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Numerical analysis of thin steel plates loaded in shear at non-uniform elevated temperatures

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ABSTRACT

The shear resistance of thin steel plate consists of three components: shear buckling, tension field and contribution of the flanges. This paper considers the first two. The shear resistance of steel plate at elevated temperatures is known when temperature is constant. However, in many practical applications temperature distribution across the plate is non-uniform. This paper presents the results of a finite element (FE) analysis of 12 steel plates at 18 non-uniform temperatures. A design method for predicting the shear resistance of thin steel plate at non-uniform elevated temperatures is also proposed. The basic idea of the method is to reduce the design strength of the plate based on so-called reference temperature, which is hotter than the average temperature but colder than the maximum temperature of the plate.

1. Introduction

Many structures in buildings, industrial products, ships, trains and other vehicles are composed of thin steel plates. A thin plate is capable of carrying considerable additional shear load in excess of its elastic shear buckling load.

The post-buckling phase was discussed by Rode as early as 1916, when he adopted a tension field width of 50 times the thickness of the plate [1]. In the 1930s Wagner presented a pure tension field theory for aircraft structures with very thin web panels [2]. However, post-buckling strength was not used directly in the design of plate girders in civil engineering. Elastic buckling remained the basis for their design until the 1960s.

The American Institute of Steel Construction (AISC) was the first to include post-buckling shear strength in its specifications [3] as a result of studies conducted by Basler and Thurlimann [4–6]. Their theory was followed by numerous failure theories.

In 1978 Rockey and his co-workers proposed a theory, which assumed that flanges could develop plastic hinges after tension field action [7]. Their method was eventually included in the British Standard [8]. In the Eurocodes [9] the shear resistance of slender plates is based on the rotated stress field theory as proposed by Höglund [10].

Fire resistance of structures has received much more attention since the collapse of the World Trade Center towers. Shear resistance of plates at elevated temperatures has also been studied. Test results and theories are available for cases where temperature across the entire plate is uniform [11–13]. However, in reality temperatures at opposite edges of the plate may vary when it is a part of a larger structure. Webs of slim floor beams in buildings and all-metal sandwich panels mainly used in ships in fire are examples of possible applications of this study. In Nordic countries it is very typical to use so-called hat beams (a welded beam with two webs, a cavity in the middle and a wide lower flange to support concrete slab) e.g. in office buildings because they enable the use of relatively slim floors. In these cases the web(s) can buckle towards the cavity. No test results or theories on shear resistance at non-uniform elevated temperatures could be found. The safe solution is to always use the maximum temperature of the plate in reducing material properties.

2. Scope of the work

The shear resistance of a thin plate involves three phases both at ambient and elevated temperatures as the load increases [12]: the buckling phase, the post-buckling phase and yielding of supporting structures. The first two are considered in this study. The buckling phase was studied first. This study [14] showed that when the reduction factor based on the average temperature of the plate at non-uniform elevated temperatures is used, the results are almost always clearly on the unsafe side compared to FE analysis. It should be noted that the results of this study have been first published in references [15–17]. This paper collects the most important results received in study completed in 2012 in Tampere University of Technology, Research Centre of Metal Structures.

Twelve steel plates with the properties given in Table 1 were analysed numerically. The boundary conditions were assumed to be either simply supported or clamped. Plates PL2 and PL7 are so-called benchmark cases for which test results at ambient and at uniform elevated temperatures are available [11]. More specific

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Table 1
Properties of the plates.

Plate	Boundaries	Geometry [mm] ($a \times h \times t$)	f_y [N/mm^2]	E [N/mm^2]	h/t	λ
PL1	Simple	305 × 305 × 1	355	210000	305	3.28
PL2	Simple	305 × 305 × 1.5	332	200000	203	2.17
PL3	Simple	305 × 305 × 2	355	210000	153	1.64
PL4	Simple	305 × 305 × 1	235	210000	305	2.67
PL5	Simple	305 × 305 × 1.5	235	210000	203	1.78
PL6	Clamped	305 × 305 × 1	355	210000	305	2.63
PL7	Clamped	305 × 305 × 1.5	332	200000	203	1.73
PL8	Clamped	305 × 305 × 1	235	210000	305	2.14
PL9	Simple	305 × 305 × 2	460	210000	153	1.87
PL10	Simple	152.5 × 305 × 1	235	210000	305	1.62
PL11	Simple	610 × 305 × 1	235	210000	305	3.24
PL12	Simple	915 × 305 × 1	235	210000	305	3.39

analyses were performed in the case of these plates. In these two cases, whole beams (Fig. 1) were also modelled in order to validate the numerical model. The plates were selected so that at ambient temperature elastic shear buckling resistance, V_{cr} according to classical Eqs. (1) and (2) was smaller than the ultimate shear resistance according to EN 1993-1-5 [9] to ensure that the post-buckling phase occurred.

$$V_{cr} = ht\tau_{cr} \tag{1}$$

where

- h is the height of the plate,
- t is the thickness of the plate,
- τ_{cr} is the critical shear stress.

Critical shear stress, τ_{cr}, is determined from Eq. (2) [18]:

$$\tau_{cr} = k_\tau \frac{\pi^2 E}{12(1-\upsilon^2)} \left(\frac{t}{h}\right)^2 \tag{2}$$

where

- E is the elastic modulus,
- ν is Poisson's ratio (0.3),
- k_τ is the shear buckling coefficient.

The shear buckling coefficient, k_τ, is obtained for simply supported plates as shown in Eqs. (3) and (4) [18].

$$k_\tau = 5.34 + 4\left(\frac{h}{a}\right)^2 \quad \text{for } a \geq h, \tag{3}$$

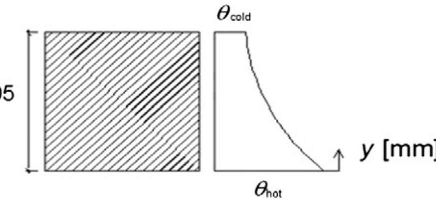

Fig. 2. Temperature distributions.

$$k_\tau = 5.34\left(\frac{h}{a}\right)^2 + 4 \quad \text{for a} \leq \text{h}. \tag{4}$$

where

- a is the distance between the stiffeners of the plate.

Historically, shear buckling in steel plates has been determined by assuming that web panels are simply supported at the joint between the supporting structure and the web. This assumption has turned out to be conservative since the geometrical properties of the structure modify the boundary conditions and influence the behaviour in shear. It has been known for long that the boundary conditions of a web plate are somewhere between simply supported and clamped, but it has not been taken into account in design, mainly due to the lack of means to evaluate it in rational manner [19]. In this study, boundary conditions were assumed to be either simply supported or clamped. The shear buckling coefficient for clamped plates can be derived from Eqs. (5) and (6) [18].

$$k_\tau = 8.98 + 5.6\left(\frac{h}{a}\right)^2 \quad \text{for } a \geq h, \tag{5}$$

$$k_\tau = 8.98\left(\frac{h}{a}\right)^2 + 5.6 \quad \text{for } a \leq h, \tag{6}$$

Table 1 also shows the slenderness parameters, λ, for each plate, which are calculated as shown in Eq. (7) [9]:

$$\lambda = 0.76\sqrt{\frac{f_y}{\tau_{cr}}} \tag{7}$$

where

- f_y is the yield stress at ambient temperature.

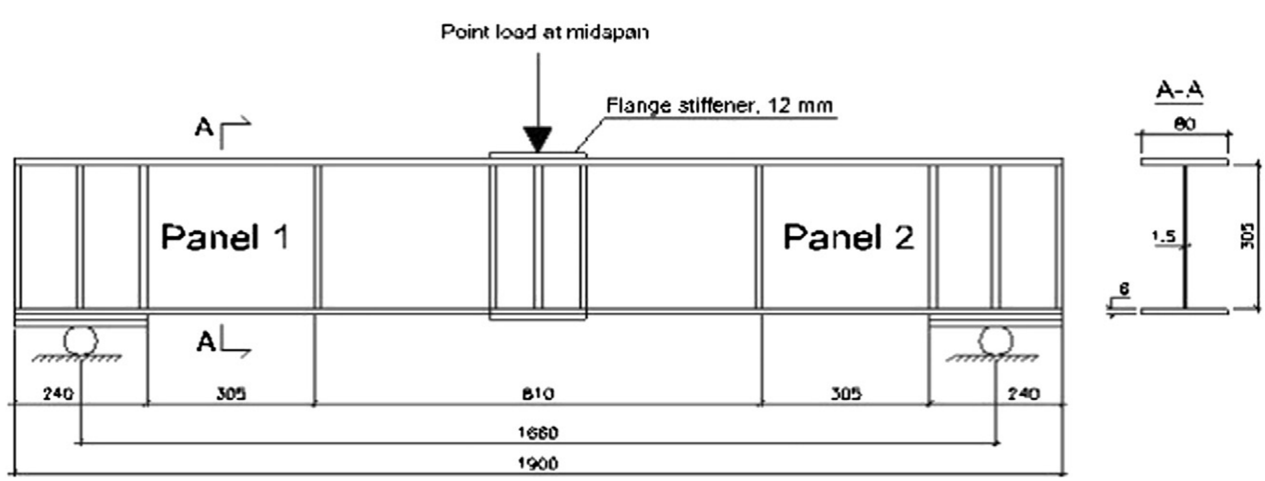

Fig. 1. Test girder configuration as reported in [11].