Effectiveness of knee-braced stud and pole frames in windbracing farm structures

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Massé, D.I. and Munroe, J.A. 1991. Effectiveness of knee-braced stud and pole frames in windbracing farm structures. Can. Agric. Eng. 33:351-354. A study was made to evaluate the long-term interactions between knee-braced pole or stud frames and conventional roofs currently recommended by the Canada Plan Service and thereby estimate the relative proportion of the lateral wind load resisted by either element. The flexibility coefficients of the frames and roof were based on full-scale tests and analog models. It was concluded that the knee-braced pole or stud frames, as assumed in design, do indeed provide for the major portion of the lateral wind resistance as compared to the conventional roof.

Cet article présente une évaluation de l'interaction à long terme entre des charpentes à poteaux ou à colombages contreventées avec des aisseliers et une toiture conventionnelle. Par conséquent les proportions de la charge due au vent résistées par la charpente contreventée et la toiture conventionnelle sont déterminées. Les coefficients de rigidité des charpentes et de la toiture conventionnelle ont été déterminés expérimentalement lors d'essais à grande échelle. Destimations ont clairement démontré que la charge induite par le vent est principalement résistée par la charpente à poteaux ou à colombage.

INTRODUCTION

In knee-braced pole or stud frame farm buildings, lateral wind resistance is provided by the pole or stud frames and the roof as well as the ceiling if installed. The relative proportion of the total resistance provided by each element depends upon the construction of the element and its location along the building length. For example, if a roof is very flexible but the knee-braced pole or stud frames are very stiff, then the frames will carry most of the lateral load particularly at midlength of the building. Conversely, if a roof is designed as a diaphragm and is well constructed, it can carry most of the lateral load, particularly in a short building. The proportion of lateral load carried by each of these elements could be determined by elastic analysis assuming compatibility at the frame-to-roof connection. That is, the lateral deflection must be the same for the frame and for the roof at any given point along the building.

The Canada Plan Service (CPS) was concerned about the long-term field performance of typical construction methods it was recommending. In particular, it was interested in trying to evaluate the proportion of lateral wind resistance provided by conventional roofs (roofs without specific diaphragm details included) as compared to knee-braced stud or pole frames. To date this appears to be the most common type of construction in the field. CPS also wanted to reflect on how the performance (strength and stiffness) of either element might be affected during the normal exposure conditions expected for agricultural structures. Such difference could be attributed to corrosion of roofing steel around roofing fasteners, or small relative movement between elements at joints causing a substantial increase in flexibility. These concerns developed following site visits where actual performance of structural elements was seen to be quite different from that assumed in design. A series of tests were carried out by Massé et al (1989) and as a result much improved connection details relating to knee-braced pole and stud frames were implemented in the CPS plans. The stiffness of frames built according to the revised plans was increased significantly (Massé et al 1989).

It remained, however, to determine how the stiffness of these frames now compared to the stiffness of roofs built according to conventional details currently shown in CPS plans, and typical of much of the construction in the field. A study was therefore initiated to evaluate the stiffness of knee-braced pole or stud frames as well as roofs and to determine the interaction of these components in providing lateral wind resistance in typical farm buildings.

OBJECTIVES

The objectives of the study were:

1. to determine the interaction of the frames and roofs, built according to CPS plans, in providing lateral wind resistance; and

2. to determine if the relative flexibilities of the frames and roof were such as to allow the frames to carry as much of the lateral load as is assumed in design.

METHODOLOGY OF PROCEDURES

Lateral flexibility of conventional roof construction

Thompson et al (1985) load-tested a 4.8 x 19.2 m long roof section and measured lateral in-plane deflections. This roof section was built using the conventional construction practice of purlins on the flat with metal roofing screwed to the purlins. No additional details such as blocking between trusses, special end splices in edge purlins, or additional stitch screws between purlins in metal roofing lap joints were added to improve the diaphragm action of the roof. It was felt that this type of roof panel represented what was commonly done in the field and shown in CPS plans, except where a roof was specifically designed as a diaphragm roof. The test set-up and resulting deflections are shown in Fig. 1. Further details relating to the test procedure can be found in Turnbull et al (1986).

The theoretical deflection of such a roof section could be
d_i = a [R - (i - 1)F] \quad \text{(1)}

where:
- \(d_i\) = deformation in panel \(i\) (mm),
- \(a\) = panel flexibility coefficient (mm/N),
- \(R\) = roof section end reaction (N),
- \(F\) = point load (N), and
- \(i\) = panel number.

On this basis, the total deflection \(\Delta_i\) at a given panel \(i\) is equal to the sum of the deformation of the individual panels from the end of the roof up to panel \(i\), or:

\[ \Delta_i = \sum_{j=1}^{i} d_j \quad \text{(2)} \]

Substituting Eq. 1 into Eq. 2, and noting that \((N - 1)F = 2R\),

\[ \Delta_i = \frac{ai}{2} (N - i)F \quad \text{(3)} \]

where:
- \(N\) = total number of panels.

The maximum deflection will occur at midspan where \(i\) equals \(N/2\). In this case:

\[ \Delta_{\text{max}} = \frac{aN^2F}{8} \quad \text{(4)} \]

Rearranging terms in Eq. 4 yields the panel flexibility coefficient as:

\[ a = \frac{8\Delta_{\text{max}}}{N^2F} \quad \text{(5)} \]
The lateral load on overdriven screws can be calculated using the following equations:

\[ Y = 0.6496 + 0.1588 \ln \Delta_s \]  
(6)

and for overdriven screws:

\[ Y = 0.4193 + 0.1033 \ln \Delta_s \]  
(7)

where:

- \( Y \) = screw lateral load (N), and
- \( \Delta_s \) = screw deformation (mm).

Assuming a screw deformation of 0.2 mm in order to be in the elastic range, and using Eqs. 6 and 7, the flexibility of the overdriven screws is 1.6 times larger than that for a standard driven screw. If it is assumed that under normal conditions during its expected life, the flexibility for a standard driven screw eventually attains a flexibility equal to that of an overdriven screw, for reasons mentioned previously, then it is realistic to assume that the long-term flexibility of a conventional roof should also be at least 1.6 times the short term flexibility. Thus the flexibility coefficient based on the short term noncyclic loading of Thompson et al (1985) could also be multiplied by 1.6 to give a more realistic long term value of 0.0016 mm/N. Assuming linearity, the long term flexibility coefficients of 2.4 and 3.6 m wide panels would be 0.0032 and 0.0048 mm/N respectively.

Based on field observations of several farm buildings, the writers feel that it is very conceivable that standard driven screws achieve at least this degree of flexibility over time.

Lateral flexibility of knee-braced pole and stud frames

Tests on full-scale knee-braced pole and stud frames carried out by Massé et al (1989) indicated good agreement between actual measured deformations and deformations predicted using an analog model. This same analog model was therefore considered adequate to determine the flexibility coefficients for the frames. The predicted flexibilities of the knee-braced stud and pole frames were 0.018 and 0.012 mm/N respectively. The truss-to-stud and truss-to-pole connection details for the frames tested are shown in Fig. 3 and are those currently recommended in CPS plans. Further information on the test procedure and construction details can be found in Massé et al (1989).

Interaction of knee-braced frames and conventional roof

Bryan (1973) has developed a method to determine the relative proportions of a lateral load carried by a particular frame or the coincident portion of a roof. He has computed these roof-frame interaction factors for buildings up to 12 frames in length, and for various flexibilities of roofs and frames.

Using the factors determined by Bryan as appropriate for the flexibilities of frames and conventional roof described in this study and typical of current CPS recommendations, the predicted relative proportion of wind load carried by the frames and roof has been calculated and is shown in Fig. 4. It is evident that these knee-braced frames are carrying most of the lateral load, particularly after the first few frames from the

Fig. 3. Truss-to-stud and truss-to-pole connection details of frames used in this study and currently recommended by CPS.
end of the building. In this analysis, the end walls were assumed to be rigid. Under actual conditions the end wall has some flexibility which in turn would cause the knee-braced frames to carry more of the load than indicated in Fig. 4. As well, the flexibility coefficients were established based on the results of tests by Thompson et al (1985) where the panel L:W ratio was 4. For L:W ratios greater than 4, the knee-braced stud or pole frames will carry a relatively greater portion of the load especially towards midlength of the building. In addition, for wide buildings where several roofing sheets are required to cover the width of the roof, the flexibility of the roof will quite possibly increase. This in turn again transfers a greater portion of lateral load to the knee-braced frames.

It should be noted that Massé et al (1989) also tested knee-braced frames that followed pole-to-truss and stud-to-truss connection details recommended by CPS prior to the development of those shown in Fig. 3. Although the earlier frames had adequate strength, their flexibility was very high; for example, the pole frame tested by Massé et al "could be moved laterally up to 100 mm by simply pushing it by hand". Nevertheless, this likely is indicative of the performance of much of the pole construction carried out in the field today. Obviously when such frames are used in conjunction with conventional roof construction, a greater proportion of the lateral load must be carried by the roof than indicated in Fig. 4, and of course greater deflections will be encountered. On this basis, the revised knee-braced frame designs as shown in Fig. 3 and now recommended by the CPS should help ensure that the frames carry the major part of the lateral load as intended by design.

**SUMMARY AND CONCLUSIONS**

A study was made to evaluate the long-term interactions between knee-braced pole or stud frames and conventional roofs (no specific diaphragm details included) and thereby estimate the relative proportion of the lateral wind load resisted by either element. The knee-braced frames and conventional roof considered were those currently recommended by the CPS. The flexibility coefficients of the frames and roof were based on full-scale tests of these components and analog models. Based on interaction factors established by others, it was concluded that, as assumed in design, the knee-braced pole or stud frames currently recommended by the CPS do indeed provide for the major portion of the lateral wind resistance as compared to the conventional roof. Only at the first interior frame did the roof carry more.

**REFERENCES**


