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Review

Anomalous properties of steel of old railway bridges in the light of diagnostic tests

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Abstract: The procedure for assessing the properties of steel in exploited bridge structures is one of the most important issues in the comprehensive assessment of the load capacity and durability of bridges used for a longer period. The paper is an attempt to recapitulate conclusions from many years of research on these structures. Literature and own research have shown that many of these structures do not strictly transfer design loads, but safely transfer the actual loads occurring on a given railway line. The new rules for assessing such bridges will allow to estimate their operational usefulness despite the progressing material degradation. It is necessary to determine the safety factor, especially when it concerns several objects globally at the same time. The authors' dream is to comprehensively interpret Polish research and to develop an appropriate standard to estimate the real pragmatic loads.

Keywords: steel properties; welds cracks; old railway bridges

Introduction

Literature quality requirements for steel in old Bridges. Due to the natural decrease in the load-bearing capacity of old bridges used by long-term users, their required further durability according to the design criteria seems unjustified. It is justified to develop new methods for assessing their load-bearing capacity. In the 21st century, between 2004 and 2011, five countries (Canada, the United States, Great Britain, Switzerland and Denmark) adopted a new probabilistic method for assessing the load-bearing capacity of old bridges. It is a method of separated, calibrated safety factors with reduced requirements for newly designed structures [1,2]. The analysis of the load-bearing capacity of these bridges should be a process enabling, as necessary, an increasingly accurate and more reliable estimate of their load-bearing capacity. The authors presented these issues with reference to Polish regulations in the article [3]. All comments included there are consistent with the recommendations of the studies [4,5], that is, based on the Joint Committee on Structural Safety (JCSS) procedure and published in 2001 [6]. The following new comments on the topic discussed in [5], unknown to the authors, deserve attention and discussion:

1. "The stress ranges of the design spectrum do not contribute to cumulative damage if they are below the cut-off limit of 29 MPa", two examples are given at m=5;
2. „The yield strength and tensile strength are higher in the web of beams than in their flanges" - according to EN 10025;
3. In the case of weldable steel (puddled iron): "Chemical analysis must be performed by wet analysis because it is impossible using emission spectroscopy", no justification.

The authors of studies [4,5], as employees of the Federal Institute for Research and Testing of Materials in Berlin (BAM), suggest at the same time using all information on the strength of structures that have been previously tested, at best obtained directly from the tested structure, and when the data significantly differ from today's standards. Surprisingly, the opposite was done in the expert opinion on the steel bridge from 1885 on the Radom-Tomaszów Mazowiecki railway line. The author of the expert opinion obtained average values from longitudinal and transversely rolled strip samples from the bridge tests, ReH = 238 MPa and Rm = 378 MPa. However, to analyze the bridge's load-bearing capacity, he adopted the value ReH = 0.74 * 235 = 174 MPa, which: "guarantees the maximum level of safety of the analyzed facility." Ultimately, the bridge was dismantled and its material and strength analysis were presented in two publications in technical journals. In the second case of publication, the author of BW, as a reviewer, carried out a thorough refutation of the content of the article, showed a mistake and denied its printing. However, the article was undoubtedly printed. The question arises: why did the "titular author and reviewers" and the Editorial Office, knowing exactly the situation, make such an unquestionable decision? Doesn't this fact qualify for the Guinness Book of Records?

According to the authors of the article, a model, priority analysis of the designed bridges was presented by the contractors of two structures on the Sava River near Zagreb in Yugoslavia, put into operation in November 1939. These were the objects shown in Fig. **1÷3** [7]:

a) road bridge with a length of 220 m (spans: 54.6 + 2 * 55.075 + 54.6) made of St44 steel,
b) double-track railway bridge, 306 m long (spans: 57.50 + 135.54 + 58.00 + 55.00). The load-bearing girders are riveted from St52 steel, and the wind girders, hangers and roadway are welded from St37 steel.

Fig. 1. Road bridge over the Sava River near Zagreb [7]

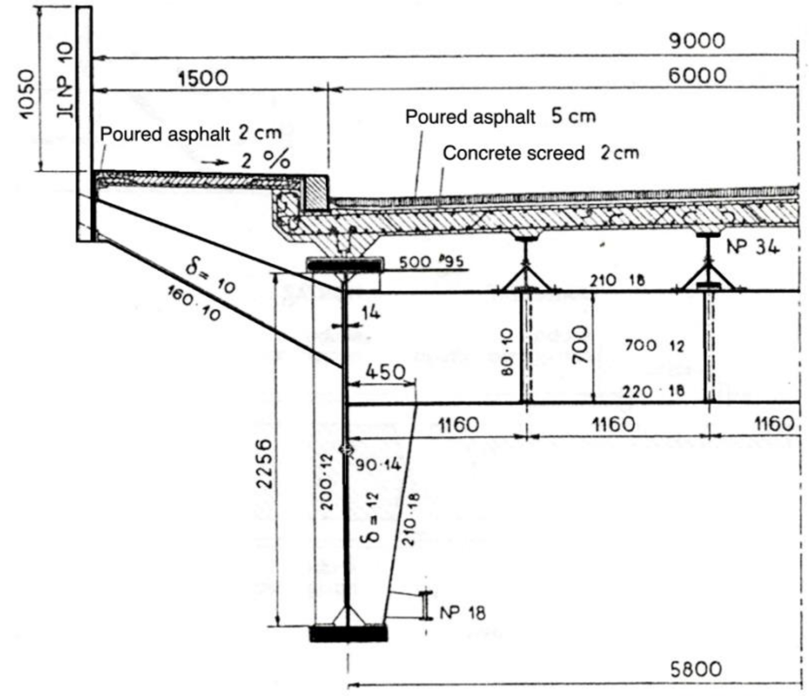


Fig. 2. Bridge cross-section (1938)

Fig. 3. Railway bridge Hendrix Bridge over the Sava River (1937) [7]

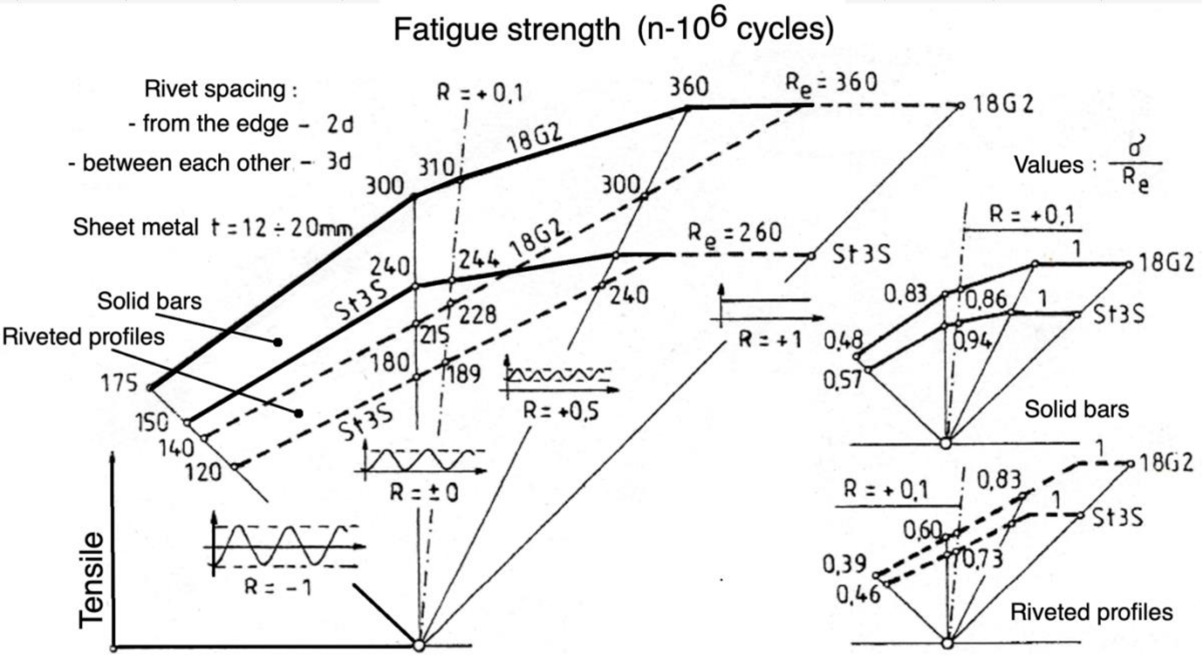


Fig. 4. Fatigue strength of bars (1925)

The preparatory analysis was carried out for 12 years (1925-1937), and the bridges were built within 3 years. The study [7] discusses in detail: material and technical properties, general construction assumptions and construction details of bridges. It is worth noting; testing of annealed bars in a stress-free state (620 °C), tested at - 16 °C and normalized (940 °C). Fatigue tests according to Fig. 4 were carried out with various types of loads, at R = -1; 0; +0.5; +1 and the number of load changes n = 10^6 cycles, according to the EMPA method. It is easy to see that with the cycle amplitude coefficient R = +0.1, the fatigue strength for solid bars is 30 to 35% higher than Z_{rj} for riveted bars.

It is regrettable that due to the different values of the number of load cycles, it is not possible to compare the values in Fig. 4 with the results obtained according to the EN ISO 5817: 2014 standard (Annex C) and presented in items [2,4]. According to this standard: "The FAT fatigue class value is the range of stress variation ◎◎c with respect to a 2 million cycle number Nc established at a 75 percent mean confidence tolerance limit with a 95 percent survival probability." The aim of this work is to synthetically present information related to the issue of abnormal steel properties that deviate from the general rule and to provide aspects affecting the correct further operation of historical steel bridges. The publication will not replace expert advice in assessing their durability, but it will enable discussion of the problem during the expert opinion and may influence the final assessment of the technical condition of the structure. The final assessment of steel consists in determining whether it still meets the requirements of industry standards and regulations in its post-service state. Unfortunately, we often encounter a lack of appropriate technical and technological knowledge, especially necessary when assessing structures that do not meet the criteria of 100% safety [3,8]. This is a global problem, and in relation to bridges it is complicated by their functional aging and structural aging of steel. There is a need to harmonize various procedures and establish general recommendations for their safe use. Information obtained from in situ examinations of old objects and laboratory tests of their structure may be useful in this analysis. The article is an attempt to draw conclusions from the authors' long-term research on the impact of aging on the durability of these structures [9,10].

Properties of aged steel from which railway bridges were built

All analyzed bridges (about 40) were made of cast steel in the years 1857-1983. According to literature data, the precursor of introducing new types of structural steel was the German metallurgy. Professor Rudolf Albrecht in [11] states that cast steel began to be used in the construction of bridges already in 1856. This fact is confirmed by English-language literature [12] and current [1,5]. The development of metallurgical processes (Bessemer and Thomas converters and Siemens-Martin furnaces) contributed to the production of new types of steel, especially in the second half of the 19th century and at the beginning of the 20th century

[4,5]. Taking this fact into account, in this study, the steel analysis of existing structures was carried out separately for:

- bridges built in the second half of the 19th century,
- for objects from the 20th century.

Material of these structures as a function of their operating time. Material testing of steel in old bridges usually comes down to determining the chemical composition, tensile testing, and fracture work measurements. However, the comparative level of changes in material properties is the comparison of their current properties with the properties from the period of construction of the object, which we obtain by normalizing annealing; in the article these are values enclosed in brackets. We heat the samples at a temperature of 930 °C (when the amount of carbon C<0.26%) for one hour and cool in the air. In this case, the smallest possible grain size and properties from the period of construction of the object are obtained [13]. Results of the above chemical and mechanical tests of steel of 9 railway bridges; five from the 19th century and four from the 20th century are presented in the three tables I÷III below.

Tab. I Chemical composition of the analyzed steel 9 bridges

Aggregate data

Century	Year	Number of bridges	Chemical composition. wt. %					
			C	Mn	Si	P	S	Cu
1	2	3	4	5	6	7	8	9
XIX	1875 -1890	5	0,016 -0,258	0,319 -1,409	0,000 -0,803	0,026 -0,088	0,016 -0,043	0,029 -0,459
XX	1920 -1983	4	0,084 -0,150	0,390 -0,676	0,006 -0,169	0,006 -0,045	0,011 -0,034	0,039 -0,156

The chemical composition of the analyzed bridges given in Table I shows a large variation in the content of individual elements. The contents of the three basic elements are: carbon from 0.016 to approximately 0.26%, manganese from 0.32 to 1.41%, and silicon from 0.00 to 0.803%. At the same time, phosphorus and sulphur except for one case (0.09%), it is less than 0.05%, i.e. the value considered harmful [14]. In extreme cases, the phosphorus content is 0.09% and sulphur only 0.043%. Until recently, for the authors, a bridge phenomenon was a railway structure made of weldable steel with a trace carbon content, elusive by measurements, built in Czechoslovakia in 1852. Surprise in 2020, we found similar steel in two bridges. The properties of these steels are presented in Table IV. According to [14], as the carbon content in steel decreases, the amount of oxygen increases during its production. Therefore, more Al and Si should be added to the metal bath, causing an increase in the content of non-metallic inclusions. These are aluminum oxides, silicates and spinels, which include SiO2, Al2O3, MnO and certain amounts of FeO and CaO, MgO, TiO2. The elongation of non-metallic inclusions during rolling causes anisotropy of ductility and impact strength, i.e. different physical properties in different directions.

Table II The results of mechanical tests and impact toughness KV of steel from 5 bridges from the XIX century

No. of bridge	Construction year	C %	R_{eH} MPa	R_m MPa	A_5 %	Impact toughness KV, (J)				
						-20 °C	-10 °C	0 °C	10 °C	20 °C
1	2	3	4	5	6	7	8	9	10	11
1	1875	0.258	343 (337)	548 (543)	27 (25)	11.9 (25.9)	15.9 (35.7)	28.6 (48.0)	41.8 (55.4)	53.9 (66.3)
2	1875	0.147	376 (365)	558 (554)	28 (28)	10,6 (57.1)	19.1 (76.2)	25.6 (124.6)	36.8 (133.6)	47.2 (140.5)
3	1882	0.030	305 (265)	389 (376)	28 (26)	7.4 (24.2)	- -	10.9 (63.4)	- -	47.8 (230.6)
4	1887	0.028	252 (260)	381 (388)	29 (39)	3.9 (8.5)	4.6 (32.9)	5.5 (39.6)	8.2 (60.7)	12.0 (116.2)
		0.037	259 (285)	387 (408)	38 (39)	3.8 (12.6)	5.8 (19,7)	6.3 (37.7)	9.0 (65.4)	13.9 (107.4)
5	1890	0.016	220 (258)	359 (373)	34 (35)	5,3 (14.4)	8.3 (17.3)	10.7 (52.1)	13.0 (86.7)	28.2 (283.3)

Aggregate data										
$\sum_{1}^{5}$	1875 -1890	0,016 -0,258	220 -376	359 -558	27 -38	3,8 -57,1	4,6 -19,1	5,5 -28,6	8,2 -41,8	12,0 -53,9
$\sum_{(1)}^{(5)}$	1875 -1890	(0,016) - (0,258)	(258) - (365)	(373) - (554)	(25) - (39)	(8,5) - (57,1)	(17,3) -(76,2)	(37,7) - (124,6)	(55,4) (133,6)	(66,3) - (283,3)

Table III The results of mechanical tests and impact toughness KV of steel from 4 bridges from the XX century

No. of bridge	Construction year	C %	R_{eH} MPa	R_m MPa	R_{eH}/R_m	Impact toughness KV w J -20 °C	-10 °C	0 °C	10 °C	20 °C
1	2	3	4	5	6	7	8	9	10	11
6	1925	0.150	244 (325)	376 (448)	0.65 (0.72)	5.6 (6.5)	7.2 (13.0)	12.1 (22.7)	24.7 (33.4)	37.7 (57.0)
6	1935	0.150	268 (289)	423 (440)	0.63 (0.66)	8.2 (21.4)	9.2 (32.4)	13.9 (41.3)	22.1 (62.8)	27.7 (87.4)
6	1938	0.084	220 (275)	356 (376)	0.62 (0.73)	9.8 (185.0)	13.6 (245.9)	15.8 (212.7)	28.4 (262.3)	56.0 (244.4)
6	1983	0.140	286 (328)	430 (456)	0.67 (0.72)	21.3 (130.6)	25.8 (137.0)	28.6 (160.0)	52.0 (164.8)	132.6 (177.5)
Aggregate data										
$\sum_{6}^{9}$	1825 -1983	0.084 -0.150	220 -286	356 -430	0.62 -0.67	5.6 -21.3	7.2 -25.8	12.1 -28.6	22.1 -52.0	27.7 -132.6
$\sum_{(6)}^{(9)}$	1925 -1983	(0.084) -(0.150)	(275) -(328)	(376) -(456)	(0.66) -(0.73)	(6.5) -(185.0)	(13.0) -(245.9)	(160.0) -(212.7)	(33.4) -(262.3)	(57.0) -(244.4)

Table IV. Chemical and mechanical data of bridge steels with a trace content of C

No.	Construction year	Location and destination	Components (steel)	Chemical composition. wt. % C	Mn	Si	P	S	R_{eH} MPa	R_m MPa
1	2	3	4	5	6	7	8	9	10	11
1	1852	Duisburg (Railway bridge)	- (welded)	0	0.06	0.17	0.156	0.03	229	347
2*	1908	Poznań-Bydgoszcz (Railway bridge, km 103.78)	L100*12 (cast)	0.0042	0.529	0.035	0.06	0.019	240	375
3	1927	Szczecin (Road viaduct) Druckiego-Lubeckiego Str.	C280 L100*11 (cast)	0 0	0.396 0.536	0.022 0.205	0.025 0.014	0.023 0.03	320 334	421 428

* Concentration of elements from subsequent hardfacing: 4, 6 i 12 (content C= 0,0053 ÷ 0,0018 ÷ 0,0055%, $C_{average}$=0,0042%)

As the volume fraction of inclusions increases, the work of breaking and the ductility of the steel decrease exponentially. The elongation of the sample in a tensile test is the largest if the direction of elongation of the inclusions coincides with the axis of the strength sample, and the smallest if the inclusions have the shape of plates and are perpendicular to the axis of the sample. According to technical literature [14,15], the current development of technology makes it possible to produce steel with a negligible content of inclusions and the possibility of adjusting their shape (pure steel), preferably a globular shape, which causes the smallest increase in stress by 2.06 times, regardless of the diameter.

Carbon is the basic component of steel; microstructure and properties depend on its content Strength and hardness increase with increasing carbon content, while ductility, crack resistance, weldability and machinability decrease (Table IV, Fig. 5, 6). Carbon increases strength and hardness the most. The influence of the carbon content dissolved in iron on the yield strength of ferrite is shown in Fig. 5, and the influence of carbon content and CEV (Carbon Equivalent) on the formation of cracks in the HAZ (Heat Affected Zone) is shown in Fig. 6, according to [14].

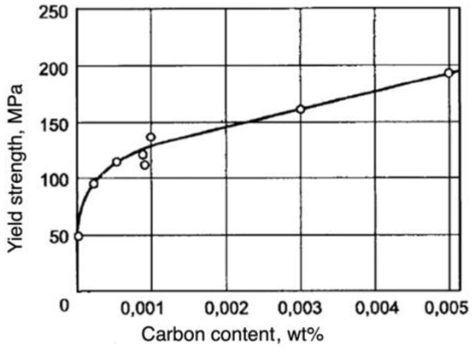
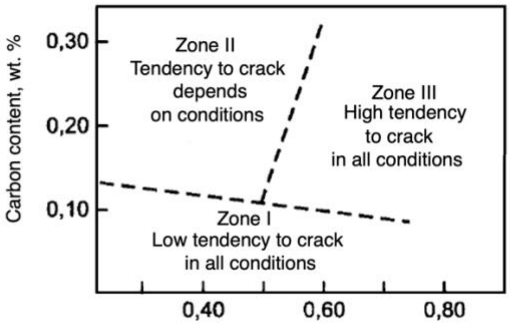


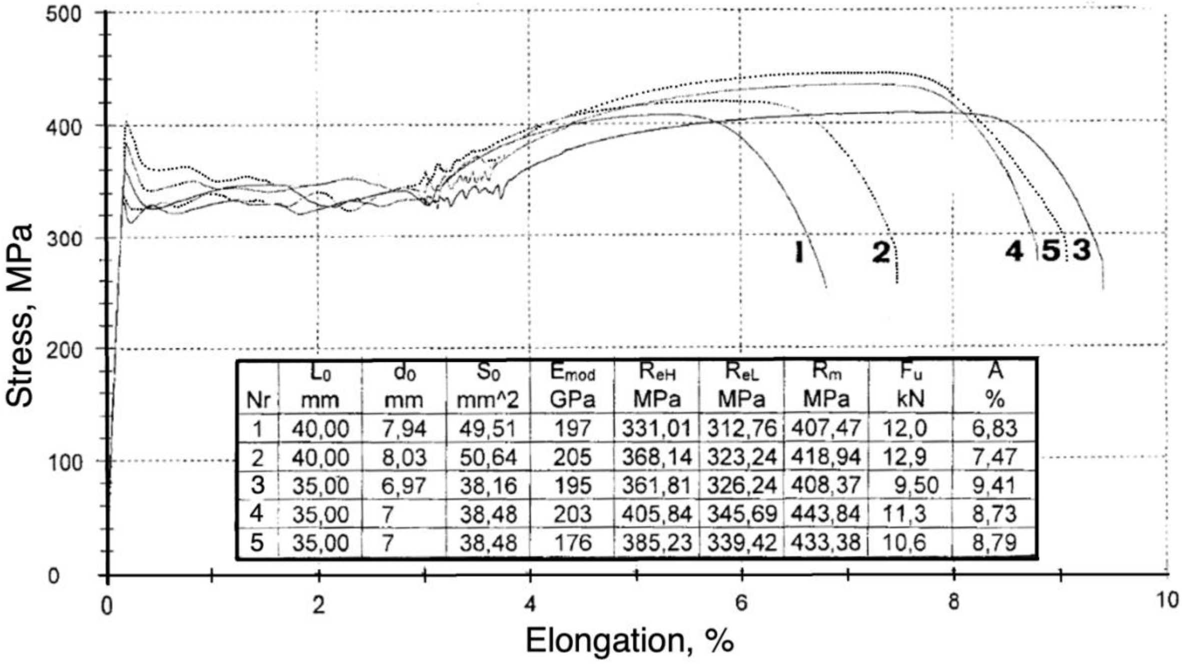
Fig. 5. The influence of a very low carbon content on the yield point of iron [14]



$$CEV = C + \frac{Mn}{6} + \frac{Ni + Cu}{15} + \frac{Cr + Mo + V}{5}$$

Fig. 6. The influence of C and CEV content on cracks formation in the HAZ [14]

The authors encountered the previously discussed property of directionality of elongation of steel with a ferritic-pearlitic microstructure during static strength test of samples during tests of the railway bridge on the Tczew-Kostrzyn line [16]. A very low value of percentage elongation after breaking all tested samples was obtained, A5 =(l/lo) 100%, ranging from 6.83 to 9.41% - Fig. 7. The existing bridge standard PN-82/S-1052 (Polish Norm) for steel of all grades recommended A5 values greater than 22%, and the EN 10025-2: 2004 for steel with t≥3mm and ≤250mm provides for a minimum percentage elongation after tearing in the range of 26÷17% depending on the type of material. The PN-54/H–84021 standard for St37S steel provided for a characteristic value of A5 = 25%.



Nr	L_0 mm	d_0 mm	S_0 mm^2	E_{mod} GPa	R_{eH} MPa	R_{eL} MPa	R_m MPa	F_u kN	A %
1	40,00	7,94	49,51	197	331,01	312,76	407,47	12,0	6,83
2	40,00	8,03	50,64	205	368,14	323,24	418,94	12,9	7,47
3	35,00	6,97	38,16	195	361,81	326,24	408,37	9,50	9,41
4	35,00	7	38,48	203	405,84	345,69	443,84	11,3	8,73
5	35,00	7	38,48	176	385,23	339,42	433,38	10,6	8,79

Fig. 7. The results of tensile tests of samples from bridge girder [16]

This issue is thoroughly explained by prof. Marek Blicharski in publication [14], when describing the directional properties of steel. Sometimes ferrite and pearlite are grouped into bands parallel to the direction of rolling and arranged alternately. The formation of a band microstructure is caused by the interdendritic segregation of manganese during the crystallization of steel. Banding strongly affects the properties of steel only in the direction perpendicular to the bands, in the normal direction. This is the cause of the anisotropy of steel properties, just like non-metallic inclusions. This impact is insignificant in the case of spherical inclusions, as evidenced by the recommendations of the EN 10025-2: 2004 standards, which provide for S275 steel grade minimum elongation A5=23% for t≥3mm and ≤40mm

Normalizing annealing of steel

According to the authors, the most subjective way that proves the extent of structural aging of steel is the simulation (pretending) of the initial structure of the material using normalizing annealing [13-15]. For this purpose, samples are heated at 930 °C for one hour and cooled in air. In this case, the best metallurgical properties of steel from the period of construction of the facility are obtained. Steel with the smallest possible grain size, increasing the yield strength and lowering the brittle transition temperature, estimated by the impact toughness method. The impact test is used to classify steel upon acceptance, it is a qualitative test. The results of impact tests, unlike Re and Rm, cannot be considered in strength calculations. Impact toughness tests are only used to compare the resistance of different steel grades to cracking - Tables II and III and Fig. 8.

Fig. 8 shows the values of the breaking work KV in five temperature ranges, each limited to two extreme bridges from the 19th and 20th centuries. For bridges 4 and 6 with the least aging and for bridges 5, 8 with the greatest aging. A surprising regularity of aging occurs in the St37-21 steel of bridge No. 8. This comment applies to negative temperature values for this bridge. There was an 18-fold reduction in the KV value over 76 years of its operation.

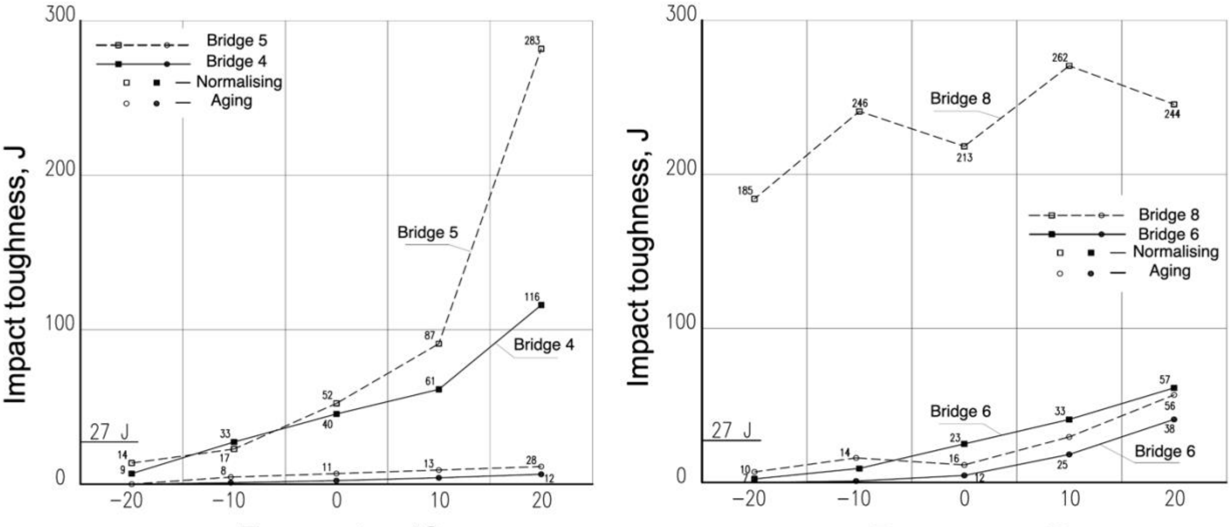
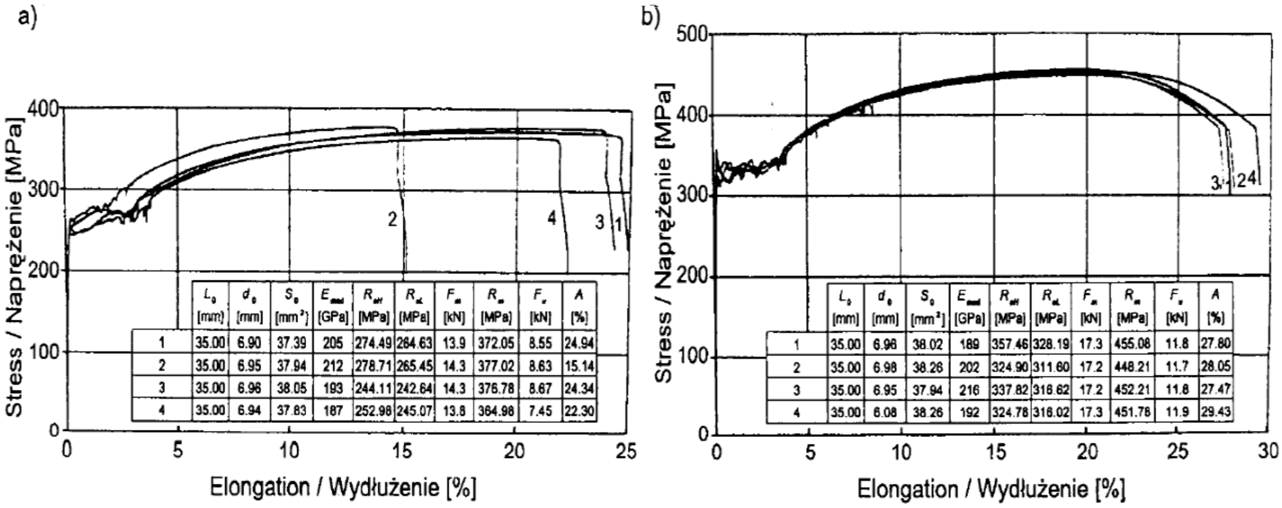


Fig. 8 Impact toughness: a) bridges 4 and 5 from the XIX century, b) bridges 6 and 8 from the XX century

Conclusions

The results of testing the ReH and Rm values in Tables II and III showed that the effect of spontaneous aging is significant in cast steels with low carbon content, less than 0.10%. The degree and rate of aging are important for any steel, even within the same grade (Figure 9). However, researchers have never obtained a value lower than ReH = 220 MPa. This value of ReH = 220 MPa and Rm = 320÷380 MPa was recommended by the International Union of Railways UIC in 1986 [17]. Eurocode 3 (EN 1993-1-10: 2007) was also right to recommend that the lowest value of the breaking work KV ≥27 J be used in the selection of steel for new structures, and not to use it in existing structures (see Fig. 8).



a)

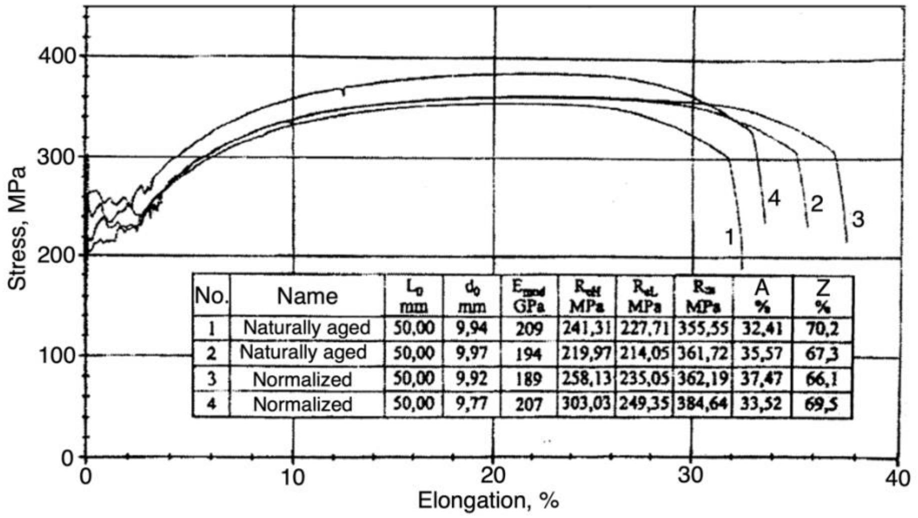
	L_0 [mm]	d_0 [mm]	S_0 [mm²]	E_{mod} [GPa]	R_{eH} [MPa]	R_{eL} [MPa]	F_m [kN]	R_m [MPa]	F_u [kN]	A [%]
1	35.00	6.90	37.39	205	274.49	264.63	13.9	372.05	8.55	24.94
2	35.00	6.95	37.94	212	278.71	265.45	14.3	377.02	8.63	15.14
3	35.00	6.96	38.05	193	244.11	242.64	14.3	376.78	8.67	24.34
4	35.00	6.94	37.83	187	252.98	245.07	13.8	364.98	7.45	22.30

b)

	L_0 [mm]	d_0 [mm]	S_0 [mm²]	E_{mod} [GPa]	R_{eH} [MPa]	R_{eL} [MPa]	F_m [kN]	R_m [MPa]	F_u [kN]	A [%]
1	35.00	6.96	38.02	189	357.46	328.19	17.3	455.08	11.8	27.80
2	35.00	6.98	38.26	202	324.90	311.60	17.2	448.21	11.7	28.05
3	35.00	6.95	37.94	216	337.82	316.62	17.2	452.21	11.8	27.47
4	35.00	6.08	38.26	192	324.78	316.02	17.3	451.76	11.9	29.43

Fig. 9. Tensile charts of steel (C= 0,025 %) of railway bridge from 1873 on line 353, a) aged steel, b) normalized steel

The "American record holders" [18] for S235JR steel obtained values of ReH=340 MPa and its reduction as a result of aging to Re=235 MPa, i.e. by 30.9%. The authors of this article found the impact of maximum steel aging in the bridge on the railway line No. 353, over the isthmus of the Pakowski Reservoir, in accordance with Fig. 9. The value of the yield strength decreased from ReH=325 MPa to ReH=253 MPa, i.e. by 22.2%. In other cases, these values were lower - see Fig. 10. On average, according to own research (Tables II and III), the operational decrease in the ReH value in the steels of the discussed bridges is 32.8 MPa, i.e. approximately 15% of original value. This is also the ReH value of the steel (C=0.016%) of bridge No. 5 from 1890, located online No. 3 Warsaw-Kunowice at km 26.806. The obtained values of ReH=258 and 220 MPa allow us to estimate a reduction in the yield strength, during the period of current operation, by 38 MPa, which constitutes 14.7% of the ReH value of normalized samples.



No.	Name	L_0 mm	d_0 mm	E_{mod} GPa	R_{eH} MPa	R_{eL} MPa	R_m MPa	A %	Z %
1	Naturally aged	50.00	9,94	209	241,31	227,71	355,55	32,41	70,2
2	Naturally aged	50,00	9,97	194	219,97	214,05	361,72	35,57	67,3
3	Normalized	50,00	9,92	189	258,13	235,05	362,19	37,47	66,1
4	Normalized	50,00	9,77	207	303,03	249,35	384,64	33,52	69,5

Fig. 10. Tensile charts of steel (C= 0,016 %) of bridge no 5

Conflicts of Interest: The authors declare no conflict of interest.

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