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QUALITY AND STRENGTH ASSESSMENT OF BUTT WELDS IN POLAND'S OLDEST WELDED RAILWAY BRIDGES

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Abstract. *Fitness-for-purpose (FFP) assessment is used for estimation the durability of welded plate girders in Poland's four oldest welded railway bridges. For the girders, the 20.2 m span length was designed. The bridges were constructed over the years 1937-1939 and now being readied for refurbishment. The first radiographic tests on the butt welds in 18 bridge plate girders were conducted in 1958 during which 101 internal cracks in 49 butt weld splices were found, and which were subsequently riveted. The usefulness of such a strengthening is assessed upon structural calculations for design and service loadings, fatigue tests on specimens with internal cracks in the welds, literature service strength of welds with cracks and design fatigue strength of welds with cracks using FFP assessment. The results provide a unique experimental database on the behavior of welds with inadmissible imperfections. They also allow to assess the likely length of their further service life.*

1 HISTORICAL OUTLINE

The transition to welding facilitated some aspects of steel construction and allowed steel weight savings by eliminating the connecting angles used in riveted structures. However, early welding technologies created new problems, both in terms of ensuring weld quality and also the qualifications of welding workers. The first welded bridge structures were of a test character, trying to solve the many problems which appeared during the design, manufacture and construction, and ensuring the proper quality as well as the strength of welded joints. The problems caused some skepticism among the engineers and scientists involved in welded bridge projects. Early attempts were quite modest, such as a railway bridge with a little over 5 m span constructed in 1921 by English structural engineers for the Newcastle and Carlisle railway line, GB [1]. Nevertheless, welding technology was moving forward in many countries. The introduction of high strength steels in bridge construction proved the worth of welding technology, and new problems associated with notch toughness and ductility appeared to be solved.

The world's first recognized welded plate girder bridge was a railway bridge over the Turtle Creek in East Pittsburgh, Pennsylvania, USA [2, 3]. The bridge was constructed in 1928 with a length of 16 m and a width of 4.67 m. The plate girders utilized web sections of 1460×10 mm and three flange plates of depths 10 and 25 mm with widths of 380 and 400 mm. Cross girders and stringers were from rolled beam sections of depths 640 mm and 457 mm. The bridge was constructed at a 60° skew with non-parallel abutments, which resulted in different girder span lengths of 15.97 and 16.29 m. Around 520 m of welds were executed, using 200 kg electrodes. The bridge was used to transport large generators by rail to the rest of the country.

The first all-welded plate girder railway bridge was constructed in Germany in 1930 with a span of 10 m [4, 5]. The construction was preceded by broad research, tests and theoretical studies. The bridge was initially installed on a track subjected to the movement of heavy locomotives, even at high speed. The bridge was also tested dynamically by a vibrating machine. Following positive results from the tests, the bridge was installed on a trunk railway line.

2 THE FIRST WELDED RAILWAY BRIDGES IN POLAND

The first welded railway bridge in Poland, designed by F.K. Szelągowski, later a professor at the Warsaw Technical University, became operational in July 1936 [6]. Two external spans of the bridge crossing over the Drwęca river on the Nasielsk and Toruń railway line were constructed as a welded plate girder structure of span length 13.0 m (Figs. 1 and 2). The main truss span of the bridge still utilized a riveted construction as it was estimated to be cheaper. The welded plate girders had a web section of 1600×11 mm and the flanges were from a single plate of 230×30 mm in the span middle sections, lowering into the supports to 210×20 mm and 190×10 mm plates.

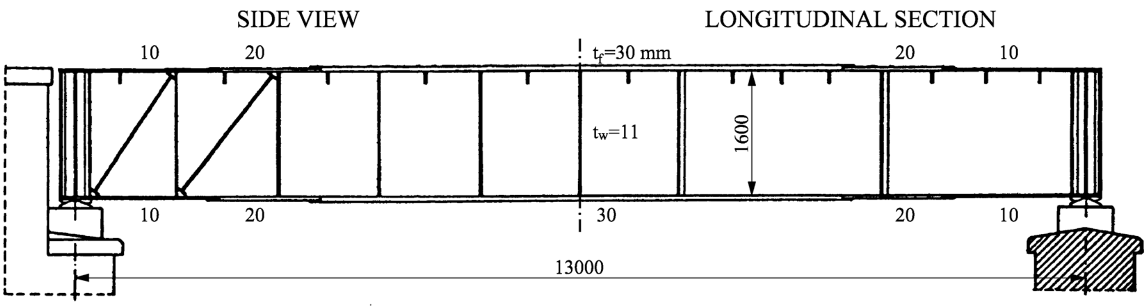


Figure 1: Plate girder for the external spans of the Drwęca river bridge

The web was connected with flanges by continuous fillet welds of leg length 6 mm. The flange joints were executed in a non-typical way according to Fig. 2. The ends of the thicker plates were whittled down at 160 mm length to the thickness of the thinner plate and then welded with a single v butt and around the fillet weld. All the welding works were executed in the workshop using a variable current and heavy coated Jotem type A electrodes for the vertical welds and Jotem type B-extra electrodes for the horizontal welds.

The successful construction of the first welded bridge encouraged the Ministry of Transport to construct a further four welded bridges on the same railway line [7, 8, 9]. The bridges were erected in the years 1937-1938 according to a design by F.K. Szelągowski and M. Witordowa (MoT). The welded structures were made by the L. Zieleniewski & Fitzner-Gamper company in Cracow. All the welding works were carried out in the factory shop, with only the riveted connections executed on site. For welding, direct current and *Baildon* domestic electrodes were used.

The bridges of semi-through plate girder construction have a span length of 20.2 m (Figs. 3 and 4). The main girders, along with the cross beams, were manufactured in the shop as fully welded components and transported, ready to use, to the construction sites. The welded plate girders have web plates of 2400×12 mm with two shop butt splices covered by two-sided vertical plates of thickness 8 mm. The flange plates are of constant width 240 mm with variable thickness from 10 to 40 mm. The flange plate sections are connected by double v-butts which were covered by two one-sided rhomboidal horizontal plates (Fig. 4 – flange splice).

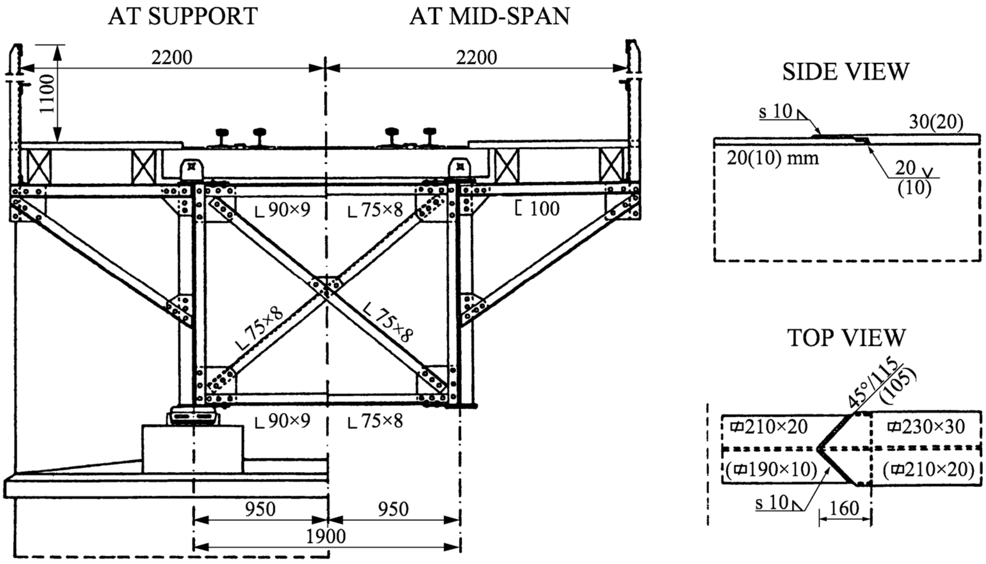


Figure 2: Bridge cross section and the welded flange splice

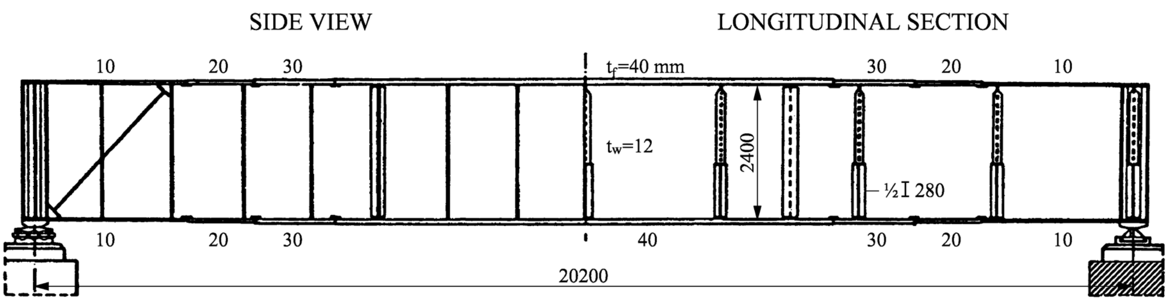


Figure 3: Plate girder for the four bridges constructed in the years 1937-1939

There are K-type wind bracings from single angles of 100×100×8 mm in the girder planes of the lower flanges. These were connected by riveted joints using 22 mm rivets. The joints were executed on site along with the riveted joints of the crossbeams, stringers and other deck members. The first bridge of this group was constructed over the Sierpienica river and became operational in October 1937 (No. I). The condition of the bridge during a butt splice radiographic examination is shown in Fig. 5.

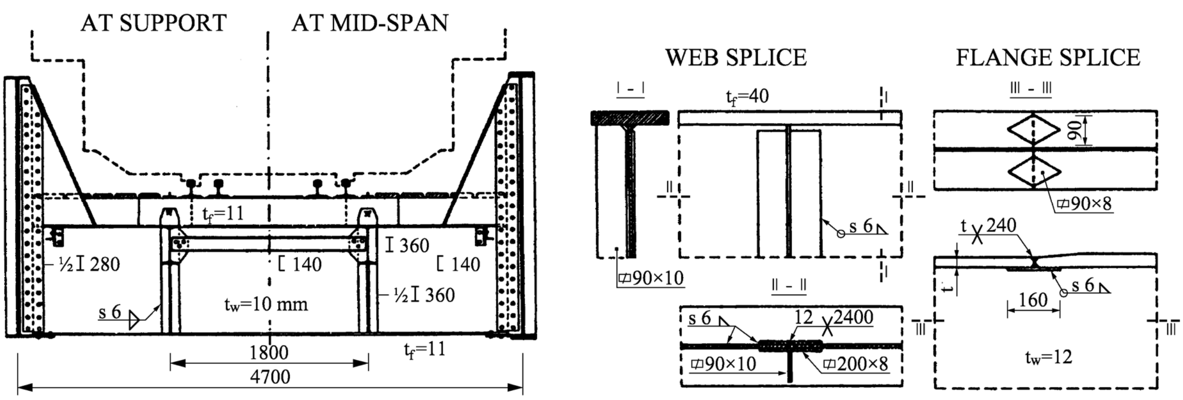


Figure 4: Bridge cross section and welded butt splices

Figure 5: View of the two-span bridge over the Sierpienica river in 1958 during radiographic examination

Over their eighty years of operational life, the bridges suffered partial structural and material degradation. In 2018 the second of the authors drew up a technical design for retrofitting one of the bridges, the two-span bridge (No. III) over the Płonka river in Płońsk (Fig. 6). While preparing the retrofit design it was necessary to estimate the grade and quality of the structural steel used for the bridge construction as well as its design strength value. The bridge steelwork was planned to be refurbished via welding so the weldability of the steel required estimating. The properties were calculated by the authors along with the methodology used for determining them. The procedure undertaken may be used for the retrofitting and repair of similar bridge structures.

Figure 6: Actual views of the two-span bridge over the Płonka river in Płońsk before retrofitting (2019)

The full range of material testing, together with notch toughness and static tensile tests, is expected to be conducted on the material from the members dismantled from the bridge during the retrofitting works.

3 RADIOGRAPHIC TESTS AND THEIR ANALYSIS

The Steel Structure Chair at the Technical University of Szczecin carried out the radiological examination on the flange butt joints in the main girders of the four bridges with the construction shown in Fig. 4 (web splice). The technical data for the bridges are given in Table 1 in chronological order of construction. Radiological examinations were carried out twice, in 1958 and in 1971. The tests were limited to the flange splices, as vertical stiffeners were welded directly in the axis of the welded web splices, thus preventing inspection (see Fig. 4 - web splice).

Table 1: General data for the bridges and their radiological examination

Bridge No. and river	Building year	Test year	Number of girders (joints)	Number of rtg.		Number of cracks		
				Σ	With cracks	Upper flange	Bottom flange	Σ
1	2	3	4	5	6	7	8	9
I Sierpienica	1937	1958 1971	4 (44)*	88	7	3	5	8
II Karsówka	1938	1958 1971	2 (24)	48	6	4	2	6
III Płonka	1938	1958 1971	4 (48)	96	46	21	27	48
IV Wkra	1938	1971	8 (72)	144	31	22	17	39
Total			18 (188)	376	90	50	51	101

* during the post-war repeairs 4 flange splices had been riveted

Table 2: Butt weld specification according to their quality levels

Bridge No.	Type of splices	Number of rtg.	Number of rtg. for the quality level					Total rtg.	
			B+	B	C	D	>D	B+ ÷ C	D and >D*
1	2	3	4	5	6	7	8	9	10
I	Upper flanges	44	2	1	2	1	38	5	39 (3)
	Bottom flanges	44	-	2	-	3	39	2	42 (4)
II	Upper flanges	24	1	3	3	4	13	7	17 (4)
	Bottom flanges	24	3	4	5	1	11	12	12 (2)
III	Upper flanges	48	1	7	3	2	35	11	37 (21)
	Bottom flanges	48	4	-	5	5	34	9	39 (25)
IV	Upper flanges	72	11	11	7	2	41	29	43 (18)
	Bottom flanges	72	11	9	9	7	36	29	43 (13)
	Total	376	33	37	34	25	247	104	272 (90)

* the values in brackes are the number of rengenographs with cracked welds

In total, 188 welded butt splices were X-rayed: 94 in the top flanges and 94 in bottom tensile flanges, taking 376 weld X-rays – col. 5÷9 in Tab. 1. 101 cracks were ascertained on 90 radiographs. The length of the cracks was from 8 mm to 240 mm, so at the extreme a breakage appeared over the total length of the splice – the flange width (Fig. 7)

Table 2 shows a summary of the quantity of radiographs with a division into imperfection classes. At the time of the tests, the imperfection classes were marked as R1 ÷ R5 according to PN-87/M-69772 Polish standards. Now the classes may be compared with the quality levels the of welds: B+, B, C, D and >D, as determined by EN ISO 5917:2014, and in Table 2 the welds are qualified according to these quality levels. In the total summary (cols. 9 and 10), the welds are divided into two groups: a group with quality levels B+, B and C which complies with the requirements of the standard EN 1090-2:2018, and a group of welds which are not allowed for new bridge constructions, D and >D. The latter is up to 72.3% of the rested sections and for the bridge over the Sierpienica river (No. I) this value is 95.5% for the lower tensile flanges. Generally, the condition of the butt splices in all bridges was ascertained to be poor: 101 cracks on 90 radiographs (23.9%) were discovered. These cracks could lead to fatigue cracks developing (Tab. 1 col. 6 and 9).

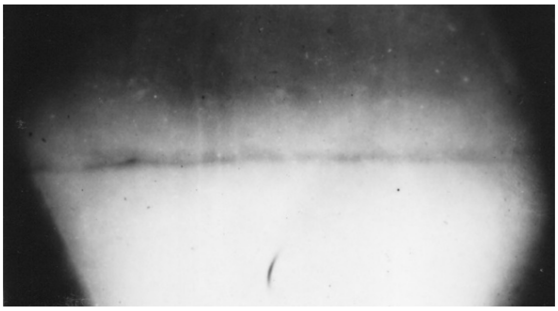



Figure 7: Two radiograms with cracks from the bridge over the Sierpienica river (No. I) - 1958 and 1971

The negative results of the tests were a surprise for bridge engineers and required the splices with cracks to be riveted for bridge Nos III and IV (Płonka river and Drwęca river). The decision to reinforce the crack flange splices was undertaken based on static standard loadings, which had never been taken into account in experimental and theoretical analysis [7-12]. The Paris equation, recognized as the first to take into account the parameters from fracture mechanic $dl/dN = C(\Delta K)^m$, was published in 1957. In 1959, 26 flange splices were riveted onto bridge No. III (13 in the upper flanges and 13 in the lower flanges). For bridge No. IV, 23 splices with cracks were riveted (12 in upper flanges and 11 in lower flanges).

4 CHEMICAL AND MECHANICAL PROPERTIES OF BRIDGE NO. III STEEL

Spectrometric chemical analysis of the steel on bridge No. III was carried out in 2018 on the basis of three samples cut out from an upper flange (U), a stringer (S) and a bracing diagonal (B). The results are given in Table 3. The analysis was carried out using a Leco GDS500A emission spectrometer. For comparison, the chemical composition of old mild steels, former Polish bridge steel (St3M) and a contemporary non-alloy structural steel S235 are given.

The comparison of the results for particular samples shows the bridge structure was manufactured from one steel type. It is a low-carbon mild steel with limited carbon, manganese and silicon content. Of note are the small differences in content of the individual elements in the samples analyzed.

Using the chemical composition content, the metallurgic and structural weldability were assessed. For this purpose, the following ratios were established [15]:

- the equivalent carbon content (for steel with C < 0.18%)

$$CEV = C + \frac{Si}{30} + \frac{Mn+Cu+Cr}{20} + \frac{Ni}{60} + \frac{Mo}{15} + \frac{V}{10} + 5B = 0.12\% < 0.41\% \quad (1)$$

- the material resistant ratio for hot cracking

$$HCS = 1000(S + P + \frac{Si}{25} + \frac{Ni}{100}) \frac{C}{3Mn+Cr+Mo+V} = 3.10\% < 4\% \quad (2)$$

- the material resistant ratio for cold cracking

$$CEV' = C + \frac{Mn}{6} + \frac{P}{2} + \frac{Mo}{4} + \frac{Ni}{15} + \frac{Cu}{13} + \frac{Cv+V}{5} + 0.0024t = 0.24\% < 0.41\% \quad (3)$$

- the hardness of the heat effected zone

$$HVmax = 90 + 1050C + 47Si + 75Mn + 30Ni + 31Cr = 219 \text{ HV} < 350 \text{ HV} \quad (4)$$

The plate girder structural steel is weldable without limitations, as all the ratios are lower than the limit values. At the same time, the phosphorus and sulfur contents are lower than 0.05% in each case and the manganese content is lower than 1%, along with a vestigial content of silicon (rimmed steel).

Table 3: Comparative chemical composition for the bridge steel

Type of steel	Chemical composition, %							
	C	Mn	Si	P	S	Cu	Cr	Ni
U - t_f = 10 mm	0.083	0.479	3.6 ppm	0.030	0.027	0.151	0.111	0.089
S – I360	0.086	0.539	4.4 ppm	0.026	0.015	0.162	0.131	0.062
B – L100×10	0.078	0.417	7.5 ppm	0.021	0.013	0.121	0.158	0.108
Mild steel	0.03÷0.35	0.04÷0.75	< 0.18	< 0.16	< 0.11	0.11÷0.14	< 0.014	0.03÷0.04
St3M	max 0.20	min 0.40	0.12÷0.30	max 0.05	max 0.05	–	max 0.30	max 0.30
S235J0	max 0.17	max 1.40	–	max 0.03	max 0.03	max 0.55	–	–

Table 4: Brinell hardness measurement

Type of sample	Thickness of sample, mm	Diameter of imprint, mm			Brinell hardness			
					HBW	R_{mB}, MPa	α	R_{eB}, MPa
U - flange	10	2.7	2.7	2.7	121	404	0.7	282
S – I360	18	2.8	2.8	2.8	111	375	0.7	262
B – L100×10	10	2.8	2.8	2.8	111	375	0.7	262

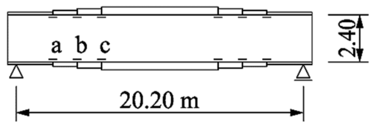
The mechanical properties of the bridge steel were assessed on the basis of Brinell hardness measurements. The hardness was measured under standard conditions using a steel ball of 5 mm in diameter. The measurements were carried out on samples for chemical analysis. The measurement results are given in Table 4. After determining the value of R_{mB} taking into consideration that $\alpha = R_{eB} / R_{mB} = 0.70$, $R_{eB} = 260$ MPa. Taking according to Polish bridge standard the partial coefficient $\gamma_s = 1.15$ with a 5% increase when $R_{eB} < 355$ MPa we obtain:

- characteristic strength $f_y = R_{eB} = 260$ MPa,
- design strength $f_{yd} = 260/1.2 = 215$ MPa.

5 FLANGE WELD STRESS ANALYSIS AND FATIGUE DURABILITY

Strength analysis was undertaken for a global analysis of the weld behavior (Table 5). The characteristic values of stresses in the butt welds were established (in the axes of the cover plates) and beyond the cover plates in the flange plates. The calculations were made according to Polish bridge standards – as for fatigue verification, the standard vertical traffic loads (LM71) were taken with a dynamic factor. Nominal values of stresses for permanent actions are given in col. 2 and for permanent and rail traffic model actions in col. 3. The stresses are calculated for the persistent design situation using the classified vertical loads with factor $\alpha = 1.21$. The stresses for service loads are also calculated for permanent actions and a heavy locomotive with dynamic effects; the results are shown in col. 4. Column 6 gives the number of load cycles on the bridge up to 1959 (the riveting of the butt splices), until the present day (2019) and after a further 50 years in service in 2069.

Table 5: Normal stresses in butt welds (value in numerators), in flange plates (value in denominaters) and number of load cycles

Girder scheme	Stresses, MPa			Number of load cycles $N \cdot 10^6$
	Permanent loads	Persistent situation		
		LM71	Locomotive ET - 21	
1	2	3	4	5
	$\sigma_a = \frac{14.3}{16.8}$	$\frac{116.1}{136.5}$	$\frac{68.4}{80.4}$	$N_1 = 0.31$[1)]
	$\sigma_b = \frac{15.1}{17.1}$	$\frac{122.1}{138.1}$	$\frac{74.4}{84.2}$	$N_2 = 1.18$[2)]
	$\sigma_c = \frac{15.4}{17.0}$	$\frac{123.7}{136.6}$	$\frac{72.4}{79.9}$	$N_3 = 1.91$[3)]

1) Number of load cycles up to 1959 (splice riveting)
2) Number of load cycles up to now (40 passages per day)
3) Number of load cycles in 2069 (after further 50 years)

The values of the service stresses σ_{ser} obtained constitute about 60% of the stresses from standard loading. The service stresses are compared with the fatigue stresses obtained during laboratory tests, given in Fig. 8 [16, 17]. The regression lines given there are concerned with:

- tests on specimens marked "a" – butt welds of quality level B+, B and C, for which the infinitive life fatigue strength (endurance limit) is 125 MPa,
- tests on specimens marked "b" – butt welds with internal cracks, for which the obtained

infinitive life fatigue strength is 90 MPa,

- tests on specimens marked "c" – butt welds with one-sided rhomboidal cover plates, for which the obtained infinitive life fatigue strength is 79 MPa.

The fatigue stress values are marked as red lines in Fig. 8 (logarithmic scale) according to the regression lines for the number of load cycles N_1 and N_2. The minimal values of these stresses are σ = 120.1 and 95.6 MPa. At each case these values are larger than the calculated service stresses $\sigma_{ser, max}$ = 74.4 MPa (Table 5).

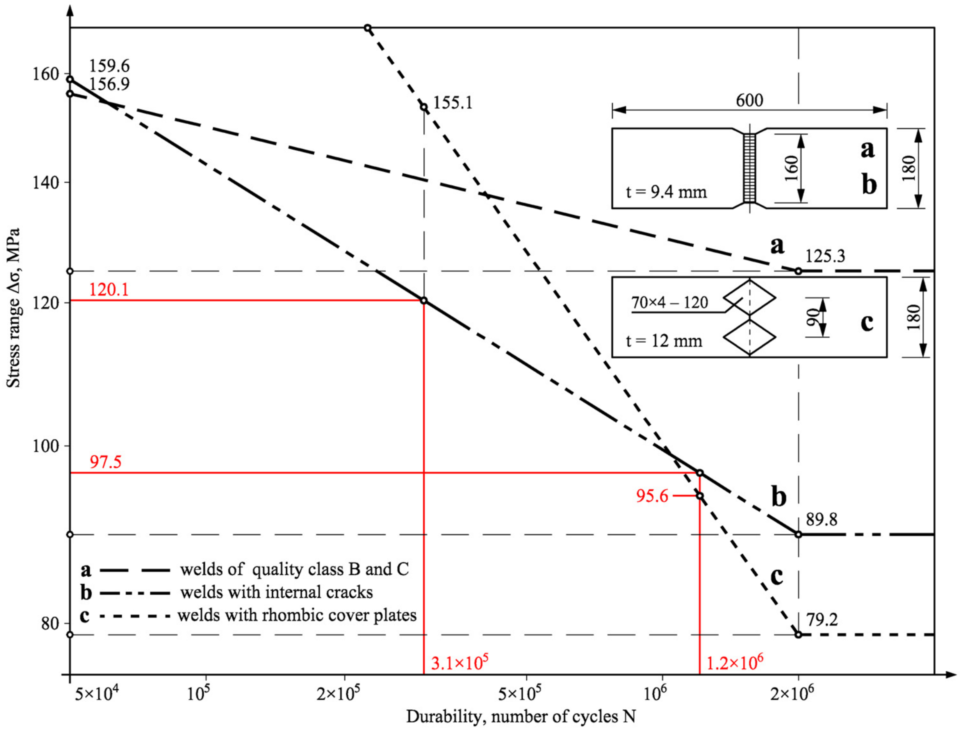


Figure 8: Regression lines and service load data

This is evidence for the safe operation of bridges Nos I and II, whose cracked welded splices were not strengthened in 1959. For a further 50 years of bridge service, in 2069, at the number of load cycles $N_3 = 1.9 \cdot 10^6$, the stress according to the regression line "c" will be σ = 80.5 MPa, so the stress will also be larger than the maximum value of the service stress $\sigma_{ser, max}$.

6 SUMMARY

The radiographic testing of welded butt splices in the first welded railway bridges in Poland revealed their very low quality. The considerable number of cracks in the flange splices on bridges Nos III and IV (48 and 39 cracks) and the stress values calculated for standard loadings led to the decision to undertake strengthening work. In total, 49 splices were retrofitted by additional cover plates and riveting. The rest of the splices were X-rayed after 13 years of further service life. These additional examinations showed no growth in the existing cracks nor the formation of new cracks. It is rightly noted in EN-19931--9: 2005 that "cracks … do not necessarily mean the end of the service life". The pulse tension from the service loading is Δ_e

= 74.4 – 15.1 = 59.3 MPa and is 25.1% lower than the infinitive life fatigue strength Δ_c = 79.2 MPa (Fig. 8).

The bridges on the railway line were constructed from low-carbon mild steel with actual characteristic values of ultimate strength R_m = 375 MPa and yield strength R_e = 260 MPa. Global tests on the structural steel of bridge No. III will be carried out during its nearest planned retrofit.

These steel bridges constitute an unique experimental base on the behavior of welds with non-tolerated welding imperfections – internal cracks.

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