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Author[s]: Scandella, Claudio; Knobloch, Markus; Neuenschwander, Martin; Fontana, Mario

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FIRE RESISTANCE OF STEEL PLATE GIRDERS

Claudio Scandella, Markus Knobloch, Martin Neuenschwander, Mario Fontana
ETH Zurich, Institute of Structural Engineering, 8093 Zurich, Switzerland
scandella@ibk.baug.ethz.ch, knobloch@ibk.baug.ethz.ch, neuenschwander@ibk.baug.ethz.ch, fontana@ibk.baug.ethz.ch

INTRODUCTION

Shear buckling has a strong influence on the load-carrying behavior of steel plate girders and is an important factor for the safe and economical design of buildings and bridges. Stability effects have a strong influence on the structural behavior and the resistance of steel structures subjected to fire. Therefore, analytical models for fire design should pay more attention to instabilities than models for ambient temperature design. Due to elevated temperatures in fire, the strength and stiffness of steel decreases rapidly, and the stress-strain relationship becomes distinctly nonlinear. These changes in material behavior cause an increased influence of stability phenomena. Furthermore, compatibility stresses caused by constrained thermal expansion can have a strong influence on the stability behavior of steel members. During a fire, thin webs heat up faster than flanges of common I-shaped steel cross-sections, which lead to compressive stresses in the web. Theses compressive stresses can aggregate or even cause shear buckling. Additionally, steel plate girders, which develop local flange buckling due to bending moments at ambient temperature may fail due to shear web buckling under fire conditions [1, 2].

Three basic analytical models, namely the Cardiff model [3], the Basler model [4] and the rotated stress field theory adopted in EN 1993-1-5 [5] are commonly used for the ambient temperature design of steel plate girders subjected to shear. It is not yet finally investigated whether common analytical models for ambient temperature design [3-5] can directly be adapted to fire design [6-8].

The paper first presents a numerical analysis on the fire behavior of steel plate girders. Then, the results of the numerical study are used to perform a comparative study with fire-adapted common analytical models used for the design of steel plate girders.

1 NUMERICAL ANALYSIS OF STEEL PLATE GIRDERS IN FIRE

The fundamental fire behavior of steel plate girders is numerically analyzed using a finite element model for four-point bending test simulations. In a parametric study, the influence of the aspect ratio, depth-to-thickness ratio and load ratio (with respect to ultimate load at ambient temperature) on the fire resistance and the failure mode is analyzed with a study of over 100 numerical fire simulations.

1.1 Numerical Model

A three-dimensional FEM-model for transversely stiffened steel plate girders loaded in a four-point bending configuration was developed using the implicit Finite-Element-Code ABAQUSStandard. The web and the flanges composing the I-shaped cross-sections of the girders were modelled as plates using four-node shell-elements with a reduced integration scheme (S4R elements in the ABAQUSStandard element-library). The girders’ longitudinal axis coincides with the global X-axis and their vertical deflection direction with the global Z-axis as depicted in Fig. 1a. An initial geometric imperfection shaped according to the first eigenmode is applied with an amplitude of h/200, with h being the clear depth of the web plates between the two flanges. The boundary conditions of the simply supported beam are implemented in the reference nodes of analytical rigid surfaces modelling stiff plates of rocker-bearing type supports. Lateral buckling is prevented by constraining accordingly the displacement of points of the upper flange equidistantly at s/15, with s being the span of the girders. The loads are applied as concentrated loads in the reference nodes of analytical rigid surfaces representing stiff loading plates. The welded joints between the different plates forming the girders (flanges, webs and stiffeners) are modelled with multi-point constraints enforcing complete coupling of all global displacements and rotations at the contacting nodes. The classical von Mises-plasticity
Fig. 1. Setup of the numerical simulation and results of girder G48 in fire
model is used for the material behavior of the steel, considering temperature-dependence of the stress-strain-relationship and the thermal expansion according to EN 1993-1-2 [9]. The numerical model is validated with results of an extensive experimental study on several steel plate girders at ambient temperature performed by Basler et al. [10]. Thereby an agreement of the calculated ultimate load with the experimental data within a range of 8% could be established [1].

1.2 Simulation procedure
The simulation procedure assessing numerically the fire resistance of preloaded steel plate girders is divided into four different calculation steps consisting of: (1) An eigenmode analysis in order to determine the shape of the initial geometric imperfection, (2) an ultimate load analysis at ambient temperature, (3) a heat transfer analysis in order to determine the varying temperature field in time of the girders when subjected to ISO 834 standard fire and (4) a fire resistance analysis of the preloaded girder by applying the varying temperature field in time in a static analysis until failure.

1.3 Fire behavior of steel plate girders
The numerical results are used to analyze the fundamental structural fire behavior of preloaded steel plate girders. The behavior of girders subjected to fire, which fail due to shear buckling, can be divided into two phases: (1) A pre-critical range starting with the beginning of the fire exposure and ending when web buckling has fully developed. The latter occurring due to the increase of compressive stresses in the web as a result of constrained thermal strains. (2) A post-critical range characterized at the beginning by constant rates of the girders’ deformations (midspan deflection, out-of-plane deflection of the web) until a runaway failure sets in at the end.

The structural behavior of steel plate girders in fire is analyzed in detail using the results of girder G48 as an example. This girder featured a change in failure mode from bending failure at ambient temperature (Fig. 2a) to shear failure in fire (Fig. 2b). The geometry and geometrical slenderness ratios of this girder are listed in Table 1 together with the prevailing load ratio (with respect to ultimate load at ambient temperature) of, $\mu_R = 0.6$, and the fire resistance reached at, $t_R = 856 \text{ s}$, under ISO 834 standard fire exposure.

![Fig. 2. Failure of girder G48 at ambient temperature (a, scale factor = 2.0) and in fire (b, scale factor = 5.0)](image)

Fig. 1 outlines the setup of the numerical simulation and the deformations calculated during the standard fire exposure starting at, $t = 0 \text{ s}$. The midspan deflection, $w$, and its rate, $\dot{w}$, (Fig. 1d), were selected as quantities detecting global failure when increasing unboundedly. Additionally an unbounded increase of: (1) the out-of-plane deflection, $v$, of the web in cross-section A-A and its rate, $\dot{v}$, (Fig. 1e), would indicate shear buckling failure, whereas (2) the rotation, $\phi$, of the upper flange in cross-section B-B and its rate, $\dot{\phi}$, (Fig. 1f), would indicate bending failure. At the beginning of the
fire exposure the girder shows a midspan deflection of, $w = 36.5 \, \text{mm}$, due to the preloading of $F_R = 176.4 \, \text{kN}$. Fig. 1g shows the temperature-time-curves at different points in the girder’s cross-section A-A (Fig. 1b) and indicates that the thin web ($t_w = 7 \, \text{mm}$) heats up faster than the relatively massive flanges ($t_f = 20 \, \text{mm}$). However, due to the compatibility condition of the cross-section, the web is partially constrained in its thermal expansion by the flanges. Therefore a self-equilibrating stress state arises within the cross-section leading to additional thermally-induced compressive stresses in the web ($\sigma_3$ and $\sigma_4$, Fig. 1h) and tension stresses in the flanges ($\sigma_1$ and $\sigma_2$, Fig. 1h).

The increase in compressive stresses in the web is limited by the onset of local web buckling starting at $t_1 = 51 \, \text{s}$, of fire exposure (Fig. 1e). Simultaneously, bending stresses develop in the web lowering the compressive stresses on the convex side of the buckle, $\sigma_3$, and increasing them on the concave side, $\sigma_4$ (Fig. 1h). Buckling of the web is accomplished at the point in time, $t_2 = 100 \, \text{s}$, when the out-of-plane deflection, $v$, ceases to increase (Fig. 1e). From this moment onwards the girder is in the post-critical range and its midspan deflection increases with an almost constant rate, $\dot{w}$, until the onset of a runaway failure at $t_3 = 700 \, \text{s}$. Failure of the girder finally occurs at, $t_R = 856 \, \text{s}$, when both the rate of the midspan deflection, $\dot{w}$, and the rate of the out-of-plane deflection, $\dot{v}$, approach a vertical asymptote. Simultaneously, the rotation of the upper flange, $\varphi$, however, remains bounded indicating thus shear failure.

Additional numerical studies were performed in order to check whether the thermally-induced compressive stresses in the web caused the change in failure mode from a bending failure at ambient temperature to a shear failure in fire. Therefore, the entire girder’s cross-section was heated up uniformly with a moderate constant rate of 7.3 °C/min. Fig. 3 shows girder G48 in its ultimate state when heated up uniformly. In this case, there is no temperature difference between the flanges and the web. Hence, the lack of additional compressive stresses in the web prevents the latter from buckling, as can be seen in Fig. 3 (in opposition to Fig. 2b).

**Fig. 3. Failure of girder G48 when heated up uniformly (scale factor = 5.0)**

**Fig. 4. Results of girder G48 when heated up uniformly**
Fig. 4 shows the results of girder G48 when heated up uniformly. After a fire exposure with a constant heating rate of 7.3 °C/min at, \( t = 4162 \) s, the fire resistance, \( t_f \), is reached when the curves of, \( w \), and, \( \dot{w} \), approach a vertical asymptote. At this point in time, the curves of the rotation of the upper flange, \( \phi_U \), and its rate, \( \dot{\phi}_U \), approach a vertical asymptote too, indicating a bending failure. This is in opposition to the change in failure mode of girder G48 when subjected to ISO 834 standard fire. Thus, the change in failure mode – from a bending failure at ambient temperature to a shear failure in fire – is induced by compatibility stresses due to a temperature difference between the flanges and the web.

1.4 Parametric Study
In a numerical parametric study, the influence of different parameters on the fire resistance and the failure mode was comprehensively analyzed with 105 numerical fire simulations. The parameters varied included: (1) The aspect ratio \( \alpha \) of the web plate, (2) the depth-to-thickness ratio \( \beta \) of the web plate and (3) the load ratio \( \mu_R \) (with respect to ultimate load at ambient temperature). Table 1 exemplarily shows the geometry, input parameters, fire resistance and failure mode of four selected girders. The results of the numerical study show that some steel plate girders which fail due to bending at ambient temperature, develop shear failure in fire (e.g. girders G48 and G88). Increasing the load ratio led to a bending failure even under fire conditions (e.g. girder G50). Steel girders with smaller depth-to-thickness ratios of the web plate also failed due to bending in fire (e.g. girder G53). Steel plate girders with a high aspect ratio, however, failed due to the development of shear buckling even for small depth-to-thickness ratios of the webs.

| Table 1. Geometry, input parameters and numerical results of selected girders |
|-----------------------------|-----------------------------|-----------------------------|-----------------------------|-----------------------------|-----------------------------|-----------------------------|-----------------------------|-----------------------------|-----------------------------|-----------------------------|-----------------------------|-----------------------------|
| Girder | Span of girder | Clear depth | of web plate | Width of flanges | Thickness of flanges | Yield stress of web plate | Yield stress at temperature | Ultimate load | at ambient temperature | Aspect ratio | of web plate | Depth-to-thickness | ratio of web plate | Load ratio | in fire | Fire resistance | Failure mode | Failure mode | Change in failure mode? |
| G48 | 177250 | 1'150 | 250 | 20 | 235 | 7 | 1'150 | 294 | \( \alpha = \frac{a}{h} \) | 1.0 | 164 | 0.6 | 856 | Bending | YES |
| G50 | 177250 | 1'150 | 250 | 20 | 235 | 7 | 1'150 | 294 | \( \beta = \frac{h}{\tau_w} \) | 1.0 | 164 | 0.8 | 680 | Bending | NO |
| G53 | 177250 | 1'150 | 250 | 20 | 235 | 8 | 1'150 | 306 | \( \mu_R = \frac{F_R}{F_{u,amb}} \) | 1.0 | 144 | 0.6 | 870 | Bending | NO |
| G88 | 177250 | 1'150 | 250 | 20 | 235 | 8 | 2875 | 306 | \( \tau_f \) | 2.5 | 144 | 0.6 | 832 | Bending | YES |

2 COMPARATIVE STUDY
Common analytical models according to “Eurocode 3” [5], “Basler” [4] and “Cardiff” [3] – originally developed to determine the shear capacity of steel plate girders at ambient temperature – were adapted for fire design using temperature-dependent material properties for the Elastic modulus and yield strength according to EN 1993-1-2 [9]. The temperatures of the web and the flanges, needed to calculate the reduction-factors \( k_E \) (Elastic modulus) and \( k_Y \) (yield strength), were taken with their maximal value from the numerical simulation at failure time \( t_f \).

For ambient temperature design, all three analytical models lead to suitable results. For fire design, however, the simplified analytical models neglecting thermal strains lead to inappropriate results for some cases. Fig. 5 compares the shear resistance in fire according to “Eurocode 3” (Fig. 5a) and “Cardiff” (Fig. 5b) to the numerical results. A simplified analytical model according to “Eurocode 3” adapted only for temperature related reduction of the Elastic modulus and the yield strength leads to higher shear capacities than the numerical results for girders with high aspect ratios (\( \alpha = 2.5 \)). For girders with small aspect ratios (\( \alpha = 1.0 \)), the simplified analytical model leads to lower shear capacities than the numerical results (Fig. 5a). On the other hand, a simplified analytical model according to “Cardiff” adapted only for temperature dependent reduction of the Elastic modulus and the yield strength slightly overestimates the shear capacities for almost all girders (Fig. 5b), regardless of whether the aspect ratio is small (\( \alpha = 1.0 \)) or high (\( \alpha = 2.5 \)).
3 CONCLUSIONS

The paper presented a detailed numerical study on the ambient temperature and fire behavior of steel plate girders. It shows that constrained thermal strains have a marked influence on the shear capacity of steel members under fire conditions. Steel plate girders which fail due to bending at ambient temperature may develop shear web buckling in fire. A comparative study reveals that commonly simplified analytical models adopted from ambient temperature design using only hot material properties for the Elastic modulus and the yield strength are not suitable and can lead to unconservative design results for standard fire situations.

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ETH Zurich, Institute of Structural Engineering, 8093 Zurich, Switzerland
scandella@ibk.baug.ethz.ch, knobloch@ibk.baug.ethz.ch, neuenschwander@ibk.baug.ethz.ch, fontana@ibk.baug.ethz.ch

KEYWORDS: steel plate girders; fire resistance; elevated temperatures; shear buckling

ABSTRACT

Shear buckling has a strong influence on the load-carrying behavior of steel plate girders. Due to elevated temperatures in fire, the strength and stiffness of steel decreases rapidly, and the stress-strain-relationship becomes distinctly nonlinear. These changes in material behavior cause an increased influence of stability phenomena. Furthermore, compatibility stresses caused by constrained thermal expansion can have a strong influence on the stability behavior of steel members. During a fire, thin webs heat up faster than flanges of common I-shaped steel cross-sections, which lead to compressive stresses in the web. These compressive stresses can aggregate or even cause shear buckling.

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