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Moment redistribution in cold-formed steel continuous beams

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A R T I C L E I N F O

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A B S T R A C T

The external envelope of steel framed industrial buildings normally involves the use of purlins and rails spanning between the main hot-rolled frames to support the roofing/cladding. These purlins are typically light-gauge cold-formed steel members of complex shape for which the thin-walled nature of the material means that local instabilities will significantly influence their structural behaviour. Economic design should be based on failure of the system, recognising the opportunity for redistribution of moments. This paper presents the findings from a numerical investigation of the degree of moment redistribution in continuous cold-formed steel beams subjected to a downward (gravity) uniformly distributed load (UDL). Three types of nonlinear finite element analysis were validated against previously reported physical tests: (i) continuous two-span beams subjected to a UDL, (ii) single span beams subjected to a central point load producing a moment gradient and (ii) single span beams subjected to two point loads producing a central region under pure bending. The interior support moments from the continuous beam models were compared against reference moment capacities from the three-point bending models. Based on various different section sizes, covering a range of cross-sectional slenderness, full moment redistribution with no drop-off in moment at the interior support was found to be possible only for stocky sections but not for slender sections. In the case of slender sections, local and distortional buckling caused a reduction in interior support moment prior to failure of the system. Hence a design formula is proposed to estimate the post-peak reduction of interior support moment from its initial peak, and this reduced moment capacity is then used in conjunction with the full span moment to determine the load-carrying capacity of the system. Comparisons show the proposed approach to offer accurate prediction of observed system failure loads.

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1. Introduction

Modern light roof construction for industrial or warehouse buildings normally comprises cold-formed steel purlins covered with sheeting. These purlins are used as secondary steelwork between the main frames. The thin-walled nature of cold-formed steel members makes them susceptible to cross-sectional instabilities such as local and distortional buckling. With buckling limiting the overall load carrying capacity, the basic assumption for the application of plastic design that the first hinge to form in a dual span system can maintain its moment capacity until a collapse mechanism is formed, is not always guaranteed. This paper reports on a study designed to provide suitable modifications to permit the basic concept of plastic theory to be applied in such cases.

Many research projects have been carried out to investigate the structural behaviour of multi-span purlin systems. Willis and Wallace [1] and Hancock et al. [2] conducted tests on purlin systems with through-fastened roof panels, while previous researchers [3–5] conducted studies to investigate the flexural behaviour of lapped moment connections for single span and 2-span Z sections. As an alternative to conducting full scale testing which can be expensive and time consuming, finite element (FE) models have also been increasingly used for research into cold-formed steel. Early publications on FE modelling of cold-formed steel members appeared more than a decade ago [6]; since then treatments have become more robust and practical with the use of faster processing power. Recently, Haidarali and Nethercot [7–9] successfully analysed cold-formed Z sections using ABAQUS [10] and investigated the local/distortional buckling behaviour for a range of different Z section arrangements subjected to pure bending.

Research into the inelastic strength of cold-formed steel members is relatively limited. Reck et al. [11] and Yener and Peköz [12–14] have shown that the inelastic strength reserve for cold-formed steel beams due to partial plastification of the cross-section can permit increases of up to 35% beyond the yield moment for many practical shapes. While Eurocode 3 [15] recognises this potential inelastic strength and the scope for moment redistribution

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through appropriate physical testing, direct guidance on moment redistribution in multi-span cold-formed steel systems is lacking. In the present study, FE models are first validated against available test data, and then utilised to generate a series of parametric results. These results are used to underpin a proposed design approach to allow for moment redistribution in 2-span continuous purlin systems.

2. Design approach for 2-span systems

For a statically indeterminate 2-span system, as shown in Fig. 1, additional load carrying capacity can be achieved if the cross-section bending moment capacity can be maintained over sufficiently large rotations to allow for moment redistribution and exploitation of the unused moment capacity within the span. This principle is commonly employed when designing hot-rolled steelwork and has been adapted for reinforced concrete and composite construction. To date, moment redistribution is rarely considered in cold-formed steel design since most sections comprise of plates with high width-to-thickness ratios that typically exceed the limits outlined for plastic design. Hence, 2-span purlin systems are generally designed based on elastic principles, which only utilise the full cross-sectional moment capacity at the central support but not within the span. However, due to the statically indeterminate nature of the 2-span system, there is the potential to use a greater proportion (possibly all) of the unused moment capacity within the span by allowing for redistribution of moments.

2.1. Moment redistribution

Given the ideal situation of full plastic redistribution of moments, the bending moment at the interior support, $M_{support}$ and within the span, M_{span} both reach their full cross-sectional capacities because the moment-rotation relationship for the support region is such that no significant drop off in capacity occurs before rotations develop sufficiently to allow the span moment to reach its capacity. For such cases, it is common for the designer to adopt an idealised elastic-perfectly plastic moment-rotation relationship, as demonstrated in Fig. 2, where M_p is the plastic moment capacity and θ_p is the rotation at M_p.

However, due to the high width-to-thickness ratios of the constituent elements, cold-formed steel purlins are susceptible to local instabilities such as local and/or distortional buckling. Thus the design issue for cold-formed steel construction is that the moment-rotation behaviour may differ from the ideal arrangement detailed in Fig. 2. Should this be the case, i.e. attaining full capacity within the span is accompanied by a reduction in interior support moment, then appropriate allowance must be made.

To study the level of moment redistribution in indeterminate purlin systems, a series of 2-span continuous and representative single span beams was modelled using ABAQUS [10]. For each 2-span FE model, two further single span models were created to generate reference values for the interior support and span moments in the 2-span model respectively. The largest sagging moment (elastically located at 0.375 L (see Fig. 3) from the beam ends) M_{span} in the 2-span system was represented by a single span model subjected to 4-point bending (i.e. uniform bending between point loads) M_1. Knowing that the moment–rotation response at the interior support is influenced by the interaction of a sharp moment gradient and a high shear force and concentrated load, the interior support $M_{support}$ was represented by the maximum moment obtained from a single span model of the same cross-section subjected to 3-point bending M_3 with length L′ and with the load introduced in the same manner as in the corresponding continuous beam such that the effect of the concentrated force is captured, as shown in Fig. 3. The length L′ is the length of the hogging region from the 2-span model. For the 4-point bending models used to determine M_1, the span length L* was kept constant at 4880 mm, which was sufficient for local and distortional buckling to freely develop within the constant moment region. M_1 and M_3 are then considered as reference bending moments for the span and interior support regions respectively, that can capture the local and/or distortional buckling response of the cross-sections and, in the case of the M_3 models, can allow for the influence of moment gradient, shear force and concentrated load, the latter of which will be a function of the cleat arrangement used to secure the purlins.

The ultimate load carrying capacity q_{ult} for a 2-span system with equal spans subjected to a UDL can be determined, from equilibrium,

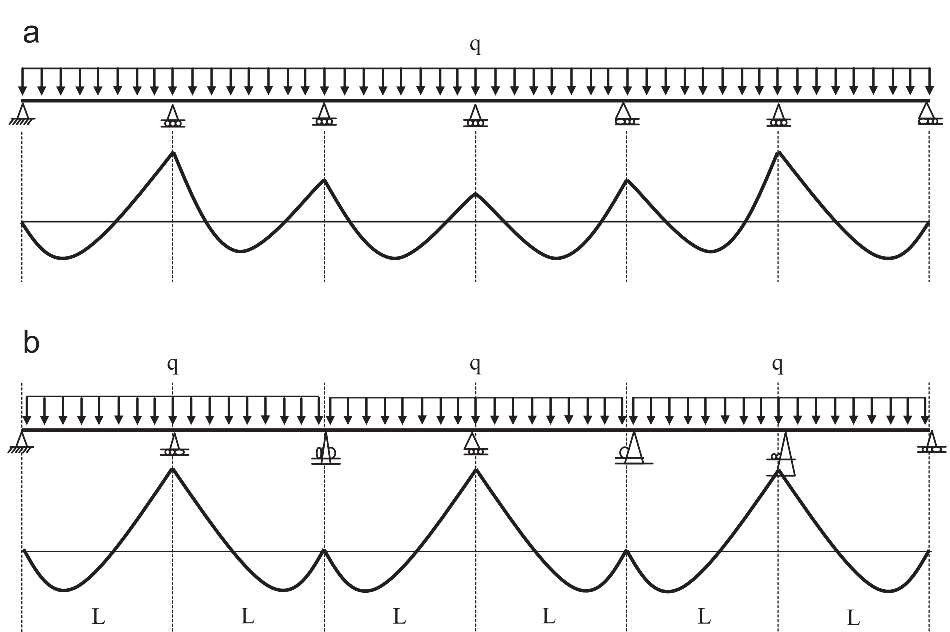

Fig. 1. Elastic bending moment diagram (BMD) for (a) multi-span and (b) a series of 2-span purlin systems.

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