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Experimental study on local buckling of axially compressed steel stub columns at elevated temperatures

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A B S T R A C T

There are few design provisions in codes and standards on local buckling of steel columns under fire conditions. To examine the local stability of steel stub columns at elevated temperatures, 12 stub columns were tested under simultaneous application of load and fire conditions. The test variables included Grade (type) of steel, buckling resistance, temperature and load levels. During fire tests, cross sectional temperatures, axial displacement, buckling deflection, and local buckling failure modes of flange and web of stub columns were recorded at various temperatures. Data from the tests is utilized to evaluate buckling resistance of flange and web both at room and elevated temperatures by applying the ultimate strain method and curve inflexion point method. Results from fire tests are utilized to validate a finite element model developed for tracing local buckling of steel columns at elevated temperatures. Results from fire tests and finite element analysis show that failure mode of columns at room and elevated temperatures follow similar pattern but the load bearing capacity of Q460 steel columns degrade much more rapidly under fire conditions than that of Q235 steel columns. Further, Eurocode 3 provisions for local buckling produce non-conservative results in certain situations.

1. Introduction

The behavior of steel structures subjected to fire loading has become an important consideration in structural design over recent years, especially after the '9 · 11' event happened in USA, and this has attracted attention of researchers to develop methodologies for rational design of steel structures under fire conditions. In order for this research to be put into practice, design methods that can be easily applied within the broad framework of structural engineering design codes of practice are needed for structures subjected to fire [1]. The lack of design rules for local buckling of steel members at elevated temperatures (as shown in Fig. 1) as well as the effect of strength of steel on local buckling in steel columns has motivated this research.

Buckling of steel members, and in particular local buckling [2–3], has a strong influence on the behavior of steel structures, both at ambient and elevated temperatures. Under fire conditions, steel structural members heat up quickly, primarily because of their usually high section factors (ratio of sectional area to volume) and due to high thermal conductivity of steel. Simplified design rules for evaluating local buckling of thin-walled structural members are included in EN 1993-1.2 [4]. These rules are based on the ambient temperature stress-based approach and utilize the effective width method-under fire conditions. Resistance of thin-walled structural members at elevated temperatures is determined taking the effective width but incorporating the decreased load-carrying capacity arising from reduced strength of steel and local buckling of flange and web occurring at ambient temperature conditions. One of the aims of the current study is to check whether the Eurocode 3 provisions for local buckling are suitable, for different grades of steel and also at different temperatures.

The local buckling phenomenon of flanges in steel members at elevated temperatures has been studied by a few researchers. Uy and Bradford [5] studied local buckling of cold-formed steel in composite structural members at elevated temperatures by applying a finite strip method. Feng et al. [6] carried out a numerical study on cold-formed rectangular hollow section columns to evaluate the sensitivity of the column capacity to initial imperfections and to the stress–strain curve of steel. Yang et al. [7–8], based on fire test on a series of H section steel columns, reported that steel columns, with slender sections, can attain their yield capacity even at elevated temperature. Knobloch et al. [2] developed a strain-based approach for evaluating local buckling of steel sections subjected to fire conditions by taking into consideration the effective width of stiffened and unstiffened elements of thin-walled cross-sections. Bradford et al. [5] modified the finite strip model and studied the local buckling of flange in thin-walled steel

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Nomenclature

h_0	height of web
t_w	thickness of web
b	outstanding width of flange
t_f	thickness of flange
ρ	reduction factor of gross cross-sectional area
$\overline{b}$	height of web or outstand width of flange for H shaped sections
t	thickness of web or flange
k_σ	buckling factor corresponding to the stress ratio and boundary conditions

beam at elevated temperature. And they also studied the local buckling of web under flexure, shear and compression action [9]. Quiel et al. [10] presented simple continuous equations to calculate the ultimate strength of thin steel plates at elevated temperature by using a stress-based approach. Most of the reported studies on local buckling of thin-walled steel members are based on finite element analyses or analytical approaches, and few full-scale experiments under fire conditions are reported in the literature. Thus there is no proper validation of the current design rules for local buckling of thin-walled sections at elevated temperatures.

This paper presents an experimental investigation on the local buckling phenomenon of flange and web of thin-walled cross sections at room and elevated temperatures. Two types of steel columns, made of mild Q235 steel and high strength Q460 steel, widely used in building structures in Far East, were tested under fire conditions to evaluate local buckling of web and flanges. Data from the tests is utilized to validate a finite element model developed to evaluate local buckling in steel columns. The buckling resistance and ultimate load bearing capacity of columns at elevated temperatures are also compared with EC3 provisions.

2. Experimental investigation

The experiments consisted of testing 12 stub columns under simultaneous application of load and temperature conditions. The test variables included Grade of steel, buckling resistance of columns and temperature.

2.1. Test set-up and instrumentation

Ambient temperature tests: The ambient temperature tests were conducted in a purpose built test set-up, shown in Fig. 2(a). The test specimen (column) was inserted into reaction frame and one hydraulic jack was connected to installing platform to apply axial load on the specimen. Detailed measurements of deformation and strain were taken at a number of locations of the panel. As shown in Fig. 2(a), 4 linear variable differential transformers (LVDT) were used to record the vertical compression of the specimen. 3 or 6 LVDTs were used to measure the horizontal buckling displacement of web or flange. 10 strain gauges were mounted on flange and web at mid-section of the column to record the strain when the column gets compressed.

Fig. 1. Local buckling in a typical steel columns.

Fire tests: To undertake fire tests, the tested specimen described above and shown in Fig. 2(a) was placed into a big furnace that measures $\phi 600 \times 900\ \text{mm}^3$ internally and about $\phi 1200 \times 1500\ \text{mm}^3$ externally. The furnace can generate temperatures up to 1200 °C,with an accuracy of ± 0.5 °C through a temperature control system. In addition to the same 4 LVDTs placed to measure the displacement of axial compression in ambient temperatures tests, additional 3 LVDTs placed outside of the furnace were used to measure the horizontal buckling displacement of web or flange through three holes on the wall of the furnace. A total of 6 thermocouples were placed within each specimen to obtain temperature distribution at three different locations as shown in Fig. 2(b).

2.2. Specimen details

In order to examine the local buckling behavior and exclude the contribution from the global behavior, all test specimens were designed as stub columns according to SSRC Technical Memorandum no. 3 [11]. The overall length of each specimen was 1700 mm. Totally 12 specimens were fabricated, and six of them were made of Q235 steel (nominal yield strength is 235 MPa) and the other six were made of Q460 steel (nominal yield strength is 460 MPa). In each type of steel columns, three columns were designed for testing flange buckling and three columns were prepared to test web buckling.

H shaped welded steel column comprises of three steel plates, namely top and bottom flanges and web. When the column is subjected to compression, all the three plates get compressed. The buckling resistance of steel plate is governed by width-to-thickness ratio, boundary conditions and mechanical properties of steel. The local buckling of web occurs prior to that of flange when the following condition is satisfied [12]:

$$\frac{h_0/t_w}{b/t_f} > 3.07 \tag{1}$$

where h_0 is the height of web; t_w is the thickness of web; b is the outstanding width of flange and t_f is the thickness of flange. When the above condition is not satisfied, local buckling of flange occurs prior to that in web.

Burgess and Plank [13] have shown that geometric imperfections in steel columns at elevated temperature to be same as that at room temperature. All tested columns were checked for imperfections by the steel fabrication company to ensure the columns meet the code requirements for acceptance [14]. It was also found that the initial imperfection is smaller than 1% of outstanding width of flange and web.

Information on the specimens are tabulated in Table 1. For each type of buckling, 3 temperatures were considered, namely ambient temperature, 450 °C and 650 °C. Four stiffeners on both