected in the populated areas of Canada. It was decided, therefore, that the wind loads would first be carried to the design velocity of 90 mph and maintained for 1 hr, then released and reapplied up to a velocity of 120 mph for both an internal suction and an internal pressure condition.

There was some doubt as to what percentage of the design snow loading should be used for the combined loading condition which is not stipulated in the National Building Code. It was decided that the maximum amount of snow which might be expected to remain on the roof under high wind velocities would be not more than $\frac{1}{2}$ the design snow loading.

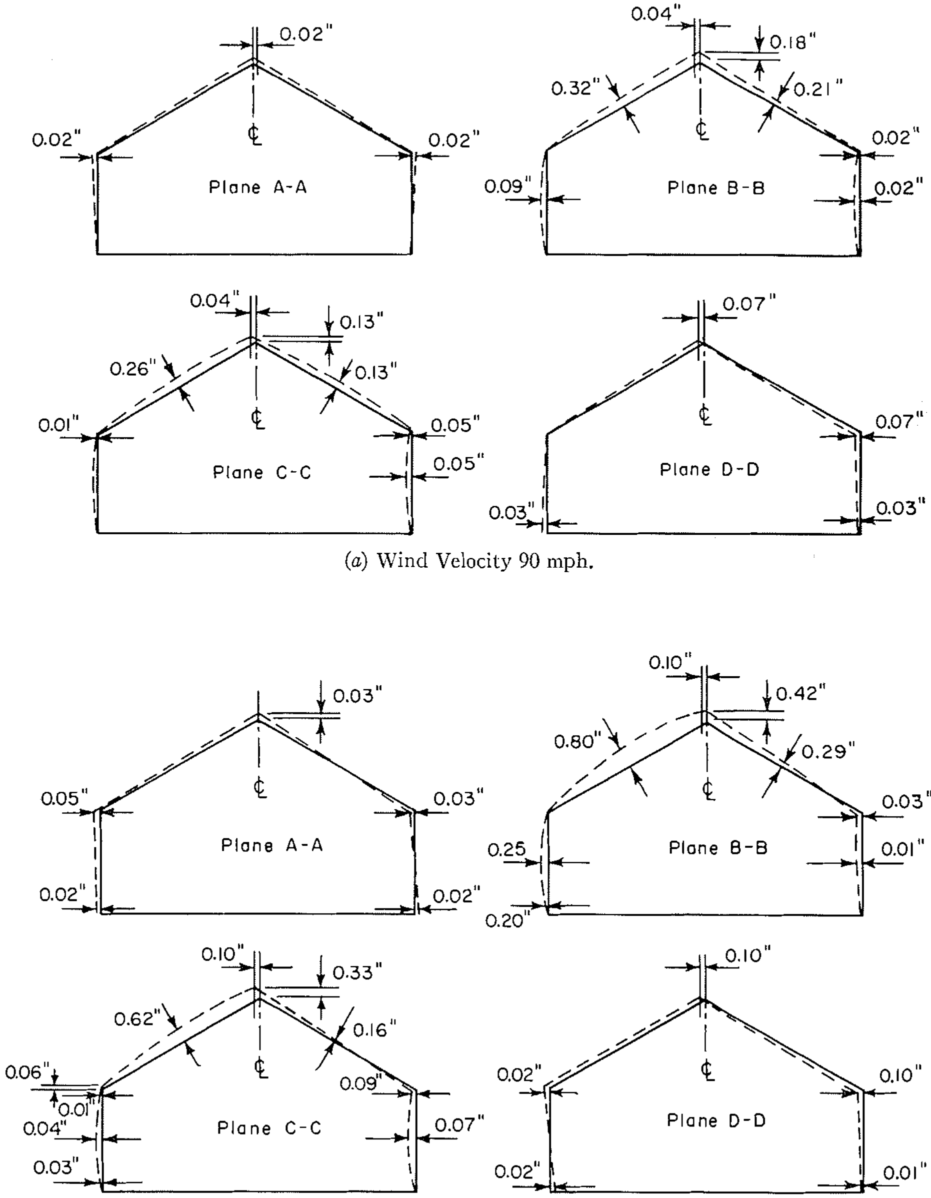
The magnitude of the design snow loading was arrived at by direct computation from the information given in the National Building Code (1953). The snow loads applied during the test were the components of the vertical loads acting at right angles to the plane of the roof.

Wind Loads—Internal Suction:

Internal suction (windows open on leeward side only) was the first wind loading condition applied. This condition is less severe than that producing internal pressure, as it results in smaller loads except on the windward wall. The loads were applied in increments of pressure corresponding to increases in velocity of 10-mph, beginning at 70 mph and ending at the design velocity of 90 mph. The loads were maintained for $\frac{1}{2}$ hr, and the loads for a velocity of 90 mph were maintained for 1 hr. Deformations were recorded on the deflection chart in the instrument hut before and after each increment of loading and after complete unloading. A visual inspection was made of each of the 13 plaster telltales before and after each loading increment to see if any cracks had developed. No cracks appeared during this loading phase. Deformations resulting from the first phase of loading are shown in Fig. 10 (*a*).

The next loading phase was a continuation of the loading schedule given above up to loads corresponding to a wind velocity of 120 mph. Each wind load was sustained for 1 hr after a 90-mph wind velocity loading was reached.

After reapplying pressures equivalent to a 90 mph wind velocity a crack in telltale No. 5 was discovered, and toward the end of the 1-hr period of sustained loading, a second crack became visible in telltale No. 9. All loads were then increased to pressures corresponding to 100, 110, and finally 120 mph and maintained for 1 hr in each case. During the 120-mph loading, cracks appeared in two more telltales, Nos. 6 and 10. The deformations resulting from this phase of loading are shown in Fig. 10 (*b*).



(*a*) Wind Velocity 90 mph.

(*b*) Wind Velocity 120 mph.

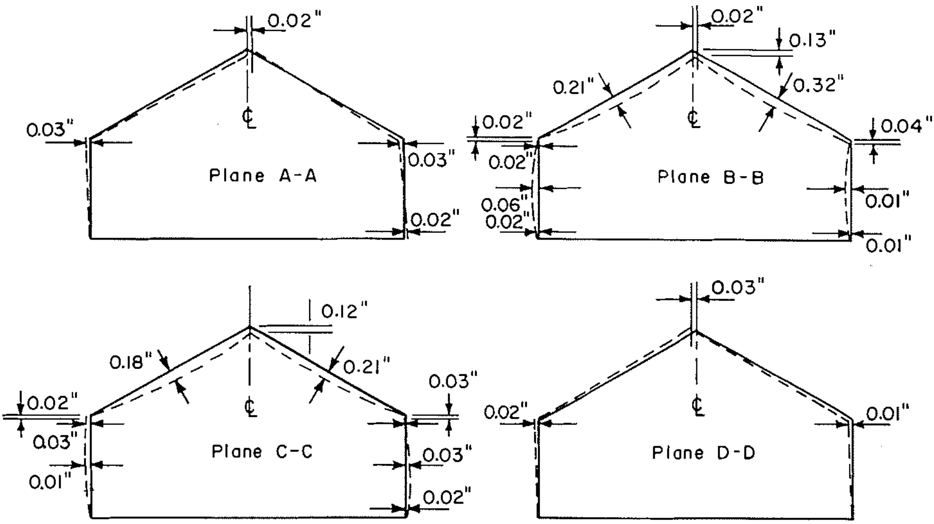
——— Undistorted Frame

— — — Distorted Frame

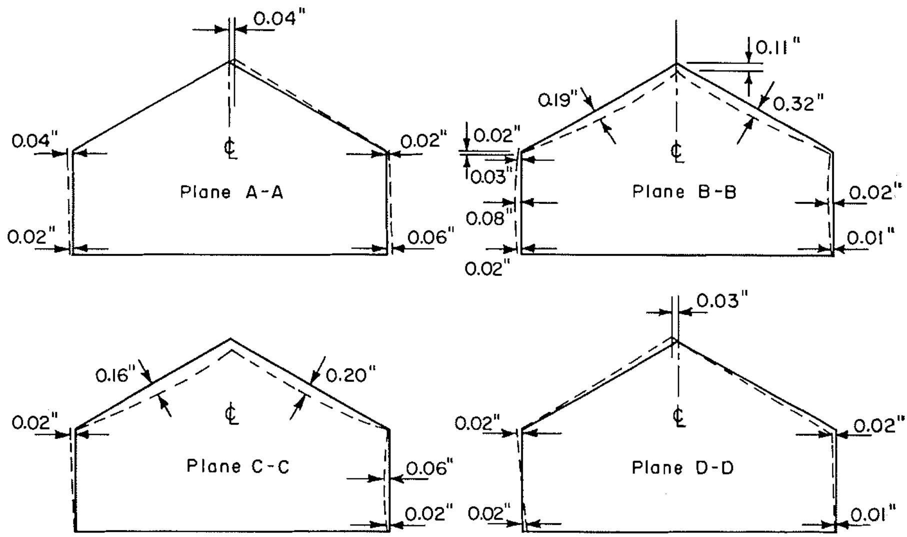
FIG. 11.—Deformations due to Wind Loads (Internal Pressure).

NOTES.—Refer to Figs. 5 and 7 for loading and deflection points. Deformations are not drawn to scale.

Wind Direction: From the right.



(*a*) Wind Velocity 90 mph.



(*b*) Wind Velocity 120 mph.

——— Undistorted Frame

— — — Distorted Frame

FIG. 12.—Deformations due to ½ Design Snow Load plus Wind Loads.

NOTES.—Refer to Figs. 5 and 7 for loading and deflection points Deformations are not drawn to scale.

Wind Direction: From the right.

At the end of 120 mph sustained loading, all pressures on the console were released in increments of one-third of the maximum pressures.

Wind Loads—Internal Pressure:

The next wind loading condition applied was that producing internal pressure (windows open on windward side only).

Loadings were again applied in increments of pressure corresponding to increases in velocity of 10 mph starting at a velocity of 70 mph and ending at the design wind velocity of 90 mph. At loads corresponding to a velocity of 80 mph, a new crack became visible in telltale No. 6. The 90-mph load was maintained for 1 hr, but no further cracks appeared in the plaster telltales. Then pressures were released in three steps to zero. The deformations resulting from this loading phase are shown in Fig. 11 (*a*).

The next phase was a continuation of this loading up to a wind velocity of 120 mph.

During the process of increasing and maintaining loads to 90, 100, and 110 mph, no further cracks appeared, but during the 1-hr period of sustained loading for a wind velocity of 120 mph, a crack which appeared in telltale No. 5 earlier in the test extended noticeably. At the end of the loading period, all loads were released to zero. The deformations resulting from this phase of loading are shown in Fig. 11 (*b*).

Up to this point in the test, the roof loads were in the form of an outward thrust at each of the loading points. Having completed the wind loading schedule, the next phase of loading in the direction of increasing severity of loading was that of a combination of wind and snow loading. Since the net result of such a combination of loading required a downward load on the roof, the 16 hydraulic rams acting on the roof surfaces were reversed. During this interlude, the plaster telltales were rephotographed to record the extent of cracking which had taken place under the applied wind loads.

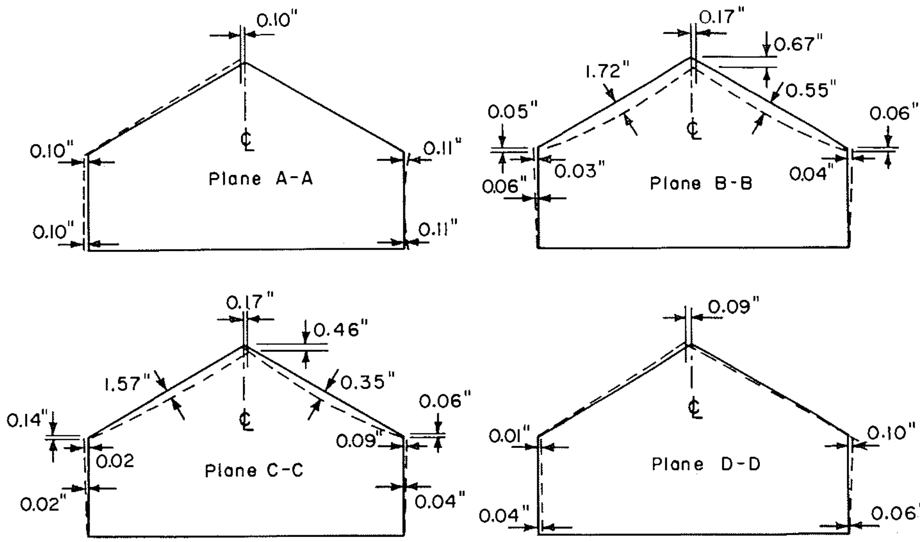
One-half Design Snow Load plus Wind Loads:

The combined loading of $\frac{1}{2}$ design snow plus wind load was applied for wind velocities varying from 70 to 120 mph. At 70 mph sustained loading, a small new crack appeared on the ceiling side of telltale No. 4. No further cracking took place in the plaster telltales as regular increases of 10-mph wind velocity were applied until an equivalent condition of a 110-mph wind velocity was reached, when a small new crack became visible in telltale No. 8. A final loading of $\frac{1}{2}$ design snow loading plus a wind velocity of 120 mph was reached without any further apparent cracking in the telltales. After being maintained for a certain period, all loads were reduced to zero in three stages as before. The deformations resulting from this phase of loading for wind velocities of 90 and 120 mph are shown in Figs. 12 (*a*) and (*b*).

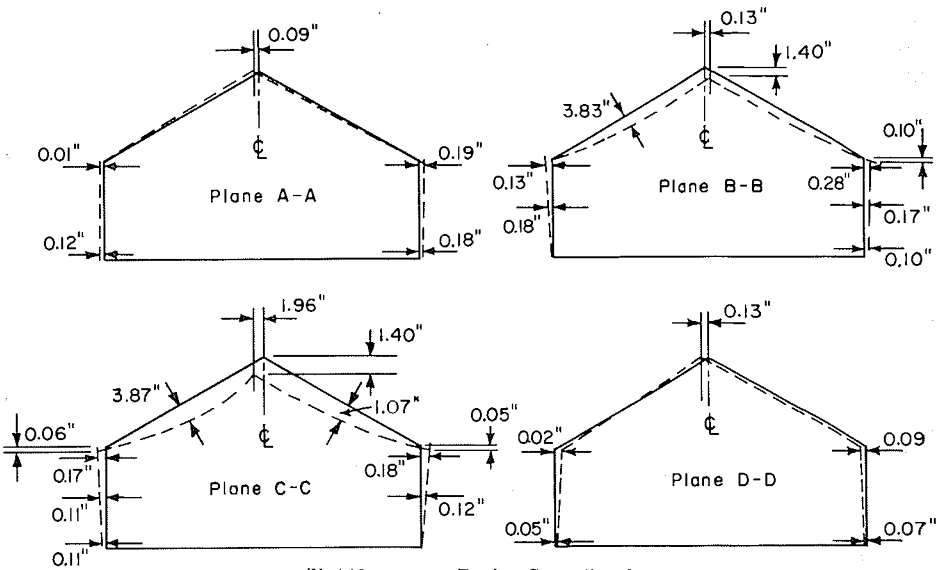
Snow Loads:

The next phase of loading consisted of the application of simulated snow loads which were applied in 25 per cent increments of the design snow load of 50.8 lb per sq ft of horizontally projected area for the Ottawa area as determined from the 1953 edition of the National Building Code.

As a first increment, the console pressures were raised in the two pressure channels actuating the hydraulic rams on the roof to a load equivalent to a 50 per cent design snow load. After a period of sustained loading, the crack which had appeared in telltale No. 9 earlier in the



(*a*) 100 per cent Design Snow Load.



(*b*) 143 per cent Design Snow Load.

——— Undistorted Frame

— — — Distorted Frame

FIG. 13.—Deformations due to Snow Loads.

NOTES.—Refer to Figs. 5 and 7 for loading and deflection points. Deformations are not drawn to scale.

Wind Direction: From the right.

test extended noticeably. The deformations were large in comparison to those of previous loadings. As the loading was increased from 50 to 75 per cent of the design snow load many new cracks occurred in the telltales, and sounds of cracking could be heard in the house. At telltale No. 9, the exterior wall separated from the solid plaster partition to the extent of $\frac{1}{8}$ in. near the ceiling. After $\frac{1}{2}$ hr of sustained loading, a further increase was made to 100 per cent design snow load. This loading was sustained for approximately 1 hr without any further serious damage. During the next increase from 100 to 125 per cent design snow load, further sounds of distress were heard, and the increase in the deformations shown on the deflection chart indicated that a failure was imminent. After $\frac{1}{2}$ hr had elapsed, the snow loading was again increased, but before 150 per cent snow loading was reached the loading had to be stopped because of the large deflections occurring (Figs. 13 (*a*) and (*b*)).

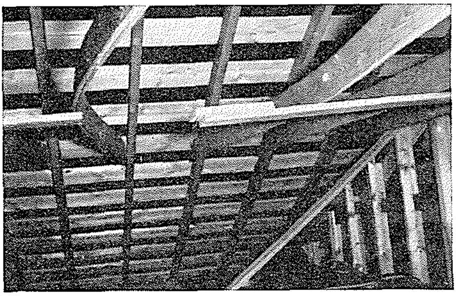


FIG. 14.—Failure in Splice of 1- by 4-in. Longitudinal Member and Buckling of Collar Ties in Attic under 143 per cent of Design Snow Load.

An inspection of the interior of the attic showed that all the collar ties were buckling horizontally to the extent of approximately 12 in. Under the maximum load reached (143 per cent of the design snow load), two collar ties snapped suddenly, one at the end of the house and one over the living room and kitchen. A splice in the longitudinal member attached to the underside of the collar ties had also given way. The break in the splice of the longitudinal member and the buckling of the collar ties is shown in Fig. 14 and the failure of the two end collar ties in Fig. 15. An additional failure occurred in one of the rafters at a knot. Except for a brief period of time during which failures were taking place in the roof assembly, it was possible to maintain stable console pressure for approximately 1 hr at 143 per cent design snow loading. During this period of sustained loading, careful observations were made of the interior of the test house and photographs were taken of the damage to the interior finishes and to the roof members. When this information had been gathered, all loads were gradually released and this phase of loading was completed.

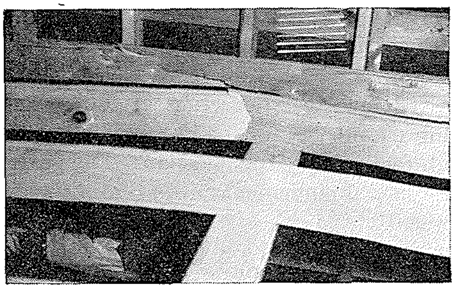


FIG. 15.—Failure of Two Collar Ties in the Attic at one End of Test House under 143 per cent of Design Snow Load.

DISCUSSION OF RESULTS

Wind Loads—Internal Suction:

No apparent damage resulted from wind loads (internal suction) equivalent to wind velocities up to 90 mph. The first crack in a telltale occurred when this loading to 90 mph was repeated. Further cracks became visible when the loads were increased up to 120 mph. This cracking was more pronounced at the junctions of the exterior wall and the solid plaster partitions than at other partitions.

The lateral movement under this wind loading was small (maximum 0.12 in.). The largest measured deflection in the rafters was 0.31 in. (outward at midspan).

Wind Loads—Internal Pressure:

During the application of the more severe conditions of wind loads with internal pressure some of the existing cracks in the telltales were extended slightly.

The greatest deflection in the rafters for a wind velocity of 120 mph was 0.80 in. (outward at the midspan). The greatest lateral movement in the exterior walls was 0.08 in.

One-half Design Snow Load plus Wind Loads:

Under the combined loading of wind and snow there was no further significant damage caused to the test house. This condition of loading produced a reversal of the direction of loading on the roof because of the downward snow loading. As the wind loading was increased, the rafter deflections decreased because of the uplift of the wind on the roof. Consequently the largest deflections in the roof under the combined loading were inward and occurred at the lowest wind velocity simulated in the test.

Snow Loads:

As the snow loads were applied in the final phase of the loading test, the deformations in the test house became more and more pronounced until failure began to occur in some parts of the roof at 143 per cent of the design snow load of approximately 50 lb per sq ft so that the load could not be increased any further.

At 100 per cent snow load the largest deflection in the rafters reached 1.72 in. on the front slope and 0.55 in. on the rear slope. The allowable deflection (National Building Code of Canada, 1953) for a rafter having a horizontally projected span of 12 ft 2 in., is $\frac{1}{240}$ of the horizontal projection of the rafter or 0.61 in., neglecting the support offered by the collar ties. Therefore, in the test, the allowable deflection was exceeded on the front slopes by 182 per cent at design snow loading for this particular roof slope. When failures occurred in the end collar ties and one rafter at 143 per cent of the design snow load, the rafter deflections had reached values of 3.87 in. on the front slope and 1.07 in. on the rear slope in plane C-C and there was a total of 1.96 in. of displacement in the ridge board towards the front slope on this plane of deflections.

The longitudinal 1 by 4-in. member which was fastened to the underside of the collar ties was not attached to the gable ends nor, because of the central location of the chimney, was it continuous over the full length of the house. Consequently as the snow load was increased, the collar ties were free to buckle in groups in each end of the house on opposite sides of the chimney, thus affording little support to the rafters after initial buckling had taken place. Figure 14 shows the failure of the splice in the longitudinal 1 by 4-in. runner and the extent of lateral buckling.

In the interior of the house, damage became progressively larger as the snow load was increased, particularly at the intersection of interior partitions and exterior walls, where the exterior walls moved outward at eave level in most cases. This pattern of deformation was produced largely by the spreading effect of the deflecting rafters and by ineffec-

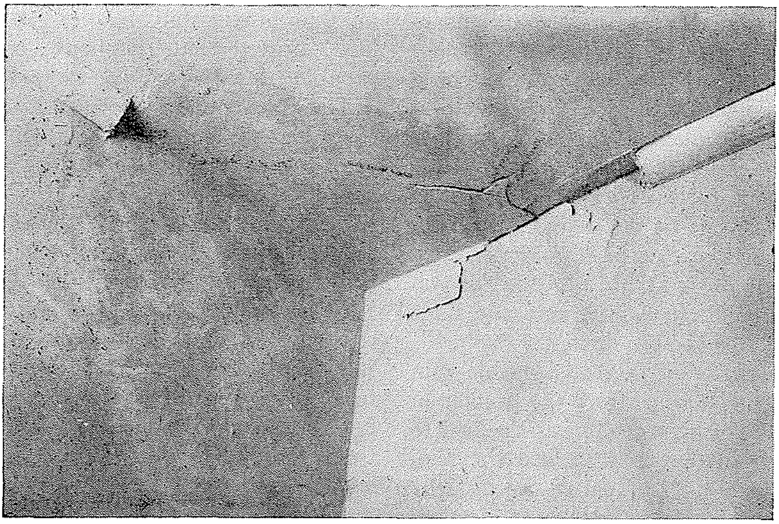


FIG. 16.—Plaster Telltale No. 4 After Completion of Test, Showing Extent of Cracking but no Signs of Separation Between Adjacent Walls.

tive resistance of the ceiling joist to this movement. Already at 75 per cent of design snow loading the exterior wall and the solid plaster partition separated for a considerable distance from the ceiling down. Under further loading this separation grew larger. Although there was extensive cracking in the plaster telltales at other points in the test house, there were no signs of apparent separation at the junctions of the conventional partition walls and exterior walls, as shown in Fig. 16. Further evidence of the spreading of the exterior walls is shown in Fig. 17 where the front wall moved outward between the conventional partition walls of the entrance and produced a fracture in the gypsum board ceiling.

Alterations to Roof to Increase Resistance to Snow Loads:

As the relatively early failure of the roof at 143 per cent of the design snow load (or 72 lb per sq ft) seemed to be mainly the result of the compression in the collar ties, it was decided to alter this part of the roof frame to find if in a simple manner the strength of the roof could be improved. As a first possibility in preventing the buckling of the collar

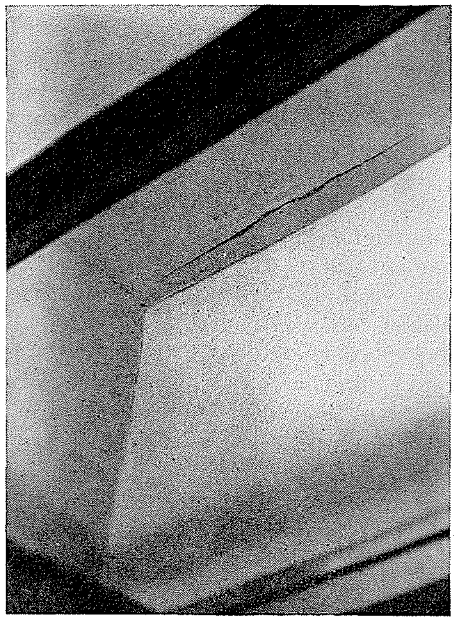


FIG. 17.—Damage Caused to Ceiling Above Entrance by Spreading of Rafters Under Snow Load.

ties the runner board of the collar ties was cross-braced in each half of the house on both sides of the chimney. The cross-bracing consisted of 1 by 6-in. members nailed to the underside of the collar ties. Then the simulated snow loading of the roof was repeated and a maximum load of 175 per cent of the design snow load was reached, an increase of 32 per cent. At this point, a failure occurred in two rafters and one collar tie, and the loads on the roof had to be released.

After reviewing the considerable gain in strength obtained from the bracing of the existing collar ties it was decided to go a step further and replace the relatively thin 1 by 4-in. collar ties by 2 by 4-in. members at every second rafter, thus using no more lumber than before. These new collar struts were also cross-braced by a 1 by 4-in. runner and 1 by 6-in. cross-braces. The snow loading on the roof was then repeated again and a load of 206 per cent of the design snow load reached. At this point several rafters broke, two of which had been spliced after the earlier failure. Under this load, considerable further cracking inside the house was observed and photographs of all plaster telltales were taken. The vertical crack between the solid plaster partition and the outside wall was approximately ½ in. wide, indicating the outward movement of the walls under the thrust developed by the rafters.

CONCLUSION

The results of the structural tests conducted by the Division of Building Research may be summarized as follows:

Resistance to Wind Loads (Racking Strength):

The application of simulated wind loads (wind direction towards front of house only) corresponding to velocities up to 90 mph (design speed) and 120 mph (approximately 80 per cent overload) proved that the house was well capable of withstanding these loads.

The movements of the end walls (parallel to the wind) due to wind loads were small. This was shown both by the deflection measurements and by the fact that of the four plaster telltales in the corners of the house only one cracked (No. 5). It was thus shown that the let-in 1 by 4-in. corner bracing, combined with the various finish materials, provided sufficient racking strength and that exterior wall sheathing was not required for additional racking resistance (if 1.8 times the design wind load is considered an acceptable test load).

Resistance to Snow Loads:

The application of simulated snow loads (symmetrical loading only, design load approximately 50 lb per sq ft) showed that the house in its original form could only carry 143 per cent of the design snow load and that, under 100 per cent design load, excessive deflections occurred in the roof structure. This indicated the following:

(1) *Rafters.*—The 2 by 4-in. rafters on 16-in. centers were structurally inadequate. It should be noted that these rafters do not meet the requirements of the National Building Code of Canada (1953) nor those of the present Building Standards of the Central Mortgage and Housing Corporation of Canada, although, for the latter, the 1 by 5-in. collar ties might be interpreted as intermediate supports and thus make the 2 by 4-in. rafters acceptable to those standards.

(2) *Collar Ties.*—The 1 by 5-in. collar ties were subjected to compressive loads (although their name implies a tension member). They were not able to resist these compressive loads effectively due to early buckling. The 1 by 4-in. longi-

tudinal runner attached to the underside of the collar ties was of little value in preventing them from buckling, because it was not continuous nor adequately braced against longitudinal movement.

After failure of the roof in its original form under 143 per cent of the design snow load the collar ties were altered to make them "collar struts" without using more lumber by replacing the 1 by 5-in. members fastened to every rafter pair by 2 by 4-in. members at every second rafter pair and cross-bracing. This increased the load-carrying capacity of the roof by 63 per cent to 206 per cent of the design snow load. It appears that in this form the collar struts can justifiably be considered as intermediate supports for the rafters.

(3) *Exterior Walls.*—Under snow loading separations began to occur at the junction of the rear exterior wall and the solid plaster partition walls under 75 per cent of design snow loading, indicating that the thrust of the rafters tended to deflect the exterior side walls outward at the ceiling level at points where insufficient resistance was afforded by the ceiling joists and the interior cross partitions. The connections between the rear exterior wall and the interior solid plaster partitions were structurally inadequate in preventing this separation.

Further Tests:

The evaluation of the strength and rigidity of house structures is difficult because of their complexity of form, variations in materials, workmanship, method of construction, and many other factors which make direct comparisons questionable. Before definite conclusions can be reached, more information is needed and a number of structural tests are necessary. The investigation carried out on this experimental house has been the first step by the Division in obtaining this information.

Future tests may be carried out in the laboratory on special full-scale test houses rather than in the field, because of the amount of time and cost involved in building up the test equipment around an existing house as was done in this first test.

Acknowledgments:

The authors wish to express their thanks to all those whose time and effort contributed to the successful completion of this project. To Mr. R. F. Legget, Director of the Division of Building Research, with whose approval this paper is published, acknowledgement is gratefully made for his encouragement and assistance in this project.

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DISCUSSION

MR. T. K. MAY.[1]—The framing members, of course, are the principal concern. Would the authors describe the framing members as being of high-grade, average-grade, low-grade or mixed grades of lumber?

MR. W. R. SCHRIEVER (*author*).—They were of average grade.

As I mentioned in the beginning, no special attempt was made to differ in any way in this house from the normal construction methods used in our area.

CHAIRMAN ROBERT F. LEGGET.[2]—May I ask what species?

MR. SCHRIEVER.—Pine.

MR. R. F. BARTELMES.[3]—The authors mentioned 120 mph resistance capability for the building, that is on the framing. What was the method of cleating the siding on to resist suction pulls?

MR. SCHRIEVER.—We did not investigate the resistance of the cleating to that suction. Originally we were trying to include a study of this factor by attaching the roof bearing pads on the aluminum by gluing. However, during the test we found that the glue which we had used was not able to resist the pull of the bearing pads and, to end our troubles of repairing the pads all the time, we finally drilled through the roof and used wire loops which went around the bearing pads and rafters.

MR. L. J. MARKWARDT.[4]—Was the effect studied on deformation of reversing the stresses by changing the direction of the loading?

MR. SCHRIEVER.—No. We applied it

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[4] Asst. Director, Forest Products Lab., Forest Service, U. S. Dept. of Agriculture, Madison, Wis.

only from one direction. The only reversal we did was from wind load to snow load.

MR. R. F. LUXFORD.[5]—Was the interior wall covering wet plaster construction or was it dry wall construction?

MR. SCHRIEVER.—It was dry wall construction.

MR. LUXFORD.—Did you use tape over the joints?

MR. SCHRIEVER.—Yes, I believe all joints were taped.

MR. IRA ASHFIELD.[6]—In reply to Mr. Luxford's question, gypsum wall board with standard cement joints and paper tape was used with the one exception that there is the plaster partition.

MR. LUXFORD.—I was speaking of exterior walls.

MR. ASHFIELD.—They were gypsum board on wood studs.

MR. JOHN F. LEWIS.[7]—I was concerned with the roof bracing system shown in Figs. 2 and 14. The use of collar ties is not particularly conventional to Southern California or to building codes and housing agencies in the States, and I wondered if that was a typical system used in Canada, or if it was tested in that way as an experimental basis of determining what might happen with just the collar ties. Conventional framing to me is made of struts and kickers and complicated bracing system that we see in most housing projects.

MR. ASHFIELD.—I do not accept the blame for the design, but I can report about it.

We selected this particular system because it was the lightest of three or four methods which were in reasonably common use, and we wanted to find out how it would perform. I think the authors have reported on that.

CHAIRMAN LEGGET.—As a result of this work, the Division of Building Research has embarked on a very extensive program of study of standard Canadian roof frame designs for house structures, as one of the several by-products of this work.

MR. J. E. DYKINS.[8]—I would like to ask Mr. Ashfield what nail spacing was used on the wallboard?

MR. ASHFIELD.—We followed the manufacturers' directions, and I believe the nails were 6 to 9 in. apart.

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[6] Supervisor, Building Materials Dept., Central Mortgage and Housing Corp., Ottawa, Ont., Canada.

[7] Structural Research Engr., Dept. of County Engineer, Building and Safety Div., County of Los Angeles, Los Angeles, Calif.

[8] Project Engineer, Environmental Division, U. S. Naval Research Establishment, Port Hueneme, Calif.