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# METHOD OF ASSESSING FATIGUE HAZARD TO STEEL RAILWAY BRIDGES

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This research has been carried out as part of the ministerial R & D program: "Improvement of the condition of roads and highway and railway bridges". This paper deals with the establishment of guidelines relating to "Checking the fatigue hazard to railway bridges". Its aim is to provide a relatively simple and coherent method of assessing the fatigue hazard to steel structural elements in both newly designed bridges and those being in service under different conditions of traffic on Polish Railways lines. This is proposed to be done on the basis of the latest results of fatigue studies. Another aim is to develop a method of assessing the allowable – from the point of view of fatigue – the service life of railway bridges in service. Finally, examples of checking the fatigue hazard to structural elements of railway bridges by means of the proposed method are given.

## 1. Introduction

Structural elements of railway bridges, due to the character of the loads acting on them, are subject to fatigue which in extreme cases leads to the appearance of cracks, usually at different structural joints [2]. To prevent this, and thus to avoid closing the bridges to traffic, fatigue should be taken into account already when a bridge structure is designed [7]. In the case of bridges in service, it is necessary to detect a fatigue hazard early enough in order to take appropriate preventive measures (including bridge replacement).

In contrast to the standards of most of the European countries, the design standard PN-82/S-10052 [6] being in force now in Poland takes into account fatigue but it is not based on the state of the art in the field of service strength (it is based on the stress ratio, differentiates between various kinds of steel but it does not take into account the kinds of elements and the static scheme, giving only limited consideration to the span of the elements, the load spectrum, etc.) [9, 10]. This problem is discussed in more detail by H. CZUDEK [2] and J. RABIEGA [8].

Due to the above, the structural elements of steel bridges designed according to the current standard cannot be expected to be fully fatigue-resistant, particularly under the constantly increasing loads on the railway lines [8].

The author – A. WYSOKOWSKI has drafted, as a part of the preparatory work aimed at the revision of the current PN-82/S-10052 standard being carried out by the Roads & Bridges Research Institute, some proposals for taking fatigue into consideration already at the design stage [13, 14, 15, 16]. The presented method has been developed on the basis of this part of the above work which applies to newly designed bridges. The author has taken into account the latest trends in standardisation in this field to make the Polish standards conform to the European guidelines [1, 4, 5, 11, 12].

## 2. GENERAL DESCRIPTION OF THE METHOD OF ASSESSING FATIGUE HAZARD

### *2.1. Introduction*

To counteract fatigue and to prevent bridges from being closed to traffic, fatigue should be taken into account already in their design. The Polish steel bridge design standard in force takes account of fatigue, but unlike the corresponding standards in most European countries, it is not based on the recent findings in the field of service strength.

To illustrate this point, let me quote the relevant **fatigue calculations according to Polish standard PN-82/S-10052**. For structural elements the following condition must be fulfilled:

$\frac{\sigma}{m_{zm}}$ [with a dynamic factor of characteristic loads] $\leq$ R calc.

Fatigue coefficient $m_{zm}$ is calculated from this formula:

$$m_{zm} = \frac{c}{(a\beta + b) - (a\beta - b)\,\rho}$$

where:

$a, b, c$ – coefficients,

$\beta$ – a notch sensitivity index (depending on the notch and the type of steel),

$\rho = \frac{\sigma_{\min}}{\sigma_{\max}}$ – an asymmetry coefficient $(-1 \leq \rho \leq 1)$,

$\sigma_{\min}, \sigma_{\max}$– respectively the lowest and highest absolute value of normal or steady stress generated by characteristic loads for a considered cross-section, with the dynamic factor, but not the stability loss coefficient, taken into account; values with proper signs (if the signs agree, then $\rho$ is positive, if otherwise, then $\rho$ is negative) are assumed for the formula. The signs in brackets in the formula for $m_{zm}$ are for a case when the tested cross-section is under tension; if the cross-section is under compression, they should be changed to the opposite signs. If $m_{\rm zm} \geq 1$, fatigue is neglected.

**Comments:**

- Standard PN-82/S-10052 does not distinguish between the kind of element (a deck or a main beam), whereas such a distinction is of fundamental importance;
- Two types of steel are considered (the high-cycle fatigue sensitivity, however, does not depend on the type of steel; the fatigue hazard to steel 18G2 is higher in comparison with that to steel St3M only because higher stress is allowable for the former steel).
- Archaic cycle asymmetry coefficient $\rho$ is used in the standard, whereas today stress range $\Delta\sigma$ is commonly used.
- Fatigue load capacity is much more affected by the span of elements than it follows from coefficient "$c$".
- The classification of notches needs updating to incorporate the latest research findings.

As regards the above, the Polish general building standard for the design of steel structures [5, 7] is closer to reality. It is not possible, however, to apply this method in full to steel bridges because of the quite different load spectrum and other features peculiar only to bridges.

Through the 80s and 90s the bridges could not be checked for fatigue because of, among others, the lack of a model of fatigue loads in Eurocode 1 – no Pan-European agreement on this issue could be reached due to the particular interests of individual countries. New Eurocodes 3 (editions [3]) take the fatigue of steel structures into account in a highly original way, but since the fatigue vehicle is strictly defined there, this method can be applied only to standard-related calculations. The method is not flexible enough for more detailed analyses connected with the testing of the fatigue hazard to the particular structural elements of bridges.

Considering the above and authors' own research:

- *Testing Fatigue Durability of Structural Elements of Bridges* – a chapter written for the draft of updated steel bridge design standard PN-/S10052 [16],
- Draft Guidelines of General Directorate of Railways: *Testing Fatigue Hazard to Steel Railway Bridges* Polish Ministry of Transport. 1990.

as well as previous analyses contained in works [14, 15] and in Sec. 3.4, the author has developed a Method of Testing Fatigue Hazard to Structural Elements of Steel Bridges which is presented below.

The method incorporates all the recent findings relating to the fatigue of steel structures and takes into account the current world standardization trends in this field.

The author of the present paper is the author of all the formulas given below.

### *2.2. General assumptions*

This method of assessing the fatigue hazard is applicable to elements of the steel spans of both newly designed railway bridges and those being in service on all the lines of Polish State Railways.

Fatigue hazard can be checked for both the main bearing elements and secondary elements (such as wind and other braces) performing other functions than carrying the standard moving loads.

### *2.3. Method of assessing fatigue hazard to structural elements of newly designed bridges*

*2.3.1. Introductory remarks.* Structural elements are checked at the fatigue hazard for the assumed bridge service life $T_n = 120$ years. Structural elements in which the maximum range of stress is smaller than 26.0 MPa need not be checked for fatigue.

*2.3.2. Checking for fatigue hazard.*

**Fatigue hazard should be checked by using the following relationship:**

$$\Delta\sigma_n < \frac{1}{\gamma_s}\Delta\sigma_{n,\mathrm{allow.}}, \tag{2.1}$$

$$\Delta\tau_n < \frac{1}{\gamma_s}\Delta\tau_{n,\mathrm{allow.}}, \tag{2.2}$$

where:

$\Delta\sigma_n$, $\Delta\tau_n$ – the range of stress produced by the standard load (for secondary elements, the loads for which these elements were designed for should be assumed);

$\Delta\sigma_{n,\mathrm{allow.}}$, $\Delta\tau_{n,\mathrm{allow.}}$ – the allowable range of stress produced by the standard load (fatigue load capacity);

$\gamma_s$ – the generalised material coefficient for steel, which for structures corresponding to the execution and acceptance standard PN-89/S-10050 is $\gamma_s = 1.00$.

Stress range $\Delta\sigma_n(\Delta\tau_n)$ is defined as the absolute value of the algebraic difference between the maximum stress and the minimum stress, produced by characteristic live loads with a dynamic coefficient at the considered point of a structural element. Compressive stress $\Delta\sigma_c$ in the case of unwelded elements can be reduced by 60% for alternating stresses or exclusively compressive stresses ($\Delta\sigma_r = 0$) by using the following expression:

$$\Delta\sigma_n = \Delta\sigma_r + 0.6\Delta\sigma_c, \tag{2.3}$$

*Checking for fatigue hazard in case of complex state of stress*

- If in a considered section of a structural element stresses $\Delta\sigma_n$ and $\Delta\tau_n$ occur simultaneously under an identical load, the fatigue hazard should be checked by using the following relationship:

$$\left[\gamma_s \frac{\Delta\sigma_n}{\Delta\sigma_{n,\text{allow}}}\right]^2 + \left[\gamma_s \frac{\Delta\tau_n}{\Delta\tau_{n,\text{allow}}}\right]^2 < 1. \tag{2.4}$$

- If in a considered section of a structural element $\Delta\sigma_n$ and $\Delta\tau_n$ do not occur simultaneously, the fatigue hazard should be checked by using the following relationship:

$$\left[\gamma_s \frac{\Delta\sigma_n}{\Delta\sigma_{n,\text{allow}}}\right]^3 + \left[\gamma_s \frac{\Delta\tau_n}{\Delta\tau_{n,\text{allow}}}\right]^5 < 1. \tag{2.5}$$

*2.3.3. Fatigue load capacity.*

- Fatigue load capacity is defined as a range of the variation of stresses $\Delta\sigma_{n,\text{allow}}$, $(\Delta\tau_{n,\text{allow}})$ occurring cyclically in the expected service life of a structure which will not cause damage to any of its elements with at least 97% probability.
- Fatigue load capacities for different notch categories are represented in the $\Delta\sigma$ – N diagram as curves in a doubly logarithmic scale with the corresponding values of $\Delta\sigma_A$ $(\Delta\tau_A)$ (fatigue categories) at two million stress cycles (Fig. 1).
- The fatigue load capacity in a structural element for a specific notch category should be calculated by using the following relationships:

$$\Delta\sigma_{n,allow} = \Delta\sigma_A * \left[\frac{2 * 10^6}{N_{\Delta_n}}\right]^{\frac{1}{m}}, \tag{2.6}$$

$$\Delta\tau_{n,allow} = \Delta\tau_A * \left[\frac{2 * 10^6}{N_{\Delta_n}}\right]^{\frac{1}{5}}, \tag{2.7}$$

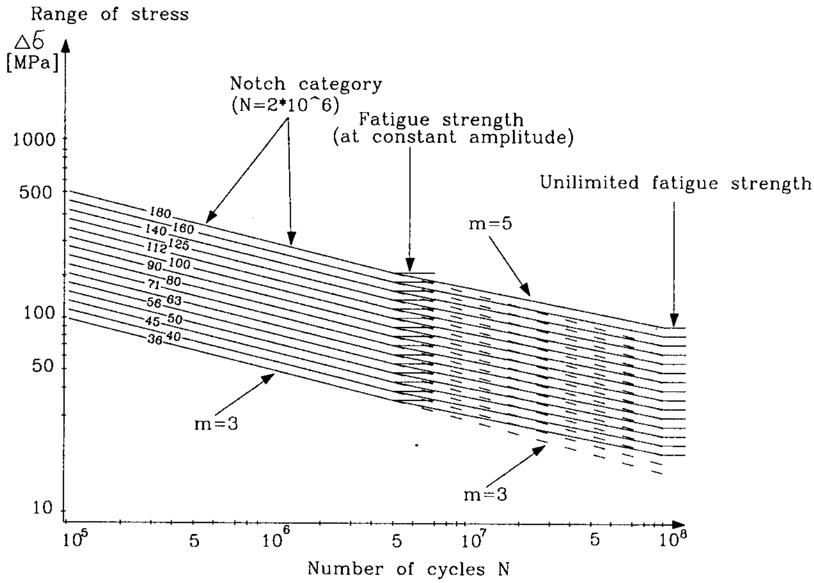



FIG. 1. Fatigue load capacity curve for different categories of notches.

where:

$\Delta\sigma_A$, $\Delta\tau_A$ – the standard fatigue strength (for N = 2 $\times 10^6$), an element fatigue category. A value for $\Delta\sigma_A$ ($\Delta\tau_A$) should be taken from appropriate tables [3] depending on the type of the notch in the considered element. Two exemplary Tables: 1 and 2 containing fatigue categories $\Delta\sigma_A$ for group B – welded elements, and for group C – connectors, can be found below.

$m$ – a slope coefficient for fatigue curves. If Tables [3] 1 and 2 do not specify otherwise, $m = 3.0$ should be assumed.

$N_{\Delta n}$ – the equivalent substitute number of cycles with ranges of stress produced by standard moving loads (an operating stress spectrum, cf. Sec. 2.3.4, parameter).

*2.3.4. Operating stress spectrum.* Operating stress spectrum parameter $N_{\Delta n}$ should be calculated by using this relationship

$$N_{\Delta n} = N'_{\Delta n} * a * b, \tag{2.8}$$

where:

$N'_{\Delta n}$ – the equivalent number of cycles of stress ranges. A value for $N'_{\Delta n}$ should be taken from Table 3 depending on the category of the railway line on

**Table 1. Group B: Welded elements. Fatigue categories $\Delta\sigma_A$ for different structural elements depending on kind of notch.**

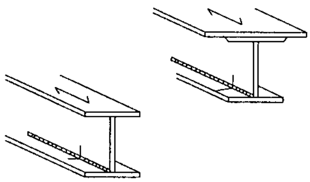
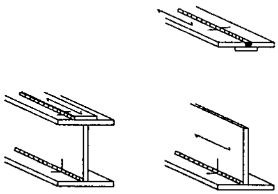
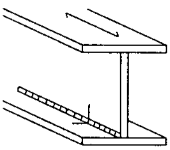
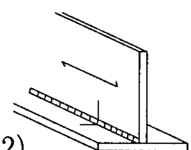
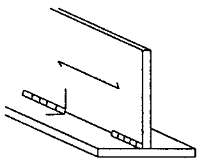
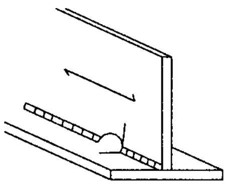
| Category $\Delta\sigma_A$ | Structural elements (arrows indicate direction of stresses in base material for which range of stress is calculated) | | Description of element |
|---|---|---|---|
| **125** | (8)<br>(9) | <br>8) | Welded sheets and box girders – – continuous longitudinal welds<br>(8) Zones of continuous longitudinal fillet welds and/or<br>(9) bilateral butt welds done automatically without interruptions.<br>(10) Zones of continuous unilateral butt welds executed automatically with backing without interruptions |
| **112** | (10) | <br>9) 10) | |
| **112** | (11) | <br>11) | (11) Zones of continuous longitudinal fillet or bilateral butt welds done automatically with interruptions during welding.<br>(12) Continuous longitudinal fillet or butt welds done manually. Zone of unilateral continuous longitudinal welds (especially in box girders).<br>Very good fit between wedge and flange and uniform thorough penetration without skips are required in order to obtain appropriate crack resistance on opposite side of weld. |
| **90** | (12) | <br>12) | |

**Table 1. [cont.]**

| | | |
|---|---|---|
| **80** |  13) | Longitudinal intermittent welds<br>(13) Zones of longitudinal intermittent welds. |
| **71** |  14) | (14) Zones within passes for longitudinal welds in joints of type T. |

which the bridge structure is situated. For nontypical bridge structures, parameter $N'_{\Delta n}$ should be calculated individually, particularly in relation to the magnitude of the loads and the intensity of service.

$a$ – a coefficient related to the type of the element:
for the main girders $a = 1.00$,
for deck elements $a = 1.50$,
for secondary elements $a = 0.50$.

$b$ – a coefficient related to the span of a considered element (the effective span should be taken for simply supported girders and the length of the one-sign influence line branches should be taken for multispan continuous girders). A value for coefficient $b$ should be taken from Table 4a for the main girders, and from Table 4b for deck elements. For secondary elements, the value of coefficient $b$ can be assumed to be equal to 0.10.

### *2.4. Method of assessing fatigue hazard to structural elements of bridges in service*

*2.4.1. Introductory remarks.* Checking the fatigue hazard to structural elements of bridges in service by means of the procedure described in this section should be preceded each time by a thorough survey of the bridge structure. The aim of such a survey is to detect cracks, material defects or improperly executed joints between elements (e.g. welds), which increase the fatigue hazard. Special

**Table 2.**

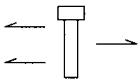
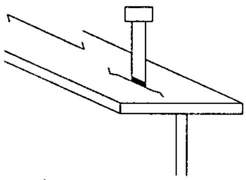
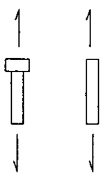
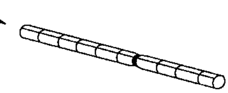
| Category $\Delta\sigma_A$ | Structural elements (arrows indicate direction of stresses in base material for which range of stress is calculated) | Description of element |
|---|---|---|
| 100 |  40) | (40) Pins and bolts subjected to welding |
| 80 m=5 |  41) | (41) Welded studs subjected to shearing (crack in weld). |
| 36 | 42)  | (42) Threaded bolts and bars subjected to tension (tensile stresses calculated for net section). Additional forces generated by eccentricity. |
| 80 |  43) | (43) Butt welds in rolled bars. |

**Table 3. Equivalent stress range cycle numbers $N^c_{\Delta n}$**

| Location of bridge structure | | Category | $N'_{\Delta n}[*10^6]$ |
|---|---|---|---|
| 1 | | 2 | 3 |
| Railway bridges | Railway trunk lines (load $\geq=$ 25 million t/annum) | K I | 50 |
| | Primary railway lines (load $<$ 25 million t/annum) | K II | 20 |
| | Other railway lines (load $<$ 10 million t/annum) | K III | 15 |

**Table 4a. Values of coefficient b for main girders**

| Kind of element | Reliable length L [m] | | | | | | |
|---|---|---|---|---|---|---|---|
| | $\leq 3,0$ | 4,0 | 6,0 | 8,0 | 10,0 | 15,0 | $\geq 20,0$ |
| Simple-supported girders | 1,00 | 0,30 | 0,20 | 0,15 | 0,10 | 0,10 | 0,05 |
| Continuous multi-span girders | 1,80 | 0,50 | 0,30 | 0,20 | 0,15 | 0,15 | 0,10 |

**Table 4b. Values of coefficient b for deck elements**

| Kind of element | Spacing of cross-bar t [m] | | | |
|---|---|---|---|---|
| | $\leq 2,0$ | 3,0 | 4,0 | $\geq 6,0$ |
| Deck plate, longitudinal ribs, cross-bar | 1,00 | 0,50 | 0,20 | 0,10 |

attention should be paid to structural joints in the main girders and to the deck elements.

Checking the fatigue hazard in bridges in service can be limited to the main load bearing elements (the main girders and deck elements). Secondary elements should be checked for fatigue only in the case of a hazard or doubts raised by a comprehensive survey.

It is advisable to select elements for fatigue testing on the basis of preliminary theoretical analyses (carried out, for example, by the methods described in Sec. 2.3).

*2.4.2. Checking the fatigue hazard after service life so far.* The fatigue hazard for bridges in service is assessed on the basis of the stress spectra recorded in selected elements during testing under the actual traffic. The way in which they are obtained is described in Sec. 2.4.4.

The fatigue hazard should be checked by applying formulas 2.1, 2.2, 2.6 and 2.7 (Sec. 2.3) whereas operating the stress spectrum parameter $N_{\Delta n}$ for the service life so far can be calculated by using the following relationship:

$$N_{\Delta n} = \gamma_f * N^e_{\Delta n} * \frac{N^e}{\sum n_i}, \tag{2.9}$$

where $N^e_{\Delta n}$ for the test period can be calculated by applying this formula

$$N^e_{\Delta n} = \sum_i n_i * \left(\frac{\Delta\sigma_i}{\Delta\sigma_n}\right)^m, \tag{2.10}$$

where:

$\Delta\sigma_i$ – the range of stress for the $i$-th level,

$n_i$ – the number of cycles for the $i$-th level,

$\Delta\sigma_n$ – the range of stress produced by standard moving loads for the considered structural point,

$N^e$ – the number of cycles in service life so far estimated on the basis of tests (loads can be assumed to be invariable in the entire service life).

The spectrum recording period related coefficient $\gamma_f$ can be calculated by applying this formula:

$$\gamma_f = 1 + 0.03 * \left(\lg \frac{N^e}{\sum n_i}\right)^2 \geq 1. \tag{2.11}$$

*2.4.3. Checking the fatigue hazard for standard service life.* The fatigue hazard for standard bridge service life $T_n = 120$ years can be checked by applying formulas 2.1, 2.2, 2.6 and 2.7 (Sec. 2.3), and the operating stress spectrum parameter $N_{\Delta n}$ can be calculated by using the following relationship:

$$N_{\Delta n} = \gamma_f * N^e_{\Delta n} * \frac{N^n}{\sum n_i}, \tag{2.12}$$

where:

$N^n$ – the number of cycles in standard bridge service life $T_n = 120$ years estimated on the basis of tests,

$N^e_{\Delta n}$ – the operating stress spectrum parameter calculated by applying formula (2.10).

Coefficient $\gamma_f$ should be calculated by applying the following formula:

$$\gamma_f = 1 + 0.03 * \left[\lg \frac{N^n}{\sum n_i}\right]^2 \geq 1. \tag{2.13}$$

*2.4.4. Method of determining the stress spectrum.* Stress spectra are the basis on which a fatigue hazard to structural elements of bridges in service on railway lines is assessed. In order to obtain such spectra, it is necessary to make appropriate measurements of bridge elements under the actual railway traffic.

The aim of such tests is to record oscillograms of unit strains that occur in bridge elements under railway traffic. It is recommended to record the kinds of the occurring loads at the same time.

Tests on one bridge structure should be conducted for at least 48 hours and on the days on which the traffic is most typical.

As mentioned in Sec. 2.4.1, mainly the elements most susceptible to fatigue and essential for the proper performance of the bridge structure should be tested. At least two strain gauges per one element should be installed in the most critical sections. Both the disposable and reusable gauges can be employed.

Instantaneous stress values are calculated on the basis of the recorded strain traces using the following relationship:

$$\sigma = \frac{E * a * C}{k} * \frac{h}{A} \tag{2.14}$$

where:

- $\sigma$ – stress in MPa,
- $E$ – Young's modulus of the tested element's material for steel E $= 2.06 \times 10^5$ MPa,
- $a$ – an extensometer bridge constant,
- $C$ – a set instrument range,
- $k$ – sensitivity of the used strain gauges (an extensometer constant),
- $h$ – a strain value read out from a recorded graph,
- $A$ – a calibration value read out from the tape for an appropriate gauge and time interval.

The measuring set-up should include a computer which will enable the immediate representation of changes in the stresses during testing according to formula (2.14), and recording of the results for the particular channels on a CD. The conversion of the recorded results into histograms of stress ranges $\Delta\sigma_i$ by means of the recommended here and generally used Rain-Flow method, leads directly to the calculation of operating stress spectrum parameter $N^e_{\Delta n}$ according to formula (2.10).

This parameter is a basic quantity used for checking the fatigue hazard to bridges in service (Secs. 2.4.2 and 2.4.3).

### *2.5. Determination of the allowable service life of railway bridges in service in relation to fatigue*

*2.5.1. Introductory remarks.* Once it has been determined that there is no fatigue hazard after the bridge's service so far, it is necessary to determine the further safe service life of the bridge in relation to fatigue.

This can be done in the way described in this section using the previously calculated service parameters.

*2.5.2. Determination of allowable service life of bridges in relation to fatigue.* The allowable service life of a bridge element can be determined by using this relationship:

$$T_{\text{allow}} = \left[\frac{\Delta\sigma_A}{\Delta\sigma_n}\right]^m * \left[\frac{2 * 10^6}{N_{\Delta_n}}\right] * T_n, \tag{2.15}$$

where:

$T_n$ – the standard service life of 120 years,

$N_{\Delta n}$ – an operating stress spectrum parameter for the standard service life of 120 years.

It should be noticed that $T_{\text{allow.}}$ applies to the whole bridge whereas for any structural bridge element $T_{\text{allow.}} = \min\,(T_{\text{allow}})$.

Further safe service life $T_e$ in years (assuming that the magnitude of load does not change) can be determined by using the relationship

$$T_e = T_{\text{allow}} - T_d, \tag{2.16}$$

where: $T_d$ – the service life so far of the bridge.

## 3. Examples of checking the fatigue hazard of bridge structural elements by means of the proposed method

To demonstrate how the proposed method is used to check structural elements of bridges for the fatigue hazard, several examples are given below.

The examples apply to structural elements of newly designed bridges and bridges in service. In addition, an example showing how to determine the allowable service life is provided.

### 3.1. Example I

A newly designed railway viaduct situated at kilometre 134.073 of the Kalety-Kluczbork-Wrocław railway line.

The designed all-welded steel plate girder structure with inclined-web main girders and a closed bridge deck in the form of an orthotropic deck plate with closed longitudinal ribs should be checked for the fatigue hazard.

The static scheme: a 13.60 m long ($l_t$) simply supported beam positioned at slant $\alpha = 64°42'$. The track on ballast running straight at a longitudinal slope (track structure S-49). The line is two-track, electrified, having classification coefficient $\alpha_{+2} = 1.21$. The loads conform to standard PN-85/S-10030. The material: steel St3M having design strength $R = 200$ MPa. A cross-section of the bridge is shown in Fig. 2.

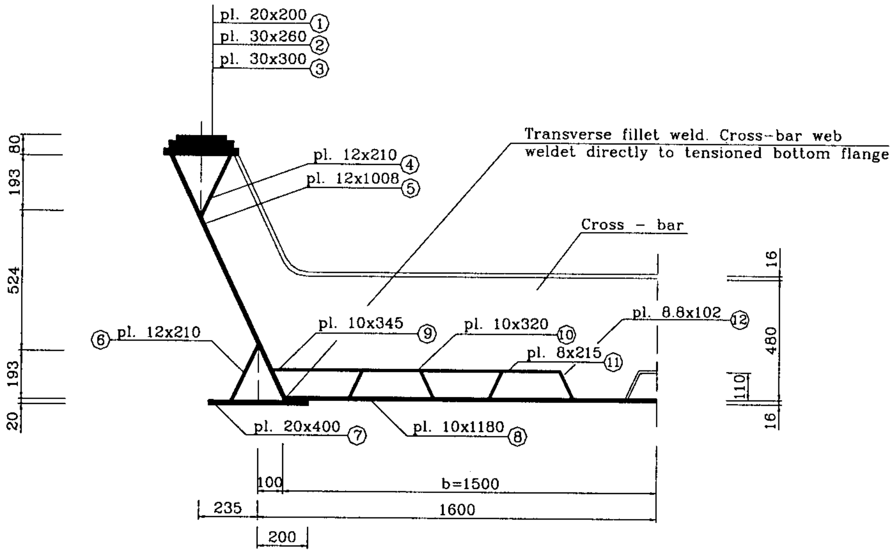



FIG. 2. Cross-section of viaduct structure.

3.1.1. *Example I.1.* A welded plate girder with an inclined web, as shown in the figure, is designed. The plate girder's bottom flange under tension, at the place where the cross-bar's web is welded with fillet welds is checked, taking into account its service life. Static quantities were determined at midspan.

a) The geometric characteristics of the cross-section are assumed according to the design:

$I_x = 1237\,628$ cm$^4$; $e_d = 42.04$ cm; $w_d = 29439$ cm$^3$.

b) Determination of the range of stress produced by the characteristic loads:

$M_{\max}^{(\mathrm{ch})} = 2922.9616$ kNm $\Rightarrow \sigma_{max} = 99.29$ MPa,

$M_{\max}^{(\mathrm{ch})} = 577.5376$ kNm $\Rightarrow \sigma_{\max} = 19.62$ MPa,

$\Delta\sigma_n = \sigma_{\max} - \sigma_{\min} = 79.67$ MPa.

c) Determination of the fatigue load capacity $\Delta\sigma_{n,\mathrm{allow}}$, Eq. (2.5):

$\Delta\sigma_A = 71$ MPa (Table 2, item 29 [3]),

$m = 3$, cross-bar (rib) thickness $t_s = 13$ mm,

$N_{\Delta n} = N'_{\Delta n} * a * b$,

$N'_{\Delta n} = 40\times10^6$ cycles $\Rightarrow$ category K I according to Table 3,

$a = 1.0 \Rightarrow$ for the main girder,

$b = 0.1 \Rightarrow$ Table 4a, $l_t = 13.6$ m,

$N_{\Delta n} = 4.0 \times 10^6$ cycles,

$\Delta\sigma_{n,\mathrm{allow}} = 56.35$ **MPa.**

d) Checking the service life conditions:

$\Delta\sigma_n < \frac{1}{\gamma_s} * \Delta\sigma_{n,\mathrm{allow}}, \quad \gamma_s = 1.0,$

$\Delta\sigma_n = 79.67 \text{ MPa} > \frac{1}{\gamma_s} * \Delta\sigma_{n,\mathrm{allow}} = 56.35$ MPa.

The condition is not satisfied. Because of a fatigue hazard, the bending factor for the cross-section must be increased by increasing the dimensions of the flanges, the web, the deck plate and the longitudinal ribs.

e) The geometric characteristics of the new cross-section:

$e_d = 41.16$ cm, $e_g = 62.04$ cm, $I_x = 1767569.6$ cm$^4$, $w_d = 42944$ cm$^3$.

f) The determination of the range of stress produced by the characteristic loads:

$M_{\max}^{(\mathrm{ch})} = 2922.9616$ kNm $\Rightarrow \sigma_{\max(\mathrm{d})} = 68.06$ MPa,

$M_{\min}^{(\mathrm{ch})} = 577.5376$ kNm $\Rightarrow \sigma_{\min(\mathrm{d})} = 13.45$ MPa,

$\Delta\sigma_n = \sigma_{\max} - \sigma_{\min} = 54.61$ MPa.

g) Checking the service life condition:

$\Delta\sigma_n < \frac{1}{\gamma_s} * \Delta\sigma_{n,\mathrm{allow}}, \quad \gamma_s = 1.0,$

$\Delta\sigma_n = \mathbf{54.61\ MPa} < \frac{1}{\gamma_s} * \Delta\sigma_{n,\mathrm{allow}} = \mathbf{56.35\ MPa}.$

The condition is satisfied and thus the main girders will not be exposed to a fatigue hazard for the whole service life assumed for the bridge structure ($T_n = 120$ years).

*3.1.2. Example I.2. Cross-bar.* A deck in the form of an orthotropic plate ribbed longitudinally and transversely from the top is designed. The closedprofile longitudinal ribs are welded to the metal plates of the bridge deck and to the webs of the cross-bars (the stringers pass through holes in the cross-bars). The bottom flange of a cross-bar, which forms the collaborating part of the metal deck plate within the cuts in the web, should be checked at the place where intermittent fillet welds end, taking into account the service life of the flange.

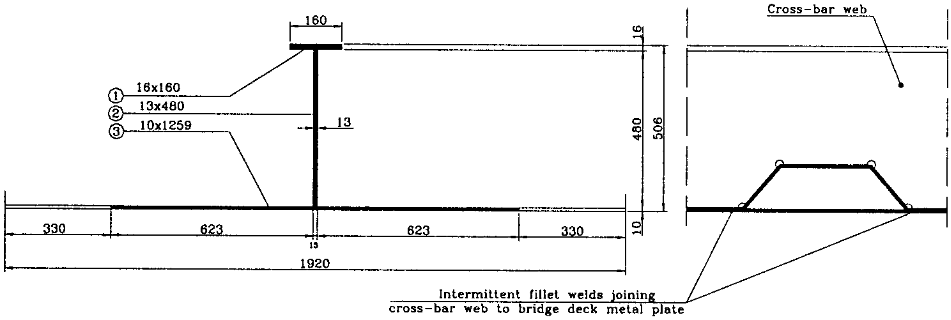



FIG. 3. Cross-bar.

a) The geometric characteristics of the cross-section are assumed according to the design:
$I_x = 75243$ cm$^4$, $e_d = 13.55$ cm, $w_d = 5553$ cm$^3$.

b) Determination of the range of stress produced by the characteristic loads:
$M^{(\mathrm{ch})}_{(\mathrm{max})} = 237.2762$ kNm, $\sigma_{\mathrm{max(d)}} = 42.73$ MPa,
$M^{(\mathrm{ch})}_{(\mathrm{min})} = 31.6467$ kNm, $\sigma_{\mathrm{min(d)}} = 5.7$ MPa,
$\Delta\sigma_n = \sigma_{\max} - \sigma_{\min} = \mathbf{37.03}$ **MPa.**

c) Determination of the fatigue load capacity $\Delta\sigma_{n,\mathrm{allow}}$, Eq. (2.5):
$\Delta\sigma_A = 71$ MPa, Table 1. item 14,
$m = 3$ (the zone within the cut at the longitudinal welds),
$N_{\Delta n} = N'_{\Delta n} * a * b$, $N'_{\Delta n} = 40\times 10^6$ cycles $\Rightarrow$ category K I according to Table 3,
$a = 1.5 \Rightarrow$ for deck elements, $b = 0.44$, $t_{\mathrm{transv.}} = 3.2$ m according to Table 4b,
$N_{\Delta n} = 26.4 \times 10^6$ cycles, $\Delta\sigma_{n,\mathrm{allow}} = \mathbf{30.04}$ **MPa**.

d) Checking the service life condition

$$\Delta\sigma_n < \frac{1}{\gamma_s} * \Delta\sigma_{n,\mathrm{allow}}, \quad \gamma_s = 1.0,$$

$\Delta\sigma_n = 37.03$ MPa $> \frac{1}{\gamma_s} * \Delta\sigma_{n,\mathrm{allow}} = \mathbf{30.04}$ **MPa**.

The condition is not satisfied. Because of a fatigue hazard, the bending factor for the cross-section must be increased by increasing the bottom plate thickness to 14 mm.

e) The geometric characteristics of the new cross-section:
$e_d = 11.33$ cm, $e_g = 39.67$ cm, $I_x = 131577$ cm$^4$, $w_d = 11613$ cm$^3$.

f) Determination of the range of stress produced by the characteristic loads:
$\sigma_{\text{max(d)}} = 20.43$ MPa, $\sigma_{\text{min(d)}} = 2.73$ MPa,
$\Delta\sigma_n = \sigma_{\text{max}} - \sigma_{\text{min}} = \mathbf{17.7MPa}$.

g) Checking the service life condition:
$\Delta\sigma_n < \frac{1}{\gamma_s} * \Delta\sigma_{n,\text{allow}}$, $\gamma_s = 1.0$, $\Delta\sigma_n = 17.7$ MPa $< \frac{1}{\gamma_s} * \Delta\sigma_{n,\text{allow}} =$ **30.04 MPa**.
The condition is satisfied. Thus no fatigue hazard will occur in the assumed entire bridge service life ($T_n = 120$ years).

*3.1.3. Example I.3. Stringer.* The top flange of the closed rib should be checked at the place where the transverse fillet welds join the metal plate to the web of the cross-bar, taking into account the service life of the flange.

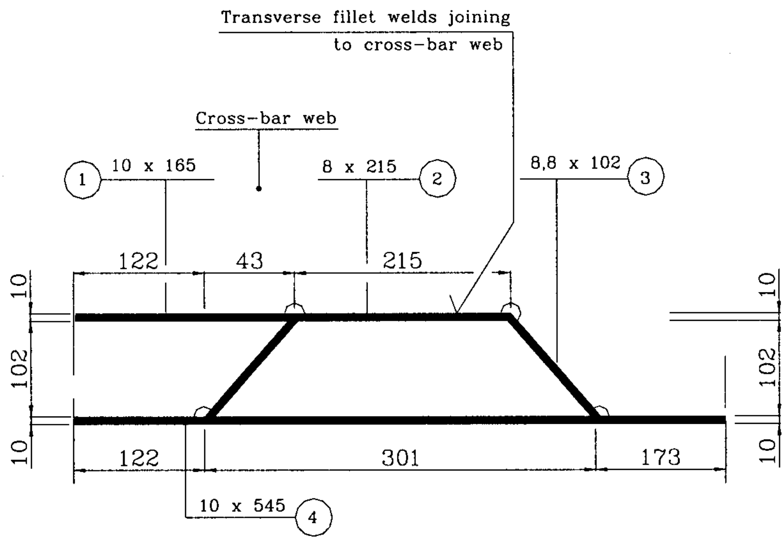



FIG. 4. Stringer.

a) The geometric characteristics of the cross-section are assumed according to the design:
$I_x = 2727.15$ cm$^4$, $e_g = 7.04$ cm, $w_g = 387$ cm$^3$.

b) Determination of the range of stress produced by the characteristic loads:

The maximum moments at the support:
$M^{(ch)}_{(max)} = 19.2509$ kNm, $M^{(ch)}_{(min)} = 2.1783$ kNm, $\sigma_{max(g)} = 49.74$ MPa, $\sigma_{min(g)} = 5.63$ MPa, $\Delta\sigma_n = \sigma_{max} - \sigma_{min} = \mathbf{44.11\ MPa}$.

c) Determination of the fatigue load capacity $\Delta\sigma_{n,\text{allow}}$, Eq. (2.5):
$\Delta\sigma_A = 71$ MPa, (according to Table 2 item 29 [3]),
$m = 3$ cross-bar web thickness $t_s = 13$ mm,
$N_{\Delta n} = N'_{\Delta n} * a * b$, $N'_{\Delta n} = 40{*}10^6$ cycles $\Rightarrow$ category K I according to Table 3,
$a = 1.5 \Rightarrow$ for deck elements, b $= 1.0 \Rightarrow$ according to Table 4,
$l_{\text{transv.}} = 1.92$ m $< 2.00$ m according to Table 4b, $N_{\Delta n} = 6.0 \times 10^6$ cycles,
$\Delta\sigma_{n,\text{allow}} = \mathbf{22.85\ MPa}$.

d) Checking the service life condition:
$\Delta\sigma_n < \frac{1}{\gamma_s} * \Delta\sigma_{n,\text{allow}}$, $\gamma_s = 1.0$, $\Delta\sigma_n = 44.11$ MPa $> \frac{1}{\gamma_s} * \Delta\sigma_{n,\text{allow}} = \mathbf{22.85\ MPa}$.
Because of a fatigue hazard, the bending factor for the cross-section must be increased by extension of its dimension.

e) The geometric characteristics of the new cross-section:
$e_d = 6.71$ cm, $e_g = 9.09$ cm, $I_x = 6827$ cm$^4$, $w_g = \mathbf{751\ cm^3}$.

f) Determination of the range of stress produced by the characteristic loads:
$\sigma_{max(g)} = 25.63$ MPa, $\sigma_{min(g)} = 2.9$ MPa, $\Delta\sigma_n = \sigma_{max} - \sigma_{min} = \mathbf{22.73\ MPa}$.

g) Checking the service life condition:
$\Delta\sigma_n < \frac{1}{\gamma_s} * \Delta\sigma_{n,\text{allow}}$, $\gamma_s = 1.0$, $\Delta\sigma_{n,\text{allow}} = 22.85$ MPa,
$\Delta\sigma_n = 22.73$ MPa $< \frac{1}{\gamma_s} * \Delta\sigma_{n,\text{allow}} = \mathbf{22.85\ MPa}$.
The condition is satisfied. Thus no fatigue hazard will occur in the assumed whole bridge service life ($T_n = 120$ years).

### *3.2. Example II*

A newly designed railway bridge at kilometre 98.470 of the Rawicz-Legnica railway line.

The designed load-bearing structure of the bridge with two all-welded, solid-walled steel girders with straight webs spaced at every 5.00 m and the reinforced concrete deck integrated with the cross-bars should be checked for the fatigue hazard. The static scheme: a 27.00 m long ($l_t$) simply supported beam without slant. The track on the deck runs straight on breakstone ballast at a longitudinal slope. The line: single track, secondary, not to be electrified, classification coefficient $\alpha + 1 = 1.1$. The load conforms to Polish Standard PN-82/S-10030. The material: corrosion-resisting steel 12HNNb having design strength $R = 269$ MPa and $R_t = 162$ MPa. The cross-section of the bridge is shown in Fig. 5.

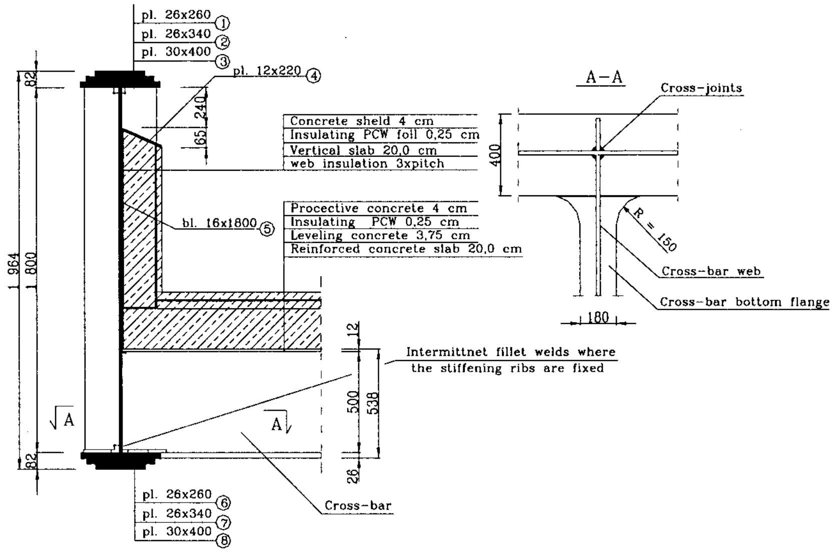



FIG. 5. Cross-section of the bridge.

*3.2.1. Example II. 1. Main girder flanges.* A plate girder with a straight web and symmetric flanges, as shown in the figure, is designed. The bottom flange of the plate girder subject to tension, at the place where the bottom flange of the cross-bar is butt-welded to it should be checked, taking into account the service life of the flange. Midspan static quantities are used.

a) The geometric characteristics of the cross-section are assumed according to the design:
$I_x = 5521463$ cm$^4$, $Y_d = 05 * 196.4 - 2 * 2.6 = 93$ cm, $w_g = \frac{5521463}{93} = 59371$ cm$^3$.

b) Determination of the range of stress produced by the characteristic loads:
$M^{(\mathrm{ch})}_{(\max)} = 11235.64$ kNm, $\sigma_{\max(\mathrm{g})} = 189245$ MPa, $M^{(\mathrm{ch})}_{(\min)} = 5023.72$ kNm, $\sigma_{\min} = \frac{M_{\min}}{w_g} = 84.62$ MPa, $\Delta\sigma_n = \sigma_{\max} - \sigma_{\min} = \mathbf{104.625\ MPa}$.

c) Determination of the fatigue load capacity $\Delta\sigma_{n,\text{allow}}$, Eq. (2.5):
$\Delta\sigma_A = 90$ MPa, (according to Tab. 2 item 26 [3]),
$m = 3$, $r = 150$ mm, $W = 400$ mm, $\frac{r}{W} = 0.375 > \frac{1}{3}$,
$N_{\Delta n} = N'_{\Delta n} * a * b$, $N'_{\Delta n} = 20{\times}10^6$ cycles $\Rightarrow$ category K II according to Tab. 3,
$a = 1.0$ – for the main girder,
$b = 0.05$ – acc. to Tab. 4a, $l_t = 27.00$ m,
$N_{\Delta n} = 1 \times 10^6$ cycles, $\Delta\sigma_{n,\text{allow}} = \mathbf{113.4\ MPa}$.

d) Checking the service life condition:

$\Delta\sigma_n \leq \frac{1}{\gamma_s} * \Delta\sigma_{n,\text{allow}}, \quad \gamma_s = 1.0,$

$\Delta\sigma_n = 104.625$ MPa $< \frac{1}{\gamma_s} * \Delta\sigma_{n,\text{allow}} = \mathbf{113.4}$ **MPa**.

The condition is satisfied. Thus the main girders of the bridge will not be exposed to any fatigue hazard in the entire service life of the structure ($T_n = 120$ years).

*3.2.2. Example II.2. Main girder web.* The web of the plate girder in the tensile zone at the place where the transverse welds which fix the stiffening ribs are fixed should be checked, taking into account the service life of the web. Similarly as in Example II.1, midspan static quantities are assumed.

a) The geometric characteristics of the cross-section:

$J_x = 5521463$ cm$^4$, $\quad Y_{dl} = 87$ cm, $\quad g = 16$ mm, $\quad w_{dl} = \frac{J_x}{Y_{dl}} = \frac{5521463}{87} = \mathbf{63465\ cm^3}$,

$S_{x1} = 26291$ cm$^3$.

b) Determination of the range of normal stress produced by the characteristic loads:

$M^{(\text{ch})}_{(\max)}$=11235.64 kNm, $\quad \sigma_{\max(\text{dl})} = 177.04$ MPa,

$M^{(\text{ch})}_{(\min)} = 5023.72$ kNm, $\quad \sigma_{\min_{(\text{dl})}} = 79.16$ MPa,

$\Delta\sigma_n = \sigma_{\max} - \sigma_{\min} = \mathbf{97.88}$ **MPa.**

c) Determination of the range of shear stress produced by the characteristic loads:

$Q_{\max} = 306.87$ kN $\Rightarrow \tau_{\max(\text{dl})} = \frac{Q_{\max} * S * 1}{I_x * g}$ =9.13 MPa,

$Q_{\min} =$ -306.97 kN$\Rightarrow \tau_{\min(\text{dl})} = -$ 9.13 MPa,

$\Delta\tau_{\max} - \tau_{\min} = \mathbf{18.26}$ **MPa**.

d) Determination of the fatigue load capacity $\Delta\sigma_{n,\text{allow}}$, Eq. (2.5) and $\Delta\tau_{n,\text{allow}}$ Eq. ((2.5)′)

$\Delta\tau = \Delta\sigma_A = 80$ MPa (according to Tab. 1.2 item 26 [3]),

$m = 3$ rib (cross-bar web) thickness $t_s = 12$ mm,

$N_{\Delta n} = 1 \times 10^6$ cycles (see p. II.1c),

$\Delta\sigma_{n,\text{allow}} = \mathbf{100.8}$ **MPa,**

$\Delta\tau_{n,\text{allow}} = \mathbf{91.9}$ **Mpa.**

e) Checking the service life condition:

Stresses $\Delta\sigma_n$ and $\Delta\tau_n$ do not occur simultaneously in the considered cross-section $\rightarrow$ Eq. (2.4), $\gamma_s = 1.0$.

$0.91591 < 1$ – the condition is satisfied. Thus no fatigue hazard will occur in the assumed entire service life ($T = 120$ years).

*3.2.3. II.3. Cross-bar bottom flange.* A bridge deck in the form of a reinforced concrete slab collaborating with steel, solid-walled cross-bars spaced at every 2.00 m is designed. The bottom flange of the cross-bar under tension in the zone of continuous longitudinal fillet welds executed automatically without skips should be checked, taking into account the service life of the flange. Midspan static quantities are used.

a) The geometric characteristics of the cross-section assumed according to the design:
$J_c = 236160\ \mathrm{cm}^4$; $Y_d = e_l + a_{st.} = 53.4$ cm, $W_d = \mathbf{4422.5\ cm}^3$.

b) Determination of the range of stress produced by the characteristic loads:
$M^{(\mathrm{ch})}_{(\mathrm{max})} = 537.60$ kNm $\Rightarrow \sigma_{\mathrm{max(d)}} = 121.56$ MPa,
$M^{(\mathrm{ch})}_{(\mathrm{min})} = 114.47$ kNm $\Rightarrow \sigma_{\mathrm{min}_{(\mathrm{d})}} = 25.88$ MPa,
$\Delta\sigma_n = \sigma_{\mathrm{max}} - \sigma_{\mathrm{min}} = 95.68$ **MPa**.

c) Determination of the fatigue load capacity $\Delta\sigma_{n,\mathrm{allow}}$, Eq. (2.5):
$\Delta\sigma_A = 125$ MPa, (according to Tab. 1.2 item 8),
$m = 3$ (welded plates – continuous longitudinal welds),
$N_{\Delta n} = N'_{\Delta n} * a * b$, $N'_{\Delta n} = 20\times10^6$ cycles, category K II according to Tab. 2.3,
$a = 1.5$ – for deck elements,
$b = 0.15$ – according to Tab. 2.4b, $t_{\mathrm{transv.}} = 5.0$ m,
$N_{\Delta n} = 4.5 \times 10^6$ cycles, $\Delta\sigma_{n,\mathrm{allow}} = \mathbf{95.4\ MPa}$.

d) Checking the service life condition:
$\Delta\sigma_n \leq \frac{1}{\gamma_s} * \Delta\sigma_{n,\mathrm{allow}}$, $\gamma_s = 1.0$, $\Delta\sigma_n = 95.68$ MPa $\cong \frac{1}{\gamma_s} * \Delta\sigma_{n,\mathrm{allow}} =$ **95.4 MPa**.
The deficiency in the range of stress amounts to only 0.3% . Thus it can be stated that the condition is satisfied, i.e. no fatigue hazard will occur in the assumed whole service life of the bridge ($T_n = 120$ years).

*3.2.4. II.4. Cross-bar web.* The web of the cross-bar at the place where it is fillet-welded to the main girder web should be checked, taking into account its service life.

a) The geometric characteristics of the web:
web thickness $g = 12$ mm,
web actual height $h_{act.} = 50 - 2 \times 3 = 44$ cm.

b) Determination of the range of stress produced by the characteristic loads:
$Q^{(\mathrm{ch})}_{\mathrm{max}} = 336.645$ kN $\Rightarrow \tau_{\mathrm{max}} = \frac{Q_{\mathrm{max}}}{g*h} = 63.76$ MPa,
$Q^{(\mathrm{ch})}_{\mathrm{min}} = 94.86$ kN $\Rightarrow \tau_{\mathrm{min}} = 17.97$ MPa, $\Delta\tau_n = \tau_{\mathrm{max}} - \tau_{\mathrm{min}} = \mathbf{45.79\ MPa}$.

c) Determination of the fatigue load capacity $\Delta\tau_{n,\text{allow}}$, Eq. ((2.5)′):

$\Delta\tau_A = 56$ MPa (according to Tab. 2 item 32 [3] cross-joints),

$N_{\Delta n} = N'_{\Delta n} * a * b$,

$N'_{\Delta n} = 20 \times 10^6$ cycles – category K II acc. to Tab. 3,

$a = 1.5$ – for deck elements,

$b = 0.15$ – according to Tab. 2.4b, $t_{\text{transv.}} = 5.0$ m,

$N_{\Delta n} = 4.5 \times 10^6$ cycles,

$\Delta\tau_{n,\text{allow}} = \mathbf{47.62\ MPa}$.

d) Checking the service life condition:

$\Delta\tau_n \le \frac{1}{\gamma_s} * \Delta\tau_{n,\text{allow}}$, $\gamma_s = 1.0$, $\Delta\tau_n = 45.79$ MPa $< \frac{1}{\gamma_s} * \tau_{n,\text{allow}} =$ **47.62 MPa**.

The condition is satisfied. Thus no fatigue hazard will occur in the assumed entire bridge service life ($T = 120$ years).

### *3.3. Example III*

A railway viaduct whose load-bearing structure has the form of a two-span continuous beam with the span length of $2 \times 25.50 = 51.00$ m. The load-bearing structure in the cross-section has the form of a steel box closed from top by a reinforced concrete deck slab. The viaduct has been in service since 1985.

*3.3.1. Example III.1. Checking bottom flange for fatigue hazard after service life so far (p. 2.4.2).*

**Checking.**

a) The design range of stress produced by the standard moving loads is $\Delta\sigma_n = 100.0$ MPa. Operating stress spectrum parameter $N^e_{\Delta n}$ was calculated, by applying formula 2.10, on the basis of service tests carried out on the viaduct. The parameter is $N^e_{\Delta n} = 46.93$ cycles and the recorded actual number of cycles is $\sum n_i = 99684$ cycles. Therefore the estimated number of actual cycles for the service life so far is $N^e = 99684 \times 183 \times 15 = 274 \times 10^6$. The coefficient related to the spectrum recording period Eq. (2.11):

$$\gamma_f = 1 + 0.03 * \left(\lg \frac{N^e}{\sum n_i}\right)^2 = 1.36.$$

b) Operating stress spectrum parameter $N_{\Delta n}$ can be calculated by applying relationship Eq. (2.9):

$N_{\Delta n} = 175435$.

c) The fatigue load capacity (according to Eq.(2.5)):

Fatigue category $\Delta\sigma_A = 71.0$ was taken from Table 1 (item 30) [3] (the diaphragms in the box girders are welded by continuous welds).

$\Delta\sigma_{n,\text{allow}} = 159.8$ MPa .

d) Checking the fatigue hazard for the service life so far Eq. (2.1):

$\Delta\sigma_n = \mathbf{100.0\ MPa} < \frac{1}{\gamma_s} * \Delta\sigma_{n,\text{allow}} = \mathbf{159.8\ MPa.}$

The condition is satisfied. Thus no fatigue hazard to the bottom flange of the main girder exists at the moment.

*3.3.2. Example III.2. Checking bottom flange for fatigue hazard for standard service life $T_n = 120$ years.*

**Checking.**

a) The operating stress spectrum parameter determined on the basis of service tests is as in III.1:

$N^e_{\Delta n} = 46.93$ cycles.

The number of actual cycles estimated for the standard service life is $N^n = 99684 \times 183 \times 120 = 2190 \times 10^6$. The coefficient related to the spectrum recording period Eq. (2.13) is:

$\gamma_f = 1.57.$

b) Operating stress spectrum parameter $N_{\Delta n}$ Eq. (2.12):

$N_{\Delta n} = 1618710.$

c) The fatigue load capacity (according to Eq. 2.6):

$\Delta\sigma_{n,\text{allow}} = 76.2$ MPa.

d) Checking the fatigue hazard for standard service life $T_n = 120$ years Eq. (2.1):

$\Delta\sigma_n = \mathbf{100.0MPa} > \frac{1}{\gamma_s} * \Delta\sigma_{n,\text{allow}} = \mathbf{76.2MPa.}$

**The condition is not satisfied.** Thus there is a fatigue hazard to the bottom flange of the main girder for the assumed bridge service life $T_n = 120$ years.

*3.3.3. Example III.3. Determining allowable safe service life in relation to fatigue hazard to bottom flange of the main girder (p. 2.5).*

**Determining service life.**

a) The allowable service life of an element can be determined by using relationship (2.15):

$T_{\text{allow}} = \mathbf{53\ years.}$

The allowable service life is $T_{\text{allow.}} = 53$ years.

b) Further safe service life $T_e$ (after service life so far $T_d$ is subtracted) Eq. (2.16)

$T_e = 53 - 15 = 38$ years.

It follows from the above analyses that because of the fatigue hazard to the bottom flange of the main girder, the bridge can stay in service for the next 38 years.

## Conclusions

The present work deals comprehensively with the fatigue hazard to structural elements of railway bridges and it covers both the bridges being in service on the Polish State Railways lines and the newly designed bridges.

The proposed method of assessing the fatigue hazard to bridges allows one to use as input data also the results of other previously done research in this field.

Since this is a novel approach to the problem in Poland, it needs to be verified in practice for a larger number of actual bridge structures (e.g. as a part of current assessments and surveys of railway bridges and during the design of bridges).

Wider analyses of bridge structures by means of the proposed method will make it possible to determine the scale of the fatigue hazard problem for both the single bridges and the whole classes of steel railway bridges depending on the type of the structure, or to reduce the load on the railway lines on which the bridges are located.

## Acknowledgements

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