cold-formed steel framing is rarely designed in most structural engineering offices. When such framing is needed, one of two things tends to happen to the engineers: They either uncritically rely on the suppliers' literature or simply avoid any cold-formed design at all by specifying hot-rolled steel members and hoping for a contractor to make the substitution and to submit the required calculations.

In this chapter, we limit our immersion into the actual Specification formulas that could easily have become obsolete by the time you read this book. Instead, we point out but a few salient concepts.

What makes cold-formed steel design so time-consuming? First, materials suitable for cold forming are usually quite thin and thus susceptible to local deformations under load. (Remember how easy it is to dent a tin can?) This mode of failure is of much less concern in the design of thicker hot-rolled members. These local deformations can take two forms: local and distortional buckling. The nature of distortional buckling (Fig. 5.2*a*) is not very well understood, at least not as well as that of local buckling (Fig. 5.2*b*). In local buckling, some part of the compression flange and the web buckles when the stresses reach a certain limit; that part then ceases to carry its share of the load. In distortional buckling, the compression flange and the adjacent stiffening lip move away from the original position as a unit, also weakening the section. Research on distortional buckling proceeds at a brisk pace, with some important work done by Bambach et al.[5] and Schafer and Pecoz,[6] among others. Second, the flanges of light-gage sections cannot be assumed to be under a uniform stress distribution, as the flanges of an I beam might be (the shear lag phenomenon). To account for both the local buckling and the shear lag, the Specification utilizes a concept of "effective design width," in which only certain parts of the section are considered effective in resisting compressive stresses (Fig. 5.3). This concept is pivotal for stress analysis and deflection calculations performed for cold-formed members.

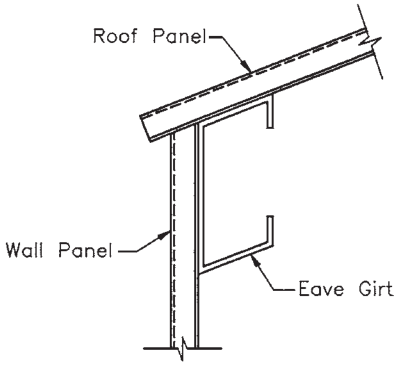

FIGURE 5.1 Typical eave strut.

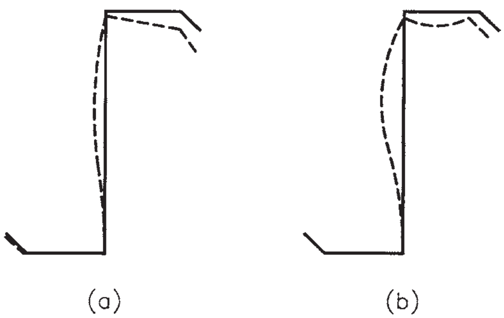

FIGURE 5.2 Local deformations of cold-formed Z sections in flexure, with top flange in compression: (*a*) distortional buckling; (*b*) local buckling. (*After Refs. 5 and 6.*)

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SECONDARY FRAMING: GIRTS AND PURLINS

The effective design width depends on the stress in the member, which, naturally, cannot be computed until some section properties are assumed first. Because of this "vicious circle," a few design iterations are needed. A common simplified yet conservative procedure for the effective width calculations assumes the level of stress to be the maximum allowable.

Another complication caused by a nonuniform stress distribution across thin, often nonsymmetrical, sections is their lack of torsional stability. Light-gage compression and flexural members can fail in torsional-flexural buckling mode by simultaneous twisting and bending, a failure that can occur at relatively low levels of stress. In plan, purlins that buckle laterally are displaced from their original positions as shown in Fig. 5.4. The maximum lateral displacement typically occurs in the middle of

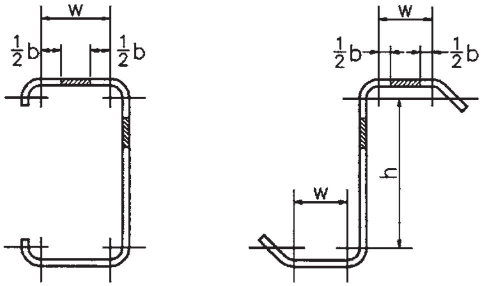

FIGURE 5.3 Effective width concept for C and Z sections (shaded areas are considered ineffective).

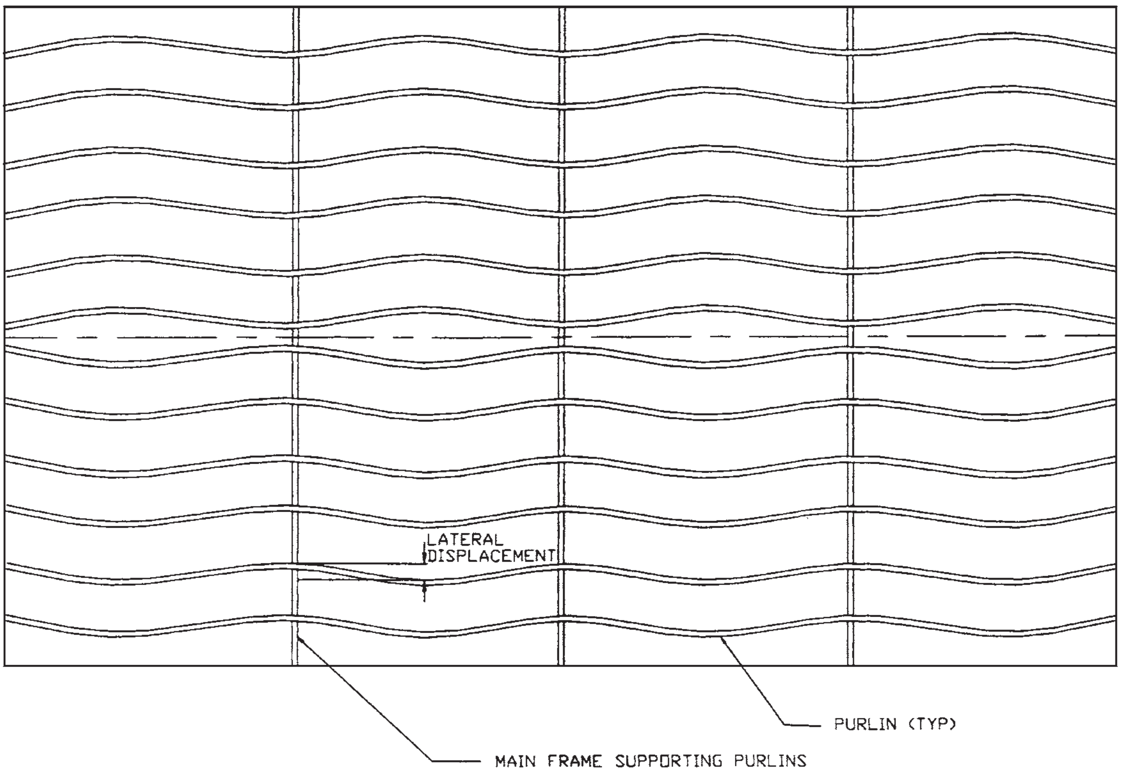

FIGURE 5.4 Purlin movement from lateral buckling. (*LGSI.*)

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