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Structural modeling of cold-formed steel portal frames

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ABSTRACT

The paper describes the modeling of pitched roof cold-formed steel portal frames with slender cross-sections. Two types of finite element models are introduced: a shell finite element model and a modified beam finite element model. The shell element model involves explicit modeling of each structural member and accounts for the semi-rigid behavior of apex and eave joints by incorporating spring-like elements. The beam element model utilizes a reduced tangent rigidity method to account for cross-sectional instability. Both models are compared with results of experimentally tested portal frames, and good agreement is demonstrated.

1. Introduction

Conventional beam finite element analysis assumes the cross-section and remains unchanged and so cannot consider the effect of cross-sectional instability such as local and/or distortional buckling. In contrast, the Generalised Beam Theory (GBT) [1] was developed as a beam-element-based analysis method capable of accounting for local instability of the cross-section. The theory has been proven capable of predicting interaction between local and overall buckling modes using beam elements [2–4]. The main drawback of GBT is its complexity and associated reluctant take-up by practicing engineers.

In this paper an alternative method [5] is used to include local/distortional buckling effects in beam element models, where local/distortional buckling deformations are considered by simply reducing the rigidities of the section. The reduction of the axial rigidity (EA), the flexural rigidities (EI_z, EI_y), and the warping rigidity (EI_w), as well as other rigidities, are determined by means of *a priori* finite element analyses of short lengths of members, which produce average values of reduced tangent rigidities for nominated lengths of section [5].

The purpose of this paper is to demonstrate the application of the stiffness-based beam element method to portal frames. This is achieved by comparing results obtained using the method with experiments and predictions obtained using full shell element discretization. In the analyses, material properties, loading, boundary conditions, and initial geometric imperfections were the same as those in the experiments. Representative load–deflection curves obtained from the analysis results are shown to agree closely with the tests. The calibrated models can be used to study the interaction of local/distortional buckling and frame sway buckling failure.

2. Tests of portal frames

Portal frame tests [6] were carried out to investigate the effect of cross-sectional instability on the two-dimensional frame behavior (stiffness, sway deflection, ultimate capacity, *etc*). Thus, the frames were designed to ensure that local/distortional buckling developed at an early stage, and that large sway displacements occurred in the tests. Besides, through discrete lateral and torsional restraints, the frames were restrained to deform in-plane.

Three pitched roof portal frames with the same nominal geometry were assembled for testing, and were named as Frames 1, 2 and 3. Frames 1 and 2 were subjected to nominal vertical loading only whereas Frame 3 was subjected to a combination of horizontal and vertical loads. The column and rafter sections are shown in Fig. 1(a) and (b), respectively. The cross-section of the column was relatively slender, with a web depth-to-thickness ratio of 183, and a flange width-to-thickness ratio of 67. The latter ratio was slightly larger than the maximum ratio (60) specified for flange elements in Section 2.1.3 of AS/NZS 4600 [7]. The cross-section of the rafter was stockier than that of the column, which caused local buckling to develop in the column and not the rafter.

The apex and eave joints which were similar to those tested by Lim and Nethercot [8] and Chung and Lau [9] featured brackets bolted to both the webs and flanges of the back-to-back channel sections forming the rafters and columns. Grade 350 mild steel 6 mm plates were used as brackets. The apex and eave joint brackets were cut at a 152° angle and 104° angle respectively. The nominal diameters of the bolts and bolt holes were 16 mm and 18 mm for the webs, and 8 mm and 10 mm for the flanges. The bolt edge distances were 50 mm for the apex joint

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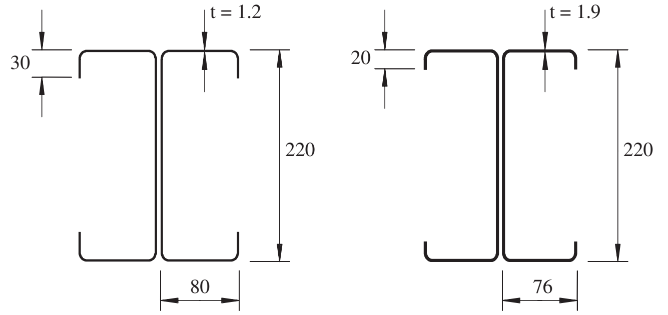

(a) Nominal dimensions of column section
(b) Nominal dimensions of rafter section

(Dimensions in mm)

Fig. 1. Column and rafter cross-sections.

and 65 mm for the eave joints. Grade 8.8 M16 and Grade 8.8 M8 bolts were used for the webs and flanges, respectively. Figs. 2 and 3 show the details of the apex and eave joints respectively.

Fig. 4 shows a general layout of the test frames. Each frame had a span of 8 m and a total height of 5 m from the base plate to the centre of the apex joint. The height from the base plate to the centre of the eave joints was 4 m, and the frame had a 14° pitch. A pinned base was used and point loads were applied in the tests. Further details of the tests can be found in [6].

3. Beam element modeling of frames subject to local/distortional buckling

The basic approach of the beam-element-based method involves considering the primary effect of local/distortional buckling as the reduction of member stiffness against overall compression, bending and torsion. Consequently, the overall behavior of the structure was achieved by using the stiffness of the locally/distortionally buckled cross-section rather than the stiffness of the undistorted cross-section. Based on the theory presented in [10], the stiffness matrix ($\mathbf{K}_l$) and internal force vector ($\mathbf{p}$) in the local system are defined as,

$$\mathbf{K}_l = \int_{L_0} \mathbf{N}_{\delta d2}^T \mathbf{G} \mathbf{N}_{\delta d2}\, dx + \int_{L_0} \mathbf{N}_{\delta d2}^T \mathbf{N}_{\delta d1}^T \mathbf{S}_t \mathbf{N}_{\delta d1} \mathbf{N}_{\delta d2}\, dx \tag{1}$$

$$\mathbf{p} = \int_{L_0} \mathbf{N}_{\delta d2}^T \mathbf{N}_{\delta d1}^T \mathbf{D}\, dx. \tag{2}$$

The matrix $\mathbf{S}_t$, which accounts for the reduction of tangent rigidities during the analysis, is termed as the tangential rigidity matrix and is defined as,

$$\mathbf{S}_t = \begin{bmatrix} (EA)_t & (ES_z)_t & (ES_y)_t & (EI_p)_t & (ES_w)_t & 0 \\ (ES_z)_t & (EI_z)_t & (EI_{yz})_t & (EI_{pz})_t & (EI_{zw})_t & 0 \\ (ES_y)_t & (EI_{yz})_t & (EI_y)_t & (EI_{py})_t & (EI_{yw})_t & 0 \\ (EI_p)_t & (EI_{pz})_t & (EI_{py})_t & (EI_{pp})_t & (EI_{pw})_t & 0 \\ (ES_w)_t & (EI_{zw})_t & (EI_{yw})_t & (EI_{pw})_t & (EI_w)_t & 0 \\ 0 & 0 & 0 & 0 & 0 & (GJ)_t \end{bmatrix} \tag{3}$$

where the tangent rigidity terms ($(EA)_t$, $(EIz)_t$, $(EI_y)_t$, *etc.*) are reduced when local and/or distortional buckling develops. Therefore, the fundamental approach of this method is to find the appropriate reduction factors (τ_g) for each tangent rigidity term, so that

$$S_t = \begin{bmatrix} (\tau_g)_{EA}EA & (\tau_g)_{ES_z}ES_z & (\tau_g)_{ES_y}ES_y & (\tau_g)_{EI_p}EI_p & (\tau_g)_{ES_w}ES_w & 0 \\ (\tau_g)_{ES_z}ES_z & (\tau_g)_{EI_z}EI_z & (\tau_g)_{EI_{yz}}EI_{yz} & (\tau_g)_{EI_{pz}}EI_{pz} & (\tau_g)_{EI_{zw}}EI_{zw} & 0 \\ (\tau_g)_{ES_y}ES_y & (\tau_g)_{EI_{yz}}EI_{yz} & (\tau_g)_{EI_y}EI_y & (\tau_g)_{EI_{py}}EI_{py} & (\tau_g)_{EI_{yw}}EI_{yw} & 0 \\ (\tau_g)_{EI_p}EI_p & (\tau_g)_{EI_{pz}}EI_{pz} & (\tau_g)_{EI_{py}}EI_{py} & (\tau_g)_{EI_{pp}}EI_{pp} & (\tau_g)_{EI_{pw}}EI_{pw} & 0 \\ (\tau_g)_{ES_w}ES_w & (\tau_g)_{EI_{zw}}EI_{zw} & (\tau_g)_{EI_{yw}}EI_{yw} & (\tau_g)_{EI_{pw}}EI_{pw} & (\tau_g)_{EI_w}EI_w & 0 \\ 0 & 0 & 0 & 0 & 0 & (\tau_g)_{GJ}GJ \end{bmatrix} \tag{4}$$

where the unreduced rigidities (EA, EI_z, EI_y, etc.) can be calculated based on the geometry of the undistorted cross-section. The calculation of the reduction factors (τ_g) is described in [11].

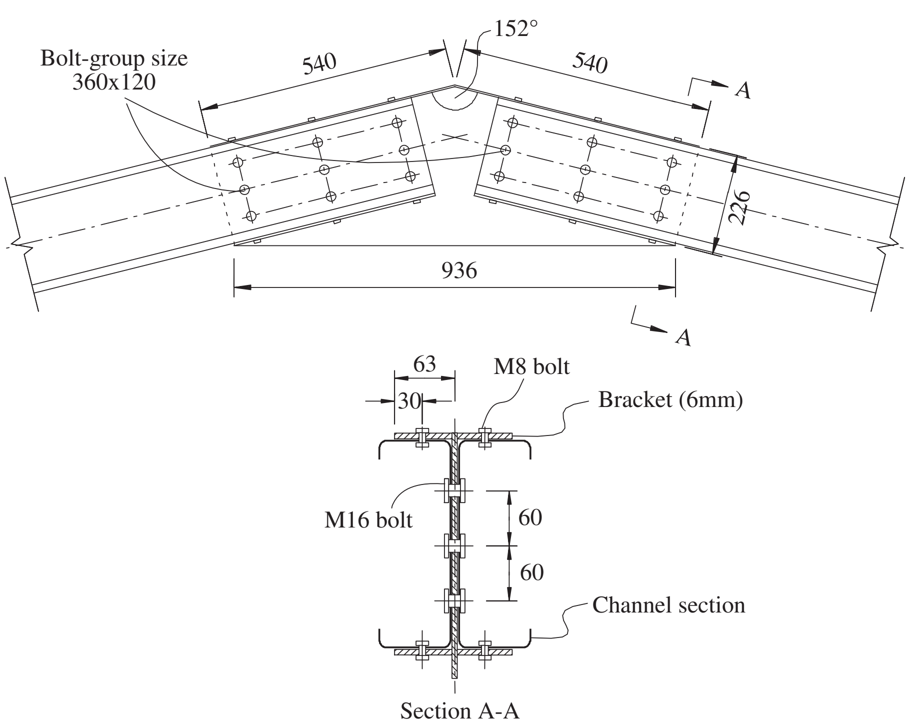

Fig. 2. Geometry of apex joint.