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Effect of stressed skin action on the behaviour of cold-formed steel portal frames

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ABSTRACT

This paper describes a series of six full-scale laboratory tests conducted on cold-formed steel portal frame buildings in order to investigate the effects of joint flexibility and stressed skin diaphragm action. The frames used for the laboratory tests were of 6 m span, 3 m height, 10° pitch and the frame spacing was 3 m. Vertical loading was applied in two tests, and horizontal loading was applied in another four tests. The laboratory test set-up represented a building having two gable frames and two internal frames. Tests were conducted on frames having two joint types, both with and without roof sheeting. It was shown that as a result of stressed skin action, the internal frame with roof sheeting resisted approximately three times more horizontal load than the bare frame and the deflection of the internal frame was reduced by 90% relative to the bare frame. When the difference in loads between 2D (bare frame model) and 3D (stressed skin model) were considered, it was shown that the joint flexibility of the frame has a significant effect on the load transfer between frames through the roof sheeting. It was found that the 'true' loads transferred to the gable frames are between three and seven times higher than the loads deriving from tributary area. By using stressed skin analysis, it is possible to assess the shear force in the roof sheeting so that damage to the fixings is prevented and a more economical design is possible.

1. Introduction

Portal frame buildings composed entirely of cold-formed steel (see Fig. 1) with spans up to 20 m, frame spacings up to 4 m and subject to relatively light loading, can be a viable alternative to conventional hot-rolled steel frames [1–6]. Uses of cold-formed steel portal frames include light industrial, sports and agricultural buildings. In such light-weight steel portal frames, channel-sections are used for the column and rafter members, and top-hat sections may be used for the purlins and side rails (see Fig. 2). Top-hat sections are considered to be more efficient than zed-purlins for cold-formed steel portal frames where the frame spacings (or purlin spans) are in the range of 3–4.5 m, compared with 6 m for conventional hot-rolled steel frames. They are also much stiffer than zed-purlins in terms of transferring shear load to the roofing (see Fig. 2) [7].

Principally under horizontal load, the roof sheeting is known to act as a shear diaphragm (see Fig. 3) [8] and by this means, loads are transferred to the end gables that should be designed to resist these forces. This stiffening effect, referred to as stressed skin or diaphragm action [9–14], explains why a clad frame behaves differently from a bare frame. Design recommendations on stressed skin action were first published by ECCS TC17 [15] and further extended by Davies and Bryan [11]. Other contributors were: Bates et al. [16], Bryan and Moshin [17], Strnad and Pirner [18] , Davies et al. [12], Heldt and Mahendran [19] and Mahendran and Moor [20]. It should be noted that this research focused on hot-rolled steel portal frames in which the haunched eaves and apex joints can be classified as rigid.

In practice, however, the effects of stressed skin action are often ignored by designers of hot-rolled steel portal frames. However, cold-formed steel portal frames have more flexible joints [21] and also use relatively stiffer top-hat purlins, which means that not including the effects of stressed skin action could lead to roof failure at serviceability loads (see Fig. 4) [22]. This could lead to excessive tearing of the fixings and water leakage into the building [13].

3D structural analysis is now a standard tool for designing complex structures as it gives a more accurate representation of the structural behaviour. However, portal frame buildings are still predominantly modelled as 2D bare frames [23]. An exception is

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Fig. 1. Typical cold-formed steel portal framing system.

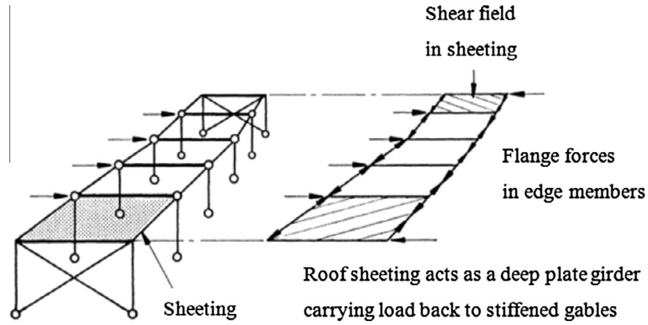


Fig. 3. Stressed skin action under horizontal load on buildings (after BS 5950-Part 9 [9]).

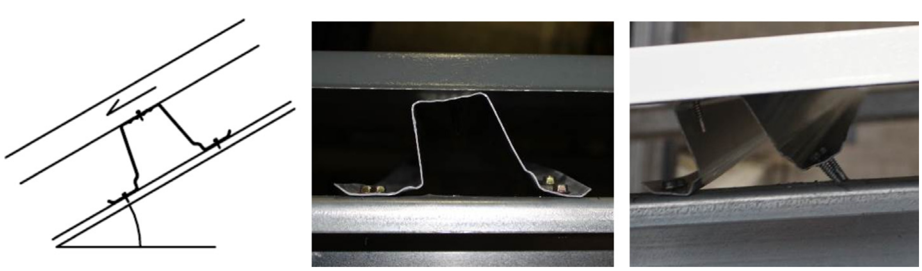
where seismic actions have to be considered, which has been highlighted in research on seismic design of cold-formed steel frames [24–28].

Furthermore, for cold-formed steel portal frames, steel designers often refer to guidance for equivalent hot-rolled steel frames [29] in terms of deflection limits of bare frames, but these are discretionary. As a result, designers sometimes relax these deflection limits to achieve a more economical design under the assumption that the roof panel will reduce the deflections, possibly by as much as 50%. However, the effect of relaxing deflection limits can lead to lighter and more flexible internal frames based on 2D design. This results in an underestimation of the forces in roofing and hence can lead to an under-design of the gable end-frames. Fig. 4b shows the consequences of diaphragm action loads on the gable rafter. This is even more important when the joint rotation adds to frame flexibility. Joint rotation in cold-formed steel portal frames is associated with the bearing of the mechanical fasteners (generally bolts) acting in shear on relatively thin steel plates. Designers in the UK often use design guides [30] in which the moment resistance of connections between cold-formed sections is assumed to be governed by bearing resistance of fasteners. The rotational stiffness of a joint and slip due to tolerances in bolt hole is often neglected in the analysis [31]. Investigated joints are therefore similar to what can be found in the practice and the tests using these connections take account of initial slip in the bolts.

Experimental investigation on portal frames using back-to-back lipped channel sections and bolted joints had been already published [28] but was focused on developing full-strength connections of much greater rotational stiffness than those reported in this paper. The behaviour of a bare frame was investigated in the seismic design context and stressed skin action was not included in this study [28]. The study highlighted the importance of component testing in establishing accurate strength and stiffness characteristic of joints which must be included in an analysis model.

In this paper, the results of six full-scale tests on cold-formed steel portal frames are presented. Details of the eaves and apex joints considered in this paper are shown in Fig. 5; such joints are typical of those used for cold-formed steel portal frames in practice. As can be seen, the joints are formed using brackets that are bolted between the webs and outer flanges of the cold-formed steel channel-sections. Two different bolt-group sizes are considered for the joints, with each bolt-group size (and therefore bracket size) having a different rotational stiffness. Firstly, tests on frames without roof sheeting are described. Vertical loading was applied in two tests, and horizontal loading was applied in four tests. Secondly, for horizontal loading only, the frame tests were carried out with and without roof sheeting to determine the effect of stressed skin action. The component tests are described separately for both the roof panel and the beam-to-column connections.

Finally, 3D non-linear frame analysis models are presented, which show that the frame behaviour can be predicted accurately if the experimentally determined joint strength and stiffness are used, as well as the stiffness of the roof sheeting. Using these models, the design of cold-formed steel portal frame buildings of 6 m span, height to eaves of 3 m and frame spacing of 3 m are considered in which the design variables are the stiffness of the internal frames and the length of the building. The simplified 2D design assumption, of the load on the end gable being half that of an internal frame is shown to be incorrect. It is demonstrated in Fig. 20 that if horizontal deflection limits [29] are ignored, such assumption will under-predict the loads acting on end gables by as much as factor of seven. It is concluded that the horizontal loading acting on 1.7 bays should be used as the minimum to design the end gables. This is a factor of 3.4 higher than the simplified assumption that the load on the end gable is half that of an internal frame (see Fig. 20c). It is estimated that for building of 12 m length, 2D design (see Fig. 18) requires 981 kg of steel which is 42% more than a portal frame building with joint type B (see Fig. 9b). The 'true' loads acting on clad buildings are a function of a length as



(a) Top- hat purlin (b) Shear deformation due to diphragm action (c) Failure of purlin-to- end-rafter connection

Fig. 2. Behaviour of top-hat sections acting as purlins in a clad frame.