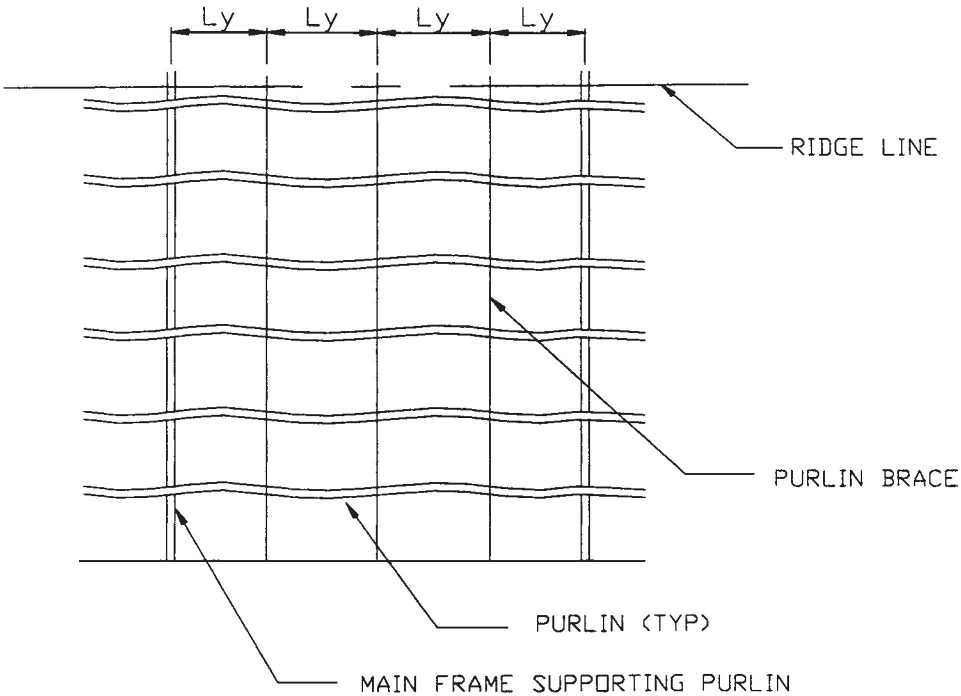

FIGURE 5.33 Purlin braces spaced at one-quarter points of the span. (*LGSI*).

Not all manufacturers agree that a close spacing of purlin braces is warranted. Some would-be low bidders might wish not to provide any bracing at all, or to provide it solely for the purpose of purlin alignment, but prudent suppliers usually understand why purlins must be braced at close intervals. Purlin bracing should not be an area where corners are cut. In the author's opinion, closely spaced purlin bracing represents one of the best investments possible in the quality of metal building systems.

5.5 PURLIN BRACING: ADDITIONAL TOPICS

5.5.1 Bracing for Uplift Conditions

Whenever purlins are stabilized by roofing or top-flange bracing, they are considered laterally supported only for *downward* loads, which produce mostly compressive stresses in the purlin's top flange. (Owing to continuity effects, some areas of the top flange located near supports will be in tension.) But what about the situations when wind produces upward forces and the *bottom* flange acts mostly in compression?

According to one model,[15] the maximum compressive stress during uplift occurs at the intersection of the purlin's bottom flange and its web. There, the purlin is unbraced. As Tondelli[16] demonstrates, the bottom flange can also be in compression under downward loading in short interior spans of uniformly loaded continuous purlin runs. A similar situation can occur under partial roof loading (see Fig. 5.12).

Behavior of roof purlins braced only on the tension side is extremely complex and poorly understood. The AISI Specification has undergone many changes in this regard; the latest major modification appeared in the 1989 Addendum. The design approach of the 1989 Addendum described here was retained in the 1996 AISI Specification[1] and was further fine-tuned in the 2002 North American Specification for the Design of Cold-Formed Steel Structural Members.[4]

In order to calculate the nominal moment strength M_n of the member with one flange attached to through-fastened roofing, a "reduction factor" method is presented in Equation C3.1.3-1:

$$M_n = RS_eF_y$$

where M_n = nominal moment strength
S_e = *effective* elastic section modulus computed with the compression flange assumed to be stressed to yielding
F_y = design yield stress
R = reduction factor.

In both the 1989 Addendum and the 1996 Specification, the R factor was given as follows:

0.4 for simple-span channels

0.5 for simple-span Z sections

0.6 for continuous-span channels

0.7 for continuous-span Z sections

The 2002 North American Specification maintained the same R values for continuous C and Z sections, but introduced more complicated provisions for simple-span members. The R values of 0.4 for C and 0.5 for Z simple-span sections were retained for structural members whose depth was between 8.5 and 11.5 in, but were increased for members of smaller depth. For C and Z sections more than 6.5 in but not more than 8.5 in deep, the R value was set at 0.65; for sections not more than 6.5 in deep, R was increased to 0.7. In a new provision, the 2002 North American Specification stipulated that R values for simple-span C and Z members were to be reduced by the effects of compressed insulation between the sheeting and the members. The rate of reduction was 1% per 1 in of insulation thickness (uncompressed). No such reduction for insulation was specified for continuous C and Z members.

The M_n in this equation is the ultimate moment strength of the section, not the moment caused by the maximum allowable service load, and the R factors are not supposed to be automatically transferred to the allowable load values. Nevertheless the R factors are widely used to convert the allowable uniform-load values for fully braced conditions found in the manufacturer's tables into the design values for purlins braced only on one side by simply dividing the "fully braced" tabulated values by 1.67, the factor of safety for flexure.

The "reduction factor" method of analysis is based on an extensive testing program[17] performed within some very specific limits. It does not apply to situations where *any* of the conditions listed in the Specification are not met. For those nonconforming cases, the user is directed to apply "rational analysis procedure" or to perform a load test in accordance with the specification provisions. The values of R factors could be changed in the upcoming Specification editions, and the engineer who gets involved in these issues should obtain a copy of the latest Specification.

Analyzing Eq. C3.1.3-1 and the R factors, one can conclude that the ultimate load capacity of a roof framed with continuous Z purlins under wind uplift loading can be taken as about 70% of its capacity for resisting downward loads. It follows that in theory the roof should be able to support a net wind uplift load equal to about 70% of the design snow or live load without any additional bracing required for the compression (bottom) flange.

For many areas of the United States, the design wind uplift loading is indeed less than 70% of the design snow or live load. Does this mean that bracing for the bottom flange of purlins is not needed when the top-flange sag angles or straps parallel to the roof are used? Certainly not: bottom-flange bracing is still required to ensure purlin stability and to prevent its rotation under load—whichever loading governs the purlin design.

5.5.2 Purlin Bracing Provided by Standing-Seam Roofing

In contrast with through-fastened roofing attached directly to purlins, standing-seam metal roofing (or, more precisely, standing-seam roofing with concealed clips) is intended to expand and contract with temperature changes and to move relative to the supporting structure. Freedom to move without