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Shear Verification of Plain and Lipped Channel Steel Sections

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Abstract. With the continuous growth of industrialization in the construction sector in recent years, steel structures have become increasingly common and economically viable. Along with this trend, the use of cold-formed steel sections has also expanded. Made from thin steel sheets, these sections offer advantages such as low weight, ease of fabrication, handling, and transportation, thereby reducing assembly and shipping costs. In steel roofing structures, cold-formed sections—particularly plain and lipped channel sections—are widely used in trusses and purlins. When used as purlins, these members are subjected to oblique bending, with loads applied outside the shear center of the cross-section—a condition not accounted for in the current Brazilian steel design standards. Given this context, the present study analyzes the maximum shear stresses in the webs of plain and lipped channel sections subjected to the boundary conditions and loading typically experienced by roof purlins. The analysis is carried out using an analytical method based on solid mechanics, the approach recommended by Brazilian steel design codes, and the finite element method (FEM), in order to validate the analytical results.

Keywords: Cold-formed steel, Shear stress, Finite element analysis, Channel sections.

1 Introduction

The industrialization of the Brazilian construction industry has brought significant changes to the way structures are designed and executed. The replacement of traditional manual processes with faster and more efficient solutions is an increasing demand, mainly driven by the pursuit of productivity and cost reduction. In this context, steel structures have emerged as an increasingly attractive alternative, standing out for their rapid assembly, reduced weight, and material reusability. Among the elements used in metal roofing systems, purlins composed of plain and lipped channel sections are noteworthy, particularly due to their favorable strength-to-weight ratio.

However, the proper design of these purlins requires considerations that go beyond traditional approaches. This is because, due to the geometry of the sections and the position of the load relative to the shear center (SC), these elements are subjected to combined bending and torsional effects. Brazilian standards, particularly ABNT NBR 8800 [1] and ABNT NBR 14762 [2], establish criteria for verifying shear stresses in sections subjected to transverse loads. However, these standards assume that the loading acts on the shear center, which in most practical situations does not occur.

This study proposes a more in-depth investigation into the shear behavior of plain and lipped channel sections used as purlins. Through analyses based on three distinct methods, the aim is to understand the effects of flexural-torsional interaction on the cross sections of these members.

2 Methodology

In order to evaluate the structural behavior of plain and lipped channel sections subjected to shear forces under conditions representative of their use as metal purlins, different analytical approaches were adopted. This chapter describes the methodology employed to conduct the study, encompassing the definition of boundary and

loading conditions, as well as the procedures adopted in the three analysis methods used: analytical, normative, and numerical using the Finite Element Method (FEM). The choice of multiple approaches aims to ensure the reliability of the results and enable a critical comparison between the solutions prescribed by standards and those obtained through more rigorous models.

2.1 Considerations on Purlins

Purlins are structural elements of roofing systems whose function is to support the roof panels and transfer their loads to the primary structural members, such as trusses or frames. In steel systems, purlins are generally manufactured from cold-formed sections, with plain and lipped channel sections being the most commonly used.

Since all types of roofs require some inclination to ensure proper water drainage, purlins are inevitably subjected to oblique bending, as they follow the roof slope. In practice, the method commonly used to minimize the effects of bending about the minor axis consists of the installation of a bracing system, which is essentially a set of rigid and flexible braces designed to reduce the unbraced length of the member about its minor axis, as illustrated in Figure 1.

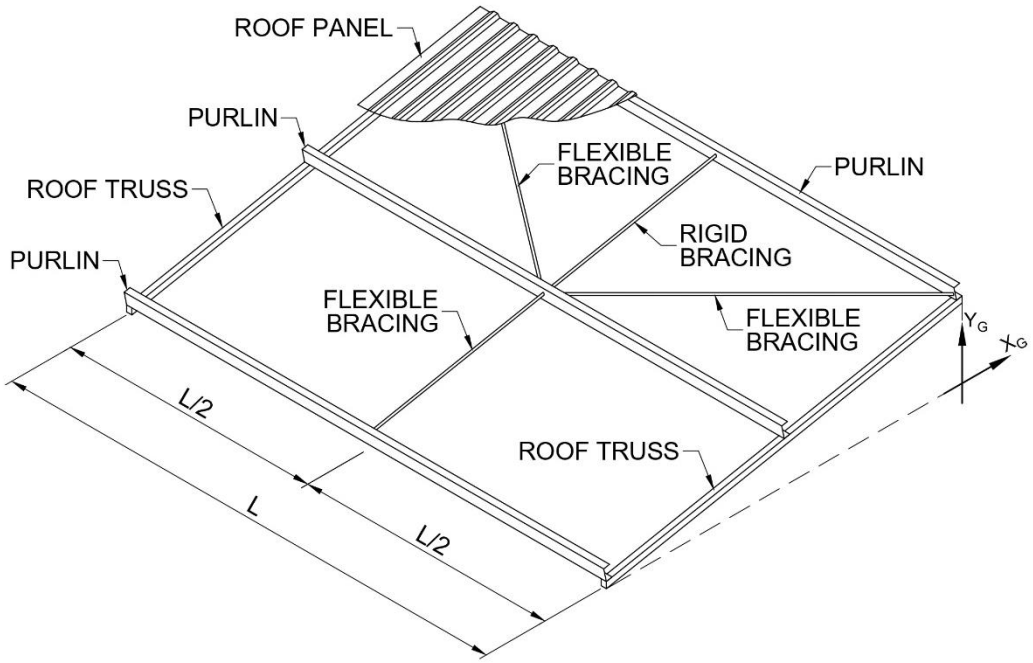


Figure 1. Commonly used steel structural model for roofing systems.

According to Chamberlain et al. [3], these braces are typically equal-angle sections or round bars that should be distributed along the purlins so as to define unbraced lengths about the minor axis between 2,50 m and 3,00 m. Bellei [4] also recommends installing one brace to the purlins with spans up to 6,00 m, and two for longer spans.

The normal loading on a purlin generally consists of the self-weight of the member and the actions arising from the roof panels, i.e., their own self-weight and that of their fasteners, the roof live load, and dynamic wind pressures, which can be either pressure or suction.

Gravitational loads originating from the roof panels are applied on the top flange of the purlin in a direction perpendicular to the horizontal plane, whereas dynamic wind pressures are also applied on the top flange but perpendicular to the plane of the roof. The self-weight of the purlin is also a gravitational action; however, its point

of application corresponds to the centroid of the purlin's cross section.

2.2 Considerations on the study

The span length L adopted, as shown in Figure 1, was 6,00 m, which is a common dimension that optimizes the member by avoiding profile cutting, thus resulting in an unbraced length in the roof plane of 3,00 m, in accordance with the recommendations of Chamberlain et al. [3] and Bellei [4].

Regarding the actions, the following values were considered:

- Self-weight of the purlin: applied according to the characteristics of each section analyzed;
- Self-weight of the roof panels: estimated at 0,10 kN/m², based on Gerdau [5] and Irmalex [6] catalogs;
- Roof live load: adopted as 0,25 kN/m², according to ABNT NBR 8800 [1];
- Wind action: estimated for a gable roof as 0,10 kN/m² of positive pressure, representing the most unfavorable condition for the purlin under analysis.

The spacing between purlins used to determine the line load along each member was 1,60 m, adopted based on the catalogs of metal roofing manufacturers Gerdau [5] and Irmalex [6]. The roof slopes considered were 10%, 15%, and 20%, which are typical values for metal roofing and recommended by roofing manufacturers.

2.3 Analytical Method

The analytical method adopted in this study is based on the principles of the mechanics of deformable solid bodies, with emphasis on the analysis of shear in thin-walled open sections, as described by Féodosiev [7]. Considering that channel sections, when used as purlins, are subjected to transverse forces applied outside the shear center, it is assumed that, in addition to simple bending, torsion of the section also occurs. The approach considers the decomposition of the shear force along the principal axes of the section and applies Juravski's formula (Equation 1) to calculate the distribution of shear stresses along the midline of the cross section.

$$\tau_1 = \frac{1}{t}\left(\frac{V_y S_x^*}{I_x} + \frac{V_x S_y^*}{I_y}\right) \tag{1}$$

The procedure used to calculate the shear stress associated with torsional moments caused by shear forces whose line of action does not pass through the shear center of the section was based on the principle of superposition of effects, by summing the products of the components of the shear force corresponding to each load and the distance from the shear center to its line of action, as shown in Figure 2 and expressed in Equation 2.

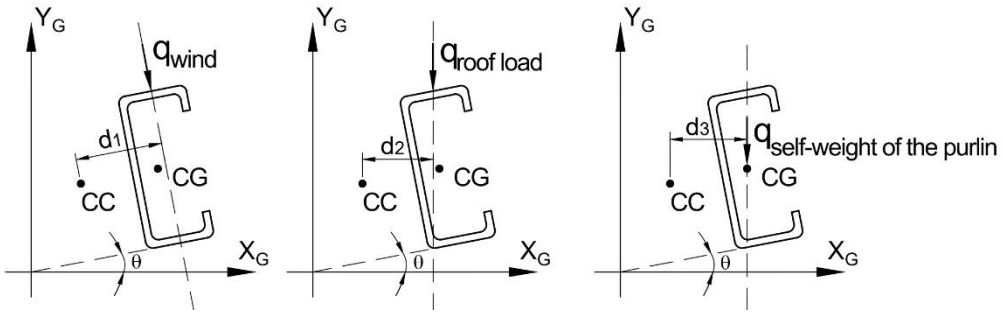


Figure 2. Lever arm length between the shear center and the line of action of the load.

$$\tau_2 = \frac{d_1 V_{q_{wind}} + d_2 V_{q_{roof\ load}} + d_3 V_{q_{self-weight\ of\ the\ purlin}}}{I_T/t} \tag{2}$$

2.4 Normative method

The ABNT NBR 8800 [1] standard, in item 5.4, provides the expressions for determining the design strength of prismatic members subjected to bending moment and shear force, but with some restrictions, such as in item 5.4.1.2, which states: "The transverse load shall always lie in a plane of symmetry, except in the case of channel sections bent about the axis perpendicular to the web, when the resultant of the transverse load shall pass through the shear center of the cross section or torsion shall be prevented." The ABNT NBR 14762 [2] standard, in item 9.8, details the conditions for designing members subjected to simple bending only with expressions derived for loads applied at the shear center of the section.

In other words, both standards that govern the design of steel structures in Brazil omit, in principle, the condition of members subjected to simple bending with loads applied outside the shear center and, in a simplified way, assume a constant shear stress acting only on the web of the section.

In both standards, shear resistance is verified by comparing the applied shear stress with the design shear strength, which depends on the slenderness of the web.

2.5 Numerical method

In order to verify the reliability of the results obtained by the solid mechanics method, a computational model was developed to determine the maximum shear stresses in some of the studied profiles, using the Finite Element Method (FEM) with the Ansys 14.0 software.

For the computational models developed in this study, among the numerous element types available in the Ansys library, the profiles were discretized using rectangular shell elements, specifically SHELL 181, which is suitable for thin and medium-thickness shells and supports nonlinear physical and geometric analysis of second order with large deformations. For the steel, the following properties were adopted: Elastic modulus E = 200 GPa; Poisson's ratio ν = 0,3; and yield strength f_y = 230 MPa.

To avoid the formation of stress concentration points in the region of maximum shear forces, the following boundary conditions were applied to the computational model, as shown in Figure 3:

a) Displacements restrained in the z direction (longitudinal axis of the profile) at the midsection elements;
b) Displacements restrained in the y direction (web direction) at the web elements at the profile ends;
c) Displacements restrained in the x direction (flange direction) at the flange elements at the profile ends and at the mid-height of the web at the midsection length, simulating the bracing system.

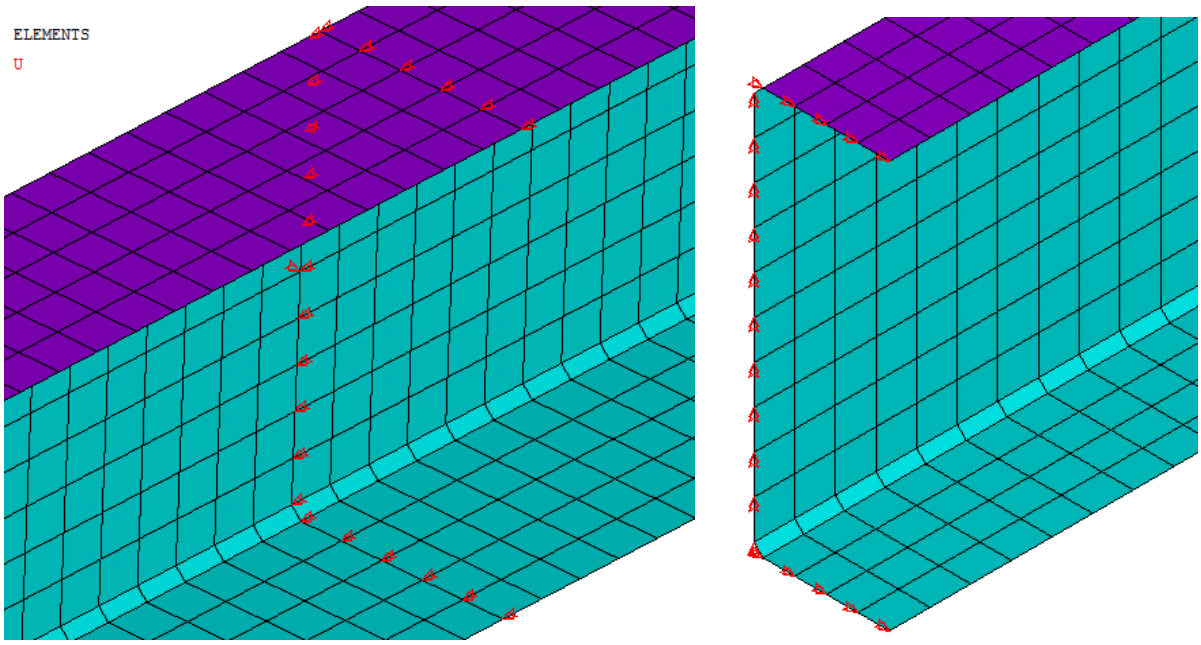


Figure 3. Boundary conditions applied at the mid-length section of the profiles (right side) and at their ends (left side).

The finite element size was selected based on a stability analysis of the maximum shear stress results in the web of the profile using different mesh sizes. It was observed that square finite elements with a side length of 2,0 cm provided good convergence of the results for the present study.

With the boundary conditions and finite element size defined, the analysis was conducted for the plain channel section series 300x100 mm, 250x100 mm, and the stiffened channel section series 300x100x25 mm, 250x100x25 mm. The results obtained by the FEM were then compared with those from the solid mechanics method. This comparison is shown in the graphs of Figures 4 and 5.

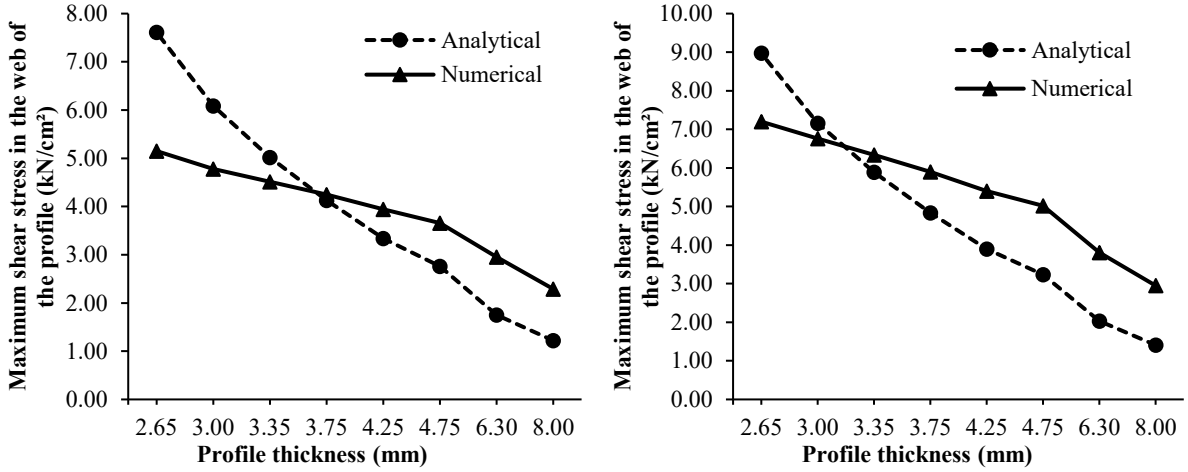


Figure 4. Results obtained for the plain channel sections of the 300x100 mm and 250x100 mm series, respectively.

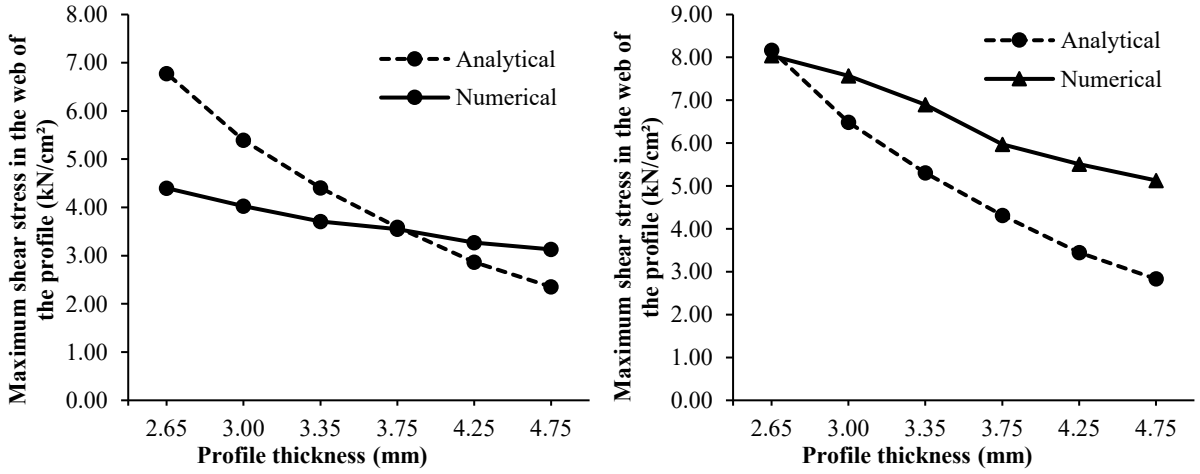


Figure 5. Results obtained for the stiffened channel sections of the 300x100x25 mm and 250x100x25 mm series, respectively.

The deformation of the profile, shown in Figures 6, indicates that, as expected from the theory of deformable solid bodies, the profile exhibited clear torsional deformation characteristics, with a pronounced tendency for the cross section to rotate around the shear center of the section. This rotational behavior results from the eccentricity of the applied load relative to the shear center, which induces additional torsional moments that combine with the primary bending effects. As a consequence, the stress distribution across the web and flanges becomes non-uniform, with stress concentrations appearing in specific regions. The maximum shear stresses were observed in the mid-height region of the web at the support section, where the combination of bending and torsion effects is most significant, as illustrated in Figure 7.

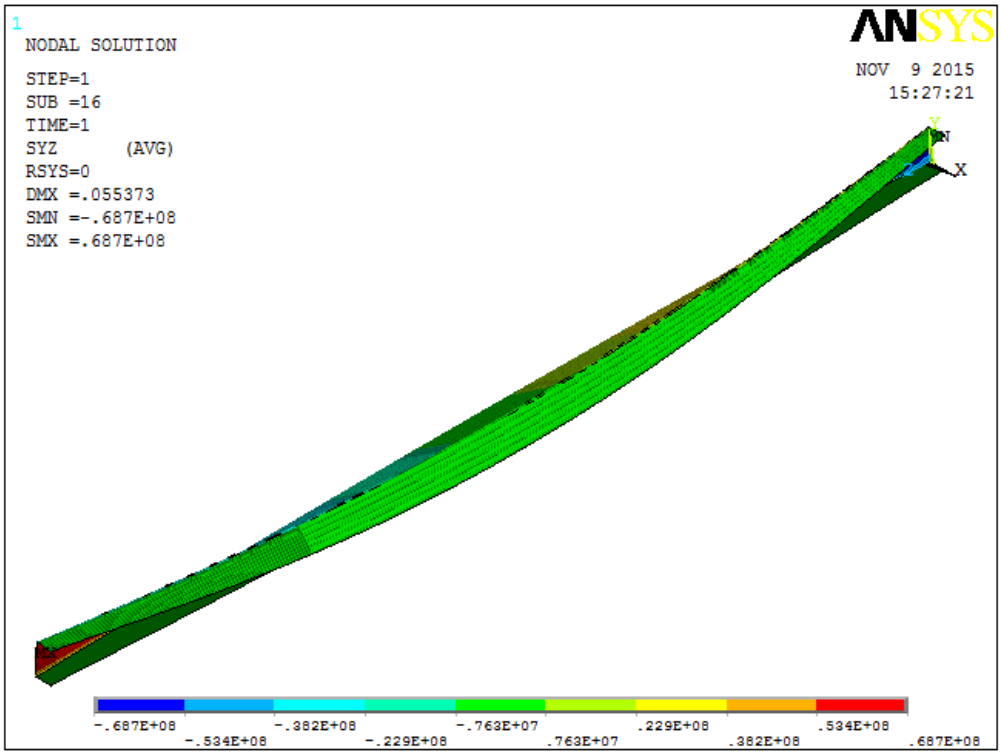


Figure 6. Shear stress distribution and deformation pattern of the profile under combined bending and torsion. The finite-element analysis confirms a rotational tendency of the cross-section around the shear center.

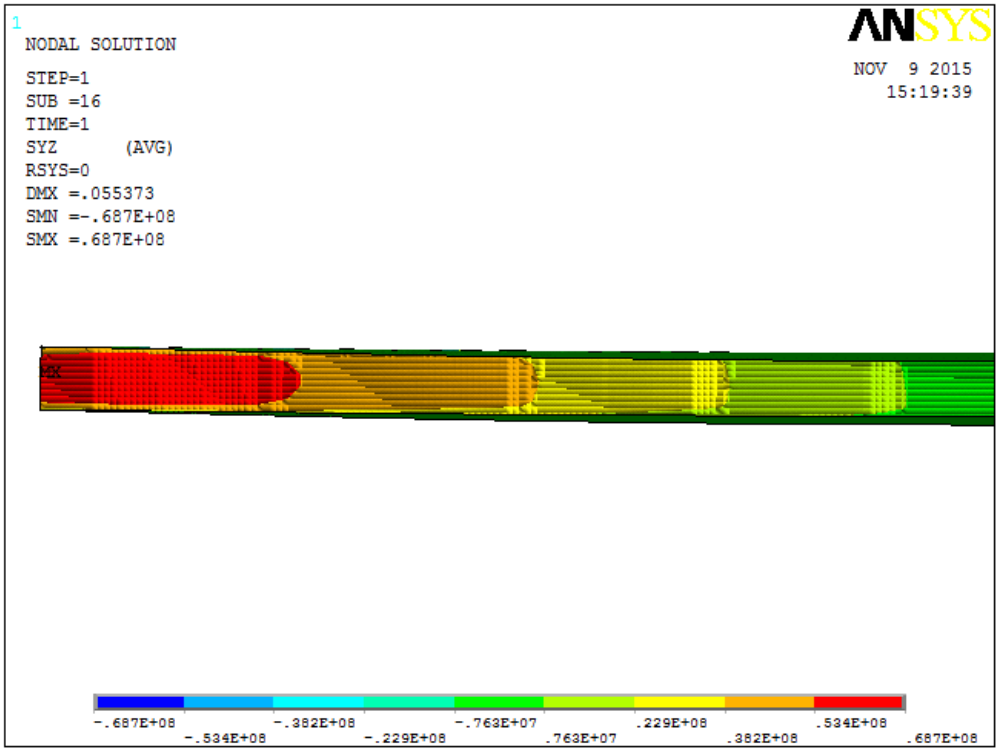


Figure 7. Shear stress distribution along the web of the profile. The maximum shear stresses occur at the mid-height region, where the interaction between bending and torsion is most significant.

3 Conclusion

The results of this study demonstrate that torsional effects arising from loads applied outside the shear center significantly influence the shear capacity of cold-formed channel sections. The comparative analyses show that the analytical approach based on solid mechanics provides results consistent with the numerical finite element simulations.

From a design standpoint, these findings indicate that the simplified shear verification procedures prescribed by Brazilian standards (ABNT NBR 8800 and NBR 14762) may underestimate local effects in purlins subjected to eccentric loading. A more refined methodology that explicitly considers the load eccentricity and its torsional consequences would lead to more accurate and safer design practices.

Although the present work is limited to numerical and analytical analyses, the close agreement between both approaches supports the validity of the adopted methodology. Future studies should include experimental validation and the investigation of a broader range of section geometries to develop practical design recommendations for cold-formed steel purlins under combined bending and torsion.

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