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Component-based model of buckling panels of steel beams at elevated temperatures

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ABSTRACT

Both bottom-flange buckling and beam-web shear buckling have been observed in many full-scale fire tests in the vicinity of beam-to-column connections. These phenomena can influence the load redistribution within the adjacent connections and the global structural behaviour, detrimentally affecting the structural overall fire resistance. However, existing models for bottom-flange buckling overestimate the structural resistance when the beam is slender. In this work, a new analytical model has been created to predict both of these types of buckling behaviour in steel beams in the vicinity of beam-to-column connections at elevated temperatures. The model considers the individual effects of both buckling modes, as well as their interaction. It is capable of predicting the force–deflection relationship of the buckling zone from the initial elastic loading stage to run-away failure. The new analytical model has been compared with the existing Dharma's model and a range of 3D finite element simulations created using the ABAQUS software. Comparisons have shown that the proposed method gives better predictions than Dharma's model. A component-based model of the buckling zone has been created on the basis of this new analysis. The component-based model can provide sufficient accuracy, and will be implemented in the software Vulcan for performance-based global structural fire analysis.

1. Introduction

The collapse of the '7 World Trade' building in New York City [1] indicates that the progressive collapse of the whole building was initially triggered by the failure of beam-to-column connections. Connection failure was also observed on many other occasions, including the well-known full-scale Cardington series of Fire Tests [2]. Beam-to-column connections have been among the key elements for the investigation of the robustness of steel structures in fire. It has traditionally been assumed that connections have sufficient fire resistance, because of their lower temperatures and slower rates of heating compared with the parts to which they are connected. However, connections are actually under complicated and extreme force combinations transferred from the connected members; these forces are not usually considered in ambient-temperature design. A number of previous research studies [3–6] have shed light on the performance of beam-to-column connections in fire, and have further proved that connections are potentially the most vulnerable parts of a steel-framed structure in fire.

The Cardington Fire Tests [7] indicated that combinations of beam-web shear buckling and flange buckling are very prevalent under fire conditions. This phenomenon can significantly influence the internal forces in the connections. Flange buckling can raise the neutral axis, which changes the force distribution in the connection's components. Although at early stages of heating the top bolt rows would logically experience higher tensile force without flange buckling than when it has occurred, this situation may be reversed in the high-temperature catenary stage when, without a significant connection moment, most of the catenary force may be carried by these rows. Local buckling at the beam ends will also have an effect on the deflection of the beam, and therefore influence the net tying force within the connection. The increased beam deflection during the heating phase of the fire will significantly increase the tension force on the connection during the cooling down period. However, the contribution of the combination of beam-web shear buckling and flange buckling in the vicinity of beam ends has not been taken into consideration by almost any of the existing research.

The behaviour of structural elements in real frames observed in the full-scale Cardington Fire Tests [2,8–10]was very different from that observed in furnace tests on isolated elements. This indicates the importance of performance-based design, which sufficiently considers the interactions between various members of the structure. However, full-scale structural testing is expensive. To carry out finite-element modelling of an entire structure, including detailed representation of the connections, is computationally demanding, and is therefore not feasible for practical design. A practical alternative approach, component-based modelling of connections, has been proposed [11,12].

The component-based method considers different parts of each connection as an assembly of individual nonlinear springs with predefined force-deformation characteristics. This method has been used to

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Notation

b	flange width
c	half flange width
d	depth of a beam web
f	width of one strut in compression zone
F	vertical shear force at the end of the buckling zone
F_{max}	maximum reaction force
$F_{p,T}$	reaction force when plastic bending moment resistance is reached
h_c	height of the area resisting axial force in a strut
k_1	the stiffness of the compressive spring in the flange buckling component
k_2	the stiffness of the compressive spring in the shear buckling component
k_E	reduction factor for young's modulus at elevated temperatures
k_y	reduction factor for yield stress at elevated temperatures
M	bending moment at the end of the buckling zone
M_p	bending moment resistance of one beam-web compressive strut
P_c	axial force resistance of one beam-web compressive strut
t_f	thickness of the flange
t_w	thickness of the beam web
α	the angle between tensile strips and the deformed upper flange
α_1	the angle between a yield line and vertical direction
Δ_1	out-of-plane deflection of one strut
Δ_{vs}	the vertical movement of the right edge of the shear panel
$\varepsilon_{l,\theta}$	limiting strain for yield strength at elevated temperatures
$\varepsilon_{p,\theta}$	strain at the proportional limit at elevated temperatures
$\varepsilon_{u,\theta}$	ultimate strain of steel at elevated temperatures
$\varepsilon_{y,\theta}$	yield strain of steel at elevated temperatures
θ_1	the rotation caused by bottom-flange buckling
θ_2	the rotation caused by shear buckling
σ_c	compressive stress in the compressive strips
σ_{cf}	compressive stress in the flange along beam length
σ_{eq}	yield strength of the flange considering reduction caused by axial stresses and shear stresses
σ_{eq1}	yield strength of the flange considering reduction caused by axial stress parallel to yield lines
$\sigma_{p,\theta}$	stress at the proportional limit at elevated temperatures
σ_r	stress normal to the yield lines
σ_t	tensile stress in the tensile strips
σ_{tf}	tensile stress in the flange along beam length
σ_y	yield strength of steel at ambient temperature
$\sigma_{y,\theta}$	yield strength of steel at elevated temperatures
$\sigma_{yf,\theta}$	yield strength of the flange without considering reduction at elevated temperatures
$\sigma_{yw,\theta}$	yield strength of steel web at elevated temperatures
τ_r	shear forceparallel to the yield lines

establish a connection element, which has been integrated into the software Vulcan [13], which was developed by the Structural Fire Engineering Research Group at the University of Sheffield. Vulcan is a three-dimensional programme, allowing engineers to conduct three-dimensional structural robustness assessments. A variety of elements (beam-column, connections, shear connector and slab) has already been implemented. Recently, Sun. *et al.* [14,15] developed a static-dynamic solution procedure for Vulcan. This procedure is able to capture re-stabilization after initial instability caused by local failure. For instance, this can be used to trackthe sequential failure of different connection components during progressive collapse of a whole structure. Sufficient validation [8,9,16–18] has been carried out to demonstrate that Vulcan is accurate and computationally-efficient software to be used in performance-based fire engineering design.

Fig. 1. Flange buckling and beam-web shear buckling in combination [7].

Elghazouli et al. [19] implemented a local-buckling model within a frame analysis programme to investigate the influence of local buckling at the beam ends on the fire response of frame members and sub-assemblies. This study indicated that, although local buckling at the beam ends may not directly trigger overall structural collapse, it can have detrimental effects on the deflections of, and load re-distributions between, structural elements. This will influence the fire resistance of the structure. However, the local buckling model presented in Elghazouli's work is based on elastic plate buckling theory, which is not appropriate for representing the buckling behaviour of Classes 1 and 2 sections. No sufficient validation of the local-buckling model presented in his paper has been provided. A body of research [20–24] has been carried out to investigate the local in-plane flange buckling phenomenon, including both the pre- and post-buckling stages. Recent research has a common solution for the pre-buckling stage, while different collapse models [21–24] of the post-buckling stage have been proposed since 1965 [20]. The local buckling collapse mechanisms in all these models are composed of yield lines and plastic zones. The choices of possible yield line patterns are based on experimental observations. All models assume that the yield lines, formed within the elastic buckling wavelength, will not change their positions in the post-buckling stage. However, these studies nearly all focus on the effects of local buckling on the rotational capacity and ductility of beam-ends,

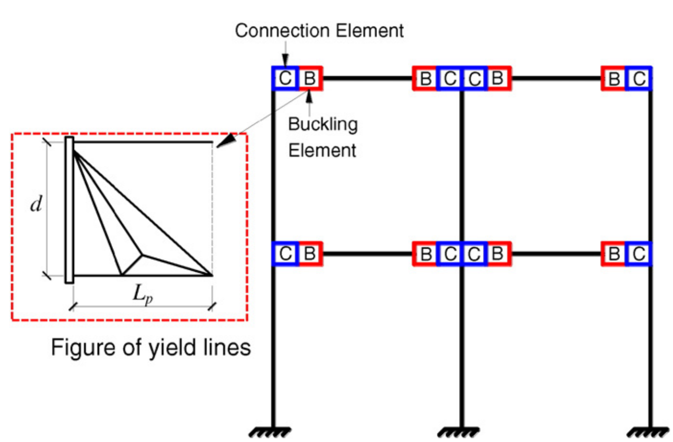

Fig. 2. Frame analysis including connections element and buckling element.