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To Professor Andrzej GARSTECKI
on His 70 birthday
and a long professional work with us
and for us

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DEFORMATIONS AND DECREASE OF LOAD-CARRYING CAPACITY OF WELDED STRUCTURES DUE TO RESIDUAL WELDING STRESSES

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Residual deflection due to restrained shrinkage of longitudinal welds is calculated in the case of an asymmetric I-section crane runway girder. The residual welding compression stresses decrease the load-carrying capacity of a welded I-section compression strut in a significant measure. The effective width of a welded stiffened plate should be calculated considering residual welding stresses. The fatigue strength of welded joints is influenced by residual welding stresses significantly. These effects are illustrated by numerical examples.

Keywords: welded structures, residual welding stresses, distortions, stability, fatigue

1. INTRODUCTION

Modern metal structures are characterized by welding of plated components. The best way to decrease their mass and cost is the decrease of plate thicknesses. This trend results in thin-walled structures. Then the designer faces the following special problems:

(a) high residual welding stresses and distortions occur due to restrained shrinkage of welds,
(b) the load-carrying capacity is influenced by instability phenomena such as overall and local buckling,

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(c) open-section rods are sensitive to torsion,
(d) vibration and noise can be high,
(e) stiffening should be used against buckling and vibration, which increases the cost (Farkas & Jármai 2003),
(f) high stress concentrations initiate fatigue cracks and failure.

To avoid these disadvantages the effect of residual welding stresses and distortions should be taken into account. The role of these residual stresses and distortions is investigated in the present paper for some structural parts.

2. RESIDUAL DEFLECTION OF AN ASYMMETRIC WELDED CRANE RUNWAY GIRDER

The investigated crane runway girder (Fig.1) (Lequien 1998) is constructed with the following welds: group of upper welds No.1: double bevel butt (V) weld connecting the upper flange with the web and two single bevel butt (1/2V) welds joining the rolled L-profiles to the upper flange, lower welds No.2: double fillet weld. Welds connecting the rail and transverse welds joining the vertical stiffeners are neglected.

For the calculation of residual deflection the Okerblom-method is used, which has been adopted by the authors (Okerblom et al. 1963, Farkas & Jármai 1997, 1998).

The specific heat induced by welding can be calculated as follows:

- for butt welds QT (J/mm) = $60.7Aw$
- for GMAW-C (Gas Metal Arc Welding with CO2) fillet welds QT = $78.8aw2$
- for SAW (Submerged Arc Welding) fillet welds $QT = 59.5aw2$,
  where A_W is the cross-section area of a butt weld (mm^2), a_w is the fillet weld size (mm).

The specific strain in the gravity center fiber for steel sections is expressed by

$$\varepsilon_G = 0.844x10^{-3} Q_T / A \tag{1}$$

and the curvature

$$C = 0.844x10^{-3} Q_T y_T / I_x, \tag{2}$$

where A is the cross-sectional area and I_x is the moment of inertia of the girder cross-section, y_T is the weld eccentricity.

In the case of the investigated girder the following numerical data are calculated:

$$A = 32440 \text{ mm}^2,\ I_x = 9.24266\text{x}10^9 \text{ mm}^4,$$

- for the upper double bevel butt weld $Q_T = 60.7\text{x}100 = 6070$ J/mm,
- for two single bevel butt welds $Q_T = 2\text{x}60.7\text{x}180 = 21852$ J/mm,
- for upper welds total $Q_{Ttotal} = 27922$ J/mm,
- for the lower double fillet weld welded simultaneously $Q_T = 1.5\text{x}59.5\text{x}5^2 = 2231$ J/mm (using the factor of 1.5 instead of 2).

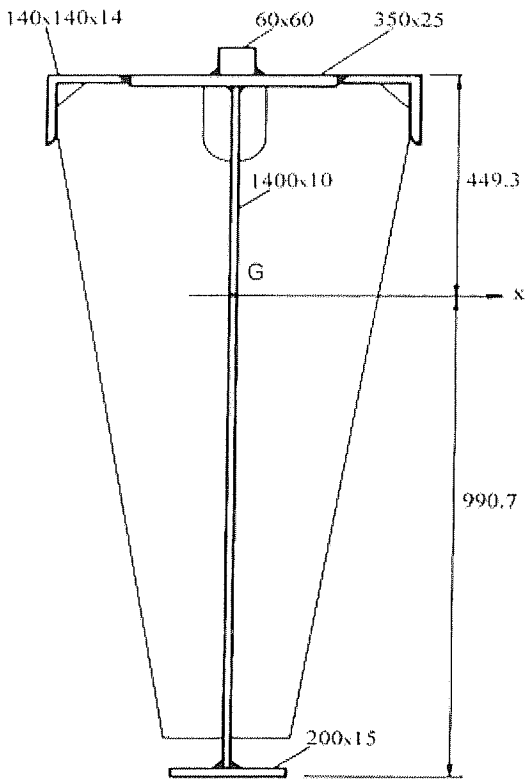

Fig. 1. Cross-section of the investigated crane runway girder

The specific strain and the curvature caused by the upper welds

$$\varepsilon_{G1} = -0.7265x10^{-3}, C_1 = 1.1317x10^{-6}$$

and those caused by lower welds

$$\varepsilon_{G2} = -0.05805x10^{-3}, C_2 = 0.2018x10^{-6}.$$

Two welding sequences are considered: upper+lower welds (1+2) or 2+1. In the case of 1+2 the effect of lower welds is calculated using a modifying factor ν expressing the prestraining effect of the upper welds

$$C_{1+2} = C_1 + \nu_{12} C_2 \tag{3}$$

$$\nu_{12} = 1 - \frac{\varepsilon_{12}}{\varepsilon_y}; \varepsilon_{12} = \varepsilon_{S1} - C_1 y_{T2} \tag{4}$$

$$\varepsilon_y \cong \frac{f_y}{E} \tag{5}$$

$$\varepsilon_{12} = 0.3947x10^{-3},\ \varepsilon_y = 235/2.1x10^5 = 1.1190x10^{-3},\ \nu_{12} = 0.6473,$$

$$C_{1+2} = 1.0011\text{x}10^{-6}.$$

For a simply supported girder of span length $L = 10$ m the deflection is

$$f_{1+2} = \frac{C_{1+2} L^2}{8} = 12.5 \text{ mm.} \tag{6}$$

Using similar formulae one obtains for the welding sequence 2 + 1

$$f_{2+1} = 11.2 \text{ mm.}$$

Since for crane runway girders the allowable residual deflection is $L/1000 = 10$ mm, the investigated girder does not fulfil the residual deflection requirement.

Note that $f_{2+1} < f_{1+2}$. It results from the larger prestraining effect of lower welds.

3. LOAD CARRYING CAPACITY OF A WELDED I-SECTION COMPRESSION STRUT

The decrease of the load-carrying capacity of an I-section compression strut due to initial imperfection (a_0) and compression residual welding stress (σ_n)

is shown by a numerical example. The distribution of the residual welding stresses can be seen in Figure 2.

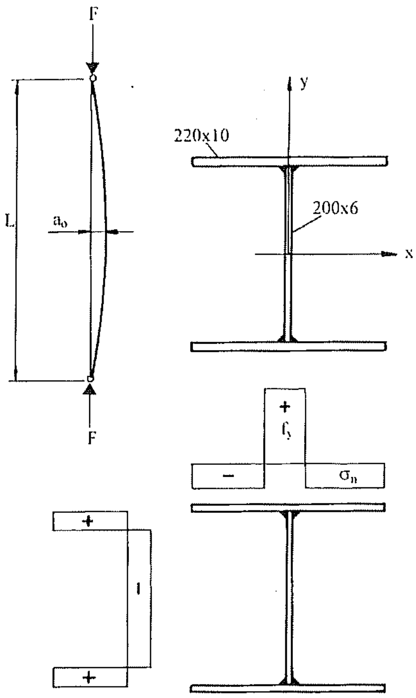

Fig. 2. Welded I-section, compression strut and the distribution of residual welding stresses

The load-carrying capacity in the case of $a_0 \neq 0, \sigma_n = 0$ (initial imperfection only) can be calculated as

$$F = \chi \frac{A f_y}{\gamma_{M1}}, \tag{7}$$

where $\gamma_{M1} = 1.1$ and

$$\chi = \frac{1}{\phi + \left(\phi^2 - \bar{\lambda}^2\right)^{1/2}}; \phi = 0.5\left(1 + \eta_b + \bar{\lambda}^2\right) \tag{8}$$

$$\bar{\lambda}_y = \frac{\lambda_y}{\lambda_E}; \lambda_y = \frac{L}{r_y}; \lambda_E = \pi\left(\frac{E}{f_y}\right)^{1/2}; r_y = \left(\frac{I_y}{A}\right)^{1/2} \tag{9}$$

$$\eta_b = \frac{a_0 A}{W_y}; a_0 = \frac{L}{1000}; A = ht_w + 2bt_f; W_y = \frac{b^2 t_f}{3}. \tag{10}$$

The following numerical data are used: L = 4000 mm, a_0 = 4 mm, A = 5600 mm^2, I_y = 17.7467x10^6 mm^4, r_y = 56.29 mm, f_y = 355 MPa, $\lambda_E = 76.41$; $\lambda = 71.06$; $\bar{\lambda} = 0.9300$; $\eta_b = 0.1388$; $\chi_y = 0.7276$,

F = 0.726x5600x355/1.1 = 1315 kN.

The effect of residual welding stresses is taken into account calculating with (Eurocode 3. 2002a).

$$\eta_b = 0.49(\bar{\lambda} - 0.2) \tag{11}$$

$\eta_b = 0.3577; \chi_y = 0.5974; F = 1080$ kN, which is 22% smaller than that without the effect of σ_n.

Let us compare the load-carrying capacity according to the Euler method, which does not take into account the two effects calculated below $(a_0 = 0,\ \sigma_n = 0)$. The critical strength according to Euler

$$\sigma_{kr} = \frac{\pi^2 E}{\lambda^2} = 410 \text{ kN}. \tag{12}$$

Since this value is larger than the yield stress, the load-carrying capacity is calculated taking χ = 1 in Eq. (7): F = 1807 kN, which is 67% larger than 1315 kN. Thus the use of Euler method gives unsafe values and is not recommended.

It should be noted that the unsafe design using Euler formulae is treated also in Farkas & Jármai 1997 and in Farkas 2005.

4. EFFECTIVE WIDTH OF A STIFFENED PLATE SUBJECT TO COMPRESSION

Figure 3 shows the residual welding stress distribution in a stiffened plate. It can be seen that the residual compressive stresses act together with the compression stresses due to external load, thus, they decrease the load-carrying capacity of the stiffened plate. Therefore the effect of residual welding stresses should be taken into account in the calculation of the compression strength of stiffened plates.

When the plate slenderness b/t is larger than the limiting slenderness (Farkas & Jármai 1997), the load-carrying capacity should be calculated considering the effective plate width. To show the effect of residual welding stresses, first we calculate without them using the Winter formula (Winter 1947)

$$\psi_1 = \frac{b_e}{b} = \frac{2}{\lambda_p} - \frac{1}{\lambda_p^2}; \lambda_p = \frac{b}{t}\left(\frac{\sigma_{\max}}{E}\right)^{1/2} \tag{13}$$

For the case of $\sigma_n \neq 0$ Faulkner has proposed a formula (Farkas & Jármai 1997)

$$\psi_2 = \frac{2}{\lambda_p} - \frac{1}{\lambda_p^2} - \frac{\sigma_n}{f_y} = \frac{2}{\lambda_p} - \frac{1}{\lambda_p^2} - \frac{6}{30\lambda_p - 6}\ . \tag{14}$$

This formula has been verified by experiments in the University of Cambridge (Bradfield 1979).

In the case of $\lambda_p = 2$, $\psi_1 = 0.75, \psi_2 = 0.6389$, the residual welding stresses cause a decrease of 17% in the effective width.

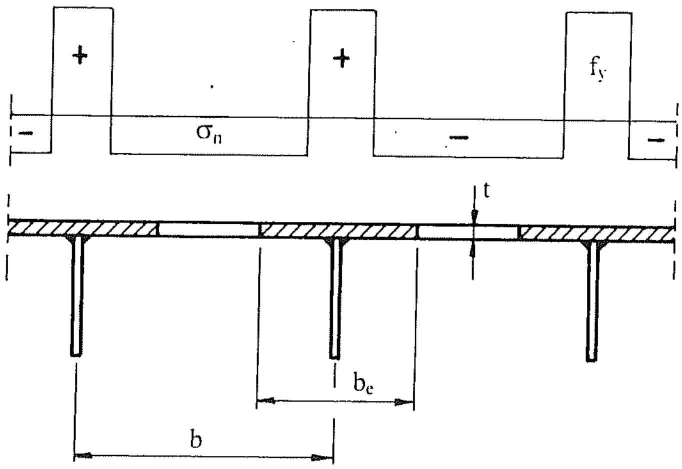

Fig. 3. Residual welding stress distribution and effective width of a stiffened plate

5. FATIGUE STRENGTH OF WELDED JOINTS

In 1960's the fatigue strength of welded joints has been determined by experiments using small specimens (Neumann 1963). Residual welding stresses did not develop in these specimens, since the shrinkage of welds has not been restrained. Later the experiments with large, real-size specimens resulted in much smaller strength due to residual stresses. Therefore, the design values should have been changed.

In thicker plates the residual stresses also occur in the thickness direction and they decrease the fatigue strength as well. Nowadays we can calculate with fatigue strengths determined statistically based on a great number of experiments.

Comparing the data given by Neumann (1963) with the newest Eurocode 3 (2002b) (EC), some significant differences can be found as follows:

(a) EC does not distinguish between fatigue strength values for steels of yield stress 235 and 355 MPa,

(b) EC calculates with fatigue stress range $\Delta\sigma = \sigma_{max} - \sigma_{min}$ and does not consider the effect of the factor $\sigma_{min} / \sigma_{max}$,

(c) in EC the fatigue stress ranges decrease for numbers of cycles larger than $N = 2x10^6$ and are constant for $N > 10^8$,

(d) Neumann has given allowable fatigue stresses, EC gives limiting fatigue stress ranges and different safety factors.

Let us illustrate the difference between fatigue strengths with a numerical example of a cruciform (or Tee-) joint (Fig. 4).

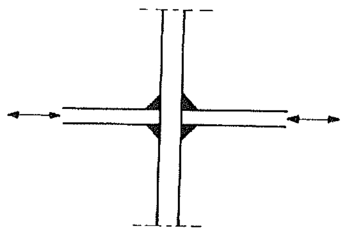

Fig. 4. A cruciform welded joint

The allowable fatigue stress for root failure in partial penetration Tee-butt joint or fillet welded joint according to Neumann (1963) is 51.5 MPa and according to EC is 36/1.25 = 28.8 MPa, which is 44% smaller. This difference is caused by residual welding stresses.

6. CONCLUSIONS

The effect of residual welding stresses on final displacements and on the load-carrying capacity is illustrated by numerical examples.

The longitudinal welding distortions in an asymmetric welded I-section crane runway girder result in the deflection, which is larger that the allowable value, thus the cross-section sizes should be changed by an optimum design procedure.

In the case of a welded I-section compression strut the realistic load-carrying capacity should be calculated according to Eurocode 3 formulae, which allow for the effect of initial imperfection and residual welding compression stresses. The Euler formula does not take into account these effects, and results in unsafe design.

The effective width of a welded stiffened plate subject to compression can be decreased due to residual welding stresses significantly.

The new Eurocode 3 gives much smaller values for fatigue strength of welded joints than the old design rules in 1960-s, since the old values have been determined by experiments with small specimens, which do not contain residual welding stresses.

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