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Second-order effects in locally and/or distortionally buckled frames and design based on beam element analysis

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ABSTRACT

In the design of steel portal frames, second order effects are usually accounted for by using a second order elastic analysis to calculate the deformations and internal actions of the frame. In this analysis, one-dimensional beam elements are employed irrespective of whether the cross-sections of members are slender or non-slender. However, frames composed of members with slender cross-sections may buckle in local and/or distortional modes before reaching the ultimate limit state. In this case, the flexural rigidity of the frame is reduced when local/distortional buckling occurs, and as a result it undergoes larger sway deflections than had local/distortional buckling not occurred. The additional second order moments thus caused by local/distortional bucking are not accounted for in current design provisions.

In this paper, the additional second-order effects caused by the development of local/distortional buckling are studied by comparing numerically determined ultimate capacities with design capacities. The numerical ultimate strengths are based on previously calibrated geometric and material nonlinear shell finite element models, and the design capacities are determined using the Australian Standard for Cold-formed Steel Structures AS/NZS 4600. A simplified approach to account for local/distortional buckling in beam-element-based design is also proposed. The focus is on portal frames subject to in-plane sway failure.

1. Introduction

Determining ultimate strength is arguably one of the most important objectives of structural design. The structure and its component members and connections are designed for the strength limit state so that the capacities of all members and connections are greater than the internal design actions. In this current design approach for steel structural frames, the design actions of members are obtained from a first-order or second-order elastic beam element analysis, whereas the member capacities are obtained using design equations derived from tests and/or geometrically nonlinear inelastic analyses.

In practice, most of the structural analyses used by consulting engineering firms for designing steel structures are premised on beam elements and as such implicitly preclude local/distortional buckling. Similarly, the provisions of national standards for the design of cold-formed steel structures [1,2] implicitly assume that the structural analysis employs beam elements.

This limitation may result in inaccurate load actions for frames composed of thin-walled sections that undergo local/distortional buckling prior to the frames failing. The presence of local/distortional buckling reduces the rigidity of the section and thus amplifies second-order effects, causing a redistribution of the internal forces. Unless accounted for, local and/or distortional buckling may lead to inadequate designs of structures prone to these modes.

The use of beam elements in the structural analysis is imposed partly because modelling local instability requires complex discretisation of the cross-section (using shell elements), and partly because, for many hot-rolled and fabricated products, the cross-section can reach its ultimate capacity without developing local/distortional buckling deformations. However, as light gauge cold-formed steel structures are being used more widely and increasingly slender sections are becoming available, a need has emerged to validate the accuracy of current design methods in predicting the ultimate strength of thin-walled steel structures. If current methods are proven inadequate, there is a need for developing guidelines for a method of design which considers the effect of cross-sectional instability in determining internal actions.

Shell element analyses have proven capable of capturing cross-sectional deformations and accurately predicting load carrying capacities. In particular, shell element models of pitched roof single bay and single storey portal frames with semi-rigid apex and eave joints have been validated [3] by comparing FE strengths and load-deflection curves with experimental results. Similar models to those presented in [3] are used in this paper for the numerical analyses. The ultimate loads of frames with a variety of cross-sections are obtained using shell element analyses, and subsequently compared to ultimate loads

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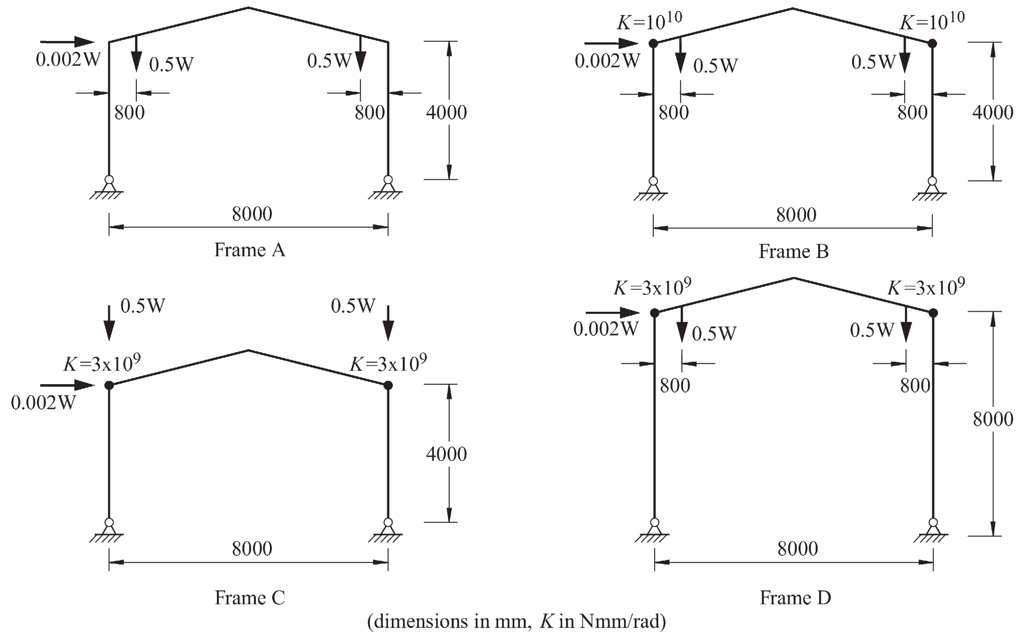

Fig. 1. Portal frame models; unit of joint stiffness K is Nmm/rad.

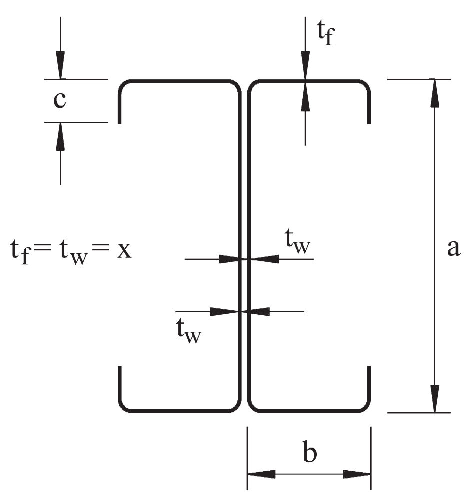

Fig. 2. Dimensions of cross-sections.

obtained using beam element analyses. They are also compared with design capacities based on the Australian standard for the design of cold-formed steel structures AS/NZS4600.

In the past decade, the analysis and design of cold-formed steel portal frames have attracted the attention of many researchers [4–8]. However, these researchers have concentrated mainly on the design optimization and joint topography of the portal frame. Two-dimensional portal frames with different geometry and load cases are studied in this paper. The rafters of each frame have relatively stocky sections to ensure that local/distortional buckling will not develop in these members. The column sections are gradually reduced in thickness to increase the section slenderness. The sections chosen range from non-slender to slender, and the effects of local/distortional buckling on the structural behaviour are studied for these sections.

In the second part of this paper, a simplified method is proposed to account for the development of local/distortional buckling in beam-element-based analysis. In this method, a simple factor τ_g which encapsulates the reduction in stiffness caused by local/distortional buckling is incorporated into the analysis. The τ_g-factor is assumed to be a function of the non-dimensional compressive force and the ratio between bending moment and axial force. In this paper, only the reduction of major axis bending rigidity (EI_z) was considered, while other rigidities were assumed to be unchanged. The method is verified by comparing results thus obtained with accurate shell element results. Three different sections are considered in the verification analyses. The study is confined to two-dimensional frames failing in the plane of bending.

2. Second-order effects in locally/distortionally buckled frames

2.1. Frame description

Current standards for the design of cold-formed steel structures [1,2] employ a member-based design check and use computer analyses to obtain design actions. The provisions implicitly assume that the structural analysis uses beam elements. As explained in the Introduction, cross-sectional buckling deformations reduce the rigidity of the section, and hence amplify deflections and cause a distribution of the internal forces in the structural frame. These effects are not accounted for by the beam element analysis programme, and so the internal bending moments are underestimated and the structural design may be inadequate.

To study the effect of these additional second-order moments, frames were modelled using both beam and shell elements. The frames failed by elastic instability and/or yielding depending on the cross-section slenderness. For frames with slender section columns, the limit state was governed mainly by elastic stability, whereas for frames

Table 1
Dimensions of cross-section for each case of column thickness.

	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Column section	BL-220-80-30-6	BL-220-80-30-4	BL-220-80-30-3	BL-220-80-30-2	BL-220-80-30-1.5	BL-220-80-30-1.2
Rafter section	BL-220-76-20-8	BL-220-76-20-8	BL-220-76-20-8	BL-220-76-20-8	BL-220-76-20-8	BL-220-76-20-8