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# CALCULATED ESTIMATION OF LOAD-CARRYING CAPACITY OF THE MAIN SPAN BEAMS OF THE E.O. PATON BRIDGE ACROSS THE DNIPRO IN KYIV BY NONDESTRUCTIVE TESTING RESULTS

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**ABSTRACT**

The paper deals with of the features of the design and loading parameters of the main longitudinal beams of the E.O. Paton Bridge across the Dnipro. It is shown that by the results of examinations, the welded joints are in a satisfactory condition, no inadmissible defects or fatigue cracks were detected in them, but rather considerable local corrosion damage appeared as a result of water flowing through the beam structures. The constructed finite element model of the main span beam was used to perform a calculated estimation of its loading and of the change of load-carrying capacity in the presence of local corrosion defects in the zone of the web and lower flange joint. Results of numerical modeling showed that the presence of these defects does not cause any significant increase of stresses in the span beam. Analysis of load-carrying capacity revealed that although the welded structure of the main span beams of the bridge has a rather high static strength margin under the action of normative distributed loading, presence of local corrosion defects of material discontinuity of rather large size considerably lowers the fatigue resistance of welded joints with detected defects that is dangerous in terms of span structure reliability and, therefore, requires immediate performance of repair operations.

**KEY WORDS:** bridge welded structure, main span beam, load-carrying capacity, corrosion defect, normative load, stressed state, numerical methods

## INTRODUCTION

The Kyiv city E.O.Paton Bridge across the Dnipro, built and commissioned in 1953, is a unique engineering structure [1]. During its construction, the then advanced welding technologies were used, in particular those of mechanized submerged-arc welding. The bridge consists of 24 span structures of 58 m length in the non-navigable portion and 87 m length in the navigable portion, its total length is 1543 m. In the cross-section each of the bridge structures has four main longitudinal I-beams. The beam consists of a vertical web 3600 mm high and 14 mm thick and girths of different thickness from 30 to 80 mm at up to 1000 mm width (Figure 1, *a*). The beam web strength is additionally ensured by longitudinal and vertical stiffeners. In the 6-span structures in the bridge navigable part the height of the web above the intermediate supports was increased up to 6200 mm, due to adding the haunches (Figure 1, *b*). The beams are made from M16S carbon steel, which corresponds to VSt3sp (killed) steel by its characteristics.

During inspection of the main longitudinal beams of the bridge in 2020, it was determined [2] that the welded joints of the main beams are in a satisfactory condition, no inadmissible defects or fatigue cracks formed in them during long-term service. However, rather significant corrosion damage developed in the beam structure as a result of seepage of water from rain and thawing snow, particularly in the lower girths, lower horizontal ribs and in the lower parts of the webs, which is also due to accumulation of debris in these areas, which retains moisture. Through-thickness corrosion damage was detected in some trusses of the main beams (Figure 2).

In order to perform the calculated estimation of the load level, as well as of the change of the load-carrying capacity of the beam span structure in the presence of local corrosion defects, a geometrical and finite element model of the main 6-span beam was constructed, in keeping with the design documentation. The model 43.5 m long consists of one span, from the haunch middle to the middle of the span structure, under the condition of symmetry at both ends (Figure 3). Boundary conditions for displacement are assigned on the haunch reference plane, the vertical component of which is equal to zero that corresponds to the conditions of the span beam resting on the bridge supports.

The bridge load-carrying structures are designed for the action of continuous loading and unfavourable combinations of temporary loading, specified in Section 2 of SBN V.1.2-15.1.2 norms [3]. Calculations are made by the limit states in compliance with the requirements of 4.3 DNB V. 2.3-22:2009 [4]. In keeping with item 8.3.1 [4], the load from motor vehicles for each lane is assumed in the form of uniformly distrib-

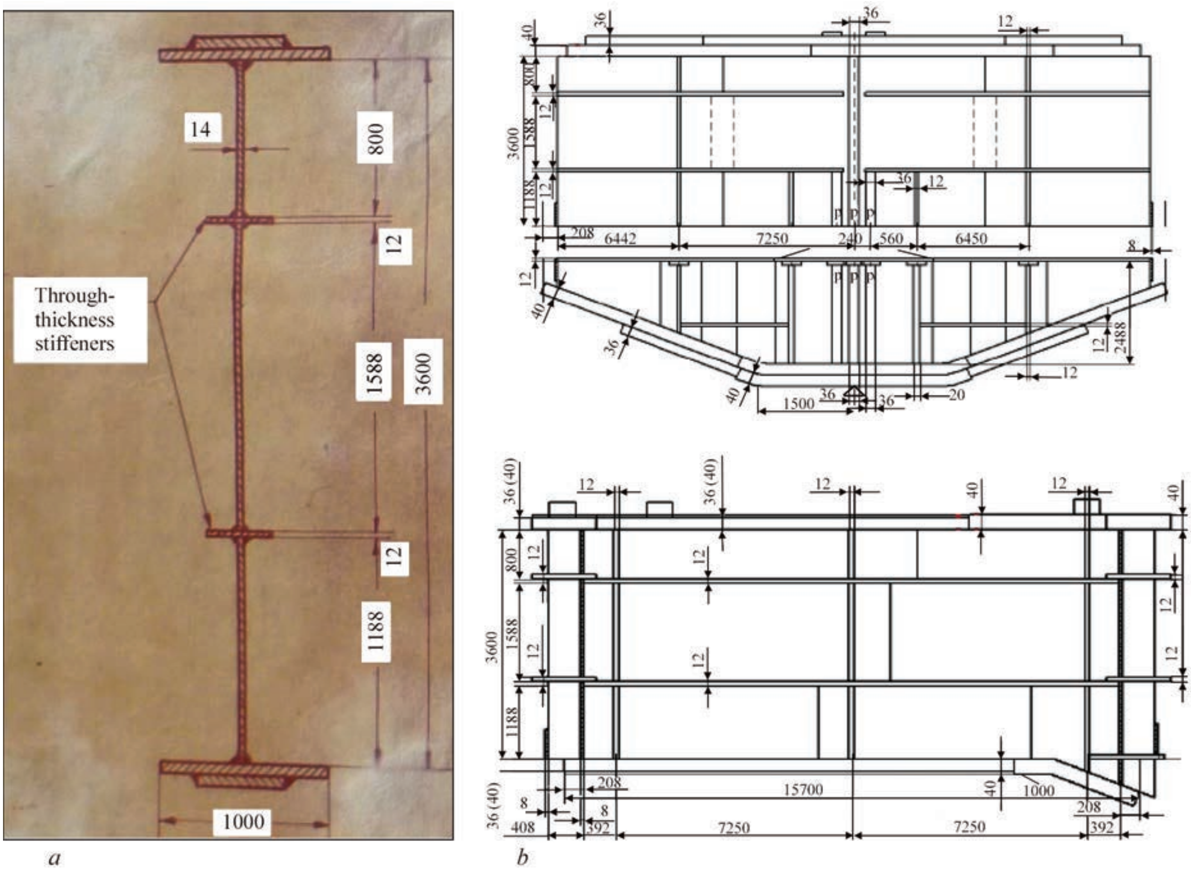



**Figure 1.** Transverse section of the main longitudinal I-beam (*a*) and drawings of fragments of the main longitudinal I-beam (*b*)

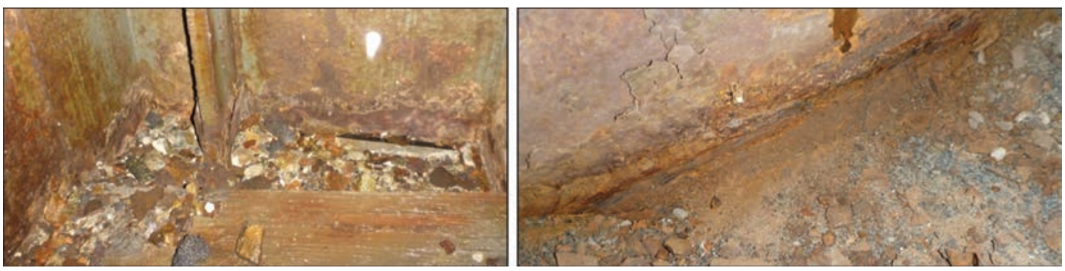


**Figure 2.** Ulcerative corrosion of the lower flange and the web in the span structure

uted load of intensity $v$ = 0.98.K kN/m (0.1·K T/m), where K is the load class specified in keeping with 8.3.2 [3]. In the highways of categories I, II, III, in the main streets of general city significance, as well as in more than 200 m long bridges and roads of categories IV and V, K= 15. The E.O. Paton bridge has 6 lanes for passage of motor vehicles, so that the intensity of uniformly distributed load on the bridge is $v$ = = 0.98·15·6 = 88.2 kN/m. Considering the presence of four main longitudinal beams in the bridge cross-section, in the developed model of 6-span beam a distributed load of magnitude $P$ = 88.2 kN/m/(4·1 m) = = 22.05 kN/m$^2$ = 0.022 MPa is applied to the upper flange of 1 m width. The model also takes into account the force of the weight of the span beam structure as a constant load.

To determine the influence of typical corrosion damage of the beam in the zone of welded joint of the lower girth and the web, through-thickness discontinuities of metal 500 mm long were introduced into the finite-element model in different points along the lower girth (Figure 4) and the web (Figure 5).

Results of numerical determination of SSS in the 6-span beam showed that the longitudinal stress component prevails as a result of the action of a distributed force (0.22 MPa), applied to the upper flange. The maximum of longitudinal stresses, located in the up-

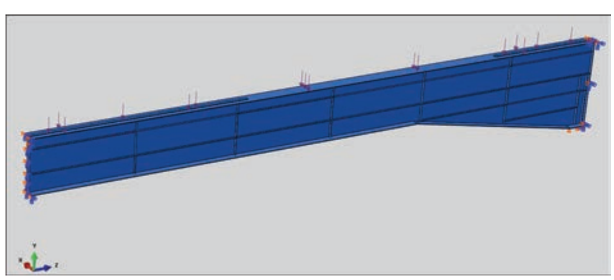


**Figure 3.** 3D model of the main 6-span beam

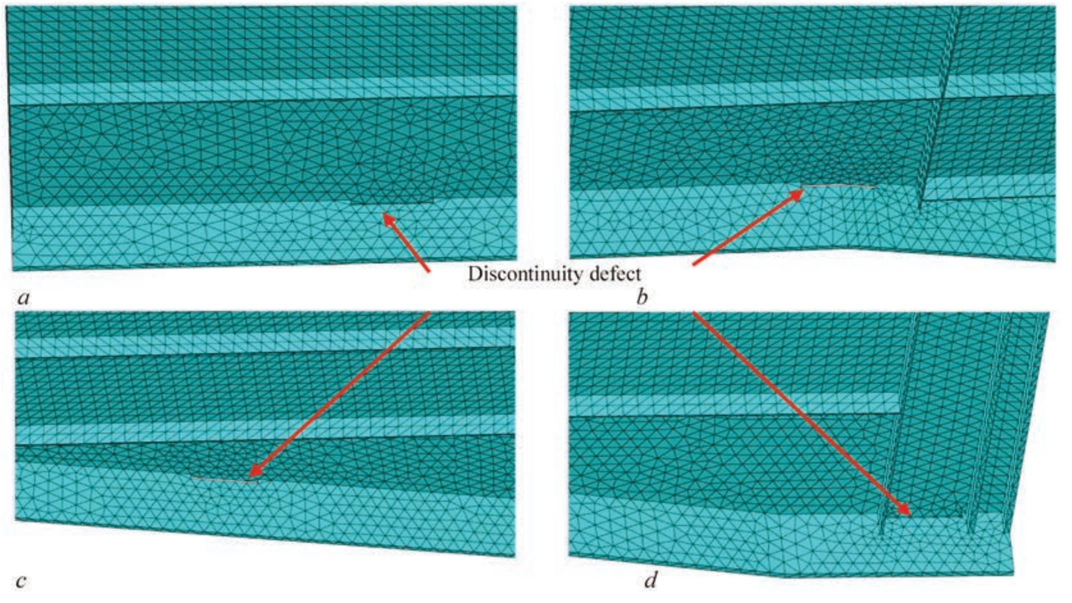



**Figure 4.** Discontinuity defects in the flange (local corrosion defect 500 mm long and 40 mm deep) in the zone of the joint of the web with the lower flange of the main 6-span beam: in the span central part (*a*); in the point of transition of the constant height beam into the haunch (through the thickness) (*b*); in the inclined part of the haunch (through the thickness) (*c*); in the central part of the haunch (*d*)

per girth above the haunch, does not exceed 65 MPa, and in the zone of the lower flange in the span central part it reaches 47 MPa (Figure 6). Presence of a local corrosion defect in this zone does not cause any significant increase of longitudinal stresses (maximum up to 50 MPa) (Figure 7).

Similar results of the influence of corrosion defects are presented also for other characteristic zones along the length of the 6-span beam, namely in the point of the haunch location and beam support, where the longitudinal tensile stresses change to compressive stresses in the lower girth zone (Figures 8–10). In the zone of transition of the constant height beam into the haunch (Figure 8), the level of maximum longitudinal stresses in the defectfree beam in the lower girth zone does not exceed 50 MPa, presence of the considered discontinuity defects causes an increase of the compressive stresses in the web up to 65 MPa (Figure 8, *a*), and in the flange in the zone of transition of constant height beam into the haunch the longitudinal compressive stresses decrease and become tensile stresses on the level of 20–37 MPa (Figure 8, *b*).

Vertical component of stresses is on a much lower level than the longitudinal component, and does not

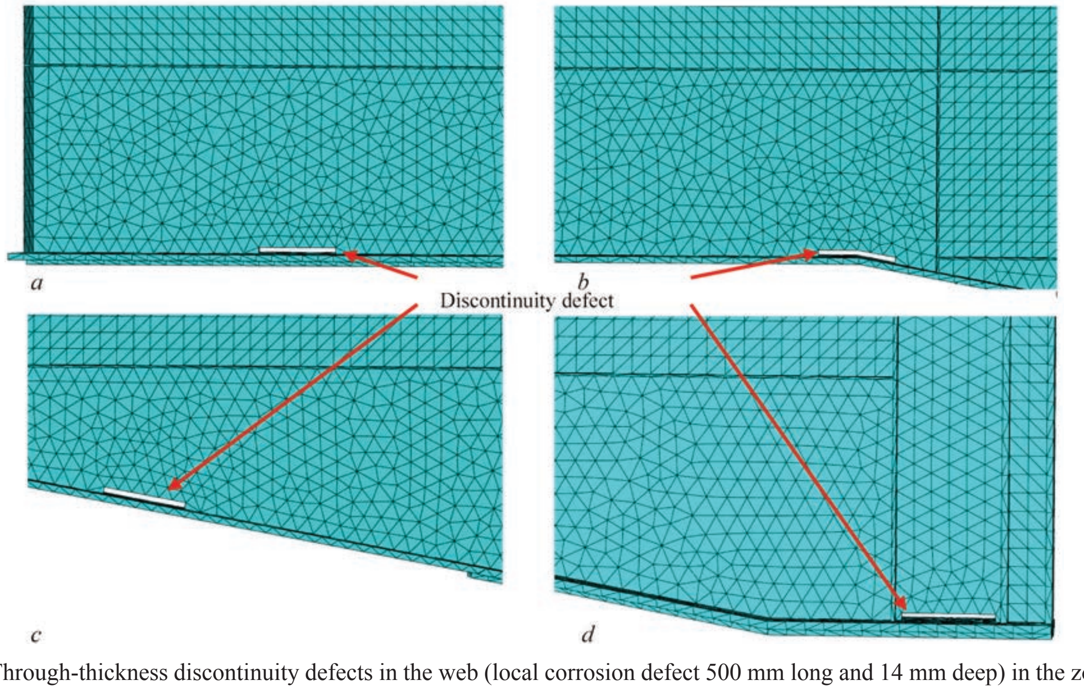



**Figure 5.** Through-thickness discontinuity defects in the web (local corrosion defect 500 mm long and 14 mm deep) in the zone of the joint of the web with the lower flange of the main 6-span beam: in the span central part (*a*); in the point of transition of the constant height beam into the haunch (*b*); in the inclined part of the haunch (*c*); in the central part of the haunch (*d*)

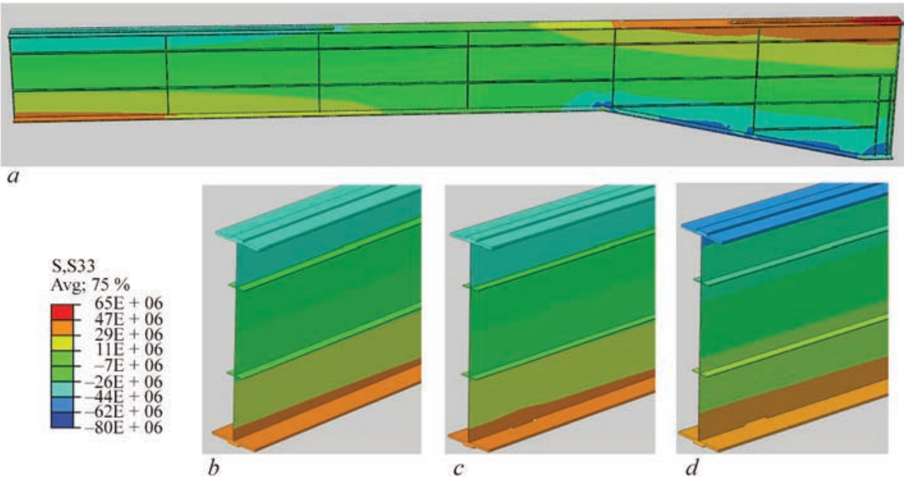



**Figure 6.** Distribution of longitudinal stresses over the span from distributed force ($P$ = 0.022 MPa), applied to the upper flange: over the entire beam (*a*); in the central part without a defect (up to 34 MPa) (*b*); with a defect in the flange (up to 37 MPa) (*c*); with a defect in the web (up to 35 MPa) (*d*)

exceed 5 MPa in its greater part, and locally in the zone of transition into the haunch and in the support zone the vertical compressive stresses reach 20 MPa.

In the haunch inclined part (Figure 9) and in its central part (Figure 10) in the defectfree beam in the lower girth zone the longitudinal compressive stresses do not exceed 50 MPa. Presence of the considered discontinuity defects may lead to a slight increase of stresses in the web and the flange up to 55 MPa.

Thus, the welded structure of the main span beams of the E.O. Paton Bridge has a rather significant strength margin (not lower than 3) in the case of ac-

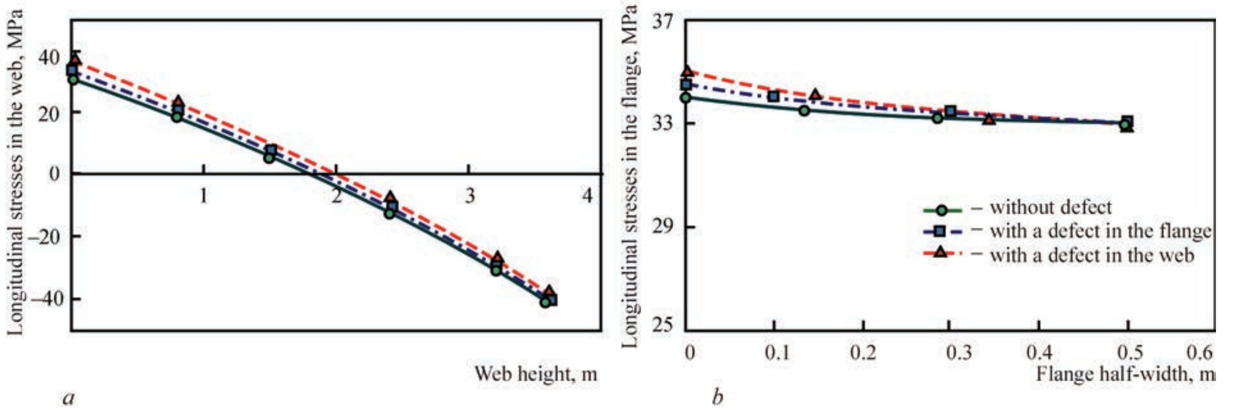



**Figure 7.** Distribution of longitudinal stresses in the central part of the span from distributed force ($P$ = 0.022 MPa), applied to the upper flange, in a defectfree beam and beam with a local through thickness defect 500 mm long in the lower flange and the web: by the beam web height (*a*); by the lower flange width (*b*)

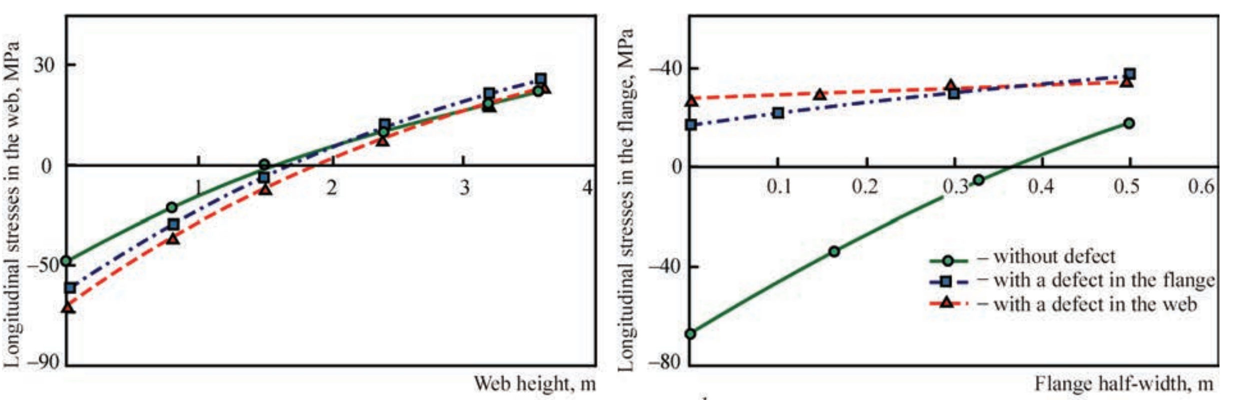



**Figure 8.** Distribution of longitudinal stresses in the zone of transition of constant height beam into the haunch from distributed force ($P$ = 0.022 MPa), applied to upper flange, in a defectfree beam and beam with a local through-thickness corrosion defect 500 mm long in the lower flange and the web: by the beam web height (*a*); by the lower flange width (*b*)

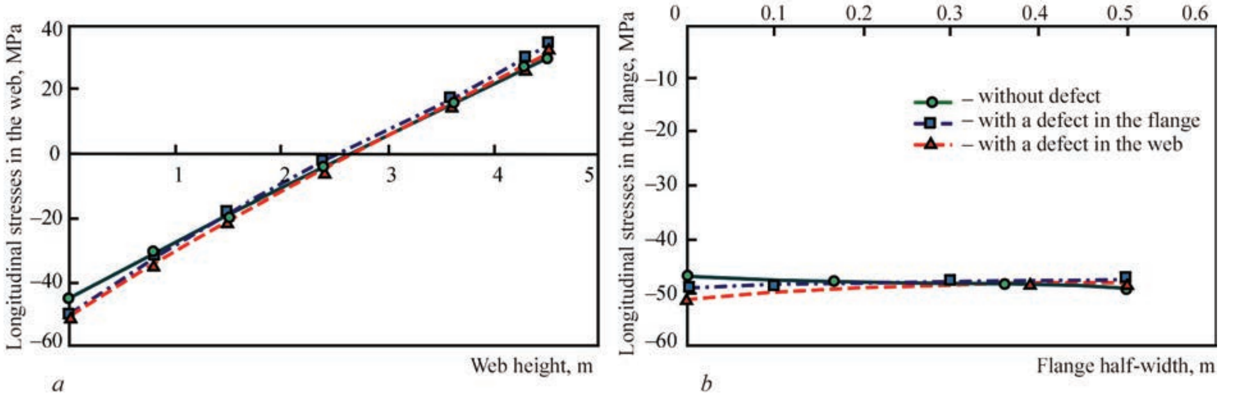



**Figure 9.** Distribution of longitudinal stresses in the haunch inclined part from distributed force ($P$ = 0.022 MPa), applied to upper flange, in a defectfree beam and beam with a local through-thickness corrosion defect 500 mm long in the lower flange and the web: by the beam web height (*a*); by the lower flange width (*b*)

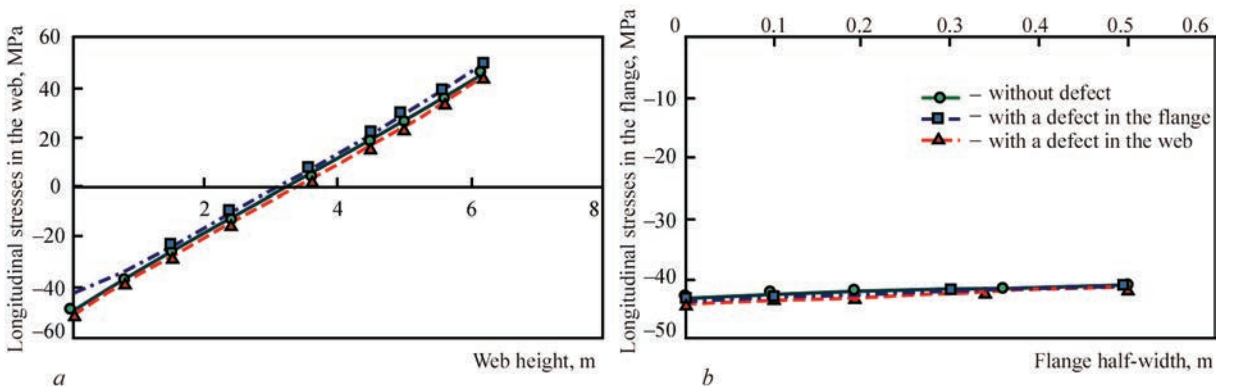
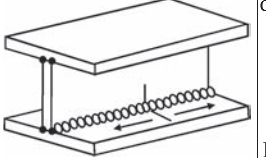
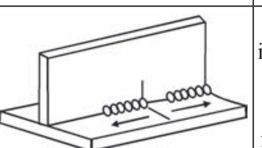



**Figure 10.** Distributions of longitudinal stresses in the haunch central part from distributed force ($P$ = 0.022 MPa), applied to upper flange, in a defectfree beam and beam with a local through-thickness corrosion defect 500 mm long in the lower flange and the web: by the beam web height (*a*); by the lower flange width (*b*)

tion of the normative distributed load, applied to the upper flange of the I-beam. Presence of local corrosion defects of material discontinuity of a rather large size in the zone of the joint of the web and the lower flange (500 mm long and through-thickness) causes a slight change of the stressed state.

Strength of bridge structures should be considered under the impact of cyclic loads during long-term service. In keeping with IIW recommendations on assessment of welded structure fatigue resistance [5], the longitudinal continuous welded joint of the beam lower girth with the web with K-shaped groove produced by continuous automatic welding with NDT performance, can be assigned to class FAT = 125 MPa (No.312 in the Table 1), where FAT is the minimum range of nominal stresses. Thus, to ensure fatigue resistance to macrocrack formation after $2 \cdot 10^6$ cycles, the maximum admissible range of longitudinal stresses in the flange should not exceed 125 MPa. In the case of formation of through-thickness corrosion defects in the welded joint of the flange and the web, such a welded joint can be regarded as intermittent (No.324 in the Figure 11), for which the admissible range of longitudinal stresses decreases to 80 MPa and lower, in the presence of shear stresses.

**Table 1.** Classification of typical; elements of welded joints based on limiting range of normal stresses [5]

| No. | Part sketch | Description | FAT, MPa | |
|---|---|---|---|---|
| | | | Steel | Aluminium |
| 321 |  | Longitudinal continuous welds with K-shaped reinforcement, automatic welding. No stops, NDT (stresses in the flange) | 125 | 50 |
| 324 |  | Longitudinal intermittent filler welds (normal stresses in the flange and shear stresses in the weld at the joint ends) | | |
| | | τ/σ = 0 | 80 | 32 |
| | | 0–0.2 | 71 | 28 |
| | | 9.2–0.3 | 63 | 25 |
| | | 0.3–0.4 | 56 | 22 |
| | | 0.4–0.5 | 50 | 20 |
| | | 0.5–0.6 | 45 | 18 |
| | | 0.6–0.7 | 40 | 16 |
| | | > 0.7 | 36 | 14 |

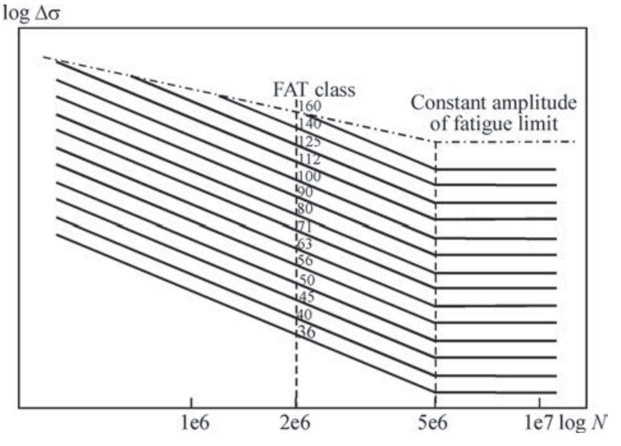



**Figure 11.** σ–*N* curves of fatigue resistance for different classes of welded joints (steel) for normal stresses [5]

Considering the rather long service life of bridge structures, it is rational to ensure such conditions that the effective stress ranges in the welded joints were below the endurance limit $\sigma_{-1}$, then the structure can be in service for an unlimited period of time. In keeping with σ–*N* curves (Figure 11) of fatigue resistance for different classes of welded joints for normal stresses [5], the endurance limit $\sigma_{-1}$ for the considered welded joints is equal to:

● without defects (No.321, FAT = 125 MPa) $\sigma_{-1}$ = = 92 MPa;

● in the presence of defects (No.324, FAT ≤ ≤ 80 MPa) $\sigma_{-1} \leq$ 57 MPa.

Thus, in the defect free beam the maximum ranges of effective longitudinal stresses (up to 50–65 MPa) are much lower than the endurance limits of the welded joint of the flange and the web. Such a level of longitudinal stresses corresponds to the modern concepts of ensuring a sufficient fatigue resistance of defect free elements and components of welded bridge structures [6, 7]. In the presence of corrosion defects, the endurance limit of the welded joint of the flange and the web of the span beam is on the level or even below the effective ranges of longitudinal stresses from normative load.

## CONCLUSIONS

Even though the welded structure of the main span beams of the E.O. Paton bridge has a rather significant static strength margin from the impact of normative distributed loading, applied to the upper flange of the I-beam, the presence of local corrosion defects of material discontinuity of a rather large size in the zone of the joint of the web and the lower flange essentially lowers the fatigue resistance of welded joints with detected defects that is dangerous in terms of reliability of the span structure and thus requires immediate performance of repair operations.

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## CONFLICT OF INTEREST

The Authors declare no conflict of interest

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