Chapter

Quality and Fatigue Assessment of Welded Railway Bridge Components by Testing

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Abstract

During a decades-long program from 1953 to 1990, the quality of welded joints in railway bridges in Poland was assessed and quantified. It was discovered that many welded joints have technological cracks, and their quality is poor, especially in old constructions. Nearly, 200 bridges were tested using X-ray examination. The number of joints tested was over 15,000; cracks were discovered in 400 welded joints in the 34 bridges tested. To solve the problem, repeated examinations on welded joints with imperfections were undertaken and laboratory fatigue tests were performed. The tests and numerical analysis allowed fatigue behavior and tensile stresses in welded butt splices with cover plates to be recognized and excluded such a structural solution in bridges. The existing discontinuities and imperfections in welded joints following many years in service show no growths or forming of new cracks, as the applied stresses are below the threshold fatigue strength. As a result of decades of service, steel bridges undergo functional aging, and their structural steels undergo structural aging. There is a need to both harmonize differentiated procedures and create national recommendations to assess their safe endurance. Therefore, of use may be the findings presented in the chapter.

Keywords: welded bridges, reliability, welded joints, fatigue, NDT, imperfections

1. Introduction

Bridges are engineering structures which are subjected to dynamic actions, variable in time—repeated millions of times, and inconsistently. The endurance of steel bridges in service is determined mainly by fatigue, which usually causes catastrophic failures, and corrosion, which ordinarily results in degradation failures [1–3]. Many early welded bridges in Europe were fractured over the period 1925–1936 and later [4]. Many failures and catastrophic events happened due to fatigue and fracture despite the pioneering works of August Wöhler and other scientists [5, 6]. As a result, the limit state analysis of structures with a probabilistic approach became necessary.

Fractures led to a prohibition on using welding for early high-strength steels [7, 8]. As a result, welding was allowed only on mild steels. It was not until the late 1950s that some fatigue requirements were introduced into the design of steel railway bridges. It took another decade for these to be considered for road bridges; this started the modern approach to fatigue.

Each welded structure possesses discontinuity, or cracks and other imperfections resulting from manufacture or welding. These do not show any growth or instability while the loading rate is relatively low, below threshold fatigue strength. This means that such a stress range does not cause crack propagation. The endurance of steel structures or fatigue details with imperfections may be determined by testing small specimens or numerical analysis.

There are still many welded steel bridges in service which were manufactured in periods of poor quality of both materials and welded joints as well as few requirements for fatigue. After many years in service, each steel structure also undergoes what is known as functional aging, and its structural steel is subjected to structural aging. The mechanism of the aging process may be described by the classical separation theory or the newer dislocation blocking theory.

The quantitative results of welded butt splice quality obtained over 37 years of the testing of welded railway bridges in Poland are given. The radiographic examinations were conducted by the Steel Construction Chair at the West Pomeranian University of Technology in Szczecin [3]. The radiographic tests on the welded joints were supplemented by laboratory fatigue tests and more recently by Finite Element Method (FEM) analysis.

2. Fatigue-induced structural changes in steels

It is assumed that the basic factor describing the properties of a material is the changes in internal material structure resulting from the structural degradation processes described among other things in [3, 9–11]. This is mainly concerned with the decreasing value of the impact strength, sometimes even by several times. The comparison level for such a phenomenon is the difference in material properties of actual steel versus normalized, as the other has material properties from the time of the structure's construction. The simulation of these properties is carried out by thermal annealing. For this purpose, specimens are annealed at a temperature of 930°C (steels of C $\leq$ 0.26%) for an hour and then cooled in air. This way the minimal possible grain size in the steel is achieved.

This process increases yield strength and at the same time lowers the ductile-brittle transition temperature, i.e. significantly increases mechanical properties (**Figure 1**). Sometimes, astonishing results are obtained. For example, from the railway bridge over the Warta River in Gorzów Wielkopolski (western Poland), two types of steel specimens were tested for Charpy impact energy.

The bridge was constructed for the German Railways in 1938, using German normalized mild steel St37-12 (**Figure 1**). The tests refer to specimens which were

- naturally aged S without any measures;
- normalized N, i.e. annealed at 930°C for an hour and then cooled in air.

A significant aging effect was found in the structural steel after 77 years in service. At −20°C, the impact energy was 19.2 times higher.

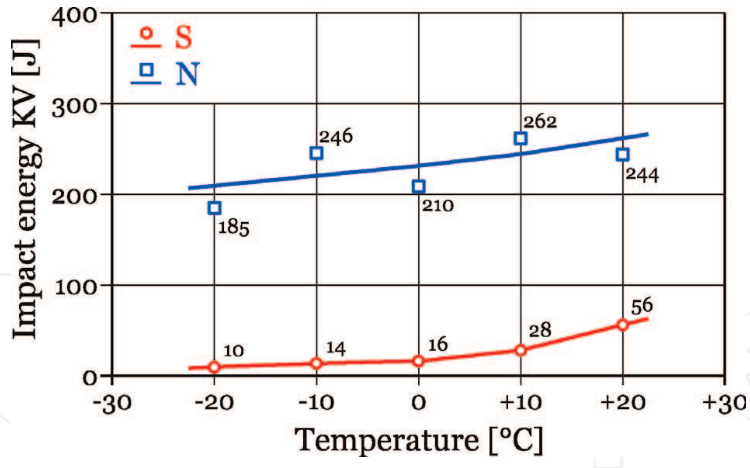


Figure 1.
Impact energy KV(T) for naturally aged (S) and normalized (N) specimens from a plate girder railway bridge constructed in 1938.

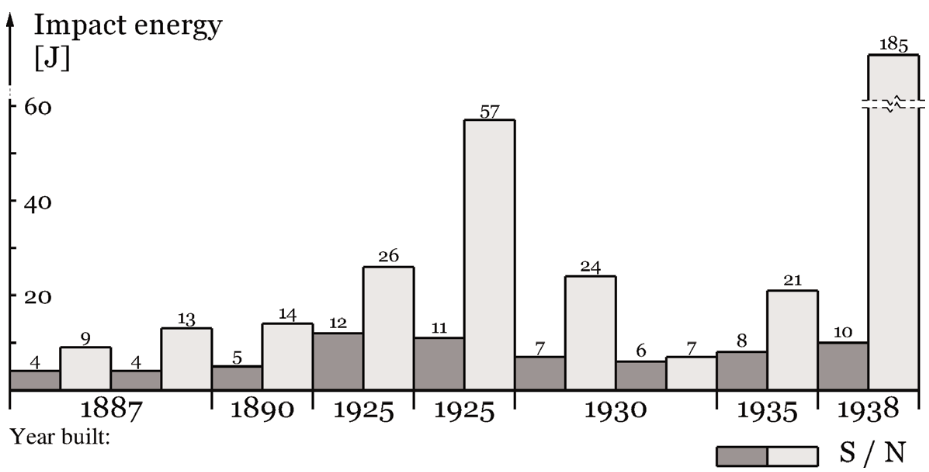


Figure 2.
Notch toughness of tested bridge steels at temperature −20°C for naturally aged (S) and normalized (N) specimens.

The results of impact energy tests at temperature −20°C for nine steel grades from eight bridges constructed in the years 1887–1938 are shown in **Figure 2**. Two types of specimens were tested: naturally aged and normalized. The steel in post-service conditions showed a very small KV impact energy value.

The actual ascertained KV values are only from 4 to 12 J. This dependence concerns all the steels tested independently of carbon content from 0.016% to 0.258%.

Such a condition shows brittleness in the material; this is a particular danger when it is located in areas of stress concentrations, for example around welding imperfections (WIs) in a weld – **Figure 3**. Welding imperfections (WIs) are crack initiators when the loads reach a prescribed critical value. The largest concentration of normal stresses σ_x is caused by ellipsoidal welding imperfections and longitudinal ones with elliptical cross sections. For these two groups of welding imperfections, the maximum stress gradient increases as the curvature radius value of the sharpest shape of welding imperfections lowers.

For example, for welding imperfections with shape characteristic $t/\rho = 100$, the shape coefficient values $u = y/t = 1$ are 13.63 and 21.00 **Figure 3**. In the case of globular welding imperfections of a small stress concentration—class III with a sharp shape—it

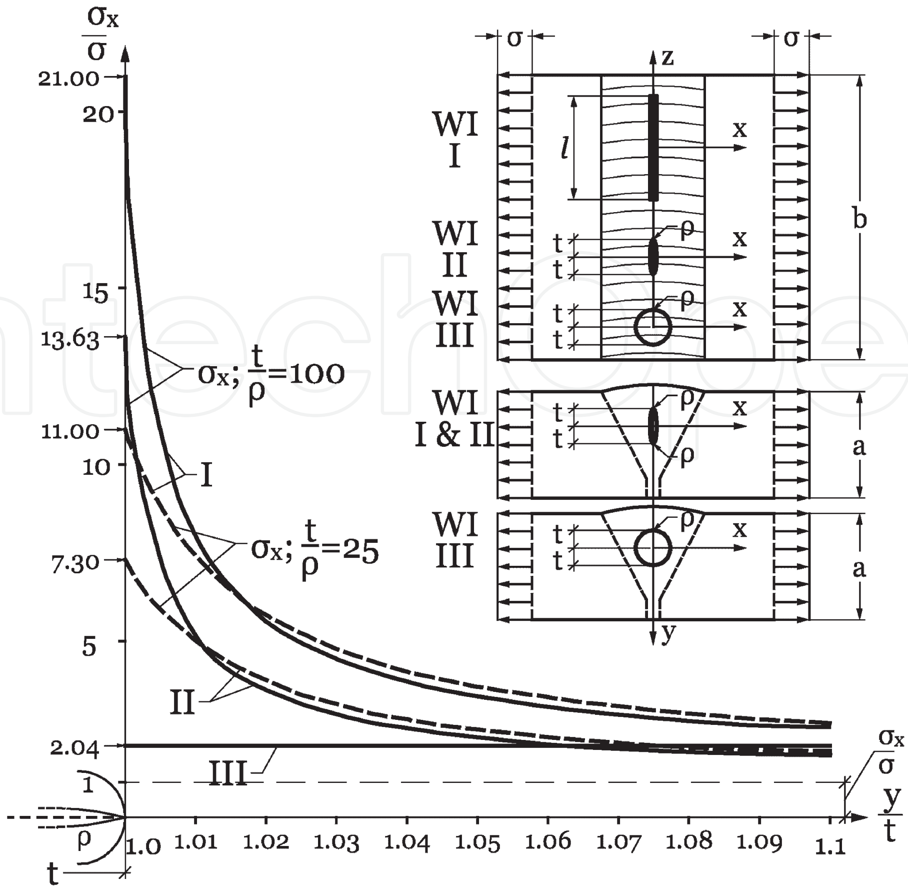


Figure 3.
Stress distribution σ_x near welding imperfections: longitudinal – I, ellipsoidal – II, and globular – III [3, 12].

is independent from the imperfection size and is $\sigma_x = 2.04 \cdot \sigma$ [3, 12], where σ is the design stress.

Tensile stresses near a notch may be the effect of external loading, residual stresses, or both simultaneously [13]. It is not possible to exclude brittle fracture even if there is no external loading.

According to the literature [14], the fatigue strength at 10^5 cycles to strength at $2 \cdot 10^6$ cycles ratio is in the range 1.44–2.45, with the average value 1.85. Similar values of 1.75 and 1.73 were obtained for Polish mild steel (St3SX: C = 0.160%, Mn = 0.498%) [15] – **Figure 4**. Tested were as follows:

- naturally aged specimens S, for 35 years (f_y = 260 MPa, f_u = 405 MPa),
- artificially overaged specimens NN (f_y = 495 MPa, f_u = 515 MPa).

Specimens NN were cold-deformed up to 10% relative elongation and then heated up to temperature 250°C, kept at this temperature for an hour, and then cooled in air.

The value of the infinitive fatigue strength Z_{rj} = 145.3 MPa of naturally aged steel for 35 years is 55.9% of its yield strength f_y = 260 MPa and for the overaged steel this value is Z_{rj} = 157.3 MPa which constitutes only 31.8% of its yield strength. As a result of additional aging (specimens NN), there was an unexpected very large increase in steel yield strength f_y of 90.4%, and ultimate strength f_u increased by 27.1%.

The specific character of bridge loadings and the structural changes in steels due to aging are the main reasons why a general hypothesis for their fatigue estimation has yet

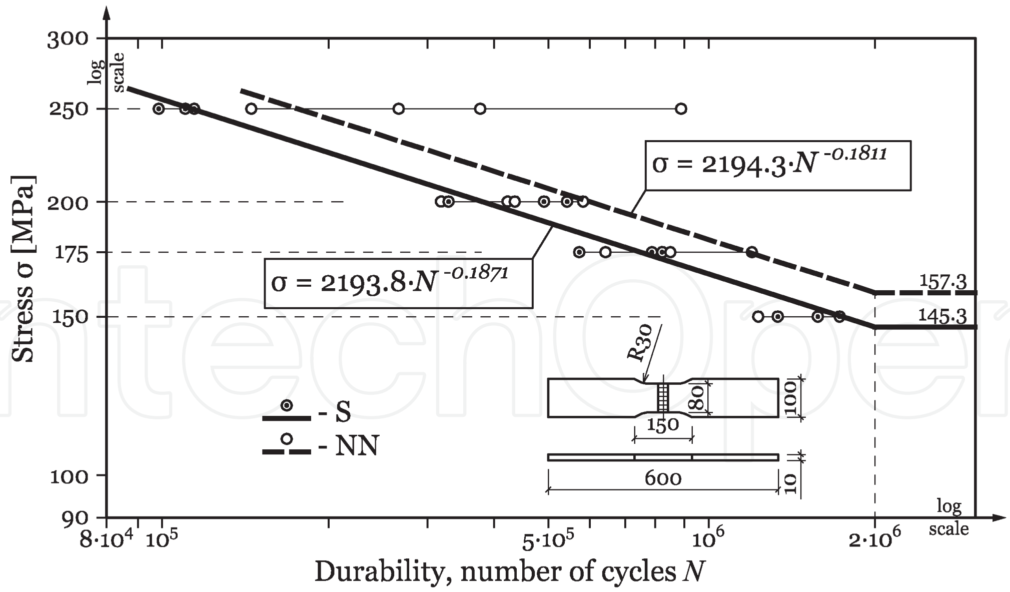


Figure 4.
Regression straights obtained from fatigue tests on: naturally aged specimens (S) and additionally aged specimens (NN) [15].

to be proposed [16]. Phenomenological models are still used despite the significant number of studies undertaken, especially for riveted structures, and more excellent research tools for testing have appeared as well as the possibility of numerical analysis. A problem has arisen as to how to adapt the information from laboratory fatigue tests to the design for the durability of structures in service. Knowledge of both the loading spectrum which a structure will be carrying and problems with butt splices have become necessary.

Welded structures under high stresses are damaged mainly by fatigue crack growth or brittleness. Hence, fracture mechanics has recognized the most important issues:

- discovering and locating flat welding imperfections in a welded joint,
- determining flat welding imperfection dimensions precisely, especially in the joint depth.

Over the years 1970–1980, the International Welding Institute introduced the "fitness for purpose" criterion, which relies on the formulation of fracture mechanics calculations for determining the permissible size of welding imperfections, thereby confirming the required quality and durability of structures according to standards and technical requirements. The determination of specific quality levels and example calculations as well as the determination of safety coefficients are given in [17, 18].

3. Quality of welds in railway bridges according to Non-Destructive Testing (NDT)

The aging of metallic materials favors the formation of brittle cracks as the ductile-brittle transition temperature clearly approaches higher values. Strains associated with aging depend on the location in the structure. They are particularly dangerous in stress concentration areas, e.g. around welding imperfections (WIs) and in heat-affected zones (HAZs). According to Neuber [3, 12], the largest concentration of stresses is associated with ellipsoidal and longitudinal welding imperfections with an elliptical cross section (**Figure 3**).These types of imperfections include cracks and lack of fusion, as well as band slag intrusion and incomplete side fusion. This issue is becoming particularly significant for bridges constructed after 1936, considering the poor quality of connecting welds. This has been confirmed by radiographic tests performed on bridges in service on Polish railway lines [3].

The Steel Structure Chair at the Technical University in Szczecin (now WPUT) carried out radiological tests on butt splices in the steel girders of around 200 railway bridges on the Polish railway network. For 154 bridges, including 124 plate girder bridges and 30 truss bridges, the exact time of construction was also established.

The range and results of in situ nondestructive radiological tests on the railway bridges are given in two histograms (**Figures 5** and **6**). The tests consist of the sum of 5-year intervals which take into account the bridge construction period from 1936 to 1975. **Figure 5**, in the upper part of the histogram, shows the number of bridges tested and the number of X-rays taken over a particular 5-year interval. At the same time, the number of internal structural cracks in connecting welds is given. These cracks were discovered on 437 X-rays. They constitute 2.8% of the total number of welds tested, equaling 15,875 units. This number includes 10,507 X-rays on butt joints in tensile components and the remaining 5368 X-rays on compress components. Simultaneously, the lower part of the histogram gives the number of welded joints tested over given 5-year intervals.

Figure 6 shows the proportional and numerical specification of hot cracks detected in bridge structures for given 5-year intervals of construction. Apart from one crack from 1974, the remaining cracks were ascertained in bridges constructed before 1960. According to Eurocode 1993-1-9 for the design of steel structures, such cracks do not

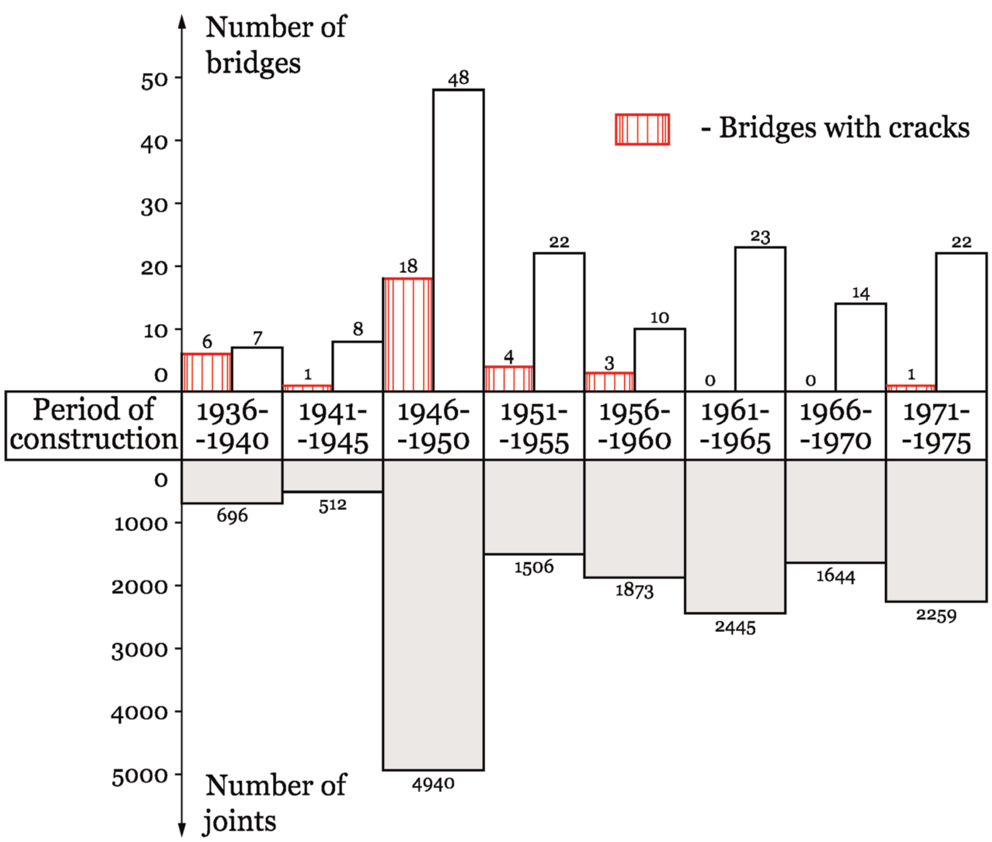


Figure 5.
Numerical specification of the bridges and welded joints tested.

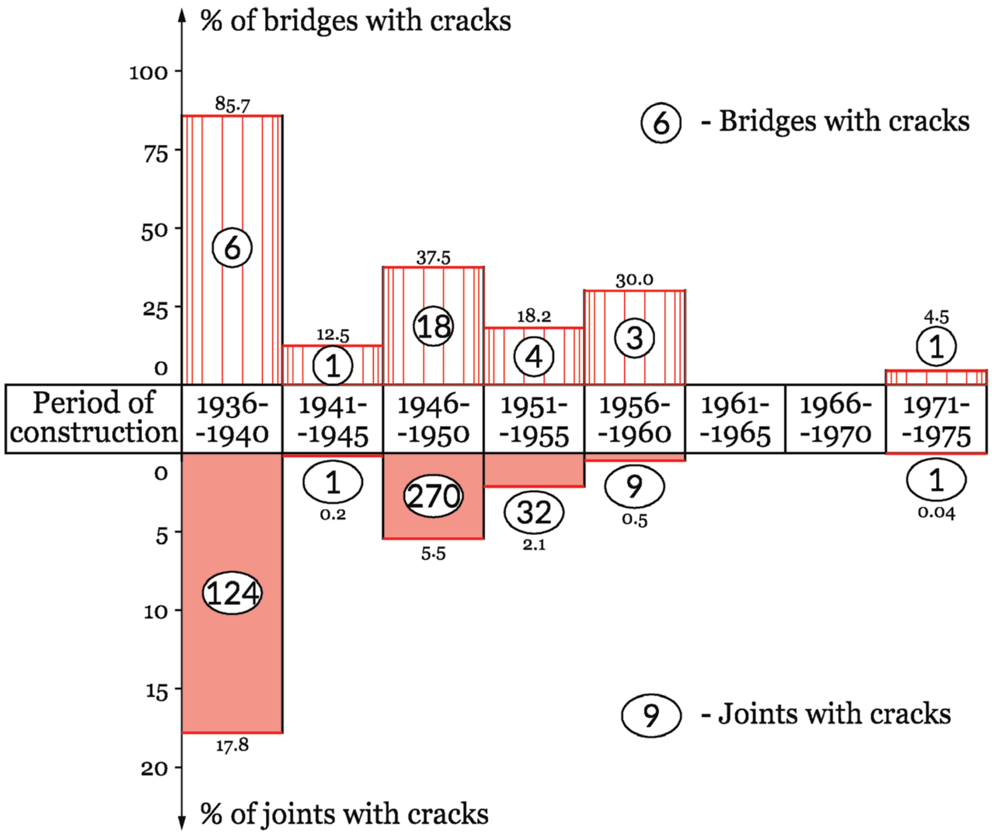


Figure 6.
Specification for bridges and joints with hot cracks.

exclude structures from service. The upper parts of the histogram (**Figure 6**) show the proportional and numerical specification of bridges with internal cracks in welded butt splices. Of note is that the majority of cracks were found in the oldest historical bridges from the years 1936–1940. For the seven bridges from that period tested, cracks were discovered in six structures, which constitute 85.7%. They were discovered on 124 welded butt splices from the general lot of 696 pieces tested.

Similar unfavorable test results were obtained during examination of the bridges constructed over the years 1946–1950. Cracks were found in 18 structures (37.5%) on 270 X-rays (5.5%).

4. Fatigue tests on butt welds

All the cracks in the butt splices of the bridge structures are internal hot cracks. They arise in the weld metal and HAZ of a joint during the crystallization process in the liquidus–solidus temperature range (**Figure 3**).

Service fatigue cracks appear in stress concentration zones caused by structural details and increase their value by a concentration factor caused by nonmetallic inclusions. Therefore, the development of a fatigue crack depends on many factors: the shape and dimensions of a structural component as well as the way and magnitude of loading. The larger the structure, the smaller the critical crack length necessary to cause the final brittle fracture [3, 18]. Development of fatigue cracking appears in stages under fluctuating loads, and their increase is caused by the weakening of a structure's strength. This is why, in the literature, the stress is put down to the influence of the imperfection's geometry and its location in the weld.

The literature gives different models for the initiation and proliferation of fatigue cracks in a nondeterministic approach. Despite many attempts to describe the fatigue mechanism, given in 64 hypotheses [19], no general hypothesis has yet to be formulated [18, 20, 21]. We are still stuck in the phenological description, despite having more and better research tools and numerical calculations. It appears that when assessing the endurance of bridges with cracks in their welded joints, it is highly useful to analyze their service behavior and the results of laboratory tests.

In Poland, the pioneer of the in situ field testing of welded butt splices on railway bridges was Professor Andrzej Fabiszewski from the Technical University of Szczecin. The procedure understood the principle that a weld is the weakest point in welded structures. The results of these tests were a great surprise to the organizers. In 34 bridges, internal technological cracks and hot cracks were ascertained in 437 welded butt splices (**Figure 6**).

To answer the question, "What do we do with theses bridges?", laboratory fatigue strength tests were carried out on three typical structural solutions which reflected the details in the early welded bridges. Specimens U, C, and P and the test results are given in **Figure 7**. The tests were carried out on 60 specimens, each time loaded at 5 loading levels. The tests are presented more precisely in [3]. They allowed, using the least-square method, fatigue class values according to EN ISO 5817: 2014 to be estimated [22, 23]. The following fatigue classes ($\Delta\sigma_C$) were obtained for individual specimens from different constructions:

- specimens U with butt welds of an acceptable quality level, fatigue class $\Delta\sigma_C$ = 125 MPa.
- specimens C with an internal crack in the butt welds, fatigue class $\Delta\sigma_C$ = 90 MPa.
- specimens P with butt welds covered by one-sided rhombic cover plates, fatigue class $\Delta\sigma_C$ = 79 MPa.

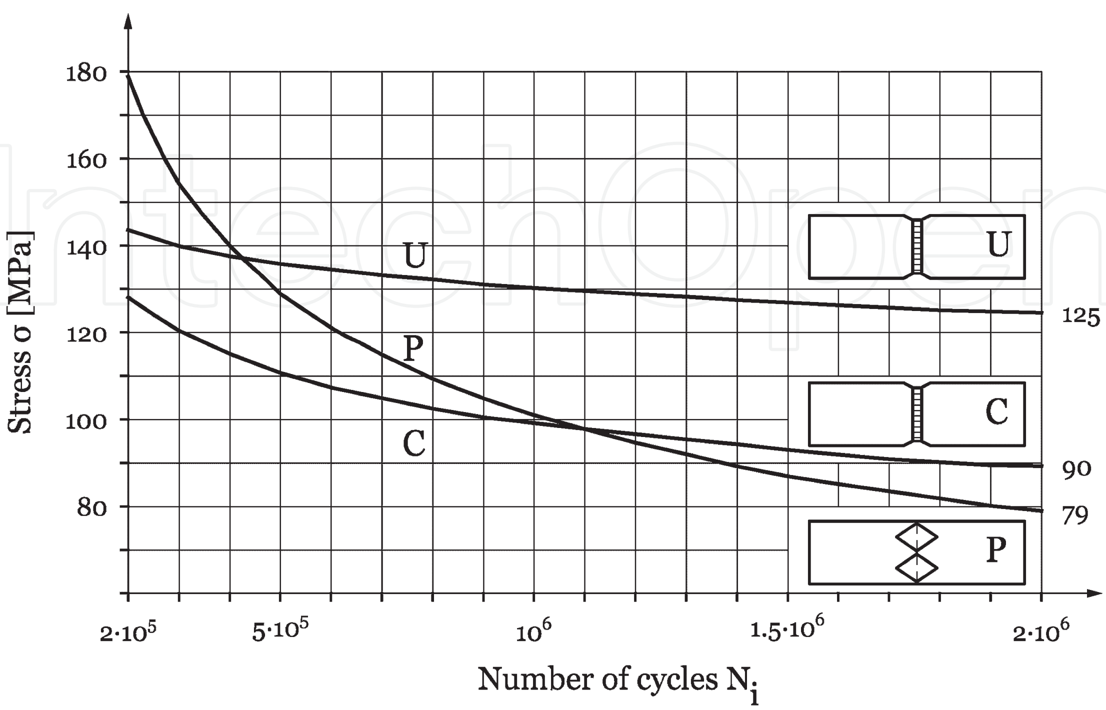


Figure 7.
Fatigue strength test results for U – Sound welds (uncracked), C – Welds with internal cracks, and P – Welds covered by one-sided rhomboid cover plates.

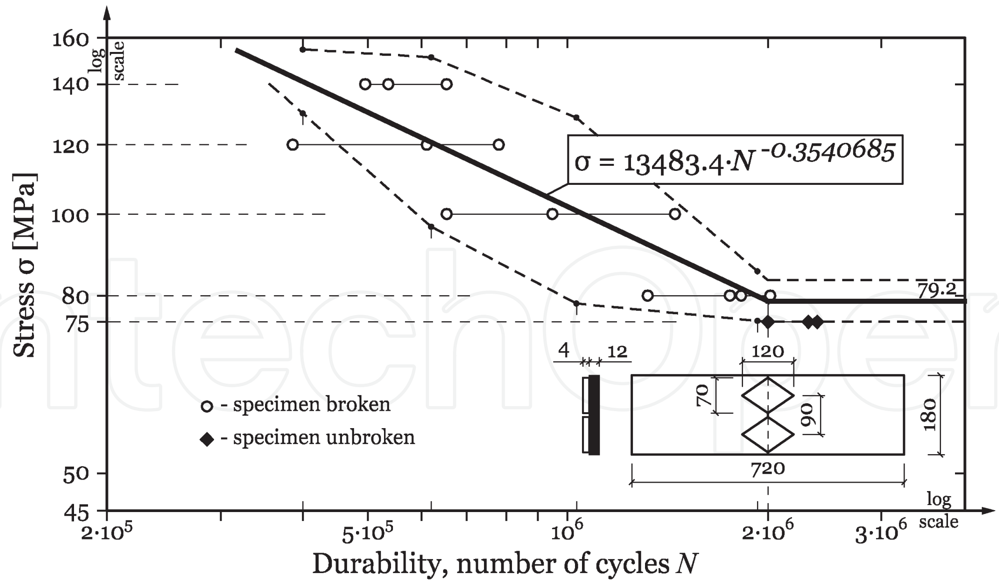


Figure 8.
Fatigue test results on 16 specimens and regression analysis.

The tests were carried out using a pulsator of frequency 5 Hz and stress ratio R = 0.1. Of note is the low fatigue class $\Delta\sigma_C$ = 79 MPa for specimens "strengthened" with rhombic cover plates. The rhombic cover plates had been intended to secure welded butt splices in early welded bridges, but the fatigue effects appeared to be quite the opposite. The results of the tests (**Figure 7**) clearly show that for the number of load cycles N_i larger than $1.1{\cdot}10^6$, the fatigue strength of the specimens with cover plates is lower than the fatigue strength of the specimens with cracked butt welds (type C).

The results of fatigue tests on 16 specimens with rhomboid cover plates give cause for reflection (**Figure 8**). Specimens with dimensions 180×12×720 mm were manufactured from Polish mild steel St3M for bridges (C = 0.19%, Mn = 0.66%) of f_y = 312 MPa and f_u = 452 MPa. The tests were carried out according to the Polish standard on fatigue tests on metals using five stress levels: 75, 80, 100, 120, and 140 MPa. The tests were performed on a pulsator with 5-Hz frequency. The first cracks appeared near the welded end of the cover plate and spread toward the specimen edges. In three specimens with stress levels 80, 100, and 140 MPa, the cracks appeared at $99{\cdot}10^3$, $168.9{\cdot}10^3$, and $20{\cdot}10^3$ cycles before total fracture. However, two specimens at stress level 80 MPa were not damaged, despite being loaded by $1819.8{\cdot}10^3$ and $836.8{\cdot}10^3$ cycles after the first cracks appeared (**Figure 9**). The test results for 13 damaged specimens allow us to work out the logarithmic regression equation.

The tests show stress concentrations by rhomboid cover plates mainly at their ends [14]. The fatigue strength value resulting from using cover plates depends on their shapes, as well as their length (**Table 1**). The lowest value is reached when the additional element is shortened up to 300 mm.

5. Stresses in welds with cracks

The fatigue behavior of metals is determined precisely in [20, 24–26]. Fatigue hypotheses, dislocation structures, fatigue cracking, and their fractures are given there. The issue of no crack growth in existing structures in service is also discussed there. Such cracks appear when the initiation crack stress is lower than the value of

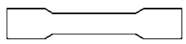
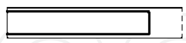
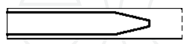
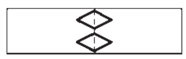
Specimen No.	Cover plate ending	Fatigue strength, MPa			Literature
		10^5 cycles	$2{\cdot}10^5$ cycles	%	
1		—	280	100	[24]
2		—	~90	32	
3		—	90–100	34	
4		182	79	28	[14]
5		234			
6		202			
7		228	79	28	[3]

Table 1.
Effect of cover plates on the fatigue strength.

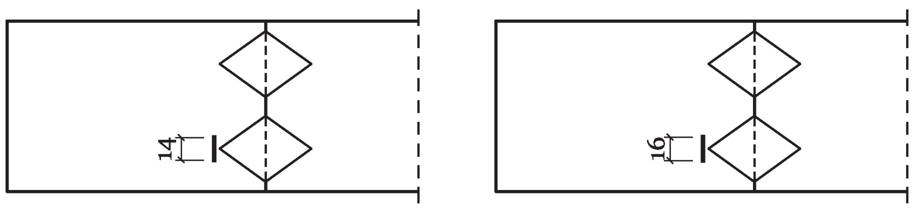


Figure 9.
Undamaged specimens after 3637·10^3 and 2827·10^3 load cycles (cracks after 1817.2·10^3 and 1990.2·10^3 load cycles).

stress necessary for crack propagation. The undamaged service of these structures and the lack of propagation cracks is because no situation has arisen during service which would lead to their appearance. Many hypotheses resulting from laboratory tests have been put forward, including the oldest tests on wagon axle models (Ø 50 mm), carried out by T.V. Buchwalter as early as 1938 [27, 28]. Generally, there is no one solution for the three-dimensional problem of fatigue fractures. However, a material experiment review laid out the directions for further research to find a more precise solution to the problem. There is the optimistic fact that as early as 1965, Kudriawcew [29] stated "structural sections in which non-propagating cracks develop may be stronger than sections constructed with notches."

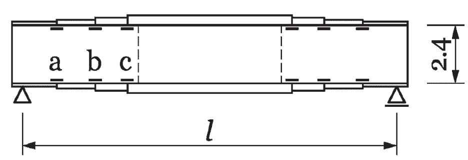
The prediction of eventual fatigue cracking in welded butt splices in the railway bridges tested was assessed on the basis of strength analysis for three selected bridges. These are plate girder bridges constructed in 1938, 1938, and 1947 on different railway lines. Their technical characteristics are given in **Table 2**. This is a compilation of requirements collated in two papers [7, 16] relating to the structures.

The analysis was undertaken on the three bridges. They were constructed either side of WWII. In total, there were six examinations of the welded butt splices, and three additional tests limited to the testing of splices with cracks. The additional examinations were executed in 5–8-year time intervals. Overall, 632 X-rays were taken and 49 technological hot cracks were discovered. The calculated stress values at the butt splice locations in the lower chords of the plate girders are given in columns 5–7 in **Table 2**. Column 8 shows the results of service stresses measured on the welded joints of bridge III located on the Katowice–Tczew coal railway line. The permanent load and the electric locomotive ST-21 (live load) are taken into account. The great similarity of stresses calculated theoretically (column 7) and stresses measured “in situ” on bridge III (column 8) are of note.

Bridge No. (year built)	Number		Span length l, m	Stress σ, MPa			
	Girder (tested)	X-rays (cracks)		Dead load	Live actions		Dead + measured
					LM 71	Service	
1	2	3	4	5	6	7	8
I (1938)	6 (2)	128 (8)	22.40	a = 10/14	81/108	47/63	
				b = 12/14	94/111	54/64	
				c = 13/15	99/114	57/65	
II (1939)	6 (1)	144 (39)	20.20	a = 14/17	116/137	68/80	
				b = 15/17	122/138	74/84	
				c = 15/17	124/137	72/80	
III (1947)	12 (3)	360 (5)	21.20	a = 9/10	80/93	47/55	50/59
				b = 10/11	89/101	52/59	52/61

Table 2.
Bridges, their technical data, and normal stresses in butt welds with cracks as well as behind cover plates.

Locations of welded butt splices:



The calculations assume the creation of a national set of standards, i.e. Polish bridge standards: PN-85/S-10030 and PN-82/S-10052 for actions and steel bridges, respectively. The characteristic values of the live action effects with dynamic factor Φ are taken into account. The standard load model in the form of the contemporary Eurocode railway traffic model LM 71 for loading class k = +2 is under consideration. The stresses were determined on the butt weld and cover plate axis as well as in the flange plates just behind the ends of the rhombic plates (values in denominators). It is easy to see that the service values of stresses in column 7 are from 60 to 70% of the stress values for the standard loading (LM 71) in column 6. This means that they are also lower than the values of $Z_{rj} = \Delta\sigma_C = 79$ MPa determined according to **Figure 7**, i.e. the service stresses are lower than the limit value for such joints determined in [3].

Considering the load spectra recommended by the JRC (Joint Research Centre) for railway bridges [1], the above value of loading will be smaller. The authors conducted such analyses and described them in [2, 30].

The load spectrum given in **Figure 10** according to the old British standard BS 153 was established in structural calculations [29–31]. The method allows the service life of bridge structures to be prolonged by as much as three times. The authors recommend this method for the endurance assessment of historical bridges.

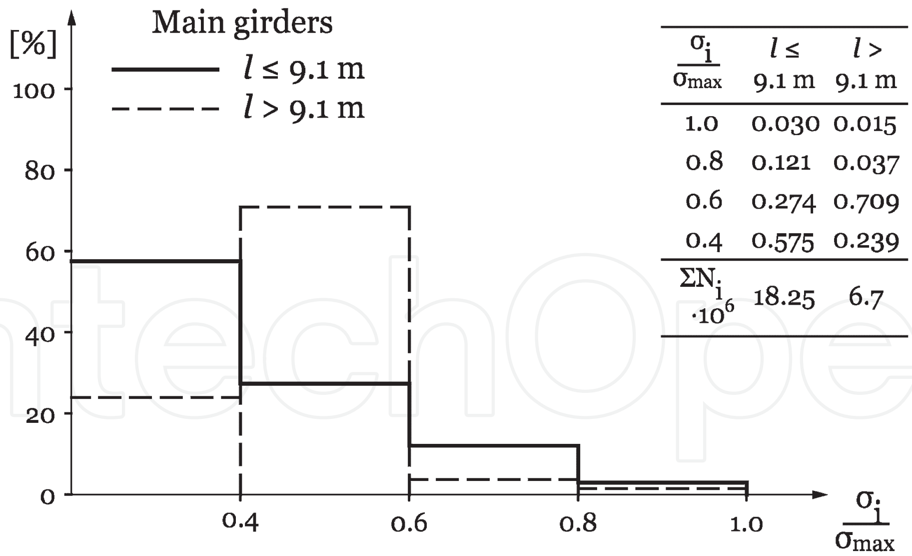


Figure 10.
Stress ratio frequency.

In some countries, the results of traffic load measurements have been published, giving the load spectra for analyzing existing bridges [32, 33]. New standards and guides for the testing and assessment of existing bridges have appeared [19, 34–37].

6. Numerical analysis of welded butt splices with cover plates in bridges

The first welded bridge designers were aware that "a weld is the weakest place in the structure." Because of welding imperfections, their resistance is lower than that of the welded material. The simplest and the most economical way to eliminate these differences seemed to be enlargement of the welded joint section by adding cover plates which compensated for the weakened section. In the welded plate girders of railway bridges constructed up to 1939 and in the period from 1945 to 1953, the butt splices of webs and flanges were covered with one- or two-sided cover plates [3].

In the 154 railway bridges which were checked radiographically, internal cracking was discovered in 438 welded joints. In this group, there were 28 plate girder structures; the constructions of their lower flanges are shown in **Figure 11**. In 18 structures, their butt splices are covered with one-sided rhomboid cover plates from the side of the girder longitudinal axis. The rhomboid cover plates are from 90 to 200 mm in width and from 160 to 340 mm long.

To assess the endurance of such types of joints, fatigue strength tests were undertaken, which were discussed in Section 4. The results of the tests and the regression line are given in **Figure 8**. The determined infinitive fatigue strength value Z_{rj} = 79 MPa at $N_i = 2{\cdot}10^6$ load cycles constitutes only 26% of yield strength f_y = 302 MPa for the steel of the specimens tested. It is worth mentioning that for three stress levels σ = 80, 100, and 140 MPa, on five specimens seven cracks appeared, as shown in **Figure 12**.

The results of the fatigue tests show a very low fatigue limit value for the welded butt splices covered with rhomboid plates. The problem was solved numerically using an FEM model as shown in **Figure 13**. More details of the numerical analysis are given in [38, 39].

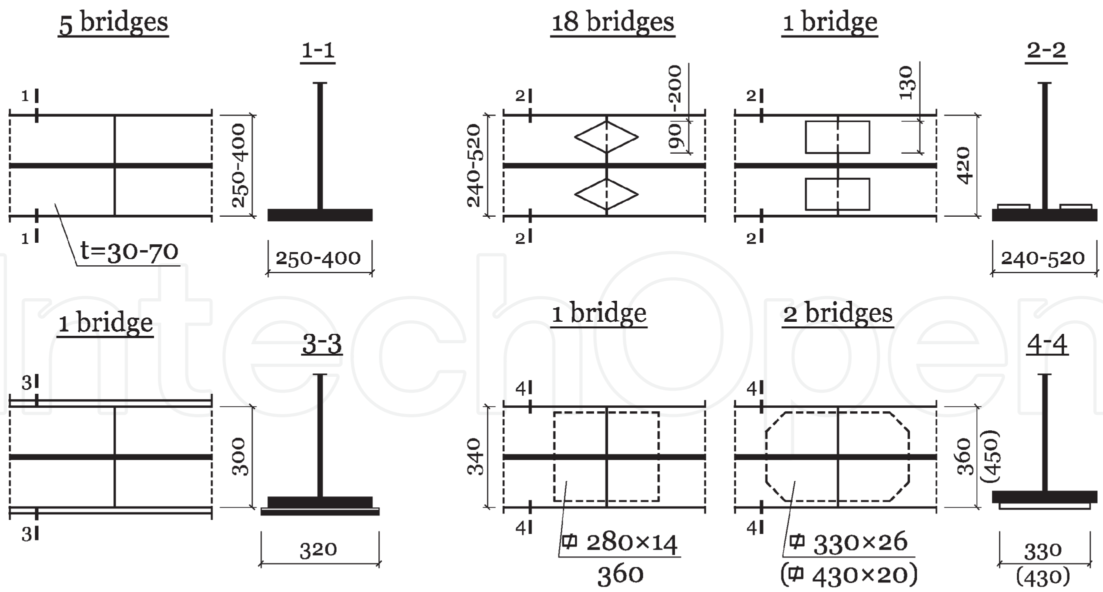


Figure 11.
Details of welded butt splices with cracks in 28 plate girder bridges.

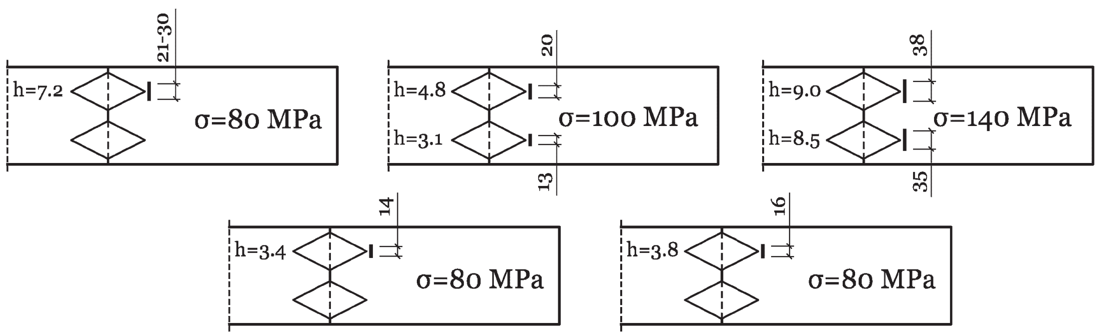


Figure 12.
Cracks in flanges with rhomboid cover plates after fatigue tests: The top three specimens – Damaged and the bottom two specimens – Undamaged.

For the numerical analysis, the welded splice was modeled using the FEM method (**Figure 14**) with Inventor Nastran software. Material parameters for structural steel are f_y = 249 MPa and f_u = 360 MPa. The stresses were calculated in four cross sections and on nine points for each section. Loading was modeled as 162, 173, 216, 260, and 303 kN tensile forces with 75, 80, 100, 120, and 140 MPa course tensile stresses in the flange. The same stress levels were formulated as for the laboratory fatigue tests.

Analysis of the tensile stresses in the welded joint allowed us to formulate some remarks:

- rhomboid cover plates do not lower the stresses in a butt weld as was initially assumed (see **Table 1**); it was expected that the resistance of the joint would be increased by 25.9%;

- cover plates appeared to be unnecessary components, causing some additional fatigue problems;

- at the ends of the cover plates, the concentration of stresses appear within a range of 1.47–1.69;

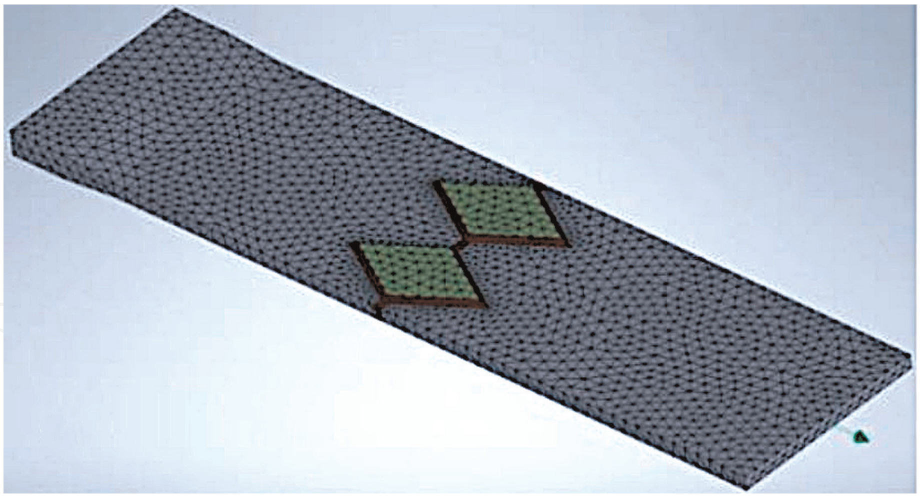


Figure 13.
Numerical model for analysis of a welded butt splice with cover plates.

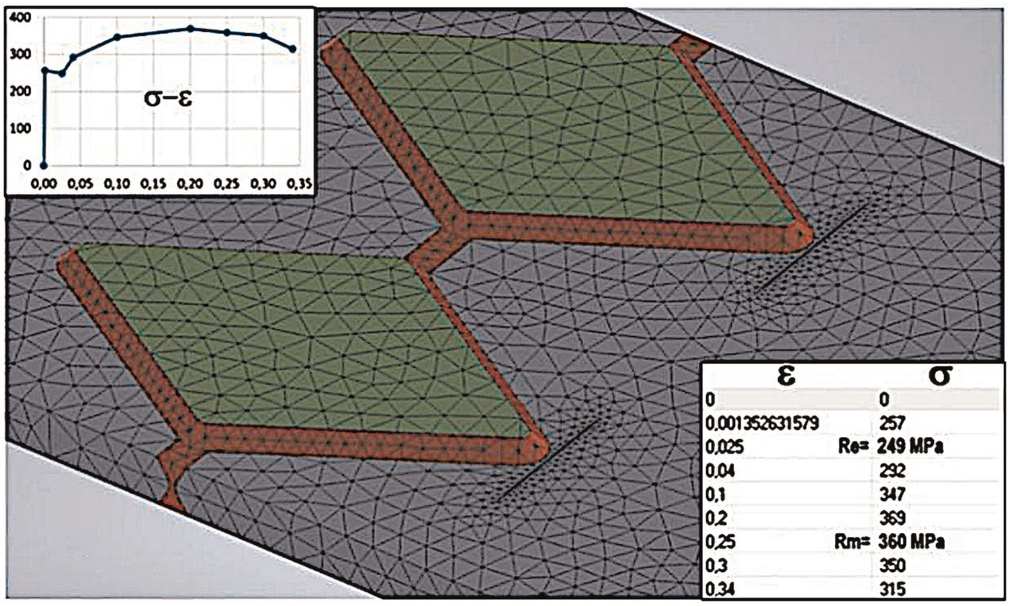


Figure 14.
Details of the numerical model with cracks and structural steel material data.

- the stress concentration together with the smallest concentration factor for globular (spherical) nonmetallic inclusions of 2.04 are the reasons for the formation of one-sided stochastic cracking already at the 80-MPa stress level (**Figure 13**); thus σ = 80.1.69.2.04 = 275 MPa which is greater than the steel yield strength f_y = 249 MPa [39].

Cracks appeared at three stress levels, σ = 80, 100, and 140 MPa, with a varied number of load cycles from 535,000 to 990,200.

These are fatigue cracks developing in stages, as opposed to the rapidly developing cracks in the fatigue tests of welded joints on specimens U, C, and P (**Figure 8**). All the cracks had a similar fracture as shown in **Figure 15**, with three developing trajectories: I – crack initiations, II – growth, and III – final fracture. The scheme of fatigue crack zones is shown in **Figure 16**.

The stress distribution on the circumference of the cracks is similar, with the smallest values in the upper zone. The values are equal to the upper values of yield

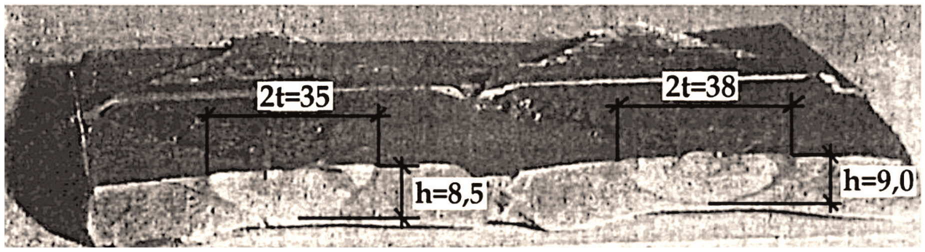


Figure 15.
*Fracture surface of a broken specimen after the fatigue testing (see **Figure 13**).*

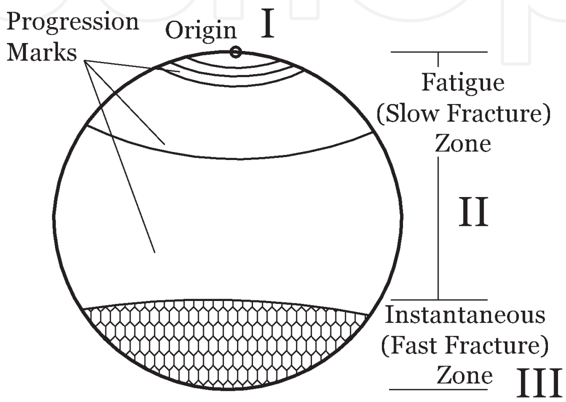


Figure 16.
Scheme of a fatigue crack: Zone I – Origin, zone II – Fatigue zone and zone III – Final fracture.

strength f_y = 280 MPa, while the maximum stress $\sigma \sim 306$ MPa appeared at the crack tip.

A study of the literature shows that no direct criterion has been established for precise cracking in zones I and II, e.g. the zones of settled crack growth, and zone III (unstable crack growth). This has not been achieved since 1913 (C. E. Inglis) despite the development of 64 growing hypotheses at the microstructure level and thousands of publications [27, 28, 40, 41]. For example, after the chapter, "Fatigue crack growth" in [27], there is a list of 469 supplementary readings. Crack growth is described there probabilistically in a way that is comprehensible only for specialists.

Considering the results of the numerical calculations of stresses in cracked joints in **Figure 17**, a new way for describing ductile fracture growth (zone II) may be suggested. The analysis takes into account two laws of physics:

- ductile fracture growth may be restrained at any moment by lowering stresses below the material's yield strength;

- the moving peaks of the upper size of the fracture with length 2 t on the top surface will first reach the elastic stress area σ = 99–100 and 100–104 MPa at stress level 100 MPa as well as σ = 172–180 and 176–180 MPa at stress level 140 MPa – see **Figure 17**. This is guaranteed by the geometry of the fracture $t/h \sim 2$ and is assured by fracture surface analysis which shows that the fracture growth is along the top surface rather than into the material.

There is a reduction in edge surface stresses on the top surface to the measured values 274–310 MPa, i.e. to the upper yield strength of the material f_{yH} = 280 MPa. The

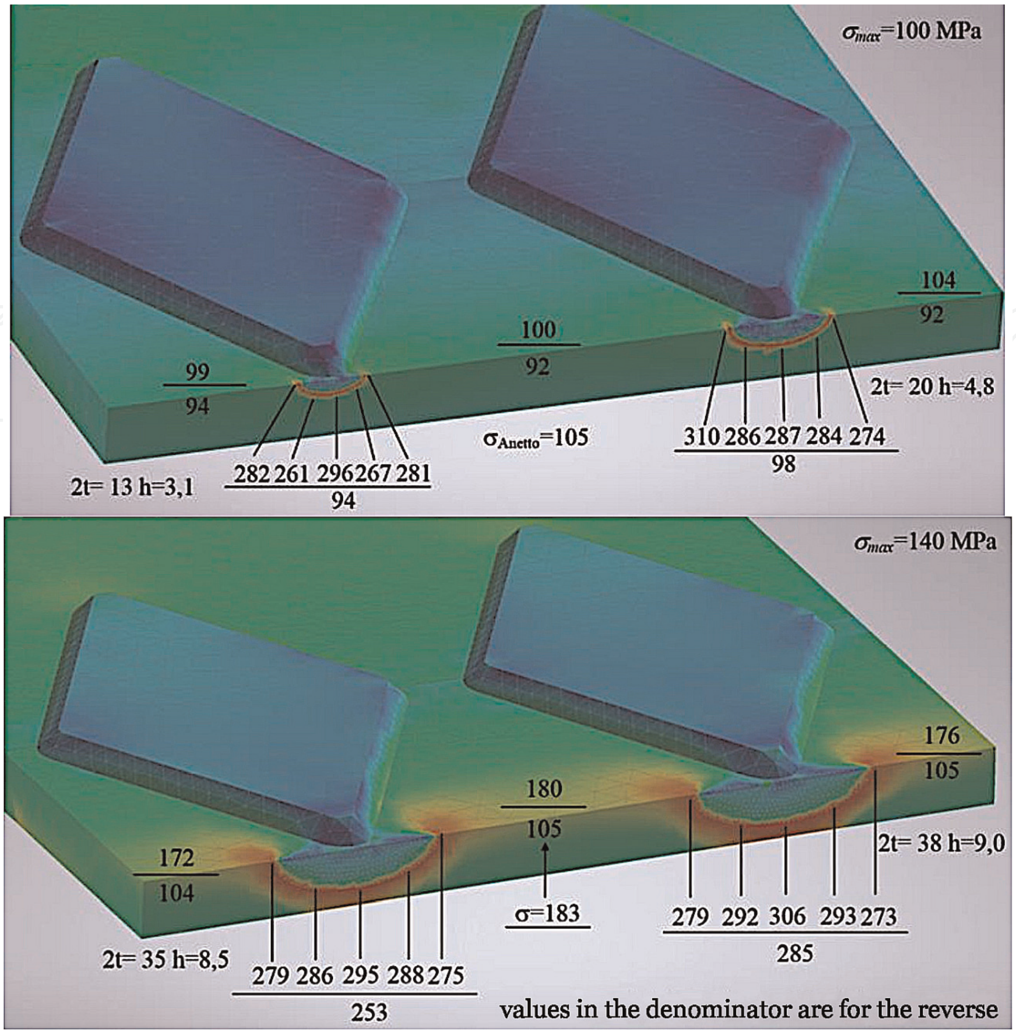


Figure 17.
Concentration of stresses at the ends of cover plates for stress levels: 100 and 140 MPa.

growth of ductile fracture disappears at the edge points on the top surface. This phenomenon evolves in the nearby "deep" points of the fracture and according to the stress equalizing rule, it gradually restrains a two-sided fracture from proceeding to the tip of the fracture. The cracking growth in zone II disappears totally.

Generally, it should be stated that no comprehensive model for a general description of fatigue fractures has yet been devised. All models described in the literature relate only to growth zones I and II. The only known model for transition from fracture zone to final fracture, zone III, was devised by A.H. Cottrel and N.J. Petch [40, 42]. The Cottrel–Petch theory describes the ductile-brittle transition properties of steel. A basis for the transition is assumed yield strength σ_{pl}.

When the yield strength is larger than the fracture growth stress, then the material is brittle and vice versa. "Brittle fracture will occur when the work of applied stress σ during fracture growth reaches the effective energy of newly formed surfaces." This means that brittle fracture will occur under stress $\sigma = \sigma_{pl}$ (**Figure 17**).

An explanation for this phenomenon in relation to the five cracks in the three damaged joints (**Figure 12**) is given in [38]. In the analysis, the results obtained during fatigue strength tests for three types of joints were used (**Figure 7**). Practically, this applies to the infinitive fatigue strength values Z_{rj} given as a function of load cycles N_i.

7. Conclusions

During their service life, bridges, like other structures, undergo functional and material degradation. In many cases, tests show that despite non-strict conformity with actual design standards, they are able to carry actual service loadings. According to [1], calculations of existing steel bridges resistance should be carried out more precisely and in a more readable manner following a three-phase assessment: a preliminary evaluation, a detailed investigation, and an expert investigation. Some other studies have recommended using five assessment levels: a bridge rating, a preliminary evaluation, a detailed investigation, an expert investigation, and advance testing [16, 35–37]. Application of the assessment procedure led to improvements in knowledge and a calibration of confidence factors.

At the same time, the project “Sustainable bridges – assessment for future traffic demands and longer lives” with nine packages, deals with the increasing capacity and service life of existing railway bridges [35]. New methods for the resistance assessment of existing bridges were developed in standards and guidelines using calibrated partial safety coefficients.

The Steel Structure Chair at the Technical University in Szczecin (now WPUT) contributed to these issues by carrying out a multiyear program for the quality-level assessment of welded butt splices in existing railway bridges in service in Poland. Radiological tests on butt splices in the steel girders of around 200 railway bridges were carried out. Nearly, 200 bridges were tested using X-ray examination over a 37-year period. The number of joints tested was over 15,000; cracks were discovered in 400 welded joints in 34 bridges tested. Repeated tests on welded joints with cracks were undertaken as well as laboratory investigations for their complex assessment. Partial results of the laboratory fatigue testing are given in the paper.

Numerical analysis of tensile stresses in welded splices was undertaken to support and confirm the results of laboratory fatigue tests. The results of the FEM analysis of tensile stresses in welded joints with rhomboid cover plates are given. The FEM method allowed us to determine tensile stresses at each service stage. The analysis allowed for:

- negative assessment of the welded joint type with cover plates – total disqualification of cover plates;
- suggesting a new hypothesis for the initiation and growth of the surface fatigue fractures;
- conformity assessment for the Cottrell–Petch theory for the transition from ductile fracture to brittle state at the final fracture.

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