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Elastic buckling of columns with a discrete elastic torsional restraint

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ABSTRACT

Cold-formed steel haunched portal frames are popular structures in industrial and housing applications. They are mostly used as sheds, garages, and shelters, and are common in rural areas. Cold-formed steel portal frames with spans of up to 30 m are now being constructed in Australia. As they are relatively new to the market, current design recommendations are fairly limited. In the specific frame system analyzed herein, the column is partially restrained against twist rotation at an intermediate point where the knee brace joining the column and rafter is connected. An experimental program was carried out on a series of portal frame systems composed of back-to-back channels for the columns and rafters. It was found that changing the knee brace and knee brace-to-column connection bracket significantly affected the buckling capacity of the column, however this was not captured in design calculations. In order to correctly predict frame behavior and ultimate loads for design purposes, the column buckling capacity must be accurately calculated. This paper presents an energy method approach to calculate the buckling load of a column with an intermediate elastic torsional restraint. Various end conditions of the column are considered including column base semi-rigidity, as well as multiple loading conditions. Displacement functions are determined based on measured experimental data. The Southwell and Meck plot methods to determine column buckling loads are discussed. The column buckling loads determined from the plot methods and calculated by the energy analysis are compared to the experimental column buckling loads. It is shown that the energy method outlined herein predicts the buckling load within 6% for columns with an intermediate elastic torsional restraint.

1. Introduction

In order to correctly predict frame behavior and ultimate loads for design purposes, the column buckling capacity must be accurately calculated. Design codes, such as the Australian standard [1] and the North American standard [2] do not explicitly consider the effect of the knee brace-to-column connection bracket, shown in Fig. 1, on restraining the twist of the unbraced column in frames with knee braces connected between the columns and rafters. The effects of the knee brace to column connection can be incorporated into design capacity calculations through the use of effective length factors. However, there is no guidance on how to correctly quantify the effective length due to the effects of this connection. The Australian Steel Institute's Design Guide for Portal Frame Steel Sheds [3] defines the column effective lengths for buckling about the minor axis and for twisting as the maximum length between adjacent bracing points. Suitable bracing points are considered to be the connections between the column and a fly brace, girt, rafter, or column base. The knee brace connection, by default, is deemed not to provide sufficient restraint to be considered a bracing point, unless it can be proven otherwise. Therefore, due to the uncertainty in determining the correct effective length resulting from the knee brace connection, the column capacity for this type of structure could be incorrectly calculated.

An experimental program on a series of eight full scale 14 m span portal frame systems with unbraced columns composed of bolted back-to-back lipped channels was conducted [4,5]. The setup of these experiments is shown in Fig. 2, where three frames were connected in parallel with purlins to create a free standing structure, and only the center frame was tested. The series consisted of frames of various configurations including modifications to the knee brace connection, the inclusion of sleeve stiffeners in the columns and rafters, and applied loading of either gravity only or combined wind and gravity. Vertical base reactions in the columns were measured by strain gauges near the column base for experiments 1 through 4, and by load cells on the column base for experiments 5 through 8. The results of the experiments with applied gravity load only are shown in Table 1, where ultimate loads, P_u, are given for the entire frame and per column, and the column base reactions in bold belong to the failed column in each experiment. Experiments 1 and 2 were nominally the same configuration. Experiments 5 and 7 were intended to be companion tests, where the

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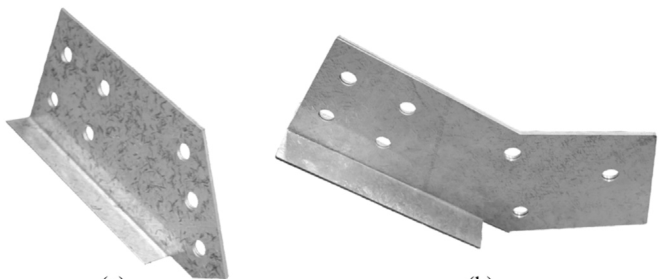

(a) (b)

Fig. 1. Knee brace-to-column (KBC) connection bracket where the left end connects to the knee brace and the right end connects to the column (a) extending through half the column depth, and (b) extending through the full column depth.

only change between them was the addition of sleeve stiffeners to the columns and rafters in experiment 7. However, the intended 3.0 mm thick plates for the knee brace-to-column connection bracket were unavailable at the time of testing for experiment 5, thus 3.3 mm thick plates were substituted.

The results of the experiments with applied wind and gravity loads are shown in Table 2. A 5 kN block, representing wind loads, was applied to the frame at the eaves connection of the north column, and held constant while gravity loads were applied to the frame until failure. Therefore, the north column was the critical column for all wind load experiments. The ultimate vertical loads, P_u, are given for the north columns due to the wind load component only, the gravity load component only, and the combined total. More details on the experimental program are given elsewhere [4,5].

It was found that strengthening the knee connection in experiment 5 resulted in a substantial increase of frame ultimate vertical load of 34% compared to the average of the nominally identical frames tested in experiments 1 and 2. However, as the effects of the knee brace connection are not reflected in design standards, increases in frame ultimate load due to increasing the thickness of the knee brace-to-column (KBC) connection bracket would be negated in design calculations. Preliminary results for frames with applied gravity loads only have been reported [6]. A more comprehensive analysis, including frames with combined loading conditions, is presented herein. The aim of this work is to determine an approach that correctly calculates column buckling capacity by accounting for the effect of the knee brace-to-column connection.

Table 1
Column ultimate vertical loads from experiments with applied gravity loads.

Experiment	t_{KBC} (mm)	Sleeve stiff.	P_u experiment (kN)		
			Frame Total	Column N	Column S
1	2 × 2.4	no	21.8	10.3	**11.5**
2	2 × 2.4	no	22.8	11.1	**11.7**
5	2 × 3.3	no	29.9	**15.9**	14.0
7	2 × 3.0	yes	29.7	**15.8**	13.9

Table 2
North column ultimate vertical loads from experiments with applied wind and gravity loads.

Experiment	t_{KBC} (mm)	Sleeve stiff.	P_u experiment (kN) induced by		
			Wind load	Gravity load	Total
3	2 × 2.4	no	0.6	9.5	10.1
4	2 × 2.4	no	0.7	6.2	6.9
6	2 × 3.0	no	1.9	9.0	10.9
8	2 × 3.0	yes	1.9	9.7	11.6

2. Energy equations

2.1. Internal actions of column

2.1.1. Gravity loads

Consider a portal frame, as shown in Fig. 3, with pinned bases, column length L, height from column bases to apex L_a, and horizontal distance from the eaves to apex L_h. The knee brace is connected at an angle θ from the vertical to the column at a distance of βL from the base, where β is the ratio of the column height between the column base and the knee connection to the total column length. There is a uniformly distributed vertical load q on the rafters.

Assuming pinned connections between all members, the frame is statically determinate and the horizontal reaction force at the base, H, can be calculated by taking the sum of moments about the apex, as shown in Fig. 4(a), and is given in Eq. (1). The actions on the column in between the knee and the eave are shown in Fig. 4(b), where F is the compression force in the knee brace. Taking the sum of moments about the eaves yields the force in the knee brace (Eq. (2)). The axial forces acting in the column are shown in Fig. 4(c). The vertical reaction force,

Fig. 2. Setup of experiment.