Engineering Structures 85 (2015) 73–82

Contents lists available at ScienceDirect

Engineering Structures

journal homepage: www.elsevier.com/locate/engstruct

An analytical approach to modelling shear panels in steel beams at elevated temperatures

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ARTICLE INFO

Article history:
Received 2 June 2014
Revised 28 November 2014
Accepted 9 December 2014
Available online 24 December 2014

Keywords:
Shear buckling
Connections
Component-based model
Fire

ABSTRACT

Shear buckling of beam webs in the vicinity of beam-to-column connections has been observed in many full-scale fire tests. This phenomenon can lead to force redistribution within the adjacent connections, and even influence the performance-based analysis of full-scale structures in fire. However, beam-web shear buckling for Class 1 beams at either ambient or elevated temperatures has not been well studied previously. In this work an analytical model has been created to predict the shear buckling behaviour of Class 1 beams in the vicinity of beam-to-column connections at ambient and elevated temperatures. The model considers the reduction of resistance of the beam after web shear buckling has occurred. It is capable of predicting the shear resistance and transverse drift of the shear panel from its initial loading to final failure. Several 3D finite element models have been created using the ABAQUS software, in order to validate the analytical model over a range of geometries. Comparisons between the theoretical and FE models have shown that the proposed method provides sufficient accuracy to be implemented and used in performance-based global modelling.

1. Introduction

The official investigation [1] of the collapse of the '7 World Trade' building in New York City has indicated that the collapse of the building was triggered by the failure of beam-to-column joints after prolonged exposure to fires. Joint failure may initiate fire spread within a building, or even progressive collapse of the whole building. Hence, the joints are among the most critical structural elements of a steel or composite framed building in fire conditions. A considerable body of research related to the modelling of joints in fire has been carried out during the last three decades [2–4]. The Cardington Fire Tests [5] indicated that the shear buckling of beams in the vicinity of beam-to-column joints, is very prevalent under fire conditions, as shown in Fig. 1, which shows shear buckling from the well-known Cardington full-scale tests. For a column-face joint, the effect of shear buckling of the beam web, which increases the transverse drift of the beam, can change the force distribution in the joint components themselves. Conversely, beam-web shear buckling, which leads to an increase of beam deflection, can be advantageous, as it may reduce the net tying force in the joint. Considering the same vertical force component, a lower local resultant shear force exists along the beam when it is acting essentially as a catenary cable. However, nearly all the existing research on joints has neglected the contribution of the shear buckling behaviour in the vicinity of beam-ends. Therefore, it is useful to be able to predict beam-web shear buckling behaviour in fire, and to include this effect in full-structure design modelling, which can be used with a view to preventing progressive collapse.

In this paper, the shear buckling behaviour of the beam web has been studied. An analytical model has been developed to predict the behaviour of shear panels for Class 1 beams, at both ambient temperature and elevated temperatures; in developments of the work this can be used to generate a component-based shear-panel element. The analytical model can initially evaluate the distance between plastic hinges on the flanges, and then reasonably predict shear capacity and vertical deflection of the shear panel. A range of 3-dimensional finite element models has been created using ABAQUS. These models can be used to produce force–transverse deflection relationships which are necessary to validate the analytical model over a range of geometries. The results from the analytical model will be seen to agree well with the ABAQUS results. As the shear stiffness of the shear panel changes significantly due to beam-web shear buckling, a new component-based shear panel element, which considers the shear panel as a separate component, will be created based on the analytical model, as shown in Fig. 2.

Since no practical research has been done on the post-local-buckling behaviour of Class 1 beams at either ambient or elevated

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http://dx.doi.org/10.1016/j.engstruct.2014.12.016
0141-0296/

Nomenclature

b_f	flange width
c	distance between plastic hinges along the bottom flange
d	height of a beam web
e	length of an arbitrary tensile strip
E	Young's modulus at ambient temperature
G	shear modulus at ambient temperature
f	width of one strut in compression zone
$f_{p,\theta}$	stress at the proportional limit
$f_{y,\theta}$	yield strength of steel at elevated temperatures
F_e	shear resistance of the beam at the end of elasticity
F_p	shear resistance of the beam at the initiation of plastic shear buckling
F_u	ultimate shear resistance of the beam
h_c	height of the area resisting axial force in a strut
I	second moment of area of a cross-section
$k_{E,\theta}$	reduction factor for Young's modulus
l	half length of the beam
L	full length of a beam
m	distance between the beam-end plastic hinge and an arbitrary tensile strip
M_0	plastic bending moment capacity of one flange
M_1	reduced plastic bending moment capacity of one flange
M_p	bending moment resistance of one strut
M_r	bending moment at the end of the beam
n	the distance between two end points of a tensile strip in Region B along the deformed flange
P_c	axial force resistance of one strut
q	uniformly distributed load on the top flange of the beam
t	thickness of the beam web
t_f	thickness of the flange
W_{At}	the internal work of tensile strips in Region A
W_{Bt}	the internal work of tensile strips in Region B
W_C	the internal work of the beam web caused by compression
W_e	the external work of the beam
W_f	the internal work of the plastic hinges on the beam flanges
W_T	the internal work of the beam web caused by tension
W_W	the internal work of the beam web
α	the angle between tensile strips and the deformed upper flange
γ	the angle between diagonal of the shear panel and deformed upper flange
δ_e	elongation of an arbitrary tensile strip
δ_{tA}	elongation of the tensile strip in Region A
δ_{tB}	elongation of the tensile strip in Region B
Δ	out-of-plane deflection of one strut
Δ_e	mid-span deflection at the end of elasticity
Δ_p	mid-span deflection at the initiation of plastic shear buckling
Δ_{rB}	the resultant movement of the right point of the representative strip in Region B
Δ_{rs}	the resultant movement of the right edge of the shear panel
Δ_u	ultimate mid-span deflection
Δ_{vb}	mid-span deflection caused by bending moment
Δ_{vB}	the vertical movement of the right point of the representative strip in Region B
Δ_{vm}	the vertical movement of the mid span caused by shear force
Δ_{vs}	the vertical movement of the right edge of the shear panel
$\varepsilon_{l,\theta}$	limiting strain for yield strength
$\varepsilon_{p,\theta}$	strain at the proportional limit
ε_t	tensile strain of a tensile strip
$\varepsilon_{u,\theta}$	ultimate strain of steel
$\varepsilon_{y,\theta}$	yield strain of steel
θ	the angle between deformed upper flange and horizontal line
σ_c	compressive stress in the compressive strips
σ_t	tensile stress in the tensile strips
τ	average shear stress along the cross section

temperatures so far, the analytical model is based on the classic "tension field theory" of plate girders. This has been adapted to the structural response of Class 1 beams, which can form a plastic hinge with the rotation capacity required by plastic analysis, without reduction of its resistance according to Eurocode 3 Part 1-1 [6]. A brief review of the development of tension field theory is presented in the next section.

Fig. 1. Shear buckling phenomenon in Cardington fire test [5].

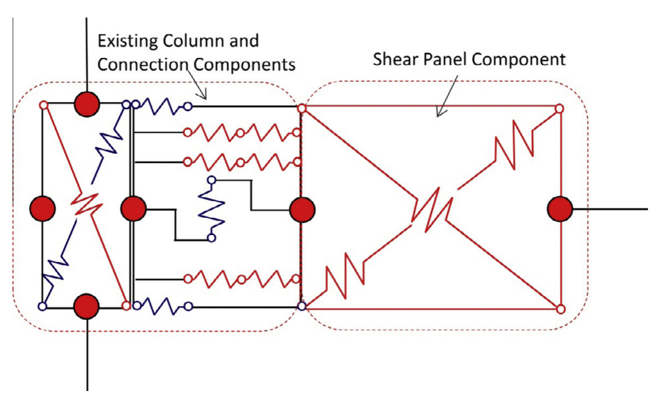

Fig. 2. Component-based model with the new shear-panel component.

2. Brief review on the tension field theory of shear web panels

As early as 1886, the possibility of utilising the post-buckling strength of plate in bridges was considered by Wilson [7]. Later Wagner [8] presented a diagonal tension theory concerning buckling and post-buckling behaviour for aircraft structures in 1931. However, the post-buckling behaviour of beam web panels was not considered as a design concept until the 1960s; until this time the elastic buckling load was used as the only design limit